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SENSITIVITY STUDY OF THE LIGHT-FRAMED WOOD SHEAR WALLS SUBJECTED TO LATERAL LOADS

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Abstract: The Light-Framed Wood Buildings are considered as one of the most economical and feasible choice for the low and mid-rise buildings in North America and worldwide. The latest changes to the National Building Code of Canada allows construction of up to six-storey light-framed wood building using panelized wood shear walls. The Light-Framed Wood Shear walls consists of different components such as studs, sheathing, chords, top, and bottom plates. These components are connected using nails and mechanical fasteners. The orthotropic characteristics of wood and the nonlinear behaviour of nails add more complexity to the numerical simulation of the Light-Framed Wood Shear walls. The current study presents detailed finite element procedures to model the wood shear walls. The finite element model is verified with two different experiments that are provided in the literature. Both strength and stiffness of the modelled wood shear walls are in good agreement with the experiments. A sensitivity study is performed to assess the critical parameters that affect the strength and stiffness of the wood shear walls. The study shows that the shear stiffness of sheathing-to-frame nails is the most sensitive parameter that affects the lateral response of the wood shear walls. A comparison between the numerical model lateral deflection results and CSA-O86-14 are provided for multi-storey shear wall. The study shows that the wall to floor connections significantly affect the lateral deflection and are not accounted for in the code equations clearly.

1 INTRODUCTION

The Wood Buildings have been standing for hundreds, maybe thousands, of years. New research shows that wood can be used for much taller buildings than normal single-family houses and low-rise buildings. An extensive literature review conducted on the Light-Framed Wood Buildings (LFWBs) analysis and design showed that the previous research only focused on the behavior of structural components, load distribution systems, seismic response, and design procedures of houses (Kirkham et al. 2014). The recent changes to the building codes in Canada and worldwide encouraged the enormous growth in the number of the mid-rise LFWBs.

Wood is an orthotropic material. In other words, the stiffness and strength of the wood vary in each orthogonal direction (Breyer et al. 2007). The main components of the LFWBs are shear walls, floor diaphragms, roof trusses, foundations, and connections. There are two kinds of the Light-Framed Wood Shear walls (LFWS): Standard shear walls and Midply shear walls. Research revealed that the Midply wall system in terms of energy dissipation and stiffness is about 3 times higher than comparable Standard wall system (Varoglu et al. 2006). However, the common system in building industry still is Standard wall system with single sheathing on one or both sides of the wall. The current study focuses on the Standard LFWS systems. The LFWS consists of frames (studs), sheathing (one or two sides), and metal fasteners (nails or staples). Studs and plates (top and bottom) are responsible for supporting the vertical loads and the sheathing transfers lateral loads (Doudak and Smith 2009). The sheathing is generally oriented strand board (OSB) or plywood. The gypsum wall board (GWB) is considered as a non-structural element (drywall) in exterior walls. Research showed that the GWB increase initial stiffness of wall up to 50% and it does not affect the overall lateral strength of the wall (Sinha and Gupta 2009). The interaction between combined in-plane racking and uplift forces reduced the racking (lateral) capacity by 25 to 40%. However, the combination of in-plane with out-of-plane bending forces did not negatively impact racking capacities (Winkel and Smith 2010). Angle brackets and hold-downs, makes structural continuity among modular elements, positively affects both stiffness and strength responses and make the effects of vertical loading on lateral capacity negligible (Germano et al. 2015, Winkel and Smith 2010).

Early the finite element (FE) simulations for the LFWBs were based on simplified two-dimensional modeling for structural components such as the shear walls and the floor diaphragms or the entire houses with linear behavior and static loading (Mahaney and Kehoe 1988, Nateghi 1988). Kasal et al. (1994) developed an equivalent 3-D FE model for a full-scale LFW-house with assembling of the roofs/floors (Linear super-elements), the shear walls (quasi super-elements), and the connection between the walls to walls and the walls to roofs/floors (inter-component connections). The quasi super element consists of the series of linear trusses, orthotropic plates, beams, and a nonlinear spring elements. The main idea of the concept was the total potential energy of the equivalent FE model was equal to the original structure. In other words, the quasi super-elements was required to experience the same displacements subjected to the equivalent loads on the number of boundary nodes as the original substructures. They concluded that the FE model was less accurate for small loads and the concept of super-elements was applicable to those linear parts of the structure. Collins et al. 2005 simulated the in-plane action and out-of-plane stiffness using commercial software package ANSYS. The diagonal hysteretic nonlinear springs represented the wood shear walls, the plate element represented out-of-plane stiffness and the coupling of nail connections was not taken into account. This model gave accurate results in distribution of mass for seismic analysis. Nevertheless, it cannot predict in- and out-of-plane responses simultaneously and the behavior of connection between floors correctly. Xu and Dolan (2009) developed a FE model of simple two storey light-framed wood structure with similar approach in commercial software package ABAQUS. The model can predict nonlinear wood structural response with acceptable errors. As part of NEESWood Project, the SAPWood program (Seismic Analysis Package for woodframe Structures, Version 2.0) was developed for seismic analysis of wood frame structures (Pei and Van de Lindt 2007). This tool was used to simulate the multi-storey LFWBs. The shear-bending model was proposed where a vertical double linear spring (or nonlinear spring) simulated the vertical stiffness and a horizontal spring simulated the lateral stiffness of wood shear walls. Also, the floor system was assumed to be rigid. The inter-storey drift and shear deformation response of the LFWBs were compared and good agreement was found (Pei and Van de Lindt 2011). However, the numerical model is only accurate if 1) the stiffness of the wood shear walls is properly calibrated with the hysteretic test data or 2) the SAPWood-Nail Pattern is used to implement the stiffness of sub-assembly members including the studs, sheathing, and nails based on the principle of virtual work (Pei and Van de Lindt 2009).

There are two kinds of nail connections: frame-to-frame and frame-to-sheathing connections. The formation of plastic hinge in the shank part of nails during cyclic tests proved the importance role of the nonlinear behavior of nails on both the strength and the stiffness of the LFWS under lateral loads (Germano, Giovanni, and Giuriani 2015). Nevertheless, the most challenging part in the FE simulation of the LFWS is how to implement the nonlinear behaviour of nails as the main source of energy dissipation and the structural continuity to predict accurate lateral performance of the LFWS. In this study, a detailed FE model including studs, sheathing, top and bottom plates, wall anchors, and nails is developed for a typical one storey shear wall. The nonlinear FE model is developed using the commercial software ETABS 2016. The FE model is verified with two experimental test results from Winkel and Smith (2010) and Sinha and Gupta (2009). A sensitivity analysis is performed to recognize the important parameters in overall behavior of wood shear walls under lateral loads. Furthermore, the FE model is used to simulate a multi-storey wood shear wall to assess the structural performance under lateral loads and top deflection values that are compared with CSA-O86-14 deflection procedure for multi-storey LFWS.

2 DESCRIPTION OF FINITE ELEMENT MODEL FOR LIGHT-FRAMED WOOD SHEAR WALLS

2.1 One-Storey Wood Shear Walls

In the current study, two different walls are selected. The description of walls is presented as the follows:

Wall A is 2440×2440 mm with one side 11.1 mm thickness OSB sheathing panels (1220×2440 mm). The frame consists of five vertical studs (38×89 mm SPF lumber @ 610 mm), a top horizontal plate (double studs with same dimension), and a bottom horizontal plate (one stud with same dimension). The wall is fixed to two steel boxes, at the top and the bottom, through four anchor bolts for each side (Figure 1). Two groups of nails are used in wall A: frame-to-frame (FF) spiral nails (3.86 mm diameter & 89 mm length) and sheathing-to-frame (SF) common nails (2.95 mm diameter & 57 mm length). The spacing between SF nails is 152 mm along panel edge and 305 mm in panel interiors. Wall A was tasted by Winkel and Smith (2010) using a monotonic loading to record the load-displacement curves.

Wall B considers the effect of hold-down anchorage system on the LFWS. The dimensions of wall B are the same as Wall A. However, the double end studs are used for anchorage system as shown in Figure 1. Sinha and Gupta (2009) did experimental research on the LFWS which had the similar configuration for Wall B.

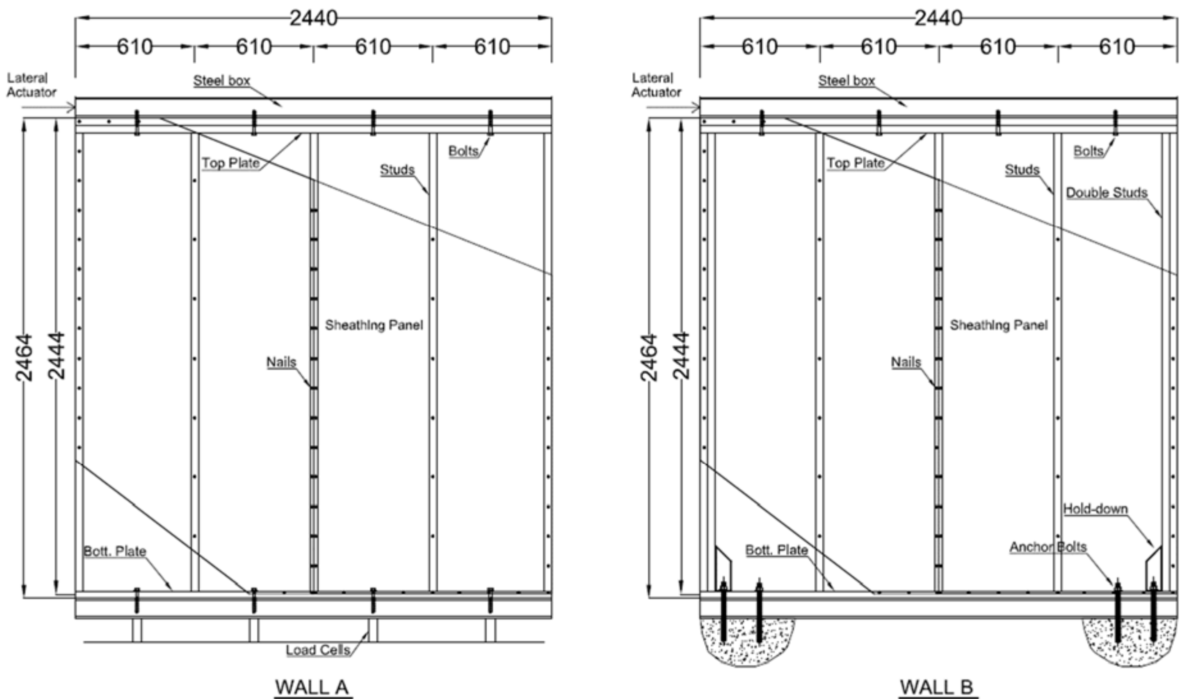

Figure 1: Schematic View of a) Wall A; b) Wall B

The developed finite element model is based on modelling all the LFW shear wall components including studs, sheathing, top and bottom plates, hold-downs, and nail connections regarding their actual material properties using ETABS software (CSI 2016). Studs, bottom plate, and top plate are modeled as orthotropic frame elements and sheathing panels modeled as orthotropic shell elements using the material properties provided in Table 5.1 by Winkel (2006). For double top plate and double studs, full strain compatibility between two members is assumed. The nails and the bolts are simulated using the link element. For the link element, the linear/nonlinear force-deformation and moment-rotation relationships for the six degrees of freedom (DOFs) (U_1, U_2, U_3, R_1, R_2 and R_3) can be defined where $U_{1,2,3}$ are the transitional DOFs and $R_{1,2,3}$ are the rotational DOFs. The FF and SF nails are modeled using the nonlinear link elements. The nonlinear force-displacement curves provided by Winkel (2006) are used. The anchor bolts are modeled using linear link element and the linear stiffness are based on the values provided by Winkel (2006). The contact surface between the (top and bottom) plates with the steel box is simulated using the link element with gap property. For the gap element (compression only), the linear compression stiffness of plates is specified for U_1 DOF and the initial zero gap opening is assumed. The friction between the two surfaces is neglected for the FE modelling and the anchor bolts are used to transfer all the loads to the steel box.

Figures 2 and 3 shows the extruded view and different components of the detailed FE model for Wall A respectively. For modelling Wall B, the same approach with same material properties noted above is used. The two brackets are simulated with a pair of compatibility link element. In both models, an incremental monotonic displacement-controlled lateral loading under nonlinear-static analysis is applied at the upper level steel box. The effect of initial imperfections and out-of-plane buckling of sheathing are not considered due to the numerical model limitation of geometric nonlinearity option for such complex models.

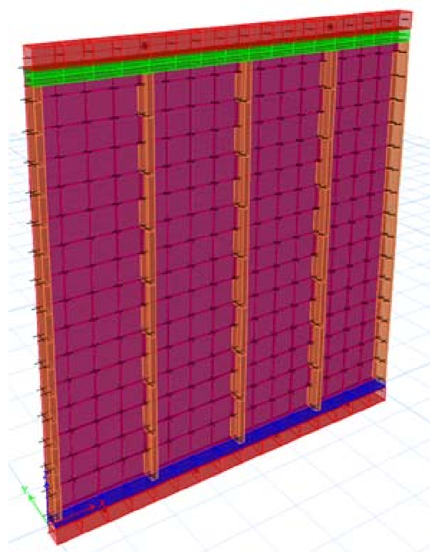

Figure 2: Three-dimensional Detailed Finite Element Model

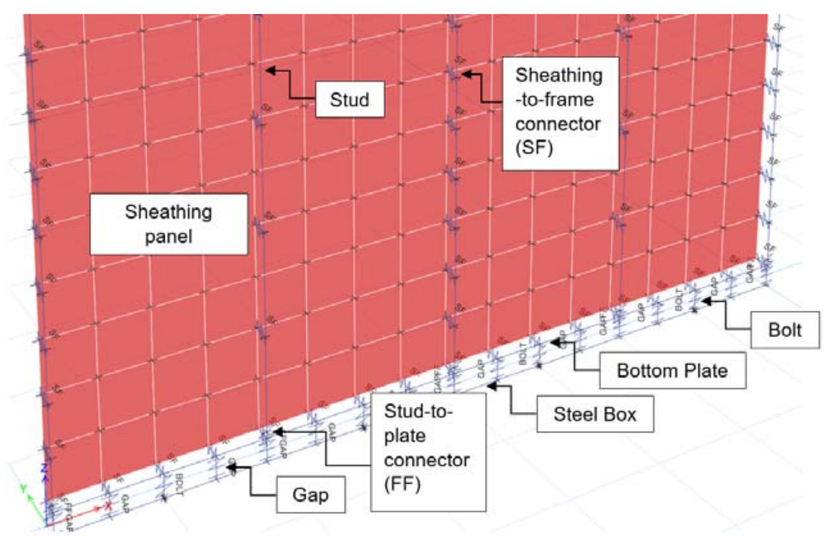

Figure 3: The FE Model Components

2.2 Multi-Storey Wood Shear Walls

In multi-storey wood shear walls, the tie-down system is added to resist overturning moments, dissipate the wood shrinkage over service time, and decrease inter-storey drift deflection. Accordingly, the tie-down elements are tension only elements and are modeled using the nonlinear hook link element. The hook element axial linear stiffness is derived from the most common tie-down system available.

Two different assumptions simulating the connections between shear walls from one floor to another at floor levels are investigated in the current study. First, both upper wall segment and wall below is connected to the floor system using nails only. The first system is a very flexible. It is recommended by wood design manual (CWC 2015) to add anchor bolts to connect both floor diaphragm to shear walls and shear walls to each other. In second system, the adjacent wall segments are connected using multiple anchor bolts. The nonlinear FE model of Wall A is used to simulate a six-storey unblocked shear walls. The two mentioned floor connection systems are examined including two cases for the anchor bolts option: a) 4 bolts plus gap elements, b) 5 bolts plus gap elements and one case nails only (Figure 4). A concentrated lateral load (5 KN) is applied to the top of each floor and the results of top deflection in the sixth floor are recorded.

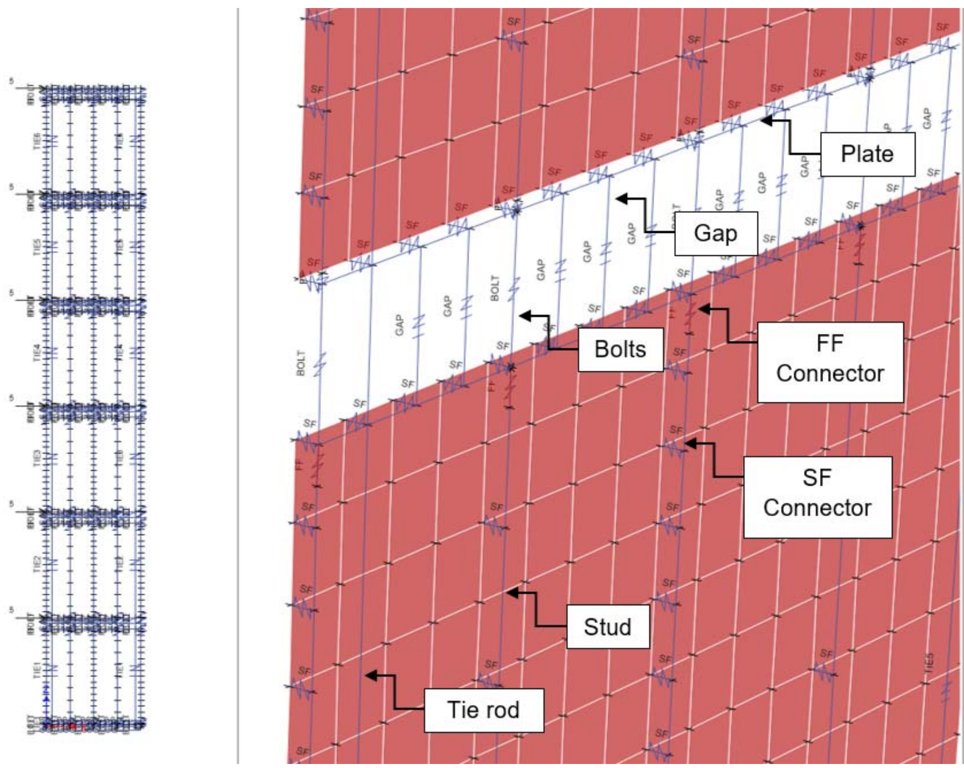

Figure 4: FE Model of Multi-storey Wood Shear Wall

3 RESULTS AND DISCUSSION

3.1 Verification of Finite Element Model

Wall A: The force-displacement curve resulting from the FE simulation in the current study is compared with the experimental test results by Winkel and Smith (2010) as shown in Figure 5. They performed two experimental tests while the test specimens are completely the same. The load-deflection curve in test #1 shows a sudden change in stiffness at an approximate load of 2.3 KN. This stiffness change can be due to the experiment boundary conditions and specimen supports. Furthermore, the secant line approach between 10 and 40% of the peak values is used to determine the stiffness coefficient. The stiffness coefficient of the finite element model is 0.24 KN/m/mm which is in good agreement with the test #2 result (0.23 KN/m/mm). However, the stiffness coefficient of test# 1 was 0.19 KN/m/mm. The post yielding results from the FE model is in good agreement with the experimental results (Only 4% difference in wall lateral strength). The FE model did not capture the descending part of the load-deflection curve accurately, but still within acceptable limits.

Wall B: The comparison between force-displacement results of the test conducted by Sinha and Gupta (2009) and the FE model developed in the current study is shown in Figure 6. The initial stiffness of force-displacement curve for both cases are in good agreement. A difference of approximately 19 % in the wall strength is noticed. This difference between the FE model and the experimental test can be due to not capturing the precise post-buckling behavior of wood shear wall by the FE model results, similar to the results shown in figure 5.

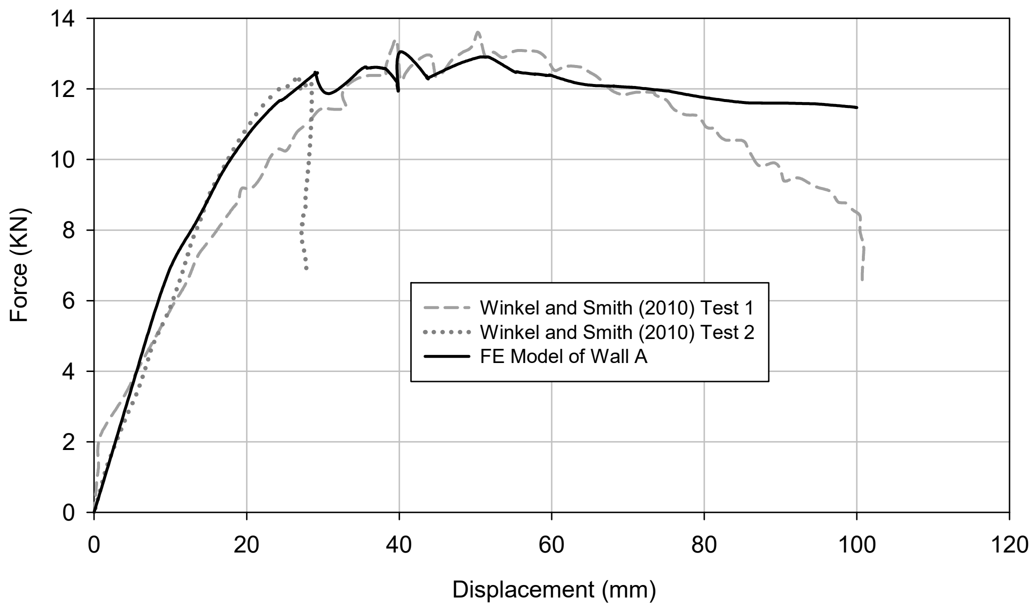

Figure 5: Wall A Verification

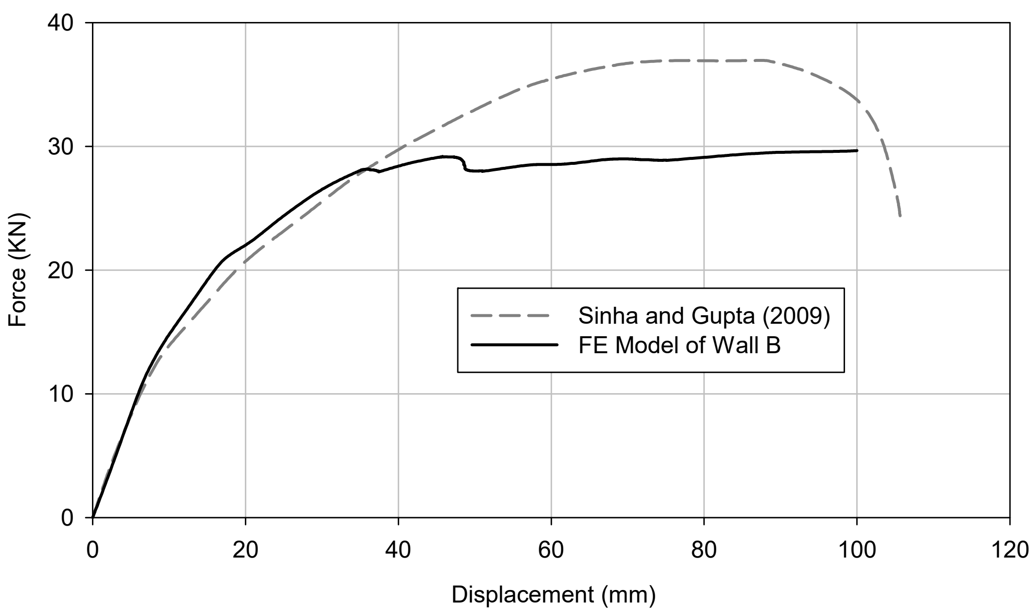

Figure 6: Wall B Verification

3.2 Sensitivity Study

A sensitivity analysis is carried out to identify how different parameters affect the lateral capacity of the LFWS. The study focused on the alteration of components' size, the nonlinear behaviour of nails, and the different element connections. The parameters considered in the sensitivity analyses are as follows:

1. The OSB sheathing thickness of 7.5, 9.5, 11.1, and 15 mm.
2. The S-P-F lumber stud sizes of 38 × 89 and 38 × 140 mm.
3. The shear perpendicular to grain properties of sheathing-to-frame (2.95-mm diameter) nails up to ±100 % changes in initial stiffness.
4. The shear parallel to grain properties of sheathing-to-frame (2.95-mm diameter) nails up to ±100 % changes in initial stiffness.
5. The shear properties of frame-to-frame (3.86-mm diameter) nails up to ±100 % changes in initial stiffness.
6. The tension properties of frame-to-frame (3.86-mm diameter) nails up to ±100 % changes in initial stiffness. Note: The compression properties of the FF nails were assumed to be govern by compression capacity of the S-P-F lumber studs
7. The shear and tension properties of the anchor (3/4 in) bolts up to ±100 % changes in linear stiffness.
8. The number of gap elements from half to two times.
9. The blocking effects between studs.

The level of sensitivity of the modeled parameters is summarized in Table 1. The results showed that nonlinear shear stiffness of the SF connectors is the most critical factor and anchor bolts get in to second place. It can be predictable that the sheathing thickness contribute more than the stud size on lateral performance of the LFWS. The blocking is least effective parameter for the lateral stiffness. Although, it prevents any premature failure of the studs by decreasing the buckling effective length.

Table 1: The Sensitivity Level of Modeling Parameters

Sheathing thickness	Stud size	Sheathing to frame connector (SF)		Frame to frame connector (FF)			Anchor bolts		Gap No. and mesh size	Blocking
		Shear ⊥ to grain	Shear ‖ to grain	Shear	Tension	Comp.	Shear	Tension		
45%	32%	71%	39%	15%	28%	9%	42%	8%	21%	7%

3.3 Top Deformation Comparison for Multi-Storey Wood Shear Wall

CSA-O86-14 Clause A.11.7.1 calculates the total deflection of blocked shear walls in multi-storey buildings, as shown in equation [1] below. The equation is the accumulated inter-storey drift at each level due to bending, panel shear, nail slip, and vertical elongation of the wall anchorage system. The equation is based on the assumption that LFW shear wall is cantilevered from its base and takes into account the cumulative rotation due to bending and wall anchorage system elongation. Panel shear deformation and nail slip deformation are calculated per floor and will not affect the drift in adjacent level above.

[1] $\Delta_{i,blocked}^{total} = \sum_{j=1}^{i} \Delta_{j}^{storey} = \sum_{j=1}^{i} (\Delta_{b,j}^{storey} + \Delta_{s,j}^{storey} + \Delta_{n,j}^{storey} + \Delta_{a,j}^{storey})$

Tables 2 and 3 show the summary of the code and the FE model results for determining top lateral displacement. The total top deflection prediction of the FE model using only five bolts for connecting the shear wall segments between the floors has only 1% difference compared with eq. [1] calculations. By decreasing the number of anchor bolts to 4, the FE model top deflection for the blocked LFW shear walls is 43% higher than CSA O86-14 eq. [1] as shown in Table 3. Furthermore, when using the nail connection system between shear walls and both floor diaphragm and rim boards, the FE model gives the lowest value of 166.84 mm for the top deflection. Based on the results shown in Table 3, it is recommended to use both anchor bolts and nails to connect LFW shear walls segments to floor diaphragm and between floors.

Table 2: The Top Lateral Displacement Calculation Based on Clause A.11.7.1-CSA-O86-14

Total Deflection						Blocked
Storey	Δb (mm)	Δs (mm)	Δn (mm)	Δa (mm)	Inter storey drift Δ(storey) (mm)	Total deflection Δ(storey) (mm)
Storey 6	4.919	0.581	1.48	3.063	10.041	182.731
Storey 5	4.758	1.162	5.91	2.963	14.794	172.690
Storey 4	4.443	1.744	13.30	2.793	22.277	157.896
Storey 3	3.804	2.325	23.64	2.518	32.288	135.619
Storey 2	2.680	2.906	36.94	1.959	44.485	103.332
Storey 1	1.029	3.487	53.19	1.137	58.847	58.847

Table 3: Top lateral displacement comparison between FE model and hand calculation

Model	Floor connection	FE model result (mm)	Difference (blocked)
1	4 Bolts + Gap	261.58	+43%
2	5 Bolts + Gap	180.2	-1%
3	FF only	166.84	-9%

4 CONCLUSION

A detailed FE model for simulating the behavior of the LFWS under lateral loads is presented. The proposed nonlinear FE model is verified with two different experimental tests that are available in literature. Sensitivity analyses are conducted to investigate different parameters that affect the lateral stiffness and the strength of the LFWS. The multi-storey LFWS FE models are developed using the same approach. The multi-storey models assess the effect of different wall to wall and wall to diaphragm connection on the building lateral drift. The following conclusions can be drawn from the current study:

- The proposed nonlinear FE model can simulate the initial stiffness of the LFWS. However, it can not perfectly envisage the post buckling behavior of the LFWS.
- The sheathing-to-frame (SF) nails nonlinear shear stiffness perpendicular to grain and the blocking effects are recognized as the most and least effective parameters in the lateral performance of the LFWS, respectively.
- The continuity or the connection system of the shear walls in adjacent floors can be critical for the lateral performance of the multi-storey LFWS. Thus, the adequate lateral load connection is needed to connect the shear walls segments.

- Adding the hold-down (anchor bolts) system in the multi-storey LFWS can decrease the maximum lateral deflection effectively.

Current studies by the authors show that the light-framed wood shear walls lateral deflection is also affected by the wall geometry and the height to length ratio. Further studies are needed to fully understand the different parameters that affect the lateral deflection and behaviour of the multi-storey light-framed wood shear walls buildings under lateral loads.

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References

Breyer, D.E., Frifley, K.J., Cobeen, K. E., and Pollock, D.G. 2007. Design of wood structures. New York: ASD, 6th Ed., McGraw-Hill.

Collins, M., Kasal, B., Paevere, P., and Foliente, G.C. 2005. Three-dimensional model of light frame wood buildings. I: Model description. Journal of Structural Engineering, ASCE 131(4): 676-683.

CSA-O86. 2014. Engineering Design in Wood. Mississauga ON: CSA Group.

CSI. 2016. Analysis reference manual. ETABS: Computer & Structures, INC, Structural and Earthquake Engineering Software.

CWC. 2015. Wood design manual. Ottawa ON: Canadian Wood Council.

Doudak, Gh. and Smith, I. 2009. Capacities of OSB-sheathed light-frame shear-wall panels with or without perforations. Journal of Structural Engineering, ASCE 135(30): 326-329.

Germano, F., Giovanni, M., and Giuriani, E. 2015. Experimental results on the role of sheathing-to-frame and base connections of a European timber framed shear wall. Construction and Building Materials, Elsevier 80: 315-328.

Kasal, B., Leichti, R.J., and Itani, R.Y. 1994. Nonlinear finite element model of complete light-frame wood structures. Journal of Structural Engineering, ASCE 120(1): 100-119.

Kirkham, W., Gupta, R., and Miller, Th. 2014. State of the art: seismic behavior of wood-frame residential structures. Journal of Structural Engineering, ASCE 140(4).

Mahaney, J.A. and Kehoe, B.E. 1988. A rational design procedure for multistory buildings with wood diaphragms. International Timber Engineering Conference. Madison WI: Forest Products Research Society. 303-316.

Nateghi, F. 1988. Analysis of wind force on light-frame timber structures. Doctoral dissertation, Department of Civil Engineering, Columbia, MO: University of Missouri.

Pei, S. and Van de Lindt, J.W. 2007. SAPWood User's Manual. Released with SAPWood Program. http://www.engr.colostate.edu/NEESWood/SAPWood.htm.

Pei, S. and Van de Lindt, J.W. 2009. Coupled shear-bending formulation for seismic analysis of stacked wood shear wall systems. Earthquake Engineering & Structural Dynamics (Wiley Inter Science) 38:1631–1647. doi:10.1002/eqe.926.

Pei, S. and Van de Lindt, J.W. 2011. Seismic Numerical Modeling of a Six-Story Light-frame Wood Building: Comparison with Experiments. Journal of Earthquake Engineering 15:6, 924-941. doi:10.1080/13632469.2010.544840.

Sinha, A. and Gupta, R. 2009. Strain Distribution in OSB and GWB in wood-frame shear walls. Journal of Structural Engineering, ASCE 135(6): 666-675.

Varoglu, E., Karacabeyli, E., Stiemer, S., and Ni, Ch. 2006. Midply wood shear wall system: concept and performance in static and cyclic testing. Journal of Structural Engineering, ASCE 132(9): 1417-1425.

Winkel, M.H. and Smith, I. 2010. Structural behavior of wood light-frame wall segments subjected to in-plane and out-of-plane forces. Journal of Structural Engineering, ASCE 136(7): 826-836.

Winkel, M.H. 2006. Behavior of light-frame walls subject to combined in-plane and out of plane loads. MS thesis, Fredercton, NB, Canada: University of New Brunswick.

Xu, J. and Dolan, D. 2009. Development of a wood-frame shear wall model in ABAQUS. Journal of Structural Engineering, ASCE 135(8): 977-984.