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NUMERICAL MODELLING ANALYSIS OF HIGH-CAPACITY SHEAR WALLS WITH MULTIPLE ROWS OF NAILS: FAILURE MODES AND PARAMETRIC STUDY

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ABSTRACT: A new high-capacity wood frame shear wall system with two and three rows of nails on sheathing edges was jointly developed by the University of Victoria and FPInnovations in response to the increased demand for stronger shear wall systems. A three-year period test program was carried out in 2020-2022. Results showed that the proposed high-capacity shear wall system can achieve lateral resistance proportional to the number of rows of nails on sheathing edges. Detailed 3D numerical models of high-capacity shear walls with multiple rows of nails had been developed via ABAQUS and verified by the test results. This study presents a parametric study based on the verified shear wall models, considering different wall configurations that were not included in the previous test programs, i.e., high-capacity shear walls with different heights, lengths, stud sizes, and sheathing arrangements. Results show that the lateral load capacity of high-capacity shear walls were proportional to the wall length, while the stiffness decreases as shear wall height increases. Stud sizes had no significant effect on wall performance, while sheathing panel size had an important role in terms of shear wall resistance and deformability.

KEYWORDS: high-capacity shear wall, numerical modelling, parametric study, failure mode

1 – INTRODUCTION

Light wood frame shear wall system, as the main vertical component resisting lateral loads caused by wind and earthquake, has been widely used in the construction of low-rise up to four-storey residential and commercial wood frame buildings in North America. With the increased height limit of wood frame buildings up to six-storey and the increased seismic design spectra in the national building code of Canada (NBCC) [1,2], the demand for higher lateral load resisting systems for mid-rise wood frame buildings has increased, especially in high seismic zones. A high-capacity shear wall system with multiple rows of nails along sheathing edges was jointly developed by FPInnovations and the University of Victoria. A three-year testing program of the high-capacity shear walls had been completed in 2020-2022 [3–5]. Results showed that the lateral resistance of shear walls with two and three rows of nails was around two and three times than that of a standard shear wall. However, the occurrence of new brittle failure modes, such as splitting of plates and studs, separation of studs from plates, sheathing panel rupture or buckling, had affected the shear wall ultimate displacement and ductility. New design configurations to prevent these undesirable failure modes are being investigated through both experimental testing and numerical modelling analysis. This paper mainly discusses the numerical modelling results.

The commonly adopted numerical modelling methods for wood frame shear walls, such as using beam and shell elements for framing and sheathing, respectively [6], and assuming pin connections between studs and plates, are usually sufficient for regular shear walls when the performance of walls is mainly governed by sheathing-to-framing nail joints. However, to study the behaviour and failure modes of high-capacity shear walls which may fail on other components of shear walls, more realistic assumptions on wall configurations, including connections between framing members, anchorage to foundations, etc. should be considered. A detailed 3D shear wall model had been developed and verified using ABAQUS to predict the overall performance and failure modes of high-capacity shear walls [7]. This paper focuses on the parametric study of high-capacity shear

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walls with configurations not covered in the previous test programs. The objective of this study is to get a better understanding on the effect of construction details and configurations on the behaviour and failure modes of high-capacity shear walls, and to provide insights for future design and testing.

2 – NUMERICAL MODELLING METHOD

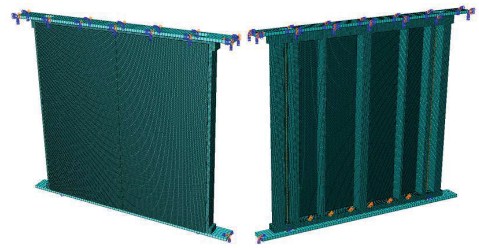
2.1 MODEL DESCRIPTION

Numerical models of shear walls with one, two and three rows of nails had been developed using ABAQUS by the authors [7]. The shear wall models consisted of Douglas-Fir framing members which were represented by solid elements (C3D8R), and OSB sheathing panels which were modelled with continuum shell elements (CS8R). The framing members and sheathing panels were both assumed to be linear elastic. The shear walls were sheathed only on one side by either 8d or 10d common nails. The sheathing-to-framing nails were represented by fastener elements with elastic, plastic and damage behaviour in shear and withdrawal directions. For elastic behaviour, spring stiffness D was assigned for each local direction. Coupled plastic behaviour with nonlinear isotropic hardening (analogous to metal plastic hardening in ABAQUS [8]) was defined using exponential law as shown in (1):

$$F_0 = F|_0 + Q_{inf}\left(1 - e^{-b\bar{u}^{pl}}\right) \tag{1}$$

where F_0 is the yield surface size defined as the equivalent force in the connector, $F|_0$ is the yield force at zero plastic displacement, Q_{inf} is the maximum change of yield surface, $\bar{u}^{pl}$ is the equivalent relative plastic displacement, b is the rate of the change of the yield surface. The same fastener elements were assigned to the nails for built-up studs and plates, and nails connecting studs to plates, with adjustment factors considering different nail sizes and end grain nailing. Table 1 summarizes the input properties for the sheathing-to-framing nails and nails used in framing members. More detailed material properties of wood components and descriptions on fitting process to derive the nail joint element properties can be found in Qiang et al.[7].

Fig. 1 shows the details of the 3D shear wall models. Steel loading beam was tied to the top plates of the shear wall, while the bottom plates were pinned to the rigid foundation beam. Pre-drilled holes in top and bottom plates represented the location and size of the anchor bolts. Constraints were applied to a surface area the same as the size of the bolt plate washers. Continuous steel rod hold-downs were modelled by axial connector elements, where the top of the steel rod was fixed with the top plates over a surface area the same as the bearing plate, and the bottom of the steel rod element was fixed at the foundation beam. A monotonic lateral displacement was applied through the end of the loading beam which was laterally supported so that its out-of-plane movement was prevented.



(a) *(b)*

Figure 1. Shear wall model: (a) Front view, (b) Back view.

2.2 MODEL VERIFICATION

The shear wall models had been verified by comparing the load-displacement curves derived from the numerical modelling analysis with that of the tested shear walls in 2021 [7], which includes walls of 8 ft × 8 ft (2.4 m × 2.4 m) with different sheathing thicknesses (11 mm and 15 mm), nail sizes (8d and 10d), nail spacing on panel edges (75 mm and 100 mm). Results showed that the curves of the shear wall model agreed well with the envelope curves of the hysteresis loops of tested walls. Fig. 2 shows one example of the comparisons. The models could well capture the yield load and peak resistance of high-capacity shear walls with low discrepancy. Meanwhile, the model could well mimic the failure modes of sheathing-to-framing nail joints, end stud separation from bottom plates (Fig. 3). However, rupture or buckling of sheathing panels and splitting of plates cannot be simulated by the current model due to the limitations on the assumption that sheathing and framing were linear elastic materials [7].

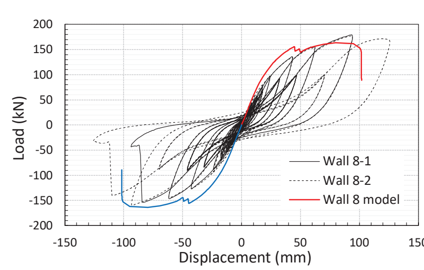


Figure 2. Comparison of test hysteresis loops and numerical modelling load-displacement curve (Wall 8) [7].

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Table 1: Properties of fastener elements used for sheathing-to-framing nails and nails connecting framing members

Connector property	Sheathing-to-framing nails		Framing nails	
	8d	*10d*	*Built-up member*	*Stud-to-plate*
D_{11} (N/mm)	734.3	980.6	811.5	543.7
D_{22} (N/mm)	734.3	980.6	811.5[2]	543.7[2]
D_{33}[1] (N/mm)	9.6	10.6	811.5	543.7
$F\|_0$ (N)	856.1	1100.1	946.0	633.8
Q_{inf} (N)	539.4	743.1	596.1	399.4
b	0.4	0.4	0.4	0.4
Plastic displacement at damage initiation (mm)	7.6	7.5	7.6	7.6
Plastic displacement at failure (mm)	30	30	30	30

1. D_{33} is the withdrawal stiffness of sheathing-to-framing nails based on the local coordinates, which is assumed to be a small percentage of the lateral stiffness of a nail joint considering that the withdrawal resistance is limited after yielding of nails in shear and losing of friction from the surrounding wood when the nail is bent and pulled out from studs.

2. For built-up members and stud-to-plate connections, D_{22} is the withdrawal stiffness based on the local coordinates. There was no reduction of the withdrawal stiffness applied.

(a) (b)

Figure 3. Separation of end studs from bottom plates: (a) Test, (b) Model [7].

3 – PARAMETRIC ANAYSIS

Using the verified shear wall model, a parametric study was carried out to expand the configurations of high-capacity walls that were not covered in previous test programs. The parameters considered in this parametric study are shown in Table 2 which include the effect of wall height, wall length, stud size, and sheathing arrangement. Each parametric model was compared to its reference case. As monotonic loading was applied to the shear walls in numerical modelling analysis, the envelope curves of tested hysteresis loops were used to derive the EEEP (Equivalent Energy Elastic-Plastic) parameters in accordance with ASTM E2126 [9]. Fig. 4 shows the typical envelope curve and it's corresponding EEEP curve of a shear wall, 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 and Δ_{yield} is the corresponding displacement; the ultimate displacement Δ_u is 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.

To prevent the typical brittle failure modes of high-capacity shear walls in the parametric models, such as separation of end studs from bottom plates, or separation of centre studs, the stud-to-plate connections and built-up stud connections were strengthened with additional nails. Detailed discussion is presented in the following sections.

3.1 WALL HEIGHT EFFECT

A standard shear wall configuration consisting of two vertically sheathed standard size panels of 4 ft × 8 ft (1.2 m × 2.4 m), and taller shear walls with over-sized sheathing panels of 4 ft × 9 ft (1.2 m × 2.7 m) and 4 ft × 10 ft (1.2 m × 3.0 m) were compared in this parametric analysis (Fig. 5). The detailed configurations are listed in Table 3. The 8 ft × 8 ft shear wall with three rows of nails on sheathing edges was used as a reference case.

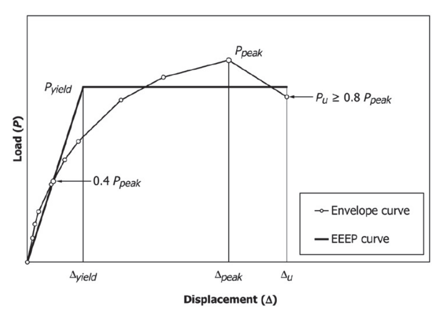

Figure 4. EEEP parameters and envelope curve [9].

Table 2: Parametric study matrix

Parameters	Configuration details
Wall height	Wall heights of 2.4 m (8 ft), 2.7 m (9 ft), and 3.0 m (10 ft)
Wall length	Wall lengths of 1.2 m (4 ft), 2.4 m (8 ft), and 4.8 m (16 ft)
Stud size	2 × 4 and 2 × 6 dimension lumber
Sheathing arrangement	Vertically sheathing, horizontally sheathing, large panel (8 ft × 8 ft)

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Table 3: Parametric analysis matrix of wall height

Parameter	Wall size (ft × ft)	Panel size (ft × ft)	Sheathing Thickness (in)	Row of nails	Nail size	Nail spacing (in)	Sheathing orientation	Stud size	Aspect ratio	Hold down
Wall height	8 × 8	4 × 8	19/32	3	10d	3	Vertical	2 × 6	1	Tie rod
	8 × 9	4 × 9	19/32	3	10d	3	Vertical	2 × 6	1.125	Tie rod
	8 × 10	4 × 10	19/32	3	10d	3	Vertical	2 × 6	1.25	Tie rod

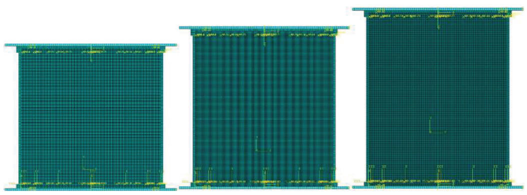

Figure 5. Shear wall models with different wall heights.

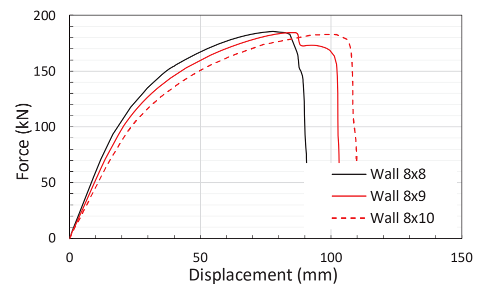

Figure 6. Load-displacement curves of Wall 8×8, Wall 8×9 and Wall 8×10.

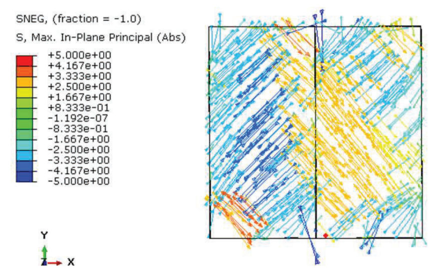

Figure 7. Principle stress in sheathing panels.

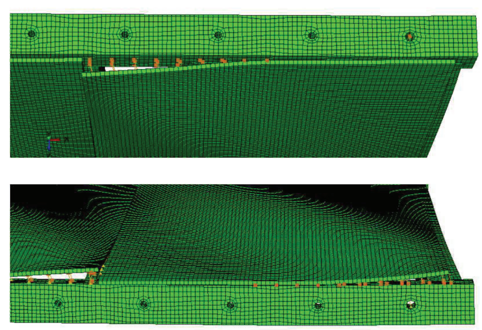

Figure 8. Sheathing deformation at panel corners

One of the common failure modes of high-capacity shear walls with three rows of nails observed in previous tests is the separation of end stud from bottom plates due to the out-of-plane moment amplified by the increased lateral load and eccentricity [5]. To prevent this failure in the real test scenario, construction details such as steel angles connecting end stud to plates were adopted. However, in the numerical model, simplified methods were preferred to achieve strengthening effect and reduce computational cost. Thus, in this parametric study, additional nails were added to end studs-to-plates (9 nails in modelling compared to 4 nails in the tests) and centre studs-to-plates (9 nails in modelling compared to 6 nails in the tests) connections, which has been proven to be efficient to prevent stud separation from top or bottom plates in the numerical models. Fig. 7 shows the principal stress developed in the sheathing panels, on which the compressive and tensile stresses are diagonal. It indicates that sheathing rupture may occur in thinner panels where panel tensile resistance is low [7]. Fig. 8 shows the separation of sheathing panels from studs and plates at the corners, which indicates that nail joints on sheathing corners have larger deformation and earlier failure compared to nails elsewhere.

The load-displacement curves of shear walls with different heights (8 ft, 9 ft and 10 ft) are similar in terms of stiffness and peak resistance (Fig. 6). From Table 4, it can be seen that the stiffness slightly decreases with the increase of wall height. While the yield displacement and ultimate displacement increase with the increase of wall height. The effect of wall height on the yield and peak loads can be ignored.

The increase of ultimate displacement with the increase of wall height is also reflected by the four-term deflection equation in CSA O86-24 [10]:

$$\Delta_{sw} = \frac{2vH_s^3}{3EAL_s} + \frac{vH_s}{B_v} + 0.0025H_s e_n + \frac{H_s}{L_s} d_a \qquad (2)$$

where v is the shear due to specified load at the top of the shear wall per unit length; H_s and L_s are the height and length of the shear wall, respectively; E, A are the modulus of elasticity and area of the boundary stud members, respectively; B_v is the shear through thickness rigidity of the sheathing panel; e_n is the nail deformation; and d_a is the vertical elongation of the hold-down system. The equation shows that with the same wall length and

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Table 4: Mechanical properties of different wall height models

Wall #	K_e (kN/mm)	Δ_{yield} (mm)	P_{yield} (kN)	Δ_{peak} (mm)	P_{peak} (kN)	Δ_u (mm)	μ (Δ_u/Δ_{yield})
8 × 8	5.9	48.2	165.2	77.8	185.3	88.7	1.84
8 × 9	5.2	53.7	163.8	85.8	184.6	102.4	1.91
8 × 10	4.5	62.6	164.3	99.5	182.9	108.2	1.73

configuration, the deflection Δ_{sw} is positively related to wall height H_s:

3.2 WALL LENGTH EFFECT

Experimental studies on standard shear walls with different wall lengths have been carried out by many researchers, ranging from aspect ratios from 2/3 to 6 [11–13]. In this parametric study, high-capacity shear walls with different lengths of 4 ft and 16 ft (1.2 m and 4.8 m) consisting of one and four 4 ft × 8 ft (1.2 m × 2.4 m) sheathing panels, respectively, were compared with the standard wall length of 8 ft (2.4 m) (Fig. 9). The detailed configurations are shown in Table 5.

It can be seen that there's a direct relationship between the high-capacity wall length and its load-displacement response (Fig. 10), which is consistent with the findings from experimental studies for standard walls [11–13]. Table 6 shows that the peak resistance P_{peak} is proportional to wall length, while the stiffnesses K_e of 4 ft long and 16 ft long walls are 0.4 and 2.4 times that of a 8 ft long wall, respectively, which is consistent with (2), as the first and fourth items are inversely proportional to wall length L_s while the other two items remain constant for the same wall configurations. The deflection Δ_{sw} at design level decreases with the increase of wall length, so as the Δ_{yield}, leading to larger increment in wall stiffness.

The ultimate displacement Δ_u decreases with the increase of wall length. Wall 4 × 8 has the largest ultimate displacement while its aspect ratio also is the largest (2.0). Similar to the observation from previous section (Table 4), larger aspect ratio leads to larger ultimate displacement of the shear wall.

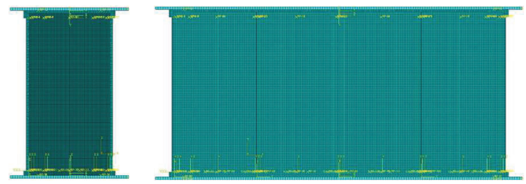

Figure 9. Shear wall models with different wall lengths (4 ft. and 16 ft.).

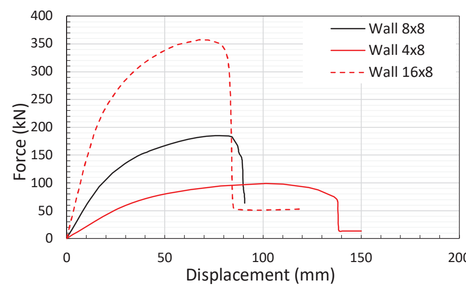

Figure 10. Load-displacement curves of Wall 4×8, Wall 8×8 and Wall 16×8.

After increasing the number of nails in stud-to-plate connections as described in the previous section, the separation of studs from plates was prevented, as shown in Fig. 11. There was no brittle failure mode observed except in Wall 16 × 8, where the sheathing-to-framing nail joints on the top edges of the sheathing panels failed simultaneously after the wall reached its peak resistance. The deformed shape of the sheathing panel shown in Fig. 12 indicates possible buckling failure of the panel. Similar failure mode was observed in one of the shear walls in test programs [5].

Table 5: Parametric analysis matrix of wall length

Parameter	Wall size (ft × ft)	Panel size (ft × ft)	Sheathing Thickness (in)	Row of nails	Nail size	Nail spacing (in)	Sheathing orientation	Stud size	Aspect ratio	Hold down
Wall length	8 × 8	4 × 8	19/32	3	10d	3	Vertical	2 × 6	1	Tie rod
	4 × 8	4 × 8	19/32	3	10d	3	Vertical	2 × 6	2	Tie rod
	16 × 8	4 × 8	19/32	3	10d	3	Vertical	2 × 6	0.5	Tie rod

Table 6: Mechanical properties of different wall length models

Wall #	K_e (kN/mm)	Δ_{yield} (mm)	P_{yield} (kN)	Δ_{peak} (mm)	P_{peak} (kN)	Δ_u (mm)	μ (Δ_u/Δ_{yield})
4 × 8	2.1	66.2	89.5	101.5	99.0	133.7	2.02
8 × 8	5.9	48.2	165.2	77.8	185.3	88.7	1.84
16 × 8	14.2	42.4	322.9	67.6	357.5	83.1	1.96

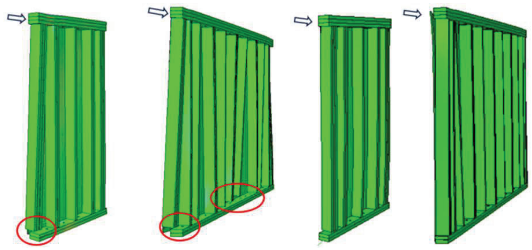
(a) (b)

Figure 11. Stud separation from plates in Wall 4 × 8 and Wall 16×8, (a) Before adding nails to stud-to-plate connections, (b) After adding nails to stud-to-plate connections.

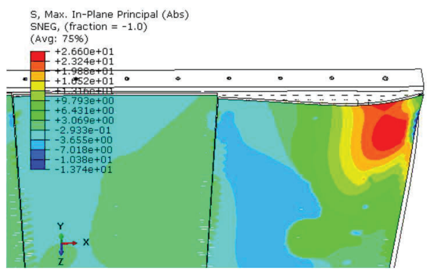

Figure 12. Sheathing panel deformation in Wall 16×8.

3.3 STUD SIZE EFFECT

In the 2020's shear wall test program [3], shear walls with two rows of nails and 2 × 4 framing members were tested, in which end stud separation from bottom plates was not the main failure modes. In the following years' test program [4,5], shear walls with two and three rows of nails and 2 × 6 lumber were tested, and separation of end studs from bottom plates was the main failure modes observed due to the increased eccentricity of the shear wall as well as the increased load in the case of walls with three rows of nails, leading to a larger out-of-plane moment [5]. Thus, performance of shear walls with 2 × 4 and 2 × 6 stud sizes were compared in this parametric study. The detailed configurations are shown in Table 7.

Results showed that, with the strengthened end stud-to-plate connection, there was no separation between stud and plates observed in 2 × 4 and 2 × 6 shear wall models. These two types of shear walls have similar performance (Fig. 13), except that shear walls with 2 × 6 studs have slightly higher lateral load resistance, stiffness and ultimate displacement than shear walls with 2 × 4 studs (Table 8). Since the use of 2 × 6 lumber can also increase the compressive resistance of end studs and bearing resistance of top and bottom plates, 2 × 6 or even larger cross-section lumber is preferred compared to 2 × 4, as long as the out-of-plane movement of end studs can be properly prevented by construction detailing in high-capacity shear walls.

3.4 SHEATHING ARRANGEMENT EFFECT

To study the effect of sheathing panel arrangements, shear walls with both staggered and non-staggered horizontal sheathings were compared to the reference shear walls with vertical sheathing panels. Shear wall with large panels (2.4 m × 2.4 m) was also modelled in consideration of the effect of panel size. The configurations are shown in Fig. 14 and Table 9. Similar configurations were tested by researchers for standard shear walls with one row of nails on sheathing edges. For example, shear walls with small panels and a single large panel were tested by Durham et al. [14]; horizontal sheathing and large panel shear walls were tested by Lam et al. [15]; and staggered and non-staggered horizontally sheathed walls were tested by Long et. al [16].

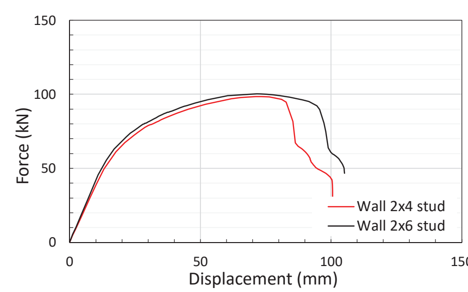

Figure 13. Load-displacement curves of 2×4 stud wall and 2×6 stud wall.

Table 7: Parametric analysis matrix of stud size

Parameter	Wall size (ft × ft)	Panel size (ft × ft)	Sheathing Thickness (in)	Row of nails	Nail size	Nail spacing (in)	Sheathing orientation	Stud size	Aspect ratio	Hold down
Stud size	8 × 8	4 × 8	7/16	2	8d	3	Vertical	2 × 4	1	Tie rod
	8 × 8	4 × 8	7/16	2	8d	3	Vertical	2 × 6	1	Tie rod

Table 8: Mechanical properties of different stud size models

Wall #	K_e (kN/mm)	Δ_{yield} (mm)	P_{yield} (kN)	Δ_{peak} (mm)	P_{peak} (kN)	Δ_u (mm)	μ (Δ_u/Δ_{yield})
2 × 4	3.9	43.8	89.0	71.1	98.4	85.6	1.96
2 × 6	4.2	44.0	91.8	71.7	100.2	97.4	2.21

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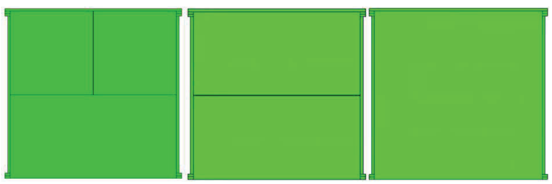
Table 9: Parametric analysis matrix of sheathing arrangement

Parameter	**Wall size (ft × ft)**	**Panel size (ft × ft)**	**Sheathing Thickness (in)**	**Row of nails**	**Nail size**	**Nail spacing (in)**	**Sheathing arrangement**	**Stud size**	**Aspect ratio**	**Hold down**
Sheathing arrangement	8 × 8	4 × 8	19/32	3	10d	3	Vertical	2 × 6	1	Tie rod
	8 × 8	4 × 4	19/32	3	10d	3	Horizontal staggered	2 × 6	1	Tie rod
	8 × 8	4 × 8	19/32	3	10d	3	Horizontal non-staggered	2 × 6	1	Tie rod
	8 × 8	8 × 8	19/32	3	10d	3	Large panel	2 × 6	1	Tie rod



(a) (b) (c)

Figure 14. Sheathing arrangements: (a) Horizontally staggered, (b) Horizontally non-staggered, (c) Large panel size.

Modelling results showed that, shear walls with large sheathing panels achieved the largest stiffness and lateral load resistance (Fig. 15). Although walls with large panels had the smallest ultimate displacement, the ductility ratio is larger than the other walls due to the much smaller yield displacement (Table 10). In the studies by Durham and Lam [14,15], it was found that oversized panel lead to substantial increase in load carrying capacity and stiffness, while the ductility factors did not vary significantly for different sheathing sizes in a standard shear wall configuration [14,15]. The performance of the reference wall model with vertical sheathing is better than horizontally sheathed walls in terms of ultimate displacement and strength (Table 10).

For horizontally sheathed walls, the non-staggered wall achieved higher stiffness, lateral resistance and ductility, but smaller ultimate displacement compared to staggered sheathing walls (Table 10). Similar findings were reported for standard shear wall configurations by Long et. al [16]. The performance of horizontally staggered wall might be affected by the size of the panels in the models, as essentially two smaller sheathing panels (1.2 m × 1.2 m) were used in the model compared to the non-staggered wall. Larger deformation was observed in the horizontally staggered shear wall model with smaller sheathing panels compared to horizontally non-staggered wall (Fig. 16). This indicates that the size of the sheathing panels is an important factor in shear wall deformability.

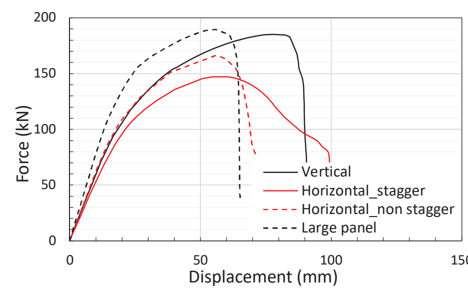


Figure 15. Load-displacement curves of walls with different sheating arragnements .

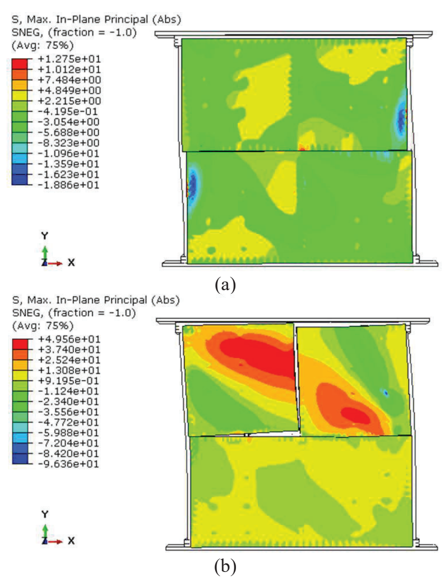


Figure 16. Sheathing deformation of horizontally sheathed wall: (a) Non-staggered, (b) Staggered.

Table 10: Mechanical properties of different sheathing arrangement models

Wall #	**K_e (kN/mm)**	**Δ_{yield} (mm)**	**P_{yield} (kN)**	**Δ_{peak} (mm)**	**P_{peak} (kN)**	**Δ_u (mm)**	**μ (Δ_u/Δ_{yield})**
Vertical	5.9	48.2	165.2	77.8	185.3	88.7	1.84
Horizontal staggered	5.3	38.8	133.8	54.6	147.2	79.5	2.05
Horizontal non-staggered	6.2	26.3	126.9	55.8	166.2	66.7	2.53
Large panel	7.8	22.8	145.2	55.8	189.7	64.3	2.83

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4 – CONCLUSIONS

Based on the verified 3D numerical models developed by the authors in previous study for high-capacity shear walls, parametric studies have been carried out in this study regarding different shear wall height, length, stud size, and sheathing arrangement. High-capacity shear walls with two and three rows of nails on sheathing edges were modelled with different wall configurations and compared to the reference high-capacity shear wall. Findings from the parametric studies are as follows:

- Increase in shear wall height from 8 ft to 9 ft and 10 ft will slightly decrease the stiffness of the walls but the effect to wall resistance can be ignored. However, the ultimate displacement increases with the increase of wall height.

- The lateral load resistance of high-capacity shear walls is proportional to wall length, while wall stiffness increases larger than proportional with the increase of wall length. The ultimate displacement, however, decreases as wall length increases, which indicates that higher aspect ratio of the shear wall leads to higher ultimate displacement.

- The use of different stud sizes in shear walls has no significant effect on the shear wall performance once the stud-to-plate connections are properly strengthened. Slight increase in peak load, stiffness, ultimate displacement can be found in shear walls with 2 × 6 lumber compared to walls with 2 × 4 lumber. It is suggested that 2 × 6 or larger dimension lumber be used for high-capacity shear walls.

- Shear wall with large sheathing panel of 2.4 m × 2.4 m has the largest stiffness and lateral load resistance compared to vertically and horizontally standard-sheathed walls, while it has the lowest ultimate displacement. The horizontally sheathed walls have lower resistance and ultimate displacement compared to vertically sheathed walls. The horizontally non-staggered shear wall performed better than the staggered wall, which indicates panel size is an important factor in shear wall performance.

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