Numerical Modeling and Finite Element Analysis of Steel Sheathed Cold-Formed Steel Shear Walls

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SUMMARY:
Shear wall panel (SWP), as one of the primary lateral load resisting components, has been extensively used in lightweight framing of low and mid-rise residential constructions. In this paper, the shear resistance of cold formed steel stud walls with steel sheathing under monotonic loading has been investigated by finite element analysis. The numerical modelling of shear wall has been conducted by ABAQUS software. In the finite element model of cold-formed steel stud wall, geometric large deformation and material nonlinearity has been considered. The results obtained from FEM have been verified against the others' experimental results. Using finite element analysis, parametric study is carried out with height-width ratio of wall, stud and sheathing thickness and screw spacing to analyze the shear carrying capacity of the wall. The numerical results have shown the good seismic performance of cold formed steel stud walls with steel sheathing.

Keywords: shear resistance, cold formed steel, shear wall, steel sheathing, finite element analysis

1. INTRODUCTION

Cold-formed steel (CFS) framed shear wall is a practical lateral force resisting system in buildings (Cheng, 2010). In general, CFS wall panels consist of CFS studs (lipped channel section), top and bottom tracks (plain channel section) and blockings covered by boards on interior and exterior faces. Gypsum, plywood, profiled metal sheets, steel sheets, sandwich panels and oriental strand boards are used as face sheathings. The concept of using cold-formed steel sheathing, however, is relatively new (Balh, 2010). The bottom tracks of the wall panels are attached to the ground supported slab by anchor bolts.

Studies on CFS wall panels sheathed with gypsum or wood based boards have been carried out by many researchers such as Fulop and Dubina (2004, 2006), Branston et al. (2006), Serrette (1997), Fiorino et al. (2011,2012), Xuhong et al. (2006).

With respect to steel sheathed shear walls, tests have only been carried out in the US by Serrette (1997), Yu et al. (2010, 2011) and Ellis (2007). The tests performed by Serrette (1997), at the Santa Clara University were limited to 2:1, 1220x2440mm , and 4:1, 610x2440mm, shear walls using 0.84mm CFS framing with nominal sheathing thicknesses of 0.46mm and 0.68mm . Yu et al. (2010), at the University of North Texas, expanded the test program for steel sheathed shear walls by including specimens constructed with 0.76mm and 0.84mm nominally thick sheathing. Balh (2010) Carried out tests on single-storey cold-formed steel frame/steel sheathed shear walls constructed from various framing and sheathing thicknesses to develop a Canadian design method for steel sheathed shear walls.

The aim of this paper is to study the shear resistance of steel sheathed CFS shear walls and some factors influencing shear resistance are also analyzed.

2. FINITE ELEMENT MODELLING

Different approaches are available to estimate the lateral response of sheathed CFS shear walls: experimental, analytical and numerical methodologies. The experimental approach is based on full scale tests carried out on typical walls and it is frequently used. In fact, nominal shear strength design values provided by building codes (AISI, 2009) in tabulated form are based on experimental test results. Due to the required large number of test, it is clear that this approach is the most expensive one and, in addition, it can be used only when the wall characteristics (geometry and materials) are within the range of experimental results. In order to overcome the limitations of the experimental approach, Finite element methods can be used to evaluate the shear response of sheathed CFS shear walls. Numerical models are usually calibrated on available experimental results and they can be used to simulate the structural response of walls having characteristics different from tested walls (Fiorino et al, 2012).

In order to employ proper FE models to analyze and study the performance of the CFS shear walls, the first step is to model CFS shear wall considering geometric and material's nonlinearity. The commercially available software package ABAQUS/ Standard, version 6.9-2, was used to develop the FE models.

The 4-node shell element with reduced integration, type S4R, was selected from the ABAQUS element library. This element uses three translation and three rotational degrees of freedom at each node. The element accounts for finite membrane strains and arbitrarily large rotations. Therefore, it is suitable for large- strain analyses and geometrically nonlinear problems. The screw connections were modelled by point based fasteners (Abaqus user's manual).

In a nonlinear analysis, ABAQUS requires the input of the material stress-strain curves in the form of true stress versus true plastic strain. The true stress (σ_{true}) and true strain (ϵ_{true}) were converted from the engineering stresses (σ) and engineering strains (ϵ) as equation (2.1) (Phama & Hancockb, 2010):

$$
\begin{aligned}
\sigma_{true} &= \sigma(1+\epsilon) \\
\epsilon_{true} &= \ln(1+\varepsilon) - \frac{\sigma_{true}}{E}
\end{aligned}
\tag{2.1}
$$

The displacements along the X, Y and Z-directions and rotations along Y and Z-directions of bottom track were restrained and the top track was assumed to have no displacement and rotation along the Y and Z-directions. The loading process was controlled by displacement load and the lateral displacement was applied on the top track nodes.

2.1. Verification of the finite element model

Experimental results related to steel sheathed CFS shear walls tested by Nisreen Balh (2010) were used to validate the accuracy of the numerical model. The characteristics of the models tested by Nisreen Balh are given in the following.

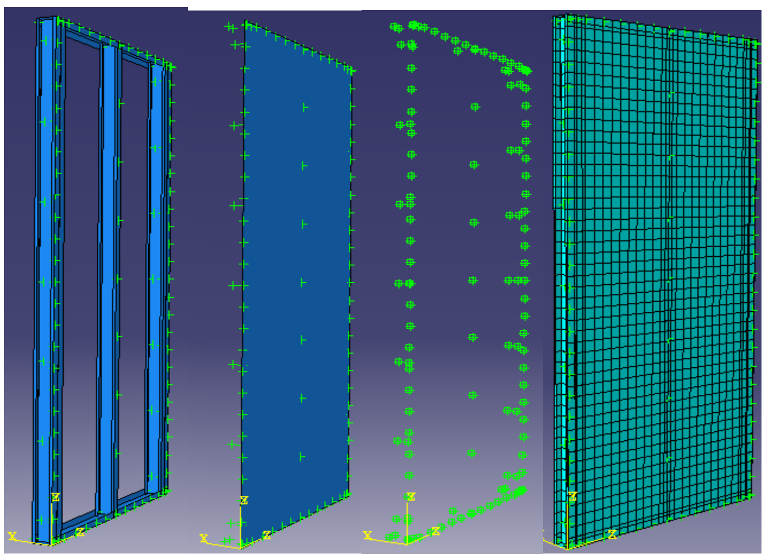
Nominal dimensions of the steel studs were 92.1mm web and 41.3mm flange and 12.7mm lip. Nominal dimensions of the steel tracks were 92.1mm web and 31.8mm flange. Except for 610mm long walls, a field stud was placed at a spacing of 610mm on-centre in the 1220mm long walls. The sheathing was then placed on the frame, marked, and installed with No.8 gauge 19.1mm pan head screws according to the fastener schedule in table 2.1. The sheathing was fastened around the perimeter of the wall specimen along the tracks and the chord studs at an edge distance of 9.5mm and along the field stud, if available. The section structural material properties are shown in table 2.2. Fig. 2.1 shows the finite element modelling of CFS shear walls (Balh, 2010).

Table 2.1. Test matrix (Balh, 2010)

configuration	sheathing thickness (mm)	wall length (mm)	wall height (mm)	fastener spacing (mm)	framing thickness (mm)
5	0.76	1220	2440	100/300	1.09
6	0.76	1220	2440	50/300	1.09
8	0.76	610	2440	100	1.09
9	0.76	610	2440	50	1.09

Table 2.2. Material properties (Balh, 2010)

specimen (mm)	member	base metal thickness (mm)	yield stress, Fy (Mpa)	Tensile stress, Fu (Mpa)
0.76	sheathing	0.76	284	373
1.09	stud/track	1.14	346	496



(a) (b) (c) (d)

Figure 2.1. Finite element model. (a) frame. (b) sheathing. (c) screw. (d) mesh.

Load-displacement curves of cold-formed steel stud walls have been shown in Fig. 2.2. Comparison of the numerical results together with the test results indicated that the Finite element analyses results were close to those of tests (as shown in Fig. 2.2).

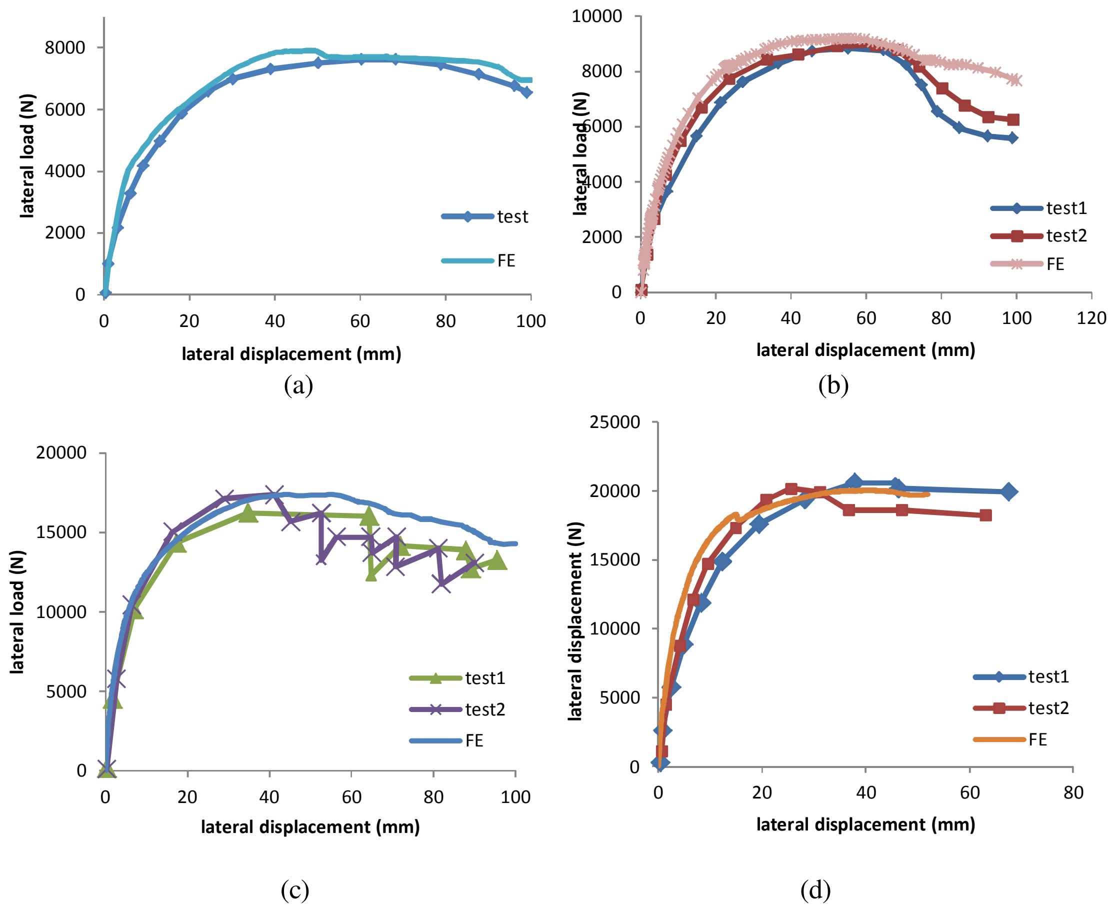


Figure 2.2. Comparison of results between test and finite element analyses. (a) Model 9.(b) Model8(c) Model 5. (d) Model 6.

3. PARAMETRIC STUDY

3.1. Material properties

Coupon tests were conducted to obtain the actual properties of the materials used in shear wall modelling. The testing procedure conformed to the ASTM A370 (2006) "Standard Test Methods and Definitions for Mechanical Testing of Steel Products". The stress-strain curve obtained from the coupon test is provided in Fig. 3.1. The yield stress $f_y = 347\ MPa$ was obtained by using the 0.2% nominal proof stress and the tensile stress is, $f_u = 400$.

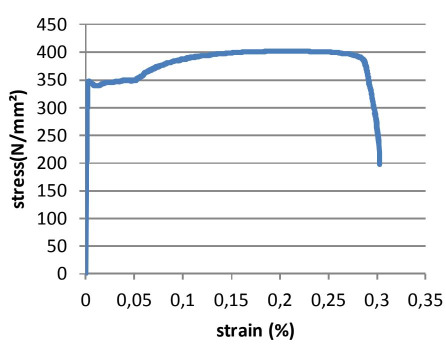


Figure 3.1. Stress-strain curve

The shear resistance of cold-formed steel stud walls is associated with many factors, such as materials of studs and sheathing, screw spacing, height-width ratio of wall, stud & screw spacing and so on. Finite element analysis verified by test is an effective method to study the shear resistance of cold-formed steel shear walls (Xuhong et al, 2006). After verification of FE models, a series of parametric analysis were done to study the effect of stud thickness, sheathing thickness, screw spacing, and wall height-width ratio on the shear resistance of walls. In the following the results of parametric study are given.

3.2. The influence of stud and sheathing thickness on the shear resistance of walls

CFS shear walls with height-width ratio of 4 (height 2440mm and width 610 mm) and height-width ratio of 2 (height 2440mm and width 1220 mm) and with stud thickness of 0.7mm, 1mm, 1.2mm and 1.5mm and sheathing thickness varied between 0.5mm to 2 mm were selected for investigating the effect of stud and sheathing thickness on the nominal shear capacity of steel sheathed CFS shear walls. Fig. 3.2 and Fig. 3.3 illustrate the load vs. lateral displacement curve for the CFS panels with height-width ratio of 4 and 2, respectively.

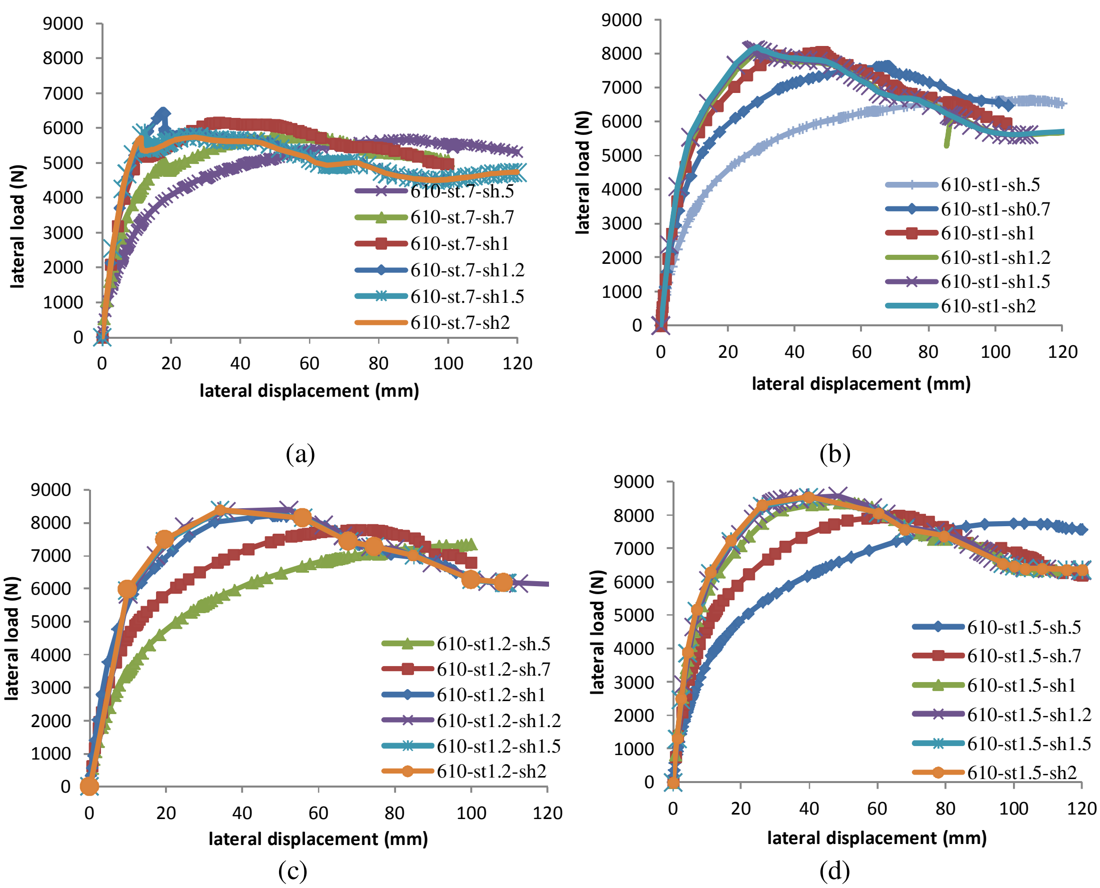
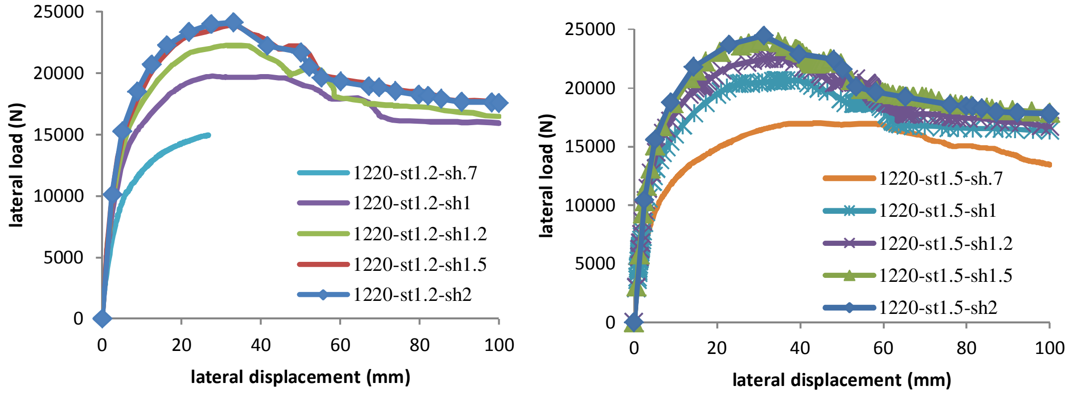


Figure 3.2. Load vs. Lateral displacement curve for the 610*1220 panels. (a) st=0.7. (b)st=1. (c) st=1.2 (d) st=1.5.



(a) (b)

Figure 3.3. Load vs. Lateral displacement curve for the 1220*2440 panels. (a) st=1.2. (b) st=1.5.

The nominal shear strengths are calculated as the peak load of load-displacement curve and are given in table 3.1. Fig 3.4 also illustrates the nominal shear strength per unit width of walls. It can be seen that increasing the sheathing thickness increases the nominal shear capacity of walls. In the case of wall height-width ratio of 4, up to the 1.2mm sheathing thickness the nominal shear strength increases linearly, but for sheathing thickness greater than 1.2mm the nominal shear capacity remains almost constant. There is no significant difference between nominal shear strength of the walls when the stud thickness increases from 1.2mm to 1.5mm.

Table 3.1. Results of analysis

Analysis label	Nominal shear strength (N/mm)	Drift ratio (%)
610-st1-sh1	13.18	1.98
610-st1-sh1.2	13.1	1.15
610-st1-sh1.5	13.31	1.16
610-st1-sh2	13.43	1.19
610-st1.2-sh1	13.45	1.96
610-st1.2-sh1.2	13.75	2.14
610-st1.2-sh1.5	13.74	1.41
610-st1.2-sh2	13.75	1.4
610-st1.5-sh1	13.71	2
610-st1.5-sh1.2	14.06	1.99
610-st1.5-sh1.5	14	1.63
610-st1.5-sh2	14.01	1.63
1220-st1-sh1	14.87	0.76
1220-st1-sh1.2	16.02	0.88
1220-st1-sh1.5	17.5	1.28
1220-st1-sh2	18	1.24
1220-st1.2-sh1	16.17	1.14
1220-st1.2-sh1.2	18.25	1.44
1220-st1.2-sh1.5	19.56	1.31
1220-st1.2-sh2	19.77	1.36
1220-st1.5-sh1	16.93	1.84
1220-st1.5-sh1.2	18.46	1.45
1220-st1.5-sh1.5	19.7	1.25
1220-st1.5-sh2	20.15	1.24

In the case of wall height-width ratio of 2, up to the 1.5mm sheathing thickness the nominal shear strength increase linearly with high slope, but for sheathing thickness greater than 1.5mm the increasing of nominal shear capacity is less. Although increasing the stud thickness from 1mm to 1.2 mm provides a significant increase in the nominal shear strength of the wall but there is not remarkable difference between nominal shear strength of the wall with 1.2mm and 1.5 mm stud thickness.

These results indicate that the thicker steel sheets did not significantly increase the shear resistance of CFS shear walls. This result has also been obtained by Yu with consideration to the experimental result (Yu & Chen, 2011).

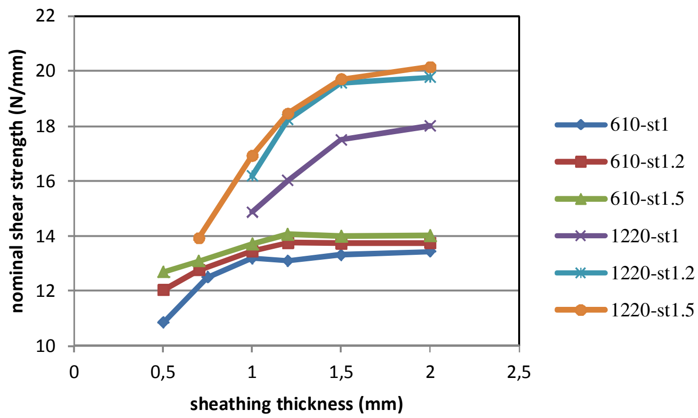
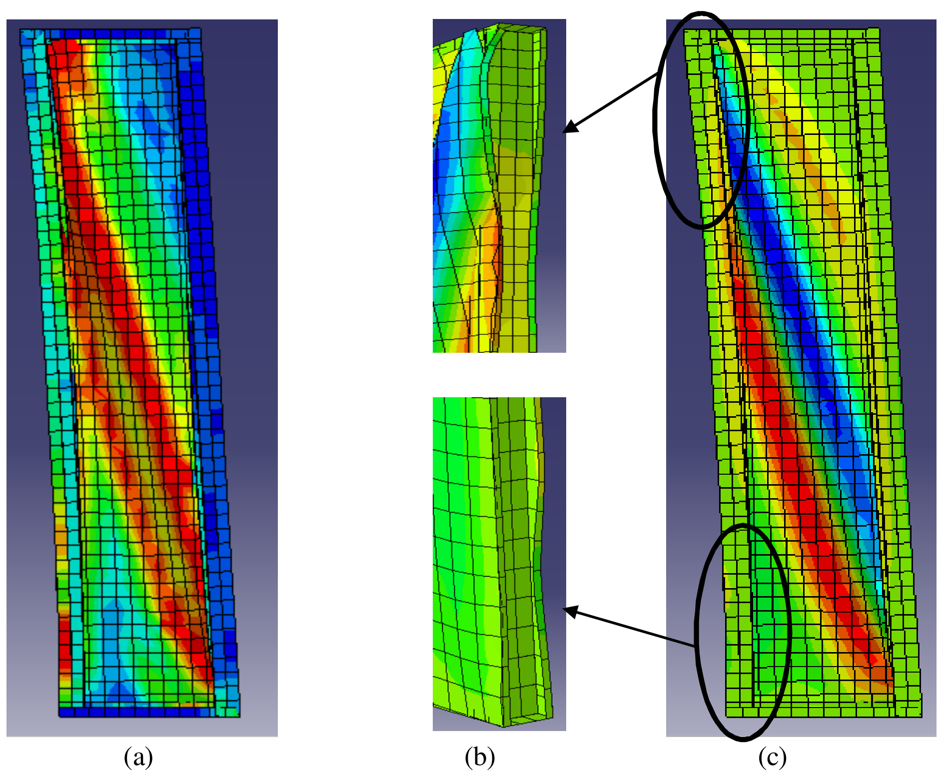


Figure 3.4. Comparison of nominal shear strength for walls with different height-width ratio and different thicknesses of stud and sheathing.

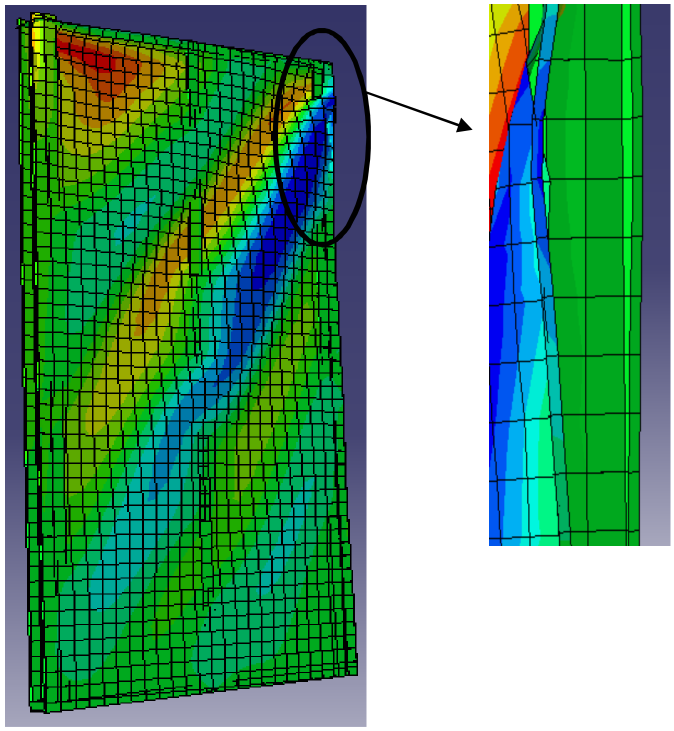
In all the tests, the in-plane shear force caused the buckling of the steel sheathing and large out-of-plane deformation of the sheathing. Fig. 3.5 shows the failure mode of a 610mm * 1220mm CFS wall with 1mm stud thickness and 0.5 mm sheathing thickness. It can be seen that diagonal buckling has occurred in the sheathing of wall. The buckling of the steel sheathing and large out-of-plane deformation of the sheathing were the primary failure modes for steel sheathed CFS shear walls. Distortional buckling of studs is also observed in this case.



(a) (b) (c)

Figure 3.5. Result of analysis for 610-st1-sh0.5 wall. (a) von-mises stress. (b) distortional buckling of stud. (c) out of plane displacement.

Fig. 3.6 shows the observed failure mode for a 1220mm*2440mm wall with 1mm stud and sheathing thickness. Steel sheet buckling and distortional buckling of boundary studs was the failure mode of this wall.



(a) (b)

Figure 3.6. Result of analysis for 1220-st1-sh1 wall. (a) out of plane displacement. (b) Flange buckling of stud.

3.3. The influence of screw spacing on the shear resistance of steel sheathed CFS shear wall

The strength and stiffness of the screw connections between CFS framing members and sheathings play the governing role in CFS wall panel behaviour and strength. Most of the non-linearity in the load–deformation behaviour of the CFS wall panel under in-plane shear is due to the non-linear response of the screw connection between the CFS members and the boards (Nithyadharan & Kalyanaraman ,2010).

In this section the effect of screw spacing on the nominal shear strength of CFS walls has been investigated. The characteristics of wall panel analyzed for investigating the effect of screw spacing is given in table 3.2. The analysis results of walls with different screw spacing as the lateral load-displacement curve are shown in Fig. 3.7. this figure illustrated that by reducing the screw spacing, the shear resistance of walls was increased. Figs. 3.8 present the plots of the nominal shear strength vs. the screw spacing at panel edges.

Table 3.2. Characteristics of Specimens

Panel	screw space(mm)	panel	screw space(mm)
	50		50
	100		100
610-st1.2-sh1.2	150	1220-st1.2-sh1.2	150
	200		200
	300		300

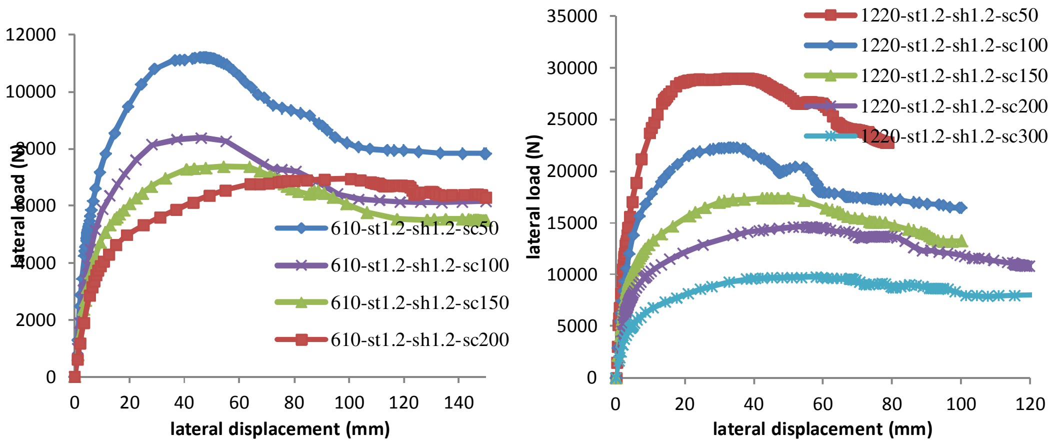


Figure 3.7. Lateral load-displacement curve of walls with different screw spacing

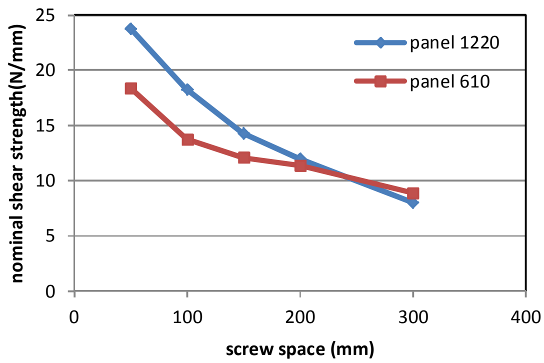


Figure 3.8. Nominal shear strength vs. the screw spacing at panel edges

4. CONCLUSIONS

In this research, nonlinear monotonic analysis on the CFS framed walls with single sided steel sheet sheathing was conducted by finite element modelling. Based on the finite element modelling, several parametric studies have been carried out for investigating the effect of some variables on the behaviour of CFS steel sheathed shear walls. The nominal shear strength of the walls for monotonic loads was established from the analysis results. It was shown that the buckling of the steel sheathing, flange distortion of the boundary studs and pullout of sheathing screws were the main failure modes of steel sheathed CFS shear walls. The analysis results also indicated that the use of thicker steel sheet and thicker stud would not improve the nominal shear strength of the shear walls effectively. It was also observed that decreasing the screw spacing at the perimeter of the walls increases the nominal shear resistance of CFS shear wall greatly.

REFERENCES

ABAQUS/standard version 6.9-2.

AISI S213-07/S1-09, North American Standard for Cold-Formed Steel Framing—Lateral Design 2007 Edition with Supplement No. 1, American Iron and Steel Institute (AISI), Washington, 2009.

ASTM A370. A370-06. (2006). Standard test methods and definitions for mechanical testing of steel products. American Society for Testing and Materials.

Balh, N. (2010). Development of Seismic Design Provisions for Steel Sheathed Shear Walls. Department of Civil Engineering and Applied Mechanics. McGill University, Canada. MsC thesis.

Branston, AE., Chen, CY., Boudreault, FA., Rogers, CA.(2006). Testing of light-gauge steel frame wood structural panel shear walls. *Canadian Journal of Civil Engineering*. **33**:561–572.

Ellis, J. (2007). Shear Resistance of Cold-Formed Steel Framed Shear Wall Assemblies using CUREE Test Protocol. Simpson Strong-Tie Co., Inc.

Fiorino, L., Iuorio, O., Landolfo, R.(2011). Sheathed cold-formed steel housing: a seismic design procedure. *Engineering Structures*.**34**:538–547.

Fiorino, L., et al.(2012). Performance-based design of sheathed CFS buildings in seismic area. *Thin-Walled Structures*. http://dx.doi.org/http://dx.doi.org/10.1016/j.tws.2012.03.022.

Fulop, LA., Dubina, D.(2004). Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading Part II: numerical modelling and performance analysis. *Thin Walled Structures*. **42**:339–49.

Fulop, LA., Dubina, D.(2006). Design criteria for seam and sheeting-to-framing connections of cold-formed steel shear panels. *Journal of Structural Engineering*. **132:4**,582–590.

Nithyadharan, M., Kalyanaraman, V. (2010). Experimental study of screw connections in CFS-calcium silicate board wall panels. *Thin Walled Structures*. doi:10.1016/j.tws.2011.01.004.

Phama, C., Hancockb, G. (2010).Numerical simulation of high strength cold-formed purlins in combined bending and shear. *Journal of Constructional Steel Research*. **66** :1205-1217.

Serrette, R. (1997). Additional Shear Wall Values for Light Weight Steel Framing. Research Report . 97 – 2, Revision 2007. American Iron and Steel Institute.

Xuhong, Zh., et al. (2006). Study on Shear Resistance of Cold-Formed Steel Stud Walls in Residential Structure. *Advances in Engineering Structures, Mechanics & Construction*. 423–435.

Yu, Ch. (2010). Shear resistance of cold-formed steel framed shear walls with 0.686 mm, 0.762 mm, and 0.838 mm steel sheet sheathing. *Engineering Structures*. **32**: 1522-1529.

Yu, Ch., Chen, Y. (2011). Detailing recommendations for 1.83 m wide cold-formed steel shear walls with steel sheathing. *Journal of Constructional Steel Research*. **67**: 93-101.