
Journal of Materials and Engineering Structures

Research Paper

Numerical evaluation of shear strength of CFS shear wall panels for different height-to-width ratios

Idriss Rouaz [a,b,*], ***Nouredine Bourahla*** [c], ***Smail Kechidi*** [d]

[a] *Geomaterials and Civil Engineering Laboratory, Department of Civil Engineering, University of Blida-1, BP270, Blida, Algeria.*

[b] *National Center of Studies and Integrated Research on Building Engineering (CNERIB), El-Mokrani, Souidania, Algéria*

[c] *Civil Engineering Department, National School Polytechnic ENP, Algiers, Algeria.*

[d] *Faculty of Engineering, University of Porto, Rua Dr. Roberto Frias s/n, 4200-465 Porto, Portugal*

A R T I C L E I N F O

Article history :

Received : 7 July 2018

Revised : 10 November 2018

Accepted : 18 November 2018

Keywords:

Cold formed Steel Shear Wall Panel

Shear strength

Finite Elementmodeling

Failure connection

A B S T R A C T

This paper presents a numerical evaluation of the shear strength of Cold Formed Steel Shear Wall Panels (CFS-SWPs) having 1.33:1 and 1:1 height-to-width aspect ratios with 0.76 mm steel plate sheathing thickness and 1:4, 1.33:1 and 1:1 height-to-width aspect ratios with 0.46 mm steel plate sheathing thickness, which are not provided by AISI S400. For this purpose, shell finite element (FE) models, validated with test results, are completed in ABAQUS v2018 with nonlinear geometry, material and connection. A good agreement is achieved between experimental and numerical results in terms of shear strength-lateral displacement and failure modes. It is concluded that, for a fixed height-to-width aspect ratio, the shear strength of SWPs having different screws spacing varying from 50.4 mm up to 152.4 can be assessed by interpolation using this FE method. However, by interloping the shear strength from 4:1 to 1:1 height-to-width aspect ratio, the shear strength can be underestimated; hence, it is more economical for practicing engineers to use the shear strength assessed by this proposed FE method for 1.33:1 and 1:1 height-to-width aspect ratios. Moreover, the effect of the sheathing thickness having 0.46 mm is evaluated and proposed as it lacks in data provided by the code (i.e., AISI S400).

1 Introduction

Recently, cold formed steel (CFS) shear wall panel (SWP) sheathed with steel plate sheets has gained its popularity in effectively resisting lateral loads (wind and earthquake) in highly seismic prone areas. Several experimental research programs have been carried out by researchers (Javaheri-Taftiet al. [1], Yu [2], Niari [3] and DaBreo el al.[4])on CFS-SWP with steel sheathing under lateral loads, with the aim of evaluating their shear strength for practicing engineers and designers, concluding that its dependency on the failure mode of the screw fasteners which governs the global behavior and strength of

* *Corresponding author. Tel.: +213 554215278.*
E-mail address: rouaz.idriss@gmail.com

e-ISSN: 2170-127X,

RESEARCH REVIEW of Sciences and Technologies

CFS-SWP. Similar conclusions have been drawn from experimental tests (AISI-2017 [5], Fiorino et al. [6], Peterman et al. [7], Ding [8] and Mohebbi et al. [9]).

Based on experiment tests done by Balh [10], AISI S400-15 code [11] provides nominal shear strength of 4:1 and 2:1 height-to-width ratios CFS-SWP having 0.76 mm sheathing thickness, and 2:1 height-to-width ratio CFS-SWP having 0.46 mm sheathing thickness. However, architectural and functionality design purposes impose different height-to-width ratios of shear wall panels less than 2:1 (longer than 1220 mm SWP width).Yet, the tabulated values of the shear strength provided by AISI code [11] do not cover the full range of CFS-SWP configurations, especially 1.33:1 and 1:1 height-to-width ratios for SWP having 0.76 mm sheathing thickness, and 4:1, 1.33:1 and 1:1 height-to-width ratios for SWP with 0.46 mm sheathing thickness. Furthermore, due to the important cost of experimental tests and time consuming, some of the shear strength with screw spacing are not provided, such as 50.8 mm screws spacing for SWP having 2:1 height-to-width ratio with 0.76 mm sheathing thickness. Therefore, the evaluation of the shear strength of the CFS-SWP is deemed necessary for these different height-to-width ratios. Hence, the numerical approach has become essential and use of finite element (FE) models is a good alternative in predicting the behavior and evaluating the shear strength of the SWP for various geometric and mechanical characteristics (Dai, X [12]).

Several FE models have been elaborated and various modeling techniques were presented. Nader et al. [13] have studied the shear strength of SWP sheathed with steel plate sheets, mentioning that members local buckling should be taken as an important mode of failure. In fact, bearing between the sheathing and the fasteners and tilting of the fasteners themselves is a desirable failure mode in the CFS-SWP, Kechidi et al. [14] have developed hysteresis models that take into account strength and stiffness degradation as well as pinching effect observed in the steel- and wood-sheathed CFS-SWPs . Rouaz et al. [15] have presented a comparison between two numerical approaches as to evaluate shear strength on the basis of connection shear failure. According to Borzooet al. [16], material nonlinearity should be introduced into the FE model in addition to the connection nonlinearity, which affects the global behavior of CFS-SWP. Furthermore, Farzampour et al. [17] suggested that the geometric nonlinearity should be introduced in the numerical simulation of the CFS-SWP lateral behavior, where its occurrence is susceptible in light gauge steel structural component. Vigh et al.[18] and Kalali et al. [19] have also presented a modeling technique of CFS-SWP sheathed but with corrugated steel sheathing.

As far as the evaluation of shear strength of CFS-SWP with different height-to-width ratios is concerned as a crucial performance, Niari [20] studied numerically the shear strength of cold formed steel shear wall panel with steel plate having 4:1 and 2:1 height-to-width ratios. Based on buckling of sheathing and local backing of members, the numerical model was validated with corresponding experimental results done by Balh [10]. However, experimental tests [10] show that the shear strength and the nonlinear response of SWPs under horizontal load were governed by the shear screw connections between members, especially sheathing-to-framing connections. Though, the screw connection nonlinearity was not considered in the numerical modeling [20]. Adding that, mechanical characteristics in terms of yield stress and tensile stress introduced in finite elements model were similar to those studs and tracks members of CFS-SWPs tested, but they did not corresponding to the sheathing mechanical characteristics.

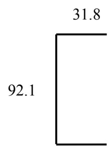
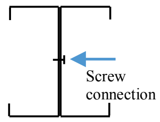
Taken into account the nonlinearity of the screws connections in the FE by take bake experimental test corresponding to those SPWs tested in addition to the material and geometrical nonlinearities, in this paper, a detailed FE model is proposed and validated using available experimental data. The shear strength with different screw spacing is evaluated for 1.33:1 and 1:1 height-to-width ratios CFS-SWP having 0.76 sheathing thickness and for 1:4, 1.33:1 and 1:1 height-to-width ratios CFS-SWP having 0.46 sheathing thickness. According to this FE modeling protocol, this evaluation which does not taken by the AISI S400-15 code and experimental tests is proposed for engineer's practices.

2 Selection of shear wall panel specimens

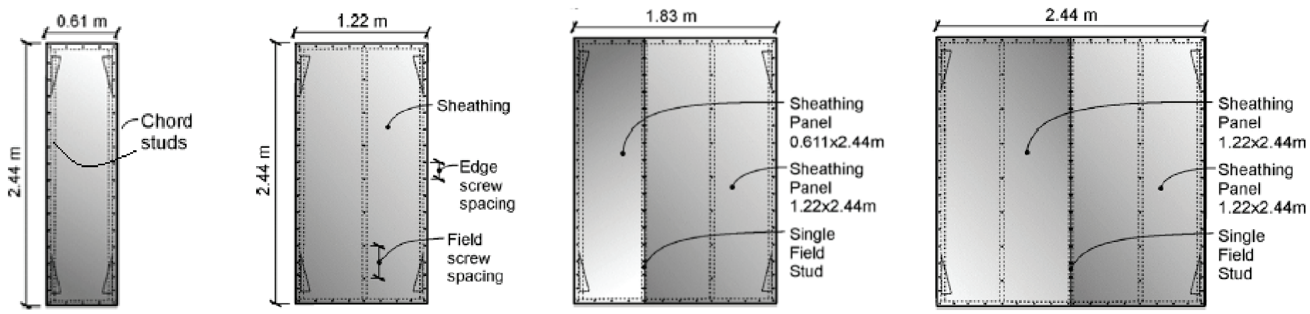
Four SWP specimens having height-to-width aspect ratio of 4:1, 2:1, 1.33:1 and 1:1 corresponding to, respectively, 610 mm, 1220 mm, 1830 mm and 2440 mm width have been selected based on the experimental study curried out by Balh [10] (see Fig 1). The framing members and sheathing sheets have 230 MPa steel grades. The geometrical characteristics properties of these members are presented in Table 1.

At each ends of the SWP, a built up back-to-back chord studs connected with two No. 10–16 x19.1 mm Hex washer head self-drilling screws at 305 mm on-center are installed in order to avoid the buckling failure modes. Moreover, the SWP is made of a single field studs at each 610 mm along the wall width. Simpson Strong-Tie S/HD10S hold-down devices are also attached to the inside web 75 mm from each end of the chord studs using No. 14 x 30 mm self-drilling hex washer head screws.

Table 1 - Geometrical characteristic members.

	Stud	Track	Back-to-back stud	Sheathing
Dimension profiles				Height-width 2440 mm x 610 mm 2440 mm x 1200 mm
Thickness	1.09 mm	1.09 mm	1.09 mm	0.76 mm or 0.46 mm

Steel sheathing sheets in two sizes (2440 mm x 610 mm and 2440 mm x 1220 mm) according to the SWP width (see Fig. 1) are attached to one face of the wall using No. 8 x 19 mm self-drilling – self-tapping pan head screw fasteners. Screws spacing of 50.8 mm, 101.6 mm and 152.4 mm are applied over the SWP perimeter the sheathing, and the spacing of screws along the field stud(s) is 300 mm.



(a) Width of 610 mm (b) Width of 1220 mm (c) Width of 1830 mm (d) Width of 2440 mm

Fig. 1 - Geometrical configurations of the selected SWPs. [10, 4]

3 Material characterization

In order to take into account the material and screw connection nonlinearities in the finite elements model, it is so important to identify these sources of nonlinearities. Therefore, the steel mechanical material is selected from normalized curve similar to those specimen members. However, sheathing-to framing and framing-to framing connection are identified by tests as below:

3.1 Strain-stress nonlinear curve of steel

Due to the lack of stress-strain nonlinear curve of steel sheathing and framing presented by Balh, a normalized stress-strain curve [21] (SAP2000, 2014) in accordance with ASTM A635 Grade 33 is selected for steel sheathing having 0.76 mm and 0.46 mm thickness as shown in Fig. 2.

In fact the tested yield and tensile stresses of the framing members (stud and track) were similar to the Grade 50 of cold formed steel [10], even though the test material had Grade 33, the Grade 50 is selected for 1.09 mm thickness of stud and track in the numerical model (Table 2), having (F_y = 344 MPa, F_u = 448MPa). The elastic modulus E_s is assumed to be $2.1*10^5$ MPa,. Density ρ is 7850 Kg/m^3.

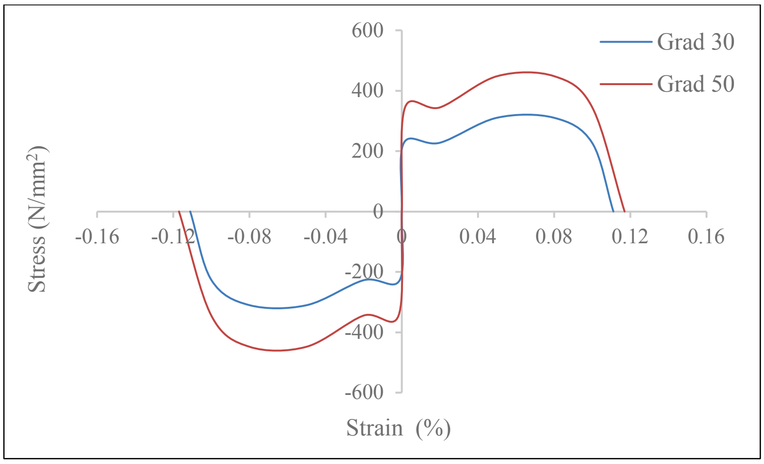


Fig. 2 - Stress-strain curve. [21]

Table 2 – Material Properties

Member	Thickness (mm)	Tests results [10]		Selected (nominal) properties for FE model	
		Yield stress F_y (MPa)	Tensile stress F_u (MPa)	Yield stress F_y (MPa)	Tensile stress F_u (MPa)
Sheathing	0.76	284	373	227	310
Sheathing	0.46	300	395	227	310
Stud/Track	1.09	346	496	344	448

3.2 Screw connection

Since an appropriately designed SWP dissipates energy by taking advantage of the inelastic behavior that develops in the connection zone between the frame and sheathing, the shear behavior of screw fasteners should be characterized. Therefore, a series of experiments tests were undertaken in CNERIB laboratory (National Center of Studies and Integrated Research on Building Engineering) in order to have the nonlinear force–displacement curves. These shear tests were done with the same conditions of thickness members, tensile grade of steel framing and sheathing, and screw diameter of Balh's experimental [10].

The tests on the screw fasteners were carried out following the European Standards ECCS TC7 TWG 7.10 (2009) [22] where the dimensions of the test specimens (Fig. 3) were adopted according to clause 3.2. The minimum number of tests was in compliance with 3.1.5 clause of ECCS for single fastener test.

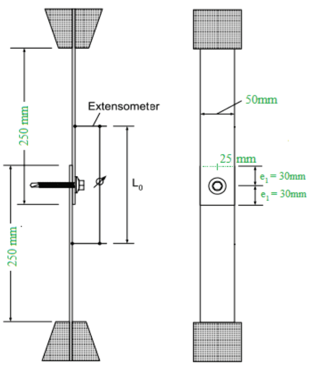
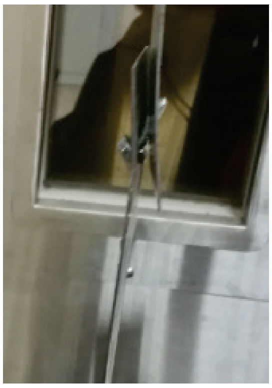


Fig. 3 - Dimension of test specimens ***Fig. 4 - Specimen under tension test.***

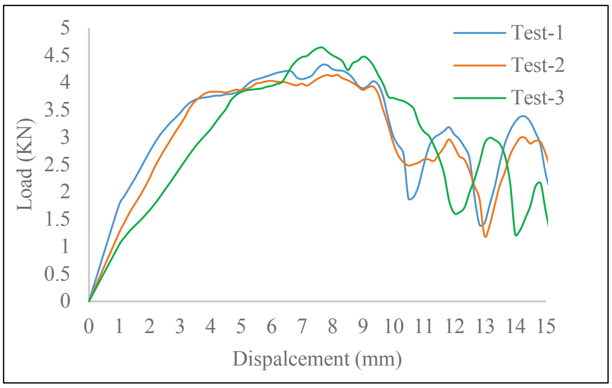
As shown in Fig. 4, the test specimens were placed in the tensile test - machine (MTS Criteriom Model 45) to determine the ultimate shear capacity and the non-linear behavior under a monotonic load of one screw fastener connecting two steel sheets.

3.2.1 *Sheathing-to-framing screws connection*

The two tests results of sheathing-to-farming screw fasteners under tensile load having 0.76 mm with 1.09 mm of thickness and 0.46 mm with 1.09 mm, respectively, confirm the tilting failure mode (Fig. 5-a and Fig. 6-a) as found and described in experimental test done by Bahl [10]. Force-displacement results are given in Fig. 5-b and 6-b. The average capacity of the test results is 4.17 kN and 2.29 kN for 0.76 mm and 0.46 mm sheathing thickness, respectively, where the difference is about 4% and 8.53%, respectively, compared to the test result obtained by Balh [10]. The large difference in specimen with 0.46 mm sheathing thickness is mainly due to the mechanical characteristics of steel plate sheathing which does not have the same properties of the corresponding tested SWP members (see table 2).

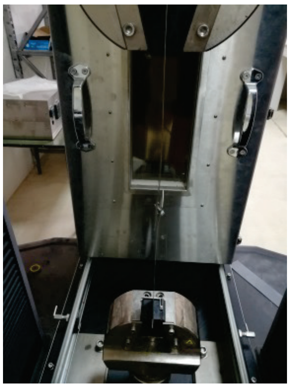


(a) Tilting of sheathing Screw

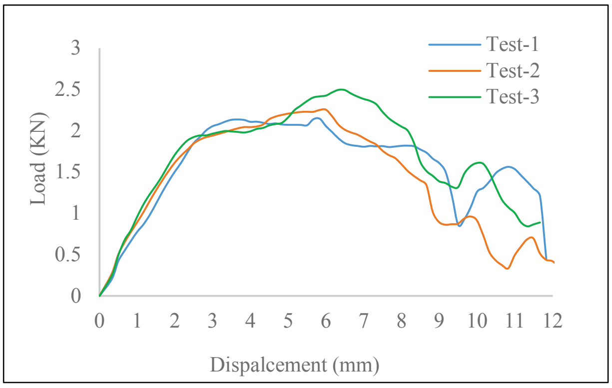


(b) Force-displacement results

Fig. 5 - Sheathing-to-Framing test connection results (0.76 mm-1.09 mm).



(a) Tilting of sheathing Screw



(b) Force-displacement results

Fig. 6 - Sheathing-to-Framing test connection results (0.46 mm-1.09 mm).

3.2.2 Framing-to-framing screws connection

The shear failure mode is more pronounced when the screws connections are driven through thick thickness of steel members (track –to-stud), it usually occurs at the corners of the wall. Fig. 7 shows the screw shear fracture failure mode, confirming the same mode of failure which has found in Balh test connections and the force–displacement curves results are given in Fig. 8.

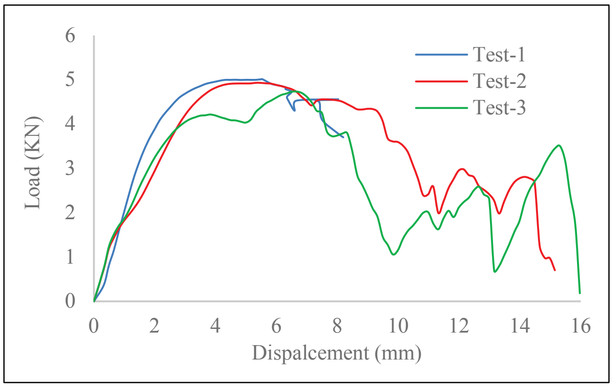


Fig. 7 - Screw shear fracture failure.

Fig. 8 - Framing-to-Framing test connection results.

Table 3 summarizes the test results of different configurations (thicknesses). A comparison of the results obtained during this experimental campaign and those from Balh [10] shows a slight difference.

Table 3 - Results of screws connection resistance.

Test	Sheathing thickness (mm)	Framing thickness (mm)	Maximum strength (kN)		Average strength (kN)	
			Balh results (kN)	Test result (kN)	Balh results (kN)	Test result (kN)
Sheathing–to-Framing connections (1.09 mm-0.76 mm)						
1	0.76	1.09	4.01	4.32	4.01	4.17
2			3.94	4.04		
3			4.10	4.26		
4			4.25	-		
5			3.73	-		
Sheathing–to-Framing connections (1.09 mm-0.46 mm)						
1	0.46	1.09	1.79	2.14	2.11	1.85
2			2.29	1.52		
3			1.86	1.89		
4			2.36	-		
5			2.25	-		
Framing –to-Framing connections (1.09 mm-1.09 mm)						
1	-	1.09	-	4.72	-	4.88
2			-	5.01		
3			-	4.92		

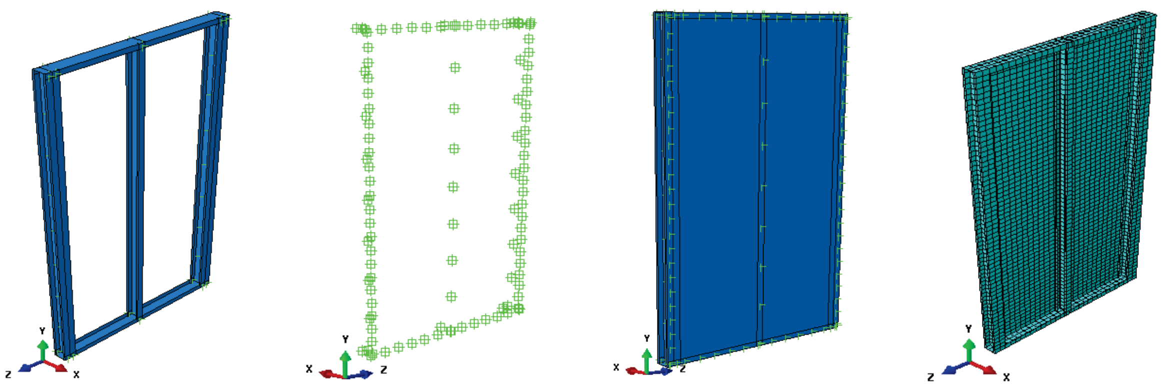
4 Finite Element Modeling

SWPs are composed of CFS C-shaped framing members (studs and tracks) attached to sheathing using screw fasteners. Modeling this structural system, for the sake of investigating its overall behavior and performance, taking into account the main failure modes such as failure of screw fasteners, and local or global buckling limit states of the framing members, needs a particular attention. In this section, a detailed description of the FE modeling protocol of the selected SWPs (see Section 2) using ABAQUS/ software [23] is presented.

4.1 Element type and mesh generation

Due to the large strain in the elements of the SWP under horizontal monotonic load, it was deemed important to take into account the geometric nonlinearity during the analysis. Hence, all elements, including sheathing, track, stud and lateral channel bracings have been modeled as S4R shell element with reduced integration scheme. Each element's node has three translational and three rotational degrees of freedom, which can be restrained according to the experimental boundary conditions.

Based on similar previous studies on CFS-SWP (Dai [24], Rouaz et al. [15], Borzooet al. [16]), in which, a sensitivity analysis has been performed on mesh size, a good compromise in terms of time and accuracy was achieved with a mesh dimension close to 50 x 50 mm for all framing members Fig. 9.



(a) Framing (b) Screws (c) SWP (d) Meshing

Fig. 9 - Finite Element modeling of CFS-SWP.

4.2 Mechanical properties of elements members

In order to introduce the material nonlinearities in the FE model, the engineering stresses (σ) and engineering strains (ε) obtained from normalized curves were processed to obtain the so-called true stress (σ_{tru}) and true plastic strain (ε_{tru}) adopting the following equations (Borzooet al. [16]):

$$\sigma_{tru} = \sigma_{nom}\left(1+\varepsilon_{nom}\right) \tag{1}$$

$$\varepsilon_{tru} = \ln\left(1+\varepsilon_{nom}\right) \tag{2}$$

$$\varepsilon_{pl} = \varepsilon_{tru} - \frac{\sigma_{tru}}{E} \tag{3}$$

4.3 Screw fastener modeling

In order to set a connection between two or more elements, Abaqus has a comprehensive set of elements to define the connection such as spot welds, rivets, screws, bolts and other types of mechanical fasteners. Furthermore, as shown in Fig. 10, the fixing elements may be located anywhere regardless of the mesh nodes, which are known as "mesh-independent fasteners". The physical characteristic of the connector such as the diameter are then introduced into the module "interaction" then defined as screws elements; taking also into account, spacing between screws and influence of the radius of screws.

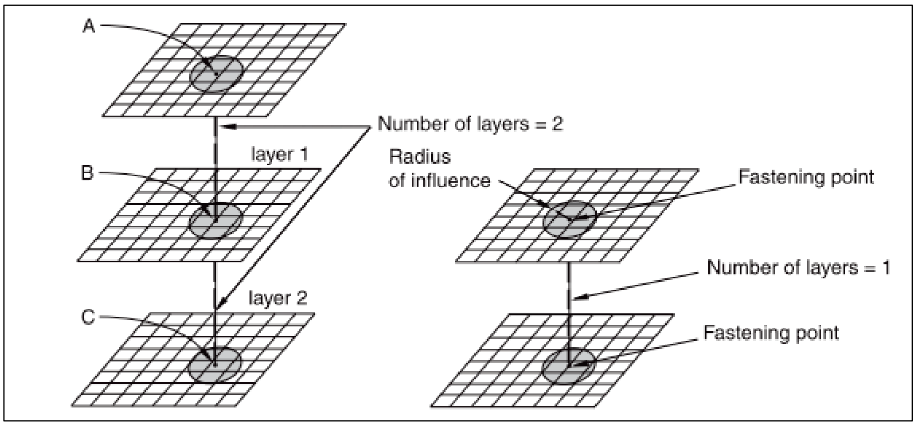


Fig. 10 - Mesh-independent fasteners ***(Abaqus Manual, 2010)***

The surface-to-surface contact was used to model the interaction relationship between the SWP elements, namely, between track flange and stud flange, assembly double studs, and sheathing to framing contact. Based on sensitivity analysis performed by Dai [24], the value of the friction coefficient does not affect the results. Therefore, a friction factor of 0.3 was adopted in this study [24].

4.3.1 Pull-out strength

The pull-out strength of the steel-to-steel screw fastener is estimated using the design provisions of the North American Specification for the Design of Cold-Formed Steel Structural Members AISI S100 (Section E.4.4.1) [25] through the following equation:

$$P_{not} = 0.85\left(t_c \times d\right).F_{u2} \tag{4}$$

Where :

d = nominal screw diameter,

t_c = lesser of the depth of penetration and thickness,

P_{not} = nominal pull-out strength per screw,

F_{u2} = tensile strength of member not in contact with screw head or washer.

4.3.2 Shear-displacement curve of the screw fastener

The nonlinear monotonic shear behavior of screw fasteners obtained from experimental results (§3.2.1) is considered as an envelope force-displacement response curve, as shown in Fig. 11 [23].

To simulate the damage evolution which initiates from point I representing the ultimate force and corresponding ultimate displacement to point D, each component is defined with ($\bar{u}_f^{pl}, F_i$) coordinates, where F_i is the damaging force, calculated as follows (Abaqus Manual, 2018):

$$F_i = (1 - d_i) F_{effi} \tag{5}$$

F_{effi}: effective force representing ultimate force, d_i damage index given by equation (6) :

$$d_i = \frac{1 - e^{-\alpha\left(\frac{\bar{u}^{pl} - \bar{u}_0^{pl}}{\bar{u}_f^{pl} - \bar{u}_0^{pl}}\right)}}{1 - e^{-\alpha}} \tag{6}$$

Where:

$\bar{u}_i^{pl}$: Equivalent plastic displacement damage at i integration step,

$\bar{u}_0^{pl}$: equivalent plastic displacement at damage initiation,

$\bar{u}_f^{pl}$: Equivalent plastic displacement at damage failure,

α : exponential coefficient based on calibration with experimental data curve.

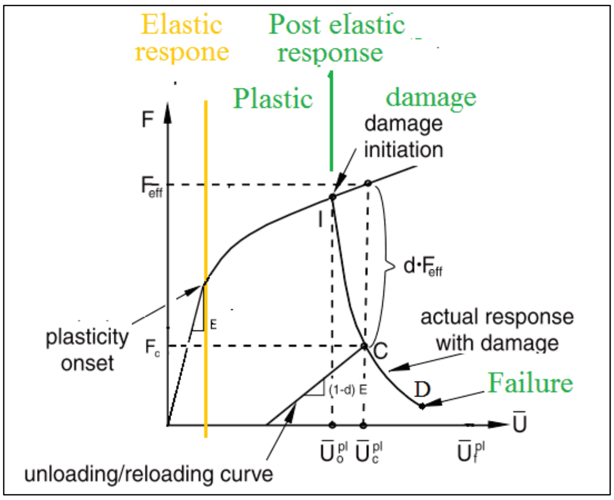


Fig. 11 - Modeling behavior connection (Abaqus) [23]

4.4 Boundary Condition and solution technique

In order to simulate the boundary conditions of the tested SWPs in the FE models, two Tie Multi Points Control “MPC” interaction were used. One was used to fasten the pinned bottom track to the ground in order to model the seven bolts placed in the vicinity of each stud, the other was adopted to allow the application of the displacement at the top track. The test rig which prevents any torsional movement of the wall or out of plane translation is modeled by restraining the translation DOF in the out of plan direction.

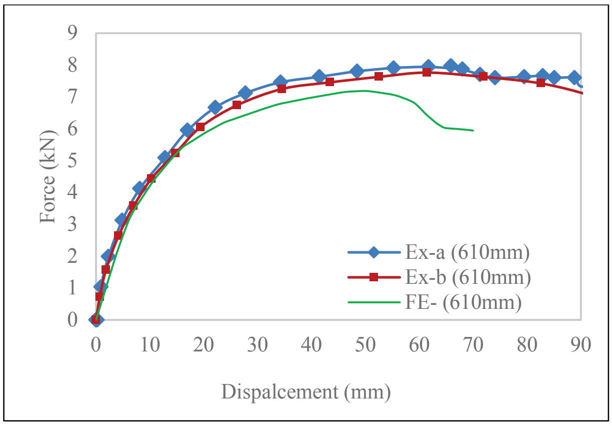
Static general and dynamic implicit for a quasi-static application were both used in order to reach convergence, with initial increment step size equal to 0.1 and minimum increment step size limited to 10e-8. To solve nonlinear equations in the analysis, full Newton–Raphson method was used.

5 Validation of the finite element model

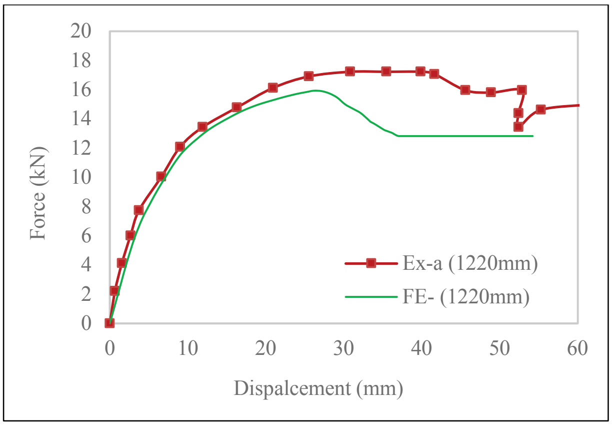
5.1 Shear strength-lateral displacement

In order to validate the proposed FE modeling protocol and evaluate the shear strength of SWP under a horizontal load, four SWPs having 4:1, 2:1, 1.33:1 and 1:1 height-to-width aspect ratio corresponding to, respectively, 610 mm, 1220 mm, 1830 mm and 2440 mm width, are selected from experimental tests [10]. The screws spacing of the steel plate sheathing is equal to 101.6 mm for all SWPs.

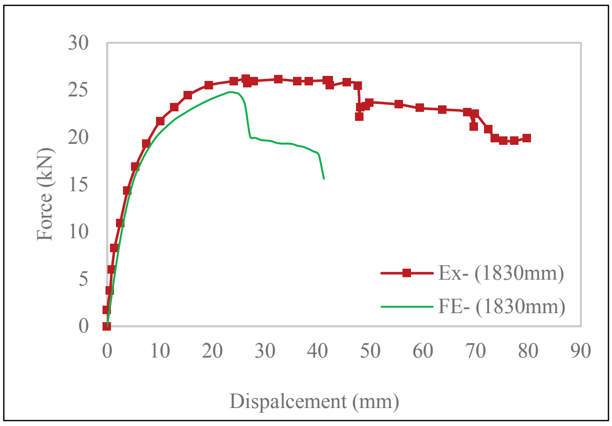
Fig 12 shows the shear strength-lateral displacement behavior of 610 mm, 1220 mm, and 1830 mm and 2440 mm width shear wall panels. A good agreement is achieved between the FE predicted responses and the measured ones for the four SWPs. Some differences in displacement at peak load and at the post peak are visible, particularly for the SWP having 1830 mm width. This is mainly due to the uncertainties’ accumulations that are caused by the assumptions related to the FE model, concerning the mechanical properties of the SWP steel members, and the hold-down stiffness connectors which were attached to the interior base of the stud in experimental test. Moreover, the boundary conditions of a small out-of-plan displacements occurred during the test at the top of the SWP were neglected in the FE model.



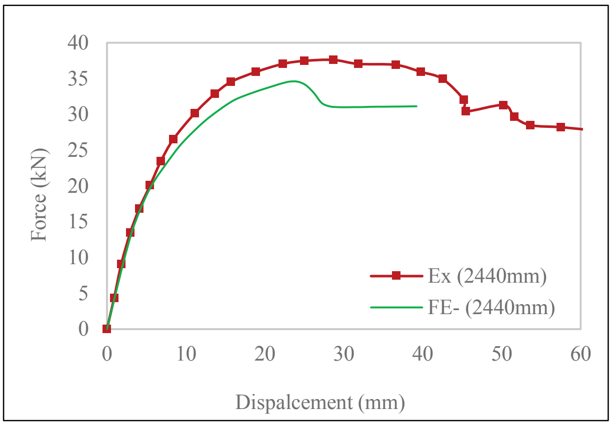
(a) 610 mm SWPwidth



(b) 1220 mm SWP width



(c) 1830 mm SWP width



(d) 2440 mm SWP width

Fig. 12 - Response of the SWP with different width.

Table 4 sumarizes the difference between numerical and experimental results in terms of shear strength and corresponding ultimate displacement for the four SWPs.

The highest difference of shear strength is reached in panels with height-to-width aspect ratio equal to 4 and 1, this difference is estimated around 7.12 % and 7.54 %, respectively, and it decreases in SWPs with an aspect ratio equal to 2:1 and 1.33:1; about 5.23 % and 5.17 %, respectively.

Furthermore, the ultimate corresponding displacement is more pronounced in the case of SWPs having height-to-width aspect ratio 4:1 and 1:1 than SWPs with height-to-width aspect ratio 2:1 and 1.33:1. This is mainly due to some simplistic modeling assumptions, which do not take into account the real mechanical material properties of the installed (hold-down) to fix the SWP against overturning moment.

Table 4 - Comparison between numerical and experimental results.

Dimensions		Experimental results		Numerical Method		ΔF (%)	Δu (%)
w (mm)	Ratio h/w	Shear strength (kN)	Displacement (mm)	Shear strength (kN)	Displacement (mm)		
610	4 :1	7.72	59.01	7.17	50.17	7.12	14.98
1220	2:1	16.83	31.01	15.95	26.83	5.23	13.48
1830	1.33:1	26.11	26.18	24.76	23.17	5.17	11.50
2440	1:1	37.38	28.63	34.56	24.69	7.54	13.76

Despite the fact that this proposed FE modeling protocol has a small difference of results with those corresponding to test results, it gives an acceptable (reasonable) prediction of the shear strength of the CFS SWP with steel plate sheathing for the four different height-to-width aspect ratios.

5.2 Screw spacing

Even though the proposed method is able to predict the shear strength for the four height-to-width ratios with 101.1 mm screws spacing with reasonable error, it is more convenient to investigate also for the sensitivity of the effect of screws spacing on the shear strength of the SWP. Available experimental data are used for the shear wall panel having 2:1 height-to-width ratio (1220 mm of width).

As shown in table 5,it is obvious that the shear strength in both experimental and numerical results increases when the screws spacing becomes narrow (moving from 152.4 mm to 50.8 mm), this is due to the increasing of the number of screws which have a significant contribution in the global strength of the SWP. It is clearly mentioned that this FE modeling protocol captures the effect of screws spacing on the shear strength and corresponding ultimate displacements of shear wall panel having 1220 mm of width.

Table 5 - Screws spacing effect for 1220 mm SWP width.

Spacing	Experimental balh		Numerical Method		ΔF (%)	Δu (%)
	Shear strength (kN)	Displacement (mm)	Shear strength (kN)	Displacement (mm)		
50.8	20.37	33.6	19.39	30.24	4.81	10.02
101.6	16.83	31.01	15.95	26.83	5.23	13.48
152.4	13.42	36.01	12.76	32.4	4.92	10.02

The difference between the numerical and corresponding experimental results (Table 5) of shear strength and ultimate displacement with different screws spacing effect does not exceed 5.23 % and 13.48 % respectively. As well as this FE modeling protocol is able to capture the maximum difference of the shear strength of SWP with 101.6 mm screws spacing, it captures also the maximum displacement error in the same screws spacing. Furthermore, the minimum error in terms of shear strength and ultimate displacement is captured in the same screws spacing with 4.92 % and 10.02 %, respectively, this can be considered as a positive point for the reliability of this proposed method.

Fig 13 shows that changing the screws spacing from 50.8 mm up to 152.4 mm has a linear variation on the shear strength of the SWP having 2:1 height-to-width aspect ratios for both experimental and numerical results. Therefore, an intermediate shear strength having screws spacing in this range such as 75 mm or others spacing can be interpolated without experimental or numerical studies.

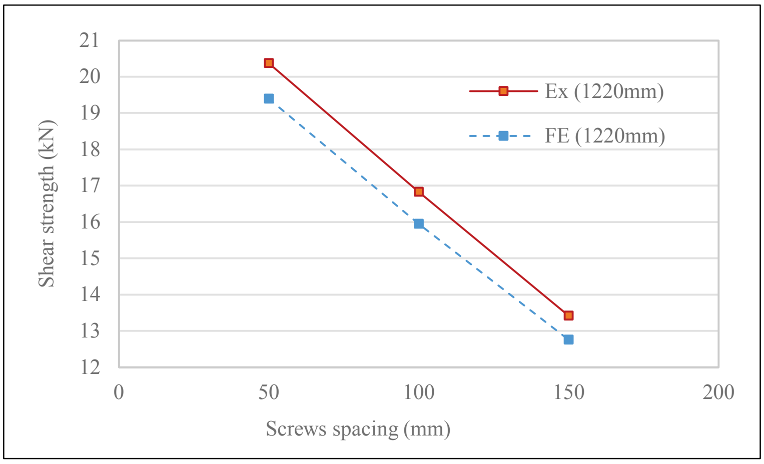


Fig. 13 - Variation effect of screws spacing 50.8 mm, 101.6 mm, 152.4 mm on shear strength

5.3 Sheathing thickness

The nonlinear response for the SWP having 2:1 height-to-width ratio and 50.8 mm screws spacing subjected to horizontal load is investigated and validated. As shown in Fig. 14**,** the comparison between the numerical and experimental results in term of nonlinearity response shows a good correlation in SWP having 0.76 mm sheathing thickness, but a slight difference in the SWP with 0.46 mm thickness of sheathing, especially in the elastic range. This might be due to the corresponding mechanical characteristics of screws tests results, which are not the same as the real mechanics characteristics. However, the global nonlinearity response is acceptable.

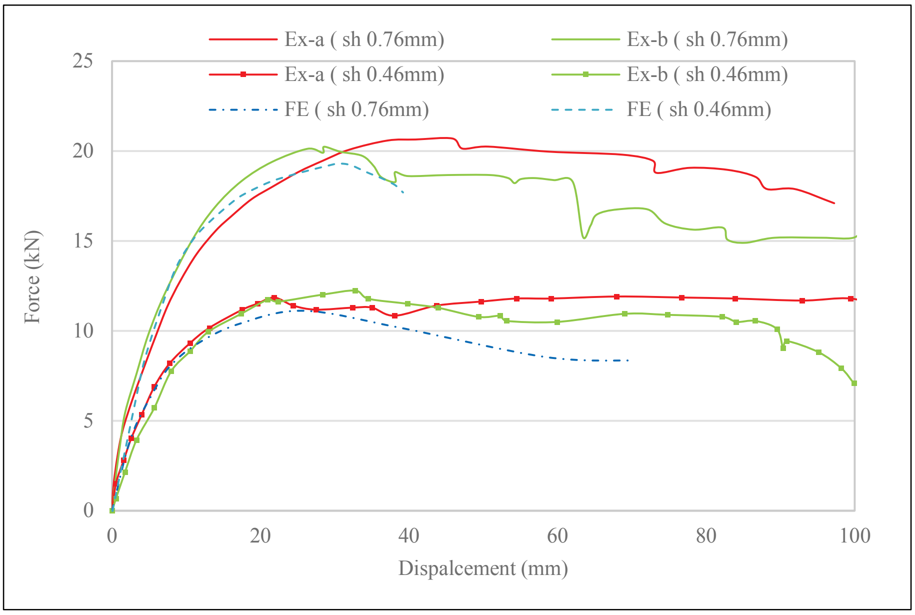


Fig. 14 - Response of the SWP with 2:1 height-to-width ratio

As for the shear strength of SWP with 0.46 mm sheathing thickness, it is evaluated as a 9.28 % as different between the numerical and experimental results, which is bigger than the difference in the SWP with 0.76 mm sheathing thickness (4.81 %). This might be due to the thin thickness of the sheathing involving an important contribution of others failure mode such as local and global buckling.

Moreover, it is noticed that the sheathing thickness has a significant effect on the shear strength of the SWP in the numerical and experimental results. Table 6 summarizes this difference of 2:1 height-to-width aspect ratio SWP having different screw spacing. For these three screws spacings, the shear strength decreases with a linear variation about 42.80 % (average difference) from SWP having 0.76mm steel sheathing thickness to SWP having 0.46 sheathing thickness. It was desirable if there were experimental results which would allow drawing a final conclusion about this linear variation of the resistance and, therefore, makes it possible to evaluate the resistance of the SWP by interpolation.

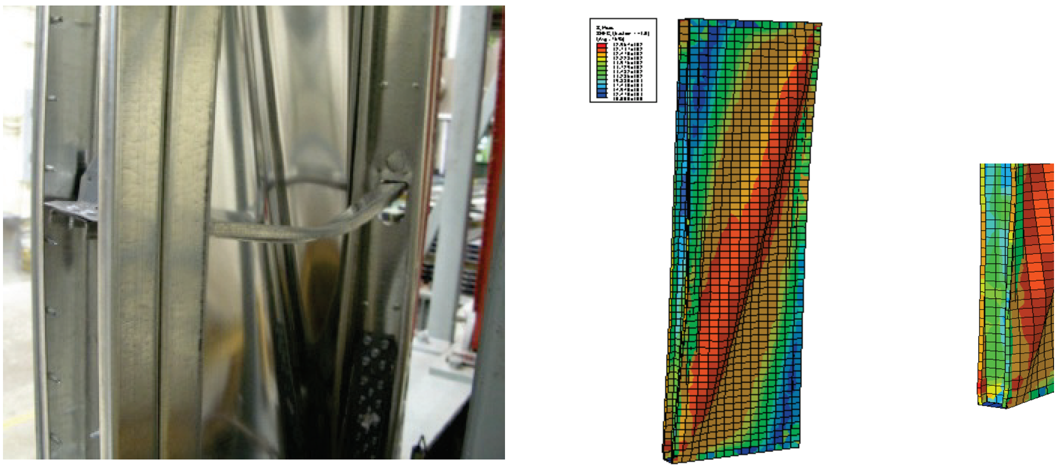
Table 6 - Shear strength of the FE model.

L (mm)	**Ratio h/w**	**Spacing (mm)**	**Shear strength (kN)** Thickness (0.76 mm)	Thickness (0.46 mm)	**ΔF %**
		50.8	19.39	11.11	42.70
1220	2:1	101.6	15.95	9.18	42.45
		152.4	12.76	7.24	43.26

5.4 Failure mode

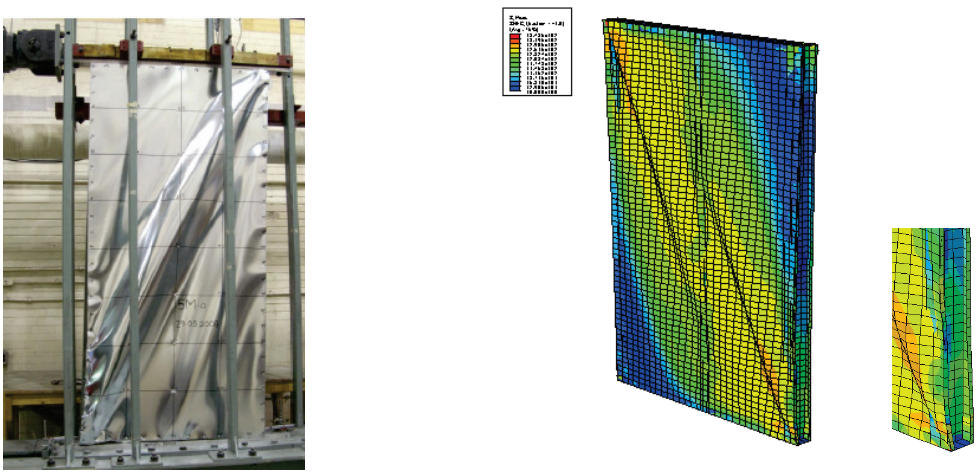
The failure of screws connections was the dominating failure mode and governed the global nonlinear behavior of the SWPs. During the experimental tests [10], it was noticed that a tension field action was developed leading to the buckling of the steel plate sheathing due to its thin thickness. Moreover, a local buckling of the end stud occurred at the corner of the SWP. Therefore, it is important to check and validate the ability of this proposed finite element model to take into account these additional failures mode in the simulation.

Figs. 15-18 show the buckling of the steel plate sheathing as a failure mode simulated by the FE model for all height-to-width aspect ratios of the SWPs under horizontal load. As shown in Fig. 15-b, one tension field is developed during the analysis in the steel sheathing as a buckling of the steel plate for height-to-width aspect ratio equal to 4:1 as well as in the experimental results. Moreover, the local buckling of the end stud is captured by the FE model (Fig. 16-c), which is due to the compression force developed in this thin member after loading. As for 2:1 height-to width aspect ratio SWP, two tension fields are developed in FE model as well as in the experimental outcomes (Fig. 16).



(a) Experimental result (b) FE: Buckling of sheathing (c) FE: Local buckling of stud

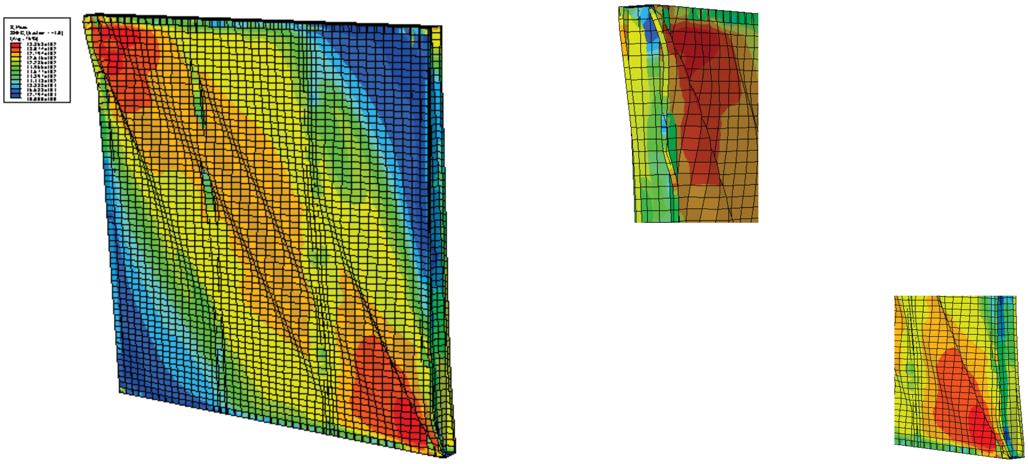
Fig. 15 - Failure mode of SWP having 610 mm of width



a) Experimental result b) FE : buckling sheathing and stud

Fig. 16 - Failure mode of SWP having 1200 mm of width

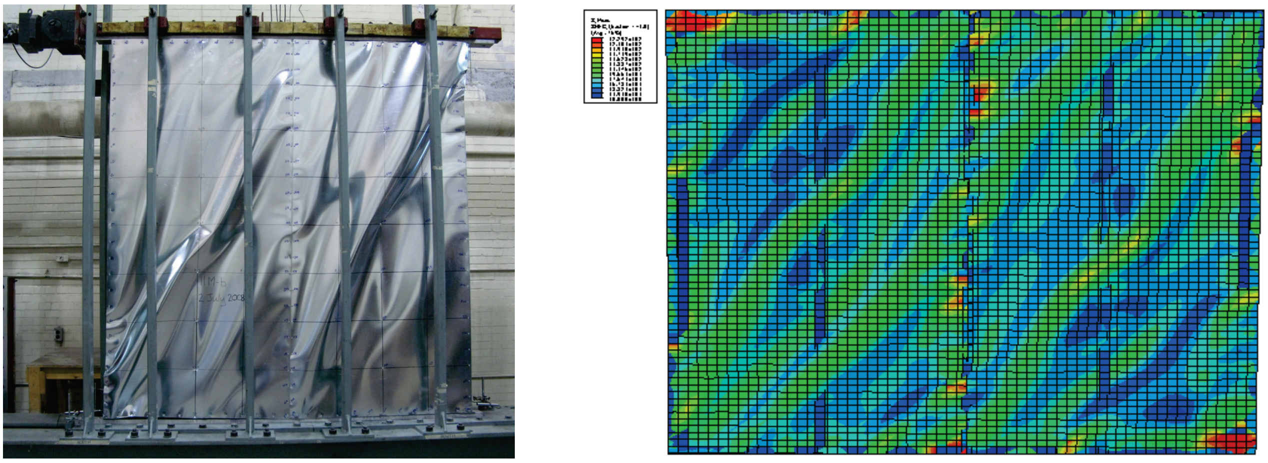
Fig. 17 shows a concentration of the stress at the corner of the sheathing for a SWP having 1.33:1 height-to width aspect ratios. However, it can be noticed a continuity of the tension field developed on the full width of SWP from 610 mm to 1220 mm sheathing width. This is a convenient point that makes these two steel sheathings as one sheathing of 1830 mm ensuring the transmission of the stresses. Additionally, a significant local buckling of the end stud is also developed at the corner of the panel (Fig. 17-b).



FE : Buckling sheathing and stud FE: Local buckling of the stud at the corner

Fig. 17- Failure mode of SWP having 1830 mm of width

As shown in Fig. 18, comparing the finite elements model with experimental results during horizontal loading, a good similarity is reached in terms of the development of tension field which involves buckling of the steel plate sheathing in 1:1 height-to-width aspect ratio SWP. Therefore, it can be concluded that this proposed FE modeling protocol takes in to account the other failure mode such as buckling of the sheathing and local buckling of the end stud in additional to shear failure of screw connections.



a) Experimental result b) FE elements simulation

Fig. 18 - Developing failure mode of SWP having 2440 mm width during loading.

6 Evaluation of SWP strength of different height-to-width ratios

According to the AISI S400-15, the nominal shear strength per unit width for seismic and other in-plane loads is proposed by Balh [35] experimental tests in order to develop the Canadian code. However, the effect of spacing screw on the strength of the SWP having 0.76 mm sheathing thickness was presented just for SWP having height-to-width ratio (h/w) greater than 2:1, but not exceeding 4:1. Adding that, the strength of SWP having 0.46 mm sheathing thickness is studied just for 2:1 height-to-width ratio. Due to the lack of experimental tests for others height-to-width ratio with different screws spacing connections and sheathing thickness, this investigation focus to evaluate and propose for designers and engineers practices the strength of 1.33:1 and 1:1 height-to-width ratios SPW with 0.76 mm sheathing thickness and 1:4, 1.33:1 and 1:1 height-to-width ratios SPW with 0.46 mm sheathing thickness. The screws spacing investigated here are 50.8 mm, 101.6 mm and 152.4 mm.

6.1 Effect of interpolation the shear strength from 610 mm width SWP

In order to check, through numerical simulations, whether the shear strength of different widths of SWP having 101.6 mm screws spacing can be assessed and predicted by interpolation from SWP having 610 mm of width, as depicted in Fig 19, increasing the SWP width from 610 mm up to 2440 mm has a linear variation effect on the shear strength of SWPs obtained by this FE modeling (FE). However, in experimental results (Ex), from 1830 mm to 2440 mm width, a slight nonlinear variation in shear strength is noticed. This is mainly due to the difference in the instrumentation, installation and boundaries conditions of the SWPs tested which are not the same in the different width, as opposed to the FE models.

However, The shear wall panels having 1220 mm ; 1830 mm and 2440 mm width are expected to have a shear strength about 2, 3, 4 times of 610 mm SWP width by interpolation, respectively. Since, the shear strength of 2440 mm SWP width is expected to have 30.88 kN and 28.68 kN for experimental (Ex expected) and numerical (FE expected) results, respectively; based on the shear strength of 610 mm SWP width. Thereby, the experimental and numerical results shows a difference about 21.8 % and 21.1 % from which is expected to have by interpolating, respectively, hence, the multiple factors for 2440 mm SWP width are 4.84 and 4.82 respectively.

This means that assessing (estimate) the shear strength by interpolation from 4:1 ratio underestimate the real shear strength of SWP having 1:1 height-to-width aspect ratio. This may bring more safety for design but may not be economical. Therefore, it is more rational to use the shear strength of the SWP having 2440 mm of width from both experimental and numerical results rather than estimating the shear strength by interpolation or using the multiple factor for SWPs.

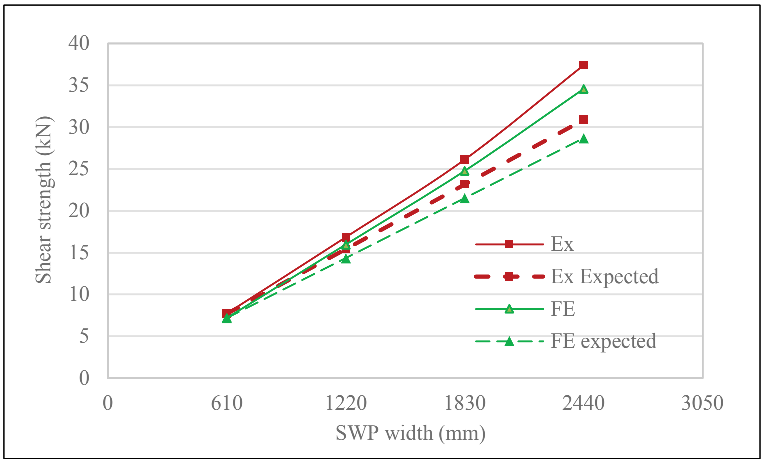


Fig.19 - SWPs shear strength for different height-to-width ratio of 101.6 mm screws spacing

6.2 Effect of screw spacing

Table 7 presents the results in terms of shear strength and corresponding displacements for 1:4, 1:1.33 and 1:1 height-to-width aspect ratio with 50.8 mm, 101.6 mm and 152.4 mm screws spacing. Some of experimental data to study the screws spacing effect on shear strength are extracted from experimental tests done by DaBreoel al.[4]. Comparing the numerical results with the experimental data of 50.8 mm screw spacing, the difference in terms of shear strength and corresponding displacement for 4:1 height-to-width ratio of SWP is about 2.13 % and 9.96 % respectively, and about 5.44 % and 29.10 % respectively for SWP with 1:1.33height-to-width aspectratio. It can noted that 1.33:1height-to-width aspectratio SWP has not a good prediction in ultimate displacement with 50.8 mm spacing screws, but it stays acceptable for 4:1height-to-width aspectratio SWP. Concerning the effect of screws spacing on the shear strength, the difference is acceptable.

Table 7 -Screws spacing effect for others height-to-width ratios shear wall panel.

Ratio h/w	Spacing (mm)	Experimental balh: Shear strength (kN)	Experimental balh: Displacement (mm)	Numerical Method: Shear strength (kN)	Numerical Method: Displacement (mm)
	50.8	8.94	53.5	8.75	48.17
4:1	101.6	7.72	59.01	7.17	50.17
	152.5	-	-	5.91	50.17
	50.8	33.85	35.5	32.01	25.17
1.33:1	101.6	26.11	26.18	24.76	23.17
	152.5			18.29	28.17
	50.8	-	-	42.18	21.39
1:1	101.6	37.38	28.63	34.56	24.69
	152.5	-	-	28.09	26.39

As depicted in Fig.20, the effect of 50.8 mm, 101.6mm and 152.4 mm screws spacing on the SWPs shear strength with different height-to-width ratios shows a linear variation response. Therefore, intermediate shear strength for screws spacing from 50.8 mm up to 152.4 mm can be computed using this numerical model.

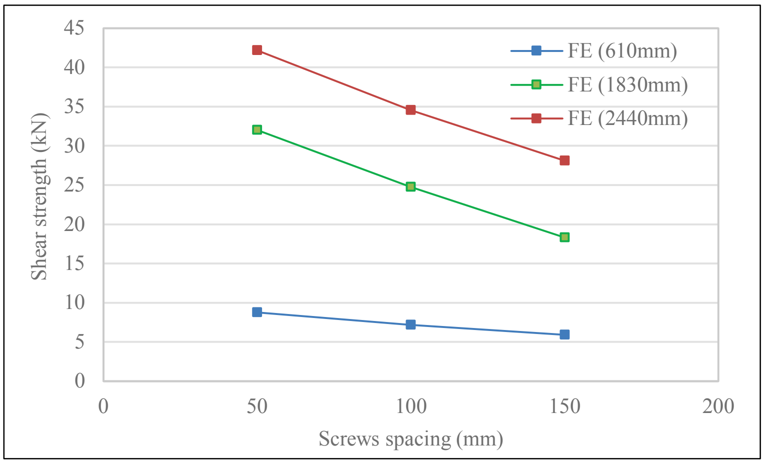


Fig. 20 - Variation effect of screws spacing 50.8 mm, 101.6 mm, 152.4 mm on shear strength

6.3 Effect of sheathing thickness

As demonstrated below (§5.3), the thickness of the steel plate sheathing has an important effect on the SWP strength. The effect of the sheathing thickness having 0.46 mm on the shear strength for SWPs having 4:1, 1.33:1 and 1:1 height-to-width ratios is evaluated with different screws spacing (0.8 mm 101.6 mm and 152.4). As represented in Fig. 21. It is noticed that the screws spacing effect on the shear strength for a fixed height-to-width aspect ratio and sheathing thickness has a linear variation.

Otherwise, the effect of sheathing thickness from 0.76 mm to 0.46 mm on the shear strength with different screws spacing and height-to-width ratio leads to decrease the shear strength with different ratios as summarized in Table 8. This difference of the shear strength is higher in the SWP with narrow screw spacing (50.8 mm) in different height-to-width aspect ratio. Moreover, the sheathing thickness effect does not have a linear variation on the shear strength for all SWPs ratios and with different screws spacing. Therefore, the shear strength for the 0.46 mm sheathing thickness cannot be interpolated directly from 0.76 mm; hence, this predictive FE method can help evaluating more precisely the shear strength of the 0.46 mm sheathing thickness.

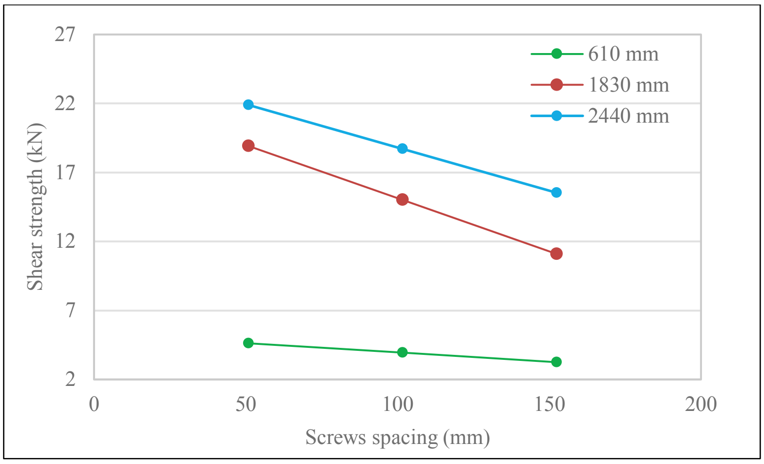


Fig. 21 - Variation effect of screws spacing 50.8 mm, 101.6 mm, 152.4 mm on shear strength

Table 8 - Shear strength of the FE model.

Ratio h/w	Spacing screw (mm)	Shear strength (kN)		ΔF %
		0.76 mm thickness	0.46 mm thickness	
	50.8	8.75	4.63	47.09
4:1	101.6	7.17	4.44	38.08
	152.4	5.91	3.25	45.01
	50.8	32.01	18.92	40.89
1.33:1	101.600	24.76	15.01	39.38
	152.400	18.29	11.09	39.37
	50.8	42.18	21.89	48.10
1:1	101.600	34.56	18.71	45.86
	152.400	28.09	15.54	44.68

7. Conclusion

This paper presents the development and validation of a predictive FE modeling protocol capable to evaluate the design shear strength of CFS-SWPs for different height-to-width ratios, sheathing thickness and screws spacing as alternative to direct interpolations from code and experimental values.

From the validation of the FE model, it can be conclude that:

- Taking into account the material, geometrical and connection assembly nonlinearities, a good agreement has been reached between numerical and experimental results in terms of the nonlinear behavior of shear strength-lateral displacement, screw spacing and sheathing thickness.
- Other than the shear connection which has governed the strength of different height-to-width SWP ratios, buckling of the sheathing and local buckling of the end studs as mode of failure has been captured by this finite element model.
- Discrepancies between numerical and experimental results for 4:1, 2:1, 1.33:1 and 1:1 height-to-width ratios of CFS-SWP with 101.6 mm screws spacing in term of shear strength are in the order of 7.12 %, 5.23 %, 5.17 %, and 7.54 %, respectively. Therefore, this proposed FE modeling protocol developed for CFS- SWP with steel plate sheathing is reliable to evaluate the shear strength of SWP.

The screws spacing and sheathing thickness effect on the strength has been also validated and the following observations and conclusions have been drawn:

- Shear strength of SWPs having 1.33:1 and 1:1 height-to-width ratios and SWPs having 4:1 1.33:1 and 1:1 are evaluated respectively and proposed with sheathing thickness of 0.76 mm and 0.46 mm for the practicing engineers in the absence of experimental results and the AISI code provisions.
- The effect of screws spacing on the shear strength of different height-to-width SWP is evaluated. For a fixed height-to-width ratio, the strength can be interpolated from 50.8 mm up to 154.8 mm screws spacing.
- The shear strength can be underestimated by interpolating the shear strength from 4:1 to 1:1 height-to-width ratios; hence, it is more economical for design purposes to use the proposed FE method to compute the shear strength for different screws spacing's.
- The sheathing thickness has a significant effect on the shear strength of the SWP. The latter cannot be interpolated directly from 0.76 mm to 0.46 mm sheathing thickness. The proposed FE model provides more rational values of design shear strengths for different sheathing thickness combined to different height-to-width ratios and screws spacing that can be used by practicing engineers.

REFERENCES

[1]- M.R. Javaheri-Tafti, H.R. Ronagh, F. Behnamfar, P. Memarzadeh, An experimental investigationon the seismic behavior of cold-formed steel walls sheathed by thin steel plates. Thin Wall Struct. 80(2014) 66–79. doi:10.1016/j.tws.2014.02.018

[2]- C. Yu, Steel Sheet Sheathing Options for Cold-Formed Steel Framed Shear Wall Assemblies Providing Shear Resistance. Report No. UNT-G70752, American Iron and Steel Institute, Washington, DC, 2009.

[3]- S.E. Niari, B. Rafezy, K. Abedi, Seismic behavior of steel-sheathed cold-formed steel shear wall: Experimental investigation and numerical modeling. Thin Wall Struct. 96(2015) 337–347. doi:10.1016/j.tws.2015.08.024

[4]- J. Da Breo, N. Balh, C. Ong-Tone, C.A. Rogers, Steel sheathed cold-formed steel framed shear walls subjected to lateral and gravity loading. Thin Wall Struct. 74(2014) 232-245. doi:10.1016/j.tws.2013.10.006

[5]- American Iron and Steel Institute, Monotonic and Cyclic Response of Single Shear Cold-Formed Steel-to-Steel and Sheathing-to-Steel Connections, Research report No. CE/VPI-ST-16-01, Virginia, 2017.

[6]- L. Fiorino, D.G. Corte, R. Landolfo, Experimental tests on typical screw connections for cold-formed steel housing. Eng. Struct. 29(8) (2007) 1761–1773. doi:10.1016/j.engstruct.2006.09.006

[7]- K.D. Peterman, N. Nakata, B.W. Schafer, Hysteretic characterization of cold-formed steel stud-to-sheathing connections. J. Constr. Steel Res. 101(2014) 254–264. doi:10.1016/j.jcsr.2014.05.019

[8]- C. Ding, Monotonic and Cyclic Simulation of Screw-Fastened Connections for Cold- Formed Steel Framing, Master of Science thesis in Civil Engineering, Virginia Polytechnic Institute, Virginia, 2015.

[9]- S. Mohebbi, R. Mirghaderi, F. Farahbod, A.B. Sabbagh, Experimental work on single and double sided steel sheathed cold-formed steel shear walls for seismic actions. Thin Wall Struct. 91(2015) 50–62. doi:10.1016/j.tws.2015.02.007

[10]- N. Balh, Development of Seismic Design Provisions for steel Sheathed Shear Walls, Master of Engineering thesis, McGill University, Montréal, 2010.

[11]- AISI S400-15, North American Standard for Seismic Design of Cold-Formed Steel Structural Systems, American Iron and Steel Institute, 2015.

[12]- X. Dai, Structural Behaviour of Cold-formed Steel Cassette Wall Panels Subject to In-plane Shear Load. J. Civil Eng. Res. 3(2) (2013) 65–74. doi:10.5923/j.jce.20130302.01

[13]- N.K.A. Attari, S. Alizadeh, S. Hadidi, Investigation of CFS shear walls with one and two-sided steel sheeting. J. Constr. Steel Res. 122(2016) 292–307. doi:10.1016/j.jcsr.2016.03.025

[14]- S. Kechidi, N. Bourahla, Deteriorating hysteresis model for cold-formed steel shear wall panel based on its physical and mechanical characteristics. Thin Wall Struct. 98(2016) 421–430. doi:10.1016/j.tws.2015.09.022

[15]- I. Rouaz, N. Bourahla, F. Kahlouche, S.A. Rafa, Numerical evaluation of shear strength for Cold-Formed Steel Shear Wall Panel. Int. J. Civil Environ. Struct. Constr. Arch. Eng. 10(3) (2016) 347–351. doi:10.5281/zenodo.1123581

[16]- S. Borzoo, S.R.M Ghaderi, S. Mohebi, A. Rahimzadeh, Nonlinear finite element modeling of steel-sheathed cold-formed steel shears walls. Steel Compos. Struct. 22(1) (2016) 79-89. doi:10.12989/scs.2016.22.1.000

[17]- A. Farzampour, J.A. Laman, Behavior prediction of corrugated steel plate shear walls with openings. J. Constr.

Steel Res. 114 (2015) 258-268. doi:10.1016/j.jcsr.2015.07.018

[18]- L.G. Vigh, A.B. Liel, G.G. Deierlein, E. Miranda, S. Tipping, Component model calibration for cyclic behavior of a corrugated shear wall. Thin Wall Struct. 75 (2014) 53–62. doi:10.1016/j.tws.2013.10.011

[19]- H. Kalali, T.G. Ghazijahani, M. Hajsadeghi, T. Zirakian, F.J. Alaee, Numerical Study on Steel Shear Walls with Sinusoidal Corrugated Plates. Latin Am. J. Solids Struct. 13(15) (2016) 2502-2514. doi:10.1590/1679-78252837

[20]- S. Esmaeili Niari, B. Rafezy, K. Abedi, Numerical study on the shear resistance of cold formed steel shear wall with steel sheathing. Asian J. Civil Eng. (BHRC). 14(3) (2013) 461-476.

[21]- ECCS TC7 TWG 7.10, The testing of connections with mechanical fasteners in steel sheeting and sections, ECCS-European Convention for Constructional Steel-work, Portugal. 2009.

[22]- CSI; SAP2000. Computers and Structures, 2014.

[23]- Dassault Systems; Abaqus analysis user's manual: Prescribed Conditions, Constraints, Interactions, USA, 2018.

[24]- X. Dai, Numerical Modelling and Analysis of Structural Behavior of Wall-stud Cold-formed Steel Shear Wall Panels under In-plane Monotonic Loads. J. Civil Eng. Res. 2(5) (2012) 31–41. doi:10.5923/j.jce.20120205.02

[25]- AISI, S100-2007, North American Specification for the Design of Cold-Formed Steel Members, American Iron and Steel Institute, Mexico, 2007.