Numerical Assessment of CFS-SWP with Different Door Opening Positions

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http://doi.org/10.29227/IM-2024-02-21

Submission date: *17.04.2024.* | Review date: *21.05.2024*

Abstract

A prevalent method of countering lateral forces in seismic regions for structures made of Cold Formed Steel (CFS) involves employing Shear Walls Panels (SWP) covered with steel sheets. The overall reaction and various failure modes during lateral loading have become apparent, continuing to capture the interest of designers and researchers. Typically, door openings are ever-present in the front and rear elevations where SWPs find their optimal position to ensure lateral stability in CFS structures. These architectural design features translate into reduced areas for lateral load resistance throughout the structure. Through numerical simulations, this paper discusses the effect of the door opening, with different positions in the SWP, on the shear strength performance of CFS-SWP. A Finite Element (FE) modelling is developed using the ABAQUS software, taking into account material and geometrical nonlinearities, as well as assembled sheathing-to-framing connections. In order to validate the FE modelling with available experimental data, comprehensive static nonlinear analyses are conducted on CFS-SWP under monotonic load. A good agreement is achieved, namely: nonlinear strength-displacement response, ultimate shear strength, and failure modes. The effect of the door positions is assessed, in which, the opening position is found to have a significant impact on the CFS-SWP performance. The results reveal that the position of the door opening between the central and the edge of the SWP produces better performance than the other positions. However, the position of the door at the edge of the SWP induces additional failure modes in the vicinity of the chord studs.

Keywords: *Cold Formed Steel, Shear Wall Panel, Shear Strength. Door opening, Nonlinear analyses*

1. Introduction

In seismic-prone areas, cold-formed steel (CFS) shear wall panels (SWP) wrapped in steel plate sheets have become increasingly popular as a means of successfully resisting lateral loads, such as wind and earthquakes. Researchers [1-2] have conducted a number of experimental research programs on CFS-SWP with steel sheathing under lateral loads in order to assess their shear strength for use by practicing engineers and designers. It was concluded that the nonlinear behavior of CFS-SWP is governed by the screw fasteners' failure mode. Further many research programs were conducted to address the connection impact on the overall shear wall response [3-7].

Indeed, the AISI S400-15 code [8] provides a nominal shear strength of 610 mm and a 1220 mm length CFS-SWP with a 0.76 mm sheathing thickness, based on experiment testing conducted by Balh [9]. Shear wall panels longer than 1220 mm, however, have various openings leading to them in the architectural and functional design. However, the complete range of CFS-SWP configurations is not covered by the tabular shear strength values published by AISI code [8], particularly for SWP with a length of 3660 mm. With respect to the experimental tests, the numerical analysis based on the Finite Element (FE) method for studying the performance of the CFS shear wall has become an effective approach to cut down on time and money consumed on further investigations. Thus, the use of FE models turns out to be a good alternative for predicting the behavior and evaluating the shear strength of the SWP for various geometric and mechanical characteristics [10-11].

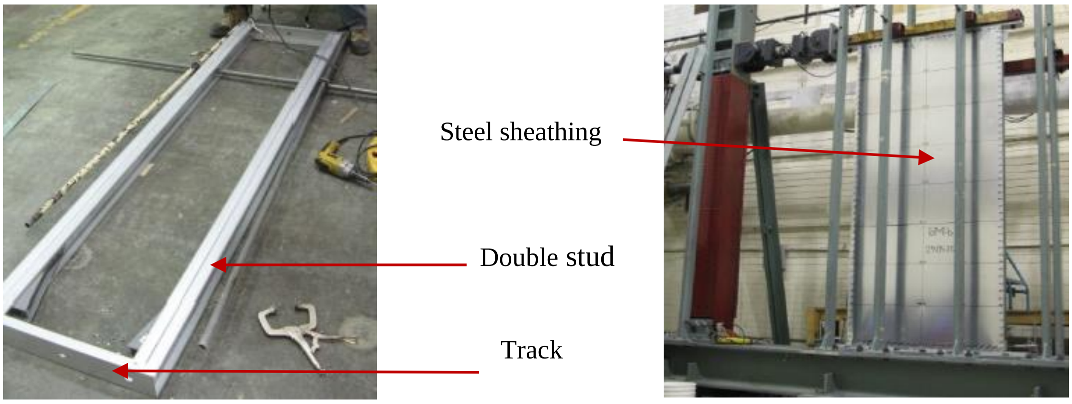
In order to develop a reliable simulation to assess the shear strength and the nonlinear behavior of the CFS-SWP, several FE modellings have been done. Ghaith et al. [12] implemented a linear kinematic hardening user-defined material model (VUMAT) in their analyses to simulate the material's nonlinearity. Due to the thin thickness of the CFS elements, the geometric imperfection has also been taken into account in the simulation. In addition, Rouaz et al. [13,14] have explored the global behavior of a 3660 mm SWP, but with corrugated steel and Oriented Strand Board (OSB) sheathing. They have concluded that the introduction of the window and door opening has a tangible effect on the shear strength and failure mode. The shear strength of cold formed steel shear wall panels with lengths of 610 mm and 1200 mm was numerically evaluated by Niari et al. [15]. To validate their simulation, they were based on the buckling of sheathing and the local backing of components according to the major experimental findings conducted by Balh [9]. Despite the experimental investigations [9] demonstrating that the shear screw connections between elements, particularly the sheathing-to-framing connections, dominated the shear strength and the global nonlinear response of SWPs under horizontal load, the numerical modeling did not take screw connection nonlinearity into account [15].

Taking into account the nonlinearity of the screw connections in addition to the material and geometrical nonlinearities, a detailed FE model is proposed in this paper, relying on the experimental test carried out by Balh [9]. The nonlinear response and ultimate shear strength of the SWPs with 1220 mm, 1830 mm, and 2440 mm lengths have been validated. However, 3660 mm SWP length with and without door opening in different positions is investigated and proposed for engineering practicing.

2. Specimens' Characteristics

Based on the experimental research conducted by Balh [9], three SWPs with lengths of 1220 mm, 1830 mm, and 2440 mm were selected in order to validate the Finite Elements modeling. To prevent buckling failure modes, built-up back-to-back chord studs are put at either end of the SWP and joined by two No. 10–16 x 19.1 mm hex washer head self-drilling screws at 305 mm on-center. Each end of the chord studs has a Simpson Strong-Tie S/HD10S hold-down device linked to the inside web by No. 14 x 30 mm self-drilling hex washer head screws. Additionally, the SWP is composed of a single field stud spaced 610 mm along the length of the wall. The nominal dimensions of the steel studs are 92.1mm web, 41.3mm flange, and 12.7mm lip. While the nominal dimensions of the steel top and bottom tracks are 92.1mm web and 31.8mm flange. The thickness of the studs and tracks is 1.09 mm.

The steel sheets, with 0.76 mm, in two sizes: 2440 mm x 610 mm and 2440 mm x 1220 mm, according to the SWP's length, 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 of the sheathing, and the spacing of screws along the field stud(s) is 300 mm. Figure 1 presents a view of the assembly and installation of the tested SWP.

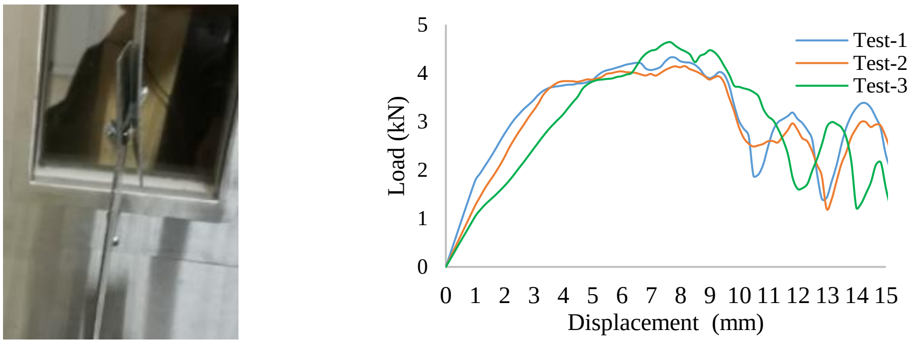


(a) Frame Assembly (b) Wall Installation in test Frame

Fig. 1. Tested SWP [9].

3. Material and Screw Connection

It is necessary to identify the nonlinearity sources in order to account for the material and screw connection nonlinearities in the finite element model. Hence, a normalized stress-strain curve [16] that resembles the specimen members' is used to choose the steel mechanical material. Moreover, the nonlinear behavior of screw fasteners needs to be determined because Balh did not report it. Therefore, a series of experimental tests were undertaken in the CNERIB laboratory (National Center of Studies and Integrated Research on Building Engineering) in order to determine the nonlinear force-displacement curves. The tests were carried out following the European Standards ECCS TC7 TWG 7.10 (2009) [17], where the dimensions of the test specimens were similar to those of SWP's members. Thereby, the test results of sheathing-to-farming screw fasteners of 0.76 mm with 1.09 mm of thickness under a tensile load confirm the tilting failure mode (Figure 2-a) as found and described in the experimental part done by Bahl [9]. The force-displacement results are given in Figure 2-b. The average capacity of the test results is 4.17 kN, where the difference is about 4 % compared to the test result obtained by Balh [9].



(a) Tilting of sheathing Screw (b) Force-displacement results

Fig. 2. Sheathing-to-Framing test connection test.

4. Finite Element Modeling

Screw fasteners are used to secure CFS C-shaped framing members, such as studs and tracks, and sheathing in SWPs. It is important to pay close attention to modeling this structural system in order to study its overall behavior and performance. This includes considering the primary failure modes, such as screw fastener failure, and local or global buckling of the frame components. In this study, through a Finite Element modeling (FE), using ABAQUS/ software [18], all the components: studs, tracks, and steel sheeting (Figure 3) are modeled as deformable Shell elements (S4R: Shell element with reduced integration). The non-linear stress-strain behavior of the material is converted into true plastic stresses and true strains to be introduced into the numerical code [18]. Predefined connectors in the Abaqus software library are used to model the assembly between the different elements of the SWP. The force-displacement strength assembly results obtained from tests are introduced into these connectors.

Following the large deformations induced in the elements of the tested SWPs under horizontal loading during the test, it was deemed imperative to take into account geometric nonlinearity during the analysis. Drawing from analogous prior research on

CFS-SWP [19-21], the mesh dimensions adopted for this study are 41 mm x 38 mm for the structural plate, 41 mm x 29 mm for the track, and 38 mm x 41 mm for the studs. In addition, monotonic displacement is applied to all the nodes of the upper track to simulate the boundary conditions of the experimental tests. This load allows the SWP to move along the (Z) and (Y) axes and rotate around the (X) axis. However, the out-of-plane movement of this upper track is blocked, according to the experimental part.

Furthermore, surface-to-surface contact is 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 the sensitivity analysis performed by Dai [22], the value of the friction coefficient does not affect the results. Therefore, a friction factor of 0.3 was adopted in this study. General static analysis is implemented with an initial increment equal to 0.1 and a minimum increment equal to 10e-8. Thus, the Newton-Raphson method is used for this analysis.

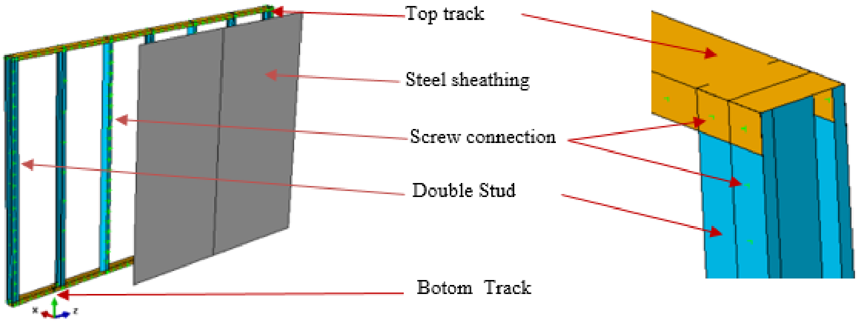


Fig. 3. Finite Element modeling of CFS-SWP.

5. Validation of the Finite Element Model

5.1 Global Nonlinear Response

Three SWPs with lengths of 1220 mm, 1830 mm, and 2440 mm were chosen from experimental tests in order to assess the shear strength of SWP under a monotonic load and confirm the reliability of the suggested FE modelling [9]. For every SWP, the steel plate sheathing screw spacing is 101.6 mm.

Figure 4 shows the nonlinear response "shear strength-lateral displacement" behavior of these different shear wall panels. For each of the three SWPs, from the stared loading until the ultimate strength, there is a good degree of correlation between the measured and FE-predicted responses. Some differences in displacement at peak load and nonlinear response after the post peak are visible, particularly for the SWP having an 1830 mm length. This is mainly due to the uncertainties' accumulations that are caused by the assumptions related to the FE modeling concerning the mechanical properties of the SWP steel members and the hold-down stiffness connectors that were attached to the interior base of the stud in the experimental test. Moreover, the boundary conditions of the small out-of-plan displacement that occurred during the test at the top of the SWP were neglected in the FE model.

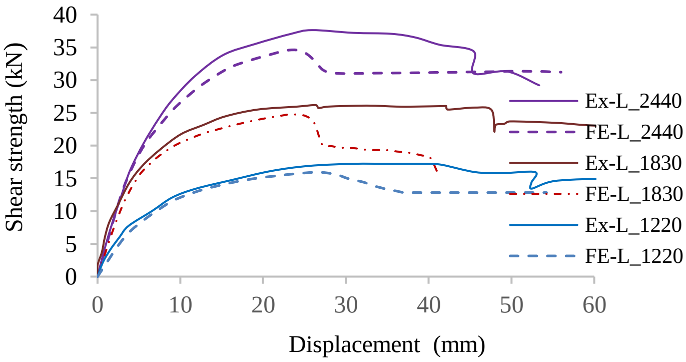


Fig. 4. Nonlinear response of SWPs.

Figure 5 presents the difference between numerical and experimental results in terms of shear strength and corresponding ultimate displacement for the three SWPs.

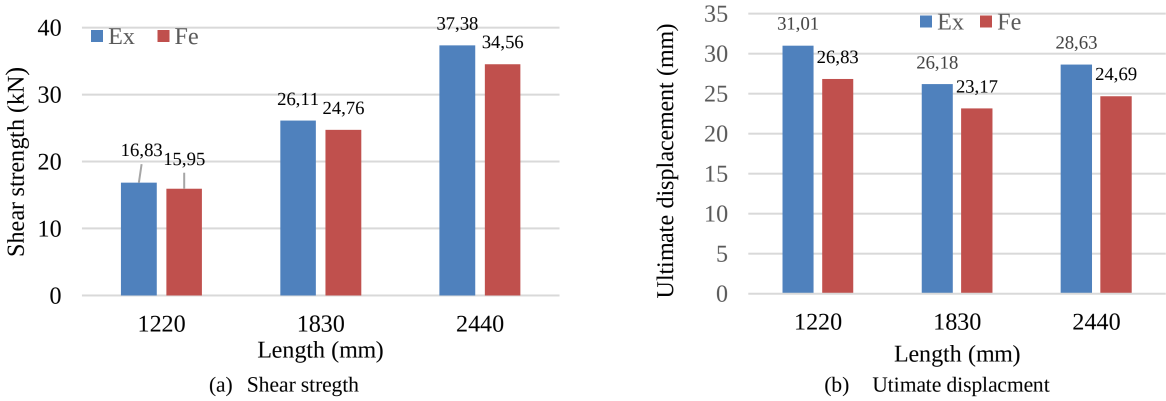


Fig. 5. Ultimate response: numerical versus experimental results.

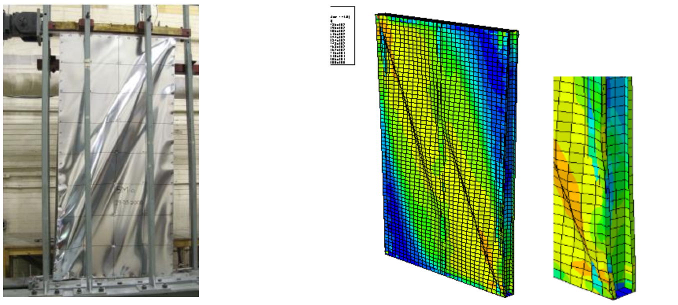
The highest difference in shear strength is reached in the SWP with a length equal to 2440 mm; this difference is estimated at around 7.54%, and it decreases in SWPs with 1220 mm and 1830 mm of length, about 5.23 % and 5.17 %, respectively. Furthermore, the ultimate corresponding displacement is more pronounced in the case of SWPs having 2440 mm of length than SWPs with 1220 mm and 1830 mm of length. 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 the overturning moment.

Despite the fact that this proposed FE modeling protocol has a small difference in results from those corresponding to test results, it gives a reasonable prediction of the shear strength of the CFS SWP with steel plate sheathing for the three different lengths of SWPs.

5.2 Failure Modes

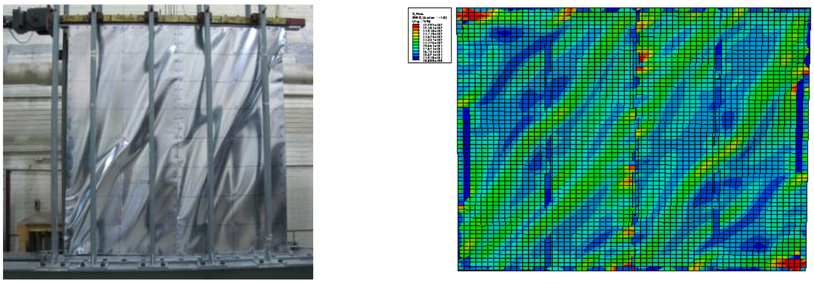
The shear failure of the screw connections was the dominant failure mode and governed the global nonlinear behavior of the SWPs. However, during the experimental tests [9], it was noticed that a tension field action was also developed, leading to the buckling of the steel plate sheathing. Moreover, a 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 modelling to take into account these additional failure modes in the simulation.

As depicted in Figure 6, the local buckling of the end stud is captured by the FE model, which is due to the compression force developed in this thin steel sheeting after loading. As 1220 mm SWP length, two tension fields are developed in the steel sheathing and simulated by the FE model as well as in the corresponding experimental outcomes. As shown in Figure 7, comparing the finite elements model with experimental results during the loading, a good similarity is reached in terms of the development of the tension fields, which involves buckling of the steel plate sheathing in 2440 mm SWP length. Therefore, it can be concluded that this proposed FE modeling takes into account other failure modes, such as buckling of the sheathing and local buckling of the end stud, in addition to shear failure of screw connections.



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

Fig. 6. Failure mode of SWP having 1200 mm of length.



(a) Experimental result (b) FE elements simulation

Fig. 7. Developing failure mode of SWP having 2440 mm length during loading.

6. Evaluation of SWP Strength with Different Door Opening Positions

A 3660 mm SWP full sheathed has been modeled based on the evaluated finite elements method. The shear strength of this SWP is compared with the previous different SWP length. Then, the effect of the position of the door opening on the shear strength is evaluated.

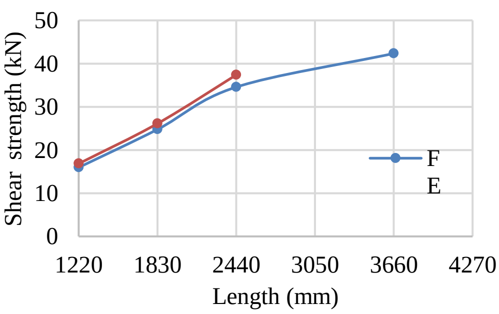


Fig. 8. shear strength of different SWPs.

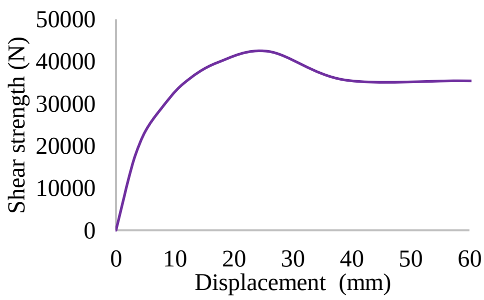
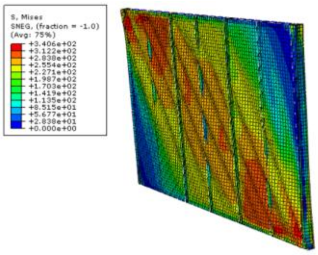


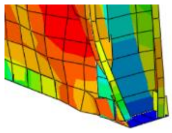
Fig. 9. Nonlinear response of full 3600 mm SWP.

So, to check, through numerical simulations, whether the shear strength of different lengths of SWP with 101.6 mm screws' spacing can be assessed and predicted by extrapolation from 1220 mm SWP length, Figure 8 shows that increasing the SWP length from 1220 mm up to 2440 mm has a slight nonlinear variation effect on the shear strength of SWPs obtained by this FE modeling (FE) and experimental results. However, for the full sheathed SWP with 3660 mm, the nonlinear effect becomes more important. It was expected that the full sheathed shear wall with 3660 mm of length has a shear strength of about 47.85 kN by extrapolation from numerical modeling of 1220 length, while the numerical results show that this shear strength is assessed at 42.34 kN (Figure 9), leading to a difference of 11.5%. This means that assessing the shear strength by extrapolation overestimates the real shear strength of SWP. Therefore, it is more rational to use the shear strength of the SWP with 3660 mm of length based on numerical modeling rather than estimating the shear strength by extrapolation.

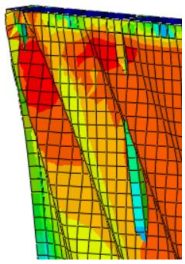
However, several deformations of the members' full SWP are simulated due to the lack of experimental tests. Figure 10 highlights the tension fields that develop in the different steel sheathings. The stress concentration, according to the Von Misses criterion, is located at the top and bottom of the SWP's corner.



(a) Tension field through different sheaths



(b) Deformation at the corner of the SWP: bottom



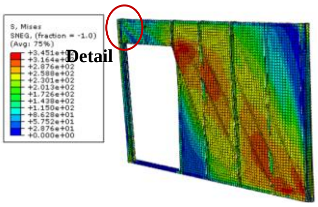
(c) Deformation at the corner of the SWP: top

Fig. 10. Stress concentration and deformation of the full sheathed SWP's members.

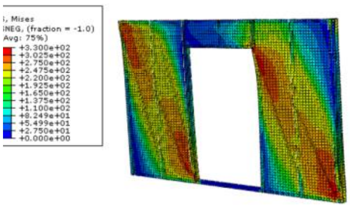
The door opening is introduced in the SWP with 3660 mm of length in order to investigated numerically its effect on the shear strength. Following architectural practice, the dimensions of the door opening are 1200 mm x 2100 mm. Three typical positions of these door openings are depicted in Figure 11. keeping all the wall's dimensions and the cross-section properties of the framing members the same as described in Section 2.

Based on this finite element method, the numerical results show that the shear strength is different in all positions. The nonlinear shear strength-ultimate displacement is depicted in Figure 12. It reveals that the SWP, with an opening located in position #3, between the central position and the edge of the SWP, has a higher ultimate shear strength than the other positions, assessed at about 29.09 kN. In fact, additional screw connections are installed on the intermediate stud to connect small sheathing (610 mm in length) to the SWP framing; this position (#3) has one more row of screw connections compared to other positions (*i.e.*, #1 and #2).

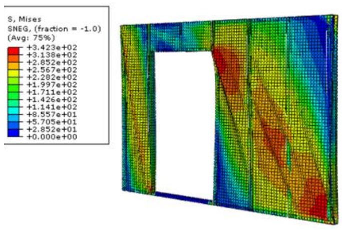
As far as door positions #1 and #2 are concerned, these two positions have almost the same number of screw connections. However, the shear strengths of door positions #1 and #2 are, respectively, 26.18 kN and 27.46 kN, leading to a difference of 4.9%. This is because of an addition failure that has appeared as a significant distortion at the end of the chord stud of SWP with door position #1 (Figure 11-d). This is due to the asymmetry of stress transfer from the unique steel sheathing, installed at the top of door position #1, to the double studs located at the edge of the SWP.



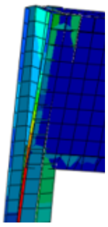
(a) Position-1



(b) Position-2



(c) Position-3



(d) Detail: distortion at the top chord stud (position #1)

Fig. 11. Plastic deformation of SWP with different door positions.

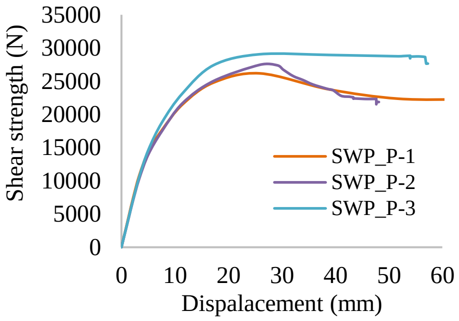


Fig. 12. Shear strength- displacement response of different SWPs' openings.

7. Conclusion

The development and validation of a predictive FE modeling procedure that can assess the shear strength of CFS-SWPs for varying lengths is presented in this study, from which the following inferences can be drawn:

- Taking into account the material, geometrical and connection assembly nonlinearities, a good agreement has been reached between the numerical and experimental results in terms of the nonlinear behavior of shear strength-lateral displacement.
- This finite element modeling has captured the buckling of the sheathing and local buckling of the end studs as modes of failure, in addition to the shear connection that has controlled the strength of various lengths.
- The discrepancies between the numerical and experimental results for 1220 mm, 1830 mm, and 2440 mm CFS-SWP' length with 101.6 mm screw spacing in terms of shear strength are of 5.23 %, 5.17 %, and 7.54 %, respectively.

Therefore, this proposed FE modeling developed for CFS- SWP with steel plate sheathing is reliable to evaluate the shear strength of SWP for other lengths.

In addition, the shear strength of the full steel-sheathed SWP with 3660 mm of length has been assessed based on the presented FE modeling without an experimental test. The numerical outcomes reveal that assessing the shear strength, from 1220 mm to 3660 mm of length, by extrapolation overestimate the real shear strength of SWP. Moreover, the position of the door opening has a tangible effect on the SWP's response. It turns out that the best performance is produced by placing the door opening between the center and the edge of the SWP. The door's location at the edge of the SWP, however, leads to an additional failure mode close to the chord studs.

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