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## **Finite Element Analysis on Shear Strength of Cold-Formed Steel Shear Walls Using Corrugated Steel Sheathing**

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### **Abstract**

Cold-formed steel (CFS) shear walls using corrugated steel sheathing is a newly proposed lateral resisting system from recent research. In this paper, Finite element models of cold-formed steel framed shear walls sheathed by corrugated steel sheets were created in Abaqus software. The validity of the numerical model was verified based on previous test results. Agreement of the FEA results and test results indicated that the proposed numerical model was able to accurately predict the shear resistance of cold-formed steel shear walls with corrugated steel sheathing. A series of parametric analysis were then conducted, including the thickness of framing members, cross section of stud members, yield strength of frame members, stud spacing, and the influence of gravity load. The detailed modeling information, relevant parametric analysis results and recommendations for practical application of this type of shear resisting system are presented in this paper.

### **1. Introduction**

Cold-formed steel (CFS) shear walls using corrugated steel sheathing is a newly proposed lateral resisting system from recent research (Fülöp and Dubina 2004, Stojadinovic and Tipping 2007, Yu et al. 2009, Yu 2013). It has been found that CFS framed shear walls using corrugated steel sheathing yielded higher strength, greater initial stiffness with similar ductility under cyclic loading when compared to the CFS walls using conventional sheathing materials.

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Currently, most shear walls are designed through empirical methods derived directly from full scale tests, which is time-consuming and costly. Numerical simulations, are an equivalent method which allow researchers to study the performance of these lateral resistant systems, and to share discoveries with designers. In this paper, finite element models of cold-formed steel framed shear walls sheathed by corrugated steel sheets were created in Abaqus software and validated according to the test results. In addition, a series of parametric analysis were conducted, including thickness of framing members, cross section of stud members, yield strength of the frame members, the stud spacing, and the influence of gravity load. The detailed modeling information, relevant parametric analysis results and recommendations for practical application of this type of shear resisting system are presented.

**2. Experimental Results**

Extensive studies on shear wall systems with corrugated steel sheathings under displacement-controlled loading were completed at University of North Texas and reported in M. Mahdavian thesis (Mahdavian 2016). Non-perforated shear wall with corrugated steel sheathings under monotonic loading, Test No.54 in M. Mahdavian’s thesis, was chosen as the prototype model in this research. The test setup is shown in Figure 1. The shear wall is tested on a 16 ft. x 12 ft. self-equilibrating steel testing frame which is equipped with a 35 kip hydraulic actuator. The shear wall is fixed to the test bed and the force is applied to the top track of the wall horizontally through a load beam. A 20 kip compression/tension load cell is used to measure the applied force. The load cell is placed between the actuator shaft and the load beam via pin connections. The top track of the wall is attached to the load beam using No. 12 hex washer head (HWH) self-drilling screws. The out-of-plane movement of the wall is prevented by the lateral supports placed on both sides of the load beam. Five position transducers are employed to measure the horizontal displacement at the top of the wall, and the vertical and horizontal displacements at the bottom of the two boundary studs.

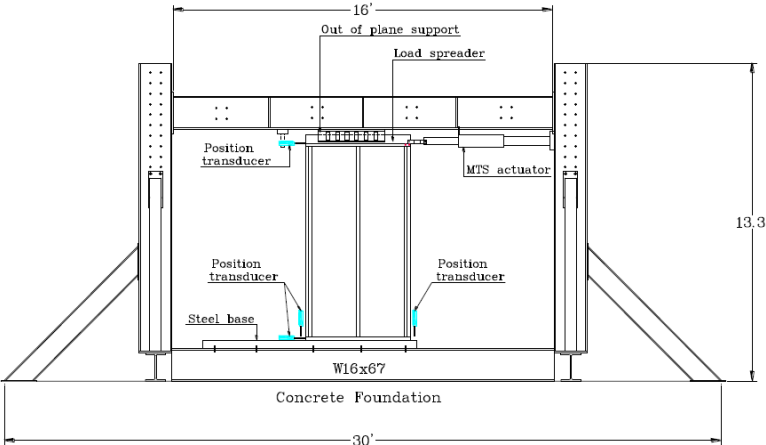


Figure 1: Test setup

The shear wall specimens studied were 8 ft. high by 4 ft. wide (2:1 aspect ratio). Steel Stud Manufacturers Association (SSMA) structural stud (50 ksi 350S162-68) and track members (50 ksi 350T150-68) were used for the framing members. The boundary studs used double C-shaped studs fastened together back-to-back with No.12  $\times$  1 in. Hex Washer Head (HWH) self-drilling screws paired at 3 in. on center. The middle stud used one single C-shaped member. Sheathing, shown in Figure 2, was Verco Decking SV36-27 mil thick corrugated steel sheet with 9/16 in. rib height. The sheathing was installed on one side of the wall using No.12  $\times$  1 in. Hex Washer Head (HWH) self-drilling screws. For each wall, the sheathing was composed of three corrugated steel sheets. Due to the metal sheathing profile, the spacing of the screws were limited to 3 in. module at the horizontal seams of the sheets, 3 in. along the perimeter and 6 in. along the interior stud.

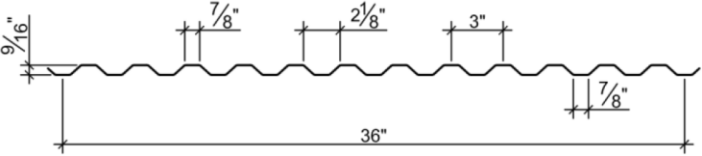


Figure 2: Verco Decking SV36 sheathing profile

The procedure of the monotonic test is in accordance with ASTM E564 (2012) “Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings”. The displacement was applied to the top of the wall at a uniform rate of 0.0075 in/sec. The observed failure modes were screw pulling over the bottom sheet at connections, and shear buckling on bottom corrugated sheet, shown in Figure 3. By the end of loading, local and torsional buckling of compression studs were also observed. The load vs. displacement curve is shown in Figure 4.



(a) Sheet buckling (b) Screw pulling over (c) Stud buckling

Figure 3: Observed Failure Modes of Shear Wall under Monotonic Lateral Loading

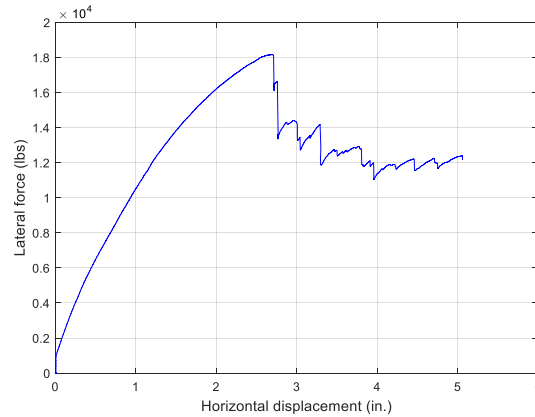


Figure 4: Load-Deformation Response

### 3. Finite Element Analysis (FEA)

#### 3.1 Modeling Technique

##### 3.1.1 Components & Geometry

The dimensions and thicknesses of each shear wall components were from the Steel Stud Manufacturers Association product technical guide (SSMA 2015). The profile dimensions of the corrugated sheathings were in accordance with those provided by Verco Decking, also seen in Figure 2. It should be pointed out that the top and bottom tracks were modeled 0.08 in. wider so the studs would fit within the tracks to eliminate contact. All components were modeled using 4-node homogeneous shell elements, type S4R, in Abaqus. The mesh size of the framing members used 0.5 in. and 1.5 in. for the corrugated sheets.

##### 3.1.2 Material Properties

Bilinear Isotropic material properties were used for both framing members and corrugated sheathings. The Young's modulus was set as 29,500 ksi and Poisson's ratio was 0.3. The modulus of the second phase was set as 1/100 of the elastic modulus, which was based on the coupon test results in Mahdavian's thesis (Mahdavian 2016). Nominal yield strength was used for all elements in this research.

##### 3.1.3 Interaction

Since no framing connection failure occurred in all the tests, tie constraints were used for stud-to-stud and stud-to-track connections. It is important to mention, members selected as master or slave are of great significance in finite element analysis. Figure 5 shows the tie constraints of the stud-to-track and stud-to-stud connections.

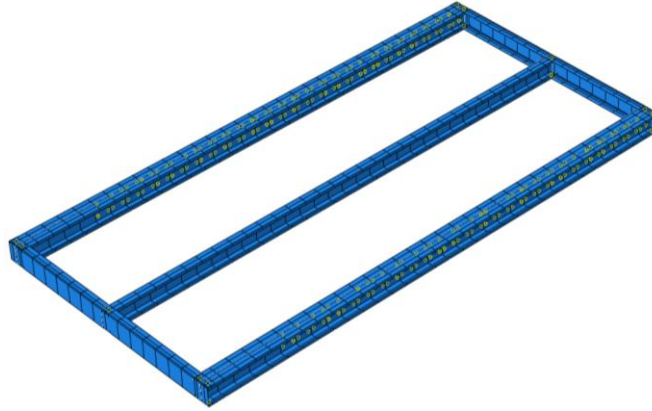


Figure 5: Framing tie constraints

#### 3.1.4 Boundary Conditions

All the nodes on the web of the bottom track as well as the bottom edges of the studs are restrained in all three directions following the test setup. Two lines of nodes on the web of the top track were restricted against the translation toward the out-of-plane direction in order to simulate the lateral support, shown in Figure 6. Also, the vertical direction of all the nodes at the hold-down area of each chord stud is restrained, as shown in Figure 7.

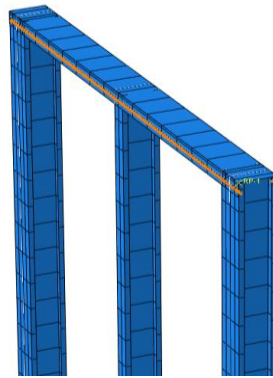


Figure 6: Out-of-plane boundary condition

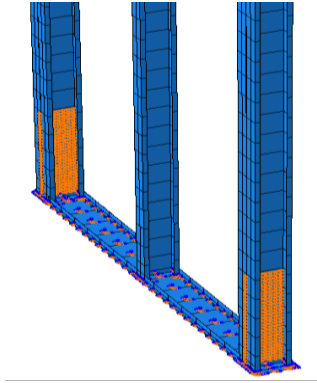


Figure 7: Bottom and Hold-down boundary conditions

### 3.1.5 Contact Properties

A contact property was introduced between the surfaces of the corrugated sheathing and the studs to prevent the sheathing from penetrating through the framing members. A “frictionless tangent” behavior and “hard-contact normal” behavior were defined at these locations. The contact locations can be seen in Figure 8.

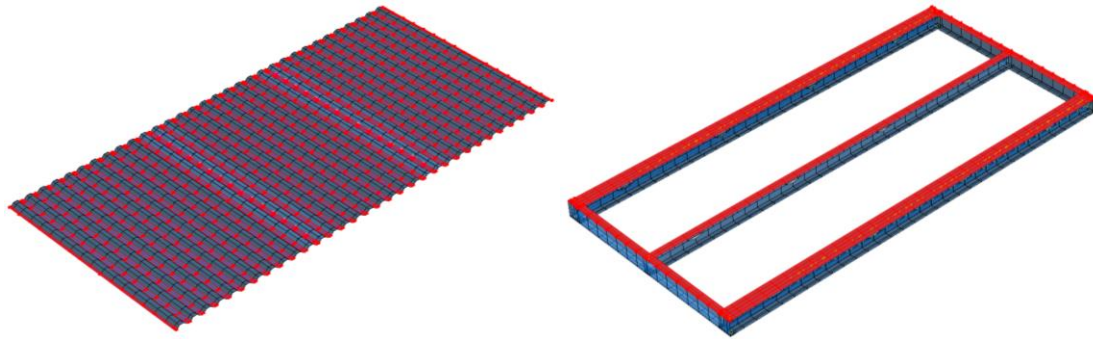


Figure 8: Contact surface locations

### 3.1.6 Sheathing Connections

The sheathing-to-frame and sheathing-to-sheathing screws were simulated by spring2 elements in Abaqus. This type of spring defines an element between 2 nodes acting in a fixed direction. Each screw connection was modeled by 3 spring elements, one withdrawal spring and two shear springs. The spring stiffness was based on connection test results. The simulation of the spring connections is shown in Figure 9.

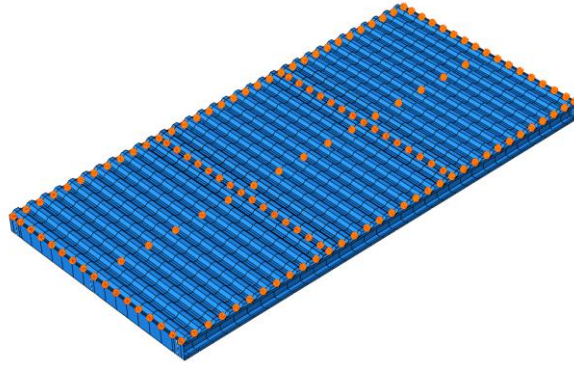


Figure 9: Spring connections

### 3.1.7 Loading Method

All the nodes on the web of top track were coupled to a reference point located on the edge of the top track, as depicted in Figure 10. A displacement controlled lateral load was applied to the reference point along the horizontal direction.

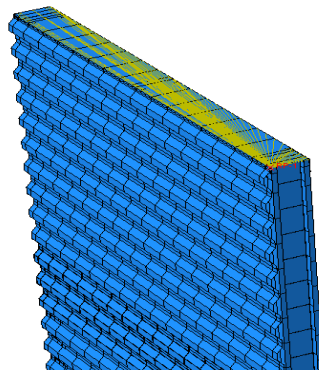


Figure 10: Loading method

### 3.2 Simulation Results

To verify the validity of the finite element model, the FEA results were compared with test results numerically as well as in terms of deformation and performance. The load-displacement responses are illustrated in Figure 11 and the comparison of the characteristic values is shown in Table 1. The Abaqus model was able to match the shear wall behavior well prior to the peak load. The initial stiffness of the Abaqus model is comparable to the full scale test initial stiffness. The displacement at the peak load determined from the test differs somewhat from that obtained by FEA, and the difference reaches 14%. However, the shear capacities are almost the same, which validates the accuracy of the FE model. In the full scale test, the shear wall failed due to shear buckling of the bottom sheet which led to the screw pull-over failure at the sheet-to-stud connections. In Abaqus, the initial failure observed was the buckling of the corrugated sheet. Stress

distribution was mainly focused on the bottom corrugated sheet which was in accordance to the full scale test results. In the test, the second loss of strength was caused by the local buckling and distortional of the chord studs. A slight torsional and local buckling of the chord stud was also noticed in the model. The comparisons of the failure modes are shown in Figure 12.

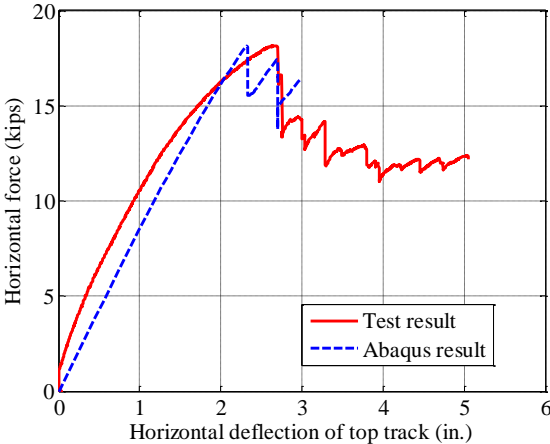
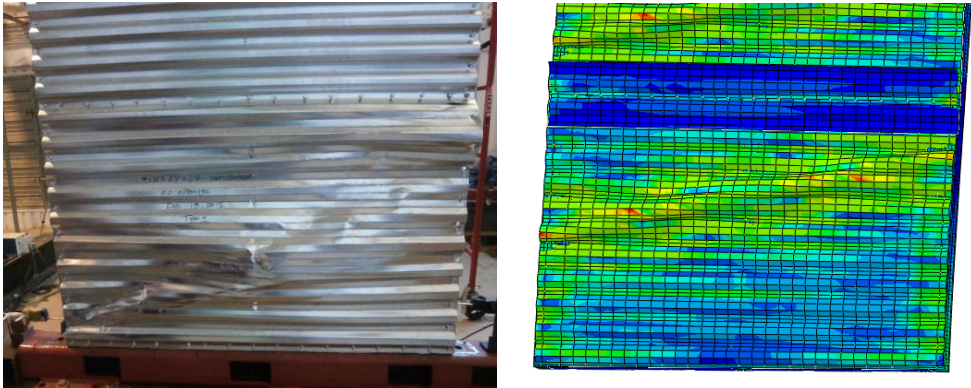


Figure 11: Load vs. displacement responses

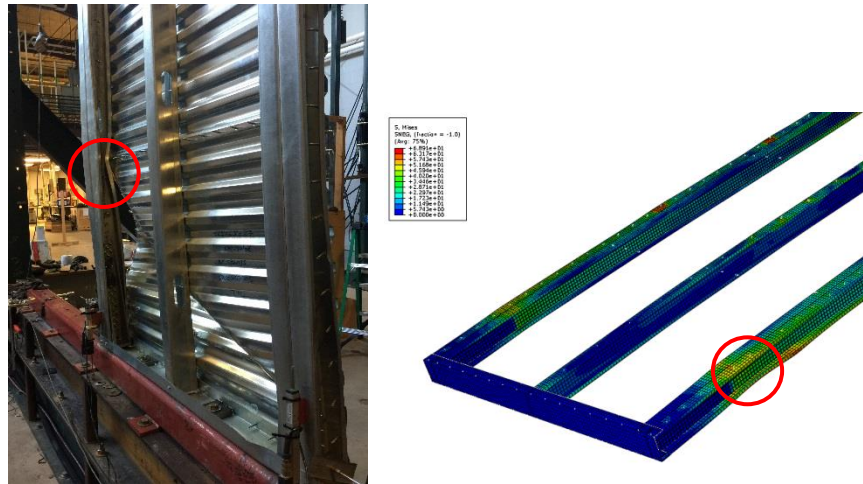
Table 1: The comparison of test results and Abaqus results

	Pmax (kips)	Ratio	$\Delta$ max (in.)	Ratio
Test results	18.17	-	2.694	-
Abaqus results	18.16	1.00	2.328	0.86



(a) Stress distribution on bottom sheet





(b) Local and distortional buckling of studs

Figure 12: Failure modes

## 4. Parametric Analysis

### 4.1 The influence of framing thickness

A desired ultimate failure state of CFS shear wall should ensure that the stress of the framing members remains at a relatively low level (elastic stage) while the sheathing buckling occurs. As a result, the adoption of framing thickness is of great importance. The framing thicknesses in this research included 54mil, 68mil, and 97mil and the sheathing thickness remained constant at 27mil for all tests. The comparison of load vs. displacement curves is shown in Figure 13, from which we can see that there is an increase in shear strength and a decrease in deflection at peak load as the framing thickness increases. Compared to the 54mil framing thickness shear wall, the shear strength of shear walls with 68mil and 97mil framing thickness improved 3.7% and 9.0%, respectively. However, the failure modes of 54mil framing thickness shear wall included sheet buckling as well as stud buckling, as shown in Figure 14.

In conclusion, the influence of framing thickness on shear capacity of corrugated steel sheathed shear walls is minimal. It can also be concluded that when using corrugated steel sheets with 27 mil thickness, the framing members must be of 68 mil or higher thickness to avoid stud buckling and framing failure.

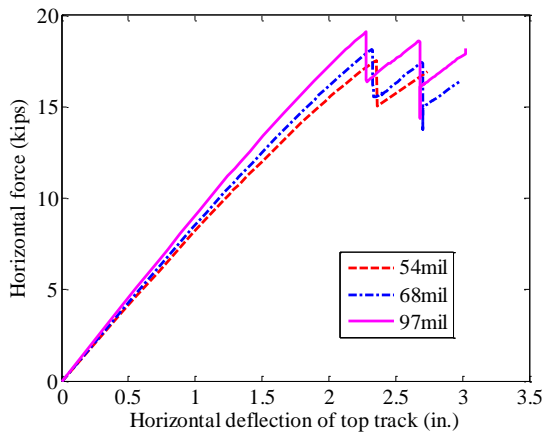


Figure 13: Load deformation responses

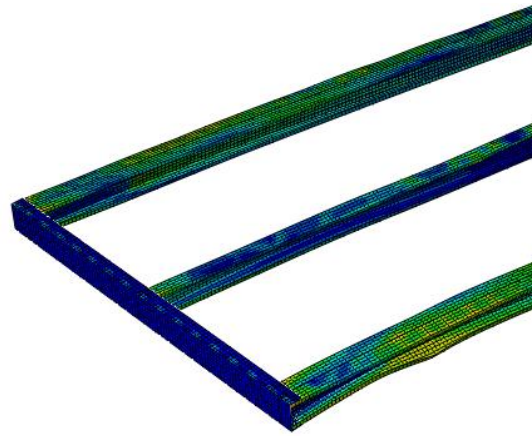


Figure 14: Stud buckling

#### 4.2 The influence of stud cross section

To explore the influence of stud cross section on the shear capacity of the shear wall, 4 shear wall models with different stud cross sections were simulated, including stud 350S162-68, 362S162-68, and 400S162-68. The framing thickness remained the same and only the height of the stud varies. The load vs. displacement curves are shown in Figure 15 and the comparison of the shear strength is summarized in Table 2. It can be observed that in comparison to 350S162-68 stud wall, the differences of the shear capacity and the displacement at peak load are no more than 1%. Therefore, it can be concluded that the stud cross section has little effect on the shear capacity of the corrugated steel sheathed shear walls and can be neglected in future analysis.

Table 2: The influence of stud cross section

Stud cross section	Pmax (kips)	Ratio	$\Delta_{max}$ (in.)	Ratio
350S162-68	18.16	-	2.328	-
362S162-68	18.18	1.00	2.329	1.00
400S162-68	18.39	1.01	2.344	1.01

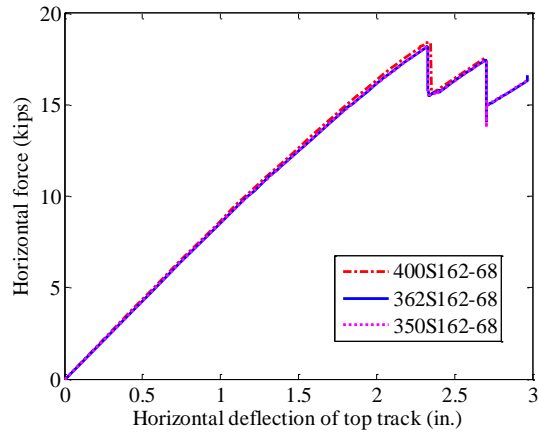


Figure 15: Load deformation responses

#### 4.3 The influence of framing material properties

The yield strength of the framing member may affect the shear capacity of the wall. Two shear wall models with different framing material properties were simulated in this paper, including yield strength of 33 ksi and 50ksi. The results are shown in Figure 16 and Table 3, from which we can see the shear capacity increased by 4% and the displacement at peak point postponed by 3% when the yield strength increased from 33 ksi to 50 ksi. However, stud buckling was also noticed at the failure point in the shear wall model with the 33 ksi yield strength, as shown in Figure 17. Thus such conclusion can be made: the influence of the yield strength of the frame member on shear capacities of corrugated steel sheathed shear walls is very limited and can be neglected. However, it's recommended that yield strength of the frame member to be 50 ksi in order to ensure the strength requirement and to avoid the stud buckling.

Table 3: The influence of framing material

Material grade	Pmax (kips)	Ratio	$\Delta$ max (in.)	Ratio
33 ksi	17.53	-	2.400	-
50 ksi	18.16	1.04	2.328	0.97

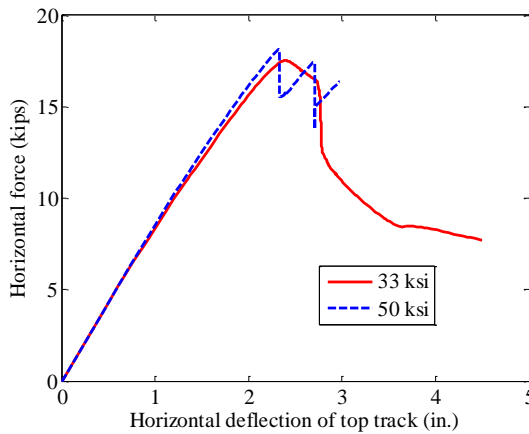


Figure 16: Load deformation responses

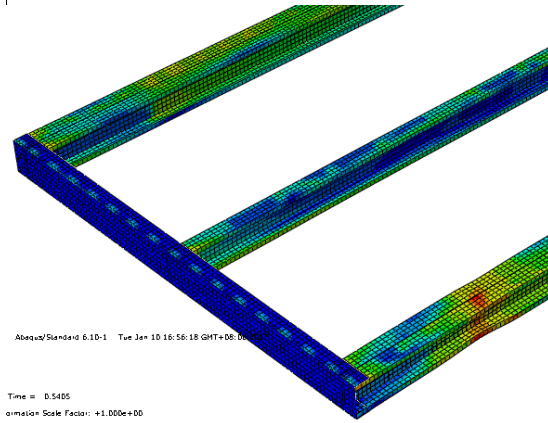


Figure 17: Stud buckling

#### 4.4 The influence of stud spacing

Studs are generally spaced between 16 in. to 24 in. on center. The influence of stud spacing on shear wall's shear capacity has always been one of the major concerns by researchers and engineers. Two shear wall models with 16 in. and 24 in. stud spacing were created and compared. The load vs. displacement curves are shown in Figure 18, from which we can see the bearing capacity of shear wall with 16 in. stud spacing was much higher than shear wall with 24 in. stud spacing. The reason for this improvement is that the number of screws between the sheathing and the frame has increased with the closer stud spacing. Besides, the displacement at failure point of shear wall with 16 in. stud spacing postponed from 2.329 in. to 3.193 in. According to ASCE 7-16 (2016) and IBC-15 (2015), the allowable story drift is 1/40 of the story height. The shear strength was calculated consequently and summarized in Table 4. The results indicated that the shear strength of shear wall with 16 in. stud spacing was 11% higher than shear wall with 24 in. stud spacing.

In conclusion, the shear capacity of the corrugated steel sheathed shear wall can be greatly improved with smaller stud spacing. It is suggested that stud spacing should be no more than 24 in. in building systems.

Table 4: The influence of stud spacing

Stud spacing	Pmax (kips)	Ratio	$\Delta_{max}$ (in.)	Ratio
24 in.	18.16	-	2.328	-
16 in.	20.14	1.11	2.4	1.03

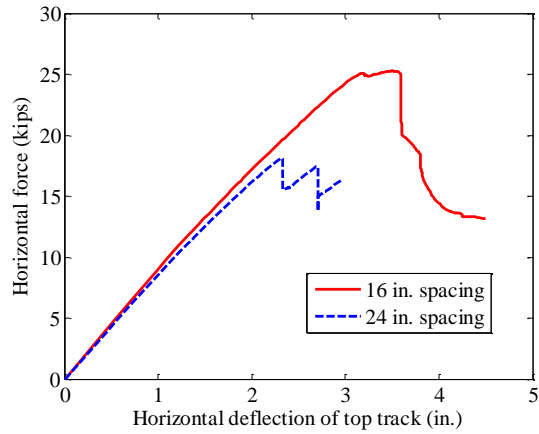


Figure 18: Load deformation responses

#### 4.5 The influence of gravity/vertical load

In actual light-frame CFS buildings, the shear walls are usually subjected to both lateral and gravity (vertical) loads. The influence of gravity load needs to be investigated. Two shear wall models were created and compared in this paper. The gravity load was applied by a uniform pressure in Abaqus. The gravity load was estimated from a typical 2-story office building in the Network for Earthquake Engineering Simulation (NEES)-CFS project (Madsen et al. 2011). The gravity load was calculated as the sum of the dead load and 25% of the live load. The load vs. displacement curves are shown in Figure 19, from which we can see these two curves almost coincide with each other. In conclusion: since the gravity (vertical) loads of low-rise cold-formed thin-wall steel structure is limited, the influence of vertical loads on shear capacity can be neglected. The reason for this could be that the gravity (vertical) loads in such low-rise CFS buildings is limited. As a result, the influence of gravity (vertical) loads on the shear capacity of shear walls can be neglected.

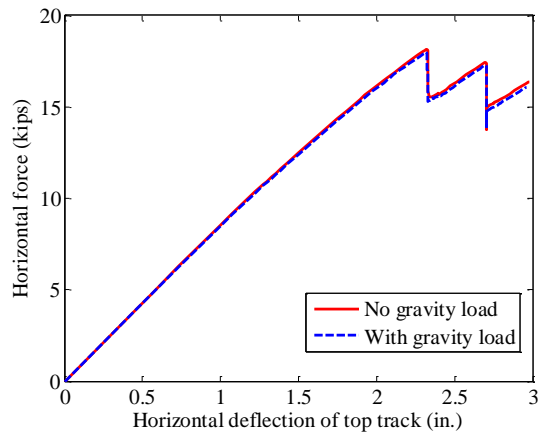


Figure 19: Load deformation responses

## 5. Conclusions

Finite element model of cold-formed steel framed shear walls sheathed by corrugated steel sheets was created in Abaqus software and validated based on previous test results. The agreement of the FEA results and test result indicated the accuracy of the proposed numerical model. A series of parametric analysis were then conducted, including thickness of framing members, cross section of stud members, yield strength of the frame members, the stud spacing, and the influence of gravity loads. Conclusions based on parametric analysis results were made and recommendations for practical applications of this shear resistance system were proposed.

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