SHEAR AND BENDING STRENGTH OF COLD-FORMED STEEL SOLID WALL PANELS USING CORRUGATED STEEL SHEETS FOR MOBILE SHELTERS

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The objective of this thesis is to determine if the single sided resistance spot weld (RSW) can be used as a feasible connection method for cold formed steel (CFS) shear walls subject to lateral force of either seismic or wind loads on mobile shelters. The research consisted of three phases which include: a design as a 3D BIM model, connection tests of the resistance spot weld, and full-scale testing of the designed solid wall panels. The shear wall testing was conducted on specimens with both resistance spot weld and self-drilling screws and the results from tests gave a direct comparison of these connections when the solid wall panel was subjected to in-plane shear forces. The full-scale tests also included 4-point bending tests which was designed to investigate the wall panel's resistance to the lateral loads applied perpendicularly to the surface. The research discovered that the singled sided resistance spot weld achieved similar performance as the self-drilling screws in the applications of CFS wall panels for mobile shelters. The proposed single sided resistance spot weld has advantages of low cost, no added weight, fast fabrication, and it is a feasible connection method for CFS wall panels.

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CHAPTER 1

INTRODUCTION

Cold-formed steel (CFS) members are made of flat plates that are bent and rolled into various shapes at room temperature then galvanized. The galvanizing process makes CFS resistive to atmospheric corrosion. CFS members are widely used in the construction of mid-rise residential and commercial building because of its many advantages. Some of which are its light weight, high-strength, and its ability to be erected easily. Plus, the dimensional stability, durable and non-combustible material is insect resistant and energy efficient. Most importantly this is a recyclable material. For this reason, CFS was chosen for the design of a new Army Standard Family of Rigid Wall Shelters (ASF-RWS) project. The main goal for this ASF-RWS project is to build a lighter weight structure that yields the high strength possible. Compared to the current design of the ASF-RWS which is composed mostly of aluminum framing materials.

The Army Standard Family of Rigid Wall Shelters ASF-RWS are 8ft x 8ft x 19ft 11in. modular shelters used by the US Department of the Army during combat operations. Different types of shelter exist which can be used as field kitchens, maintenance shops, command post or hospital operating room. The main goal of the overall research done here at the University of North Texas was to build a prototype for a new and improved version of these collapsible and expandable types of rigid wall shelters. The challenge for the CFS lab at the University of North Texas was to design a shelter that is expandable and be assembled quickly from modular a 4ft x 8ft walls to form the large 8ft x 8ft x 19ft 11in. structure. Then the larger 8ft x 8ft x 19ft 11in. structure can be pieced together to create one large multipurpose complex which will be lighter and stronger than the original version.

This thesis has three different types of tests that were conducted on each 4ft x 8ft wall which included a connection, shear wall, and transverse load tests (bending test). A connection test is a standardized test which two CFS plate are pulled by opposite repelling forces in order find the ultimate strength of the connections or fasteners holding the plates together. The second test is a shear wall test and will undergo both monotonic and cyclic lateral load testing to determine the shear strength of the designed CFS wall. The third type of test was a bending test to determine the uniform load density for each tested wall. For the ASF-RWS project the choice of a non-penetrating connection; such as spot welds, was made in order to lengthen the life of the RWS from any form of excessive corrosion. It was decided that spot welds would be a better choice than self-drilling screws for this project. In order to confirm that spot welds are comparable to the known self-drilling screws, various comparison tests were conducted.

CHAPTER 2

LITERATURE REVIEW

The study of CFS shear walls with corrugated steel sheathing was started by Dubina and Fulop (2004). Fulop and Dubina studied a series of full-scale tests on shear walls with different sheathing materials which included corrugated steel sheets. The same test framing was used during all of the wall testing. Dubina and Fulop (2004) concluded that the CFS walls were rigid and capable of resisting lateral loading. It was discovered that the seam fasteners were the failure mechanism for corrugated sheet specimens.

Adding to the literature was Stojadinavic and Tipping (2007) who conducted a series of 44 cyclic tests on CFS shear walls with corrugated steel sheathing. They tested six design configurations including different thicknesses of corrugated sheet steel and frame members. The type, size, and spacing of fasteners were also tested. Stojadinavic and Tipping reported that in all the tests, the failure mode observed was of screw pull out due to the warping of the corrugated steel sheets.

The design of inside the frame (Sheet In) CFS corrugated sheet shear wall used is this research was first proposed by Mahdavian (2016). In Mahdavian (2016) several shear wall tests were conducted to explore the structural performance in both outside the frame (Sheet Out) and sheet in configurations. The findings from these tests concluded that the sheet in configuration was a reliable design for practical usages in the construction industry. One of the main advantage in using sheet in wall designs is the ability to add finished material to the outside of the wall without adding thickness to the wall. This allows for a flush exterior which makes CFS wall a feasible option for ASF-RWS project.

CHAPTER 3

CONCEPT DESIGN AND BUILDING INFORMATION MODELING (BIM)

3.1 Concept Design

In the conceptual design for the ASF-RWS project, it was decided that CFS was going to be the main material used for the structure of the mobile shelter. CFS will reduce the overall weight and increase the interior volume of the shelter. The material in the earlier reversion of the ASF- RWS internal structure using aluminum as its base material. Mahdavian (2016) discovered the combination of CFS framing and sheet in configuration of corrugated sheathing produced higher yields of strength in lateral loading. For this reason, it was determined that a sheet-in configuration would be best for this design. The term sheet-in is a way to describe how a corrugated sheet is placed within a CFS frame. For this design, two CFS tracks were placed back to back as vertical support members, then a sheet of corrugation steel was placed between the vertical members to create a wall. A cross section view of a sheet-in wall can be seen in Figure 3.1. One of the main reasons this type of framing was used was that the thinner walls would create a space inside the wall for insulation, wiring, or piping to be installed later.

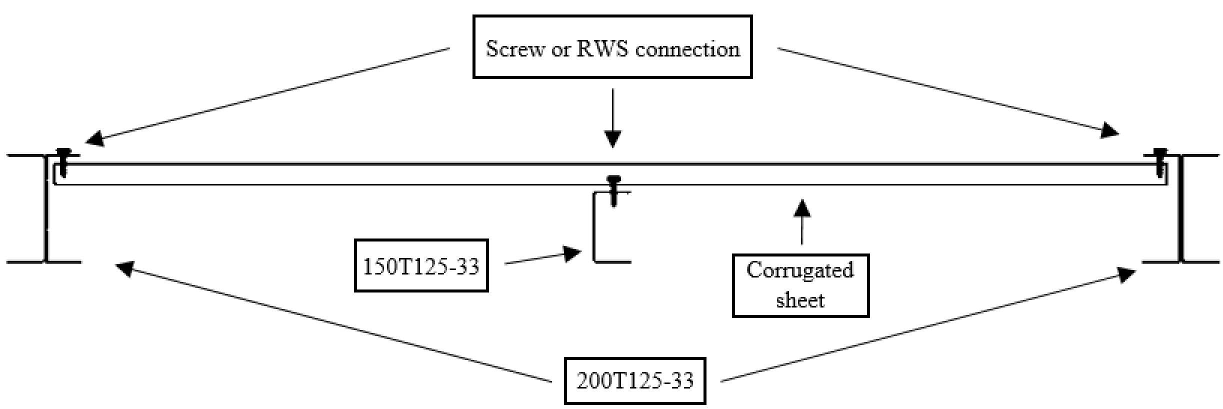


Figure 3.1: Sheet in Cross-Section View

For the design of 8ft x 8ft x 19ft 11in. mobile shelter one of the requirements in the US Department of the Army (2013) was that the mobile structure be collapsible. The testing for this research was conducted on 4ft x 8ft wall sections in order to demonstrate the individual unit strength. To provide more rigid strength to each wall a stud will be placed between each vertical track member given a spacing of 24 inches. Once it was decided that the sheet-in design was going to be the bases of the new version of the ASF-RWS a full-scale model was going to be made but due to time and material a computer generate model was used.

3.2 Building Information Modeling

The first full-scale design of the new ASF-RWS was made in Autodesk Revit 2017. Building Information Modeling or BIM is a tool used in the construction industry to present a 3D representation model of a project with information integrated and attached to the model. Valuable information can be stored in each model which includes schedules, material identification and their quantities to help with the management of a construction project. The full model of the ASF-RWS design can be seen in Figure 3.2:

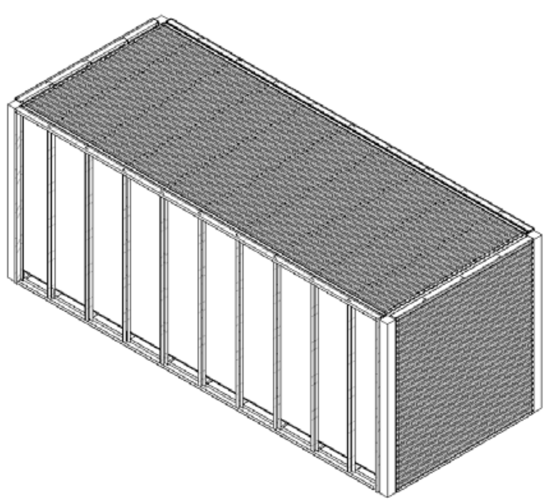


Figure 3.2: Full RWS Model in Autodesk Revit

For the new BIM design for the ASF-RWS several new CFS members needed to be added to the Families database. From research done by Johnson, A., Ramirez, R. and Yu, C. (2016) many of the members needed for the design of RWS were already made. However, several dimensions were added, adjusted, and changed to complete the model. The main goal of the BIM model for the ASF-RWS project was getting the dimensions correct in order to get a better idea of the space and weight requirements for the overall project design. The first step was changing the dimensions of each member within the model. This required the use of the already made CFS family members on file and adjusting the dimensions in the edit type function in the member's properties tab. Under the dimension section the values for D and B can be found. These values are the length of the inter web and flanges of each member. These value can be seen in Figure 3.3 and Figure 3.4 located in the edit- type- tab section within the Revit program. This is where the dimensions of B and D can be adjusted. These adjustment was needed on the member 150T125-33, 200T125 and 225T125 to show the correct dimension for the model.

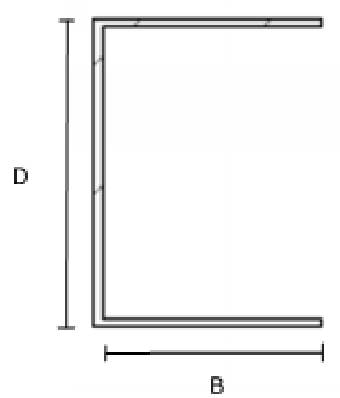


Figure 3.3: Dimension Locations of CFS Member

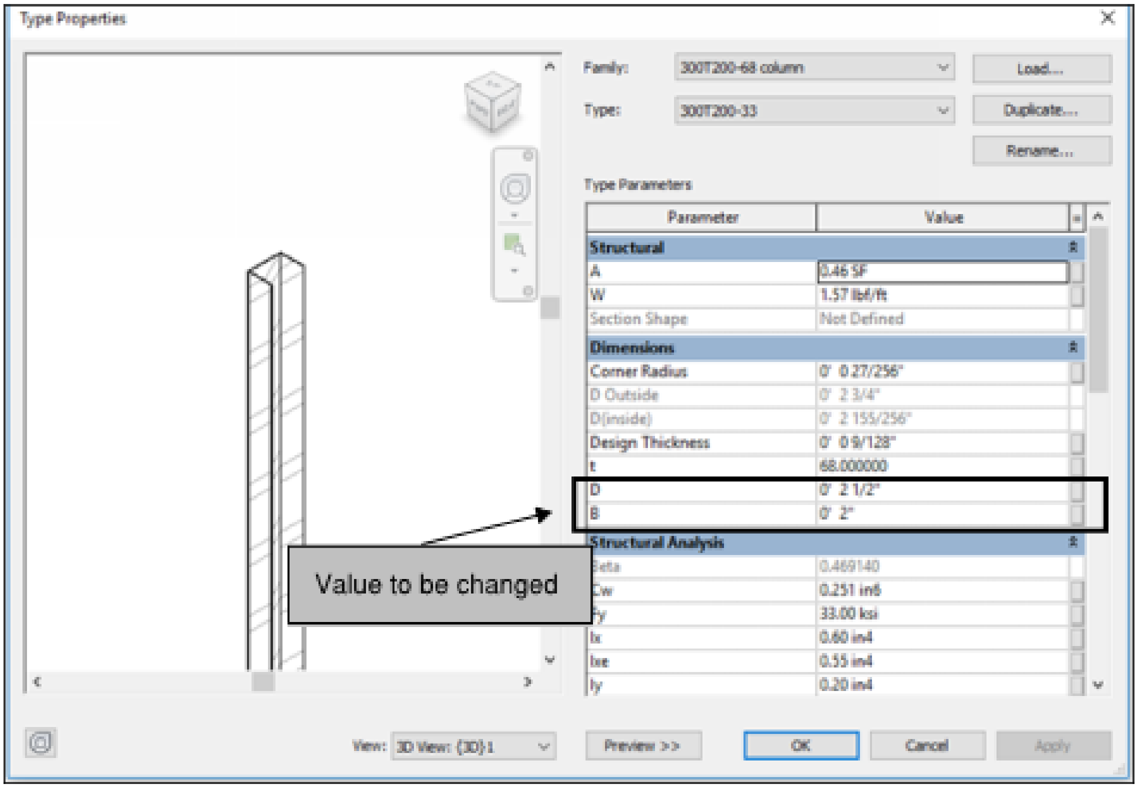
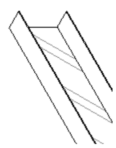
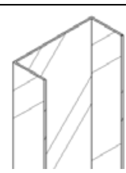
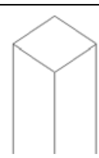
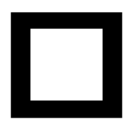
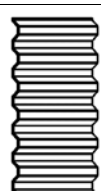


Figure 3.4: Location on Type of Properties Type

The corner posts will be the main support of the ASF-RWS system. In order to stack the shelters on top of each other, which is one of the requirement set out by the Department of the Army, the concentration of force will be focused on the four corners of the shelter. For this reason, each corner was fitted with a HSS 4 in. x 4 in. structural steel member to hold the compressive force of the added weight. The Revit model has a solid generic member with the dimension of 4 in. x 4 in. as a place holder until future research can determine the best solution for reducing weight but keeping the structural integrity. Therefore, there was no need to adjust or place an already designed HSS member. The last component that was added to the design model was corrugated steel sheets. The corrugated sheets were already a family but like the other CFS members there were several adjustments that needed to be made. The model has two different dimensions for the height and width of the corrugated sheets determined by which wall it was inserted into. For the longer span of the wall a continuous corrugated sheet with the dimension of

19 ft 11 ½ in. x 7 ft 11 ½ in. was inserted within the CFS framing and for the smaller walls a 7 ft 11 ½ in. x 7 ft 11 ½ in. was placed. The smaller dimensions were set in order for each corrugated sheet to fit within the framing which is a sheet-in configuration. Table 3.1 shows all dimensions that were adjusted in the Revit model.

Table 3.1: CFS Member Dimensions

Member Type	3D model	2D model	Dimensions
150T125-33			D = 0’ 1.5” B = 0’ 1.25”
250T125-33			D = 0’ 2.5” B = 0’ 1.25”
200T125-33			D = 0’ 2” B = 0’ 1.5”
HSS 4” X 4”			Height = 4” Width = 4”
Corrugated sheet (28 GA and 26 GA)			Roof/Side Walls: 19’ 11 ½” x 7’ 11 ½” End Walls: 7’ 11 ½” x 7’ 11 ½”

CHAPTER 4

CONNECTION TEST

4.1 Introduction to Resistance Spot Welding

Resistance spot weld (RSW) is a welding process which uses pressure without the use of any type of filler material unlike arc welding. In order to produce the best possible RSW the concept of how these type of welds are created must be looked at. RSW are based on four basic principles which are electric current, time, pressure and heat energy. The base of using these four principles are rooted in the basic concept of the Ohm's law which can be seen in the Equation 4.1:

Equation 4.1 Ohm's law

$$I = \frac{E}{R}$$

I = current in amperes

E= voltage in volts

R= resistance of the material in ohms

The principle of Ohm's law states that voltage and current are directly proportional and the proportional constant is the resistances. In CFS to insure that a strong weld will occur a force needs to be applied before, during and after the welding process. A steady amount of pressure was required throughout the welding cycle to ensure a constant electrical circuit. The amount of time and current used directly effects the heat produced to overcome any heat loss. This will insure that the heat will raise the temperature of the material to a sufficient welding temperature. (Cary 2002) The total energy is expressed by the formula for heat energy which is provided below (Equation 4.2):

Equation 4.2 Total Heat Energy

$$H = I^2 x\ R\ x\ T\ x\ K$$

I = current in amperes

R= resistance of the material in ohms

T = time of current flow in seconds

H = heat energy in joules

K= heat losses through radiation and conduction in joules

4.2 Test Set-up

Testing of a spot weld connection on a 4 ft x 8 ft wall was the main goal for the connection test. The Resistance Spot Weld (RSW) was made with an adjustable duel tip handheld spot welder made by Lors Machinery INC. model number 4010 with TECNA model 3935 transformer. There are two adjustable setting on the transformer which include one for controlling the amount of volts through the welder and can be set between 1 and 10 volts. The second setting is an adjustment of time for the welder to engage during the actual weld. The adjusting ranges from 2 to 65 cycles. Once the settings are adjusted a downward force must be applied to the back of the handheld device to create pressure and a good connection between the two pieces of steel. Once the bonding weld is produced at the specified current and time then the downward force can be removed. In Figure 4.1 a handheld device and transformer can be seen.

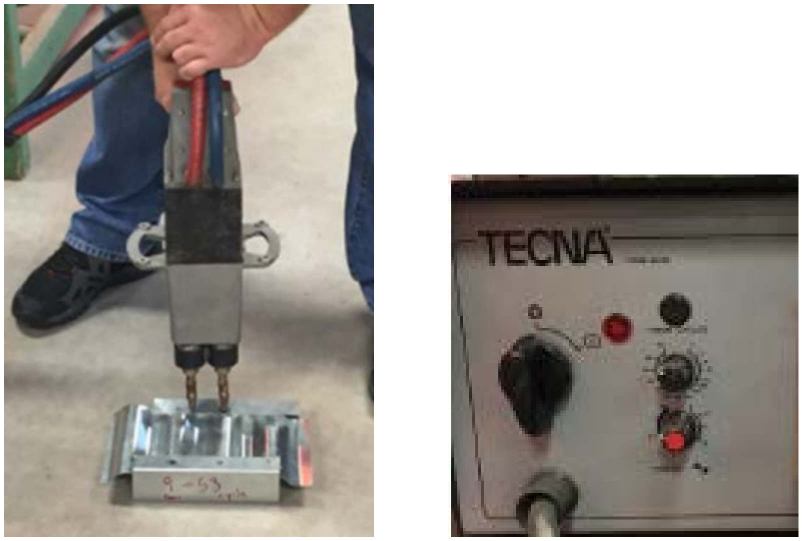


Figure 4.1: RSW Hand Held device and Transformer

Specimens for this test were made by cutting a strip of 33 mil CFS material from a 150 T 125– 33 CFS track. The second material consisted of two different types of thickness of CFS, one being 26 gauge and the second being 28 gauge CFS corrugated sheets. A single spot weld was then created to connect the two strips of metal together. Each test specimen was tested in an INSTRON 4482 Universal Testing Machine and an INSTRON 2630-106 extensometer was used to measure the tensile strain. The set up can be seen in Figure 4.2.

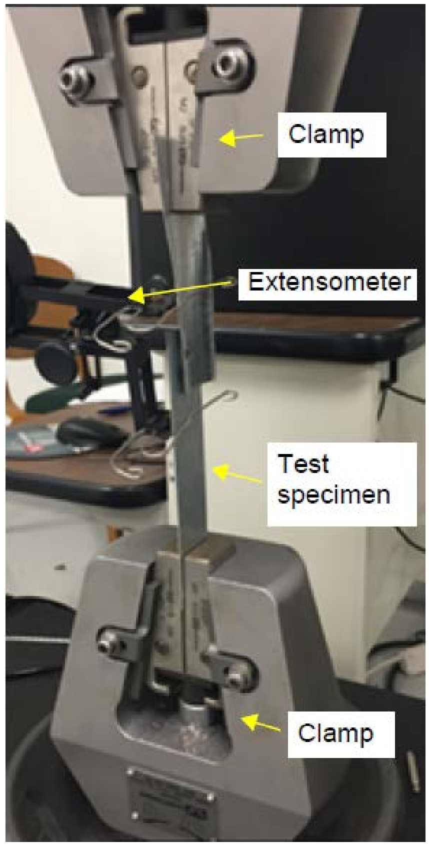


Figure 4.2: Instron Universal Testing Machine Set-up

4.3 Test Results

Two different tests were conducted on each setting to find the optimum spot welded strength to be used for later testing of 4 ft x 8 ft CFS walls during the shear and bending testing. A third test was also conducted to confirm the highest tensile strength on the 9v 53 cycles and 9v 50 cycles spot welds. The results are shown in Table 4.1 where each setting is represented with corresponding max tensile strength.

Table 4.1: Spot Weld Connection Test Results

Spot Weld Connection Test				
Volts	Cycles	Steel Gauge	Test No.	Max Tensile Load (lbs)
9	53	26	1	626
9	53	26	2	852

9	53	28	1	713
9	53	28	2	567
9	53	28	3	695
8	50	26	1	810
8	50	26	2	548
8	50	28	1	652
8	50	28	2	583
9	50	28	1	680
9	50	28	2	367
9	50	26	1	873
9	50	26	2	859
9	50	26	3	908

Each yielded strength was found by a strength over displacement curve. After the data was processed through the computer program MatLab the highest tension strength was compiled for each test specimen and found the best setting for the RWS. Figure 4.3 shows the load vs deflection curve on Test #1 of the spot weld setting at 9v 53 cycles.

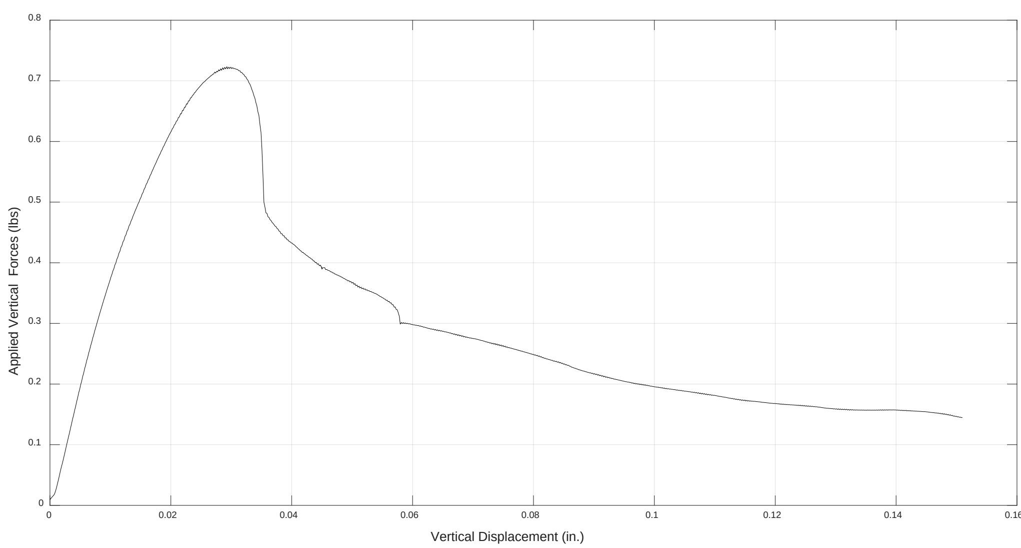


Figure 4.3: Spot Weld Test Results

The spot welds on the test specimens for all tests had a dominant failure mode of a shear material bearing build-up without a full material disconnection. A typical test fail can be seen in Figure 4.4 and Figure 4.5, which show the failure of 9v 53 cycle T1 test sample at 713.17 lbs.

Figure 4.4: Test Specimen before Test

Figure 4.5: Test Specimen after Test

4.4 Connection Test Conclusion

After reviewing the data for the spot weld connections it was determine that for the 26 gauge corrugated sheets the strongest connections were at 9 volts with a rate of 50 cycles. The data also shows that for a 28 gauge corrugated sheet 9 volts with a rate of 53 cycles was the optimal setting. These connections will be applied in the construction of 4 ft x 8 ft walls for shear and bending tests.

CHAPTER 5

SHEAR TEST

5.1 Test Set Up

The shear wall tests were conducted on a 4 ft x 8 ft CFS wall. Two different types of tests (cyclic and monotonic) were performed on a 16-ft. span, 13-ft. high adapted structural steel test frame. All the shear wall specimens were assembled in a horizontal position and then installed vertically into the test frame. The wall was bolted to the base beam and loaded horizontally at the top. The base beam was a HSS 5 in. x 5 in. x ½ in. structural steel tubing that was attached to a W 16 x 67 structural steel beam. Then the whole structure was attached to the concrete floor slab with ¾ in. anchor bolts at 24 in. on center. The web of the HSS 5 in. x 5 in. x ½ in. base beam has cut-outs in several locations on one side to provide access for shear walls anchor bolts. A T bar was attached to the top track of the shear wall by 20- No. 12 x 1 ¼ in. hex washer head (HWH) with self-drilling tapping screws placed every 3 inches on center. The out-of-plane displacement of the wall was prevented by a series of steel rollers on each of the side of the T bar. A gap of approximately 1/8 inch was provided between the roller and the T bar to avoid significant friction during the testing process. The anchor system for the cyclic tests consisted of ASTM A307 ¾ in. diameter shear anchor bolts with standard cut washers (ASME B18.22.1 (1998)) and one designed hold down were placed at each bottom end of the shear wall. The anchor system also includes two ATSM A490 ¾ in. diameter shear bolts with standard cut washers (ASME B18.22.1 (1998)) that were placed equal distances apart between the hold downs on the bottom track. To accurately measure the displacement there were five NOVOTECHNIC 10 transducers placed around the wall as shown in Figure 5.1. A MTS 35 kip hydraulic actuator with a 10 inch stroke was used to apply horizontal force to the wall. The force was measured by the 30 kip TRANSDUCER TECHNIQUES SWO universal compression/tension load cell. The testing set up can be seen in Figure 5.1:

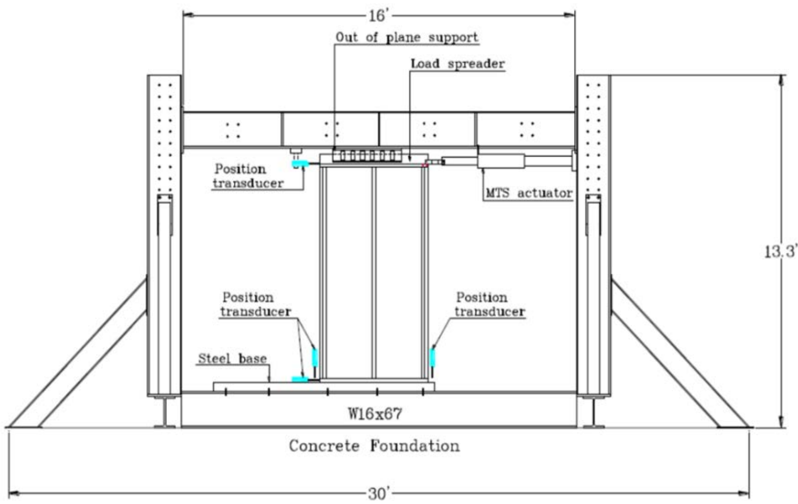


Figure 5.1: Shear Wall Test Set-up

5.2 Test Method

For this research both monotonic and cyclic loading tests were conducted. Monotonic testing was in accordance with ASTM E564 (2012) "Standard Practice for Static Load Test for Shear Resistance of Framed Walls for Buildings". The rate of displacement was at a uniform rate of 0.0075 in/sec. The cyclic tests referred to CUREE protocol with 0.2 Hz (5 seconds) loading frequency which was in accordance with Method C in ASTM E2126 (2012) "Standard Test Methods for Cyclic (Reversed) Load Test for Shear Resistance of Vertical Elements of the Lateral Force Resisting Systems for Buildings." A 40 cycle of displacement for each test was used as in the standards of CUREE testing procedures. Cyclic testing was done on four walls in order to determine the shear strength of the shear wall during seismic loads or other dynamic loads. Monotonic testing is used in determining the shear strength for wind loads or other static loads.

5.3 Test Specimens

The walls that were used in this research project were 4 ft x 8 ft framed with sheet-in corrugated sheathing. In order to place the corrugated sheet within the boundaries of the framing wall a CFS track was vertically placed instead of S designed members. The design of the wall framing consists of two 8 ft - 200T125-33 tracks placed back to back then connected with two (2)- # 12 x 1 ¼" hex washer head (HWH) self-drilling tapping screws which were set parallel to each other then placed every 6 inches for the length of the 200T125-33 member. The last set of screws were not placed on the bottom of member in order to leave space for the placement of the wall hold-downs. An identical member was also made with the same specifications to complete the vertical framing boundary for the wall. The top and bottom cords of the wall framing were a 4 ft 225T125-33 track member placed and connected with a #8 x ¾" modified truss head (MTH) self-drilling screws. A CFS stud was then placed in the center of the framed wall at a spacing of 24 inches and was also attach to the top and bottom cords with a #8 x ¾" MTH screws. This is the standard configuration for all the tested walls but the different variations of the corrugated sheet were tested. The corrugation sheets are Vulcraft SV36 in two different thickness which are 26 gauge (16 mil) and 28 gauge (13 mil) with the yield strength of 60 ksi. Figure 5.2 shows how each CFS member was identified for this research.

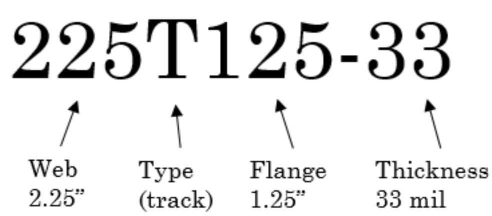


Figure 5.2: Member Identification

Wall configuration for ASF-RWS project consists of several variations between two different types of thicknesses of corrugated sheets and two different types of connections. The corrugated sheets have a thicknesses of 28 ga (13 mil) and 26 ga (16 mil) which both are set as a sheet-in configuration. The sheet to frame connection consists of either RSW or #8 x ¾” MTH self-drilling screws. There were 7 tests run to determine which thickness and connection will provide the optimal results during cyclic and monotonic testing. In Table 5.1 the test matrix for the shear wall tests and the shear test details can be found in Appendix A.

Table 5.1: Shear Test Matrix

Test #	Test Frame	Thickness of Corrugation	Type of Connection	Type of Test
1	4 x 8 -24	26 ga	#8 x ¾” MTH self - drilling screw	Cyclic
2	4 x 8 -24	28 ga	#8 x ¾” MTH self - drilling screw	Cyclic
2b	4 x 8 -24	28 ga	#8 x ¾” MTH self - drilling screw	Cyclic
2c	4 x 8 -24	28 ga	#8 x ¾” MTH self - drilling screw	Cyclic
3	4 x 8 -24	26 ga	#8 x ¾” MTH self - drilling screw	Monotonic
4	4 x 8 -24	26 ga	Resistance Spot Weld	Monotonic
5	4 x 8 -24	28 ga	Resistance Spot Weld	Monotonic

5.3.1 Material Properties

Coupon tests of each member were conducted according to the ASTM A370 (2006) “Standard Test Methods and Definitions for Mechanical Testing of Steel Products” to obtain the actual properties of the test materials in this project. The coating of the steel samples was removed by hydrochloric wash prior to coupon tests. The tensile test of coupons were conducted on an INSTRON 4482 Universal Testing Machine and an INSTRON 2630-106 extensometer was used to measure the tensile strain. The coupon tests were conducted in displacement control mode at a constant tension rate of 0.05 in./min. Three tests were made of each member sample then the totals from each were then averaged. The result are provided in Table 5.2.

Table 5.2: Material Properties

Member	Uncoated Thickness (in)	Yield Stress Fy (ksi)	Tensile Strength Fu (ksi)
150S125-33	0.0351	45.59	55.38
250T125-33	0.0349	45.58	54.38
200T125-33	0.0346	44.91	54.92
0.6C26	0.0179	108.85	106.84
0.6C28	0.0157	102.85	101.12

5.3.2 Designed Hold-Downs

Due to the test specimen thickness of two inches a standard hold-down was not used. Two version of the hold-downs were designed for the seven test conducted. The first version of the hold-down was made from bending a 1 ¾ in. x 8 ½ in. CFS plate with the thickness of 118 mil (10 ga) into a 90 degree angle. Next 10 - ¼ in holes were drilled into the upper portion of the right angle. This was to allow the # 12 x 1 ¼” hex washer head (HWH) self-drilling tapping screws to be placed and attaching the hold-down to the test specimen. Connecting the lower and upper portions to each other was a 1 ¾ in. x 1 ¾ in. right triangle which was then welded to provide strength to the hold-down. Lastly, a single 5/8 in. hole was drilled in the lower portion of the hold-down to allow the shear anchor bolts secure the test specimen to the testing frame. The first version of the hold-down can be seen in Figure 5.3. The second version of the hold-down was needed to be made after a total failure in the first version of the hold-down which occurred in Test 2. The second version of the hold- down was made from a hot-rolled angle iron with the dimension of 6 in. x 6 in. with the thickness 5/16 in. The lower portion of the angle iron was cut to 3 ¾ in. giving the angle iron a total dimension of 6 in. x 3 ¾ in – 5/16 in. The upper portion of the angle 12- ¼ in holes were drilled to allow the attachment of the hold-down to the test specimen by the # 12 x 1 ¼” hex washer head (HWH) self-drilling tapping screws. A single 5/8 in. hole was drilled in the lower portion of the angle and two 2 ½ in x 3 in. triangle with the thickness of ¼ in. was cut and then welded to the angle iron piece to create the second hold-down. The second version of the hold-down can be seen in Figure 5.4.

Figure 5.3: First Version of Hold-down (unfinished)

Figure 5.4: Second Version of the Hold-down

5.4 Test Results

From the raw data produced during the cyclic and monotonic testing, values of ductility factor and initial stiffness were obtained from the raw data. These values were used in order to compare results between RSW and screw connections in determining if spot welds can be a viable option in the design of the ASF-RWS. Although a standard value of a ductility factor for the ASF-RWS is not required the comparison between max yielding strength, ductility, and stiffness was an acceptable benchmark to compare to. The ductility factor is defined by the Equivalent Energy Elastic Plastic (EEEP) concept which first was proposed by Park (1989) and can be seen in Figure 12 is the EEEP concept used to find the ductility on test 3:

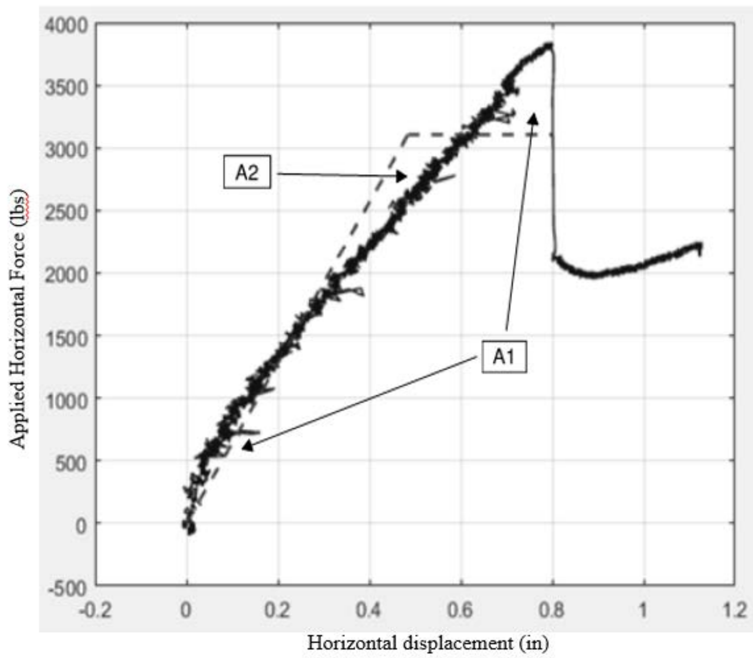


Figure 5.5: EEEP Curve of Test 3

To calculate the ductility factor an equation to find the ratio between ultimate displacements over the maximum elastic displacement must be used. The bases of calculating ductility factor, can also as be seen in Figure 5.5, after the result curve is plotted for the test then a dotted line plotted will make the area of A1 equal to the area of A2. This equation can be seen in Equation 5.1.

Equation 5.1 Ductility Factor

$$\mu = \frac{\Delta u}{\Delta y}$$

where:

μ = Ductility Factor

Δu = ultimate displacement

Δy = maximum elastic displacement

The raw data for the shear wall test was also entered into the computer program MatLab in order to analyze the graph output for determining the stiffness of each wall. The max peak load was determined from the graph made by the program. Next, the max peak load was used to obtain the 40% location of both load and displacement to determine the stiffness of each wall. Using the following Equation 5.2 the stiffness or K value was found.

Equation 5.2 Stiffness

$$K = \frac{.4P}{\Delta}$$

where:

K = Initial Stiffness

P = Peak Load

Δ = Displacement at 40% Peak Load

5.4.1 Discussion of Cyclic Loading Failures

Three shear walls were tested under cyclic loading. Test 1 showed several screw pull-outs along the bottom track which held the corrugated sheet in place. The final failure of the wall was found at both wall hold-downs. Buckling and tearing of the framing member happen just above each hold-down. Inconclusive results were found during test 2 because of an early failure in the designed hold-down itself. The ASTM A307 ¾” diameter shear anchor bolts was able to pull through the designed hold-down on the left side, because of this failure new hold downs were designed for the remaining tests. The new design changed the material of the hold-downs from the CFS with a thickness of 118 mil to ¼” thick hot-rolled steel. The failure for the retest or test 2b caused a new problem. The top cord track that was attached to the T bar tore away from the vertical framing members leading to an early failure in the test. The final test of cyclic loading was Test 2c, the failure in this test was that both vertical track members tore just above the designed hold-down. These results mimic the results found in Test 1. There was no failure in the connection only in the framing of the wall.

5.4.2 Discussion of Monotonic Loading Failures

There were a total of three monotonic loading tests, one using MTH screws and two tests using RSW as connections. The first monotonic loading test was Test 3, the failure in this test consisted of local buckling at the compression side hold-down with no connection failures. Test 4 showed the same type failure, local buckling at the compression side hold-down with no connection failure either. Since the object of these test was to achieve a connection failures out of the shear wall. For Test 5, the final test, extra bracing along the outside vertical framing members were added. The purpose of adding extra bracing was to add strengthen to the wall frame and get actual connection failure of RSW connections. As predicted, there were several connection failures with corrugated sheet tearing.

The results of each wall test under both monotonic and cyclic loading is provided in Tables 5.3 and 5.4.

Table 5.3: Shear Test Results

Test #	Test Frame	Thickness of Corrugation (ga)	Type of Connection	Type of Test	Average Peak Load (lbs)	Average Displacement at Peak Load (in)	Ductility Factor	Initial Stiffness K (lbs/in)
1	4' x 8' -24	26	screw	Cyclic	3929	0.89	3.61	4291
2	4' x 8' -24	28	screw	Cyclic	3860	1.52	2.54	4423

Test #	Test Frame	Thickness of Corrugation (ga)	Type of Connection	Type of Test	Average Peak Load (lbs)	Average Displacement at Peak Load (in)	Ductility Factor	Initial Stiffness K (lbs/in)
2b	4' x 8' -24	28	screw	Cyclic	3513	1.47	2.74	3713
2c	4' x 8' -24	28	screw	Cyclic	4062	0.95	3.89	5787
3	4' x 8' -24	26	screw	Mono	3845	0.79	1.65	6274
4	4' x 8' -24	26	Spot Weld	Mono	4061	0.91	1.77	4641
5	4' x 8' -24	28	Spot Weld	Mono	3871	0.84	2.40	6194

Table 5.4:Shear Test Failure Modes

Test #	Observed Failure Mode
1	Screw pull out along bottom track and sheet connection, sheet tearing, chord and middle track local buckling, buckling and tear at hold-downs
2	Hold-down and washer failure early in test, caused a bad test.
2b	Top track tore from frame and sheathing. Crippling and buckling of 200T, and 150T at top. Hold-down pulled away from frame at the very bottom.
2c	Tearing of outside studs on both sides above hold-downs. Middle stud buckling at bottom
3	Compression side buckling above hold-down
4	Compression side buckling above hold-down
5	Spot Weld Failures multiple areas, Sheathing Tears

5.5 Test Conclusion

After a total of 7 tests were conducted in both monotonic and cyclic loading the results of the test was determined that failures in all but one test occurred in the framing of each wall. It was only with the addition of extra bracing to the wall that a failure in connection occurred. For Test 1 and Test 2c two different thicknesses of corrugated sheathing were tested and from the results it was found by adding a thicker corrugated sheet there was a 25% increase in initial stiffness and a 7% increase in the ductility factor. The last three tests were conducted under monotonic loading with Test 3 having MTH connection and Test 4 and 5 consisting of RSW connection. The results between tests 3 and 4 shows a 6% increase in ductility factor but a 26% decrease in stiffness. Test 5 was the only test that showed failure in the spot weld connections due to the addition of extra bracing. The ductility factor and stiffness for this test showed increases compared to the other two monotonic loading tests. These increases were caused by the extra bracing rather than the spot weld connections.

CHAPTER 6

BENDING TEST

6.1 Test Set Up

To determine how a wall section of the designed ASF-RWS would react to wind loads acting on the wall surface a bending test program was conducted. The bending tests setup and procedure for this test followed ASTM E72-15 “Standard Test Methods of Conducting Strength Tests of Panels for Building Construction.” The wall specimen was simply supported at both ends by steel roller with a 4-in. wide steel plate welded to the top of the roller which allowed a better surface area for the wall to be placed on. At the two loading locations on top of the placed wall the same designed steel rollers with a 4-in. wide steel plate was placed 25 inches from each edge of the wall. A steel I beam was used to apply two equal loads to the loading rollers. The Steel I beam was then attached to a 30-kip hydraulic cylinder with an 8-in. stroke and a 20-Gpm MTS hydraulic power unit to support the loading system. A 30-kip TRANSDUCER TECHNIQUES SWO universal compression/tension load cell was placed to connect the hydraulic cylinder to the load beam. Two NOVOTECHNIC position transducers were placed on each side of the wall specimen and at the center point to measure the vertical displacement of the center of the wall specimen and the vertical displacement of load beam. The data was collected by a system consisting of a National Instruments unit (including a PCI6225 DAQ card, a SCXI1100 chassis with SCXI1520 load cell sensor module and SCXI1540 LVDT input module) and a desktop PC. The applied force and the displacements were measured and recorded instantaneously during the test. The test can be seen in Figures 6.1 and 6.2:

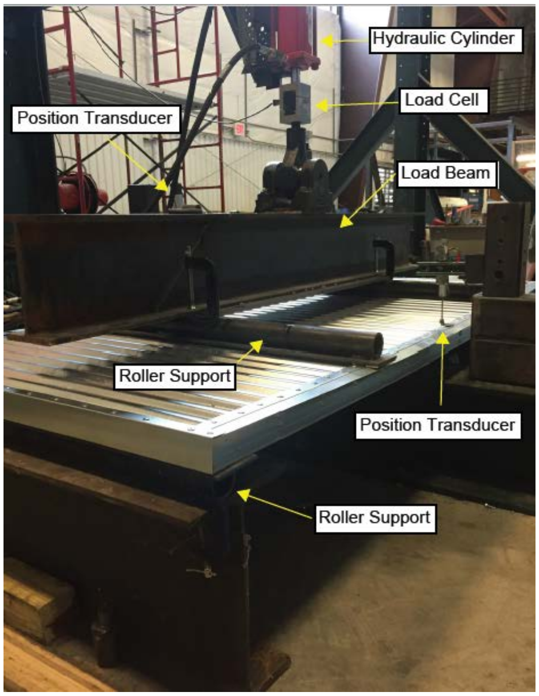


Figure 6.1: Bending Test Set-up

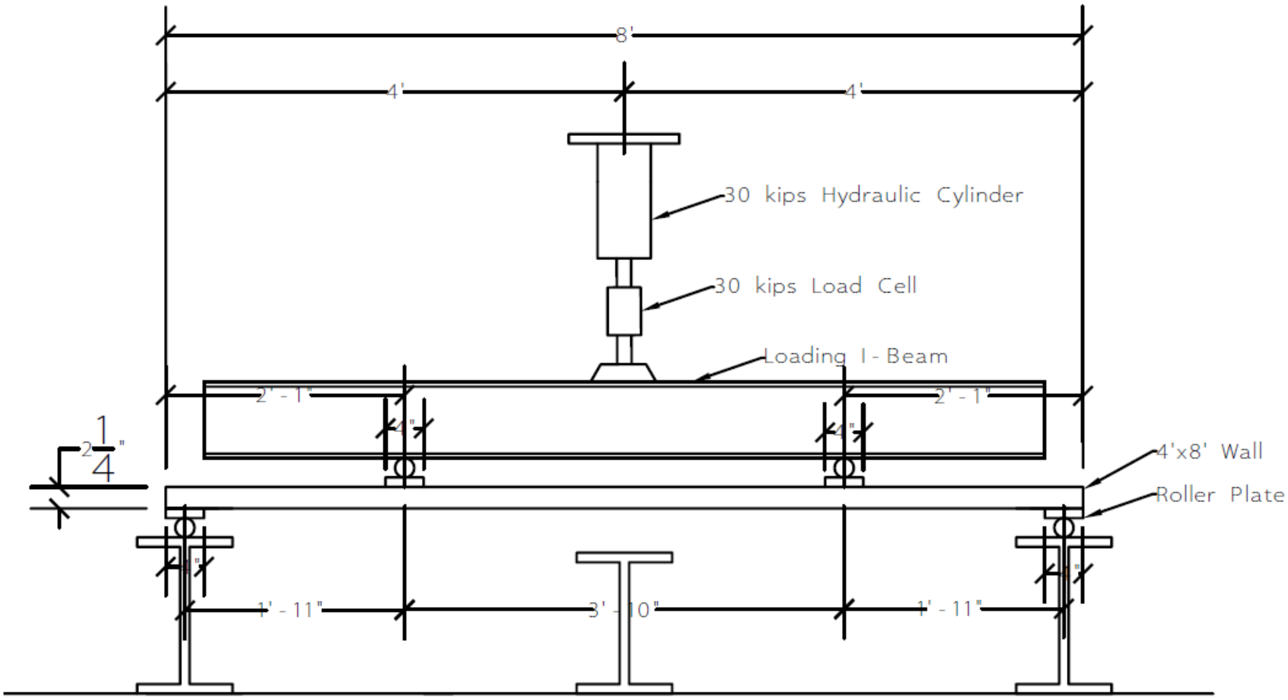


Figure 6.2: Bending Test Elevation View

6.2 Test Specimens

A total of four walls were tested for the bending tests. The first two 4 ft x 8 ft walls used in the bending tests followed the same framing configuration as in the shear wall test. The walls were built with the corrugated sheet placed within the boundaries of wall framing with a #8 x ¾" modified truss head (MTH) self-drilling screws as connections. A center stud was placed in the center of the framing at a 24 inch spacing. This is the standard configuration for the first two tested walls but the only difference between each test was the thickness of the corrugated sheets. The corrugation sheets were Vulcraft Decking SV36 in two different thickness which were 26 gauge (16 mil) and 28 gauge (13 mil) with the yield strength of 60 ksi. The last two test specimen the different thicknesses of corrugated sheets were both connected by a RSW plus an additional center stud was placed making the spacing between framing 12 inches. This was in order to raise the peak load of the test specimen and get a higher yield strength. Table 6.1 shows the test matrix for the bending tests.

Table 6.1: Bending Test Matrix

Test #	Test Frame	Thickness of Corrugation	Type of Connection
1	4' x 8' - 24"	26 ga	#8 x ¾" MTH self - drilling screw
2	4' x 8' - 24"	28 ga	#8 x ¾" MTH self - drilling screw
3	4' x 8' - 12"	26 ga	Resistance Spot Weld
4	4' x 8' - 12"	28 ga	Resistance Spot Weld

6.3 Test Results

The calculations needed from the results of the bending test were needed to find the Uniform Load Density (ULD) in pounds per square foot (psf). The ULD for the ASF-RWS project is one of the standards required by the Department of the Army. In the requirements stated in 5.2.27 (U) APA 7b. Roof Support "The roof assembly of the ASF-RWS shall withstand a snow load of 40 pounds per square foot" (US Department of the Army 2013). At this time, the design of the roof for the ASF-RWS was the same as the design of exterior shear wall, so meeting or exceeding the requirement of 40 psf was the main goal of the bending test in this research. As for a better design and strength it was determine that a safety factor of 2 needed to be added to the results. The expected results for these tests the ULD should be above 80 psf. In order to find the ULD, first the data results from the bending tests must be analyzed to find the max peak load. There are two loading points placed 25 inches for the end of the wall on the wall specimen. For these results the equations to calculate the ULD are:

Equation 6.1 Moment at Point Loads

$$M_p = \frac{P}{2l}$$

Equation 6.2 Max Moment over a Distribution Load

$$M = \frac{PL^2}{8}$$

Equation 6.3 Uniform Load Density

$$ULD = \frac{8(M_p)}{L^2}$$

Equation 6.4 Incorporating Width into the Uniform Load Density

$$ULD_W = \frac{ULD}{W}$$

Equation 6.5 Convert Uniform Load Density into Pounds per Square Foot

$$ULD_{psf} = ULD_W * (144)$$

where:

M_P = Moment at point loads

M= Max moment

P = Peak load

l = Loading point distance from end of wall

L = Total Length

W = Total Width

ULD = Uniform Load Density

ULD_W = Incorporating Width in to the Uniform Load Density

ULD_{psf} = Converting the Uniform Load Density to Pounds per Square Foot

In Table 6.2 the test results with each failure model can be seen. These four tests were conducted to determine the required ULD specified by the Army. The definition of the framing members are shown in Figure 6.3. The bending test details can be found in Appendix B.

Table 6.2: Bending Test Results

Test #	Test Framing	Thickness of Corrugation	Type of Connection	Peak Load (lbs)	Uniform Load Density (ULD) (lb/ft^2)	Failure Mode
1	4' x 8' -24	26	screw	1396	41.8	Local Buckling on the outside framing near the center with no connection failures
2	4' x 8' -24	28	screw	1384	41.4	Local Buckling on the outside framing near the center with no connection failures
3	4' x 8' -12	26	Spot Weld	1750	52.4	Local Buckling on the outside framing at the applied load with no connection failures
4	4' x 8' -12	28	Spot Weld	1665	49.8	Local Buckling on the outside framing at the center with no connection failures

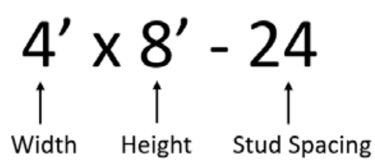


Figure 6.3: Framing Definition

6.4 Discussion of Bending Test Failures

During all four bending tests the failure resided in the wall framing with no failures of the connections. Tests 1 and 2 were constructed with one center stud giving a spacing of 24 inches. For tests 3 and 4 a second center stud was placed at 12 inches on center to strengthen the wall framing. Due to the nature of a bending test the results are expected to have failure in the framing only but no failures in the connections. Tests 1, 2 and 4 all had a local buckling failure near the center on the wall in between both loading points. Test 3 was different due to its failure occurring at the right side loading point.

6.5 Test Conclusions

The results of the bending test were as expected. The addition of a second framing member did not add the needed strength to hold the max load required. Although all test specimens did reach the required standard for ULD of 40 psf the objective of this research was to have a safety factor of two which makes the goal to raise the ULD to 80 psf still within reach. The failures in the framing structure of the shear wall were caused by the use of thinner materials during the testing. The ULD was greatly reduced due to the fact that the design of the ASF-RWS and 20 ga CFS material was reduced and the total weight of the mobile shelter being lightened. The tests also showed that neither the RSW nor the MTH screw connection had any or little to no effect on the yielding strength of the shear wall. The failure modes were all contained within the CFS framing.

CHAPTER 7

CONCLUSIONS AND FUTURE WORK

The primary objective of this research was to determine if resistance spot welding connections were equal to or better than screw connections for a designed 4’ x 8’. The research discovered that the singled sided resistance spot weld achieved similar performance as the self-drilling screws in the applications of CFS wall panels for mobile shelters. The proposed single sided resistance spot weld has advantages of low cost, no added weight, fast fabrication, and it is a feasible connection method for CFS wall panels. The failure modes for each designed RSW wall in the shear wall testing was found to be in the CFS framing rather than the actually connections. It was only when extra bracing was added to a shear RSW wall test a connection failure was found. As for the bending testing the same failure modes was found to be in the frame material and not the connections. Due to the need for a larger weight reduction in the ASF-RWS project, a thinner framing material were used and the spot weld connection was able hold its strength without failure.

For future research in Resistance Spot Welding of CFS walls I believe a heavier thickness wall framing would be the key to finding connection failure in the future. This will add weight to the overall structure but the weight to strength ratio may need to be better researched to get more accurate results in the future. Another area of future research for this project would be testing a better way to apply pressure to resistance spot welds on a CFS framed wall. The limitation found during the research and testing was the downward force needed to make a secure weld when using the adjustable duel tip spot welder. It was difficult to consistently provide the needed pressure to secure an accurate spot weld. For further research I would like to add thicker framing members can be studied to further strengthen the shear walls. This will hopefully achieve a more accurate failure of the spot weld connections and add an overall higher shear and bending strength of the walls.

APPENDIX A

SHEAR TEST DETAILS

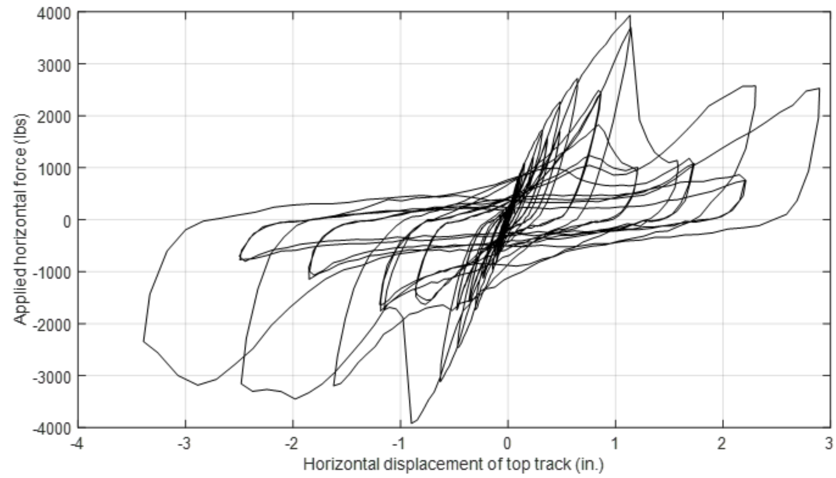


Figure A.1: Shear Test 1 Results

Failure Mode:

Screw pull out along bottom track and sheet connection, sheet tearing, chord and middle track local buckling, buckling and tear at hold-downs

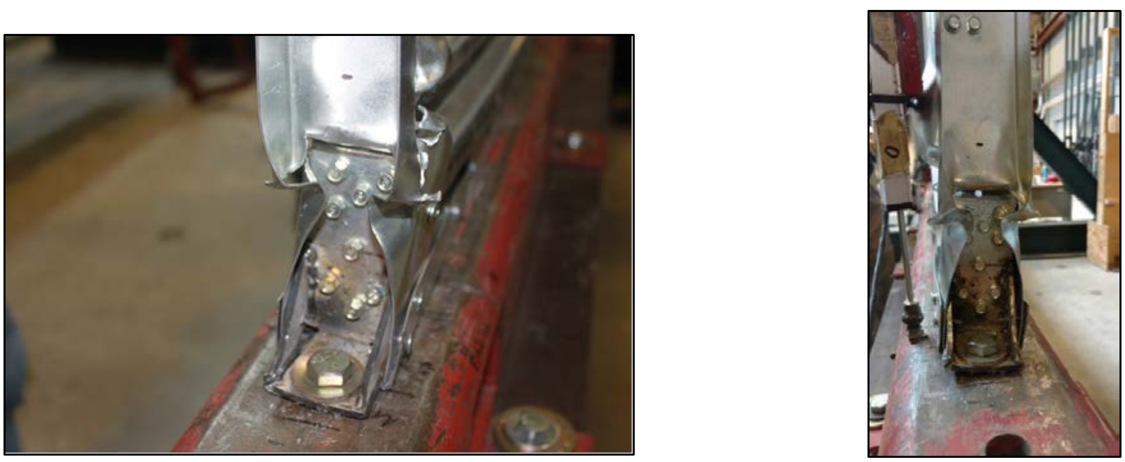


Figure A.2: Shear Test 1 Failure Mode

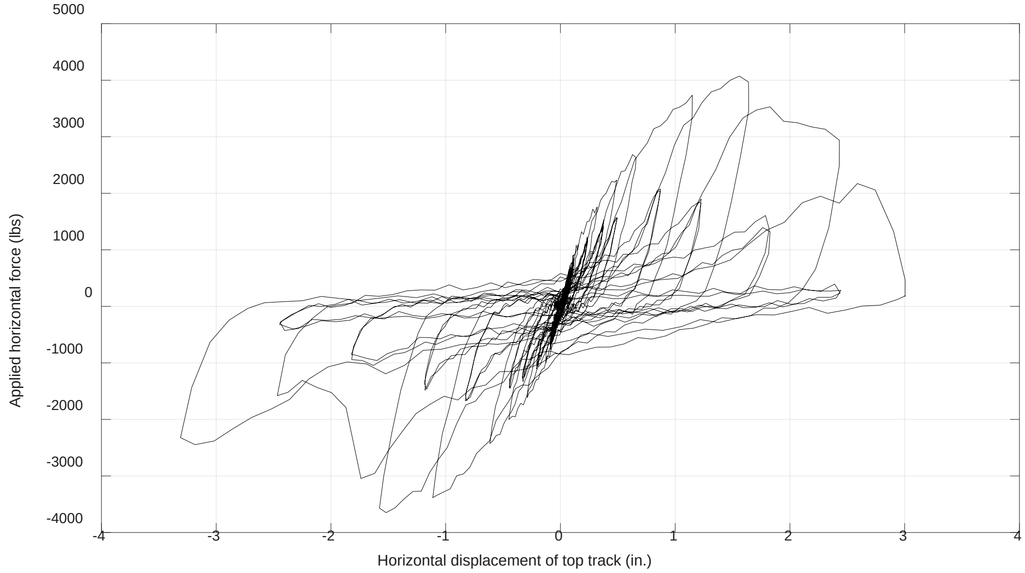
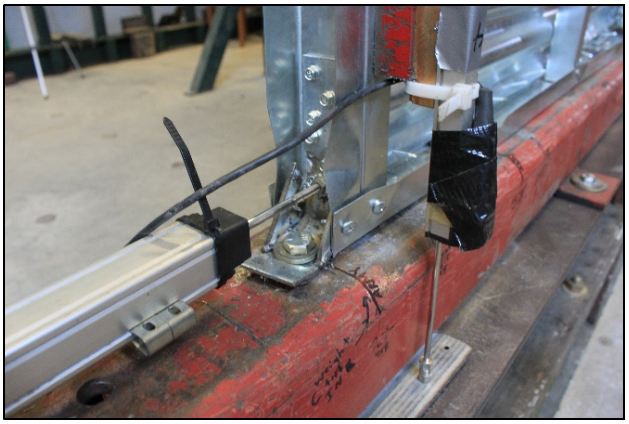


Figure A.3: Shear Test 2 Results

Failure Mode:

Hold-down and washer failure early in test, caused a bad test.



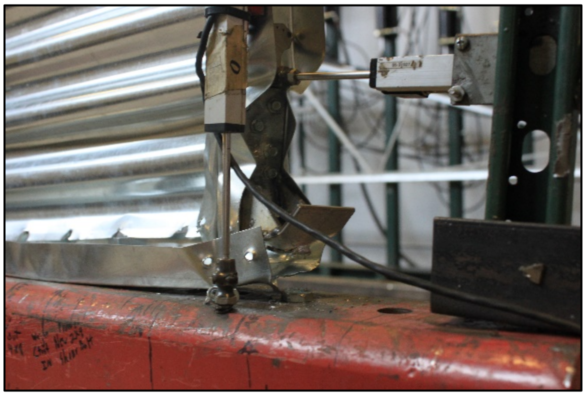


Figure A.4: Shear Test 2 Failure Mode

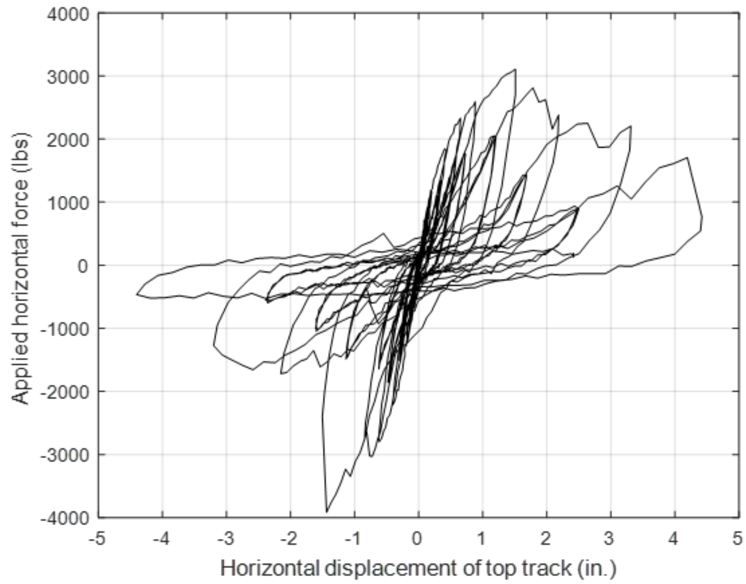


Figure A.5: Shear Test 2b Results

Failure Mode

Top track tore from frame and sheathing. Crippling and buckling of 200T, and 150T at top. Hold-down pulled away from frame at the very bottom.


Figure A.6: Shear Test 2b Failure Mode

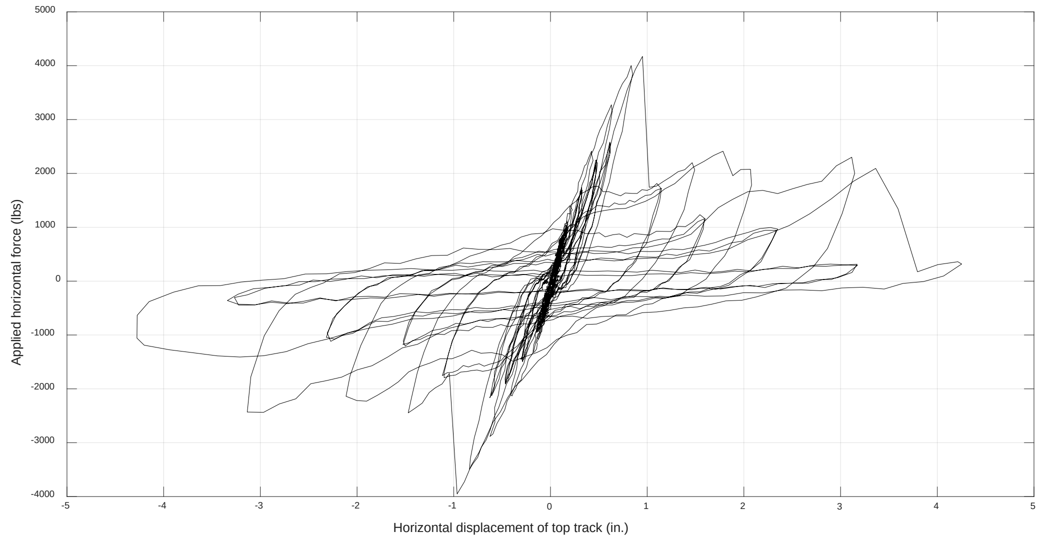


Figure A.7:Shear Test 2c Results

Failure Mode:

Tearing of outside studs on both sides above hold-downs. Middle stud buckling at bottom

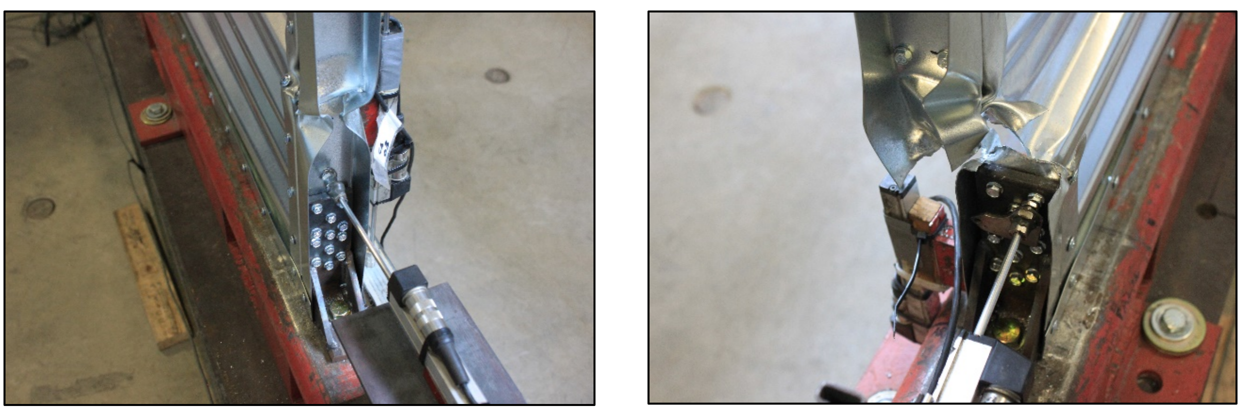


Figure A.8: Shear Test 2c Failure Mode

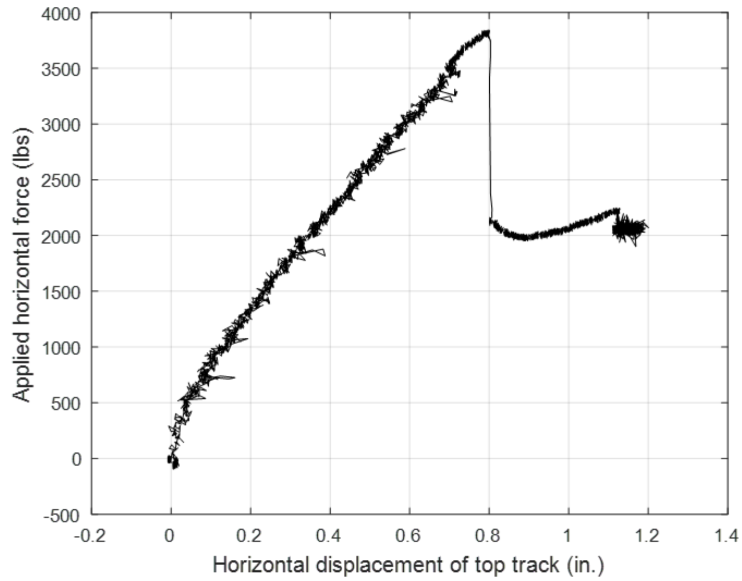


Figure A.9: Shear Test 3 Results

Failure Mode:

Compression side buckling above hold-down



Figure A.10: Shear Test 3 Failure Mode

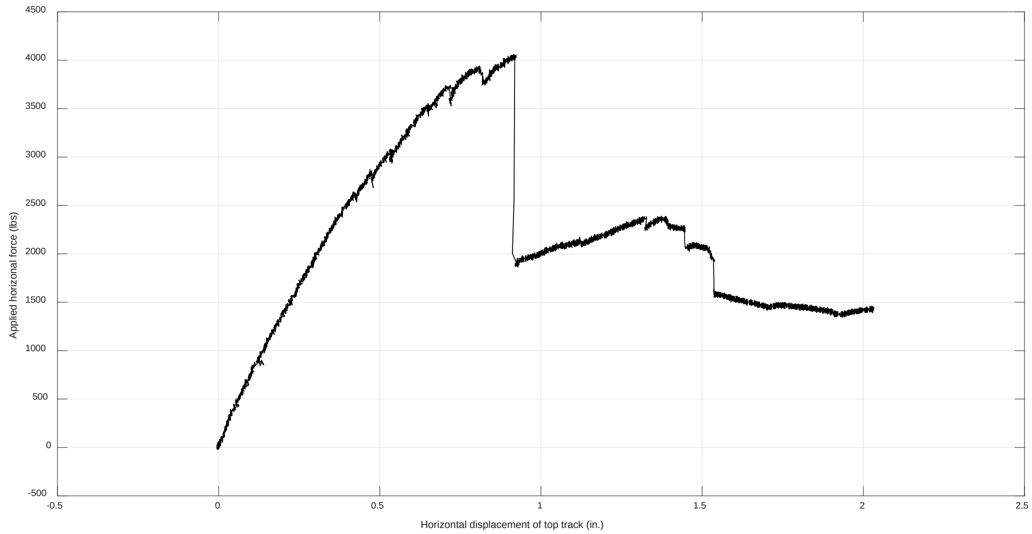


Figure A.11: Shear Test 4 Results

Failure Mode:

Compression side buckling above hold-down

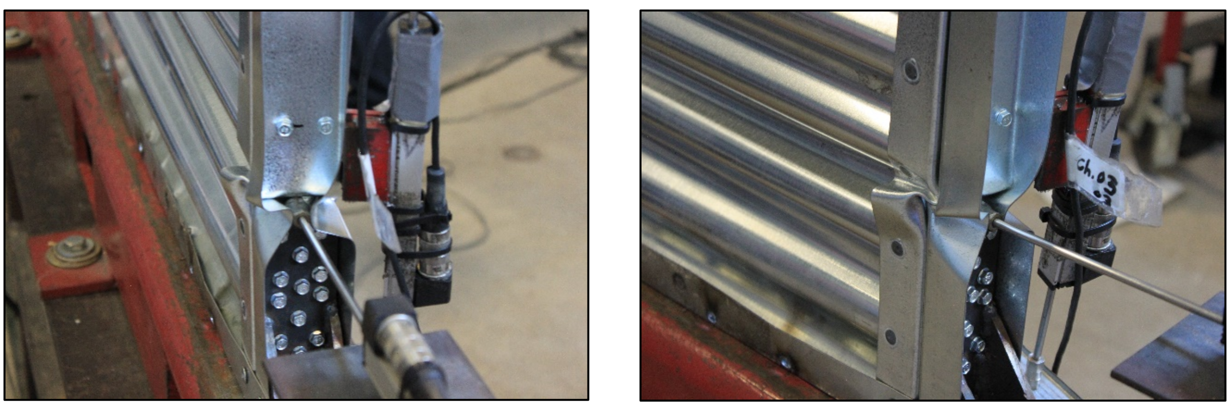


Figure A.12 Shear Test 4 Failure Mode

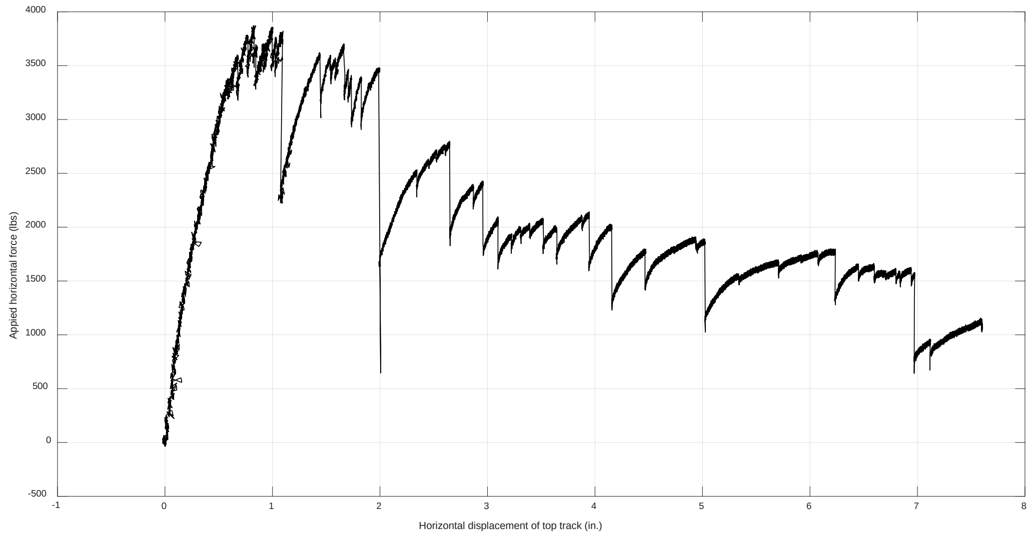


Figure A.13: Shear Test 5 Results

Failure Mode:

Spot Weld Failures multiple areas with Sheathing Tears

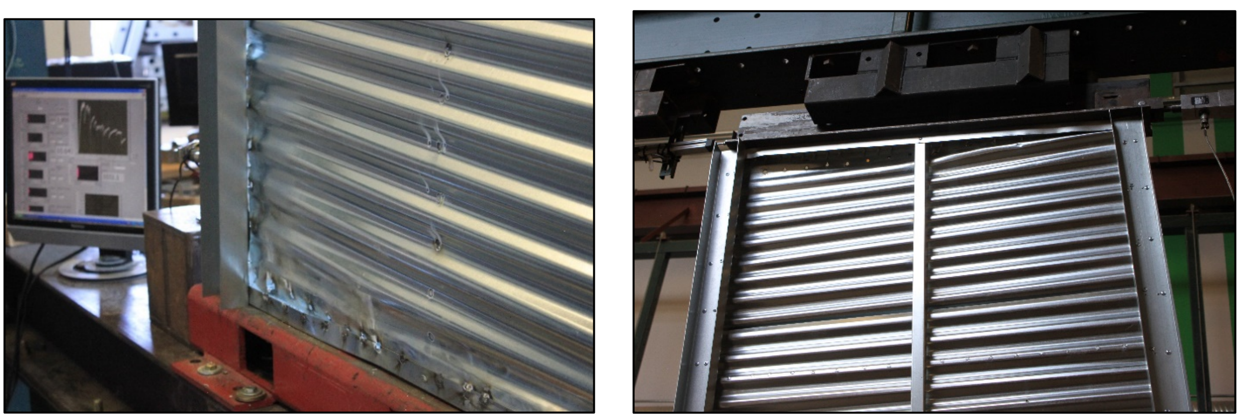


Figure A.14: Shear Test 5 Failure Mode

APPENDIX B

BENDING TEST DETAILS

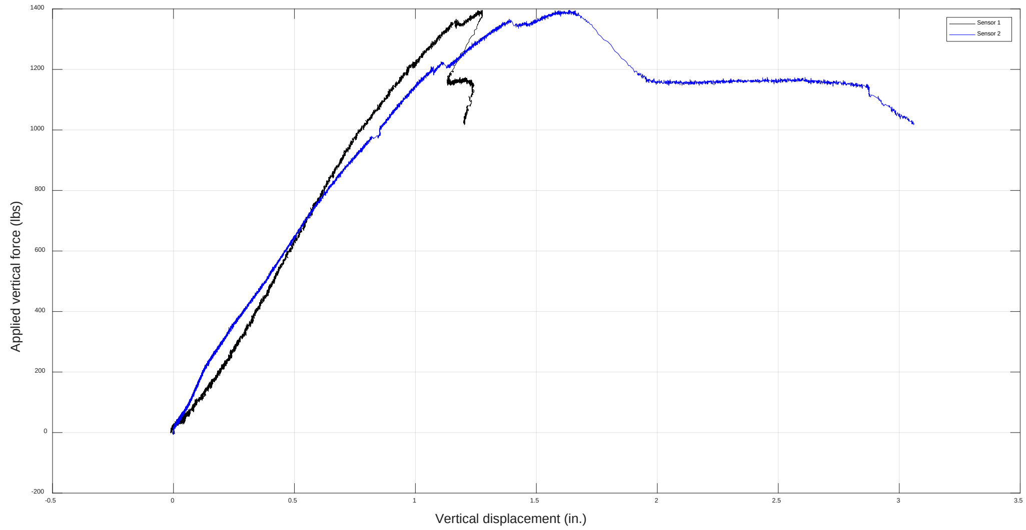


Figure B.1: Bending Test 1 Results

Failure Mode:

Local Buckling on the outside framing near the center with no connection failures

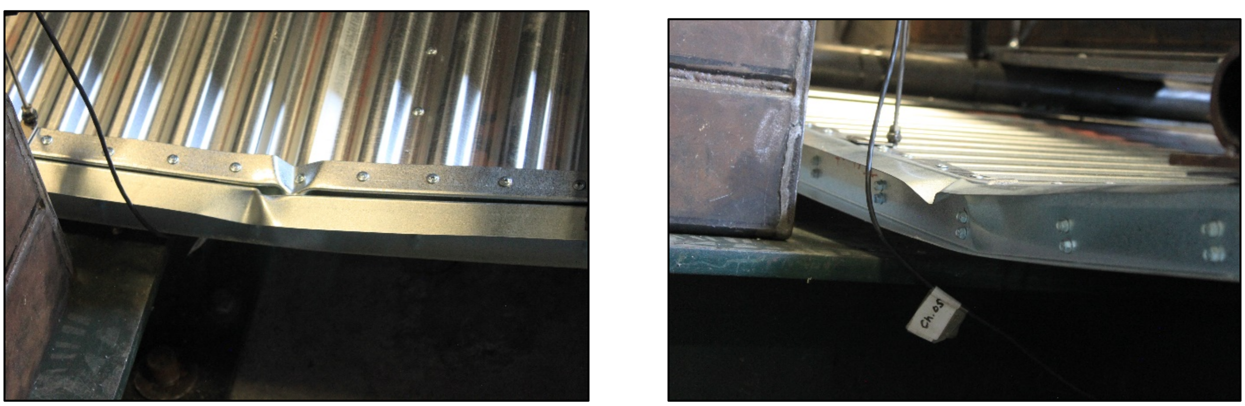


Figure B.2: Bending Test 1 Failure Mode

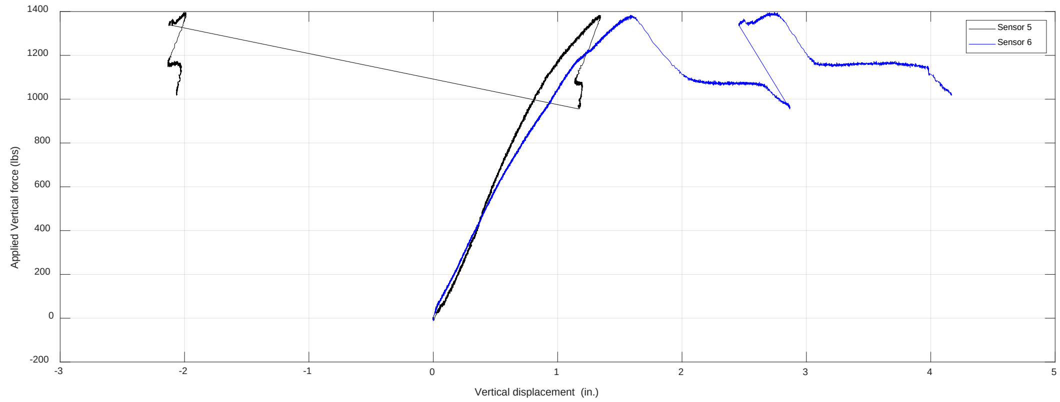
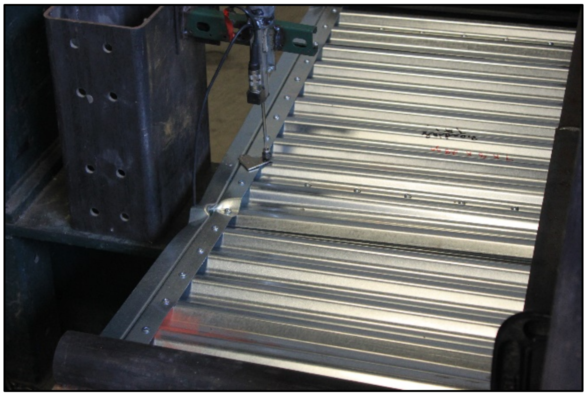


Figure B.3: Bending Test 2 Results

Failure Mode:

Local Buckling on the outside framing near the center with no connection failures



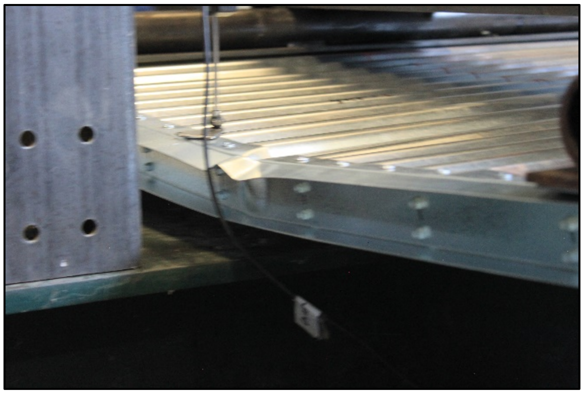


Figure B.4: Bending Test 2 Failure Mode

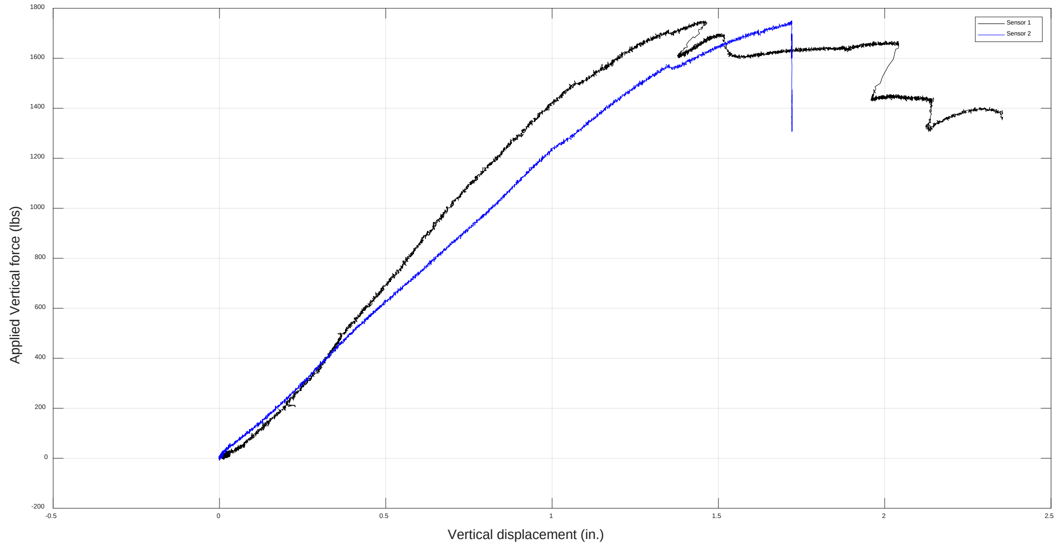


Figure B.5: Bending Test 3 Results

Failure Mode:

Local Buckling on the outside framing at the applied load with no connection failures

Figure B.6: Bending Test 3 Failure Mode

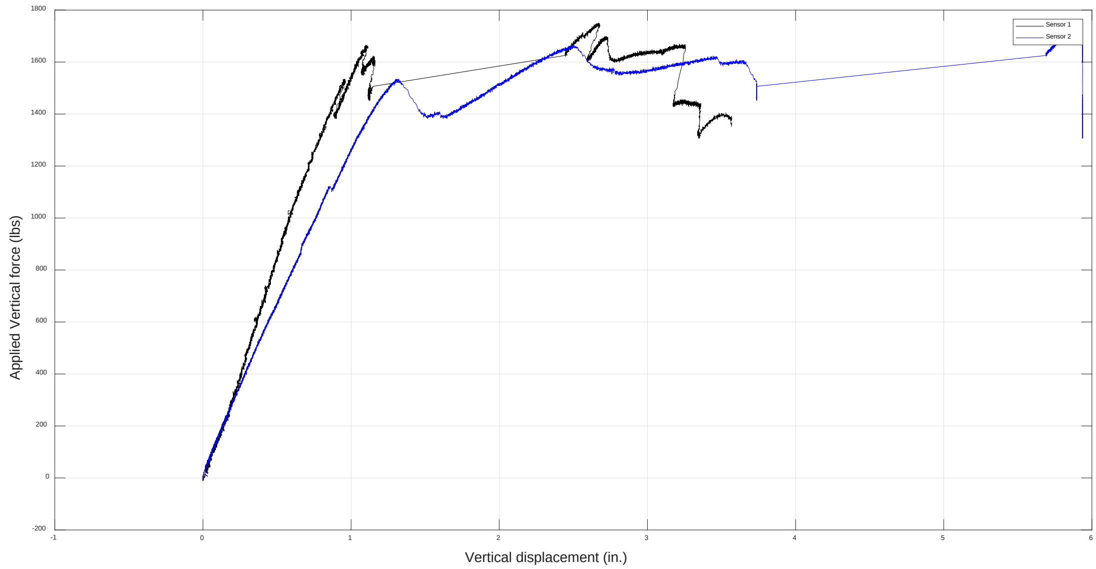


Figure B.7: Bending Test 4 Results

Failure Mode:

Local Buckling on the outside framing at the center with no connection failures

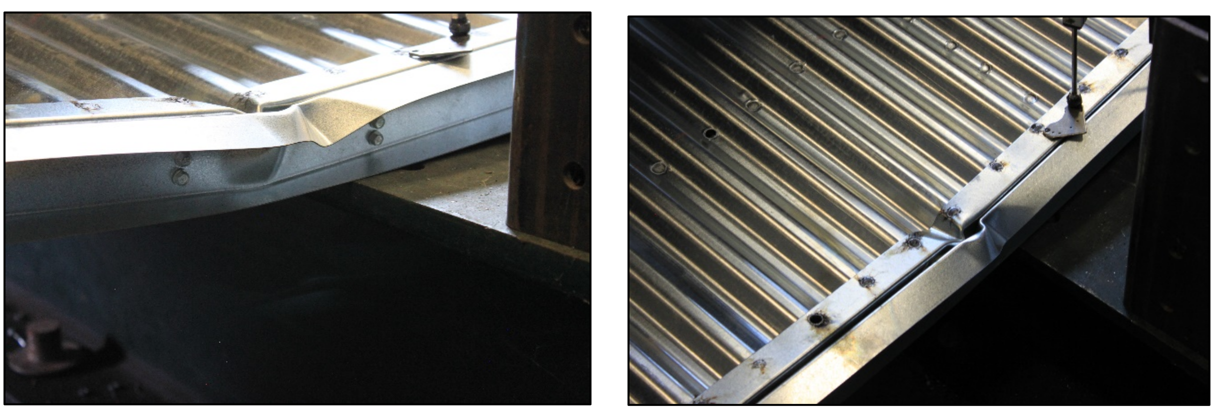


Figure B.8: Bending Test 4 Failure Mode

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