Full-scale testing of sheathed cold-formed steel wall stud systems in axial compression

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August 2009

A supplemental report for
AISI-COFS Project on
Sheathing Braced Design of Wall Studs

Abstract

A series of twelve full-scale walls (cold-formed steel studs with different sheathing configurations) were tested under axial compression. This study concentrated on the different kinds of sheathings attached to the side of the wall, specifically bare (no sheathing), OSB or Gypsum. Wall sheathing combinations Bare-Bare, OSB-Bare, Gyp-Gyp, OSB-Gyp and OSB-OSB were tested. Results revealed that the attachment of boards to the side of the wall can result in an increase of 91% of strength, when comparing the case of Bare-Bare to that of OSB-OSB. It was found that the OSB-Bare walls had no post-buckling reserve. In walls with symmetric sheathing (the OSB-OSB and Gyp-Gyp cases), the failure mode observed was local buckling, and in both cases it was identical and predictable. However, the asymmetric sheathing (OSB-Gyp) can result in different failure modes in some studs.

1 Introduction

This report provides the result of recent research conducted at JHU in association with AISI on sheathing braced design of walls. The project started with a full literature review of research focused on sheathing braced design of cold-formed steel walls. An experimental testing rig (machine) capable of testing full scale walls at different degrees of freedom was developed. The machine design and construction were successfully accomplished, and this report provides the results of a series of tests on cold-formed steel walls, with different sheathing configurations, tested in axial compression.

The walls are composed of a cold-formed steel frame fastened to sheathing. The sheathing is generally OSB, plywood and/or Gypsum and can play an important structural role in bracing the cold-formed steel studs. The walls were designed based on current design practice. The design of the walls was a result of the interaction between the researchers and the AISI project monitoring task group.
2 Background

Winter’s (1960) research into bracing systems for cold-formed steel structures, was the first to formalize the increase of stud capacity due to its connection to sheathing. In 1962 the AISI design code incorporated the design method developed by Winter, which focused on flexural buckling of the studs, and a companion experimental method for determining the lateral restraint stiffness of the connector and sheathing.

Winter (1960) developed a design method based on checking the strength and rigidity of the connection. Details such as the maximum space between fasteners are checked to provide adequate lateral bracing to the stud. The research covered only studs connected to both sides and with the same material on both sides. The requirements, even though rational, include arbitrary checks, such as considering the buckling length equal to twice the fastener spacing (known as the “2a” assumption).

Based on a significant amount of work examining sheathing as a shear diaphragm Simaan and Pekoz (1976) performed tests and developed a design method focusing on shear deformations in sheathing. The Simaan and Pekoz (1976) design method was used from 1980 to 2004 by the AISI specification, the method consider the contribution provided by the board shear stiffness to the flexural, torsional, or torsional-flexural buckling. The shear diaphragm action is translated to the stud via a rotational spring in the same plane of the board. The ability of fasteners in traditional materials to provide bracing support in this manner has been difficult to quantify or verify. As discussed in Schafer et al. (2008) “The abandonment of the method by AISI in 2004 was justified practically and theoretically”.

Miller (1994) tested fastener stiffness in Gypsum boards using the same procedure proposed by Winter (1960). The tests, though limited in scope, provide more information about the connection behavior and stiffness that is utilized in bracing wall studs. Fiorino et al. (2006) and Okasha (2004) concentrated their research on the effect of cyclic load. Iuorio et al. (2008) compiled the lateral and rotational stiffness values that have been published to date.

AISI-COFS (2007) defines a design methodology based essentially on Winter (1960) and therefore AISI (1962). Basically the method insures that connection stiffness and spacing is enough to ensure that the limit state covered by the method is achieved, regardless of the limit state. The method also does not provide a design approach for asymmetric sheathing attached to the sides.

3 MDOF

The MDOF (Multi Degree of Freedom) testing rig also called the Big Blue Baby (or BBB) has a set of seven hydraulic actuators (50kips/actuator). The actuators concomitantly move a cruciform beam which is attached to the top of the specimen. The bottom of the specimen is fixed to a steel beam which is connected to the floor.

Four actuators are mainly responsible to move the beam vertically, a fifth actuator can move the beam horizontally applying a shear force to the wall, and the other two also move the beam horizontally but perpendicularly to the last one. They are responsible to twist the beam, thus correcting eventual misalignment.
The machine is controlled by a Labview program developed by the authors, the same program also collects the data as load and displacement.

4 Wall stud

4.1 Design

The studs used in the test are 362S162-68’s (50 ksi) (SSMA/ASTM nomenclature) throughout. Two types of sheathing are employed: OSB (7/16 in., rated 24/16, exposure 1) and Gypsum (½ in. Sheetrock). Number 6 screws (Simpson #6 x 1 5/8”) were used to connect to the Gypsum boards and number 8 screws (Simpson #8 x 1 15/16”) to connect to the OSB boards. (The detailed design is shown in Figure 36 and Figure 37).

To construct the walls it was necessary to build a jig. The jig was placed on the floor, and the studs and tracks are aligned. The gap between the studs and tracks was closed using a rod clamp nine feet long; the clamp was tightened until the gap was fully closed (webs of the wall studs were bearing against the web of the track) and then the studs and tracks were then connected, following the design in Figure 36. After all the studs and tracks were connected the boards were placed over one side of the steel frame, and they were marked and screwed following the design showed in Figure 37. After the installation of the boards on one side, the wall is ready to be installed in the testing rig.

4.2 Installation inside the MDOF

At this point, the wall has a board on only one side, however, for several tests sheathing was needed on both sides. To install the wall inside the MDOF, first the wall is placed on the bottom beam, and then the wall is elevated over the bearing plates to insure only the track and not the wall sheathing may bear, Figure 1. Figure 1 details how the wall is connected to the top beam of the testing rig (blue), and a similar connection is used on the bottom. Previous studies (Shifferaw et al. 2009) showed that if the wall (boards plus studs) is allowed to bear against the loading beam, in addition to the stud, , there is an increase of 20% in the strength. In the actual building the continuity of boards cannot be guaranteed, hence only the steel frame should be loaded, even though this result in a lower tested capacity. Thus, the installation procedure is designed to insure that the wall boards never engage in direct bearing.

The cruciform (top) beam is lowered until the distance between top track and the beam can be measured with the gages available in the lab. The distance can be slightly different due to misalignment and constructions flaws. To correct this plate that is connected to the top is shimmed (again see Figure 1). The bearing plate is connected to the track at every stud through a ½” inch bolt.
4.3 Instrumentation

Position transducers and string pots were utilized to record supplemental displacements. Depending on the test and the previous data collected, the position transducer setup and location were changed. A record indicating where the position transducers (PT) were located was created for each test. Five string pots and eleven position transducers were installed in each test.

Four string pots were positioned on the top of the upper cruciform beam and one was placed over the horizontal shear actuator. The string pot data is useful to confirm the displacement of the actuators and to report eventual accommodation of the joint that connects the actuator to the beam. To have a “fixed” reference point the string pots were connected to the ceiling of the lab (which supports a mezzanine area which sees little if any use and may be regarded as static), see Figure 2a.

The PTs were grouped in different formations. There is a single PT that measures the out-of-plane displacement, Figure 2b, and a single PT that measures the displacement of the web (local buckling of the web), Figure 2c. This PT sits on a T aluminum section that is connected to the board, and the data is collected from the middle of the web, which is 3.5 inches up from the track. There are the groups of three PT that are able to capture eventual twisting, local and global buckling; one of the PT is placed in the middle and the other two (edge) are placed right after the outside corner web and flange, Figure 2d. The groups of two PTs are placed at the same place, but they are only capable of capturing twisting and global buckling, Figure 2e. In some tests a webcam was placed close to the single PT that reads the displacement at the end of the stud, Figure 2c.
5 Results and Discussion

The load–displacement curves and summary results for each test are provided in the Appendix (Figure 42). A condensed summary of test results is shown in Table 1. As expected the ascending order of values for peak load is Bare-Bare, OSB-Bare, Gyp-Gyp, OSB-Gyp and OSB-OSB. The walls with Gypsum on both sides support more load than the walls with OSB board on only one side, increasing the peak load 10%. The attachment of boards on both sides, independently of which kind, (which will be explained in the next section), is experimentally observed to provide post-buckling reserve. If the wall has one side OSB and the other Gypsum, there is a boost in the peak load of 9% compared to the Gyp-Gyp walls, and if both sides are covered with OSB there is a boost of an additional 3% compared to the OSB-Gyp wall. This means that for the walls with sheathing on both sides strength varies 12% from the weakest (Gyp-Gyp) to the strongest (OSB-OSB), additionally relatively stable post-buckling and post-peak response is observed and the wall gradually lose its capacity to support the load under deformation controlled load application, unlike the OSB-Bare wall which abruptly fails. It should be noted that in all cases there is a significant change in the peak load compared to the Bare-Bare wall.

As shown, the coefficient of variation (COV) was small for the walls with boards on both sides. Only in the OSB-Bare case there is a bigger variation due to the limit state
mode, which basically does not redistribute the load and the wall fails as soon as the first stud fails.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Peak Load (kips)</th>
<th>Limit State</th>
<th>Mean</th>
<th>COV</th>
</tr>
</thead>
<tbody>
<tr>
<td>2-BARE-BARE</td>
<td>56.33 FT and F</td>
<td>56.33</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>12-OSB-BARE</td>
<td>81.57 FT</td>
<td>87.67</td>
<td>0.06</td>
<td></td>
</tr>
<tr>
<td>1-OSB-BARE</td>
<td>89.21 FT</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6-OSB-BARE</td>
<td>92.23 FT</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7-GYP-GYP</td>
<td>94.07 Local</td>
<td>96.39</td>
<td>0.02</td>
<td></td>
</tr>
<tr>
<td>11-GYP-GYP</td>
<td>96.66 Local</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4-GYP-GYP</td>
<td>98.44 Local</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10-OSB-GYP</td>
<td>103.05 Local</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3-OSB-GYP</td>
<td>105.71 Local</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>8-OSB-GYP</td>
<td>105.99 Local</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>5-OSB-OSB</td>
<td>106.04 Local</td>
<td></td>
<td></td>
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</tr>
<tr>
<td>9-OSB-OSB</td>
<td>109.55 Local</td>
<td></td>
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</tbody>
</table>

5.1 Comments

5.1.1 1-OSB-BARE

Figure 3 shows a side view of the wall being tested as well as a close view of one set of position transducers and a picture after the wall failed in flexural-torsional buckling. Figure 4 shows the position transducers (PTs) displacement measurement plot and their location. The horizontal axis in the graph is relative to the vertical displacement of the actuators and the vertical axis is the displacement measured by the PTs. As it can be seen in the “zoomed-in” section of the plot, PT number 7 is observed to have the greatest increase in displacement, thus best indicating the failure of the wall. PT number 7 is the farthest PT from the board at stud number 7. The difference between PT 7 and 9 gives the rotation of the stud, and the difference between the imaginary line connecting PT 7 and 9, compared to PT 8, gives the local buckling wave formed.

All of the PTs (Figure 4) followed the same trend (except number 10 which was measuring the out of plane displacement and as it was predicted PT10 displaced from the beginning of the test). At approximately 43 kips PT 7 showed an abrupt change in direction and indicated failure of the wall, at stud 7, in twist. The wall failed in flexural-torsional buckling with no post-buckling (or post-peak) reserve.
Figure 3 – 1-OSB-BARE

Figure 4 – Position transducer for wall 1-OSB-BARE

5.1.2 2-BARE-BARE

Figure 5 shows the test set up and one of the studs undergoing unrestrained flexural-torsional buckling, Figure 5c. All the position transducers data is relative from a fixed point, in this case the ground.

At around 45 kips position transducer number 7, which was placed at stud number 2 (S2), Figure 6, was the first stud to initiate buckling (see inset of Figure 6) in flexural-torsional buckling. This was followed by stud 4, which also buckled in flexural torsional buckling (see Figure 6b for stud locations in the wall). The peak load was reached at 56.33 kips, and although the other studs were already showing some deformation according to the PT data, only after the peak were the other studs (S1, S3, S5) visually buckling. Stud number 3 and 5 buckled in a pure flexural mode and all the others buckled
in a flexural-torsion mode. The test showed that at the length of 8 feet the stud can buckle in either flexural-torsional or just flexural buckling; the buckling mode is strongly related to the initial imperfections and how the load is redistributed after one of the studs starts deforming in relation to the others. The studs were primarily loaded axially, but after they have different deformations, the stress distribution changes and the stud can be loaded not only in pure compression but also in bending.

As related in the report by Shifferaw et al. (2009) the single column test resisted 16 kips, if the results are extrapolated to the wall, it should theoretically be able to carry 65 kips (5 studs · 16 kips), however the wall carried only 56.33 kips. It is postulated that this lower result is because of the lack of redistribution. The weakest of the 5 studs led to the failure of the entire wall, and after the weakest stud failed the load had to be redistributed to the other 4 studs – where it could not be carried, immediately leading to full collapse of the wall.

Another important observation is the behavior at the connection between the track and stud, Figure 7. The track and studs are connected through two screws, one in each flange. Those two points are fixed and they stay connected throughout the whole test. The stud is free to lift off the track but it does not have enough strength to penetrate the track. The restriction, even though unique to the test, can be considered a kind of partial fixity against twist at the stud ends.
5.1.3 3-OSB-Gyp

Figure 8 shows the test set up and the failure mode. The peak load was 105.71 kips and the limit state that led to failure was local buckling at the ends, even though the studs in the field (fasteners at every 12 in) twisted and stud number 14 finally failed in flexural-torsional buckling. Figure 8b and c shows the cut in the Gypsum board formed by the fasteners and Figure 8e shows the wall after removing one side of sheathing and the stud which failed in flexural-torsional buckling.

The studs that were restrained by fasteners every 6 in (edge studs and middle studs) failed by local buckling because the restrictive force was enough to prevent flexural-torsional buckling. However one of the studs that was less restrained, stud 14, fasteners every 12 in (field stud) had enough restriction from the OSB board but not from the
Gypsum board, and the fastener was able to tear off the Gypsum board and thus the stud failed in flexural-torsional buckling.

Figure 9 shows the limit state that led to the failure, which was local buckling at the end of stud 10. After one of the studs failed the stresses were redistributed and a finite element model is needed to give more information about the failure’s development.

Figure 8 – 3-OSB-Gyp
5.1.4  4-Gyp-Gyp

Figure 10 shows pictures of the local buckling at the studs’ ends. The peak load was 98.44 kips, Figure 10b and c shows the wall after being opened, where it is more clear how the local buckling develops: first the flanges start opening until the yielding lines at the web are created, at the same time that the stud is opening, a little higher up the stud starts closing due to the need to keep supporting the load and find a new load path. Another interesting thing shown in the pictures is how a “bubble” is formed on the Gypsum board due to the flanges forcing the board. The implication is that when it happens all the connections in this region are damaged and it cannot hold any load and therefore is unable to keep restraining the stud.
Figure 11 shows in the zoomed plot that stud number 18, the stud in the middle of the wall, was the one that led the failure due to local buckling at the bottom. However, it is interesting to note that PT number 3, at the top of stud 17, was also showing signs of local buckling at the same level of deformation. Although both locations had similar deformations, likely stud number 18 had greater initial imperfections at the bottom and this was what triggered the local failure. This will be confirmed later as extensive measurements of the initial imperfections were conducted.

This test compared to the last test (OSB-Gyp) shows the implications inherent to the asymmetry of the boards installed to the sides. In the last test, the different stiffness in each side led to a failure mode other than only local buckling.

5.1.5 5-OSB-OSB

Figure 12 shows a side view followed by a picture of the same stud which failed at the bottom in local buckling. The peak load was 106.04 kips and all studs failed in local buckling at the bottom. In Figure 13 the zoomed-in plot shows that PTs number 10 and 9 were the first ones to capture the studs failing in local buckling, but stud 22 at the bottom (PT 10) was the one that showed larger displacements. As stated before, the symmetry of the boards installed prohibits the studs from twisting.

This test, compared to the previous test: 4-Gyp-Gyp, had a slightly higher peak load, showing that the OSB board is able to hold axially more load than the Gypsum, but both were able to restrain the stud thereby failed by local buckling. Hence, the difference (between OSB-OSB and Gyp-Gyp) is not in how much the boards are able to restrain but in how much loads the boards are able to carry axially even when not allowed to bear directly, but still connected to the stud. Thus, some load sharing occurs, although in the usual model (e.g. Winter 1960) it is presumed that the boards only provide elastic restraint and do not themselves contribute to the load carrying capacity.
5.1.6 6-OSB-BARE

This test was the one which presented the highest peak load of the OSB-BARE tests, at 92.23 kips. What occurred during the test was that stud number 27 started to twist towards the flange side instead of towards the lips side (usual side that buckles), which probably was the key for the increased peak load. After the stud started to buckle towards the flange side the stud had to reverse its initial twist before finally twisting to the lip side and failing. However, stud number 29 led the failure mechanism in the wall, as can be seen in the PT plot, Figure 15. Figure 14 shows the wall after being tested and the
position transducers measuring twisting and local buckling. As in all OSB-BARE tests there was no post-buckling reserve and the failure was abrupt after the peak load.

If compared to the previous tests, this test has one more piece of data to help understanding the tests. A PT was placed over the cruciform beam that loaded the specimen. The discussion about the data collected from this PT is explained in detail after the comments of each test, but the authors judged it to be necessary to separate specimen axial deformation from the overall axial deformation (specimen and machine) and this is the main importance of the position transducers.

![a) Wall after test](image1)
![b) Position transducers capturing twist](image2)
![c) Position transducers capturing twist and local buckling](image3)

**Figure 14 – 6-OSB-BARE**

![a) Position transducers plot](image4)
![b) Location of position transducers, upper view](image5)
![c) Location of position transducers, side view](image6)

**Figure 15 – Positions transducers for wall 6-OSB-BARE**
5.1.7 7-Gyp-Gyp

Figure 16a, b and c shows pictures after the test, after the wall was opened. It should be noted that every stud failed in local buckling at the ends except for stud number 32 which failed in distortional buckling. In Figure 16b and c it is clear how the distortional buckling happened; the fasteners tore out of the Gypsum board which was not able to restrain the stud from buckling.

The peak load was 94.07 kips. Stud number 35 was the first to buckle locally, but there was no PT in this place. Stud 34 was the second stud to buckle, which is clear in Figure 17. After stud 35 buckled there was a drop in the load of the two actuators closer to the stud but the other two actuators kept loading until the whole wall failed. Figure 16 also shows what the board looks like when the stud buckles locally; it causes the board paper to separate forming the “bubbles” as shown.

![a) Wall after test](image1)
![b) Distortional buckling](image2)
![c) Closer view of screw tearing out the board](image3)

![d) Bubble forming on the board due to the stud buckling](image4)
![e) Buckled flanges tearing the board](image5)

Figure 16 – 7-Gyp-Gyp

Figure 17 also gives an additional piece of data that helps to understand the results. This time two position transducers were placed over the cruciform beam on the studs at the edges. The goal was know how much the cruciform beam rotates if each side is loaded differently, which was the case. Even though it looks like the loading should be the same, it is not, because the whole machine deforms and the side less loaded has less machine deformation and at same time the actuator stroke stays constant, and therefore the points in discussion displace differently.
5.1.8 8-OSB-Gyp

Test 8-OSB-Gyp failed in local buckling at a load of 105.99 kips. Figure 18 shows three different views of what the stud looks like when it fails in local buckling at the ends and also show the displacement read by the position transducer. The zoom (inset) plot in Figure 19 shows that PT2 (top of stud 39) and PT8 (bottom of stud 39) were both moving the same amount, but likely the initial imperfection was determinant and the stud buckled at the top of the stud. (Again the specific role of initial imperfections will be investigated further in the future – as individual imperfection measurements were completed for each stud.)

Another interesting fact is that the wall, even though it had asymmetric boards installed on the sides, the studs did not show a considerable amount of twist.
5.1.9 9-OSB-OSB

The 9-OSB-OSB wall failed in local buckling at the top where stud number 44 led the failure. Similar to the previous test, both top and bottom position transducers at the same stud had similar displacements but the top was the first to fail, Figure 21.

Figure 20 shows pictures after the wall was tested and opened. The yielding lines were marked, as shown in the figure. Figure 20a shows the yielding line formed at the web, Figure 20b shows the yielding lines at the flange. Two yielding lines formed at the flange; one is a continuation of the local buckling failure in the web and the other, as already commented before, is an attempt of the stud to keep supporting the load through another load path.
5.1.10 10-OSB-Gyp

The wall 10-OSB-Gyp had a peak load of 103.05 kips. Stud 49, which failed locally at the top, led the wall to failure, Figure 23. Figure 22a shows the board over stud 49 and Figure 20b shows the same stud after the board was removed. An interesting observation occurred with stud 47 (field stud, connected every 12 in.); the stud did not buckle only locally at the end like all the others, it also presented distortional buckling, Figure 22c. Figure 22d shows the board torn out by the fastener. The distortional buckling failure probably happened because of the asymmetry of the boards installed; the OSB board and Gypsum were restraining with different stiffness which led the fastener tearing off the Gypsum board since the OSB was providing a higher stiffness at the same location.
a) Buckled flanges tearing the board
b) Local buckling at the top
c) Local buckling followed by distortional buckling
d) Gypsum was torn out in order to restrain the stud

Figure 22 – 10-OSB-Gyp

b) Location of position transducers, upper view

c) Location of position transducers, side view

Figure 23 – Position transducer for wall 10-OSB-Gyp

5.1.11 11-Gyp-Gyp
The wall 11-Gyp-Gyp failed in local buckling at the bottom at a load of 96.66 kips, Figure 24. PT 7 and 8 captured the wall failing in local buckling at the same time, Figure 25. Even though it is not possible to show in a written report the video from the webcam, for the first time the webcam installed inside the wall was able to capture the waves forming on the web and evolving to a local failure and yielding.

![Figure 24 – 11-Gyp-Gyp](image1)

a) View from the back of studs, local buckling on the bottom  
b) View from the front of studs

**Figure 24 – 11-Gyp-Gyp**

![Figure 25 – Position transducers for wall 11-Gyp-Gyp](image2)

a) Position transducers plot  
b) Location of position transducers, upper view  
c) Location of position transducers, side view

**Figure 25 – Position transducers for wall 11-Gyp-Gyp**

5.1.12 12-OSB-BARE

The wall 12-OSB-BARE failed in flexural-torsional buckling at a load of 81.57 kips, Figure 26. As in all OSB-BARE walls there was no post buckling reserve and the failure was abrupt after the peak load. In Figure 27 is clear that stud number 59 first started to twist and led all the others.

![Figure 26 – 12-OSB-BARE](image3)
5.2 Difficulties in collecting and processing the data

Although the difficulty in assessing the axial wall deformation was already commented on, the authors feel that it should now be explained in better detail, Figure 28 compares all the methods. Usually the position is considered the average of all the built-in LVDTs from the four vertical actuators (plot 11-Gyp-Gyp.txt), nonetheless the position plotted is actually the displacement of machine and specimen. In order to isolate only the specimen deformation, two position transducers where placed over the cruciform
beam on the edge studs, here called 11-Gyp-Gyp-PT10 and PT11. It can be noted that those two position transducers give different values. This is because each side of the specimen does not necessarily have the same deformation. The wisest method would be to plot load versus the average of both position transducers.

The curves were also compared to the single column test (25-Gyp-Gyp). To do this, the load supported by the wall was divided by five to compare to the single column test. The single column test, even though it was also tested in the MDOF machine, had less load applied (only one column) and so less machine deformation, and as it can be seen the position data is closer to the PT data values. This information is very important to compare to a FE model. The difference of deformation has to be considered. For example, at the peak load the position (machine plus specimen) is close to double the position acquired from the position transducer.

Figure 42 shows the load displacement curves for all the walls tested. The position in the plot is that of the machine plus wall, since not every test had a position transducer measuring the vertical displacement.

![Graph showing load displacement curves](image)

**Figure 28 – Position assessment.**

Another difficulty encountered is in post-processing the data. Figure 29 shows one example of how the curve fitting was done for a curve of load versus position. The MDOF machine has a time step control, which means that the actuators move at a determined speed and stop until it is time to initialize the next time step, and it also does not start the next time step if one of the actuators did not reach the target position. What occurs is that during every time step the actuators load a small amount and wait for the next time step. When the rough data is plotted there is a small amount of noise and a
relaxation of the load, a new curve is traced from the rough data and this curve is an average approximation, but the approximation is small enough to be insignificant.

![Figure 29 – Curve fitting of raw load-displacement data](image)

5.3 **Coupon Test**

Columns and tracks where chosen randomly from the single column specimens already tested, since they used the same material. Steel plates where roughly cut from the web and outside the yielded area, the dimension entered in the CNC milling machine to precisely cut the coupon specimen is showed in Figure 30.

![Figure 30 – Tensile coupon dimensions (Moen (2008)).](image)

*nominal, actual dimension will vary slightly*
After cutting the specimens they were immersed in hydrochloride acid, concentration of 15% per volume (85% distilled water and 15% hydrochloride acid) for 10 minutes. The hydrochloride bath is necessary to remove the zinc coating so that the specimen dimensions could be measured without the coating. Figure 31 shows the test set up and one of the specimens after being tested.

Two methods were used to find the yielding stress, the 0.2% offset method and the autograph method. The 0.2% method considers the yielding stress equal to the stress where the stiffness line (Young’s modulus = E = 29500ksi) is offset from the origin to the 0.2% engineering strain intersects with the stress-strain curve. The autograph method uses the same offset technique but the lines are offset to 0.4% and 0.8% of the strain and the yielding stress is the average value in the range between the two lines.

All the data is presented in Table 2. The authors consider the 0.2% offset method the most appropriate method to find the yielding stress for the steel used in the studs because there is no yielding plateau, Figure 38 and Figure 39. However, the steel used in the track has the yielding plateau and therefore the autograph method is more appropriate; if the 0.2% method is used to define the yielding stress the value will be a peak value, Figure 40 and Figure 41.

![Test set-up](image1.png) ![Coupon failed](image2.png)

**Figure 31 – Tensile coupon test.**

### 6 General discussions

#### 6.1 Bare-Bare

The Bare-Bare wall had its importance in setting a lower bound for the load, Figure 32. In the same figure the load curve from the OSB-OSB walls is also plotted, which shows the upper bound for the loads. The Bare-Bare walls were also important in helping to understand the end conditions and how the track restrains the stud. As a next step in this research the test will be helpful to calibrate the FE models.

The Bare-Bare wall was also important to keep aware of how the load distribution changes during the test; axial compression versus compression and bending. During the test the load distribution changes when there is a difference in the behavior of one stud. It
was evident in this test because some studs failed in flexural-torsional buckling while others in just flexural buckling, which is evidence of change in the applied load.

![Graph showingComparison between OSB-OSB and BARE-BARE walls](image)

**Figure 32 – Bare-Bare**

6.2 **OSB-Bare**

The OSB-Bare walls, Figure 33, were important since they did not have post-buckling reserve, the failure was always abrupt. In order to not allow an abrupt failure, the common practice is the use of a steel strap to brace the studs. Supposedly the strap would restrain the studs to each other and the failure would not be determined by individual failure but by failure of the whole system. Given the nature of the observations, the adequacy of such a strap is unclear.
6.3 Gyp-Gyp

The walls with symmetric boards attached to the sides, Gyp-Gyp and OSB-OSB, were very predictable, always failing by local buckling at the ends. The local failure at the ends is most likely a result of the way that the wall is assembled; the ends of the stud had to be squeezed to fit between the flanges of the track which can create considerable “fabrication” initial imperfections at the ends (as opposed to “manufacturing” imperfections).
6.4 OSB-Gyp

The OSB-Gyp requires special attention since the asymmetric boards attached to the sides had an effect in the failure mode, but it needs to be quantified. The first failure mode encountered was local buckling at the ends, but the field studs had different failure modes; not only can the asymmetry be responsible for that, but also how the load distribution acts during the test. The load versus position curve for all the tests are compared in Figure 35.
The load versus position curves are compared in Figure 32. The important conclusion about the OSB-OSB tests is seen when the results are compared with the Gyp-Gyp test. Both tests used fully the resistance of the studs, which in both cases failed in local buckling, but the actual board is what bears more load. OSB, as it was supposed to be, is able to carry more load and therefore increase the peak load by a small amount.

7 Conclusions

Several conclusions were already commented above, the main conclusions are listed below: i) how the end conditions behave, ii) the OSB-Bare failure with no post-buckling reserve, iii) the implications of asymmetric boards and iv) the similarity in the failure mode and resistance when the boards attached are symmetric. The data collected from this report as well as the translational stiffness report and the single column test report are part of the evolution of the larger project that aims to propose a comprehensive and reliable design method for sheathing-braced walls. Based on the tests results future research will have concrete data from actual tests to reinforce final conclusions.

8 Future Research
The development of a finite element model will be necessary to explore and better understand the sheathing braced walls. Several questions still remain, such as how the load distributes, the impact of initial imperfection, and expected reliability.

9 Acknowledgments

The authors are indebted to AISI (American Iron and Steel Institute) for the grants awarded, Simpson Strong-Tie for the fasteners donated and Nickolay Logvinovsky, Lauren Thompson and Hannah Blum for all the help and determination during the lab tests.

10 References


Figure 36 – Steel frame detailing
Figure 37 – Details of connection between boards and steel frame
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* Specimen that failed inside the gauge length (length covered by extensometer)

$\Delta u$ (in./in.) - the big value of COV for $\Delta u$ is due to the position of the extensometer compared to where it fails. If the crack is in the gauge length, $\Delta u$ is bigger than if it is outside the gauge length.
Figure 38 – Yielding stress determined using 0.2% offset method (Studs)
Figure 39 – Yielding stress determined using autographic method (Studs)
Figure 40 – Yielding stress determined using 0.2\% offset method (Traxs)

Figure 41 – Yielding stress determined using autographic method (Traxs)
Figure 42 – Curve load displacement