Increasing Strength of Plywood Shear Walls

ARCE 453 SENIOR DESIGN PROJECT
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Introduction

Continuing a project completed in ARCE 451, the Timber and Masonry Structures Design and Constructability Laboratory, this project explored two methods to enhance the performance of single sided, solid timber shear walls through large scale testing. The first design examined the impact of added metal gauge strap at panel edges and the second design studied the impact of multiple rows of nail fasteners at panel edges in a high load diaphragm configuration. Both alternatives were designed, constructed, tested and results were compared to results from a typical shear wall constructed and tested in ARCE 451.

Design

In total, four 8’ × 9’ ¾” solid timber shear walls were designed, constructed, and tested: one conventional code specified wall, two modified Metal Gauge (abbreviated to “Mega”) Strap walls, and one High Load Wall. Design of timber shear walls and diaphragms are guided by 2018 National Design Specification (NDS) for Wood Construction and the 2015 Special Design Provisions for Wind and Seismic (SDPWS).

The focus of this investigation was to identify methods to address common failure modes, such as nail head pull through and panel edge tear out. To isolate the sheathing and/or nail shear failure mechanism, wall components such as the end posts, anchor bolts, and hold-downs were all designed to resist a higher load than the ASD capacity of the nails and sheathing for all wall designs.

The increased load factor applied to the design of these other wall components was derived from findings published in APA Report 154. A load factor of three is suggested by APA Report 154. This value was used to determine the expected ultimate capacity of each of the walls. See Appendix A, Section “Specimen Description and Design” for further discussion of the APA 154 safety factor and how it was applied to design.

Conventional Wall and High Load Wall

See Appendix A, Sections 2.0 “Conventional NDS Wall Design,” 3.0 “Metal Gauge Strap Wall 1,” and 4.0 “Metal Gauge Strap Wall 2” for specific design descriptions of the Conventional NDS Wall, the Metal Gauge Strap Wall 1, and Metal Gauge Strap Wall 2. Wall drawings, details and material take offs are provided in Appendices B and C.

High Load Wall

Since high load diaphragms are an existing system in the code, the NDS was consulted for design of the High Load Wall. The design capacity was determined using seismic values from Table 4.2B of the SDPWS for a high load diaphragm and the relevant ASD seismic reduction factor of 2.0 was applied to the nominal capacity listed in the table. Given the selected nailing and sheathing, the high load diaphragm had an ASD capacity of 1790 plf or a total of 14,320 pounds applied laterally. As shown in Figure 1, the High Load Wall was designed with 19/32” Structural 1 Sheathing, three rows of 3” long 10d gun nails spaced at 2 1/2” on center at panel edges, common 10d field nailing at 12” on center, a 4x stud at the adjoining panel edge, and 4x blocking at panel edges.

The end posts, mud sill, top plate, hold downs, and anchor bolts were designed to withstand the expected ultimate load of three times the ASD design value at a total of 42,960 lbs. The posts, studs, and mud sill were kept consistent with the design of Mega Wall 2 as shown in Figure 10 from Appendix A, Section 4.1 “Specimen Description and Design”. The top plate was increased to a No. 1 DF-L 4x6 instead of a double
2x6 top plate to accommodate the high load nailing pattern. 4x6 blocking was installed at the horizontal adjoining panel edges.

Due to material limitations, 19/32” Structural 1 panel was only available in 4’ x 8’ sheets instead of the 4’ x 10’ sheets that were installed on the other shear walls. Since full 4’x8’ panels could not be installed side by side vertically without changing the dimension of the wall, instead four separate panels were installed as shown in Figure 1.

![Wall Components](image)

**Wall Components**
1. 4x6 Top Plate
2. 4x6 Adjoining Panel Stud
3. (2) 6x8 Posts
4. (4) 2x6 Studs
5. (6) 4x6 Blocking Components
6. (4) Simpson Strong-Tie HDU 14’s
7. 4x6 PT Mud Sill
8. (8) Anchor Bolts

**Figure 1. High Load Wall design and components**

**Fabrication**

The Conventional NDS Wall and the Mega Wall 1 were constructed by two sections of ARCE 451 students in the fall. The Mega Wall 2 and the High Load Wall were constructed by the senior project students. All four shear walls were constructed in the High Bay Laboratory and CAED Wood Support shop on the Cal Poly campus. Shear walls were constructed using common building practices and safety protocol utilized in industry. Figure 3 depicts various phases of the construction sequence of the ARCE 451 class, which was repeated for the Mega Wall 2 and the High Load Wall.

Walls were first drafted using CAD software; these drawings were used to develop material take offs and fabricate the shear walls. The walls were constructed on a flat, slab-on-grade and then lifted onto a steel foundation beam in the testing area. Before the walls were put into place, threaded rod was cut and welded onto the top flange of the foundation beam to act as anchor bolts. For additional details and figures of the typical test setup for each of the walls, see Appendix A, Section 2.2 “Test Setup.” In order to crane the wall into place on the anchor bolts, holes were pre-drilled into the mud sill. A template was created to mark the location of the anchor bolts on the foundation beam, this template is pictured in Figure 2 below. This template was lined up with the mud sill of each of the walls and holes were predrilled into the mud sill during fabrication.
All members and metal straps were cut to the correct size and labeled. The wall framing (posts, studs, seam studs, mud sills, and top plates) was constructed and verified to be square by measuring the diagonal distance between corners as seen in Figure 3 (a) and (b). After each wall was squared, the sheathing was nailed into place. For the Mega Walls, a nailing pattern was marked on the metal strap before it was nailed into place by hand, shown in Figure 3 (c) and (d). A nail gun could not be used for the Mega Walls due to the interference of the metal gauge straps.

For the High Load Wall, a nail gun was used to install the sheathing to emulate common practices in industry. To improve construction efficiency, a template of the nailing pattern was developed, (shown in Figure 3(e)) and spray painted onto the wall to ensure the proper number and location of nails were installed.

The construction of the High Load Wall proved to be more difficult than the Conventional NDS Wall and Mega Wall due to the low tolerances of the precise nailing pattern. These issues were most evident at the adjoining panel edges. Nails on the edges of the adjoining panels had to be installed at an angle to ensure full embedment into the 4x6 stud member.
Test Set Up

Once the walls were fully fabricated on the ground, a ledger was attached using 3” SDS screws across the studs and post as an attachment point for the crane. The walls were then lifted into place on the foundation beam. Once the wall was in place, stabilizing steel beams were placed on both sides of the wall. At this time, the nuts and washers were also installed on the anchor bolts and tightened down. Once the walls were leveled, wood shims were placed between the stabilizing beam and the wall to add out of plane bracing. A loading beam was then placed on top of the wall and screwed into the top plates with (32) 2.5” SDS screws. With the wall stabilized, hold downs were placed symmetrically on both sides of the post. All connections were tightened and double-checked to ensure the wall was ready for testing.

The Conventional Wall, Mega Wall 1, Mega Wall 2, and High Load Wall all had identical test setups. For additional details and a diagram of the final test setup, see Appendix A, Figure 2.

Instrumentation

The instrumentation for the Mega Walls and the conventional NDS wall are explained precisely in Appendix A, Sections 2.3, 3.2, and 4.3 “Instrumentation.” The High Load wall was instrumented according to the instrumentation plan of Mega Wall 2. The relative sheathing panel displacement was...
measured quantitatively with adjacent linear displacement sensors located on the bottom north and south panel and qualitatively with a solid thick line drawn across each of the two sets of vertical panels. See Figure 4 for a visual diagram of the instrumentation set up.

Figure 4. Instrumentation Set-Up Diagram

1. String potentiometer (abbreviated to pot.) at the south end of top plate to measure lateral deflection.
2. Second string pot. at the south end of top plate to measure lateral deflection. Two instruments were installed for redundancy and added accuracy.
3. String pot. at mud sill to measure slip between mud sill and foundation beam.
4. String pot. at foundation beam to measure slip between beam and concrete foundations.
5. String pot. at southern concrete foundation to measure slip between foundation and shop floor.
6. Linear displacement sensor at northern post to measure vertical displacement.
7. Linear displacement sensor at southern post to measure vertical displacement.
8. Horizontal line to visually measure panel racking of top panels.
9. Linear displacement sensors to measure vertical displacement of southern plywood panels.
10. Linear displacement sensors to measure vertical displacement of northern plywood panels.
11. Horizontal line to visually measure panel racking of bottom panels.
Loading and Testing Procedure

Lateral loads were applied uniformly across the top plate of each wall using an MTS actuator. See Appendix A, Sections 2.4, 3.3, and 4.4 “Loading and testing procedure” for the testing procedure of each wall. The High Load Wall followed the same general testing procedure as Mega Wall 2 with modifications in the set displacement during the deflection-controlled portion of testing.

High Load Wall Test Procedure

The High Load Wall was loaded monotonically and pulled in the southern direction. It endured four test cycles, 100% design load, 200% design load, 400% design load of a single sided shear wall, and one deflection control cycle. The High Load wall was tested to the same load benchmarks as prior walls to allow for direct comparison between walls. Once these three benchmarks were satisfied, the wall was switched to displacement control. Displacement control procedure is based on the behavior of each wall and how much the specimen displaces during force control. Set displacements are tailored to each wall and are increased during testing until the specimen experiences failure. During the first displacement-controlled cycle, the wall experienced sudden failure. Once this occurred, the testing protocol was ended. See Table 1 below for a detailed description of the High Load Wall loading protocol.

<table>
<thead>
<tr>
<th>Table 1. High Load Wall – Loading Protocol</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Force Controlled</strong></td>
</tr>
<tr>
<td>1) Pull to 100% design load.</td>
</tr>
<tr>
<td>2) Hold load for 15 seconds.</td>
</tr>
<tr>
<td>3) Unload to zero lbs applied and let wall rest for five minutes.</td>
</tr>
<tr>
<td>4) Repeat for 200% Design Load and 200% Double-Sided Shear Wall Design Load.</td>
</tr>
<tr>
<td><strong>Switch to Displacement Controlled</strong></td>
</tr>
<tr>
<td>5) Load wall to specified displacement benchmark.</td>
</tr>
<tr>
<td>6) Hold displacement for 15 seconds.</td>
</tr>
<tr>
<td>7) Unload to 0 lbs applied and let wall rest for five minutes.</td>
</tr>
</tbody>
</table>

Results and Discussion

The individual discussion of results for the Conventional NDS Wall, Mega Wall 1, and Mega Wall 2 are found in Appendix A, Sections 2.5, 3.4, and 4.5 “Results and Analysis.” The individual results of the High Load wall will be discussed below before comparing the overall performance of the specimens.

High Load Wall Results and Analysis

The hysteresis curve for the High Load Wall is shown in Figure 5. The hysteresis curve depicts the four loading cycles the wall endured before failure. The highest ultimate load achieved by the High Load Wall was 33,407 lbs before its sudden failure resulting in an immediate drop in force resisted.
The High Load Wall was projected to have an ultimate capacity of 42,960 lbs after accounting for both the ASD reduction and APA 154 safety factor of three. The sudden failure at 33,407 lbs was below the expected capacity of the High Load Wall. It is hypothesized that the wall failed prematurely due to local out-of-plane buckling of the plywood panels due to excessive field nailing spacing. Due to the wall’s sudden failure mechanism, that this wall could have seen greater forces had this limiting factor been addressed. Unlike Mega Wall 2, the High Load Wall did not resist any load after its failure.

\[\text{Force (lbs)} \quad \text{Displacement (in)}\]

\[\text{High Load Hysteresis} \quad \text{High Load Backbone Curve}\]

\[1\text{Dotted line portion of hysteresis curve is to illustrate data recorded after the wall failed.}\]

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{hysteresis_curve}
\caption{High Load Wall – Force vs. Deflection}
\end{figure}

The panels of the High Load Wall showed observable racking, or relative displacement between panels. The vertical panel racking of the bottom two panels at load benchmarks are summarized in Table #. Although minimal racking occurred between the bottom panels in the 100% and 200% Design Load cycles, significant displacement between panels was observed in the 400% Design Load cycle with a displacement of around 0.39”. As shown in Figure 6, the panel displacement was observable at high loads. The most pronounced panel racking occurred at center of the wall, where all four panel corners aligned, shown in the enlarged image at left in Figure 6.

During the third load cycle, it was evident the northern panels and southern panels began to act as separate rigid bodies. Shown in the left image of Figure 6, the enlarged images show a gap forming along the vertical seam as the panels began to separate laterally. It also shows that the top panels showed much greater relative displacement to each other than the bottom panels, approximately displacing 1.25” from its original orientation.

\begin{table}[h]
\centering
\caption{Relative Sheathing Panel Displacement (North vs. South)}
\begin{tabular}{|c|c|}
\hline
Load Cycle & Displacement (in) \\
\hline
100% Design Load & 0.044” \\
200% Design Load & 0.099” \\
400% Design Load & 0.388” \\
\hline
\end{tabular}
\end{table}

\[\text{Table 2. Relative Sheathing Panel Displacement (North vs. South)}\]
High Load Wall Failure Mechanisms

The High Load Wall underwent a sudden and unique failure. In the fourth load cycle, the adjoining panel edge stud, a continuous 4x6, split apart vertically, as shown in Figure 7 (a) and (b). Although the six lines of nails attaching the sheathing to the face of the 4x6 stud were spaced according to NDS specifications, the member fasteners were highly congested. This congestion made construction more difficult and proved to be a controlling failure mechanism during testing.

The high volume of nails in the adjoining panel stud caused the member to split. Once the seam stud split, the southern panels continued to be loaded separately from the northern panels which caused the top plate to split, pictured in Figure 7(d).

Outside of the nailing at the adjoining panel stud, the field nails and other panel edge nails were relatively undamaged and undeformed. The sheathing also sustained little damage. These observations suggest that the multiple rows of fasteners were effective in distributing the force resisted by each individual nail. The multiple rows of fasteners increased the stiffness of the panels, and their minimal deformation shows that they dissipated less energy. Since less energy was dissipated by the nails this was absorbed by the framing members which caused the ultimate failure at the seam stud.

Other failure mechanisms observed in the High Load Wall were nail withdrawal of the studs located along the mudsill as depicted in (d) of Figure 7 as well as slight crushing in the southern post, where compressive forces were at maximum. The limited damage of the nails and plywood also suggests that the failure of the seam stud caused the wall to fail far before the full capacity of the nails and sheathing could be achieved.
Figure 7. (a) Front view of adjoining panel edge stud splitting; (b) Rear view of adjoining panel edge stud splitting; (c) Nail withdrawal from mud sill at base of studs; (d) Top plate splitting

Overall Results and Analysis

The results for the Conventional NDS Wall, Mega Wall 1, and Mega Wall 2, and the High Load Wall are discussed below. Maximum forces and displacements are identified and compared for each specimen. From these results, linear stiffness values are calculated, and failure mechanisms are discussed to better understand how the specimens performed.

Load Benchmark Comparisons

The results indicate that the both the Mega Wall and High Load Wall designs had a higher ultimate capacity and smaller corresponding deflection when compared to the conventional NDS wall. Due to the unique and sudden failures of the High Load and Mega Wall 2, Mega Wall 1 achieved the highest load of 36,188 lbs, with the High Load Wall following with a load of 33,407 lbs. A summary of each wall’s behavior at the load benchmarks is provided below in Table 3.
From the results, it is estimated that Mega Wall 2 should have accumulated a greater load than what was achieved during testing. See Appendix A, Section 5.1 “Load Benchmark Comparison” for specific discussion about the results of testing, ultimate capacities, and behavior for the Conventional Wall and Mega Walls. The backbone curve for each wall is plotted in Figure 8 below and shows that the backbone curve of the High Load Wall followed a similar trend to the backbone curve of Mega Wall 1. The sudden failure of the High Load Wall paired with backbone curve behavior similar to that of Mega Wall 1 suggest the High Load Wall has a greater capacity than what was achieved in this test. If premature failure had not occurred and the High Load Wall continued with similar behavior, the backbone curve may have continued to extend until a similar ultimate load to Mega Wall 1.

![Figure 8. Backbone curves of all specimens](image)

\(^1\)Mega Wall Test 1 and Test 3 combined into one curve for continuity. See Mega Wall 1 Results and Limitations section in Appendix A, Section 5.2 for further explanation.

**Specimen Stiffness Comparison**

Using the ASTM E2126 equivalent energy elastic-plastic (EEEP) methodology, stiffness values were derived for the conventional NDS Wall, Mega Wall 2 and the High Load Wall. Since a quantifiable curve could not be created for Mega Wall 1, linear stiffness values were determined for each specimen to compare the stiffnesses of the specimens. See Appendix A, Section 5.2 “Specimen Stiffness Comparison” for further discussion about the limitations of stiffness calculations for Mega Wall 1. As shown in Table

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**Table 3. Results at Load Benchmarks**

<table>
<thead>
<tr>
<th></th>
<th>Conventional Wall</th>
<th>Mega Wall 1 (^2)</th>
<th>Mega Wall 2</th>
<th>High Load Wall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement at 100% Design Load (6,960 lbs)(^1)</td>
<td>0.4849”</td>
<td>0.3307”</td>
<td>0.3733”</td>
<td>0.3507”</td>
</tr>
<tr>
<td>Displacement at 200% Design Load (13,920 lbs) (^1)</td>
<td>1.0505”</td>
<td>0.8215”</td>
<td>0.7860”</td>
<td>0.7203”</td>
</tr>
<tr>
<td>Displacement at 400% Design Load (27,840 lbs) (^1)</td>
<td>n/a</td>
<td>2.2133”</td>
<td>1.8950”</td>
<td>1.9613”</td>
</tr>
<tr>
<td>Displacement at Ultimate Load</td>
<td>3.1285”</td>
<td>5.2432”</td>
<td>2.6157”</td>
<td>3.6207”</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>24,540 lbs</td>
<td>36,188 lbs</td>
<td>32,086 lbs</td>
<td>33,407 lbs</td>
</tr>
</tbody>
</table>

\(^1\)Displacements at exact design load values were found through linear interpolation of measured data.

\(^2\)100% and 200% design load displacements were obtained from Test 1 of Mega Wall 1 to capture the behavior at the first instance of these loads. Ultimate load and displacement were obtained from data collected during Test 3.
4, the EEEP stiffness value and the 100% Design Load linear stiffness values differ by an insignificant amount, suggesting that the linear stiffness values are a valid metric to compare the specimens.

The High Load Wall and both Mega Wall design show a clear increase in stiffness from the conventional NDS Wall. This increase in stiffness demonstrates that the Mega Wall and High Load Wall design are more elastic than the conventional design. The stiffness and elasticity between the Mega Wall and High Load design did not differ notably.

Table 4. Wall Stiffness Values

<table>
<thead>
<tr>
<th></th>
<th>100% Design Load Linear Stiffness Value</th>
<th>% Difference from Conventional(^1)</th>
<th>EEEP Stiffness Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional NDS Wall</td>
<td>14.52 k/in</td>
<td>N/A</td>
<td>14.92 k/in</td>
</tr>
<tr>
<td>Mega Wall 1</td>
<td>20.71 k/in</td>
<td>43%</td>
<td>N/A</td>
</tr>
<tr>
<td>Mega Wall 2</td>
<td>18.47 k/in</td>
<td>27%</td>
<td>18.39 k/in</td>
</tr>
<tr>
<td>High Load Wall</td>
<td>19.54 k/in</td>
<td>35%</td>
<td>19.41 k/in</td>
</tr>
</tbody>
</table>

\(^1\) Percent difference of stiffness between Mega Walls and Baseline Wall using 100% Design Load Linear Stiffness values.

The linear stiffness based on 100% Design Load are presented graphically for each specimen in Figure 9 below. The stiffness curves are extended past 100% Design Load to clearly illustrate the difference in stiffness between each wall. For discussion of the Mega Wall 2’s decrease in stiffness as compared to Mega Wall 1 see Appendix A, Section 5.2 “Specimen Stiffness Comparison.” The High Load Wall proved to be 35% stiffer than the conventional NDS Wall which validated its design as a stiffer and more elastic option when compared against the conventional wall.

Figure 9. Specimen Backbone Curves and Relative Linear Stiffness

\(^1\) Mega Wall Test 1 and Test 3 combined into one curve for continuity. See Mega Wall 1 Results and Limitations section for further explanation.
EEEP Curve Comparison

The purpose of this discussion is to compare the energy dissipation and ductility of each shear wall design. Using ASTM 2126, EEEP curves were created for each specimen, excluding the Mega Wall 1 for reasons previously stated. The curves in Figure 10 depict idealized elastic-plastic curves that circumscribe the area enclosed by the envelope or backbone curve. Areas under the EEEP curves are quantified in Table 5. below. As expected, the conventional NDS wall, achieving the lowest ultimate load dissipated the least amount of energy. What is more intriguing is the results for Mega Wall 2, Mega Wall 2 dissipated about 10% more energy despite having an ultimate load that was 31% greater than the conventional NDS wall. Since Mega Wall 2 did not absorb a significant amount of energy greater than the conventional wall, the Mega Wall 2 design cannot be characterized as a more ductile system. In contrast the High Load Wall absorbed 69% more energy while achieving a load that was 36% greater than the conventional NDS wall. When compared to the Mega Wall 2, the High Load Wall dissipated 53% more energy while only increasing in ultimate capacity by 4%. The High Load design wall clearly demonstrated enhanced ductility than both the Mega Wall design and the conventional NDS wall.

Table 5. Area under EEEP Curve

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Area (lb-in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional NDS Wall</td>
<td>48,764</td>
</tr>
<tr>
<td>Mega Wall 2</td>
<td>53,827</td>
</tr>
<tr>
<td>High Load Wall</td>
<td>82,602</td>
</tr>
</tbody>
</table>

![High Load Wall](image1)

![Mega Wall 2](image2)

![Conventional NDS Wall](image3)

\*Due to the limitations of Mega Wall 1 testing, a quantifiable EEEP curve could not be produced.

Figure 10. Specimen EEEP Curves
The ductility of the specimens can also be compared using ASTM E2126’s ductility ratio (D). This is a ratio of the specimen’s ultimate displacement ($\Delta_u$) versus its yield displacement ($\Delta_{yield}$) during testing. The conventional NDS wall has the highest ductility ratio with the High Load wall following after. As seen before with the area under the EEEP curves, the conventional NDS and High Load Wall tend to be more ductile systems when compared against Mega Wall 2. The Conventional NDS wall’s ductility ratio is about 1.75 times greater than the conventional wall. See Table 6 below for further comparison.

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Ductility Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conventional NDS Wall</td>
<td>3.051</td>
</tr>
<tr>
<td>Mega Wall 2</td>
<td>1.750</td>
</tr>
<tr>
<td>High Load Wall</td>
<td>2.606</td>
</tr>
</tbody>
</table>

**Discussion and Conclusions**

The tests show that metal gauge strapping and high load diaphragm nailing patterns are an effective method to increase the strength of single-sided plywood shear walls. This was evident by a roughly 49%, 32%, 36% increase in load capacity in Mega Wall 1, Mega Wall 2, and High Load Wall, respectively, compared against the conventional NDS wall. Using the safety factor of 3.5, demonstrated by the Conventional NDS Wall, the metal strapped wall designs have a design load of 10,340 lbs (1,293 plf) for Mega Wall 1, 9,170 lbs (1,146 plf) for Mega Wall 2, and 9,545 lbs (1,193 plf) for the High Load Wall. Although, the single sided reinforced walls did not reach the capacity of a double-sided conventional wall, 1,740 plf, they appear to offer a higher strength capacity than any available code specified single-sided plywood shear wall. The results from the Mega Wall 1 and Mega Wall 2 testing, illustrate the potential of utilizing metal gauge straps to increase the lateral load carrying capacity of conventional plywood shear walls. The High Load wall demonstrated the enhanced strength and ductility multiple lines of nailing add to a conventional shear wall. However, more research is necessary to further legitimize the use of these solutions in practice.

To eliminate or reduce observed failure modes of the tests previously conducted, new control variables can be introduced to future Mega Wall and High Load tests. Based on the results of Mega Wall 2, future research should consider the potential influence of stud spacing, increased field nailing (such as 6” on center field nailing rather than 12” on center), and panel thickness. Other modifications to the number of nails within the metal strap, nailing patterns, and size of metal strapping should also be investigated. Future testing of the high load wall should investigate the upsizing of the adjoining panel edge seam stud and size/number of plywood sheathing panels.

From a technical perspective, these findings have many positive implications on the engineering community due to the increased strength they lend to a common lateral system. From a less empirical standpoint, undergraduate research has been a unique and positive opportunity for many of the students involved and has positive implications on the academic community.

**Implications**

**Cultural**

This research has been the continuation of an ongoing project in ARCE 451, an upper division timber design course offered at Cal Poly, San Luis Obispo. Multiple iterations of this class have been involved in this Design, Build, Test (DBT) project. In each iteration, students participate in the hands-on process of
design, construction, and testing of large-scale shear walls. In addition to these experiences, students and faculty from other classes were encouraged to watch the live testing or observe the final tested specimens. Many civil and architectural engineering research projects do not feature research with significant participation or leadership from undergraduate students. Participating in research has many benefits for both students and university programs, however, the current culture surrounding academic research prevents many undergraduate students from accessing these benefits. In this way, the DBT research project contributes to the dominant academic culture by showing that undergraduate research can be completed at an equivalent caliber as graduate research and that a more diverse range of students learn from the process.

Furthermore, on a more personal note, the hands-on experience gained from this project as two young, female engineers has been a unique and invaluable opportunity. The construction industry is a male dominated field and as a result the culture around these tasks can make women feel slightly uncomfortable in the wood and machine shop environments. This project required many hours working in the shop, using many different tools, and an understanding of constructability. Having to confront these tasks and the discomfort of being in a new environment helped to break down these unspoken barriers of entry. This project improved our familiarity and skills with construction tasks that were once thought as daunting. Projects like these allow women to have a more equal footing in the construction environment. With luck, this project will inspire other women in the ARCE department to pursue technical fabrication projects.

**Constructability**

Both the High Load Wall and Mega Wall designs made use of typical wall framing with slight modification to the fasteners. Both these designs utilized common materials and construction techniques that are already familiar to many contractors and engineers.

Looking at the Mega Wall specifically, the metal strap made it easy to line up to the panel edge and determine the precise location of nails in a straight line. Although the strap made the nail pattern straightforward to follow, they are not compatible with nail guns translating to hundreds of nails having to be hammered by hand. On a job site, this is a major feasibility obstacle, especially if multiple walls are being constructed. Future research into this design could address this issue by studying the effect of metal strapping that is compatible with a nail gun.

Since the High Load Wall makes use of a design that already exists in code, there is already a familiarity with its construction in industry. Creating a template to locate the nails improved constructability while the nail gun eased the installation of the large sum of nails.

Currently, the High Load and Mega Wall design offer a higher load than any conventional code design. Implementing this design would allow engineers to choose a single sided shear wall where they would have had to design a double-sided shear wall in the past. This improves construction scheduling and overall constructability because less coordination between plumbing, insulation, and electrical trades is necessary during the wall’s construction.

**Economic**

Both the Mega Wall and High Load Wall design offer relatively cheap solutions to effectively increasing the strength of single-sided shear walls. In addition, both designs achieved ultimate capacities closer to that of a double-sided shear wall while avoiding the added costs associated with additional inspection,
phasing conflicts, and added material. The economic implications of this project are exciting because the lower price point of both designs means projects of all budgets have access to improved performance.

Social

Due to the constructability and economy of both wall designs, the Mega Wall and High Load wall designs make stronger single-sided shear walls accessible to a wider range of budgets. Efforts to make engineering accessible also work to make engineering more socially equitable.

Environmental

The increased strength and elasticity of the Mega Wall and High Load Wall designs are more resilient to damage. This resiliency means these systems are less likely to be critically damaged when loaded and are less likely to need to be demolished after a seismic or wind event. Improved resilience in structural systems requires less material and construction long term which reduces impact on the environment. In addition, these walls have comparable strength to double-sided shear wall systems but require half the plywood and less hardware which reduces material impact on the environment.

Acknowledgements

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References


Appendix A:

Increased Strength and Stiffness in Plywood Shear Walls Through Steel Straps

Cal Poly Pomona Conference Paper
INCREASED STRENGTH AND STIFFNESS IN PLYWOOD SHEAR WALLS THROUGH STEEL STRAPS

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In a recent iteration of an upper-division engineering design lab course at Cal Poly SLO, students were challenged to identify and understand the probable failure mechanism of conventionally built plywood shear walls and hypothesize how they could delay or prevent common failure mechanisms to increase the overall lateral load capacity. After observing the results of a test on a conventional, code specified wall, a solution to integrate metal gauge strapping into edge nailing was developed to address the common failure mechanisms of nail head pull through and panel edge tear out. A shear wall with metal strapping installed between the nails and plywood face at panel edges was designed, built, and tested to investigate the outcomes of this solution. The results of “Mega Wall 1” testing warranted further investigation and a similar, more robust system, “Mega Wall 2” was designed, built, and tested. The results of both tests show metal gauge strapping increases the ultimate capacity and stiffness of solid timber shear walls. The results of these tests are promising, and future areas of inquiry are discussed.

Keywords: metal gauge strapping; plywood shear walls, timber

1 Introduction

Lightweight wood-frame structures are widely used in residential buildings due to their cost-effectiveness and resilience to resist lateral loads. As timber becomes a more widely used material for mid-rise buildings, methods to increase the strength and stiffness of timber lateral systems should be evaluated. The main lateral load resisting systems for lightweight wood-frame structures are shear walls. Conventional lightweight wood-frame shear walls are sheathed with oriented strand boards (OSB), plywood panels, and/or gypsum wallboard. Sheathing can be applied to one side, referred to as single-sided, or sheathed on both sides, referred to as double-sided. Double-sided shear walls offer higher strength and stiffness than single-sided shear walls; however, because sheathing on both sides fully encloses the interior cavities of the wall, there are additional obstacles with construction phasing, insulation, coordination with MEP trades, and additional inspection requirements. Given the distinct advantages of utilizing single sided sheathed walls over double sided walls, there appears to be significant value in exploring alternatives to increase their strength.

Understanding how single-sided shear walls typically fail is the first step to improve their performance and address their limitations compared to double-sided walls. Students in an upper-division timber design laboratory at Cal Poly, San Luis Obispo have participated in an ongoing project to design, build, and test full-scale timber shear walls to identify and investigate these failure mechanisms. Before failure, yielding occurs in the nails and plywood and is evidenced by a combination of nail bending and wood crushing (Anderson, Lam). After yielding, common failure mechanisms of timber shear walls subjected to monotonic loading are: nail withdrawal, nail head pull through, and panel edge tear out. Many of these failure modes occur at the panel edge connections so it is important to study these common failure zones (Lam).

A possible solution to address nail head pull through is installing washers between the nails and plywood to increase the bearing area of the nail head. Despite the potential of this solution, the construction challenge of installing hundreds of separate washers in accurate locations limits the feasibility of this solution. Building on the idea of washers, the use of metal straps as a continuous washer between the nails and plywood panels could achieve a similar effect using common, easy to install materials with far less labor. To study the outcomes of this solution, three shear walls were designed, built, and tested: one conventional code specified wall and two walls with steel metal gauge straps integrated into the edge nailing. The details of this research and results of the testing are discussed in this paper.
To better understand how conventional shear walls perform, students in an upper-division timber design laboratory at Cal Poly, San Luis Obispo began their investigation with the design and testing of a conventional solid timber shear wall. The purpose of the conventional wall was to illustrate the behavior of a code-specified design, demonstrate failure mechanisms, and establish an empirical baseline to compare to the modified designs.

The design of the conventional wall was guided by the 2018 National Design Specification (NDS) for Wood Construction and the 2015 Special Design Provisions for Wind and Seismic (SDPWS). To understand the upper capacity and behavior of walls permitted by the NDS, the strongest single-sided shear wall configuration was selected.

### 2.1 Specimen description and design

To emulate a common wall geometry, the conventional wall was 8’ wide x 9’ 0-¾” tall. The lateral design capacity was determined from Table 4.3A of the SDPWS (1,740 plf) then divided by the relevant Allowable Stress Design (ASD) seismic reduction factor (2.0) to determine the reduced ASD capacity of 1,740/2 plf x 8’-0” = 6,960 lbs. To achieve this capacity, 15/32” Structural 1 sheathing, 10d common nails spaced at 2” on center at panel edges, common 10d field nailing at 12” on center, and a 3x stud at the adjoining panel edge were used as per Table 4.3A.

The focus of this investigation was to identify methods to address common failure modes, such as nail head pull through and panel edge tear out. To isolate the sheathing and/or nail shear failure mechanism, wall components such as the end posts, anchor bolts, and hold-downs were all designed to resist a higher load than the ASD capacity of the nails and sheathing.

The increased load factor applied to the design of these other wall components was derived from findings published in APA Report 154. Empirical results from testing timber shear walls of similar configurations had an average load factor of three (Tissell). Using this safety factor of three, the end posts, anchor bolts, and hold-downs were designed to the ultimate expected lateral load of 20,880 lbs. The mud sill was checked against this higher load and failed marginally in crushing perpendicular to grain, however, it was determined that this failure mechanism would not limit the shear wall from achieving its ultimate capacity. Based on this ultimate load the following members were used: No.1 DF-L 6x8 end posts, stud grade DF-L 2x6 studs, No.1 DF-L 2x6 double top plate, No. 2 DF-L pressure treated 2x6 mud sill, and four Simpson Strong-Tie HDU14 hold-downs which are illustrated in Figure 1.

![Wall Components Diagram](image)

**Figure 1. Conventional NDS Wall Specimen Diagram**

### 2.2 Test setup

The wall was constructed on a concrete slab-on-grade to provide a flat, supported surface before being tilted up and craned into place. Two 3’x6’ concrete foundations were post-tensioned down to an isolated strong floor and a wide flange foundation beam was bolted across these concrete footings to provide a base for the wall as shown in Figure 2 below. Anchor bolts were then welded to the top of the wide flange. Holes were pre-drilled through the mudsill of the wall to match the pattern of anchor bolts on the wide flange foundation beam. Once the framed wall construction was complete, the wall was lowered onto these anchor bolts and bolted down to create a connection with minimal slip. A wide flange loading beam was then attached to the top plates using SDS screws along the length of the wall to uniformly distribute the applied load. The load actuator was then connected to
the south end of the loading beam to apply monotonic lateral loading. Hold-downs were installed at the post of either end of the wall and bolted to the flange of the foundation beam.

As illustrated in the plan view of Figure 2, stabilizing steel wide flange beams were bolted into place on either side of the wall to provide precautionary lateral support of the wall, loading beam, and actuator in the case of unexpected out of plane forces. Wood shims were cut to specific dimensions and installed snug between the beam and specimen to ensure the wall was plumb and maintained its alignment throughout testing.

![Figure 2. Test Set-Up](image)

2.3 Instrumentation

The wall was instrumented to detect displacements at seven locations. A detailed illustration of the instrumentation is provided in Figure 3. Displacement potentiometers (abbreviated to pots.) were placed to measure lateral displacements of the footings, the foundation beam, the mud sill, and top plates on the northern side of the specimen (see instruments 1 through 5 in Figure 3). Lateral slip between the footings, foundation beam, and mud sill was calculated from these readings. Vertical linear displacement sensors were located at the base of the southern and northern posts (noted as instruments 6 and 7 on Figure 3) to measure the uplift and vertical compression of the wall. Lastly, a horizontal line was drawn between the two panels to visually illustrate relative panel racking denoted by note 8 in Figure 3.
1. String potentiometer (abbreviated to pot.) at the south end of top plate to measure lateral deflection.
2. Second string pot. at the south end of top plate to measure lateral deflection. Two instruments were installed for redundancy and added accuracy.
3. String pot. at mud sill to measure slip between mud sill and foundation beam.
4. String pot. at foundation beam to measure slip between beam and concrete foundations.
5. String pot. at southern concrete foundation to measure slip between foundation and shop floor.
6. Linear displacement sensor at northern post to measure vertical displacement.
7. Linear displacement sensor at southern post to measure vertical displacement.
8. Horizontal line to visually measure panel racking.

2.4 Testing and loading procedure

The conventional wall was loaded with a monotonic push in the northern direction following guidelines outlined in APA Report 154 (Tissell) and ASTM E72-15 using an MTS (Material Tests System) hydraulic actuator. The wall was subjected to two force-controlled load cycles: one to 100% Design Load (6,960 lbs) and a second to 200% Design Load (13,920 lbs). During the force-controlled loading procedure, the wall was pushed to a prescribed load then unloaded. After these force-controlled cycles, the wall was subjected to a series of displacement-controlled loading cycles until failure occurred. During displacement-controlled loading cycles, the wall was pushed to a prescribed deflection then unloaded. Failure was marked by a sudden decrease in load capacity and/or observation of severe damage. A summary of the testing protocol for the conventional wall is provided in Table 1.

As per the ASTM E564 protocol, displacement-controlled loading increments were estimated to best capture the ultimate strength of the wall. Between each displacement-controlled load cycle, the wall was pulled back to a specified residual displacement based on the behavior from the previous load cycle. During analysis of the data, it was determined that there was an error in the loading procedure because the wall should not have been pulled back to a specified residual displacement and should have been left unloaded between cycles.
Table 1. Conventional Wall – Loading Protocol

<table>
<thead>
<tr>
<th>Force Controlled</th>
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<tbody>
<tr>
<td>1) Push to 100% Design Load</td>
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<tr>
<td>2) Hold load for 15 seconds</td>
</tr>
<tr>
<td>3) Unload to zero lbs applied and hold for 5 minutes</td>
</tr>
<tr>
<td>4) Push to 200% Design Load</td>
</tr>
<tr>
<td>5) Hold Load for 15 seconds</td>
</tr>
<tr>
<td>6) Unload to zero lbs applied</td>
</tr>
<tr>
<td>7) Pull to residual net displacement and hold for 5 minutes</td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Switch to Displacement Controlled</td>
</tr>
<tr>
<td>8) Push wall to specified displacement benchmark</td>
</tr>
<tr>
<td>9) Hold Displacement for 15 seconds</td>
</tr>
<tr>
<td>10) Unload to zero lbs applied</td>
</tr>
<tr>
<td>11) Pull to residual displacement benchmark and hold for 5 minutes</td>
</tr>
<tr>
<td>12) Repeat until failure is reached</td>
</tr>
</tbody>
</table>

2.5 Results and analysis

The conventional NDS wall achieved a maximum load of 24,540 lbs before a sudden drop in strength. Although load cycles were applied monotonically in one direction, the negative forces shown in Figure 4 were caused by the test protocol error described in section 2.4 “Testing and loading procedure” when the wall was pulled to a specified residual displacement.

The results from the conventional wall validated the decision to design components using the estimated APA safety factor of three. Comparing the design load of 6,960 pounds to the ultimate empirical load of 24,540 pounds (illustrated in Figure 4) resulted in a safety factor of roughly 3.50.

![Figure 4. Conventional NDS Wall – Force vs. Deflection](image)

2.5.1 Conventional wall failure mechanisms

The wall failed due to nail withdrawal, nail head pull through, and tear out originating at the panel edges. In the lower tension zone of the southern panel, the nail heads began to embed into and pull through the plywood during initial load cycles. At the ultimate load, the forces in this tension zone increased and caused panel edge tear out, pictured at left in Figure 5. Nail withdrawal was concentrated at the vertical wall boundaries, shown at right in Figure 5. Mud sill crushing occurred at the northern (compressive) end of the wall. Significant racking between panels was visually observed during loading and unloading and there was permanent displacement between the two panels after testing was complete. After testing, there was no noticeable damage.
to the plywood sheathing aside from damage caused by panel edge tear out. These failure modes produced a gradual failure, and the wall relatively maintained its form during and after failure.

![Figure 5. Panel edge nail pull through and panel displacement (Left); Nail pullout and deformation (Right)](image)

3 Metal Gauge Strap Wall 1
Testing of the conventional wall resulted in isolated damage through nail withdrawal, nail head pull through and tearing at the plywood edge, otherwise the plywood panels remained undamaged. These failure modes were concentrated at panel boundaries of the conventional wall and not in field nailing. Based on these findings, an effective strategy to increase the strength of timber shear walls is to delay failure caused by nail head pull through and prevent panel edge tear out. The integration of a metal gauge strap as a continuous washer between the nail heads and plywood panels could prevent nail head embedment and subsequent failure by increasing bearing area of the nail head onto the plywood. The metal gauge strap wall 1 (abbreviated to “Mega” Wall 1) integrates metal strapping into the edge nailing.

3.1 Specimen description and design
In order to isolate the influence of the added metal strap, Mega Wall 1 had identical geometry, framing members, hold-downs, anchor bolts, field nailing, and sheathing as the conventional wall. CMSTC16 strap was selected to reinforce the top, left, right, and adjoining panel edges as illustrated in Figure 6. CS16 strap was selected to reinforce the lower panel edge because the CMSTC16 strap had a wider nailing pattern than the 2x6 mud sill could accommodate. CMSTC16 and CS16 straps come with pre-punched hole patterns for nails. Since CMSTC16 and CS16 strap have a prefabricated pattern, a pattern was created to emulate the spacing and total number of nails in the conventional wall, 10d common nails were installed according to the pattern shown in Figure 6.

The ultimate capacity of Mega Wall 1 was expected to have an increased capacity compared to the conventional wall. This new capacity was determined according to the new expected failure modes. Research based on past testing shows that the expected design yield mode of nails in shear walls is NDS Mode III (NDS 2018 Figure I1; Anderson). Assuming the strap eliminates nail head pull-through as a failure mechanism and the nails instead engage in a Mode III limit state first, the wall is expected to have an 11% increase in strength. With this 11% increase, Mega Wall 1 was expected to have a design load of 7,726 lbs and an ultimate capacity of 23,177. All other framing elements were checked against this increased capacity and found to be adequate.
3.2 Test setup and instrumentation

Mega Wall 1 had an identical test setup and identical instrumentation as the conventional wall. For a diagram of the test setup, see Figure 2 in section “2.2 Test Setup.” For an illustration and description of instrumentation, see Figure 3 in section “2.3 Instrumentation.”

3.3 Testing and loading procedure

The testing protocol for Mega Wall 1 was modeled after recommendations in APA Report 154 and ASTM E72-15. Mega Wall 1 was initially pushed monotonically in the northern direction using an MTS hydraulic actuator. The wall was subjected to two force-controlled cycles: first to 100% Design Load (6,960 lbs), then to 200% Design Load (13,920 lbs). The wall was then subjected to displacement-controlled loading cycles until failure occurred. Testing ended before failure occurred due to issues caused by out of plane buckling of the actuator at high loads.

For Test 2 of the same Mega Wall 1 specimen, loading was switched to monotonic pulling rather than pushing to avoid out of plane buckling. The wall was subjected to two cycles of force-controlled loading to 100% and 200% Design Load using the same procedure from Test 1. The wall was then subjected to displacement-controlled loading. Test 2 ended before failure occurred because the actuator reached its full displacement stroke before ultimate capacity could be achieved.

The actuator stroke was reconfigured to accommodate larger displacements and the same Mega Wall 1 specimen was subjected to a final test. During Test 3 of Mega Wall 1, the wall was pulled monotonically. The wall was only loaded to 100% Design Load before switching to the displacement-controlled procedure to avoid excessive fatigue of the nails. The wall was subjected to three displacement-controlled load cycles before ultimate capacity was achieved.

In total, the same Mega Wall 1 specimen was subjected to three total tests: Test 1- push, Test 2- pull, and Test 3- pull. Between each test, long periods of time elapsed where Mega Wall 1 remained unloaded. The protocol for all three tests is summarized in Table 2.
Table 2. Mega Wall 1 – Loading Protocols

<table>
<thead>
<tr>
<th>Test 1 – Push</th>
<th>Test 2 – Pull</th>
<th>Test 3 - Pull</th>
</tr>
</thead>
<tbody>
<tr>
<td>Force Controlled</td>
<td>Force Controlled</td>
<td>Force Controlled</td>
</tr>
<tr>
<td>1) Push to 100% Design Load.</td>
<td>1) Pull to 100% Design Load.</td>
<td>1) Pull to 100% Design Load.</td>
</tr>
<tr>
<td>2) Hold load for 15 seconds.</td>
<td>2) Hold load for 15 seconds.</td>
<td>2) Hold load for 15 seconds.</td>
</tr>
<tr>
<td>3) Unload to 0 lbs applied and hold for 5 minutes.</td>
<td>3) Unload to 0 lbs applied and hold for 5 minutes.</td>
<td>3) Unload to 0 lbs applied and hold for 5 minutes.</td>
</tr>
<tr>
<td>4) Push to 200% Design Load.</td>
<td>4) Pull to 200% Design Load.</td>
<td>Switch to Displacement Controlled</td>
</tr>
<tr>
<td>5) Hold Load for 15 seconds.</td>
<td>5) Hold Load for 15 seconds.</td>
<td>4) Load wall to specified displacement benchmark</td>
</tr>
<tr>
<td>6) Unload to 0 lbs applied and hold for 5 minutes.</td>
<td>6) Unload to 0 lbs applied.</td>
<td>5) Hold Displacement for 15 seconds</td>
</tr>
<tr>
<td>Switch to Displacement Controlled</td>
<td>Switch to Displacement Controlled</td>
<td>6) Unload to 0 lbs applied</td>
</tr>
<tr>
<td>7) Load wall to specified displacement benchmark.</td>
<td>7) Load wall to specified displacement benchmark.</td>
<td>7) Push to residual displacement benchmark measured from preceding cycle.</td>
</tr>
<tr>
<td>8) Hold Displacement for 15 seconds.</td>
<td>8) Hold Displacement for 15 seconds.</td>
<td>8) Hold displacement for 5 minutes.</td>
</tr>
<tr>
<td>9) Unload to 0 lbs applied and hold for 5 minutes.</td>
<td>9) Unload to 0 lbs applied.</td>
<td>9) Repeat until ultimate capacity achieved.</td>
</tr>
<tr>
<td>10) Pull to residual displacement measured at 200% Design Load.</td>
<td>10) Push to residual displacement benchmark measured at 200% Design Load.</td>
<td>10) Push to residual displacement benchmark measured at 200% Design Load.</td>
</tr>
<tr>
<td>11) Hold displacement for 5 minutes.</td>
<td>11) Hold displacement for 5 minutes.</td>
<td>11) Hold displacement for 5 minutes.</td>
</tr>
<tr>
<td>12) Test 1 ended due to out of plane buckling.</td>
<td>12) Test 2 ended due to inadequate actuator stroke.</td>
<td>12) Test 2 ended due to inadequate actuator stroke.</td>
</tr>
</tbody>
</table>

3.4 Results and analysis

Despite the testing complications, Mega Wall 1 still yielded interesting results. Hysteresis curves were developed for the three-test series and analyzed. Backbone curves were also created for each test series to better understand the wall’s performance and relative behavior between cycles regardless of the direction of loading. Failure mechanisms are also discussed in detail to gain a thorough interpretation of the metal strap’s effect on the wall’s physical behavior.

3.4.1 Limitations and results

Due to the reversal of loading on Mega Wall 1 and the necessity for multiple tests before the ultimate capacity was achieved, the hysteresis of Mega Wall 1 was different than expected. As illustrated in Figure 7, the hysteresis of Test 1 becomes nonlinear during the 200% Design Load cycle, which indicates the nails and plywood began to yield in the direction of the push loading. During Test 2, the hysteresis in Figure 7 shows a significant reduction in stiffness with the load reversal and an unexpected increase in stiffness during Test 3. This reduction in stiffness is theorized to be attributed to the reversal in load during Test 2. Since the nails and plywood had yielded in the push direction, they required less force to displace back to a neutral position under the opposite pull loading during Test 2. Once the residual deflection from Test 1 had been reversed, the nails began to engage in load resistance in the pull direction during Test 3, which caused the increase in stiffness. Further inspection of the hysteresis shows clear pinching at every load cycle during Test 3. Pinching is a characteristic response of timber shear walls at larger deformations and serves as evidence that the nails and plywood began to yield and effectively engaged in force resistance during Test 3 (Van de Lindt).
The backbone curves of Mega Wall 1’s Test 1 and Test 3 follow a similar shape and intersect at about 22,000 lbs as illustrated in Figure 8. Once the wall was displaced in the new pull direction and the nails were effectively engaged, the overlap of Test 1 and Test 3 suggest symmetrical strength in either load direction. For the purpose of comparison, Test 1 and Test 3 are combined into a single, continuous backbone curve. The backbone curves from each test are illustrated in Figure 8.

Despite the limitations caused by testing, the failure mechanisms and behavior at load benchmarks of Mega Wall 1 still provide valuable information about the general effect metal gauge strap has on the performance of timber shear walls and is relevant to the discussion of results. The results of Mega Wall 1 suggested that the “Mega Wall” design needed further inquiry, so a second iteration, Mega Wall 2, was developed.

Prior to testing, the initial estimated increase in strength for Mega Wall 1 due to the addition of the metal strap was approximately 11% which equated to an ultimate capacity of 23,177. Mega Wall 1 ultimately failed at a load of 36,188 lbs, which is a 47% increase from the ultimate load achieved by the conventional wall. The drastic difference between the expected and empirical increase is likely caused by the decrease in the displacement and overall racking of the wall panels inducing less bending and pull through of the nails.
3.4.2 Mega wall 1 failure mechanisms

Failure modes were detected to be nail withdrawal, buckling and deformation of the metal gauge strap, and mud sill splitting. The addition of the metal strap effectively eliminated some of the previous failure mechanisms seen in the conventional wall, as there were no instances of nail head pull through or panel edge tear out.

During the final Test 3 of Mega Wall 1, the specimen was pulled in the southern direction which generated a compression zone in the lower, southern corner of the wall. Significant buckling of the straps was observed at this location as well as at the adjoining panel edge. As the plywood racked at high displacements, the sheathing began to bear on the foundation beam below and eventually crushed. The high lateral loads also generated high axial forces in the straps which caused the CS16 strap to buckle out of plane, pictured at left in Figure 9. The CS16 strap at the mudsill panel edge was narrower than the CMSTC16 strap installed at all other panel edges and may have caused premature failure. Noticeable buckling also occurred at the metal gauge strap along the mud sill at the adjoining panel edges as seen in the bottom picture of Figure 9.

Compared to the conventional NDS Wall, the two panels displaced together and slight racking between panels was observed. Metal strapping added rigidity to the panels which reduced panel racking. Since the two panels displaced together, the nails could not displace as much, and yielding was delayed.

Nail pullout was more exaggerated at the mudsill when compared to the top plate. The nails at the mud sill experienced greater curvature and translation as shown in the bottom picture of Figure 9 than the top plate panel connections. The shape of the nail yielding is consistent with that of the NDS yield Mode IV.

Splitting of the mud sill perpendicular to grain was observed at the tension side of the wall. The cross section had been reduced at northern side of the mud sill to accommodate the hold-down rods which may have reduced strength at this location. In addition, the metal strap prevented the nail heads from pulling through as the panels racked. With the metal strap installed, the nails displaced with the panels vertically and split the mud sill perpendicular to grain near the reduced cross section.

![Image](image1.png)

![Image](image2.png)

![Image](image3.png)

Figure 9. Buckling of CS16 strap and plywood crushing at compression zone at base of wall (Top Left); Buckling of CS16 strap at base of adjoining panel edge (Top Right); Notable nail withdrawal and yielding at mud sill (Bottom)

4 Metal Gauge Strap Wall 2

The results of Mega Wall 1 suggested that the “Mega Wall” design needed further inquiry. Metal Gauge Strap Wall 2 (Mega Wall 2) was developed to further investigate the potential of utilizing metal gauge strapping in design. To build on this concept, failure mechanisms exhibited in Mega Wall 1 were addressed in the design of Mega Wall 2 to try to generate failure in the plywood sheathing. Specific wall components were modified to ensure that Mega Wall 2 could reach its full capacity.
4.1 Specimen description and design

Mega Wall 2 possessed the same dimensions as the previous walls and similarly the secondary components (posts, hold downs and mudsill) were designed to not be the expected controlling failure mechanism. Due to the buckling of the CS16 strap at the mud sill previously described, the mud sill strap was replaced with a more robust CMSTC16 strap to prevent this failure occurring again in Mega Wall 2. As denoted in Figure 10, the mud sill size was increased from a No. 2 BTR DF-L 2x6 PT to a No. 2 BTR DF-L 4x6 PT to accommodate the larger strapping. Nails were installed in every hole of the prefabricated strap to further reduce buckling and achieve the full capacity of the CMSTC16 strapping. This resulted in a greater number of total nails installed than in the conventional NDS shear wall. Additionally, the outermost studs were installed at a 16” from the center of the end post, which increased the outer stud bay width to allow for easier installation of the hold-downs. A total of two HDU14 hold-downs were symmetrically installed at the north end of Mega Wall 2 to resist a pull in the southern direction as shown in note 6 of Figure 10.

![Figure 10. Mega Wall 2 Specimen Diagram](image)

Wall Components
1. (2) 2x6 Double Top Plates
2. CMSTC16 Strapping (throughout)
3. 3x6 Adjoining Panel Stud
4. (2) 6x8 Posts
5. (4) 2x6 Studs (spacing changed from previous specimens)
6. (2) HDU14’s
7. 4x6 PT Mud Sill
8. (8) Anchor Bolts

4.2 Test set up

Besides slight adjustments, the test set up for Mega Wall 2 was identical to the test set up described in section 2.2 “Test Setup of the conventional wall”. Two additional anchor bolts were added to the foundation beam to account for the expected increase in capacity for Mega Wall 2 (shown in Figure 10 above). Stud spacing was changed to accommodate additional anchor bolts and aid constructability. See Figure 2, for the test set up diagram.

4.3 Instrumentation

Instrumentation of Mega Wall 2 was identical to instrumentation in the previous test specimens with the addition of two new sensors. In addition to the visual tool (the horizontal line drawn between the two panels) to illustrate panel racking, two linear displacement sensors (noted as instruments 8 and 9 on Figure 11) were installed to quantitatively measure displacement between panels.
1. String potentiometer (abbreviated to pot.) at the south end of top plate to measure lateral deflection.
2. Second string pot. at the south end of top plate to measure lateral deflection. Two instruments were installed for redundancy and added accuracy.
3. String pot. at mud sill to measure slip between mud sill and foundation beam.
4. String pot. at foundation beam to measure slip between beam and concrete foundations.
5. String pot. at southern concrete foundation to measure slip between foundation and shop floor.
6. Linear displacement sensor at northern post to measure vertical displacement.
7. Linear displacement sensor at southern post to measure vertical displacement.

Two additional instruments installed on Mega Wall 2 to measure relative displacement of both panels.

8. Linear displacement sensor to measure vertical displacement of southern plywood panel.
9. Linear displacement sensor to measure vertical displacement of northern plywood panel.

4.4 Loading and testing procedure

Mega Wall 2 was loaded with a monotonic pull in the southern direction following the guidelines outlined in APA Report 154 and ASTM E72-15 using the MTS hydraulic actuator. The wall was subjected to three force-controlled load cycles: one to 100%, 200%, and 400% Design Load. The third force-controlled cycle corresponds to 200% Design Load of a double-sided shear wall for purposes of direct comparison. Once the three benchmarks were satisfied, the testing protocol was switched to displacement control. See Table 3 for a detailed description of the Mega Wall 2 loading protocol. When sudden failure was observed, the protocol was terminated, and the wall loading was paused at the force associated with the sudden failure. The wall was held at this load for observation and then decreased back to zero applied force. Only one displacement-controlled load cycle was necessary to achieve ultimate failure.
Table 3. Mega Wall 2 – Loading Protocol

<table>
<thead>
<tr>
<th>Force Controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Pull to specified design load.</td>
</tr>
<tr>
<td>2) Hold load for 15 seconds.</td>
</tr>
<tr>
<td>3) Unload to zero lbs applied and let wall rest for five minutes.</td>
</tr>
<tr>
<td>4) Repeat for 200% Design Load and 200% Double-Sided Shear Wall Design Load.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Switch to Displacement Controlled</th>
</tr>
</thead>
<tbody>
<tr>
<td>5) Load wall to specified displacement benchmark.</td>
</tr>
<tr>
<td>6) Hold displacement for 15 seconds.</td>
</tr>
<tr>
<td>7) Unload to 0 lbs applied and let wall rest for five minutes.</td>
</tr>
</tbody>
</table>

4.5 Mega Wall 2 results and analysis

Figure 12 depicts the hysteresis curve for Mega Wall 2 and shows the four load cycles applied before failure. Ultimate capacity was achieved in one series of tests which resulted in a single, continuous hysteresis curve pictured in Figure 12, in contrast to the three discontinuous curves produced by Mega Wall 1 in Figure 7. Mega Wall 2 underwent sudden failure, illustrated by the dramatic decrease in force in Figure 12. After failure, Mega Wall 2 continued to support a significant but lower load with increased displacement.

![Mega Wall 2 Hysteresis Curve](image)

Figure 12. Mega Wall 2 – Force vs. Deflection

4.5.1 Mega Wall 2 failure mechanisms

Mega Wall 2 displayed unique and uncharacteristic failure mechanisms compared to the previous specimens. Both buckling (south panel) and a horizontal shear rupture (north panel) occurred in the plywood sheathing. These failures occurred simultaneously and suddenly. In the third loading cycle, the southern sheathing panel began to bow at its lower left corner where the compressive forces were greatest. Since the bowing occurred in the corner where the vertical and horizontal metal gauge straps met, the panel was stiff enough to resist local buckling in this zone. In the final load cycle, the panel bowing began to expand toward the center of the panel at a diagonal. The plywood finally buckled out of plane at the location where the unbraced...
length between stiff panel edge nailing and field nailing was greatest. Once the southern panel buckled out of plane, the entire lateral applied load to the wall was forced into the northern panel which caused a sudden shear rupture, which subsequently caused the southern buckled panel to rupture as well. Inspection of this horizontal rupture shows it occurred along a horizontal line of knots near the bottom of the wall as depicted in the lower right of Figure 13(b).

The vertical metal gauge strap at the northern panel displayed clear deformation and displacement induced by the panel shearing as well as slight torsional deformation, depicted in Figure 13(a). It is likely the northern panel corner twisted to counteract the out of plane movement produced by the southern panel’s buckling.

After the test terminated, it was discovered that the field nailing between the panel edge nails and the first row of stud field nailing was greater than 16” on center but less than the code maximum of 24” on center. Although the studs were spaced at 16” on center from the center line of the end post, the unbraced distance between the panel edge nailing and first row of field nailing was about 2-3” greater. This increased distance between the first line of field nails and the panel edge nails by roughly 2-3”, which increased the unbraced length between nails within the outermost stud bays to about 19” on center. Under high compressive force, the southern panel in this region provided less resistance and stiffness to out of plane forces becoming a probable cause of the southern panel buckling out of plane. In the panel buckling zone, three field nails pulled through the plywood sheathing, unable to resist the forces of the bowing sheathing. Mega Wall 2 was expected to achieve a higher load than Mega Wall 1, however, this uncommon failure mechanism suggests Mega Wall 2 failed prior to reaching its ultimate capacity.

The influence of stud spacing is accounted for in the NDS design guidelines and has been investigated by the APA. Footnote 2 of Table 4.3A of the SDPWS discusses the reduction in shear strength for walls with both (1) panels thinner than 15/32” and (2) stud spacing greater than 16” on center. Empirical findings from APA Report 154 support the application of this reduction in walls with the above listed criteria (1 & 2) due to the tendency of thin panels (panels thinner than 15/32”) to buckle. However, the buckling failure observed in the 15/32” sheathing of Mega Wall 2 demonstrated that stud spacing greater than 16” on center directly causes a reduction in shear capacity when under higher lateral force than a conventional shear wall is capable of. Since the buckling behavior of Mega Wall 2 was not consistent with APA guidelines, further examination should be conducted to specifically study the effect of stud spacing for walls such as the metal gauge walls that achieve higher loads than conventional code walls.

5 Overall Results and Analysis

The results for the Conventional NDS Wall, Mega Wall 1, and Mega Wall 2 are discussed below. Maximum forces and displacements are identified and compared for each specimen. From these results, linear stiffness values are calculated, and failure mechanisms are discussed to better understand how the specimens performed.
5.1 Load benchmark comparisons

The results indicate that both modified Mega Wall designs had a higher ultimate capacity and smaller corresponding deflection as compared to the Conventional NDS wall. The conventional NDS Wall achieved an ultimate load of roughly 24,000 lbs while Mega Wall 1 and 2 achieved an ultimate load of roughly 36,000 lbs and 32,000 lbs respectively. A summary of each wall’s behavior at the load benchmarks is provided below in Table 4.

Table 4. Results at Load Benchmarks

<table>
<thead>
<tr>
<th></th>
<th>Conventional Wall</th>
<th>Mega Wall 1</th>
<th>Mega Wall 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Displacement at 100% Design Load (6,960 lbs)(^1)</td>
<td>0.4849”</td>
<td>0.3307”</td>
<td>0.3733”</td>
</tr>
<tr>
<td>Displacement at 200% Design Load (13,920 lbs)(^1)</td>
<td>1.0505”</td>
<td>0.8215”</td>
<td>0.7860”</td>
</tr>
<tr>
<td>Displacement at Ultimate Load</td>
<td>3.1285”</td>
<td>5.2432”</td>
<td>2.6157”</td>
</tr>
<tr>
<td>Ultimate Load</td>
<td>24,540 lbs</td>
<td>36,188 lbs</td>
<td>32,086 lbs</td>
</tr>
</tbody>
</table>

\(^1\) Displacements at exact design load values were found through linear interpolation of measured data.

\(^2\) 100% and 200% design load displacements were obtained from Test 1 of Mega Wall 1 to capture the behavior at the first instance of these loads. Ultimate load and displacement were obtained from data collected during Test 3.

Mega Wall 1 and Mega Wall 2 experienced similar ultimate loads yet differed significantly in displacement at ultimate load. Since Mega Wall 2 was similar in structure to Mega Wall 1 and was projected to be even stronger than Mega Wall 2, results from the Mega Wall 2 lead to greater discussion.

Mega Wall 2 experienced unexpected and immediate failure when the plywood sheathing buckled out of plane due to the increased unbraced length in the panel (caused by the increased field nail spacing discussed in section 4.5.1 “Mega Wall 2 failure mechanisms”). The backbone curves from Mega Wall 1 and Mega Wall 2 also yield intriguing results shown in Figure 14. The failure of Mega Wall 2 is directly illustrated in its short backbone curve as compared to the elongated backbone curve of Mega Wall 1. Upon further observation, the backbone curvature of Mega Wall 2 follows just above the backbone curve of Mega Wall 1. From this pattern, it can be assumed that if Mega Wall 2 did not undergo unexpected failure, its backbone curve would have followed more closely to the backbone curve of Mega Wall 1. With this assumption, Mega Wall 2 would have achieved a higher load than Mega Wall 1 at Mega Wall 1’s ultimate displacement. It is speculated that Mega Wall 2 has greater capacity than what was achieved during testing. Further testing should be conducted to investigate this matter.

\(^1\) Mega Wall Test 1 and Test 3 combined into one curve for continuity. See Mega Wall 1 Results and Limitations section for further explanation.

Figure 14. Backbone curves of all specimens.
Appendix B :

Project Calculations
High Load Shear Wall #1

- Premise: Use nailing and blocking of NDS high load diaphragm in a shear wall application

<table>
<thead>
<tr>
<th>Sheathing</th>
<th>Structural 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Common Nails</td>
<td>10d</td>
</tr>
<tr>
<td>Fastener Penetration in Blocking</td>
<td>1½”</td>
</tr>
<tr>
<td>Panel Thickness</td>
<td>19/32</td>
</tr>
<tr>
<td>Width of Face at panel edges</td>
<td>4 x</td>
</tr>
<tr>
<td>Lines of Fasteners</td>
<td>3</td>
</tr>
<tr>
<td>Spacing at Diaphragm Boundaries</td>
<td>2½” o.c.</td>
</tr>
<tr>
<td>Other panel edges</td>
<td>3” o.c.</td>
</tr>
<tr>
<td>Y₀</td>
<td>3580 PLF</td>
</tr>
</tbody>
</table>

Not applicable for Case 6 since all edges are continuous

Load

- \( \frac{3580 \text{ PLF}}{2} = 1790 \text{ PLF} \)
- \( 1790 \text{ PLF} \times 8’ = 14240 \# \)
- Use safety factor of 3
- \( 14240 \# \times 3 = 42720 \# = 43 \text{ k} \)

Design Load: 43 k

- Diaphragm load case 6

Safety Factor = 3 based on APA testing results of NDS specified shear walls
Top Plate: 2 2x top plates

Mud Sill Plate: 4 x 6

Shear: \( 45^{\circ}/1 = \frac{5.025}{1} \text{ klf} \)

Assume Kd Common penny nails

\( P = 141^{\circ}(1.5) = 225.0 \text{#/} \)

Load Path: \( 45^{\circ}/2 = 22.5^{\circ} \)

\# of nails: \( 22.5^{\circ}/0.220^{\circ} = 100 \) nails

Spacing req'd for load // to grain: 1.5D

\[ 1.5(\frac{1}{3}) = 0.52^{\prime} \]

Spacing req'd for fasteners in arc\( \omega \):

\[ 4 \frac{1}{2} \]

End dist. reqd = 3.5D = 1.205\`

Edge dist reqd = 1.5D = 0.515\`

- Typical lap splice is 4\`
- Will use 2 rows of nails

Bearing on Sill Plate:

For stud grade 4 x 6

\( F_{ct} = 625 \text{ psi} \)

\( f_c = \frac{F_{ct}}{A} = \frac{51}{41.25} \text{ psi} \)

\( A = 41.25 \text{ in}^2 \)

\( f_c = \frac{123}{60} \text{ psi} \)

DC ratio = \( \frac{123 \text{ psi}}{60 \text{ psi}} = 1.98 \), this may be a limiting factor

\( f_c = \frac{51}{3} \text{(safety factor)} \)
Hold Down Posts:

\[ f_u = (450)(9 \times 3/4\text{ in}) \]
\[ f_u = 50,974 \text{ psi} \]

Material: Non-Cambered Beam Lumber

\( S_i = 8\) in

\[ A = \frac{41.25\text{ in}^2}{3.5^2} = 51.56\text{ in}^2 \]

Reference Design Values: [NBS 201B - Supplement Table 4D]

\[ f_c = 1,000 \text{ psi} \]
\[ E_{min} = 580,000 \text{ psi} \]
\[ f_e = 1,600,000 \text{ psi} \]
\[ d = 7.5'' \]

Factors:

\[ C_p = 1.6 \]
\[ C_u = 1.0 \]
\[ C_t = 1.0 \]
\[ C_l = 1.0 \]
\[ C_p = 0.837 \]

Cp Factor Determination: [NBS Eqn 3.7-1]

\[ C_p = \frac{1 + (F_{ce}/F_{c*})}{2c} = \sqrt{\left[1 + \left(\frac{F_{ce}/F_{c*}}{2c}\right)^2\right]} - \frac{F_{ce}/F_{c*}}{c} \]

\[ F_{ce} = 0.822 E_{min} \left(\frac{l_e}{d}\right)^2 \]

\[ l_e = 9\times 3/4'' - (2 \times 3.5'' - 3.5'') = 8.171'' \]

\[ \frac{l_e}{d} = \frac{8.171''}{12''} = 0.68 \]

\[ E_{min} = 580,000 \text{ psi} \]

\[ F_{c*} = (1,000 \text{ psi})(1.0) = 1,000 \text{ psi} \]

\[ C = 0.8 \text{ (sawn lumber)} \]

\[ C_p = \frac{1 + 1.701}{2(0.8)} = \sqrt{\left(1 + \left(1.701\right)^2\right)} - 1.701 \]

\[ C_p = 0.198 - \sqrt{(1.678)^2 - 2.13} \]

\[ C_p = 0.837 \]

\[ f_c' = 0.837(1600 \text{ psi}) = 1339.84 \text{ psi} \]

\[ f_c = 1236.36 \text{ psi} \]

\[ d/c = 0.923 \]

\[ f_c' = 1339.84 \text{ psi} \]
Holddowns

\[ T = \frac{43K(9')}{3'} \]

\[ T = 43.4K \]

- HDU 14 = 14,445 lbs
- Simpson Safety Factor 3

\[ 14.4K \times 3 = 43.2K \]

(2) HDU 14 Holddowns
Nail Spacing:

Rows of fasteners on a 4x member

10 penny common box nail
\[ d = 0.146'' \]

Spacing reqd // to grain: \[ 1.5d = 0.222'' \]
Spacing reqd for fastness in row: \[ 4d = 0.584'' = 0.592'' \]

end dist reqd: \[ 3.5d = 0.516'' \]
edge dist reqd: \[ 1.5d = 0.222'' \]

Necessary width:

\[ 2(0.222'') + 3(0.146'') + 2(0.516'') \]
\[ w = 1.724'' \text{ necessary} \]
\[ w < 3.5'' \]

4x is good for rows of fasteners.

Countersink:

\[ 4.5'' \times 3'' \]
\[ 3\frac{3}{16}'' \]
\[ 3.5'' - 1'' = 2\frac{1}{2}'' \]
Wall Anchorage:

3/4" Diameter Bolt w/ 2 1/2" Main Member Model w/ A36 Side Plate  
(NDS 12B)

Shear Failure Mode II:

\[ Z = 1230 \text{#} \]

\[ Z_{11} = (1230 \text{#})(0.46)(1)(0.8)(R_d) \]

\[ R_d \text{ (NDS 12.3.1.8):} \]

\[ m \lambda_3 = 4K_0 = 4(1.1) = 4 \]

\[ m \lambda_2 = 3.6K_0 = 3.6(1) = 3.6 \]

\[ m \lambda_{3s} = 3.2K_0 = 3.2(1) = 3.2 \]

\[ K_0 = 1 + 0.25(8/95) \]

\[ \theta = 0^\circ \text{ to grain} \]

\[ K_0 = 1 \]

\[ Z_{11} = (1230 \text{#})(1.6)(1)(0.8) \]

\[ Z_{11} = 1270 \text{#} \]

Demand = 141.32\#  \quad d/c = 1.17

---

- need 2 more anchor bolts
Loading Beam Fasteners

- Sds 1/4" x 3" Screws
- 1 1/2" DF Wood Side Plate

- Capacity:
  \[ C_p = 10.6 \]
  \[ \text{Safety Factor} = 3 \]
  \[ (3)(1.6)(280 \text{ #/screw}) = 1344 \text{ #/screw} \]

\[ \frac{48,000 \text{ #}}{1344 \text{ #/screw}} = 36.0 \rightarrow 32 \text{ screws} \]

- 28 screws existing @ 6" o.c. on both sides. Add 4 screws in off space
During Experiment: (Worst Case)

\[ M_c = 45 \times (9.16257) + T (16 + 18/12) = 0 \]
\[ T = 54.38\text{kN} \]
\[ C = -54.38\text{kN} \quad (\& \text{Fy} = 0) \]

Block Evaluation:

\[ \frac{d}{\text{A}} = \frac{\text{demand}}{2(\text{bolts})(\text{bolt capacity})} = \frac{54.38\text{kN}}{2(59.1\text{kN})} = 0.956 \quad \text{good} \]

Flexure of Beam Concerns:

\[ M_A = -54.38\text{kN}(1.6'' + 1.3'') + A_2 (5' + 6') + 54.38\text{kN}(3 + 6') \]
\[ A_2 = 73.01\text{kN} \]
\[ A_1 = 35.75\text{kN} \]

\[ M_D = 1.07 - 18.63\text{kN} \]

\[ M_D = 113.8\text{kN} \cdot \text{ft} \]

\[ 9 \geq 0.3897 \quad \sqrt{\text{W/C}} \leq \omega \]
\[ M_{\text{ind}} = (24\text{ in})(4\text{ in} / 2) = 8\text{ ft}^3 \]

\[ M_{\text{ind}} = (24\text{ in})(3\text{ in} / 2) = 6\text{ ft}^3 \]

\[ N_p = \frac{(0.5)(4)^2}{36} = 0.56\text{ psi} \]

\[ N_p = \frac{(5.5)(5)^2}{36} = 4.78\text{ psi} \]

Assume 3/8\text{ in} STL fl 1/2\text{ in} thick plate

Find moment capacity of STL plate

Plan for now, evaluate solutions for high load shear wall
ARCE 453
WINTER/SPRING 2022
Michelle Dennin
Elizabeth Claypool

Mega Wall 1 Calculations
DBT Shear Wall Cals

10d box nails @ 2" o.c. typ
** panel edges
8'
9.3/4" both walls C" thick

10d box nails @ 2" o.c. typ

Simpson CMST16 strap at all panel edges
Simpson CS16 at bottom only

Wall 1 - Control
Wall 2 - MegaWall

Philosophy: by eliminating the failure state of nail pull-through, we can force a new limit state, giving the megawall higher capacity.

Expected ultimate strength

Wall 1:

\[ V_u = 870 \text{ psf} \times 8' = 6960 \text{ lb} \]

From APA Report 158 on shear wall testing, the ultimate strength of a wall was between 1.5 - 2.7 times greater than the design strength. For our test, we will assume a factor of 3.0 to ensure proper failure.

\[ V_u = (870 \text{ psf} \times 8' \times 3.0) = 20,880 \text{ lb} \]
Expected ultimate strength - wall 2

Because the addition of metal strap is expected to eliminate nail pull through, the next limit state for a nail in single-shear will govern the wall's strength.

We expect the ratio of:

\[
\frac{Z_{\text{next limit state}}}{Z_{\text{head pull-through}}}
\]

to give a good approximation of the ratio

\[
\frac{V_o \text{ wall 2}}{V_o \text{ wall 1}}
\]

When the wall is detailed to have failure occur in the nails.

Head pull-through: 10d nails, 15/32 DF-L plywood

\[Z_{PT} = 79^*\]

Single shear limit state: 10d nails, 15/32 DF-L plywood

\[Z_{\text{II}} = 88^* \text{ (Mode III)}\]

\[
\frac{Z_{\text{II}}}{Z_{PT}} = 1.114 \rightarrow \text{expect an 11.9% increase in strength}
\]

\[V_o = 21^x (1.114) \approx 23.4^k\]

Round up due to lack of confidence in method.

\[V_o = 25^k\]
Wall Design - design both walls for $V = 25^k$ for simplicity

- Edge stud

\[ T = C = \frac{9.34}{8} \cdot 25^k = 28.4^k \]

Design for tension - DF-L + 1 posts & 3 timbers

\[ F_t = 825 \text{ psi} \]

\[ C_0 = 1.6 \]
\[ C_w = 1.0 \]
\[ C_f = 1.0 \]
\[ C_p = 1.0 \]
\[ C_L = 1.0 \]

\[ F' = 1320 \text{ psi} \]

\[ \frac{28.400^*}{1320 \text{ psi}} = 21.5 \text{ m}^2 \rightarrow \text{use a } 6 \times 6 \text{ minimum} \]

Design for compression

\[ F_c = 1000 \text{ psi} \]

\[ C_0 = 1.6 \]
\[ C_f = 1.0 \]
\[ C_p = 0.62 \]

\[ F_c = 994 \text{ psi} \]

\[ \frac{28.400^*}{994 \text{ psi}} = 28.6 \text{ m}^2 \rightarrow \text{a } 6 \times 6 \text{ will work on gross section, but accounting for net section at HD,} \]

\[ A_{6 \times 6 \text{ net}} = 11.25 - 1" \times 7.5" = 33.75 > 28.6 \text{ m}^2 \]

Use a 6 \times 8 DF-L #1 for edge studs
Check mud sill

\[ f_{c,1} = \frac{28400^*}{41.25 \text{ m}^2} = 688 \text{ psi} \]

\[ F_{c,1} = 625 \text{ psi} \]

\[ C_L = 1.00 \]

\[ C_B = 1.00 \]

\[ F_{c,1}^* = 625 \text{ psi} \]

\[ C_{allow} = (625 \text{ psi})(41.25 \text{ m}^2) = 25,781^* \]

This is less than the ultimate 28,400^*, but since crushing is a very ductile and minor failure, and will only contribute to a small amount of add'l deflection, we can say this limit state will not govern the failure of our walls.

Use a 2x6 DF-L = 1 pressure treated mud sill
**Post P-2 Design**

**Determine Loads and Load Combinations**

\[ P_u := 28 \text{ kip} \]

**Select Material to Be Used**

No.1 & Better Douglas Fir-Larch Timber (P&T)

**Determine Reference Design Values**  [NDS 2018-Supplement Table 4D]

\[ F_c := 1000 \text{ psi} \]

\[ E := 1.6 \cdot 10^6 \text{ psi} \]

\[ E_{min} := 0.58 \cdot 10^6 \text{ psi} \]

**Determine Preliminary Post Size**

Try a 6x8 Post

\[ d_x := 5.5 \text{ in} \]

\[ d_y := 7.5 \text{ in} \]

\[ A := 41.25 \text{ in}^2 \]

**Determine Post Capacity**

\[ C_D := 1.6 \quad \text{Roof Live Load} \]

\[ C_M := 1.0 \quad \text{MC<19\%} \]

\[ C_t := 1.0 \quad \text{T<100 degrees F} \]

\[ C_F := 1.0 \quad \text{for all members except dimensional lumber} \]

\[ C_i := 1.0 \]
\[ l_{e_{\text{1st}}ry} = (9 \text{ ft} + 0.75 \text{ in}) - (3 \cdot 1.5 \text{ in}) = 8.688 \text{ ft} \]

\[ \frac{l_{e_{\text{1st}}ry}}{d_y} = 13.9 \]

\[ E'_{\text{min}} = E_{\text{min}} \cdot C_M \cdot C_t \cdot C_i = (5.8 \cdot 10^5) \text{ psi} \]

\[ c = 0.8 \quad \text{Sawn lumber} \]

\[ F_{cE} = \frac{0.822 \cdot E'_{\text{min}}}{\left( \frac{l_{e_{\text{1st}}ry}}{d_y} \right)^2} = (2.468 \cdot 10^3) \text{ psi} \]

\[ F_{c,\text{star}} = F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_i = (1.6 \cdot 10^3) \text{ psi} \]

\[ \frac{F_{cE}}{F_{c,\text{star}}} = 1.542 \]

\[ 1 + \frac{F_{cE}}{F_{c,\text{star}}} = 1.589 \]

\[ C_p := \frac{1 + \frac{F_{cE}}{F_{c,\text{star}}}}{2 \cdot c} \cdot \left( 1 + \frac{F_{cE}}{F_{c,\text{star}}} \right)^2 - \frac{F_{cE}}{F_{c,\text{star}}} = 0.816 \]

\[ F_c' = F_c \cdot C_D \cdot C_M \cdot C_t \cdot C_F \cdot C_P \cdot C_i = (1.306 \cdot 10^3) \text{ psi} \]

\[ P_{\text{allow}} := F_c' \cdot A = 53.88 \text{ kip} \]

\[ DC_{\text{Ratio}} := \frac{P_u}{P_{\text{allow}}} = 0.52 \quad < \quad 1 \quad \text{(OK)} \]

Use a 6x8 Select Structural Douglas Fir-Larch Timber
Check Bearing on Sill Plate

For a stud grade 2x6 sill plate...

\[ F_{c,\text{perp}} := 625 \text{ psi} \quad \text{[NDS 2018 Table 4A]} \]

\[ C_b := 1.0 \quad \text{Lb}>6" \]

\[ F_{C,\text{perp}'} := F_{c,\text{perp}} \cdot C_M \cdot C_t \cdot C_1 \cdot C_b = 625 \text{ psi} \]

\[ \frac{P_u}{A} = \frac{1.086}{F_{C,\text{perp}'}} \quad \text{(OK)} \]

Use a 2x6 Douglas Fir-Larch Stud for ST-1
Simpson safety factor on hold downs: \( \frac{3}{2} \)

14.44 kg \( \times \) 3 = 43,335 lb \( \times \) 2

\( \theta = 0.47 \) \( \times \) 28.32 \( \times \)

\( c = 0.32 \) \( \times \)

Two HDU14-SDS 2.5 w/ 1" diam anchor rods will give more than enough capacity to combat the uplifting tension force. The demand capacity ratio ensures that they will not be the shear walls' failure mechanism. If the Simpson safety factor was not applied, the two hold downs would still have enough capacity (\( c/c = 0.98 \)).
WALL ANCHORAGE

3/4" DIAMETER BOLT W/ 2x MAIN MEMBER
MODEL W/ A36 SIDE PLATE (NDS 12.3.3)

SHEAR FAILURE MODE II

\[ z = 870 \text{ lbs} \]

\[ z_{II} = 870 (1.6) (6)(3.0) = 30K \]

SEISMIC QUANTITY

DEMAND = 21K

30 > 21

\[ D/c = 21/30 = 0.7 \]

* R \text{d} \text{ is a reduction factor specific to each failure mode. Since we will be taking the wall to ultimate we can ignore that factor.}

USE (6) 3/4" ANCHOR BOLTS W/ 2x MEMBER
STEEL BEAM CHECK

CHECK THE STEEL BEAM FOR WORST CASE SCENARIO LOADING AND SUPPORT CONDITIONS.

DETERMINE MAX POINT LOAD:

\[ 2.5K \]

\[ \Sigma M_A = 0 \]

\[ (2.5K)(9.06') = (P)(8') \]

\[ P = 2.83K \]

REACTIONS:

\[ \Sigma M_A = 0 \]

\[ (2.83K)(1.35') = (B_y)(9') \]

\[ B_y = 4.25k \]

\[ \Sigma F_y = 0 \]

\[ 2.83k - 4.25k - A_y = 0 \]

\[ A_y = 2.405k \]

\[ M_v = 32.5k\text{-ft} \]

\[ \phi M_p = 29.2k\text{-ft} \]

\[ D/C = \frac{32.5k\text{-ft}}{29.2k\text{-ft}} = 0.11 \sqrt{OK} \]
Determine \( W_{12} \times 53 \) \( I_{REDC} \) w/ 4" DIA. BOLTS (ASSUME 1-\( \frac{1}{8} \)"

\( I_x = 425 \text{ in}^4 \) (NO HOLES)

\[
I_{REDC} = 425 \text{ in}^4 - 2 \left[ \left( \frac{1.125}{12} \right) \left( 0.575 \text{ in} \right)^3 - \left( 1.125 \right) \left( 0.575 \text{ in} \right) \left( 1.77 \text{ in} \right) \right]
\]

\( I_{REDC} = 380 \text{ in}^4 \)

\( \Delta_{INC} \) (USING \( I_{REDC} \)) = (SEE RISA MODEL)
**CHECK FOUNDATION BEAM DEFLECTION:**

**REDUCED SECTION:**

$\frac{1}{12} - 5\frac{3}{8}$ yields 2 $\frac{1}{2}''$ holes in flange to allow for holdown rod to attach under the beam.

![Diagram of reduced section]

**R3A2D INPUT:**

![Diagram of R3A2D input]

**R3A2D OUTPUT:**

**Joint Deflections (By Combination)**

<table>
<thead>
<tr>
<th>Joint Label</th>
<th>X [in]</th>
<th>Y [in]</th>
<th>Rotation [rad]</th>
</tr>
</thead>
<tbody>
<tr>
<td>N1</td>
<td>0</td>
<td>0</td>
<td>0.455e-05</td>
</tr>
<tr>
<td>N2</td>
<td>0</td>
<td>0</td>
<td>2.060e-04</td>
</tr>
<tr>
<td>N3</td>
<td>0</td>
<td>0.034</td>
<td>-3.818e-04</td>
</tr>
<tr>
<td>N4</td>
<td>0</td>
<td>0.033</td>
<td>-4.199e-04</td>
</tr>
<tr>
<td>N5</td>
<td>0</td>
<td>0</td>
<td>-4.873e-04</td>
</tr>
<tr>
<td>N6</td>
<td>0</td>
<td>0</td>
<td>-1.090e-04</td>
</tr>
<tr>
<td>N7</td>
<td>0</td>
<td>0</td>
<td>-4.199e-05</td>
</tr>
<tr>
<td>N8</td>
<td>0</td>
<td>0</td>
<td>4.804e-05</td>
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<tr>
<td>N9</td>
<td>0</td>
<td>0</td>
<td>1.499e-04</td>
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<tr>
<td>N10</td>
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<td>0</td>
<td>-2.883e-04</td>
</tr>
<tr>
<td>N11</td>
<td>0</td>
<td>0</td>
<td>-1.705e-04</td>
</tr>
<tr>
<td>N12</td>
<td>0</td>
<td>0</td>
<td>-1.581e-04</td>
</tr>
</tbody>
</table>

Definitions of $<\text{0.05}''$ at ultimate load are deemed acceptable.

**Courtesy of Fall ARCE 451 Timber Design Lab Shear Wall Design-Build-Test Report**
**Current Conditions:**

* There are two bolts on each block connecting the block to the ground.

**Dimensions:**

```
<table>
<thead>
<tr>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>2' x 9'</td>
</tr>
<tr>
<td>3' x 9'3/4</td>
</tr>
</tbody>
</table>
```

**During Experiment: (Worst Case)**

\[ \Delta \Sigma M_c = \text{Shear (Moment Arm)} + T \text{(Moment Arm)} \]

\[ = 25k(9'3/4'' + 12'' + 18'') + T(9' + 3') \]

\[ T = 22.4k \]

\[ \Sigma F_y: C - T = 0 \]

\[ C = T = 22.4k \]

\[ \frac{A}{C} = \frac{\text{Demand}}{\text{Bolt Capacity}} = \frac{22.4k}{2(250k)} = 0.49 \quad \checkmark \quad \text{Good} \]
Appendix C:

Project Drawings
CONVENTIONAL NDS WALL
3/8" = 1'-0"

MEGA WALL 1 SPECIMEN DWG
3/8" = 1'-0"
HIGH LOAD WALL DETAILS

1. ADJOINING PANEL EDGE NAILING
   - 1" = 1'-0"

2. TOP PLATE NAILING
   - 1" = 1'-0"

3. MUD SILL NAILING
   - 1" = 1'-0"

HIGH STRENGTH SHEAR WALLS

ADJOINING PANEL EDGE
- 6 LINES 10D @ 2 1/2" O.C.
- STAGGERED 4X6 CENTER STUD

2 1/2" TYP.
1/2"
3/8"
1/2"
2 1/2" TYP.
1/2"
3/8"
1/2"
2 1/2" TYP.
1/2"
3/8"
1/2"
2 1/2" TYP.
1/2"
3/8"
1/2"
2 1/2" TYP.
1/2"
3/8"
1/2"