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Cyclic Testing of Reinforced Earthbag Walls

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I hereby recommend that the
SENIOR DESIGN PROJECT REPORT
prepared under my supervision by

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&
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entitled

CYCLIC TESTING OF REINFORCED EARTHBAG WALLS

Be accepted in partial fulfillment of the requirements for the degree of

BACHELOR OF SCIENCE IN CIVIL ENGINEERING

Tonya Nilsson — June 12, 2018

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CYCLIC TESTING OF REINFORCED EARTHBAG WALLS

by

David Aguilar Rodriguez, Jeff Stein
&
Taylor Darby

SENIOR DESIGN PROJECT REPORT

submitted to
the Department of Civil Engineering

of

SANTA CLARA UNIVERSITY

in partial fulfillment of the requirements
for the degree of
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This project could not have moved forward without the patience, help and guidance extended to us by the following individuals. We sincerely appreciate all you have done this past year.

Thank you.

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Department of Civil Engineering
Santa Clara University, Spring 2018

ABSTRACT

Earthen construction is the most popular building method around the world. One particular building method, using earthbags, has shown promise in performing well against seismic activity. This project undertook the goal of developing a preliminary seismic response modification factor, R, to be used in the design of homes in seismically active areas. Two 4' wide x 6' tall x 1' deep walls were cyclically loaded using a Three-Degree-of-Freedom (TDOF) Test Frame provided by Santa Clara University to determine the in-plane shear capacity of each wall. Testing revealed an average yield force of 419 lbs, an average ultimate force 1058 lbs, and an average R value of 6.

Wall design and construction was focused on three aspects of the project that were modeled to replicate common building practices while still being modular enough to test multiple samples. These aspects were the base, bond beam, and wall. Wall bases were designed to withstand up to 3500 lb-ft bending moment during forklift transport, the bond beam was designed to transfer up to 9,000 lbs of shear force into the wall, and, the wall was designed using common building practices used in earthbag construction.

Upon completion of the Consortium of Universities for the Research of Earthquake Engineering (CUREE) testing protocol, it was observed that the walls failed in buckling due to compression resulting from the force couple created by the loading arrangement. Despite failure, the walls continued standing even after the pin connection was removed from the tops of wall. This unexpected resiliency and behavior of the walls during testing led the team to believe that

earthbag walls are much more ductile a material that was initially anticipated. A deeper understanding is needed to better understand how earthbag buildings behave against seismic forces. This project is encouraging for future research and the development of a more standardized building method.

TABLE OF CONTENTS

TABLE OF FIGURES

1.0 Introduction

The goal of the project was to design, build and test two earthbag walls that contained adequate seismic reinforcement to withstand a typical design earthquake for a seismically active area such as Nepal. The dimensions of both walls were 4' wide x 6' tall x 1' deep. The project addressed not just Nepal's need, but a global need for sustainable housing and public safety while living in an earthquake prone region. This project's focus was primarily focused on Nepal due to the connections the project's advisor had with building earthbag homes in Nepal. Sustainability is accomplished by using an abundant, local building material, such as the local soil, as the home's primary means of construction, leaving behind a minimal carbon footprint. In terms of disaster response, the earthbag wall would be able to be constructed quickly due to locally sourced materials. Furthermore, these alternative building materials are economical, as they do not require builders to import a large amount of building heavy materials. This technology can then be spread widely throughout even the most remote regions of Nepal. Public safety is addressed by the testing of the seismic response of these earthbag structures. As part of this project, the desired seismic response was tested on earthbag walls constructed with barbed wire layered between the rows of earth bags and rebar driven from the top to the base of the wall. The intent of this project was to validate the reinforcement that has been tested on small scale structures and contribute to the overall research in the earthbag field. The main goal of the project was to use the results of the project to aid local home builders in Nepal and worldwide to build resilient, safer earthbag homes.

1.1 Background

As part of the work the project team did while deciding the scope of work for the project, the affected area of the 2015 Gorkha earthquake was researched to see how the common building materials used in the region affected. This research led the team to Guilan Liang's and Zhou Niandong's article "Background And Reflections On Gorkha Earthquake Of April 25, 2015". In their article, Liang and Zhou discussed the geological conditions that cause earthquakes of the intensity seen in that 2015 earthquake. The authors also looked at the damage caused by the 2015 Gorkha earthquake and reflected on how damage from future earthquakes could be mitigated. In April 2015, a magnitude 7.8 earthquake struck the Gorkha province of Nepal, destroying 299,588 homes in the country and damaging 269,107 more (Liang and Zhou, 2016). Most homes in Nepal at the time were not designed to withstand earthquakes, even though Nepal sits in a seismically active area. Just in the past century, four earthquakes with a magnitude of 6.5 or higher have occurred in Nepal (Liang and Zhou, 2016). Nepal also has a lack of building code regulation, which can be seen by comparing the aftermath of seismic events in other countries. This disparity can be seen when the 2015 Gorkha earthquake is compared to the 2015 Coquimbo earthquake located in Chile. For example, the 2015 Gorkha earthquake killed 8,800 people, while the 2015 Coquimbo earthquake with ten times the strength of the Gorkha earthquake saw far fewer deaths with the loss of 500 Chileans. Chile's strict building codes, both in writing and in implementation, prior to the 2015 Coquimbo earthquake ensured most, if not all, new buildings complied with the code. Nepal also passed its own new building codes in the 1990s but has done little to no work to enforce those new codes (Liang and Zhou, 2016). As a result, most of the buildings still lacked the proper reinforcement and design to withstand earthquakes. Analysis of the governmental structure that allowed for the dangerous building conditions was important to the team, however, the team sought analysis on the structures that both did and did not survive the 2015 Gorkha earthquake. The team therefore turned to Katsuichiro Goda's paper, "The 2015 Gorkha Nepal Earthquake: Insights from Earthquake Damage Survey" which provided useful insight on the building methods most susceptible to damage. Goda and his colleagues collected data on the damage caused by the earthquake through geo-tagged photos and comments made through in person observations (Goda, 2015). Most of the damaged buildings that were observed were built from stone or brick masonry. However, most of the reinforced concrete (RC) buildings were not damaged. Reinforced concrete in this case referred to the use of rebar as reinforcement in the concrete beams and slabs in the building. Buildings built with masonry materials, such as stone or brick, have low ductility and are in danger of collapsing during a seismic event. This low ductility factor means the building will not move with the ground acceleration caused by an earthquake. In turn, the building will attempt to resist the movement caused by the earthquake. With brittle and unreinforced masonry, this means a total collapse of the building is likely. The RC buildings that did survive all had a similar structural feature, the use of moment resisting frames. Reinforced Concrete moment resisting frames perform well in earthquakes as to the members and connections in the frame designed to resist the bending forces imparted during ground accelerations and lateral loads. Reinforced concrete moment resisting frames built in accordance with the Indian standard code performed well (Goda, 2015). In Kathmandu, there was significant damage to historical buildings, but not to surrounding buildings built with RC moment resisting frames. Another noteworthy observation of the structures in the affected region was the fact that buildings damaged during the main earthquake collapsed during aftershocks which underscores the necessity of building evacuation.

As it relates to this project, earthbag construction must first and foremost be designed to withstand the next seismic event in the region. There was a sense of urgency to rebuild as quickly as possible and house the now millions of homeless. It is crucial, however, that earthbag construction is implemented correctly. It was this project team's responsibility to address the factors that caused buildings to collapse during the 2015 Gorkha earthquake and to design earthbag structures that will be able to withstand not only the next seismic event in the Nepal region but in other earthquake prone areas of the world as well.

The research done by Goda and Zhou above have educated the methods used with earthbag construction to become more economically viable and safe. Nepalese home builders and builders in other seismically active regions of the world have been building with this material since the devastating 2015 Gorkha earthquake. This earthquake left 3.5 million people homeless in Nepal alone and destroyed tens of thousands of homes. Furthermore, the seismic aspect of earthbag construction has been largely unexplored. This project drew upon a past Santa Clara University (SCU) senior design project completed in 2017, *Design of a Single Family Home and Rooftop Rainwater Catchment System in Nepal Using Earthbag Technology,* by Makena Wong, Olivia Carreon and Nabila Farah Franco, in which the group assumed a value for seismic performance. The team sought to conduct this research with the goal of informing builders working in Nepal about the proper construction of earthbag structures and looked at existing research to ensure these homes will be ready for the next potential earthquake.

1.2 Sustainability Characteristics

Building homes out of earthbags means using an abundant and easily accessible material, such as the local soil, which allows families access to an affordable and permanent home if properly designed. This type of construction also means using minimal concrete, drywall or other materials that leave behind a large carbon footprint. The team sought to further validate the use of earthbag walls for home construction; doing so contributed not just to the academic field but to the humanitarian field as well. With this project, the team has contributed to the building of communities with sustainability and safety at the forefront. Through the research performed here, this project became one of the first in the field of earthbag construction that has built and tested a large scale, seismically reinforced earthbag wall. A lack of research into earthbag construction has inhibited its mainstream use, as little is known about their response to various loading cases.

Much of the carbon emissions related to construction come from transportation of materials. This process can be even more difficult in Nepal, which has very mountainous terrain in many areas. Additionally, minimal infrastructure can make transporting materials around the country even more difficult. This use of earthbags for homes would drastically reduce the need to transport traditional building materials. The soil found in Nepal has typically been found to have acceptable levels of clay for cohesion (20-30%), which makes for acceptable soil to build with (Geiger, 2015). These conditions make Nepal ideal for the use of earthbag homes.

2.0 Summary Alternative Analysis

During the development phase of the project, the team considered numerous different variations on the standard confined earth wall. The team's main priority was to test the method of construction that earth home builders have been utilizing in the field. Testing this building method would allow the project to provide relevant data on previously constructed earthbag structures, as well as provide builders a measure of confidence in their building techniques. Initially, the team considered using only barbed wire reinforcement between the bag layers. This method was later combined with vertical rebar reinforcement driven through the courses of bags. The use of vertical rebar reinforcement is common practice in the field, which influenced the decision to choose this hybrid design. The following designs were considered:

1. Earthbag wall with barbed wire as reinforcement. Concrete or plaster base. (chosen design)
2. Earthbag wall with rebar driven through bags as reinforcement.
3. Earthbag wall with bamboo driven through bags as reinforcement.

Use of barbed wire as reinforcement is the most common building practice and requires few materials that are difficult to source. This method would be a lower cost and sustainable, due to the simple materials required to build the wall. The second option the team evaluated was the use of rebar to help transfer shear loads and provide reinforcement against out of plane loading. This option was expected to provide the most structural reinforcement but was also seen as not as sustainable, since many of the places that use earthbag construction do not have easy access to quality rebar. The third alternative, bamboo, would provide similar structural reinforcement when compared to rebar, while at the same time being a sustainable material. Rebar and bamboo would have served the same purpose, as they would both act as dowels holding together the courses they would penetrate.

2.1 Selection of Project's Design

After evaluating the different aspects of each alternative, a hybrid between the first two alternatives was chosen due to their compatibility with each other and theoretically more seismically resistant structure. The use of barbed wire matched common building practices, as did the use of rebar. Both techniques combined provided the team structures with a higher expected seismic resistance. The option of using bamboo in place of rebar was eliminated due to workability concerns during construction, despite being more sustainable than rebar.

The selected building method for testing met the project's needs, as it blended current earthbag building practices with added reinforcement to better protect against damage from earthquakes. The inclusion of rebar in the walls allowed for a more consistent performance during testing. This consistency gave the team the confidence required to use the values from testing in a design setting. As the use of earthbag building continues to increase, experimentation with the material will follow as the community's insight on the material's performance grows. As the team sought to research the unknown seismic capabilities of earthbag structures, the chosen test set-up satisfied both the need for research into the seismic resistance of these structures and provided a meaningful contribution to the larger scholarly conversation of standardizing earthbag walls with consistent testing practices.

3.0 Design Components of Project

There were three distinct portions of the system which needed to be designed; the base, the bond beam connection to the testing rig, and the wall itself. Each of these portions were designed using a different material (or combination of materials), and as a result required different assumptions in the design process. An overview of the wall design can be seen in **Figure 1**.

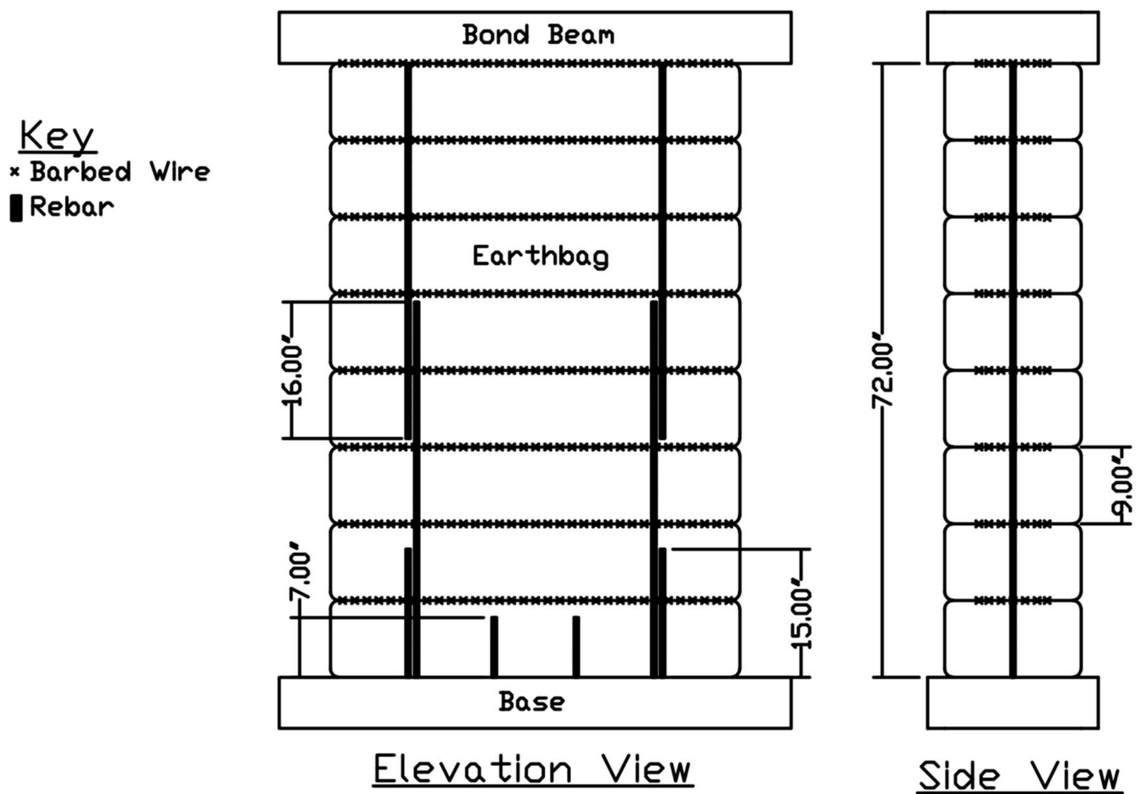

Figure 1: Front & Side View of Wall Showing Rebar Placement & Barbed Wire.

The wall bases utilized a combined concrete and structural timber system, which required the design of various design components. First, the base design required that connections between the concrete and wood were sufficient to transfer the loads the team was expecting to see during the test. These constraints resulted in a designed base that had the capacity to resist both shear and moment stresses that were to be generated by the test frame onto the wall. Connections between the two systems were provided by a series of six (6) five eighth inch (⅝”) lag bolts, which were found to be adequate to transfer the maximum expected shear loads in the base. Additionally, it was determined that the concrete of the base would never experience a significant negative moment, resulting in minimal tensile forces in the top portion of the concrete beam. As a result, reinforcing bars were placed only in the bottom portion of the beam. The culmination of these design considerations resulted in the final design values which are elucidated later in this report.

3.1 Soil Profile

The soil used for the project was sourced from Zanker Landfill located in South San Jose. The soil composition was a Sandy Clay Loam. The percentage breakdown of the soil's composition was as follows: 45% Sand, 30% Clay, 15% Silt, and 10% Gravel**.** Limiting organic matter was important to prevent growth of organisms within the earth bags which could have led to a decrease in the overall bag strength.

Compression experiments on the soil used for the project were performed in the SCU Structures Lab. **Figure 2,** below, shows the testing setup used to determine an f'c value; this value is used to represent the compressive strength of a soil. The numerical results of these tests can be found in **Table 1,** where the f'c value was found to be 320 psi. The test results shown in **Table 1** were from soil with similar compaction to the earth bags the team made for the completed walls.

The cylinders of soil that were tested were 1.5" tall by 2" wide. These samples were created by cutting a toilet paper roll in half, then compacting soil into the tube in half inch increments. The standard test specimen size, used for testing earthen materials in a laboratory setting, is 4 x 8 in. (100 x 200 mm) or 6 x 12 in. (150 x 300 mm) cylinders for strength tests provided that the requirements of ASTM C31 are met (ACI, 2014) This standard was not met as the team sought to validate the field methods used by earthen builders to test their soil on site. Through the use of the SCU Civil Engineering Structures Labs' compression machine, the team sought to accomplish this goal. Ultimately, the size of test cylinder was chosen at the direction of our external advisor, Patti Stouter, from Build Simple Inc as she had used this size of cylinder as the team's external advisor had been using in the field for years. The decision to use five samples per mix of soil in **Table 1** was done to obtain a reliable average f'c value. This was also done to allow for adequate time to complete the full range of soil tests the team was expecting to complete. At the end of the compression testing, both the external advisor and the project team were satisfied with the f'c results obtained using the lab's compression machine.

Table 1: Soil Compression Tests Results

Normal Soil w/ Normal Compaction Sample	Diameter 1 (in)	Diameter 2 (in)	Height (in)	Pounds (lb)	Radius (in)	Area (in^2)	PSI	Average PSI (5 Samples)
1	1.650	1.642	1.959	533.3	0.823	2.128	250.6	318.8
2	1.555	1.722	1.830	596.1	0.819	2.109	282.7	
3	1.511	1.634	1.966	437.7	0.786	1.942	225.4	
4	1.698	1.619	1.951	991.2	0.829	2.160	458.8	
5	1.682	1.654	1.940	843.5	0.834	2.185	386.0	
6	1.735	1.717	2.012	709.5	0.863	2.340	303.2	

Figure 2: Setup of soil compression tests.

3.2 Test Frame to Wall Specimen Connection

The bond beam, which caps the top of the wall and provides a connection to the testing rig, was formed exclusively from wood, which simplified the design to resist the lateral force placed upon the bond beam. Considerations of the functionality of the beam and determining the best way to facilitate the desired connection to the wall, required several design iterations. The use of a concrete bond beam was ruled out early in the project as the weight of the beam would pose a safety hazard. This hazard is due to the fact that these concrete beams, when used on a full scale home project, are poured onto buttressed walls (Hart, 2015). An image of concrete bond construction in the field can be found in **Figure 3**, below**.**

Figure 3: Earthbag home built with concrete bond beam.

While this method is feasible for the construction of a house, the team was concerned with constructability of a concrete bond beam. The most notable constructability issue the team had with the concrete beam design was designing the formwork for use while the concrete was poured and cured for 28 days. When the design scopes of both the concrete and timber bond beams were assessed, the timber bond beam was ultimately chosen. From the outset of the design process, the project team along with their advisors determined a singular pinned connection to the wall would best simulate the loading pattern of an earthquake. This pin connection was paired with the use of two steel T-plates that prevented any out of plane movement of the wall during testing. This was done to minimize the amount of torsional deformation that could occur in the wall. While earthquakes do cause torsion to occur in a building. This work focused on the direct shear forces that would be imparted on the wall during in-plane loading.

A problem posed by the single point of connection between the wall and the test frame was how the bond beam would be kept connected to the top course of the wall specimen throughout the duration of the testing cycle. The project team predicted correctly, that loading the wall through a single point of connection would result in a seesaw motion which would cause the bond beam to lift and separate from the top course. The team sought to prevent this by using rebar couplers attached to the rebar inserted into the wall and placing threaded rod on the open end of the coupler. This threaded rod would then go through the bond beam and terminate at the top. This is where a nut and washer combination was used to keep the bond beam connected to the top course. This connection is talked about in greater detail in Section 4.4.

3.3 Expected Loads for Walls During Testing

Design of the wall proved to require the most design choices of any component of the system. No universally accepted design standards exist for earthbag construction, this meant the project team's design was based upon commonly accepted best practices from builders in the field. The design was also influenced by the team's faculty and external advisor, which lead to the choices for rebar placement and barbed wire usage, to be detailed later in this report. At the end of this preliminary design phase, the team concluded these best practices did not include all the information needed for design, as they contain little quantitative information on the wall design. For example, little information was available regarding the expected ultimate strength of the wall, which was key for designing the bond beam and base. One test from the University of Bath (Vadgama, 2010) reported a maximum load of approximately 6,000 lbs on their test walls. As a result, a conservative maximum strength estimate of 9,000 lbs (1.5 times the

Bath tests max load) was used. The hope was that this conservative value would ensure that failure occurred in the wall and not in the bond beam or the base.

3.4 Design Standards Used for Wall Design

In order to complete the design of the system, several design standards were used. For all wood design, especially values for wood strength and equations for different shear analyses, values were taken from the AWC 2015 design standard (AWC, 2015). This is the industry standard in the United States for all wood design. This was key for design of the wooden bond beam and its connection to the testing apparatus. Similarly, for the base concrete, the design was based off the ACI 318-14 concrete building code standards (ACI, 2014). The California Building Code 2016 (CBC, 2016) was used for seismic requirements and performance goals for the wall.

4.0 Description of the Design Components Used for Testing

For the project, an initial schedule was developed using Microsoft Project, which allowed for real time tracking of the project's completion as well as manipulation of workflow and completion dates. The team planned to start basic design concept work before the academic year began. As the team and the project advisor, Dr. Nilsson, were in the area, this allowed the team to get a helpful head start on the work involved with the initial design. Several weeks were also devoted to the procurement of materials and the construction of the bases of the walls. Ultimately the decision was made to build and test two identical (or as close to identical as possible when working with earth) earthbag walls, following the typical best practices outlined in Section 3.2. The purpose of building two identical walls was to determine with a reasonable certainty the expected design strength of a reinforced earthbag wall. Prior to design, the team expected a peak test strength around six thousand (6,000) pounds and decided to design the wall sections to withstand up to nine thousand pounds (9,000) of shear force.

4.1 Overview of Walls Components for Testing

The walls were designed to incorporate three main sections: the base, wall, and bond beam. Bases for each wall were designed to best replicate friction to normal ground while still allowing for easy transport. The walls themselves were designed with common earthbag building practices in mind with the inclusion of rebar for added strength and hopefully more consistent values. The bond beam design included the transfer of all shear forces from the Three Degree of Freedom (TDOF) test frame, through the beam, and into the wall.

4.2 Base Design of Wall

The wall's base design was based off the work that a past SCU senior design group had done with straw bale walls (Ackerson, 2017), with additional concrete to help carry the heavier earthbag walls. The base design consisted of a 5.5" thick reinforced concrete beam with embedded rebar anchors that would pierce the first three courses of earthbags placed. This beam was formed using four by six (4 x 6) timber members which were left in place to provide additional strength during testing. **Figure 4,** below, provides an image of the completed bases. The design of the base allowed the use of six by twelve (6 x 12) timber members that were placed across the ends of the base to prevent overturning during testing. These beams were secured to the strong floor with the use of threaded rod, nuts and washers. **Figure 5** illustrates how these larger timber members were laid across the base to prevent overturning during testing.

Figure 4: Finished concrete in completed bases.

Figure 5: Timber beam members laid across base and connected to strong floor.

4.3 Building Method Used for Wall Construction

The building process consisted of compressing soil into polypropylene bags, tying each bag closed with metal wire. Bags were compressed at each end with a short piece of lumber to prevent crumbling in this key area of compression. Each bag was then tamped in place on the wall using a 10 lb. tamping plate. The first four courses were compacted on the floor, then lifted onto the rebar pins that can be seen in **Figure 4.** Once the rebar pins were covered, the following courses were compacted on the previously laid course, allowing for greater ease of construction. Barbed wire was also implemented during the construction of the walls. This meant using one strand of four-point barbed wire (totaling 9' of barbed wire), placed below each bag before the next course was placed on top. The 9' strand of barbed wire was formed into an ellipse shape to allow for adequate coverage of the earthbag course. This process was repeated until the walls reached the desired height for testing; 15 layers tall for Wall 1 and 16 layers tall for Wall 2.

The timber members used in the base allowed for the attachment of vertical and horizontal forms which aided in the construction of plumb and compacted earthbag walls. While compacting bags in place, sheet metal was used to prevent the barbed wire from tearing the bag material. **Figure 6**, below, showcases what these construction practices looked like. Construction of the walls started inside of the SCU Civil Engineering Structures Lab but was moved outside to due to space constraints. After construction, the walls were relocated inside the SCU Structures Lab to dry until testing, which was scheduled for the first week of April 2018.

Figure 6: Construction practice used to build both wall specimens.

4.4 Bond Beam Design of Wall

The bond beam had to be able to withstand the forces generated by the machine, as well as be compatible with the layout of the machine. The team opted for a six by eight (6 x 8) timber beam and a ¾” sheet of plywood for the bond beam, as it met the criteria that was listed previously. Nails were shot into the sheet of plywood to embed with the top course of the wall. Nails were spaced 4” in each direction, based on shear tests which indicated roughly 100 lbs. of capacity for nails in cured earthbags. The nail layout is shown in **Figure 7.** After the nails were in place the six by eight (6 x 8) member was attached to the top of the plywood using six (6) ⅝” lag bolts. Additional height was required on the bond beam to reach the minimum height of the TDOF testing rig, so an additional section of six by four (6 x 4) lumber was added to the top of the bond beam. Finally, the pinned connection was secured to the beam using four (4) 1 ½” lag bolts. Additionally, channels were cut into each end of the beam so that T-plates on the testing apparatus could minimize torsional deformation. To prevent the T-plates from lifting out of the channels during testing, two 16” long 2x4 timber members were added to the sides of the channels. **Figure 8** demonstrates the finalized channel design that was cut into the bond beam.

Figure 7: Nail connection to top earthbag layer.

Figure 8: Guide channel at ends of bond beam.

Figure 9 displays the completed bond beam used during the first test. This was the final design that was used for both wall tests.

Figure 9: Bond beam from top side with plate-pin connection.

4.5 Coupler System Used for Bond Beam to Wall Interaction

The team used a coupler system to connect the rebar inside the wall to a piece of threaded rod. This threaded rod then had a washer and nut connection to prevent the bond beam from uplifting and twisting; **Figure 10** below displays this connection. Appendix A, page A-21 contains an image of the installation of the nut and washer used to connection the bond beam to the top course of the wall. After collecting and synthesizing results from the first test, the bond beam was redesigned for the second wall to better transfer forces from the testing machine to the wall. These changes included expanding the guide channels and attempting to better secure the rebar couplers. **Figure 11** shows the coupler inside the bond beam, and this was the final design the team decided on.

Figure 10: Rebar coupler to threaded rod connection.

Figure 11: Threaded rod connection set into underside of beam.

4.6 Reinforcement Used for Walls

The reinforcement of the wall consisted of using barbed wire between each course of bags and the use of rebar driven into the courses at three foot height intervals. Nine foot strands of barbed wire were used between each course of bag in order to facilitate the friction force that would occur between courses trying to slide past each other. **Figure 12**, below, shows the layout of barbed wire that was used for both walls. The friction factor between the barbed wire and polypropylene bags was found to be 0.67 (University of Bath). This means that for every one pound (1 lb) of normal force applied at the interaction between bags, 0.67 lbs of shear resistance were generated. Two 4.5' lengths of #4 rebar were driven vertically into each end of the wall at 3' foot high intervals with the use of a fence post driver. This installation method resulted in a lap of approximately 18" between this rebar. **Figures 12** and **13**, below, shows the process used for both walls.

Figure 12: Standard barbed wire configuration between layers.

Figure 13: Driving sharpened rebar into wall.

A lap splice of 16 inches was sought as the rebar was driven into the walls; this was based off ACI 318-14 concrete building code standards (ACI, 2014). This standard was not an ideal representation of the interaction that would occur between the rebar and soil, but it provided a starting point for this project. Further research could be undertaken to determine what the ideal lap distance would be within soil.

The team also employed the use of 1/4” diameter twine that was tied around four courses of earthbag at a time. This use of twine was done with the goal of joining individual courses into groups so they could act as a unit. This procedure was done at the recommendation of Patti Stouter to help bind the individual courses together and allow them to act as more of a unit. This goal was not achieved, as the courses shrunk over time due to the drying of the clay in the soil. The twine was no longer taut around the courses, and thus did not provide the goal of unifying individual courses into one unit. The team discussed tightening the strings before testing, but it was decided that the effects would be minimal.

5.0 Description of Testing Frame Used in Testing

The wall specimens were each tested in the Three Degree of Freedom hydraulic testing frame in Santa Clara University's Structures Lab, which can be seen in **Figure 14**, below.

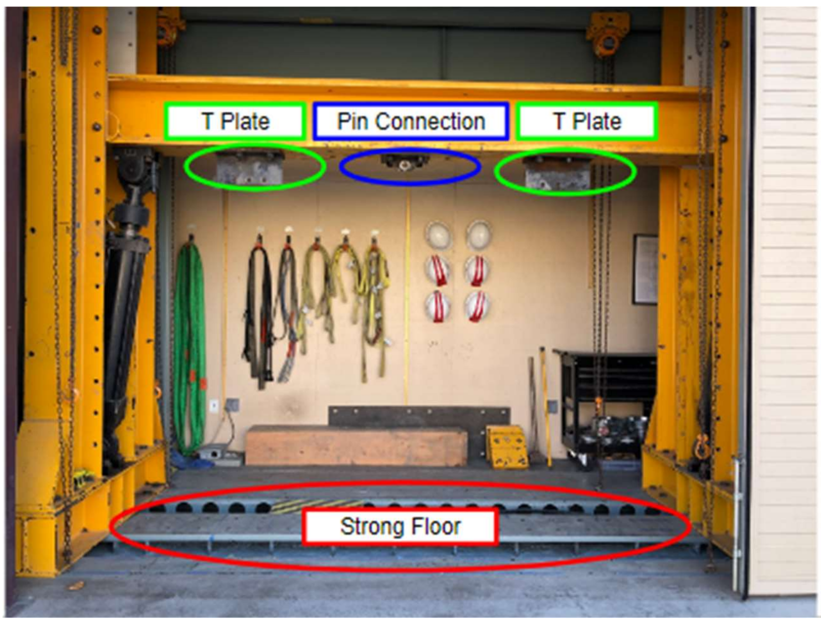

Figure 14: Anatomy of Three Degree of Freedom Test Frame.

This testing frame's top beam was connected to the specimens using the pin circled in blue in the above image, with the T-plates circled in green preventing out of plane movement during the loading cycle. The wall bases were bolted into the strong floor using one inch (1") threaded rods, which prevented sliding and overturning forces. Forces were applied upon the wall using three large hydraulic actuators.

5.1 Description of Testing Protocol Used in Experiments

The walls were tested utilizing a standard CUREE loading protocol. The CUREE protocol was developed by the Consortium of Universities for the Research of Earthquake Engineering as a standardized way of simulating seismic loading on shear walls. The testing protocol simulates earthquake loading patterns by laterally deflecting test specimens incrementally and cyclically until an ultimate deflection is reached. The pattern of deflection is shown in **Figure 15.**

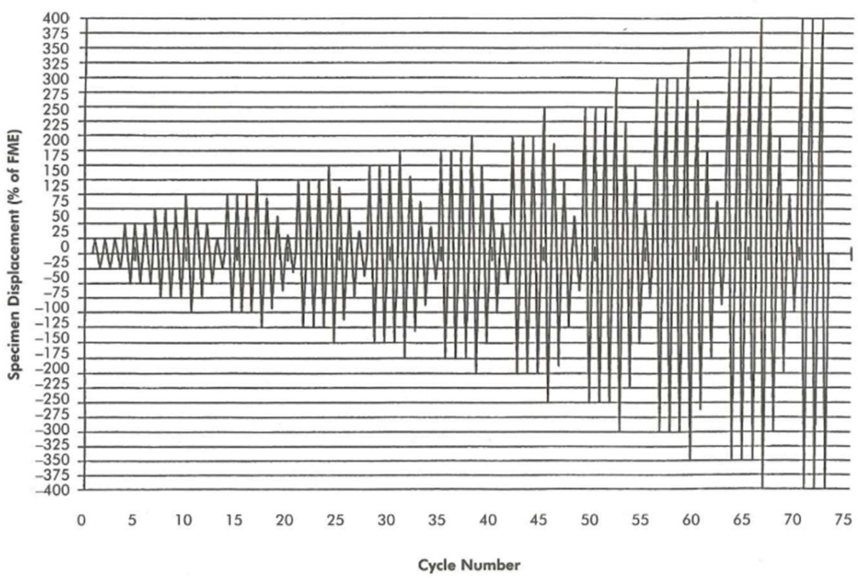

Figure 15: CUREE Testing Protocol deflections for each cycle.

As these cyclic deflections occur, load cells in each actuator record the vertical and horizontal loads resulting from these deflections. This force vs. deflection is plotted as a hysteresis loop similar to **Figure 16**, below.

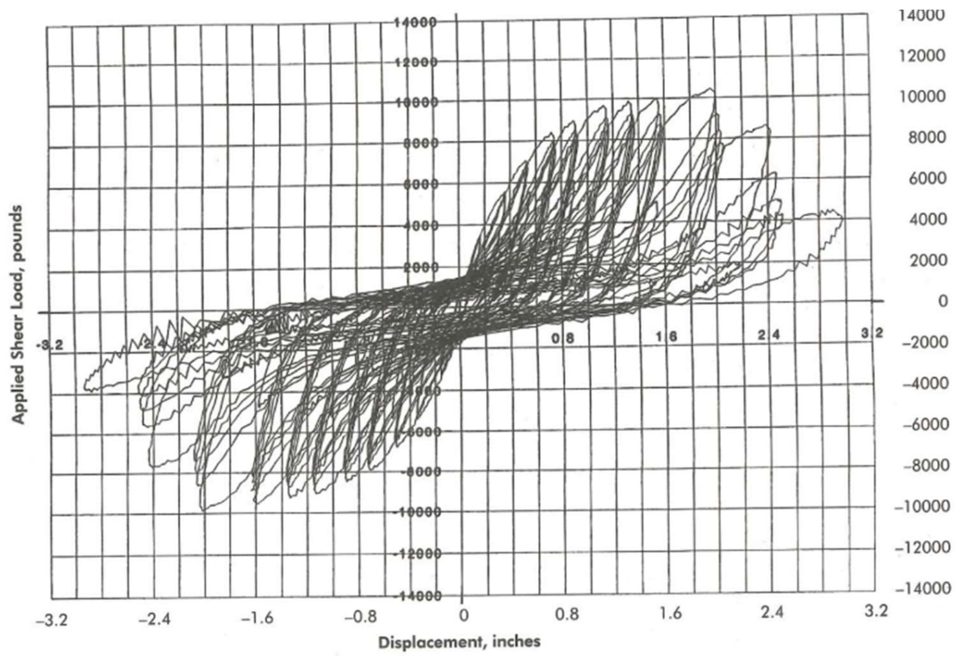

Figure 16: A typical hysteresis loop for shear walls tested with CUREE loading protocol.

5.2 Coordination of Facilities Used in Project

One key aspect of the project was the coordination that was required between the team and Santa Clara University's Facilities and Environmental, Health & Safety (EH&S) Departments. The team also coordinated with the lab manager, Brent Woodcock, and the other senior design teams working in the lab. Coordination with the lab manager and the other teams in the lab was key to ensuring every team in the lab had access to the best resources and the most time available to the respective projects. The coordination with Facilities and EH&S was key to the team's work outside of the lab, primarily during construction of the walls. The work was done on Sherman Street, which was an active street, and was essential to the completion of the project. The team made sure proper personal protection equipment (PPE) was used during all phases of construction and testing of the walls. This meant using gloves during the installation of barbed wire and hard hats when working inside the Three Degree of Freedom test frame.

5.3 Site Safety and Waste Disposal

The wall's aspect ratio was chosen not only for research purposes but for safety of the team as well. An aspect ratio of 2:1 would have led to an unstable structure during construction and transportation that would have placed the team and the public in danger. Site safety was a major concern of the team and for all those involved in the project. The project's success relied on the safe and efficient construction process, as much as on the experimentation of the walls.

The construction waste from the walls, such as the base and the soil contained inside the polypropylene, were disposed on the dirt lot located behind Bannan Engineering at SCU. This disposal was done with the permission of SCU facilities due to the impending demolition of Bannan Engineering.

6.0 Test Results

The test samples were tested with the CUREE loading protocol described above using the Three Degree of Freedom testing rig. The samples were deflected to an ultimate deflection of 1.5% of the specimen height, or approximately 9.5". As the cyclic deflections from the CUREE cycles were applied by the testing rig, the loads experienced by each of the three hydraulic rams were recorded. These three forces were then reduced to isolate their combined shear component applied within the plane of the wall. The applied shear loads versus in plane deflection were plotted to form the hysteresis loops shown in **Figure 17**.

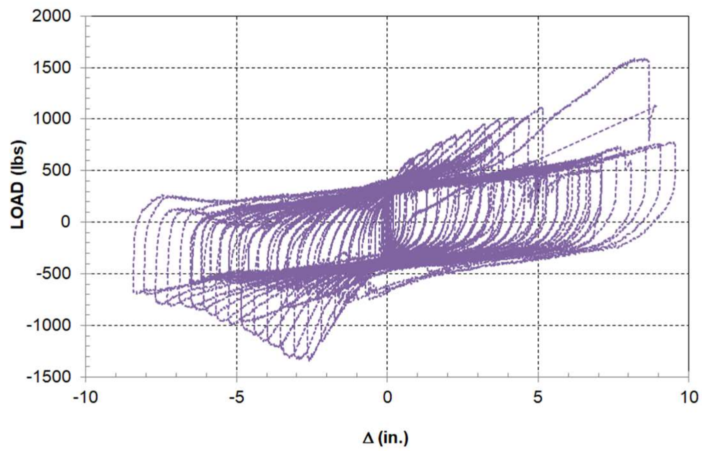

Figure 17: Hysteresis Loops for first test specimen.

The spikes in load seen on the positive deflection side of this graph are likely due to crushing of the wooden bond beam, which occurred when the previously mentioned T-plates slid out of the channels on the bond beam and failed to cleanly reenter the channel on the next portion of the loading cycle. As a result, these peak points were not considered when calculating seismic response modification factor, with a lower peak of 1242 lbs used. This issue was alleviated for the second specimen, resulting in a lower peak load, as can be seen in **Figure 18.**

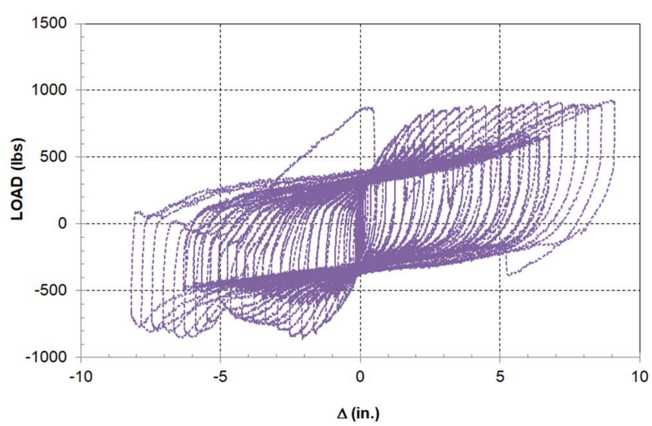

Figure 18: Hysteresis Loops for second test specimen.

6.1 Observed Failure Mode

The failure mode observed for both test specimens was buckling in the north edge of the wall. This buckling was due to the force couple created by the in-plane shear forces upon the wall. While buckling is typically a relatively inflexible failure mode, these specimens were able to withstand severe buckling while remaining standing after removal of the top pin. This is a positive, as flexible failures are typically preferred for withstanding seismic loading. The out of plane deflection of the walls is shown in **Figure 19**, below.

Figure 19: Out of plane buckling of Wall 1.

There was also some sliding in between bag layers, especially between the top two layers. Where interbag sliding was noticed, there was also significant deformations of barbed wire reinforcement, and minimal deformations of the soil. This sliding indicates that when bag slippage occurs, it is due to deflections in the barbed wire, not failures within the soil matrix. To minimize this slippage rebar pins were driven at 45 degree angles into the top four courses of Wall 2 prior to testing. This reduced the slip in the top courses somewhat, though did not increase overall strength of the wall.

6.2 Numerical Analysis of Test Results

Using the recorded experimental data, values for ultimate and yield deflections and forces were determined. Based upon these values, a value for seismic response modification factor, R, value was calculated. This is a standardized measure of a system's ductility during seismic loading and is based on the ratio of yield to ultimate strength and deflection. A higher R value is desired for earthquake loading, as it indicated a more ductile system. In code based seismic design, as prescribed by the CBC, the seismic design force is divided by the R value, reduces design force. The average R value found for these walls was 6.0 and is shown in **Table 2.** This R value was calculated using the APA formula where $R = R_d * R_0$. R_0 is taken as the ratio of 0.8 * ultimate strength over the yield and strength, $R_d = \sqrt{2\mu - 1}$ and $\mu = \Delta ult \div \Delta yield$. (APA, 1998)

Table 2: Numerical test results for both walls.

	Wall 1	Wall 2	Average
Yield Force (lbs)	440	396	419
Ultimate Force (lbs)	1242	873	1058
Yield Deflection (in)	0.21	0.43	0.32
Ultimate Deflection (in)	2.54	2.20	2.38
R	7.7	4.3	6

The R values calculated for these walls should not be used as design values for a full scale earthbag structure. This uncertainty is due to the fact that the R values noted in design codes worldwide are values based on full scale tests of entire systems. In addition, the wide variance between the two values makes it difficult to say for certain what the true R of the system is. Future testing would give a better indication of what the actual R value of earthbag systems is. The R values for this system can be said with some confidence to show the earthbag system that was tested was found to be more ductile than unreinforced masonry, as can be seen in **Table 3 (**CBC 2016).

Table 3: R values for common building materials.

Building System	**R Value**
Reinforced Concrete Shear Wall	5
Ordinary Reinforced Masonry Shear Wall	2
Light Frame Wood	6.5
Steel Plate Shear Wall	6.5

In addition, the ultimate loads are in excess of what a similar wall might be expected to withstand based upon a design earthquake in San Jose, CA. Based on USGS spectral accelerations and the California Building Code, these walls should be able to withstand roughly 174 plf, or 696 pounds per four foot (4') section. While loads of this magnitude would likely cause the earthbag walls to yield they are well below the ultimate failure forces. This is aligned with modern targets for seismic design of buildings, which allows for permanent damage but not complete collapse. All of this data indicates that these walls would likely be structurally acceptable for use in seismically active regions, provided the entire system is detailed properly.

6.3 Soil Characteristics after Testing

Wall 1 cured inside the SCU Civil Engineering Structures Lab for 75 days at an average humidity of 63%. Wall 2 cured inside the SCU Civil Engineering Structures Lab for 85 days at the same humidity as Wall 1. Moisture content tests were also performed on the bottom three courses of the walls as well as the middle course of the walls. The moisture content results for Walls 1 and 2 can be seen in **Table 3** and **Table 4**, respectively**.** These soil samples were oven dried for 24 hours before they were weighed for a second time.

Table 4: Moisture content results for Wall 1, Tested @ 75 days.

	1st Weight (g)	2nd Weight (g)	MC (%)	MC Average (%)
Bottom Course	46.3	41.93	9.44	
	39.57	36.1	8.77	9.06
	33.36	30.37	8.96	
2nd to Bottom	42.2	38.79	8.08	
	50.74	46.84	7.69	7.95
	36.07	33.15	8.10	
3rd to Bottom	45.36	41.57	8.36	
	38.21	35.11	8.11	8.24
	37.48	34.39	8.24	
Middle Course	47.17	43.14	8.54	
	42.31	38.76	8.39	8.36
	40.82	37.49	8.16	

Table 5: Moisture content results for Wall 2, Tested @ 85 days.

	1st Weight (g)	2nd Weight (g)	MC (%)	MC Average (%)
Bottom Course	17.24	15.9	8.35	
	24.15	22.35	7.87	8.11
	30.84	28.44	8.11	
2nd to Bottom	30.82	28.67	7.30	
	33.93	31.54	7.34	7.51
	32.61	30.14	7.88	
3rd to Bottom	25.11	23.18	8.08	
	25.44	23.46	8.18	8.13
	30.7	28.3	8.14	
Middle Course	34.33	31.57	8.33	
	34.42	31.58	8.54	8.44
	23.31	21.44	8.45	

6.4 Observed Status of Soil in Earthbags during Deconstruction

An area of great interest for the project team was the condition of the soil in the bags post testing. Great care was taken by the team to ensure the soil was preserved in the state it was in after being tested. This meant cutting open each course starting from top to bottom, removing one course at a time. Overall, the soil in each of the courses for both walls was found to have held together in large cohesive chunks, separated by several large cracks. There was minimal crumbling of the edges of the courses, and the soil itself was dense and hard packed, with significant effort required to break the courses apart. There was concern within the team that movement of the walls on the forklift would damage the earthbags. After deconstruction of both walls was complete, there was minimal indication of damage to the wall due to transportation. The typical status of the soil in the courses can be seen in **Figure 20,** below**.**

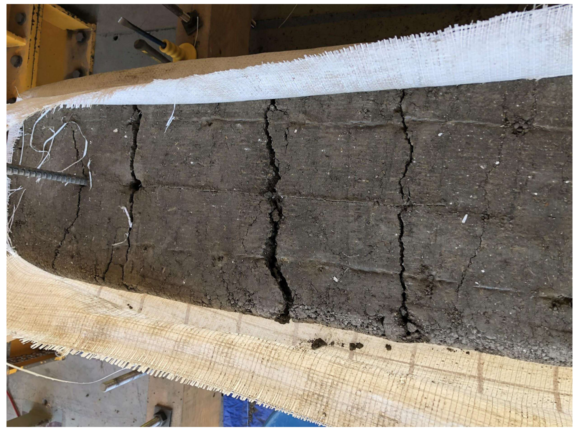

Figure 20: Typical status of soil in courses of walls, Wall 2.

Figure 20 was taken at mid-height of Wall 2. This course had many cracks along the width of the course, similar to what was observed for the majority of Wall 1's courses. The lower portion of Wall 2 tended to have fewer cracks, but the upper and middle portions were similarly damaged.

The soil around the rebar splice from the first wall test can be found in **Figure 21.**

Figure 21: State of soil around rebar lap splice, Wall 1.

One of the key components the group considered was the effects of the rebar splice on the nearby soil. The cracks that are observed in **Figure 21** were found at both lap splices on Wall 1. The team believed these cracks developed from the rebar moving out of plane during testing. This out of plane movement, the team believes, was caused by the rebar being loaded by the bond beam at the top of the wall. The rebar was connected to the bond beam via the aforementioned coupler system, this is what allowed the rebar to be loaded in such a way that allowed for the out of plane movement observed in Wall 1. This back and forth movement created the same cracks observed in **Figure 21** two to three courses above the rebar splice. This was due to the out of plane movement of the rebar occurring throughout the length of the stick of rebar. The bottommost piece of rebar did not experience the same out of plane movement. The marks left by the post driver, which was used to drive the rebar down the length of the wall, can also be seen in **Figure 21**. It is not known to the team exactly what effect the impacts from the post driver had on the strength of the course. The team did try to mitigate damage to the course during the placement of the rebar. This took the form of slowly driving the rebar as the fence post driver approached the earthbag course as well as avoiding any alteration of the courses once the walls started the drying process.

Figure 22, below, shows the soil around the rebar lap splice for Wall 2.

Figure 22: State of soil around rebar lap splice, Wall 2.

Wall 2 did not develop the same level of cracks that were observed at Wall 1's lap splice, but there was still some movement as can be seen by the small cracks originating from the rebar lap splice. Wall 2's rebar did not experience the same level of buckling as was seen in Wall 1. The team believes this was because Wall 1 failed due to the force couple that occurred at the rebar lap splices. The compression component of the force couple created in the wall caused this buckling deflection in the wall that was discussed previously. This same force couple was not seen in Wall 2, as can be seen by the lack of cracked soil around the lap splice in **Figure 22.** The variations in the installation of the rebar, along with the variations in how the soil dried in the bags, created the conditions that allowed for the creation of the force couple observed in Wall 1 and not in Wall 2. Without the force generated within the wall by the force couple, the rebar within the walls would not have sufficient enough force to create the movement required to generate the level of cracking observed in Wall 1.

6.4.1 Observed Status of Barbed Wire during Deconstruction

During deconstruction, the team also took note of the state of the barbed wire that was placed between each course. The team found that on average, 15 barbs out of the 20 were deformed due to the movement that occurred between the courses located at the rebar lap splices. The average in this case refers to the level of deformation listed above being constant for most layers in both Wall 1 and 2. A special case for the barbed wire deformation were the four courses located at the rebar lap splices for both walls. Although most of the barbs were found to be deformed at these lap splices, the barbs themselves did not cause a significant amount of damage to the polypropylene bags. This observation can be seen on **Figure 23.**

Figure 23: Marks left by barbed wire placed, Wall 1.

These same sized holes were found throughout Wall 1, as well as on Wall 2. The team does believe the barbed wire played a significant role in preventing bag slippage, however, the team was expecting to see more substantial holes in the polypropylene bags by the barbed wire. The deformation observed in the barbs of the barbed wire showed how much the barbed wire prevented the shifting of the bags in both walls during testing. Additional images of the bag slippage that was observed for both walls can be found in Appendix A, pages A-17 through A-20. The barbs of the barbed wire located at the bottom four courses of both walls were found to be less deformed than the barbs located at other points in the walls. The team believed this was due to the shear forces failing to fully transfer from the top of the wall to the base. Most of the shear forces were found to have focused around the rebar lap splices, which accounted for the lack of transfer of forces to the base.

6.4.2 Observed Status of Rebar during Deconstruction

The rebar used in Wall 1 can be found in **Figure 24** and **25.**

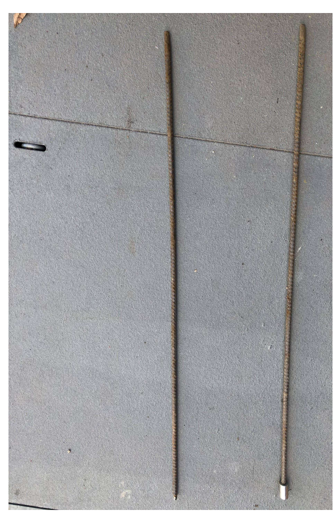

Figure 24 (L) and **25 (R):** Rebar used in Wall 1.

The rebar in Wall 1 was bent due to the testing protocol the wall was subjected to. This can be clearly seen in **Figure 24** as this rebar was heavily subjected to the load being transferred from the bond beam down into the rebar**. Figure 25** shows the rebar slightly bent, but this could be attributed to the lack of force couple that developed in this rebar lap splice. The team believes this was the case due to the aforementioned force couple that was developed in the wall, this led to the buckling failure that was observed. Wall 2's rebar can be seen in **Figure 26** and **27,** below.

Figure 26 (L) and **27 (R):** Rebar used in Wall 2.

Wall 2's rebar did not experience the same bending that the rebar from Wall 1 experienced. The team believes this result was because Wall 2 did not experience the same buckling failure that was observed in Wall 1.

6.4.3 Observed Status of Top Course of Walls during Deconstruction

Another point of interest for the project team was the top course and the bond beam interaction. The team had expected to see minor crushing of the soil located at this interaction, however what was observed was more crushing than what was expected. **Figure 28** shows the crushed soil that was seen after Wall 2's test. The crushing seen in **Figure 28,** below, was observed in both Walls 1 and 2.

Figure 28: Crushed soil found at the top layer and bond beam interaction, Wall 2.

The team can credit most of the crushing of the soil at the top course and bond beam interaction due to the movement of the bond beam throughout testing. The bond beam also began to lift and separate from the top course, which resulted in a seesaw motion that would crush one side of the top course. This resulted in the crushing of the side opposite of the direction the TDOF was loading in as the test progressed from side to side.

It is not known exactly how much of the soil crushing that occurred at the top course was due to nail shear, crushing from movement of the bond beam and installation of the bond beam onto the course. Crushing of the top course soil from nail shear and crushing from the movement of the bond beam should be the focus of a future team, as this project team did not have enough time to investigate this.

The bond beam was placed using sledgehammers as the nails in the bond beam had to be pushed firmly into the top course. The team believes this caused a significant amount of crushing of soil in the top course for both walls. Future teams should take greater care in the installation of the bond beam to control for this unwanted crushing soil. The team recommends placement of the bond beam as soon as the top course is compacted and placed, if possible. If not, the top course's soil should be rehydrated to allow for greater ease of installation, then allowing for enough time for the top course to dry around the nails from the bond beam.

7.0 Humanitarian Impact of Project

The original motivating factor to pursue a project related to earthbag building was a response to the housing crisis in Nepal following the Gorkha earthquake in 2015. This factor played a huge part in outlining the scope of the project because determining the earthquake response of earthbag buildings will help to protect the people of this region after potential future earthquakes. With so many people around the world without housing due to natural disasters, more knowledge about earthbag homes could make a huge difference by providing adequate housing to people in need.

The nature of earthen building has always been sustainable since the main building material is soil. Soil can be sourced from anywhere and in practice is typically gathered directly from the site where a building is to be built. The project team took this fact into account when deciding where to source soil from to build the earthbag walls. Unable to gather soil directly from the

Santa Clara University campus, a decision was made to purchase soil from a nearby company to cut down on transportation and best mimic the local soil profile.

8.0 Cost Estimate

The team's budget was designed within the constraints of the grant that was received from the Santa Clara University School of Engineering. The materials that were the most difficult for the team to acquire were the soil and the lumber for many of the design components. The need for high strength materials required the team to purchase structural select lumber, which increased the costs of the project. The team's access to the SCU Civil Engineering Structures Lab greatly reduced the costs due to the lab's resources being made available to the team. **Table 6**, below, indicates the components used during the project with prices and quantities included.

Table 6: Cost estimate of project.

Item	Breakdown			Description
	Quantity		Cost	
Earth Bags + Sewing Machine (Split between two teams)	1 Roll Bag + 1 Sewing Machine	@	\$350 \$100	250 yd Superadobe Roll from calearth.org + Shipping and Taxes VEVOR Bag Closer - Amazon
Soil	7 yd^3	@	\$250	Soil from Zanker Landfill
Barbed Wire	1 Roll	@	\$90	1320 ft roll from Home Depot/Lowes
Rebar	12 Rods	@	\$4	10 ft rods from Home Depot/Lowes
Rebar Anchors	-	@	-	Included in Rebar cost
Wood	Various Sizes	@	\$400	One 3/4" Plywood Sheet Six 2 x 4 x 8' Six 4 x 8 x 12' All from Home Depot/Lowes Four 4 x 6 x 12' Two 6 x8 x 8'
	TOTAL:		\$1200	

9.0 Conclusions

After testing was completed and data could be analyzed, the team had a couple major takeaways from the senior design project. First and foremost, an average seismic ductility factor of six (6) shows that earthbag walls are much more ductile than the team originally anticipated. This ductility gave the team confidence that the common building practices currently used in earthbag construction, which involve the use of metal / wooden dowels with barbed wire between courses, are providing adequate ductility in the structures they are building. In the future, more testing will need to be done to continue to validate these findings and accurately determine a design strength value, such as an R value, for an entire earthbag home instead of just one four foot (4') section of wall.

The team is confident in the R value of six (6) that was calculated for the four foot (4') wall that was tested. This value, however, should not be used and extrapolated upon to design a full scale earthbag home, as this value is unique to the walls that were built for this project.

As more research into the strength of earthbag walls is conducted, the team believes that it would be beneficial to expand styles of testing to a shake table. While a Three-Degree-of-Freedom (TDOF) test frame provides valuable information, a shake table could more accurately represent an earthquake. This accuracy is because a shake table would be able to simulate ground accelerations, which would load the walls from the bottom, as opposed to loading from the top, as was seen with the TDOF test frame. An advantage of using the TDOF test frame for lateral load tests is that it allows for the use of the CUREE testing protocol. The CUREE testing protocol lasts for three hours while a typical shake table test lasts only for a minute or two at most, as most large earthquakes last roughly this long.

Earthbag building shows a lot of promise regarding earthquake response, and more research is needed in the future to get a better idea of exactly how these walls perform. Not only is earthbag building promising structurally, but it is a very sustainable building style as the soil can typically be sourced from the building site reducing carbon emissions for transportation and processing. This frugal building method could offer help to people displaced from their homes by earthquakes and any other factors around the world.

9.1 Recommendations for Future Project Teams

More research regarding earthbag walls is needed before any major recommendations or changes to code can be implemented and as a result, the team hopes that future students at SCU can continue building from this project. Should students choose to do so, the team this year learned a couple of valuable lessons that would have helped the project run smoother had they been considered beforehand. First, a better bond beam design is encouraged. More specifically, a better connection between the rebar and threaded rod would be helpful because all these connections failed during testing of this project. Perhaps a higher quality coupler would suffice, but this project team highly advises a future team to consider this coupler design in a future, similar project. In addition, using longer walls would be worthwhile, as it would likely result in a more shear controlled failure mode. This would result in lowered compressive forces, and less buckling in the walls. Second, the team this year failed to consider facility restraints in the initial design of the project. This failure was especially true with the height of the walls because if the team was unable to work outside, the ceiling height would not have allowed enough space to drive the top strand of rebar into the walls. Third, when moving the walls about on a forklift, the team realized that it is advantageous to add bracing to prevent tipping.

The team only did this while moving one wall, and the entire process was much less stressful when there was no concern that months of work might tip over in a matter of seconds. These few issues, if fixed, should help future groups prevent some of the problems this team experienced and make the project run smoother.

Should a team desire to research the mechanics of the reinforcement used in earthbag walls, one area of research recommended by the project team is looking into the relationship between rebar and the soil in the bags. This project team observed how the behavior at the interface between these two materials could be affected by both the clay content of the soil and the drying time. The team believes the drying time given for both walls in these experiments was sufficient enough for the clay in the soil to bond with the rebar, resulting in a strong connection between the two materials. As a result, this dynamic between soil and rebar and how friction in addition to the composition of the soil bonds the two together would be a great area for additional research. A pull-out test would be the best place to start with this experimentation. Further increasing the earthbag building community's knowledge on this topic would be a great benefit to the field.

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Appendix A

Sieve Analysis of Soil

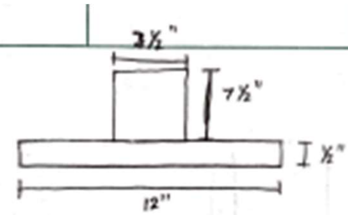
SIEVE No.	MASS RETAINED(g)	Percent Retained on each Sieve	Cum. % retained
4	15.4	3.08	3.08
10	245.2	49.04	52.12
20	124.6	24.92	77.04
40	53.2	10.64	87.68
60	27.2	5.44	93.12
140	25.4	5.08	98.2
200	0.4	0.08	98.28
Pan	8.6	1.72	

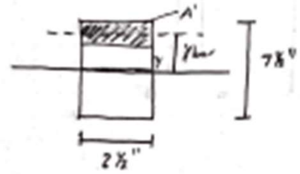
FM: SAMPLE WEIGHT : 501.05g

$\frac{509.52}{100}$

FM = 5.1

Concrete Base Beam Design.





$$A' = \left(\frac{7.5}{4}\right)'' \cdot 3.5'' = 6.56''$$

$$Y_{bar} = 0.9375''$$

$$Q = A' \cdot Y_{bar} = 6.56'' \cdot 0.9375'' = 6.15''$$

$$I = \frac{bh^3}{12} = \frac{3.5'' \cdot 7.5''^3}{12} = 123.05\ in^3$$

$$q = \frac{VQ}{I} = \frac{900\ lb \cdot 6.15''}{123.05\ in^4} = 449.8\ lb/in^2$$

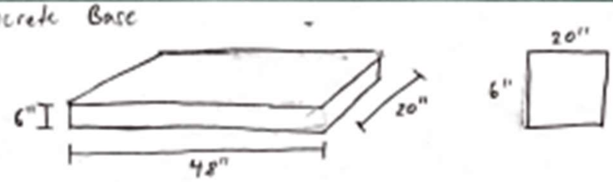
3/8" diameter capacity

$$s \leq \frac{V_{screw}}{1.5 \cdot q} = \frac{1990\ lb/in^2}{1.5(449.8\ lb/in^2)} = 2.95'' \approx 3''$$

Concrete Base Beam Design.

11/13/17

Concrete Base



$E_c = 2376000 \qquad I = \frac{bh^3}{12} = \frac{20'' \, 6''^3}{12} = 360 \text{ in}^4$

*Assuming #4 rebar

$d_t = 4.5\text{in} \qquad A_s = 0.40$

$c = 3\text{in} \qquad f_y = 60 \text{ ksi}$

$\varepsilon_t = 0.0015 \rightarrow \phi = 0.65$

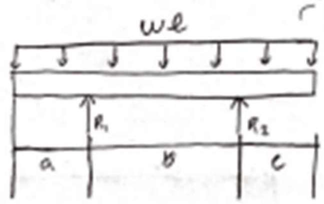
$0.40 \text{ in}^2 \cdot 60 \text{ ksi} \left(4 - \frac{0.85 \cdot 2}{2}\right)$

$= 65400 \text{ lb} \cdot \text{in} \cdot 0.65$

$= 42510 \text{ lb} \cdot \text{in}$

*Equation from NDS beam design formulas

7000/4 = 1750 plf



$R_1 = \frac{1750 \cdot 4 (4 - 2 \cdot .5)}{2 \cdot 3}$

$R_1 = R_2 = 3500 \text{lbs}$ if forks roughly equispaced

$R_1 = \frac{wl\,(l - 2c)}{2b}$

$= \frac{7145.83 \text{ lb/in} \cdot '' \, (48'' - 2(6''))}{2(36'')}$

$= 1650000 \text{ lb} \cdot \text{in} / 12$

$= 14000$ plf

Moments

$M_1 = \frac{1750 \cdot .5^2}{2} = 218.75 \text{ lb ft}$ - at support #1

$M_2 = -\frac{1750 \cdot .5^2}{2} = 218.75 \text{ lb ft}$

$M_3 = 3500 \left(\frac{3500}{2 \cdot 1750} - .5\right)$

$3500 \cdot .5 = 1750 \text{ lb ft}$

Max Mom.

$X = \frac{R_1}{w} - a = \frac{3500}{1750} - .5 = 1.5$

$3500 \cdot 1.5 - \frac{1750(.5 + 1.5)^2}{2} = 1750 \text{ lbft}$ - max moment

Concrete Base Beam Design.

Shear

$V_1 = w\ell$

$1750 \text{ lb/ft} \cdot .5 \text{ ft}$

$V_1 = 875$ lbs will be the same @ both supports

Check for stirrup need $\frac{.65 \cdot V_c}{2}$

$V_u = 875$ lb

$V_c = 2 \cdot 20 \cdot 4.25 \sqrt{3{,}000}$

$V_c = 9{,}311.3$ lbs

$V_u = 875 < \frac{.65 \cdot 9{,}311.3}{2}$

$875 < 3{,}026.2$ – moment controls as expected

Rebar Lap Splice in Walls.

11/20/17

$$l_d = \frac{3}{40}\left(\frac{f_y}{\lambda\sqrt{f'_c}}\right)\left(\frac{\Psi_t\,\Psi_e\,\Psi_s}{\left(\frac{c_b+K_{tr}}{d_b}\right)}\right)d_b$$

f_y = 60000 psi
λ = 1.0 (normal-weight concrete
f'_c = 4000 psi
Ψ_t = rebar location factor = 1.0
Ψ_e = uncoated = 1.0
Ψ_s = #6 or smaller = 0.8
d_b = bar diameter = 0.500 in
K_{tr} = 0 for design simplification
C_b = 1

$$l_d = \frac{3}{40}\left(\frac{60000}{1\cdot\sqrt{4000}}\right)\left(\frac{1.0\cdot 1.0\cdot 0.8}{\left(\frac{1}{0.5}\right)}\right)0.5$$

l_d = 14.2 in

Tension

Class B = $l_s = 1.3\, l_d$ = 1.3 · 14.2 = 18.5 in lap

$\frac{A_s \text{ provided}}{A_s \text{ required}} = \frac{0.4}{0.4} = 1$, if < 2 then class B must be used

Compression

$f'_c > 3000$ psi

f_y = 60,000 psi

$l_s = 0.0005 f_y d_b$

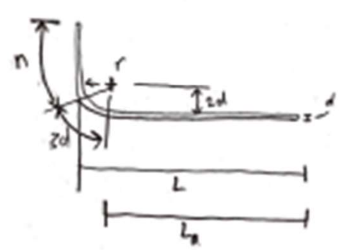
l_s = 0.0005 (60,000) (0.5)

l_s = 15 in lap

Tension controls

lap splice = 18.5 in

L_a = 18.5 in + 5.5 in = 24 in
embedded in beam



L_a = 24 in   L = 25.5 in
d = 0.5 in   n = 4 in
3d = 1.5 in   r = 0.8 in
total length = 29.5 in

Fastener Connections Used in Wall.

Mechanical Connections	2/27/18	D. Aguilar

Specific Gravity of:

Dauglas Fir - Larch : 0.50
Plywood sheathing : 0.57
OSB : 0.62

Widthdrawal Voleus, W, pounds per inch of penetration into side grain of wood

For plywood : 93 - 100 lbs per inch for single nail
→ 5/8" ≈ 60 lbs per nail
From bag shear test : 100 lbs per nail (w/ bag + wood shear)

For ½" lag bolts in bond beam : 378 lbs per inch

Design of Structural Timber Members for Wall Bases.

$\sigma = P/A$

$40{,}000 = 2600/$

$F = 2600 = 2F_{Fr}$

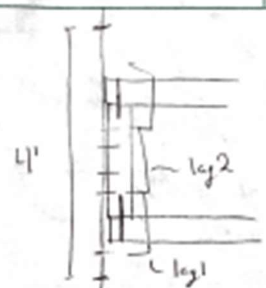
$2 \cdot F_D \cdot$ Coeff friction

-Assume F_{Fr} & F_D are equal for both down forces

$F_D = 1300/.4$

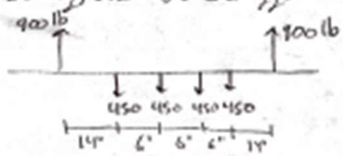
6' 4x6" side members x 4) 4 8' 4x6"
20" 4x 6" cap members x 4
3' 4x4 bottom members x 6 - 4 8' 4x4'
L brackets x8
19/32" plywood 4'x8' x 2
8' #4 rebar rod x 1

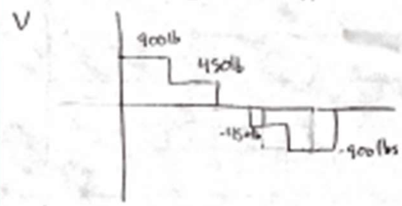
Expected Forces to be Seen in Wall During Testing.

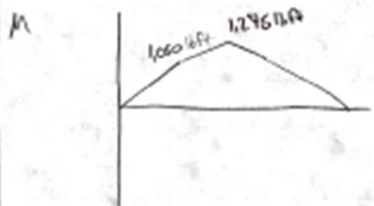


- Wall Weight - 3,600lb
(incl. base)

Bending/Shear on Outrigger







Shear Parallel to grain -

$F_y = 180\,psi$

$F_b = 850\,psi$

Shear failure of wood

Leg 1

$f_y = 180\,psi \cdot \underbrace{2.5in}_{embed} \cdot \underbrace{1.75in}_{depth} \cdot 2 = 1575\,lbs$ - well above 900lb needed

Leg 2

$f_y = 180\,psi \cdot \underbrace{2.5in}_{embed} \cdot \underbrace{1.5in}_{depth} \cdot 2 = 1{,}350\,lb$ - well above 450

Appendix A

Sieve Analysis of Soil

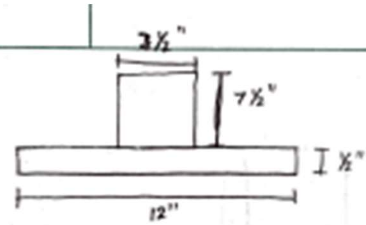
SIEVE NO.	MASS RETAINED(g)	Percent Retained on each sieve	Cum. % retained
4	15.4	3.08	3.08
10	245.2	49.04	52.12
20	124.6	24.92	77.04
40	53.2	10.64	87.68
60	27.2	5.44	93.12
140	25.4	5.08	98.2
200	0.4	0.08	98.28
Pan	8.6	1.72	

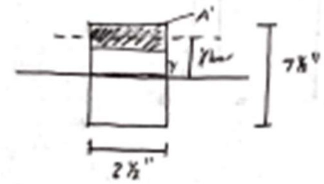
$$\frac{509.52}{100}$$

FM: SAMPLE WEIGHT : 501.05g

FM = 5.1

Concrete Base Beam Design.





$$A' = \left(\frac{7.5}{4}\right)'' \cdot 3.5'' = 6.56''$$

$$Y_{bar} = 0.9375''$$

$$Q = A' \cdot Y_{bar} = 6.56'' \cdot 0.9375'' = 6.15''$$

$$I = \frac{bh^3}{12} = \frac{3.5'' \cdot 7.5''^3}{12} = 123.05\ in^3$$

$$q = \frac{VQ}{I} = \frac{9000\ lb \cdot 6.15''}{123.05\ in^4} = 449.8\ ^{lb}/_{in^2}$$

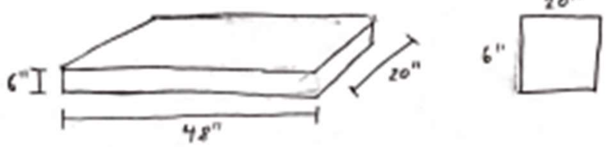
3/8" diameter capacity

$$s \le \frac{V_{screw}}{1.5 \cdot q} = \frac{1990\ lb/in^2}{1.5(449.8\ ^{lb}/_{in^2})} = 2.95'' = 3''$$

Concrete Base Beam Design.

11/13/17

Concrete Base



$E_c = 2376000$ $I = \frac{bh^3}{12} = \frac{20'' 6''^3}{12} = 360\ in^4$

*Assuming #4 rebar

$d_t = 4.5in$ $A_s = 0.40$

$c = 3in$ $f_y = 60\ ksi$

$\varepsilon_t = 0.0015 \rightarrow \phi = 0.65$

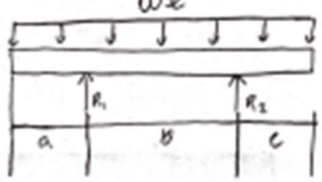
$0.40\ in^2 \cdot 60\ ksi\ (4 - \frac{0.85 \cdot 2}{2})$

$= 65400\ lb \cdot in \cdot 0.65$

$= 42510\ lb \cdot in$

* Equation from NDS beam design formulas

$7000/4 = 1750\ plf$



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$R_1 = R_2 = 3500lbs$ if forks roughly equispaced

$R_1 = \frac{wl\ (l - 2c)}{2b}$

$= \frac{145.83\ lb/in\ (48'' - 2(6''))}{2(36'')}$

$= 1\ 680\ 000\ lb \cdot in\ /12$

$= 14000\ plf$

Moments

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$3500 \cdot .5 = 1750\ lb\ ft$

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$X = \frac{R_1}{w} - a = \frac{3500}{1750} - .5 = 1.5$

$3500 \cdot 1.5 - \frac{1750(.5+1.5)^2}{2} = 1750\ lbft$ - max moment

Concrete Base Beam Design.

<u>Shear</u>

$V_1 = w a$

$1750 \text{ lb/ft} \cdot .5 \text{ ft}$

$V_1 = 875$ lbs will be the same @ both supports

$\frac{.65 \cdot V_c}{2}$

Check for stirrup need

$V_u = 875$ lb

$V_c = 2 \cdot 20 \cdot 4.25 \sqrt{3{,}000}$

$V_c = 9{,}311.3$ lbs

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Rebar Lap Splice in Walls.

11/20/17

$$l_d = \frac{3}{40}\left(\frac{f_y}{\lambda\sqrt{f'_c}}\right)\left(\frac{\Psi_t\ \Psi_e\ \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b}\right)}\right) d_b$$

f_y = 60000 psi
λ = 1.0 (normal-weight concrete
f'_c = 4000 psi
Ψ_t = rebar location factor = 1.0
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d_b = bar diameter = 0.500 in
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$$l_d = \frac{3}{40}\left(\frac{60000}{1 \cdot \sqrt{4000}}\right)\left(\frac{1.0 \cdot 1.0 \cdot 0.8}{\left(\frac{1}{0.5}\right)}\right) 0.5$$

l_d = 14.2 in

Tension
Class B = $l_s = 1.3\ l_d$ = 1.3 · 14.2 = 18.5 in lap

$\frac{A_s \text{ provided}}{A_s \text{ required}} = \frac{0.4}{0.4} = 1$, if < 2 then class B must be used

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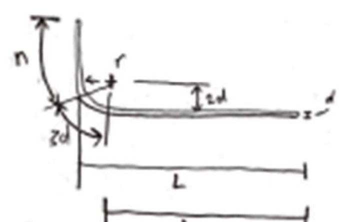
f'_c > 3000 psi
f_y = 60,000 psi
$l_s = 0.0005 f_y d_b$
l_s = 0.0005 (60,000)(0.5)
l_s = 15 in lap

Tension controls

lap splice = 18.5 in

L_a = 18.5 in + 5.5 in = 24 in
embedded in beam



L_a = 24 in
d = 0.5 in
3d = 1.5 in
L = 25.5 in
n = 4 in
r = 0.8 in
total length = 29.5 in

Bond Beam Fastener Connection + Nail Shear Test.

Mechanical Connections | 2/27/18 | D. Aguilar

Specific Gravity of:

Douglas Fir - Larch : 0.50
Plywood sheathing : 0.57
OSB : 0.62

Withdrawal Values, W, pounds per inch of penetration into side grain of wood

For plywood : 93 - 100 lbs per inch for single nail
→ 5/8" ≈ 60 lbs per nail
From bag shear test : ~~100 lbs per nail~~ (w/ bag + wood shear)

For ½" lag bolts in bond beam : 378 lbs per inch

Design of Structural Timber Members for Wall Bases.

$\sigma = P/A$

$40{,}000 = 2600/$

$F = 2600 = 2F_{Fr}$

$2 \cdot F_D \cdot$ Coeff friction

-Assume F_{Fr} & F_D are equal for both down forces

$F_D = 1300/.4$

6' 4x6" side members x4) 4 8' 4x6"
20' 4x6" cap members x4
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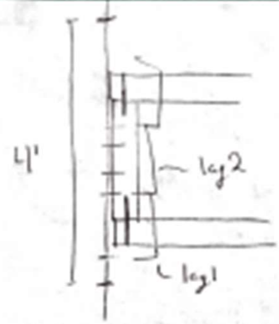
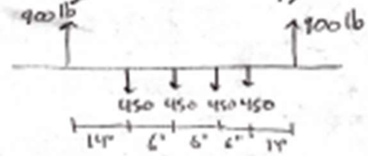
L brackets x8
19/32" plywood 4'x8' x2
8' #4 rebar rod x1

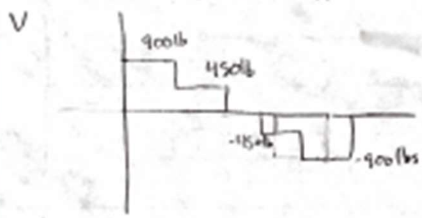
Expected Forces to be Seen in Wall During Testing.



- Wall Weight - 3,600lb (incl. base)

Bending/Shear on Outrigger

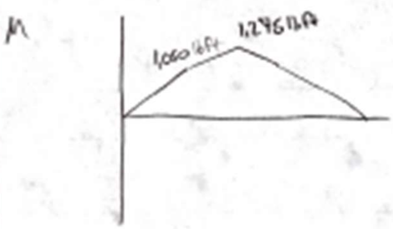




Shear Parallel to grain -

$F_v = 180$ psi

$F_b = 850$ psi



Shear failure of wood

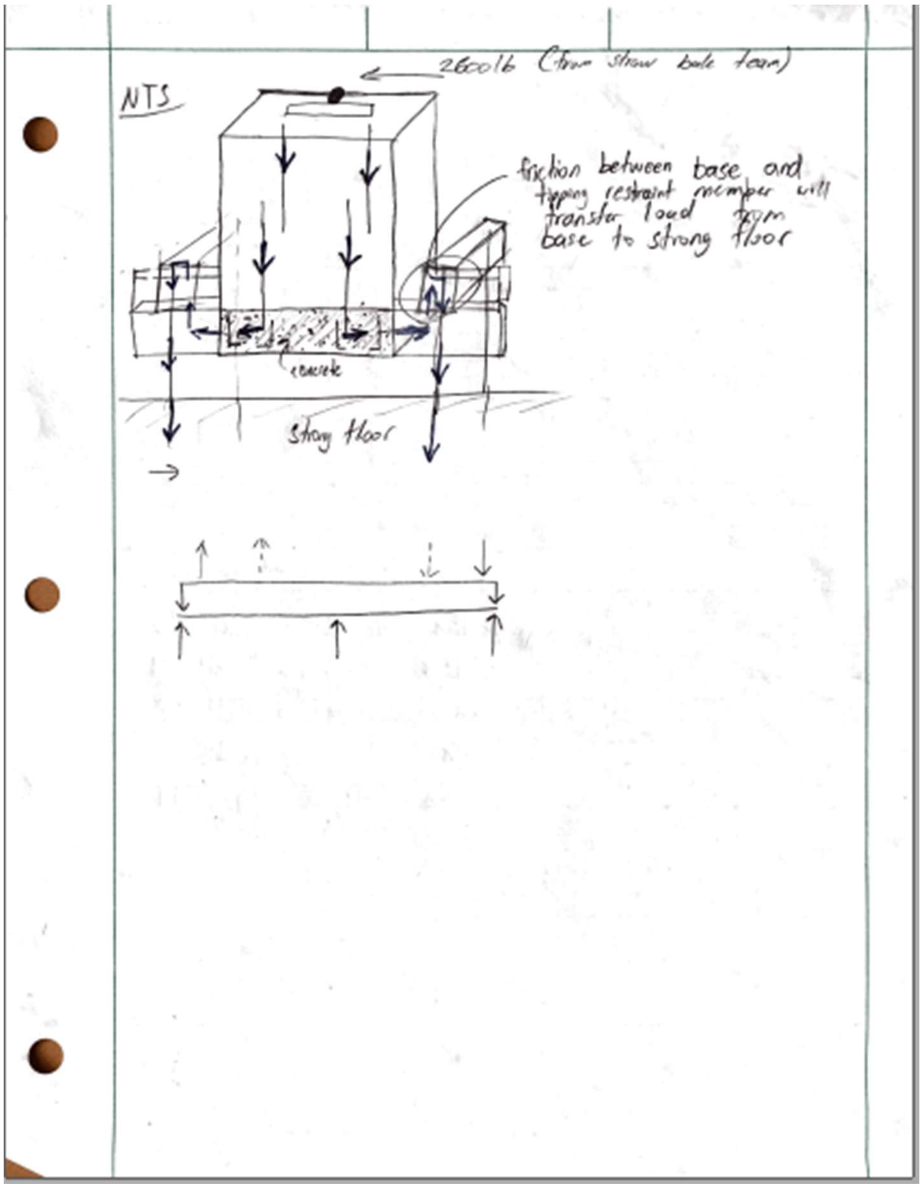
Leg 1

$f_v = 180\text{psi} \cdot \underbrace{2.5\text{in}}_{\text{embed}} \cdot \underbrace{1.75\text{in}}_{\text{depth}} \cdot 2 = 1575$ lbs - well above 900lb needed

Leg 2

$f_v = 180\text{psi} \cdot \underbrace{2.5\text{in}}_{\text{embed}} \cdot \underbrace{1.5\text{in}}_{\text{depth}} \cdot 2 = 1{,}350$lb - well above 450

Free Body Diagram of Forces from TDOF to Wall.



R Value Calculations

Test 1

$R = \frac{V_{tot}}{V_y} = \frac{1551.8}{440.3} = 3.6$

$u = \frac{7.99}{.21} = 38.05$

$R_{cl} = \sqrt{2u-1} = 8.6$

$R = 30.1$

Test 2

$R = 2.2$

$u = \frac{8.5}{.32} = 26.6$

$R_{cl} = \sqrt{2u-1} = 7.22$

$R = 15.9$

Friction Force

Coeff of friction $\mu = .67$

Wall weight = 2089 lbs - approximate

- At base of wall

$F_F = \mu \cdot F_n = .67 \cdot 2089 = 1399.63$

- At Mid height

$F_F = \mu \cdot F_n = .67 \cdot 1044 = 699.5$ lbs

assuming even weight dist

R Value Calculations

Test 1 $\quad 0 = -1150 = -5.3$ $\quad$ Test 2

$\Delta_y = -4.28 = 1.02$ $\quad$ $\Delta_y = -2.79 = 1.31$ $\quad 0 = -1010$

$F_y = -514.74 = 635.26$ $\quad$ $F_y = -424.06 = 585.94$

$\Delta_{ult} = 3.31 \quad 7.99$ $\quad$ $\Delta_u = 4.94 = 9.02$ $\quad 0 = -4.1$

$F_{ult} = +401.8 = 1551.8$ $\quad$ $F_u = -136.60 = 873.4$

Test 2

$$R_o = \frac{V_{str}}{V_y} = -\frac{1551.8}{635.26} = 2.4$$

$$\mu = \frac{7.99}{1.02} = 7.83$$

$$R_d = \sqrt{2\mu - 1}$$

$$\sqrt{2 \cdot 7.83 - 1} = 3.8$$

$$R = R_o R_d = 9.2$$

Test 2

$$R_o = \frac{V_{str}}{V_y} = \frac{873.4}{585.94} = 1.5$$

$$\mu = \frac{\Delta_u}{\Delta_y} = \frac{9.02}{1.31} = 6.89$$

$$R_d = \sqrt{2\mu - 1} = \sqrt{2 \cdot 6.84 - 1} = 3.6$$

$$R = R_o R_d = 1.5 \cdot 3.6 = \underline{5.4}$$

$$\frac{5.4 + 9.2}{2} = \boxed{7.3}$$

R Value Calculations

R-Value: Final Acutn

Test 1

Deflection = -2.54

Force = -1323

$\Delta_y = .21$

$F_y = 635$

Test 2

Deflection = -2.20

Force = -825

$\Delta_y = .43$

$F_y = 586$

Test 1 R

$R_o = \frac{V_u}{V_y} = \frac{1323}{635} = 1.6$

$\mu = \frac{-2.54}{.21} = -2.49$

$R_d = \sqrt{2\mu - 1} = \sqrt{2 \cdot 2.49 - 1}$

$R_d = 4.8$

$R = R_o R_d = 7.7$

Test 2 R

$R_o = \frac{V_u}{V_y} = \frac{825}{586} = 1.4$

$\mu = \frac{2.20}{.43} = 5.11$

$R_d = \sqrt{2\mu - 1} = \sqrt{2 \cdot 1.9 - 1}$

1.5

$R = 4.3$

Expected PLF for Walls.

Anticipated Design Lateral force

$W = 2089$ lbs

$h_n = 8'$

$S_{DS} = 1.0$

$S_{D1} = .52$

$S_1 = .6$

Approx. Period

$T_a = C_t h_n^x$ $\quad C_t = .02$

$t_a = .10s$ $\quad h_n = 8'$, $x = .75$

Seismic Response Coeff

$$C_s = \frac{S_{DS}}{(R/I_e)} \quad \frac{1.0}{(3/1)} = .33$$

$$W = 2089\,\text{lbs}/4 \cdot .33 = \boxed{174\ \text{plf}}$$

Concrete Mix Design.

STEP 1: FIND F'CR

$F'_{cr} = f'_c + 1.34s$

$4000psi + 1.34(.07 \times 4000psi)$

$4000psi + 375.2\ psi$

$F'_{cr} = 4375.2\ psi$

STEP 2: DETERMINE WATER-CEMENT RATIO (TABLE 7.2)

$\frac{.44 - .38}{4500 - 4000} = .00012$

$w/c = .44 - (.0012 \times 375.2psi) = .395$

STEP 3: MAX COURSE AGGREGATE SIZE

MSA = 1"

SPACING UNDER REBAR ≈ 1.5" V_{clear}

STEP 3B: BULK UNIT VOLUME (TABLE 7.5)

NMSA = ½"

COARSE AGGREGATE BULK VOL FACTOR = .53

USING FINENESS MODULUS OF 4.18

STEP 4: AIR ENTRAINMENT (TABLE 7.6)

2.5%

STEP 5: WORKABILITY REQUIREMENTS

SLUMP = 1½ in

STEP 6: WATER CONTENT REQUIREMENTS (TABLE 7.8 AND STEP 6)

$335\ lb/yd^3 - 20\ lb/yd^3 = 315\ lb/yd^3$

Concrete Mix Design.

STEP 7: CEMENT MATERIAL CONTENT (TABLE 7.4)

590 lb/yd^3 – BASED ON ½" NMSA

$.395 = \dfrac{315 \text{ lb } H_2O}{\text{CEMENT WT}}$ – BASED ON W/C RATIO

$\hookrightarrow = 797 \text{ lb/yd}^3$ – GOVERNS

STEP 8: ADMIXTURE REQUIREMENTS

NONE

STEP 9: FINE AGGREGATE REQUIREMENTS

$Vol_{H_2O} = \dfrac{315 \text{ lbs}}{(1)(62.4 \text{ pcf})} = 5.04 \text{ ft}^3$

$Vol_{CEMENT} = \dfrac{797 \text{ lbs}}{(3.15)(62.4 \text{ pcf})} = 4.05 \text{ ft}^3$

$Vol_{COARSE} = \dfrac{(.53)(27 \text{ ft}^3)(101.7 \text{ pcf})}{(2.59)(62.4 \text{ pcf})} = 9.00 \text{ ft}^3$ – USING STEP 8B

$Vol_{AIR} = 2.5\%(27 \text{ ft}^3) = .675 \text{ ft}^3$

$Vol_{FINES} = 27 \text{ ft}^3 - (5.04 + 4.05 + 9.00 + .675 \text{ ft}^3) = 8.23 \text{ ft}^3$

$W_{FINES} = 8.23 \text{ ft}^3 (2.63)(62.4 \text{ pcf}) = 1350 \text{ lb/yd}^3$

STEP 10: MOISTURE CORRECTIONS

$315 \text{ lb}_{H_2O} - 1350 \text{ lb}_{FINES}(.0695 - .0207) - 1455 \text{ lb}_{COARSE}(.0099 - .0104)$

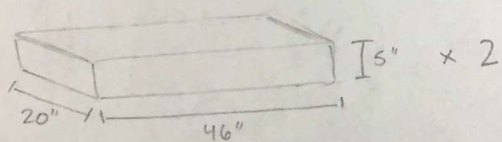
$315 \text{ lb}_{H_2O} + 23.2 + 1.46 = 339.7 \text{ lb}_{H_2O}$

Concrete Mix Design.

SUMMARY

MATERIAL	WT PER 1 yd^3 (lb)	NEEDED WT (lb)*
WATER	339.7	88.3
CEMENT	797	207
FINES	1350	351
COARSE AGG	1455	378

* CALCULATED BASED ON .26 yd^3 REQUIRED FOR BASES



Movement of Courses in Lower Half of Wall 1.

Movement Observed at Top Courses for Wall 1

Movement Observed at Top Courses for Wall 2

Movement of Courses in Lower Half of Wall 1.

Installation of Nut + Washer for Bond Beam to Top Course connection at Top of Wall 2.