

**SANTA CLARA UNIVERSITY**

Department of Civil Engineering

I HEREBY RECOMMEND THAT THE SENIOR DESIGN PROJECT REPORT PREPARED  
UNDER MY SUPERVISION BY

Daniel Eberhard, Joseph Novara, and Brandon Popovec

ENTITLED

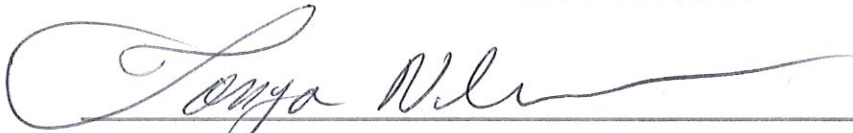
**Cob: A Sustainable Building Material**

BE ACCEPTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE  
DEGREE OF

**BACHELOR OF SCIENCE**

IN

**CIVIL ENGINEERING**



Thesis Advisor

*JUNE 13, 2018*

Date

---

Acting Department Chair

Date

# Cob: A Sustainable Building Material

By

Daniel Eberhard, Joseph Novara, and Brandon Popovec

## **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  
Bachelor of Science in Civil Engineering  
Santa Clara, California  
Spring 2018

# Cob: A Sustainable Building Material

Daniel Eberhard, Joseph Novara, and Brandon Popovec

Department of Civil Engineering

Santa Clara University, Spring 2018

## ABSTRACT

*Cob is a sustainable building material used in residential applications as a replacement for typical one-story timber or concrete structures. Cob heavily lacks the testing and modern research required for permissible construction under current residential building codes. Further tests involving earthquake and lateral loading are required to implement cob into the California Residential Building Code. In this report, four cob walls, each with different reinforcement mechanisms, were tested to observe their failure modes and find the maximum deflection and load of each wall. Through this testing, it was determined that steel reinforcement within cob walls can provide more ductile behavior and increase both maximum loads and displacement of the material. Analysis showed the ideal reinforced cob wall had an estimated Response Modification Factor of 2.5 which reduces design loads significantly when considering lateral seismic loading. The combination of vertical steel and lateral wire mesh reinforcement throughout a cob wall proved to be the most efficient way to reinforce the structure. The key to properly reinforce cob involves using materials that are large enough to transfer load but small enough to not create large voids within the cob matrix. There is potential for the material to be used in permitted buildings, yet much more full scale research must be done to solidify its capabilities.*

## ACKNOWLEDGEMENTS

We would like to extend a special thank you to Brent Woodcock, Tonya Nilsson, Anthony Dente, John Fordice, Jose Espinoza, Gabi Brunello, Alex Golitz, Verdant Structural Engineers, the Cob Research Institute, Kita Glass and all of the cob enthusiasts that helped us along our journey.

## Table of Contents

Certificate of Approval.....	i
Title Page.....	ii
Abstract.....	iii
Acknowledgements.....	iv
Table of Contents.....	v
List of Figures.....	vi
List of Tables.....	vii
1.0.0 Introduction.....	1
1.1.0 Project History.....	1
1.2.0 Project Need.....	1
2.0.0 Project Overview.....	2
2.1.0 Project Scope.....	2
2.2.0 Analysis of Alternatives.....	2
3.0.0 Design Criteria and Standards.....	3
3.1.0 Constructability of Cob.....	3
3.2.0 Mix Design.....	4
3.3.0 Reinforcement Systems.....	7
3.3.1 Wall 1 (Common Practice).....	7
3.3.2 Wall 2 (Heavy Reinforcement).....	9
3.3.3 Wall 3 (Medium Reinforcement).....	10
3.3.4 Wall 4 (Light Reinforcement).....	12
3.4.0 Base Design.....	13
3.5.0 Top Plate Design.....	14
3.6.0 Wall Restraint Design.....	16
3.7.0 Project Challenges.....	17
4.0.0 Testing Data and Analysis.....	18
4.1.0 Wall 1 (Common Practice).....	19
4.2.0 Wall 2 (Heavy Reinforcement).....	20
4.3.0 Wall 3 (Medium Reinforcement).....	22
4.4.0 Wall 4 (Light Reinforcement).....	24
4.5.0 Discussion.....	26
4.6.0 Seismic Response Modification Factor.....	27
5.0.0 Future of Cob.....	31
6.0.0 Conclusion.....	32
7.0.0 References.....	33
8.0.0 Appendix.....	34

## List of Figures

Figure 1: Example of Thumbed Cob .....	4
Figure 2: Wall 1 Reinforcement System (Common Practice) .....	8
Figure 3: Wall 1 Base Design (Common Practice) .....	8
Figure 4: Wall 2 Reinforcement System (Heavy Reinforcement) .....	10
Figure 5: Wall 3 Reinforcement System (Medium Reinforcement) .....	11
Figure 6: Wall 4 Reinforcement System (Light Reinforcement) .....	12
Figure 7: Completed Base for a 1.25:1 Aspect Ratio Cob Wall .....	14
Figure 8: Top Plate Design Configuration .....	15
Figure 9: Wall Testing Restraint System .....	16
Figure 10: Wall 1 (Common Practice) After Testing .....	19
Figure 11: Wall 1 (Common Practice) Hysteresis Diagram .....	20
Figure 12: Wall 2 (Heavily Reinforced) After Testing .....	21
Figure 13: Wall 2 (Heavily Reinforced) Hysteresis Diagram .....	22
Figure 14: Wall 3 (Medium Reinforced) After Testing .....	23
Figure 15: Wall 3 (Medium Reinforced) Hysteresis Diagram .....	24
Figure 16: Wall 4 (Light Reinforced) After Testing .....	25
Figure 17: Wall 4 (Light Reinforcement) Hysteresis Diagram .....	26
Figure 18: Wall 4 (Light Reinforced) Maximum Peak Values .....	28
Figure 19: Stress Strain Curves for All Walls .....	29
Figure 20: Wall 3 Linear Regression Analysis .....	30

## List of Tables

Table 1: 1:1 Sand to Soil Ratio .....	5
Table 2: 1:1.5 Sand to Soil Ratio .....	5
Table 3: 1:2 Sand to Soil Ratio .....	6
Table 4: Wall Test Results .....	18
Table 5: Wall Test Results .....	30

## **1.0.0 Introduction**

### **1.1.0 Project Need**

With the world's population rapidly increasing and harmful carbon emissions rising, it is important to implement sustainable construction materials to help protect the environment. Typical construction materials such as concrete and steel, have a heavy impact on the environment, which is why research in alternative, sustainable materials is being brought to the public eye more and more. Typical construction materials are not only harmful to the environment, but they also are also expensive to manufacture and construct. Cob is a material that offers a solution to both problems. It does not take carbon to produce and is a low cost alternative to steel or timber houses. Cob is cost effective because of its four common ingredients: high clay content soil, straw, sand and water. Cob does not require technical training to construct, which reduces the construction costs and enables a variety of people to build structures from the material. Cob structures are a great option for construction in both eco-friendly communities and developing nations. Although cob has been around for thousands of years, modern cob building techniques and mix ratios lack consistency and extensive research. Since there are many people living in unpermitted cob houses, there is a need for research on various characteristics of the material.

### **1.2.0 Project History**

Cob as a building material is said to have originated in England sometime around the 16th century. It can be found across the globe in temperate to extreme climates. Many of the oldest buildings in Afghanistan and England are constructed with cob or cob-like earthen building materials. Cob has been tested in various settings in the past on both small scale and full scale levels. Previous tests have typically had high levels of uncertainty due to the varying properties of the sand soil and straw used to make cob. New Zealand has achieved successful results with cob and other earthen materials. As a result, cob has been implemented into the New Zealand code (NZS 4297:1998: Engineering Design of Earthen Buildings). The code has reasonable starting points for shear and uplift values that are needed for the construction of the walls. In addition, many small scale tests have been done in universities within the United States. University of San Francisco's "Effect of Straw Length and Quantity on Mechanical Properties of Cob" report yielded results for compression testing that serve as initial base values and

expectations for the small scale tests. Although the work has been done before, cob is very dependent on the specific type of soil, sand, and straw that make up the specific mix. As a result, it is necessary to acquire values that are ideal for the areas materials.

## **2.0.0 Project Overview**

### **2.1.0 Project Scope**

The scope of work for this project was the construction and testing of four different reinforced cob wall designs. The objective was to create usable experimental data on full sized cob walls to ultimately determine the effectiveness of the structures during seismic activity. This scope included initial compression tests of different cob mix designs to determine the desired percentage of sand, clay, and straw for the full scale walls with varying reinforcement systems and aspect ratios. Bases, top plates and restraint systems were designed for wall testing. Four full scale walls were built using these designed components and were tested in the Santa Clara University Multi Degree of Freedom (MDOF) test frame. Through data collection of displacement, maximum loading, and failure mechanisms, a Response Modification Factor used to determine lateral design loads was estimated. This report specifically aimed to provide evidence to support the inclusion of cob into the California Residential Building code. In addition, the failure modes of each wall are a vital piece of data that will help analyze how cob interacts with reinforcement elements. By constructing walls that contain different aspect ratios or reinforcement systems, predictions and subsequent results of the wall tests can reveal insight into the nature of the material when interacting with ductile supporting materials and lead future research in the direction of improved building techniques.

### **2.2.0 Analysis of Alternatives**

In recent years, cob has seen an increase in popularity due to its low cost of materials and sustainability of construction. There are certainly pros and cons to cob construction when compared to the various common building practices. One large drawback for cob during construction compared to wood, concrete, and straw bale construction is the time required to erect a cob structure. Between the acquisition of materials as well as the batch by batch mixing and application process, a several hundred square foot structure can take a dozen workers, working full days, several weeks to complete. Compared to the alternatives, however, cob is a

successful material when it comes to sustainability. Concrete requires skilled labor, additional equipment, and industry wide is responsible for 5% of the world's carbon dioxide production. Wood frame structures are not fireproof and can generally be more expensive to maintain in the long run. Cob structures can be constructed with no release of carbon or greenhouse gases into the atmosphere if done completely with human power.

The walls built for this project had varying aspect ratios and reinforcement systems. One alternative to this design would be to have four identical walls, providing iterative tests. Iterative testing provides data that can be compared together, which gauges the precision of the results. Studying the results for only one system reduces the amount of information gathered about the different wall designs. Since there is limited information about full scale cob wall tests, the information about the different reinforcement systems and failure mechanisms are valuable. Future projects could implement iterative testing, but for this project, it was decided that it was more valuable to pursue information about multiple wall systems to set a baseline for future testing.

### **3.0.0 Design Criteria & Standards**

#### **3.1.0 Constructability of Cob**

The greatest challenge of building with cob is that modern building techniques rely heavily on touch and feel, instead of quantifiable measurements. In the field, sand and soil are mixed together with various ratios and then water is added until the material “feels right.” During the first stage of this project, set amounts of high clay content soil, sand, and straw had to be determined for consistency. The initial mixture of soil, sand, and water was done in a concrete mixer to increase efficiency. To determine if the mix was at the appropriate moisture level, a handful of the mix was taken out of the concrete mixer and compacted into a handful sized ball. The ball was then thrown up two to three feet in the air four times and if the ball was able to hold together when caught, the mix was ready to have straw added. The mixture of soil, sand, and water was then placed onto a tarp. The straw was manually integrated by coating the sand and soil with straw and then stomping the straw into the mix. From there the mix was rolled and folded. More straw was used to coat the batch, and this process was repeated until the entirety of the straw was integrated evenly throughout the batch (the exact quantity of straw is discussed in

section “3.2.0 Mix Design”. Once the mixture was complete, the cob was then manually formed onto the walls and integrated into the previous lift with a technique called “thumbing.” Thumbing involves placing a lump of cob onto the structure and then using a stick or thumb like object to push the new cob into the previous layer. This technique is done every few inches to insure the straw was thoroughly integrated into the previous layer. CRI Director, John Fordice, provided a demonstration for proper thumbing techniques. A visual of a layer of cob that has been thumbed is shown below in *Figure 1*.



*Figure 1: Example of Thumbbed Cob.*

Due to the high moisture content of material during construction, lifts were limited to four inches per session to avoid slump. The cob then had to dry for two to three hours before any additional lifts could be added. If the cob was allowed to dry for any more than eight hours, water would have to be applied to the top lift to retain the desirable moisture content so that the cob was well integrated between layers.

### **3.2.0 Mix Design**

Finding the highest strength mix of sand, straw, and soil involved testing three different types of cob samples in compression. The samples were created with sand to soil ratios of 1:1, 1: 1.5 and

1:2 (straw was kept as a constant) and labeled samples A, B, and C respectively. The compression tests were performed based on ASTM C 39 (ASTM C39 / C39M. – 15 2015). The protocol for the testing standard was followed in regards to how the test was performed, however the cob specimens were rectangular shaped and uncapped as opposed to cylindrical with capping. The width of the testing cylinders do not allow straw to fully integrate into the cob matrix and are not representative of the material in a full scale structure. As a result, the larger cob prism was used to provide ample volume for the straw to interact with the sand and clay, creating a sample that was more indicative of full scale behavior. The samples were compressed at a rate of 0.030 inches per minute until load peaked. The average maximum peak loads and standard deviation were recorded for each mix ratio as can be seen in *Tables 1-3*.

*Table 1: 1:1 Sand to Soil Ratio.*

Sample	Width (in)	Length (in)	Height (in)	Density (lb/ft <sup>3</sup> )	Peak Load (lb)	Peak Stress (lb/in <sup>2</sup> )
A1	8.0	8.0	7.0	95.3	10000+*	160+*
A2	8.0	8.0	7.0	96.8	3860	60.3
A3	8.0	8.0	7.0	90.6	10000+*	160+*
Mean					8120+*	125+*
STD					3690+*	58+*

*\*machine limit was reached before sample broke*

*Table 2: 1:1.5 Sand to Soil Ratio.*

Sample	Width (in)	Length (in)	Height (in)	Density (lb/ft <sup>3</sup> )	Peak Load (lb)	Peak Stress (lb/in <sup>2</sup> )
B1	8.0	8.0	7.0	102.6	4336	67.8
B2	8.0	8.0	7.0	102.2	4611	72.0
B3	8.0	7.5	7.5	97.9	4785	79.8
B4	8.0	8.0	7.5	94.7	4472	69.9
Mean					4551	72.4
STD					192.2	5.2

*Table 3: 1:2 Sand to Soil Ratio.*

Sample	Width (in)	Length (in)	Height (in)	Density (lb/ft <sup>3</sup> )	Peak Load (lb)	Peak Stress (lb/in <sup>2</sup> )
C1	8.0	8.0	7.5	99.7	8501	132.8
C2	8.0	8.0	7.0	106.8	9423	147.2
C3	8.0	8.0	7.5	101.9	9377	146.5
Mean					9100.3	142.2
STD					519.55	8.12

The 1:1 mix ratio was selected due to the compressive strength of samples A1 and A3, which each resisted over 10,000 pounds. Sample A2 did not perform to the same level as the other two, receiving 3,860 pounds before failing. Sample A2 was damaged prior to testing, which likely contributed to its reduced strength. Samples A1 and A3 outperformed all other samples. An exact peak load was not found, because of the limitations of the compression apparatus. Due to the success of those samples, the 1:1 ratio was selected as the mix design to use for the duration of the research.

Further small scale testing was conducted on the 1:1 soil to sand ratio, the details of which can be found in the Santa Clara University Cob Property Analysis 2018 Senior Design Report.

In common practice, straw is added until the product “feels right” so there was a focused effort on quantifying how much straw was required, with heavy reliance on the expertise of John Fordice from the Cob Research Institute. Through careful measuring, it was determined that 120 grams of straw was required per 0.71 cubic feet of soil and sand.

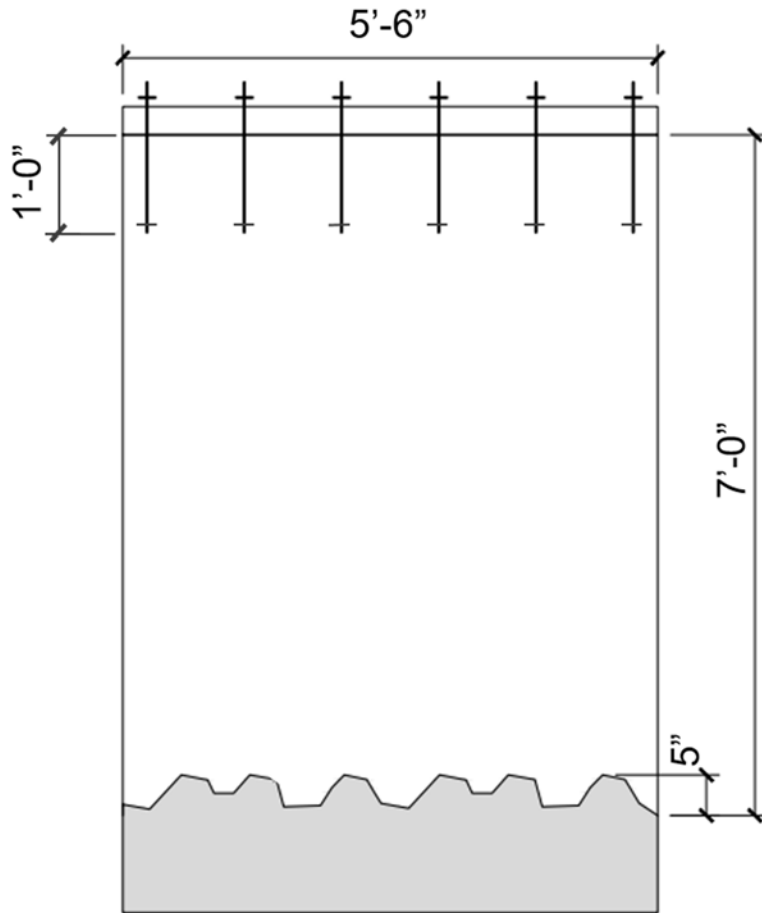
The exact amount of water added to the batches of cob varied from day to day. Since the soil was stored outside, the moisture content of the material was inconsistent, and water was added until the appropriate consistency was created. This is one area that could be quantified further for future research, although it would likely require daily moisture tests or storing all materials in a controlled environment.

### **3.3.0 Reinforcement Systems**

Cob is traditionally built in the field with no tensile reinforcement (excluding the straw mixed into the material). Due to its brittle characteristics, four unique reinforcement systems were selected to observe the ductility of the structure and examine the different failures caused by the lateral loading from the MDOF test frame.

#### **3.3.1 Wall 1 (Common Practice)**

The first wall system was a common practice wall shown in *Figure 2*. Wall 1 (Common Practice) has no steel reinforcing structure, apart from the six (6) pieces of #5 threaded rod spaced 12 inches on center at the top of wall. The threaded rods were added to transfer load from the top plate timber members to the cob wall and were embedded 12 inches into the wall with 2.5x2.5 bearing plates at the bottom. At six feet of height, a scaffolding system was constructed to hold the rods in place as cob was packed around them. To avoid shearing through the cob, the threaded rod had a minimum edge spacing of 3 inches. A foundation of rocks protruding five to six inches out of the concrete base shown in *Figure 3* was designed to mimic common cob practices. The rocks were placed while the concrete was setting at the perimeter of where the cob was to be placed as well as laterally throughout the wall at 12 inch intervals. The purpose of this wall was to give insight into how existing cob structures perform when laterally loaded. Wall 1 had a height of seven feet (7'), a length of five feet six inches (5'6"), and a depth of 16 inches (16"). The wall had a 1.25:1 aspect ratio to encourage a shear failure in the wall.



*Figure 2: Wall 1 Reinforcement System (Common Practice).  
Drawing Courtesy of Verdant Structural Engineers*



*Figure 3: Wall 1 Base Design (Common Practice).*

### 3.3.2 Wall 2 (Heavy Reinforcement)

The reinforcement method for Wall 2 was a rebar matrix, with horizontal #5 rebar installed 12 inches (12") on center and four vertical pieces running the full seven foot (7') height. The configuration can be seen in *Figure 4*. The #5 rebar was embedded into the concrete base and secured with scaffolding until the wall could support the rebar cage. Similar to Wall 1, there was a minimum three inch (3") coverage between the rebar and outside of the wall. Horizontal pieces of rebar were placed on top of the wet cob and tied to the vertical rebar with tie wire as construction progressed. The top two feet (2') of rebar were replaced with couplers and threaded rod protruded six inches (6") out of the top of the wall, allowing for tie in to the top plate. The purpose of the rebar in this wall was to replicate the kind of reinforcement found in a typical concrete wall. This wall was constructed and tested to determine how the cob interacts with rebar and determine its viability to add strength and ductility to a cob structure. Wall 2 had a reinforcement configuration and aspect ratio different from the previous wall. At a length of three feet and six inches (3'-6"), Wall 2 had an aspect ratio of 2:1 to induce a flexural failure.

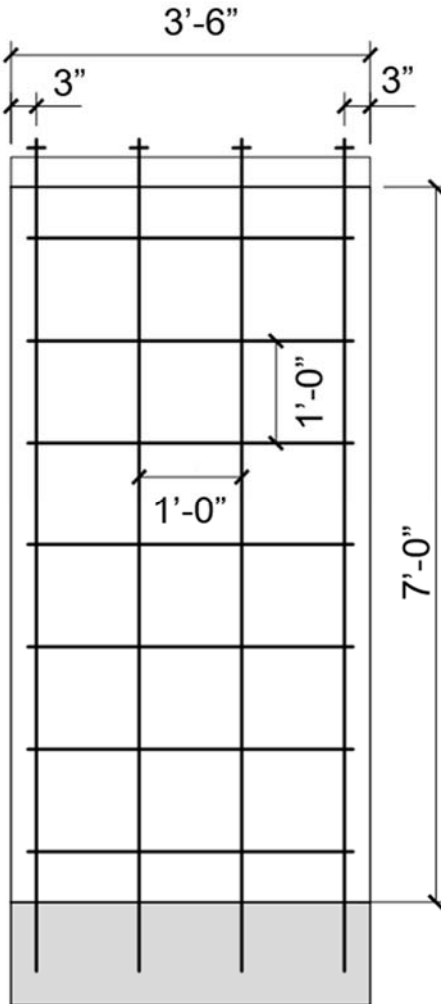
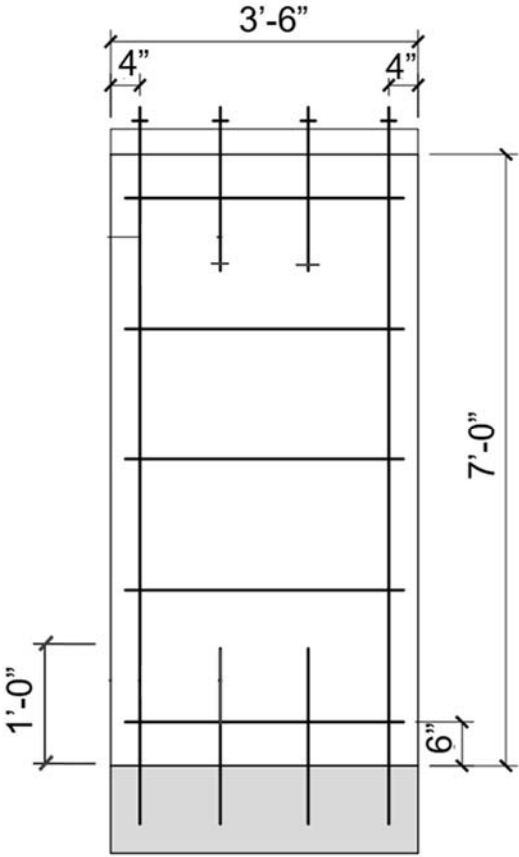


Figure 4: Wall 2 Reinforcement System (Heavy Reinforcement).  
 Drawing Courtesy of Verdant Structural Engineers

### 3.3.3 Wall 3 (Medium Reinforcement)

The reinforcement method for Wall 3 utilized rebar and welded wire mesh, with 14 gauge welded wire mesh installed horizontally every 18" on center and two (2) vertical pieces of #5 rebar running the full seven foot (7') height. In addition to the full length rebar, two pieces of #5 threaded rod were placed in between at 12 inches (12") on center on both the top and bottom of the wall, penetrating the cob 12 inches (12"). Bearing plates, which were 2.5" x 2.5", were placed at the bottom of the all thread rods at the top of the wall to help transfer the load from the MDOF test frame into the wall. The configuration can be seen in *Figure 5*. The #5 rebar was embedded into the concrete base and secured with scaffolding until the wall could support the rebar cage. Similar to Wall 1, there was a minimum three inch (3") coverage between the rebar and outside of the wall. Horizontal pieces of welded wire mesh were placed on top of the wet

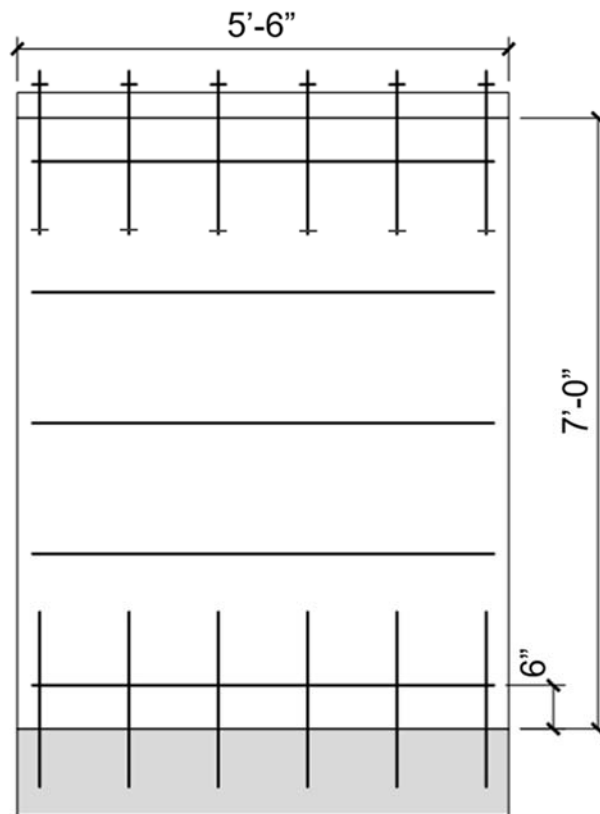
cob as construction progressed. The top two feet (2') of rebar within the wall was replaced with couplers and threaded rod protruding six inches (6") out of the wall to allow for tie in to the top plate. The purpose of the rebar and welded wire mesh was to reinforce the wall with materials that would be easier to use in the field. Welded wire mesh is a lighter material that is easier to transport and cut compared to rebar. This wall was constructed and tested to determine how the cob would interact with the welded wire mesh and rebar to see if these materials would help increase the ductility and strength of the wall. Wall 3 had a different reinforcement configuration, but the aspect ratio was the same as the previous wall. At a length of three feet and six inches (3'-6"), Wall 3 had an aspect ratio of 2:1 to induce a flexural failure.



*Figure 5: Wall 3 Reinforcement System (Medium Reinforcement).  
Drawing Courtesy of Verdant Structural Engineers*

### 3.3.4 Wall 4 (Light Reinforcement)

The reinforcement methods used for Wall 4 combined all thread rods and welded wire mesh, similar to Wall 3. Vertical #5 threaded rods penetrated 12 inches (12") into the cob at the top and bottom of the wall. Bearing plates, which were 2.5" x 2.5", were added to the bottom of the threaded rods at the top of the wall to help transfer the load from the MDOF test frame into the cob. The threaded rods were placed 12 inches (12") on center along the middle of the base and top of the wall. Welded wire mesh was used for the horizontal reinforcement of Wall 4. The first layer of welded wire mesh was placed 6 inches above the top of the base and the following layers of the mesh were placed 18 inches (18") on center for the remainder of the wall. The reinforcement of Wall 4 can be seen in *Figure 6*. Wall 4 was constructed and tested to determine how the cob would interact with the welded wire mesh if there was no vertical reinforcement that ran the entire height of the wall. Wall 4 had a height of seven feet (7'), a length of five feet six inches (5'6"), and a depth of 16 inches (16"). The wall had a 1.25:1 aspect ratio to encourage a shear failure.



*Figure 6: Wall 4 Reinforcement System (Light Reinforcement).*

*Drawing Courtesy of Verdant Structural Engineers*

### 3.4.0 Base Design

In order to test the material in the MDOF Testing Frame, a base for the walls had to be designed. There were a few primary design concerns that dictated the parameters of the bases. The first concern was the weight of the completed walls bearing on the bases. The walls had to be moved from the storage locations to the testing machine using a forklift. Deflection of the base between the forks had to be considered to protect the walls from damage during transportation. To prevent this deflection, the allowable deflection of the bases were determined using *Equation 1*, the allowable concrete floor deflection equation per ACI 318 (ACI 318 2014).

$$\Delta = L/480, \quad (\text{Equation 1})$$

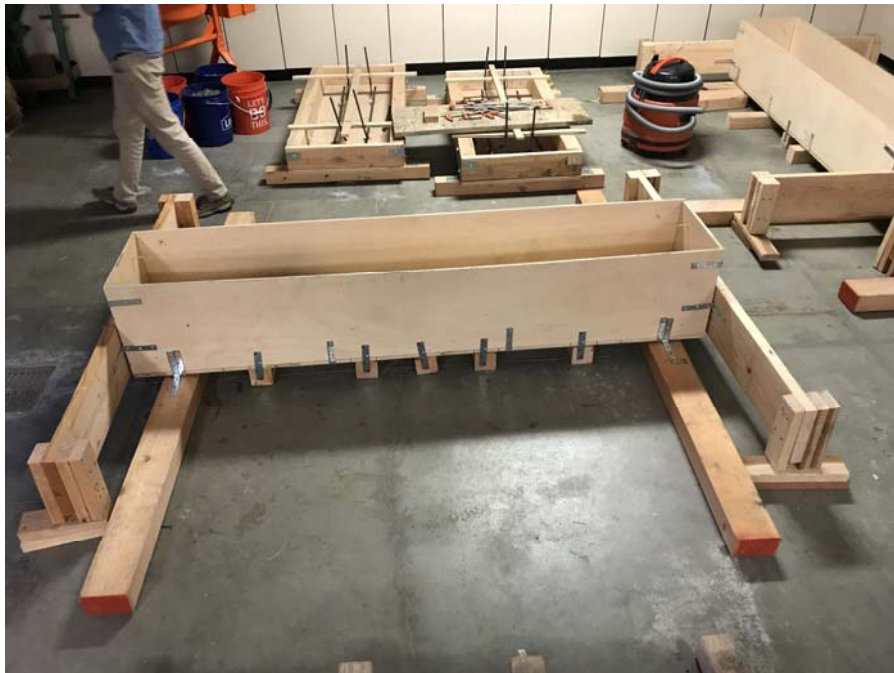
where  $\Delta$  is the allowable deflection and  $L$  is the length of the base between supports (inches). The forks were assumed to be at the edges of the longest wall (66 inches), as this is the worst case scenario. It was found the bases would have an allowable deflection of 0.1375 inches.

The bases were over designed so that all failure during testing would occur in the cob and not in the concrete. One potential failure in the concrete was the rebar pullout due to uplift. Designing the required embedment of the rebar required an estimate of the expected uplift force during testing. After discussions with Anthony Dente of Verdant Structural Engineers, some assumptions were made, with help from the New Zealand Building Code NZS 4297 (Morris 1998), about the expected maximum lateral force before the walls failed. The estimated lateral force was 1,145 pounds per linear foot of wall and due to the limited research on cob, an additional factor of safety of three was applied for a final design lateral force of 3,440 pounds per linear foot, yielding a total of 18,900 pounds in Walls 1 and 4 (1:1.25 ratio) and 12,080 pounds in Walls 2 and 3 (1:2 ratio).

Using the assumed 18,900 pounds of force for Walls 1 and 4, a maximum uplift force of 22,350 pounds was calculated, resulting in a required a depth of rebar of 12 inches (12"). Four inches of coverage between the vertical and horizontal rebar and the bottom of the base was decided after discussion with CRI, resulting in a final base depth of 16 inches (16").

To finalize the bases, an outrigger system was designed. The outriggers were included to add stability to the walls both during construction and for the three months of drying. The system included two 4x6 timber beams, which were placed underneath the edge of each base and extended out three feet on each side as shown in *Figure 7*.

Type equation here.



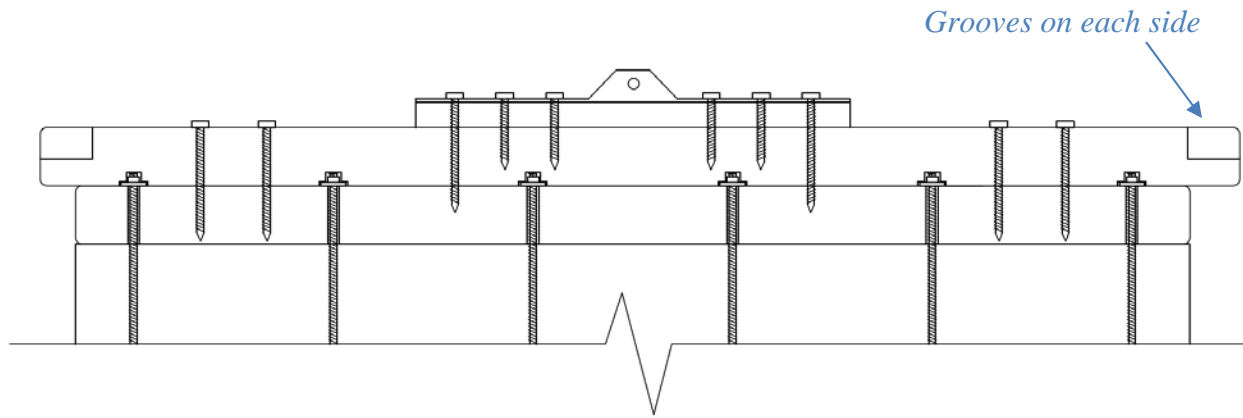
*Figure 7: Completed Base for a 1.25:1 Aspect Ratio Cob Wall.*

In addition, 4x4 pieces of lumber were placed beneath the wall at six inch (6") intervals. These 4x4 pieces did not extend beyond the width of the base, supporting the base and preventing deflection between the 4x6 beams. This design allowed enough space for forks to get underneath the wall. The limited space in the MDOF test frame required the outriggers to be removable and the base to sit flush to the ground.

### **3.5.0 Top Plate Design**

Unlike shake tables, the MDOF test frame applies load cyclically through a pin connection at the top of the wall. The walls require a header that is able to transfer load through the top of the wall and into the cob matrix. In the initial design phase, timber was chosen as the ideal material for

the top plate for both ease of construction and adequate strength. An elevation view of the top plate can be seen in *Figure 8*.



*Figure 8: Top Plate Design Configuration.*

The top plate, as with the base of walls, was designed for 3,440 pounds per linear foot of lateral load, to ensure that failure occurred in the cob wall system. Immediately after the final lift of cob was applied, a 4x8 flat piece of Douglas fir north, running the length of the wall, was embedded into the wet cob. Holes were drilled so that the six pieces of threaded rod in the cob could penetrate through the embedded 4x8. Threaded rods were then secured with washers and bolts. After the cob dried, an additional 4x8 flat piece was placed above the original for constructability of the pin connection to the MDOF test frame. The two pieces of timber were then secured together using eight three quarter inch ( $\frac{3}{4}$ " ) lag screws to act as one cohesive top plate. In addition, six one inch (1") diameter lag screws were used to fasten the steel plate pin connection to the upper 4x8 member. Shear force in the bottom 4x8 transferred by the threaded rod, governed the design of the top plate. Shear force through the member was found using *Equation 2*

$$\tau = \frac{V}{2 \times L_{embed} \times edge\ distance}, \quad (Equation\ 2)$$

where  $\tau$  represents allowable shear stress in a member,  $V$  represents the shear force applied,  $L_{embed}$  represents the embedded distance of the lag screw into the wood member, and the edge distance represents the minimum spacing between the lag screw and edge of wood. With a minimum edge distance of three inches (3") and a minimum allowable shear stress of 170 psi for

Douglas fir north based on 2012 National Design Specifications for Wood Construction (NDS 2012) values, the allowable shear force in the wood was 4170 pounds per threaded rod. The estimated maximum lateral force of 18,900 pounds, produced a value of 3150 pounds per threaded rod. Lastly, the lag screws were checked for shear failure and crushing of wood. Grooves 1.5"-2" deep were cut four to five inches (4"-5") into the top plate on either end of the of the upper 4x8 as pictured in *Figure 6*. T-plates attached to the MDOF test frame shown clearly in *Figure 14* sat within these grooves, reducing any torsional or out of plane movement of the wall during testing.

### 3.6.0 Wall Restraint Design

Each wall needed to be restrained in the MDOF testing frame to prevent overturning and sliding. To resist overturning, a select structural Douglas fir 6x10 beam was placed horizontally across the concrete base and tied into the strong floor with two one inch (1") diameter threaded rods at each end, as shown in *Figure 9*. Calculations were completed to verify that the timber and rods were sufficient to resist the estimated maximum lateral force of 18,900 pounds.



*Figure 9: Wall Testing Restraint System.*

Based off of the estimated design load of 18,900 pounds, the maximum moment was 16,000 pound feet. The maximum allowable moment of the timber was 18,000 pound feet. The estimated forces required each rod to be able to resist a maximum uplift force of 4,560 pounds. The limiting factor of this system is the wooden beams. To verify that the bearing plate would not cause the wooden beam to fail, the maximum shear calculation was completed for the metal plates. Using the NDS (NDS 2012), it was determined that it would require 68,750 pounds for the metal plate to shear through the 11” height of the timber beam. These calculations showed that the restraint system would be able to resist the forces produced by the MDOF testing frame, during the cyclical loading of the four walls.

### **3.7.0 Project Challenges**

One of the major challenges of this project was dealing with limited working space. The layout of the walls was critical because each had to be built inside of the lab and then moved three months later after they had finished drying. The layout had to provide enough space to construct the walls and also leave enough room for the forklift to have access to each wall.

The final base design had a depth of 16 inches, a width of 16 inches and a length of either five feet six inches (5’6”) or seven feet seven inches (7’7”). The bases required 1.2 cubic yards of concrete, which was donated by Tom Albanese at US Concrete and delivered to the Santa Clara University campus. The concrete cured for one week before the construction of the walls began.

The soil was stored outside, which affected its moisture content and resulted in slight daily adjustments to the mix design. Each day, the correct water amount was determined by adding water incrementally to the first batch until the cob mixture was at the desired consistency. The same amount of water was used for the remainder of the mixes produced that day. The moisture content of the mix affects workability of the cob.

There were issues finding threaded rods that were long enough to run the full height of the wall. Continuous threaded rods were used in the two smaller walls from the base to the top plate. Since a threaded rod of the required length was expensive, #5 rebar was used with couplers at the top, which allowed the rebar to transition into the required rod. The couplers caused cracking in Walls 2 and 3.

Using the calculated cob density of 96 pounds per cubic foot, shown in *Table 1*, the final weight of the small walls with the base were each roughly 5200 pounds, and the large walls were each 7500 pounds. Santa Clara University has a forklift with a lifting capacity of 5600 pounds. One of the 7500 pound walls was built in the MDOF testing frame but the other large wall had to be moved, which required a rented forklift. There were an abundance of administrative issues to be resolved before the rented forklift could be ordered, including appropriate signatures and final confirmation that it was acceptable to place the order for the forklift.

Transporting the walls was an additional challenge because the walls had to be taken out of the structures lab and driven 150 feet down the road to the MDOF testing frame. To help minimize risk during moving the walls, the road was closed down with cones, and flaggers were placed on both ends of the road to redirect pedestrian traffic. The wall was placed on four wooden blocks so that the forklift could pull its forks out without knocking the wall over. Tubular webbing was used with rigging hoists to lift half of the wall up off the block. Once the two blocks were removed, the wall was slowly lowered to the ground. Once secure, the other half of the wall was then lifted so that the remaining blocks could be removed, and then the wall was placed on the ground.

#### 4.0.0 Testing Data and Analysis

*Table 4* provides a summary of the four tests. In addition to reinforcement styles, aspect ratios, maximum load, and displacement of walls, failure modes and testing observations can be found in the following sections.

*Table 4: Wall Test Results.*

Wall	Reinforcement	Aspect Ratio (H:L)	Max Lateral Load (lbs)	Lateral Load per Foot (plf)	Maximum Displacement (in)
1	None	1.25:1	6509	1183	2.5
2	Heavy	2:1	3104	887	5.08
3	Medium	2:1	4693	1324	8.19
4	Light	1.25:1	6971	1267	4.87

#### 4.1.0 Wall 1 (Common Practice)

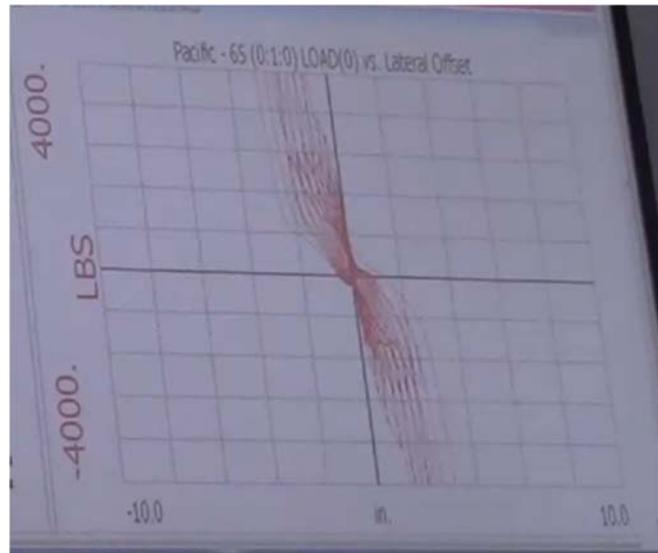
The first wall tested was the common practice wall (1.25:1 ratio). In the cyclic loading machine, the wall took a lateral force of 1183 pounds per foot or 6509 pounds total. The wall failed prior to reaching the maximum displacement in the machine (8.4 inches). Wall 1 took more lateral force than its self-weight of 5,745 pounds. This wall failed in shear, and the diagonal cracks can be seen below in *Figure 10*.



*Figure 10: Wall 1 (Common Practice) After Testing.*

The ultimate failures in the wall developed along a lift, indicating these may be a weak point in the material. Before failure, the wall had some minor uplift off of the stone connections in the base, indicating that load was being transferred through all seven feet of the wall.

Unfortunately during this test, the data tracking the stress and displacement of the wall was not recorded. Some data involving peak loads was able to be extracted from the video data thanks to Santa Clara University student Jose Espinoza. The hysteresis loops show a maximum displacement of 2.5 inches, which can be seen in *Figure 11*.



*Figure 11: Wall 1 (Common Practice) Hysteresis Diagram.*

#### **4.2.0 Wall 2 (Heavily Reinforcement)**

The heavily reinforced (2:1 aspect ratio) received a maximum lateral force of 887 pounds per foot or 3104 pounds total. This heavily reinforced wall utilized couplers in the construction of the rebar cage. A horizontal crack was observed prior to testing Wall 2. The couplers added to this wall were likely the source of the crack and may have decreased the maximum lateral load the wall could resist. Failure occurred at the base of the couplers which can be seen in *Figure 12*.



*Figure 12: Wall 2 (Heavily Reinforcement) After Testing.*

In addition, the rebar did not integrate well with cob. As the test continued, the rebar created voids in the cob, indicating the steel cage and cob were moving independently. Unlike concrete, the cob did not bond with the rebar, suggesting a full rebar cage is not the ideal reinforcement mechanism for cob. The hysteresis loops in *Figure 13*, show Wall 2 had five inches (5”) of lateral offset. The hysteresis loops for Wall 2 also show a significant decrease in load as the lateral displacement increases.

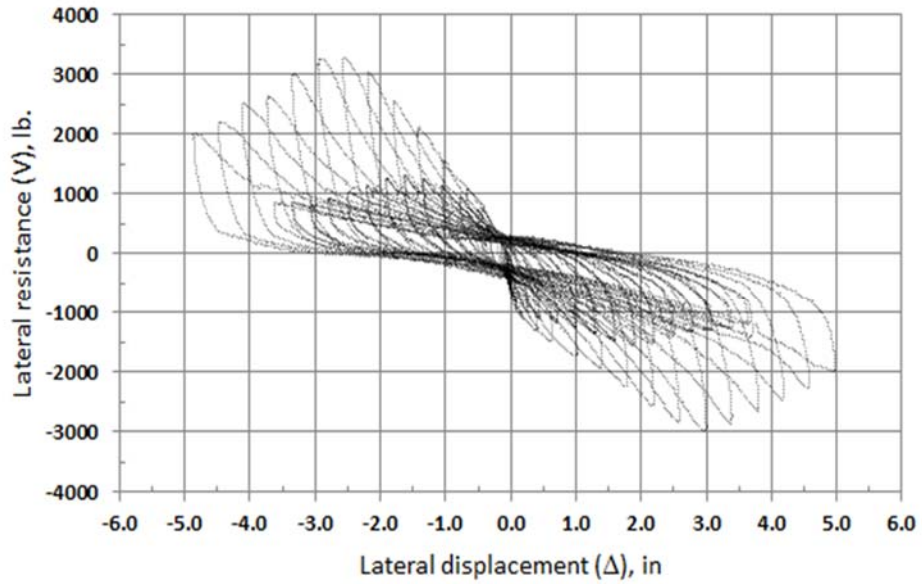


Figure 13: Wall 2 (Heavily Reinforcement) Hysteresis Diagram.

#### 4.3.0 Wall 3 (Medium Reinforcement)

The medium reinforced wall (2:1 aspect ratio) experienced a peak load of 1342 pounds per foot and a total force of 4693 pounds. Wall 3 failed in the plane where the couplers were installed, as can be observed in *Figure 14*.



*Figure 14: Wall 3 (Medium Reinforcement) After Testing.*

The welded wire mesh offered many different points of contact for the surrounding cob, helping the two move as one unit. The rebar, which ran the total height, appeared to distribute forces horizontally to each piece of mesh. The hysteresis loops in *Figure 15* show eight inches (8") of displacement, completing all cycles on the MDOF test frame without collapsing.

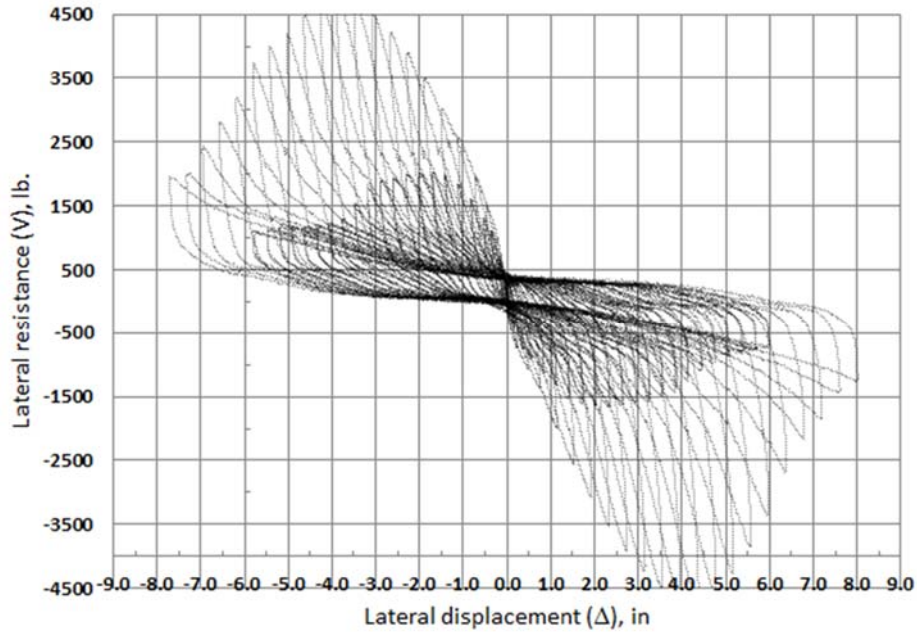


Figure 15: Wall 3 (Medium Reinforcement) Hysteresis Diagram.

#### 4.4.0 Wall 4 (Light Reinforcement)

The lightly reinforced wall (1.25:1 aspect ratio) reached a maximum load of 6971 pounds of lateral force before failure, or 1267 pounds per lineal foot of wall. During the test, the metal plates attached to the testing frame began to bear on the wood top plate. The metal plates can be seen in *Figure 16*. As a result of the bearing, the wall was no longer being loaded solely through the pin. This may have created a higher peak, since the cob wall was not taking all of the load through the pin connection. Ultimately, it is nearly impossible to determine how the cob would act if the test frame did not come in contact with the wood member, so the values received must be analyzed accordingly.



*Figure 16: Wall 4 (Light Reinforcement) After Testing.*

The failure in Wall 4 occurred along a horizontal plane near the bottom of the wall roughly one hour into the test. The location of the failure was along the same plane as a lift, where construction was completed for the day. Shortly after cracks began to form, a large crack spread through the entire length of the wall 2 feet (2') about the base. The crack divided Wall 4 into two halves, only transferring load to the top of the wall. The base and bottom two feet (2') of cob only received a portion of the load. The portion of the wall above the crack was moving independently of the bottom for the remainder of the test. As a result, the hysteresis curve shows a drop of in load as the displacements grows. Had the test been concluded immediately after the failure of the wall, the maximum displacement of the system would be substantially less. As shown in the hysteresis loops in *Figure 17*, Wall 4 had a maximum lateral offset of 4.5 inches.

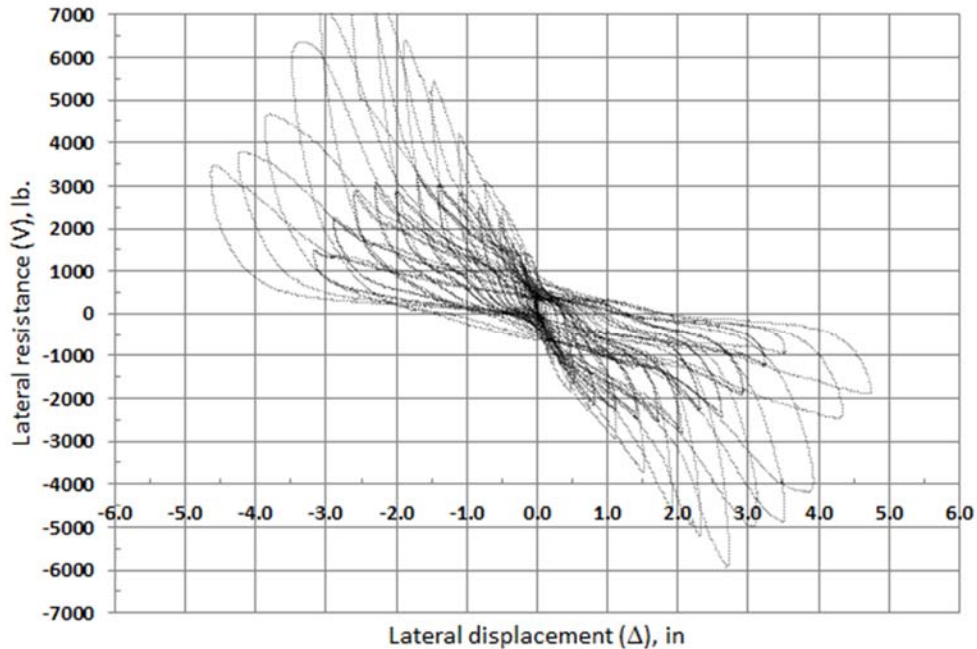


Figure 17: Wall 4 (Light Reinforcement) Hysteresis Diagram.

#### 4.5.0 Discussion

Looking at an overview of all tested wall systems, Wall 3 produced the highest displacement and load per linear foot. The high performance was likely due to the welded wire mesh and its interaction with the vertical rebar. Compared to Wall 4, which only had welded wire mesh, Wall 3's vertical rebar only added 57 pounds per linear foot. Wall 3 however, had 3.32 inches more maximum lateral offset, a 68% increase over Wall 4.

Wall 3 had the highest performance, although there are other factors to consider. First, this wall only had two couplers installed, while Wall 2 had four total. This fact may have adversely affected Wall 2, skewing the results. Although Wall 2 had a reduction of 437 pounds per linear foot compared to Wall 3, the rebar did add a significant amount of ductility to the cob, with the second highest lateral displacement of the walls. The added ductility in Wall 2 was the desired effect of the added reinforcement, although the reduction of the peak lateral force adds a significant trade-off.

Based on the hysteresis loops (Figures 11, 13, 15, 17), Wall 1 was the most brittle with a maximum lateral displacement of 2.5 inches. Compared to Wall 3's maximum lateral

displacement of 8.19 inches, Wall 1 had a reduction of 228 %. The brittleness of Wall 1 indicates the steel reinforcement added a significant amount of ductility to the walls where it was implemented. Wall 1's peak lateral load per linear foot was 141 pounds less than Wall 3's. This suggests the steel reinforcement did not add a significant amount of strength to each wall.

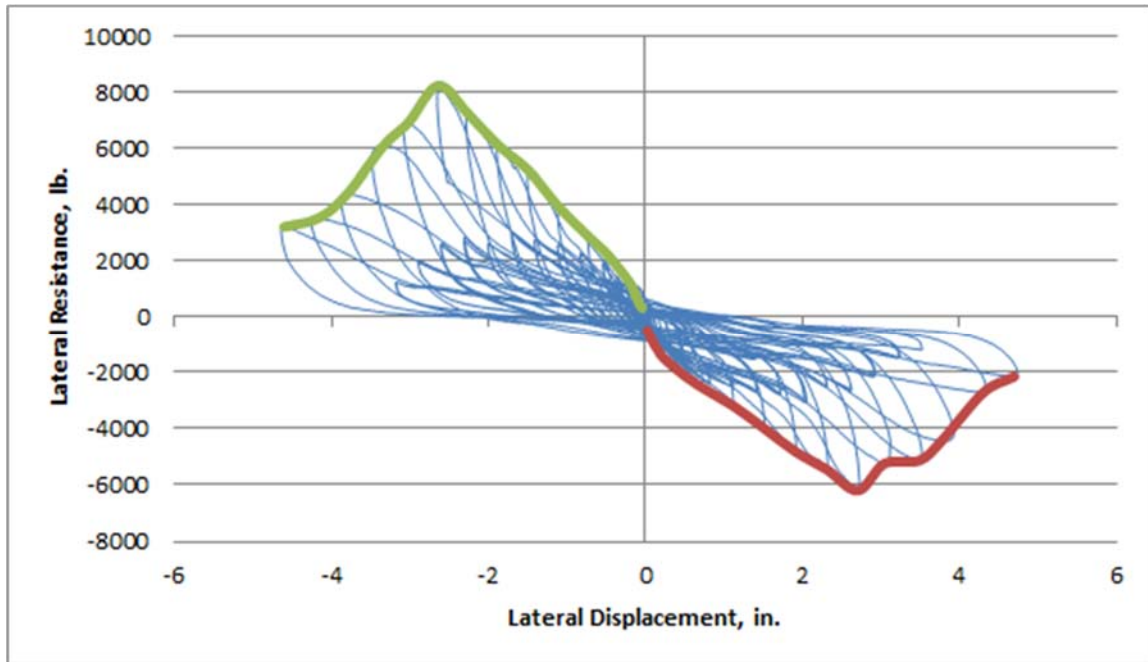
A common failure mechanism observed across multiple walls was horizontal failure along a lift. The lift locations throughout the walls do not have the same level of integration of straw that the general cob matrix would have. Time must be allotted to allow the walls to dry and gain strength before another layer of cob is added. Wet cob that is applied to dryer cob from the day before does not integrate as well, resulting in a reduction in lateral strength along that plane. The locations of the lifts were weak spots for the walls, but different construction techniques could be used to strengthen these planes, such as staggering the lifts at the end of each day.

The couplers were a potential point of weakness in the walls. The couplers were bulky, and the smaller walls developed cracks along the bottom of the couplers. The couplers did not fail during testing, but their effect on the smaller walls is a complication that needs to be looked into further.

The MDOF test frame induced uplift in each of the walls due to rocking shear. In an actual cob structure this reaction would be mitigated by the added weight and stiffness of the entire structure. Each wall's lateral resistance was likely conservatively measured because of the test frames method of loading. In the field an earthquake load would be evenly distributed through the wall's base, rather than through four to six points of contact through the top plate.

#### **4.6.0 Seismic Response Modification Factor**

Using the hysteresis graphs found from the testing, approximations of the seismic response modification factors could be found. Before analyzing the data, bare frame corrections had to be accounted for to remove error in the data, created by friction in the testing frame. After completing these adjustments, envelope curves for each wall's graph could be determined. The maximum peak values and the envelope curves for Wall 4 can be seen below in *Figure 18*.



*Figure 18: Wall 4 (Light Reinforced) Maximum Peak Values.*

By taking the absolute value of each of the data points graphed, two stress strain curves for the wall were created, shown in red and green on *Figure 18*, which were averaged to find a final load vs. displacement plot for each wall. Each wall's stress strain curve was then plotted together as shown on *Figure 19*.

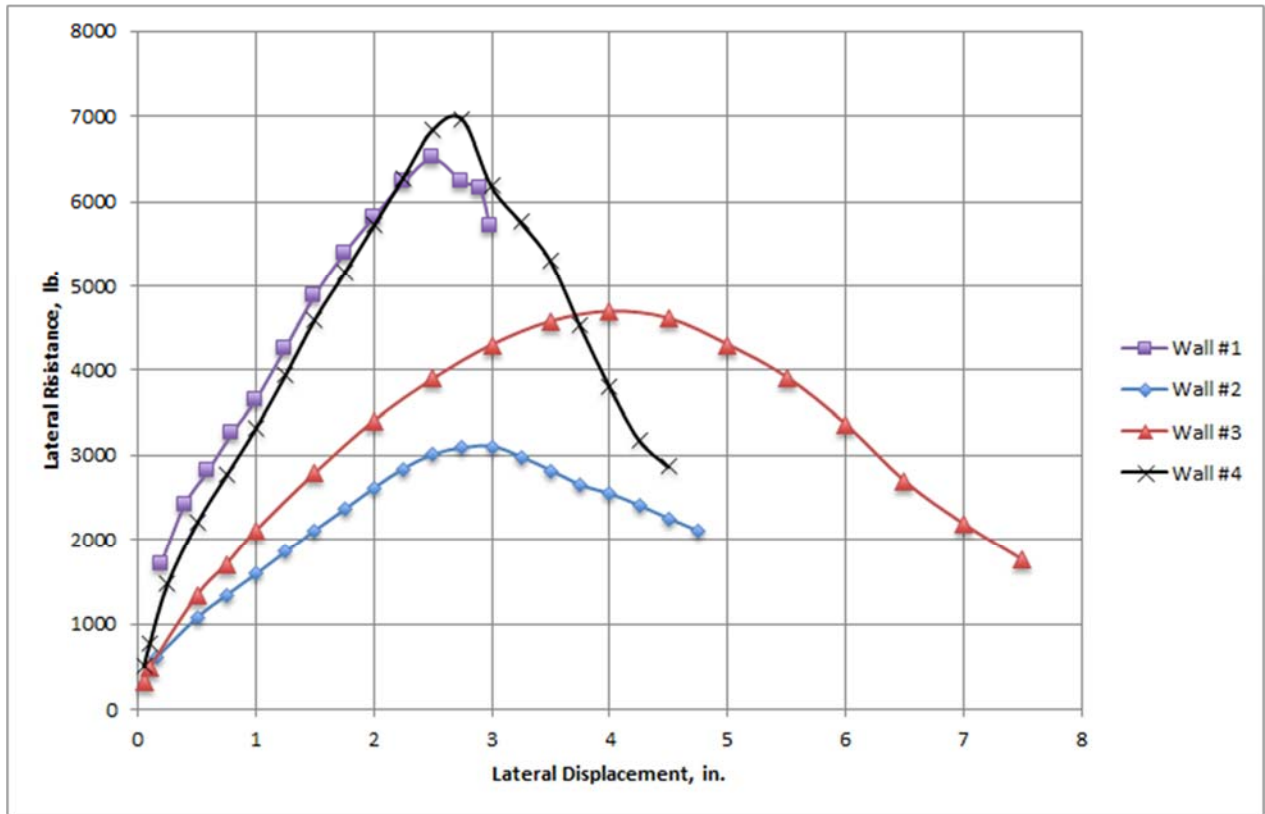


Figure 19: Stress Strain Curves for All Walls.

It is important to note that the data for Wall 1 was recovered using the video data and may contain more error than the others.

The above figure compares how each wall performed, as well as provides a means to estimate the Seismic Response Modification Factor, or R value. The method used to calculate the R value for each wall was the APA Report 158 Method (Rose 1998), using

$$R = (\sqrt{2\mu - 1}) \frac{0.8 \times V_{st}}{V_y} \text{ where } \mu = \frac{\Delta_{st}}{\Delta_y}, \quad (\text{Equation 3})$$

where  $V_{st}$  is the force at the peak load and  $\Delta_{st}$  is the maximum displacement ( $V_{st}$  values can be seen in Figure 20).  $V_y$  and  $\Delta_y$  are the force and displacement at the yield point, respectively. This method was used for each wall. The main difficulty in calculating these values came from determining where the yield point is on the graphs. The yield point exists where the graph becomes non-linear. These points were found using both visual inspection, and by using linear

regressions lines to determine  $R^2$  values for each wall's graph. Each line was considered linear until the  $R^2$  value dropped below 0.98. Any portions beyond the 0.98 value were considered to no longer be yielding. These values were averaged with a visual inspection of where the non-linear portion begins, to find the final values.

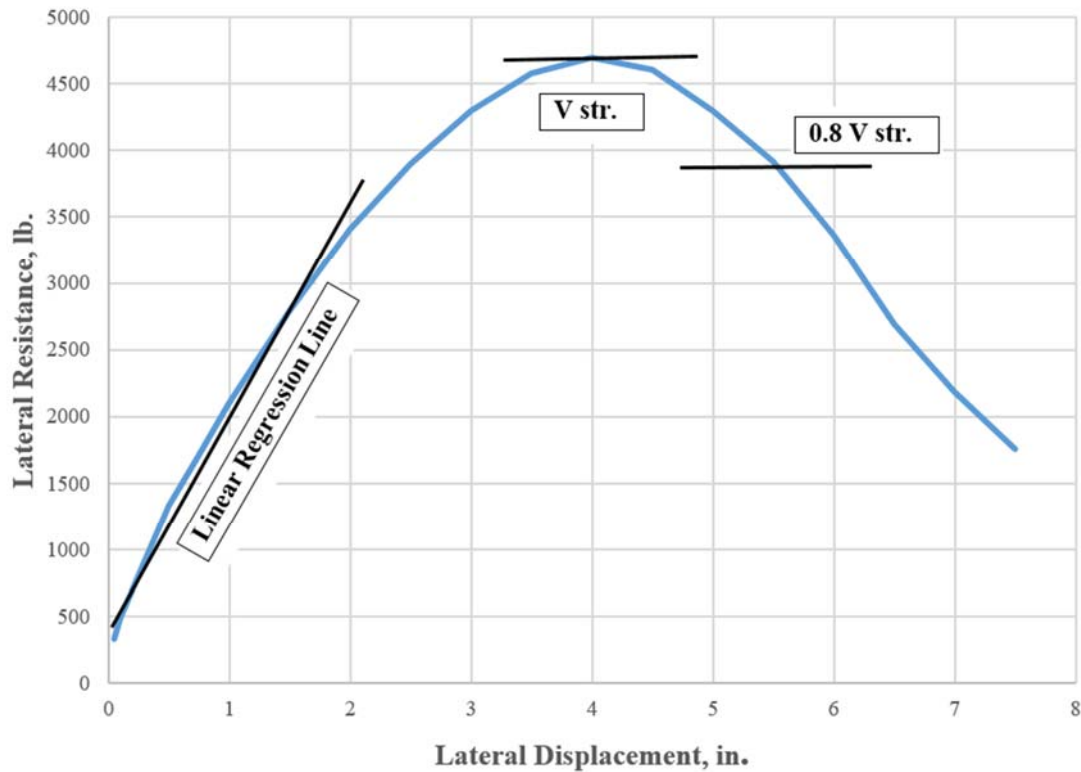


Figure 20: Wall 3 Linear Regression Analysis.

This process was used for all walls, and the results were recorded as shown in Table 5.

Table 5: Wall Test Results.

Wall	Reinforcement	Seismic Response Modification Factor
1	None	N/A
2	Heavy	2
3	Medium	2.5
4	Light	1.5

The test data shows that Wall 3 had the highest R value of 2.5. This is expected, as Wall 3 had the highest lateral displacement and load per linear foot. Due to the data loss for Wall 1, only a fraction of the data points were collected from the hysteresis diagrams. The R value was not

calculated because of the higher uncertainty. Wall 2 and Wall 4 received an R value of 2 and 1.5 respectively.

### **5.0.0 The Future of Cob**

Currently, cob structures are not a permitted building material in the United States, but groups such as the Cob Research Institute are making strides to get the material into the California Residential Code. More research and iterative testing has to be done before cob will be able to make it into the Residential Code but the results from these tests have helped lay a solid foundation for future testing. The political aspects that this material will have to overcome revolve around the inconsistency of the material. Since all soil sources are slightly different, there is always going to be a concern about the consistency of the soil used in cob construction. Verifying the clay content of different soil sources requires testing that some people will not be able to complete in rural areas. The main point of using cob as a building material is that it is inexpensive to use, and the materials needed to construct the structure are accessible in most places on the globe. It is important that more research is done with different types of soil to see how the different sources affect the properties of the cob wall. The main concerns of the material revolve around the safety of cob structures. If people construct cob incorrectly or are inconsistent with their mixes, the structure could be compromised and the occupants of the structure could be in danger. Iterative testing would help minimize these risks because it would show how the material properties react under different circumstances. It is also extremely important that very detailed guidelines for constructing with cob are written and distributed to anyone who wants to build a cob structure. These guidelines will help make sure that people construct the cob structure properly. The other aspect that needs to be looked into is how the structural integrity of the walls is affected by different environments. Even though a solid foundation and roof structure will protect the cob from most rainfall, some more humid environments may prevent drying which would in turn weaken the cob.

Cob research draws many people in because there is a huge push for sustainable building practices in today's society. Construction is an environmentally unsustainable industry due to the large carbon footprint from the production of steel and concrete. Even though the use of steel and concrete is unavoidable with the building materials that we currently have access to, it is

still essential that engineers research materials such as cob, which can provide a sustainable alternative for certain types of structures. Cob could become a great building material for small residential structures, and the production of these structures is much less harmful to the environment than traditional building materials. Cob could also have a huge social impact on certain societies because the structures can be produced for a fraction of the price of current homes. A fully operational three bedroom cob house can be construction for roughly \$26,000. The cob walls act as a great insulating material and can be finished with plaster and painted over to conceal the earthen building material. Since cob is not currently in the California Building Code, people are currently building unpermitted structures or are forced to undergo a very expensive and lengthy design process to build a cob structure. If cob is approved for the Residential or Building Code someday, more people will be able to legally construct cob houses without spending extra money to prove that their house design is safe.

## **6.0.0 Conclusion**

Cob is a great alternative to current materials being used in residential communities. The lack of testing done on this material, however, is currently preventing cob from becoming a part of the California Residential Code. The testing done for this project helped lay the foundation for getting cob into the Residential Code but further testing is needed to see the true values of the material. The results from this research show some of the ways to improve the properties of cob, such as internal reinforcement and using specific sand to soil ratios, but iterative testing needs to still be completed to verify the results discussed in this report. Being able to maximize the peak load and displacement allows designers to use a higher “R” value when designing structures for earthquake loads. The resulting “R” value given from the data of these wall tests was roughly 2.5. This “R” value was higher than originally anticipated but with further testing and innovation, the “R” value could increase. The use of horizontal and vertical reinforcement inside of the cob walls helped increase the ductility of the structure, but further testing would help narrow down the most efficient way of reinforcing cob walls. There is a chance that cob will be able to make it into the California Residential Building Code, but iterative testing will be needed to verify the data provided in the report.

## 7.0.0 References

ACI 318. (2014). “Building Code Requirements for Structural Concrete and Commentary.”  
American Concrete Institute.

ASTM C39 / C39M. - 15. (2015). “Standard Test Method for Compressive Strength of  
Cylindrical Concrete Specimens.” American Society for Testing and Materials.

Morris, Hugh and Miles, Allen. (1998). “NZS 4297:1998 Engineering Design of Earth  
Buildings.” Standards New Zealand, section 7.3.1.1.

Rizza, Michael S. and Bottger, Hana M. (2013) “Effect of Straw Length and Quantity on  
Mechanical Properties of Cob.” University of San Francisco Press.

Rose, John D. (1998). “Report 158: Preliminary Testing of Wood Structural Panel Shear Walls  
Under Cyclic (Reversed) Loading.” APA: The Engineered Wood Association.

NDS. (2012). “ASD/LRDF NDS: National Design Specification for Wood Construction.”  
American Wood Council.

# Appendix

Structural Calculations:

Bearing Stress

3

\* All diagrams Not To Scale

$V = 18,900 \text{ lbs}$

84"

87"

11"

$V = 18,900 \text{ lbs}$

84"

87"

\* Ignoring self weight to determine worst case scenario

6 1/2" 7 1/2" 44.12" 7 1/2" 6 1/2"

$$+\sum M_y = 0 = 18900(84) - x(87)$$

$$87x = 1587600$$

$$x = 18248.28 \text{ lbs}$$

$$x_{\text{required}} = 4562.07 \text{ lbs}$$

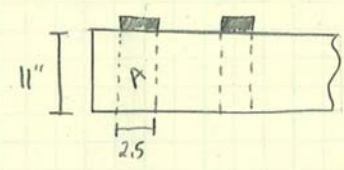
\*  $x_{\text{required}} = \frac{x}{4}$

$C = 625 \text{ psi}$

→ table 4D NDS Supplement  
- most conservative value for  
Douglas fir Wood type

$$C = \frac{V}{A}$$
$$625 = \frac{V}{2.5(4)(11)}$$

$V_{max} = 68,750 \text{ lbs}$



$A = 11(2.5)$

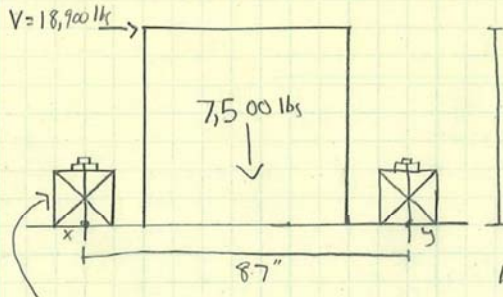
\*  $A(4)$  for  
equation because  
there are four  
sides in  
perimeter

$68,750 \text{ lbs} > 74,562 \text{ lbs}$

$V_{max} > V_{required}$

PASS

# Max Moment on Support Beams

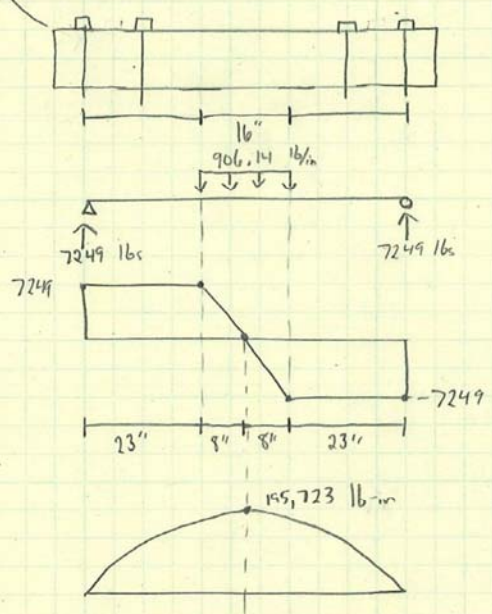


$$\sum M_y = 0 = -7,500\left(\frac{87}{2}\right) + 18,900(87) - 87x$$

$$87 \sum M_y: 1,587,600 - 326,250 = 87x$$

$$x = 14,498 \text{ lbs}$$

Distributed load:  $\frac{14,498}{16} = 906.14 \text{ lb/in}$



$$\text{Max Moment} = 7249(23) + 7249(.5)(8)$$

$$\text{Max Moment} = 195,723 \text{ lb-in}$$

\*largest Moment will be in the center of the beam

## Allowable Moment

$$M_{\text{allowable}} = F_b(S) \rightarrow S = \frac{bd^2}{6}$$

$$S = \frac{5.5(11)^2}{6}$$

$$S = 110.91$$

$kF_b = 1900 \text{ psi}$  (Located in NDS Table 4D)  
 - Dense Select Structural Douglas Fir Beam Confirmed with Lab Manager

$$M_{\text{allowable}} = 1900(110.91)$$

$$M_{\text{allowable}} = 210,729 \text{ lb-in}$$

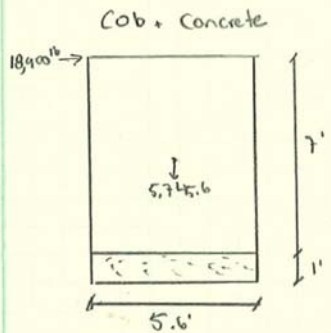
Passes!

$$210,729 > 195,723$$

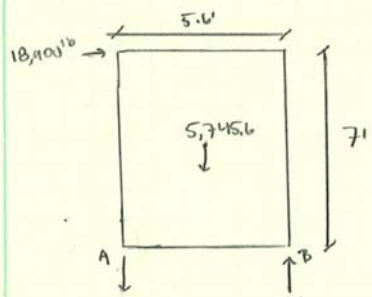
$$M_{\text{allowable}} > \text{Max Moment}$$

\*This calculation does not take into account additional lateral reinforcement supplied by metal plates bolted on the outside of the plate. This aspect would increase allowable.

OVER TURNING CALCULATIONS (7x5.6 Wall)



FORCES IN COB



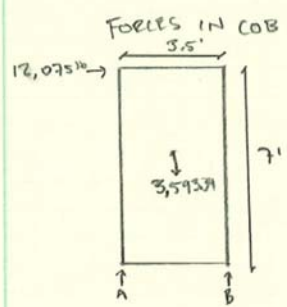
$$\sum M_o B = 18,900(7') - F_A(5.6) - 5,745.6(2.8) = 0$$

$$132,300 = F_A(5.6) + 16,090.40$$

$$5.6F_A = 116,209.52$$

$$F_A = 20,751.71 \text{ lb}$$

OVER TURNING CALLS (7x3.5 WALL)



$$1150 \text{ pcf} \cdot 3.5' = 4025 \times 3^{\text{S.F.}} = 12,075 \text{ lb}$$

$$\sum M_o B = 12,075(7') - F_A(3.5') - (3,593.75 \cdot 1.75') = 0$$

$$84,525 = F_A(3.5') + (6,287.75)$$

$$3.5F_A = 78,237.25$$

$$F_A = 22,353.51 \text{ lb}$$

Concrete breakout of anchor in tension.

\* BUILDING CODE REQUIREMENT FOR STRUCTURAL CONCRETE (ACI 318-05)

$$* N_{cb} = \frac{A_{nc}}{A_{nc0}} \psi_{ed,n} \cdot \psi_{c,n} \cdot \psi_{cp,n} \cdot N_b$$

Assume embedment of 12"

$$* A_{nc} = (c_{a1} + h_{ef}) \cdot (2 \cdot c_{a2})$$

where  $c_{a1}$  and  $c_{a2}$  = distance to edge of concrete and  $h_{ef}$  = embedment

$$= (9.6 + 18") \cdot (2 \cdot 8") = 441.6$$

$$A_{nc0} = 9 h_{ef}^2 = 1296$$

$$* \psi_{ed,n} = .7 + 3 \left( \frac{8"}{18} \right) = .833$$

\*  $\psi_{c,n} = 1.4$  for post installed anchors

$$* \psi_{cp,n} = \frac{1.5 h_{ef}}{c_{ac}} = \frac{18}{20} = .9$$

$$* N_b = k_c \sqrt{f_c'} h_{ef}^{1.5}$$

$k_c = 17$  for post installed anchors

$$f_c' = 4000 \text{ psi}$$

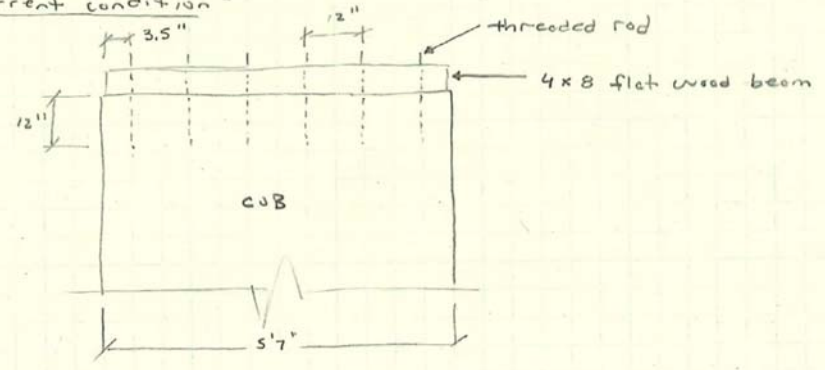
$$N_b = 44,694.16$$

$$N_{cb} = \frac{441.6}{1296} \cdot .833 \cdot 1.4 \cdot .9 \cdot (44,694)$$

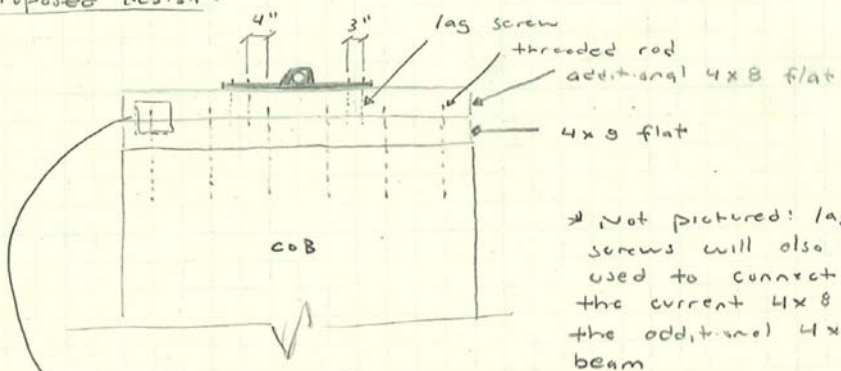
$$N_{cb} = 15,984.1$$

### Top Plate Design

current condition -

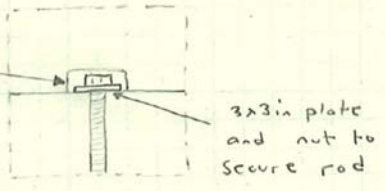


proposed design -



Not pictured: lag screws will also be used to connect the current 4x8 to the additional 4x8 beam

natched wood to accommodate plate and nut

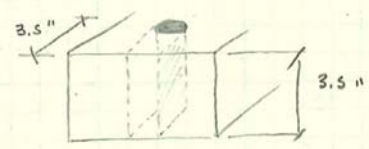


check threaded rod shear on wood -

$V_{demand} = 18,900 \text{ lbs}$  - from CR1 as design shear value

$$\tau = \frac{V}{A}$$

A = area of shear through wood at worst case scenario



$$A = (3.5 \text{ in})(3.5 \text{ in})(2 \text{ shear surfaces})$$

$$A = 24.5 \text{ in}^2$$

$\tau = 170 \text{ psi}$  - from NDS 2015 for Douglas Fir Larch (North) table 4D

$$V = (170 \text{ psi})(24.5 \text{ in}^2)$$

$$V = 4165 \text{ lbs / rod}$$

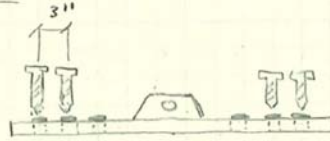
$$V_{demand} = \frac{18,900 \text{ lbs}}{6 \text{ rods}}$$

$$V_{demand} = 3150 \text{ lbs / rod}$$

$$\boxed{4165 \text{ lbs} > 3150 \text{ lbs}} \quad \checkmark$$

check lag screws shear on wood -

- 4 5" lag screws



$$A = (5 \text{ in})(3 \text{ in}) / 2 \text{ surfaces}$$

$$A = 30 \text{ in}^2$$

$$\tau = 170 \text{ psi}$$

$$V = (170 \text{ psi})(30 \text{ in}^2)$$

$$V = 5100 \text{ lbs/screw}$$

$$V_{\text{demand}} = \frac{18900 \text{ lbs}}{4 \text{ screw}}$$

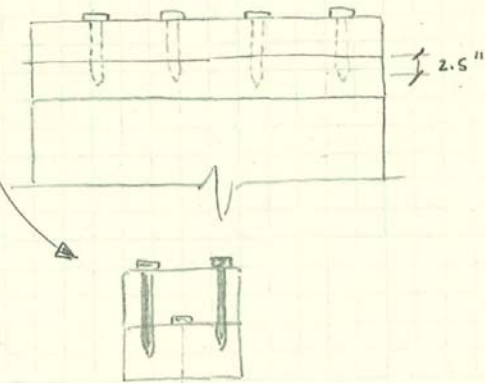
$$V_{\text{demand}} = 4725 \text{ lbs/screw}$$

$$5100 \text{ lbs} > 4725 \text{ lbs} \quad \checkmark$$

- Top connection steel configuration has 6 bolt holes. Due to decrease we would receive in shear capacity, check to see if only 4 screws are necessary in the outermost bolt holes.

check to determine how many lag screws are needed -

- to hold 4x8 to additional 4x8
- Assume using 6" lag screw, so it will penetrate the bottom 4x8 appx 2.5" (use 2" conservatively)
- use 4 lag screws per side, for a total of 8 lag screws at ~12" on center



$$\tau = \frac{V}{A}$$

$$A = (2 \text{ in})(12 \text{ in})(2 \text{ surfaces})$$

$$A = 48 \text{ in}^2$$

$$\tau = 170 \text{ psi}$$

$$V = (170 \text{ psi})(48 \text{ in}^2)$$

$$V = 8160 \text{ lbs / screw}$$

$$V_{\text{demand}} = \frac{18900 \text{ lbs}}{8 \text{ screws}}$$

$$V_{\text{demand}} = 2362.5 \text{ lbs}$$

$$\boxed{8160 \text{ lbs} > 2363 \text{ lbs}} \quad \checkmark$$

Determine capacity of bolts

$$F_{\text{bolt}} = \frac{18900}{4}$$

$$F_{\text{bolt}} = 4725 \text{ lbs/bolt}$$

$$V_{\text{min}} = 19832 \text{ lbs/bolt} \quad \leftarrow \text{From Fastenal online load calculator for a 1 in diameter bolt}$$

$$\boxed{4725 \text{ lbs} < 19832 \text{ lbs}} \quad \checkmark$$

check crushing of wood

crushing strength of Douglas Fir North based on wood-database.com

$$= 6950 \text{ lb/in}^2$$

$$A = (5 \text{ in})(1 \text{ in})$$

$$\rightarrow F = (6950)(5) = \boxed{34750 \text{ lbs}} \quad \checkmark$$

5 inch penetration with a 1 inch diameter bolt

Confirm strength of bolt connections

bearing capacity:

\*  $F_c = 5500 \text{ psi}$  - For Douglas Fir (orch. N) parallel to grain with a diameter of 2 inch

$$F_b = 5500 (5) (1)$$

$$F_b = 27500 \text{ lbs/bolt}$$

shear capacity:  
wood-wood connection -

$Z_{||} = 2210 \text{ lbs}$  - From table 12A in NDS 2015 for  $3\frac{1}{2}$  &  $3\frac{1}{2}$  thick member

$$V_{des} = 18900 \text{ lbs} \rightarrow \frac{18900}{2210}$$

= 8.5 bolts necessary

use 5 lag screws per side

wood-steel connection -

$Z_{||} = 2840 \text{ lbs}$  - From table 12B in NDS 2015 for  $5\frac{1}{4}$  &  $\frac{1}{4}$  thick members.

$$V_{des} = 18900 \rightarrow \frac{18900}{2840}$$

use 6.6 lag screws X

\* Design capacity is too high

Applicable Resources:

Attachment #1

**DESIGN OF REINFORCED CONCRETE COMPONENTS (ACI 318-11)**

U.S. Customary units

**Definitions**

- $a$  = depth of equivalent rectangular stress block, in.
- $A_g$  = gross area of column, in<sup>2</sup>
- $A_s$  = area of tension reinforcement, in<sup>2</sup>
- $A_{st}$  = total area of longitudinal reinforcement, in<sup>2</sup>
- $A_v$  = area of shear reinforcement within a distance  $s$ , in.
- $b$  = width of compression face of member, in.
- $\beta_1$  = ratio of depth of rectangular stress block,  $a$ , to depth to neutral axis,  $c$   

$$= 0.85 \geq 0.85 - 0.05 \left( \frac{f'_c - 4,000}{1,000} \right) \geq 0.65$$
- $c$  = distance from extreme compression fiber to neutral axis, in.
- $d$  = distance from extreme compression fiber to centroid of nonprestressed tension reinforcement, in.
- $d_t$  = distance from extreme compression fiber to extreme tension steel, in.
- $E_c$  = modulus of elasticity =  $33w_c^{1.5} \sqrt{f'_c}$ , psi
- $\epsilon_t$  = net tensile strain in extreme tension steel at nominal strength
- $f'_c$  = compressive strength of concrete, psi
- $f_y$  = yield strength of steel reinforcement, psi
- $M_n$  = nominal moment strength at section, in.-lb
- $\phi M_n$  = design moment strength at section, in.-lb
- $M_u$  = factored moment at section, in.-lb
- $P_n$  = nominal axial load strength at given eccentricity, lb
- $\phi P_n$  = design axial load strength at given eccentricity, lb
- $P_u$  = factored axial force at section, lb
- $\rho_g$  = ratio of total reinforcement area to cross-sectional area of column =  $A_{st}/A_g$
- $s$  = spacing of shear ties measured along longitudinal axis of member, in.
- $V_c$  = nominal shear strength provided by concrete, lb
- $V_n$  = nominal shear strength at section, lb
- $\phi V_n$  = design shear strength at section, lb
- $V_s$  = nominal shear strength provided by reinforcement, lb
- $V_u$  = factored shear force at section, lb

**ASTM STANDARD REINFORCING BARS**

BAR SIZE	DIAMETER, IN.	AREA, IN <sup>2</sup>	WEIGHT, LB/FT
#3	0.375	0.11	0.376
#4	0.500	0.20	0.668
#5	0.625	0.31	1.043
#6	0.750	0.44	1.502
#7	0.875	0.60	2.044
#8	1.000	0.79	2.670
#9	1.128	1.00	3.400
#10	1.270	1.27	4.303
#11	1.410	1.56	5.313
#14	1.693	2.25	7.650
#18	2.257	4.00	13.60

**BEAMS—SHEAR**

$$\phi V_n \geq V_u$$

Nominal shear strength:

$$V_n = V_c + V_s$$

$$V_c = 2 b_w d \sqrt{f'_c}$$

$$V_s = \frac{A_v f_y d}{s} \text{ (may not exceed } 8 b_w d \sqrt{f'_c} \text{)}$$

Required and maximum-permitted stirrup spacing,  $s$

$$V_u \leq \frac{\phi V_c}{2}: \text{ No stirrups required}$$

$$V_u > \frac{\phi V_c}{2}: \text{ Use the following table (} A_v \text{ given)}$$

	$\frac{\phi V_c}{2} < V_u \leq \phi V_c$	$V_u > \phi V_c$
Required spacing	Smaller of: $s = \frac{A_v f_y}{50 b_w}$ $s = \frac{A_v f_y}{0.75 b_w \sqrt{f'_c}}$	$V_s = \frac{V_u - V_c}{\phi}$ $s = \frac{A_v f_y d}{V_s}$
Maximum permitted spacing	Smaller of: $s = \frac{d}{2}$ OR $s = 24"$	$V_s \leq 4 b_w d \sqrt{f'_c}$ Smaller of: $s = \frac{d}{2}$ OR $s = 24"$ <hr/> $V_s > 4 b_w d \sqrt{f'_c}$ Smaller of: $s = \frac{d}{4}$ $s = 12"$

**SHORT COLUMNS**

Limits for Main Reinforcements

$$\rho_g = \frac{A_{st}}{A_g}$$

$$0.01 \leq \rho_g \leq 0.08$$

Design Column Strength, Tied Columns

$$\phi = 0.65$$

$$\phi P_n = 0.80 \phi [0.85 f'_c (A_g - A_{st}) + A_{st} f_y]$$



Fig 10. Cylinder tested in splitting tension to determine tensile capacity



Fig 11. "snake-like" cobbing method around wattle frame



Fig 12. Ductile nature of long-straw cylinders

4.1: Results of Compression Test

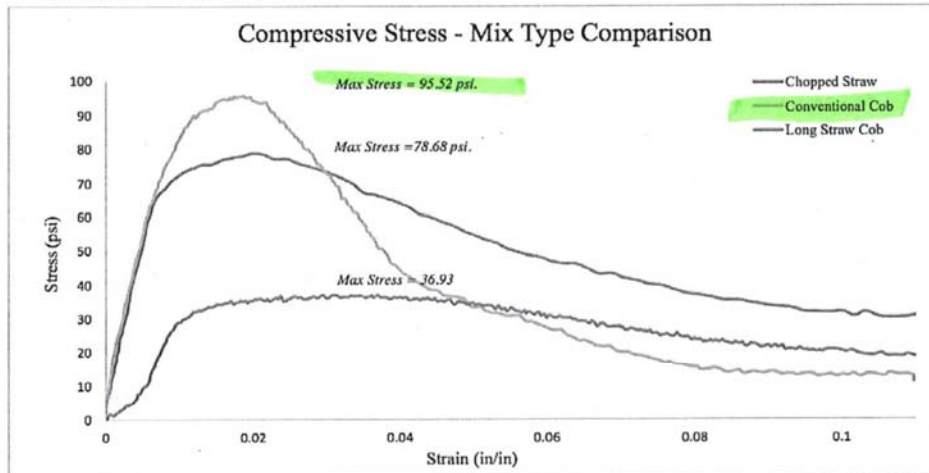


Fig 13 - Compressive Stress by Mix Type

Young's Modulus:

Conventional (P7): 10371 psi/(in/in)

Chopped Straw (P1): 9414 psi/(in/in)

Long Straw (P11): 5405 psi/(in/in)

Young's Modulus (E) calculated using the following equation:

$$E = (F/A)/(\Delta L/L_0)$$

where F = maximum load at failure (lbs)

A = cross sectional area (in<sup>2</sup>)

L<sub>0</sub> = original specimen height (in)

ΔL = change in specimen height at failure (in)

**Table 4D Reference Design Values for Visually Graded Timbers (5" x 5" and larger)<sup>1,2</sup>**  
**(Cont.)** (Tabulated design values are for normal load duration and dry service conditions, unless specified otherwise. See NDS 4.3 for a comprehensive description of design value adjustment factors.)

**USE WITH TABLE 4D ADJUSTMENT FACTORS**

Species and commercial Grade	Size classification	Design values in pounds per square inch (psi)						Modulus of Elasticity		Specific Gravity <sup>3</sup> G	Grading Rules Agency
		Bending	Tension parallel to grain	Shear parallel to grain	Compression perpendicular to grain	Compression parallel to grain	E	E <sub>min</sub>			
		F <sub>b</sub>	F <sub>t</sub>	F <sub>v</sub>	F <sub>c⊥</sub>	F <sub>c</sub>					
<b>DOUGLAS FIR-LARCH (NORTH)</b>											
Select Structural	Beams and Stringers	1,600	950	170	625	1,100	1,600,000	580,000	0.49	NLGA	
No.1		1,300	675	170	625	925	1,600,000	580,000			
No.2	875	425	170	625	600	1,300,000	470,000				
Select Structural	Posts and Timbers	1,500	1,000	170	625	1,150	1,600,000	580,000			
No.1		1,200	825	170	625	1,000	1,600,000	580,000			
No.2	725	475	170	625	700	1,300,000	470,000				
<b>DOUGLAS FIR-SOUTH</b>											
Select Structural	Beams and Stringers	1,550	900	165	520	1,000	1,200,000	440,000	0.46	WWPA	
No.1		1,300	625	165	520	850	1,200,000	440,000			
No.2	825	425	165	520	550	1,000,000	370,000				
Select Structural	Posts and Timbers	1,450	950	165	520	1,050	1,200,000	440,000			
No.1		1,150	775	165	520	925	1,200,000	440,000			
No.2	675	450	165	520	650	1,000,000	370,000				
<b>EASTERN HEMLOCK</b>											
Select Structural	Beams and Stringers	1,350	925	155	550	950	1,200,000	440,000	0.41	NELMA NSLB	
No.1		1,150	775	155	550	800	1,200,000	440,000			
No.2	750	375	155	550	550	900,000	330,000				
Select Structural	Posts and Timbers	1,250	850	155	550	1,000	1,200,000	440,000			
No.1		1,050	700	155	500	875	1,200,000	440,000			
No.2	600	400	155	550	400	900,000	330,000				
<b>EASTERN HEMLOCK-TAMARACK</b>											
Select Structural	Beams and Stringers	1,400	925	155	555	950	1,200,000	440,000	0.41	NELMA NSLB	
No.1		1,150	775	155	555	800	1,200,000	440,000			
No.2	750	375	155	555	500	900,000	330,000				
Select Structural	Posts and Timbers	1,300	875	155	555	1,000	1,200,000	440,000			
No.1		1,050	700	155	555	875	1,200,000	440,000			
No.2	600	400	155	555	400	900,000	330,000				
<b>EASTERN HEMLOCK-TAMARACK (N)</b>											
Select Structural	Beams and Stringers	1,450	850	165	555	950	1,300,000	470,000	0.47	NLGA	
No.1		1,200	600	165	555	800	1,300,000	470,000			
No.2	775	400	165	555	500	1,100,000	400,000				
Select Structural	Posts and Timbers	1,350	900	165	555	1,000	1,300,000	470,000			
No.1		1,100	725	165	555	875	1,300,000	470,000			
No.2	650	425	165	555	600	1,100,000	400,000				
<b>EASTERN SPRUCE</b>											
Select Structural	Beams and Stringers	1,050	725	135	390	750	1,400,000	510,000	0.41	NELMA NSLB	
No.1		900	600	135	390	625	1,400,000	510,000			
No.2	575	275	135	390	375	1,000,000	370,000				
Select Structural	Posts and Timbers	1,000	675	135	390	775	1,400,000	510,000			
No.1		800	550	135	390	675	1,400,000	510,000			
No.2	450	300	135	390	300	1,000,000	370,000				
<b>EASTERN WHITE PINE</b>											
Select Structural	Beams and Stringers	1,050	700	125	350	675	1,100,000	400,000	0.36	NELMA NSLB	
No.1		875	600	125	350	575	1,100,000	400,000			
No.2	575	275	125	350	400	900,000	330,000				
Select Structural	Posts and Timbers	975	650	125	350	725	1,100,000	400,000			
No.1		800	525	125	350	625	1,100,000	400,000			
No.2	450	300	125	350	325	900,000	330,000				
<b>HEM-FIR</b>											
Select Structural	Beams and Stringers	1,300	750	140	405	925	1,300,000	470,000	0.43	WCLIB WWPA	
No.1		1,050	525	140	405	750	1,300,000	470,000			
No.2	675	350	140	405	500	1,100,000	400,000				
Select Structural	Posts and Timbers	1,200	800	140	405	975	1,300,000	470,000			
No.1		975	650	140	405	850	1,300,000	470,000			
No.2	575	375	140	405	575	1,100,000	400,000				

## NZS 4297:1998

### 6.5.2 Composite columns

Where other material column sections are built into earth walls, or columns, the strength of the earth shall not be included in the calculation of the strength of the column section.

## 6.6 Principles and requirements additional to 6.3 for members designed for seismic loading

### 6.6.1 Strength calculations

Columns of earth shall not be used to provide main structure lateral load restraint unless they contain other material sections which can resist the whole of the lateral load.

## 7 SHEAR

### 7.1 Notation

$A_b$  area of earth cross section

$A_s$  area of tensile reinforcement

$A_{sv}$  area of shear reinforcement

$d$  depth of section in the direction of shear

$f_d$  compressive stress acting on section under the design loading

$f_{es}$  shear strength of earth

$f_n$  total nominal shear stress

$f_{sy}$  lower characteristic yield strength of shear reinforcement

$f_y$  lower characteristic yield strength of reinforcement

$h$  height of member

$k_v$  shear factor as given in 7.3.1.2

$L$  clear length of the wall or pier

$s$  spacing of shear reinforcement measured perpendicular to direction of shear force

$t$  thickness or depth of wall perpendicular to the axis under consideration

$V^*$  design shear force acting on the cross section of a member at ultimate limit state, assessed from NZS 4203

$V_n$  nominal shear strength of the section (kN)

$\delta$  coefficient of variation

$\mu$  structural ductility factor

$\phi$  capacity reduction factor

**7.2 Scope**

**7.2.1**

Provisions of this section apply to design of earth walls for shear and torsion with flexure and with or without axial load. Design of earth walls for shear and torsion shall be in accordance with established principles for reinforced concrete, as set out in NZS 3101, section 9 modified by the requirements of this section. Where no specific requirements are given in this section, the appropriate requirements of NZS 3101 section 9 shall apply.

**7.3 General principles and requirements**

**7.3.1 Unreinforced earth**

**7.3.1.1 Shear capacity**

The design of an unreinforced earth wall subject to shear forces, with or without simultaneous compressive forces acting across the shear plane, shall be such that the following relationship is satisfied under each combination of simultaneously acting design shear force,  $V^*$ , and (minimum) compressive stress ( $f_d$ ) acting at the cross section under consideration:

$$V^* \leq \phi [f_{es} A_b + k_v f_d A_b] \dots\dots\dots (\text{Eq. 7-1})$$

or

$$V^* \leq 5\phi f_{es} A_b \dots\dots\dots (\text{Eq. 7-2})$$

whichever is less.

Limited ductility seismic design principles shall not be applied to unreinforced earth.

**C7.3.1.1**  
*The factor of 5 term in equation 7-2 places an upper limit on the influence of vertical load. Vertical reinforcement pretension may be taken into account in calculating  $f_d$  provided the pretension is maintained after shrinkage has taken place.*

**7.3.1.2 Shear factor ( $k_v$ )**

The value of  $k_v$  for use in 7.3.1.1 shall be:

(a) At membrane-type damp-proof courses, flashings, and similar locations having low friction resistance:

$$k_v = \text{zero} \dots\dots\dots (\text{Eq. 7-3})$$

(b) At mortar bed-joints:

$$k_v = 0.30. \dots\dots\dots (\text{Eq. 7-4})$$

**C7.3.1.2**  
*Testing may be used to establish values for  $k_v$  different to those given by 7.3.1.2. Such testing is outside the scope of this Standard. The value of  $k_v$  from equation 7.4 necessitates reinforcement penetrating damp proof membranes to ensure shear continuity.*

**7.3.1.3 Development of longitudinal shear strength**

Where it is necessary to transfer shear forces across wall intersections and vertical mortar joints in earth brick construction, the bonding or tying across the shear plane shall be an overlapping bond.