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Material effects on strawbale wall seismic capacity

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By

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Abstract

Strawbale construction is a sustainable, viable alternative to conventional building practices. As a newly introduced appendix to the International Residential Code (IRC), the strawbale construction requirements may benefit from further evaluation and possible refinement. Such evaluation and refinement may lead towards code change proposals that will improve the provisions and make strawbale construction safer and more accessible to the general public. This seismic test series addressed the effect of mesh wire type on ductility and the validity of the existing wall slenderness limits. The tests focused on slender walls dominated by flexural deformations. Welded wire mesh wall performed better than the woven wire mesh wall of the same detailing, yet fell short of expected values. Slenderness must continue to be analyzed as the results of a wall using 14" bales were impacted by bale irregularity. The additional tests done as part of this thesis, including vertical load tests and materials testing, added to the understanding of strawbale construction performance and expanded the corpus of strawbale wall test data. All tested walls performed satisfactorily under vertical loading in post-seismic conditions. The purpose of this test series was to validate and potentially suggest improvements to the building code provisions to enhance the prevalence and safety of strawbale construction.

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1.0 Introduction

As a sustainable alternative to conventional building practices, strawbale construction has steadily grown in popularity throughout the United States. Its recent addition to the International Residential Code (IRC) has catapulted strawbale construction into the field as a viable material option for homes internationally. Building primarily on research completed at the University of Illinois and Santa Clara University, as discussed in the Literature Review, the experiments described here aim to make strawbale construction safer and more accessible to populations across the world. This focus on lateral seismic testing adds valuable data to the body of information currently available on strawbale structures.

1.1 Importance and Ethics of Strawbale Research

A lack of knowledge regarding strawbale seismic performance underlies the need for a better understanding of strawbale construction. A significant effort to distill the current understanding of strawbale performance and construction methods led to the approval of the Appendix on Strawbale Construction, which was published in the 2015 International Residential Code (IRC Appendix S). The Appendix provides uniformly applicable provisions for strawbale construction that impact the wellbeing of people using strawbale structures in seismic zones. As knowledge of strawbale construction continues to develop, including thorough experiments on strawbale walls and buildings, proposals to update the Appendix may be necessary. Additionally, more time and data is required to sufficiently understand strawbale wall systems so that they can also be incorporated into the International Building Code (IBC), as well as validating existing provisions and analyzing potential improvements. The test series outlined in this paper intends to further that knowledge by examining the influence of mesh type on the ductility of slender walls (dominated by flexure) and the validity of code limitations on bale thickness and slenderness limits.

The importance of pursuing strawbale construction lies in the material's accessibility and the sustainability of its components. Strawbale construction has been lauded for its environmentally friendly nature. Straw bales are formed from the stalks of agricultural products such as wheat, rice, oat, barley, rye and other cereal grains (IRC, 2015). Generally seen as a waste product or byproduct of agricultural production of cereal grains, straw bales are often used for livestock bedding and fuel. Straw is also left in fields or baled and sent to landfills, where they decompose (Sodagar, et al., 2011). Straw bales used for construction act as carbon sinks, however, instead of releasing CO_2 via flame or decomposition. According to a 2011 study on "the carbon-reduction potential of straw-bale housing," straw bale sequesters 1.35 pounds of CO_2 per pound of bale; if a bale weighs approximately 50 pounds, then it sequesters approximately 68 pounds of CO_2 (Sodagar, et al., 2011). The same study states that timber also boasts a negative carbon footprint, but its sequestration rate was estimated to be 1.2 pounds of CO_2 per pound of timber. These values correspond purely to the material properties, and do not incorporate carbon emissions associated with the transportation of materials. If using locally sourced straw bales, the potential environmental benefit could be even greater per pound of material than the carbon savings of using timber if it comes from several states away. In either case, both materials work together in strawbale structures to contribute a negative carbon footprint.

With regard to the mechanical properties of strawbale walls, most existing data comes from a limited number of tests, with even fewer performed specifically to test strawbale wall seismic capacity. Prior to the consensus of the 2015 IRC, the maximum allowed vertical load varied by jurisdiction: 17.2 kPa [2.5 psi] per Arizona Standards, 19.1 kPa [2.8 psi] per Austin Standards, and 11.7 kPa [1.7 psi] per the California Building Standards Code (CBSC) (Swan, et al., 2011). Additionally, different jurisdictions required different details, plaster mixes, strengths, and other strawbale wall properties, which led to general confusion regarding which regulations applied where. Despite the addition of IRC Appendix S, some confusion still remains. For example, failure modes cannot always be accurately predicted; common modes of failure include buckling, cracks in the plaster, and excessive displacement (Kim, et al., 2012). The lack of scientific data concerning strawbale construction currently limits its use, and potential inaccuracies within the code can endanger those who use it. The intention of this project was to add to the growing body of experimental data, and to fulfill our responsibility as engineers to contribute to public safety.

Strawbale construction is a sustainable alternative to standard building practices, both by using locally sourced materials and materials that act as carbon sinks (Moga, 2015). Additionally straw bales in buildings act as thermal insulation, which reduces electricity demands for heating and cooling during winter and summer months (Sodagar, et al., 2011). A lack of regulatory knowledge, however, currently prevents strawbale construction from gaining more mainstream use and acceptance. The thickness of strawbale walls also acts as a negative against strawbale construction. In practice, to shelter the walls from rain, more roofing and timber is required on strawbale homes to cover the same interior space protected by masonry or timber walls. Further testing will help establish a firmer strawbale database, allow for a more universal set of standards across multiple jurisdictions, and give builders clearer expectations on the seismic performance of strawbale structures. This testing will increase the safety and accessibility of a sustainable construction method, which is key to tackling the climate change issues the world faces.

1.2 Objectives

In 2013 a strawbale code was proposed to the International Building Code (IBC), which provided criteria for engineering strawbale wall systems for gravity, wind, and earthquake loads. These provisions were met with resistance and rejected by the IBC council. The IRC proposal was adopted into the 2015 IRC, and it includes required lengths of walls with different plasters for wind and earthquake resistance. To further refine the information in the IRC, the experiments in this test series look at two main objectives:

1. Evaluate the performance of two wire mesh types: welded and woven wire mesh.

2. Empirically examine the code slenderness limits using small straw bales.

The background for these objectives is described below.

1.2.1 Background on Objective 1

The plaster reinforced with wire mesh that encases the bales in strawbale walls provides flexural and shear strength necessary to withstand powerful seismic loading. The straw bales themselves and the plaster itself provide little, if any, tensile strength. Flexural resistance imposes tensile demands on the wire mesh within the plaster. Currently there are two types of mesh allowed in code: hexagonal woven wire mesh (termed “woven wire mesh,” rather than the much thinner “chicken wire” or “poultry netting” which may be made using 20-gauge wire) and 2” x 2” [50mm x 50mm] welded wire mesh. The woven wire mesh is also used in practice for the application of stucco (a cement-lime plaster) and is made using 17-guage wire, woven to form 1-½” hexagons. Welded wire mesh is made in different gauges; the mesh used in this series and recognized in the IRC is made from 14-gauge wire, in compliance with ASTM A641 (2003).

One concern that arises with using woven wire mesh is the possibility that after the reinforced plaster mesh cracks, there may be a tendency for the hexagons to elongate, representing a potential loss of stiffness. In contrast, the 2”x2” mesh consists of a grid of straight wires that are welded to one another.

The 2015 IRC allows woven wire mesh to be used with cement-lime plaster and lime plaster, while the welded wire mesh is allowed for use with all plasters (Appendix S, Table AS106.13). While test data provides a basis for the mesh-plaster combinations provided in the IRC, results for some mesh-plaster combinations were not available and expectations were deduced by interpolation or augmented by calculation. Thus, one objective is to validate the behavior of a cement-lime plaster reinforced with woven mesh and quantify the impact of using woven wire mesh instead of welded wire mesh.

1.2.2 Background on Objective 2

Slenderness limits proposed in the IRC were based on limited empirical data, so the second objective of this test series is to assess the slenderness limits when walls are built to the limit. The Santa Clara University test frame can fit a maximum wall height of approximately 9 feet, which would be too short to test a wall made using full-scale bales. Thus, reduced-size bales were used to characterize the behavior of the most slender wall allowed. The slenderness limit prescribed in the code for walls made with cement-lime plaster is:

$$H \leq 9T^{0.5} \qquad \text{(eq. 1)}$$

where T is the thickness of the bales and H is the vertical height of the wall, both measured in feet (2015 IRC Appendix S, Table AS105.4). A full-scale wall made of 18” wide bales can be a maximum of 11.0 feet high. To build a wall that fits in the test frame at the same slenderness limit, a 9” thick wall with a nominal height of 8 ft was constructed. Specifically, the limit corresponding to a 9” wide bale is 7.8 feet.

Additionally, the 2015 IRC Appendix S (AS 106.13.1) requires the bales to be at least fifteen inches thick (excluding the plaster). In some parts of the country, balers produce narrower bales that are 14” wide, which would not meet the Appendix S provisions. Thus, related to the slenderness limit question is a question whether 14” thick bales are acceptable.

2.0 Literature Review

This section examines five papers on strawbale wall seismic capacities. A significant lateral testing program studied the behavior of six full-scale walls in 2003. While this test program provided much valuable data, many questions remained. Several test programs have been conducted at Santa Clara University (SCU). Three senior design papers are discussed, as well as one master’s thesis, which extended the work done by the same students when they were undergraduates at Santa Clara University. This formal literature review, though not fully comprehensive on all research done at SCU and elsewhere, serves to contextualize the research presented in this paper.

2.1 University of Illinois

2.1.1 In-Plane Cyclic Tests of Plastered Straw Bale Wall Assemblies

At the University of Illinois, Urbana-Champaign, Cale Ash, Mark Aschheim, and David Mar constructed six full-scale plastered straw bale walls, three with cement stucco skins and three with earth plaster skins (2003). The walls had different levels of detailing in the connections, sill plates, and reinforcement. All walls were 8 feet long and 8 feet tall (1:1 aspect ratio).

The cement stucco mix had a sand:cement:lime:water ratio of 1 : 0.25 : 3.75 : 0.8 by volume. The slaked lime had been hydrated for five days prior to mixing. The seven-day strength of their plaster cubes was 1850 psi on average, and ranged between 2200psi to 2210psi after 36 days of curing. The cubes were moist-cured. The average plaster strength for the earth plaster was inconsistent, as it was 290 psi at 44 days and 160 psi at 94 days. Though the SCU cement-lime plaster mix differs from theirs, their results present a useful measure for comparison.

The results of the three cement stucco walls varied depending on the detailing, though there was little difference between the medium- and heavy-detailed walls, Wall E and Wall F respectively (Table 1). The authors concluded that the additional nails and staples of the “Heavy” detail design did not significantly contribute to wall strength, and the increased labor and materials did not justify the use of a “Heavy” detail over a “Medium” detail.

Table 1: Description of detailing and performance of the three cement stucco straw bale walls in Ash, Aschheim, and Mar’s paper.

Wall Name	Detailing Level	Mesh Type	Maximum Lateral Force (kips)
Wall D	Light	17 gauge woven wire mesh	6.4
Wall E	Medium	14 gauge 2” x 2” welded wire mesh	19.0
Wall F	Heavy	14 gauge 2” x 2” welded wire mesh	17.9

The desired failure mode, a ductile failure in flexure, occurred only in walls with the cement stucco skin, specifically Wall E and Wall F. The failure of Wall D was dominated by rocking as the wall unhinged from its sill plate due to cross-grain bending failure at the base. Flexural cracks appeared in both Wall E and Wall F, which both experienced a similar ductile failure despite differences in detailing. Because all walls were stable at lateral drifts exceeding 5% (approximately 5 inches), the tests in our research were comfortably ended when the drift reached 5-7% (approximately 5-7 inches).

The conclusions of this research included recommendations for continued research on material testing, specifically (1) component testing of the reinforcement and (2) examination of detailing requirements. This conclusion has heavily influenced the direction of research at Santa Clara University, described in the following section.

2.2 Santa Clara University

2.2.1 Straw Bale Seismic Design Capacities 1

In 2014 Christopher M. Heckert and Zach T. Looney completed their senior design project examining connection reinforcement materials and several boundary conditions. They worked with two other senior design groups completing projects in the same year, and the three groups built upon each other. Heckert and Looney's research focused on a series of small-scale tests.

First, they looked at several wire mesh types. The ones relating to the current research are their 14C ("welded wire mesh") and Stucco Netting ("woven wire mesh"). In Table 2 are the average tensile ultimate strengths of their tests.

Table 2: Listed are the average tensile ultimate strengths for the transverse and longitudinal wires of the welded wire mesh (14C) and woven wire mesh (Stucco Netting).

Average Ultimate Strength (lbs)		
	Transverse	Longitudinal
14C	318	320
Stucco Netting	132	N/A

They also tested the mesh intersections in both tension and compression. They found that the Stucco Netting "weave pulled apart" while the weld in the 14C wire mesh resisted 402 lbs in compression and 198 lbs in tension.

Their final series of testing involved testing a 12" x 16" swatch of wire mesh attached to a small wooden cap and base and loaded in such a way to experience both tension and shear. Heckert and Looney's results lead to the design of the "strong" and "workhorse" detailing used in subsequent testing. The "strong" detail included 6"x6" W1.4 welded wire mesh and 2"x2" 16-gauge woven wire mesh along the height of the wall. The connections were secured with a combination of 5/16" x 3" hot-dipped galvanized (HDG) lag screws and 16 gauge 7/16" crown 1-¾" leg electro-galvanized staples. The "workhorse" detail used less material, and was comprised of 14 gauge 2"x4" welded wire mesh across the bales and 16 gauge 7/16" crown 1-¾" leg electro-galvanized staples as the connection to the base. These two details would be used in the other groups' tests.

2.2.2 Straw Bale Seismic Design Capacities 2

Beth Avon and Brittnie Swartchick looked specifically at plaster mix designs as well as the design of full-scale post-and-beam strawbale wall tests (2014). After several iterations of plaster mix designs, they concluded on the plaster mix design used for all subsequent testing done at SCU. The cement:lime:sand ratio (by volume) was 1 : 0.75 : 5.25, and its strength when water- and lime-bath cured is summarized in Table 3. This mix design conforms to ASTM C926 according to Avon and Swartchick, determining the compliant volumes as three parts sand per part cementitious material.

Table 3: Average strength of 2” test cubes in water and lime-saturated baths at two different cure times.

Average Compressive Strength (psi)		
	7-day Cure Time	41-day Cure Time
Water bath	620	N/A
Lime-saturated bath	1030	1140

The water curing resulted in significantly less strength, and there was not an appreciable amount of strength gain between 7 and 41 days of curing in a lime-saturated water bath. Once the plaster mix design proportions had been finalized, resulting in the mix design described in the Materials section (Table 6), two more sets of cubes were tested. One set dried alongside the wall without a moisture bath, and its strength was 1732 psi, which met the 1000 psi minimum (IRC, 2015). The second set was lime-cured and reached a strength of 2746 psi on average. These tests were completed at the time of wall testing, at longer than 28 days.

Avon and Swartchick completed a full design, analysis, and construction of two 8’ tall by 8’ wide post-and-beam strawbale walls - one with the “strong” details and the other with the “workhorse” details. This work is less relevant to the present research because their walls utilized the post and beam framing system, and their wire mesh reinforcement differed from those detailed in the 2015 IRC and the modified “workhorse” connection detail that this test series used.

2.2.3 Straw Bale Seismic Design Capacities III

The third 2014 SCU senior design group, comprised of Molly Summers and Michael Zaleski, focused on the complete design and construction of two walls: a 4’ long wall with “strong” detailing and an 11’ long wall with openings and “strong” detailing. Both specimens were 8’ tall and designed to examine the behavior of slender walls (such as between a door and window, i.e. the 4’ wall) and wall panels having different aspect ratios (such as can occur due to window openings, i.e. the 11’ wall). The window openings on the longer wall were reinforced with No. 3 rebar along their perimeters. These walls were not tested as part of their senior design project.

The walls were tested in the 3DOF (degrees of freedom) Structural Test Frame in the SCU structural laboratory as part of an MS thesis by Heckert, Summers, and Swartchick (2015).

2.2.4 Straw Bale Seismic Design Capacities: The Culmination

In 2015 Heckert, Summers, and Swartchick completed a master’s thesis that built upon their collective undergraduate research for their senior capstone projects. They completed the testing and post-test analysis of four walls begun by their respective groups in 2013: two 8’ walls, one 4’ wall, and one 11’ wall with openings. All four walls were constructed using post-and-beam framing, made primarily of 4x4 members. Three of the walls used Heckert and Looney’s “Strong” detailing, and one 8’ wall used their “Workhorse” detailing as a comparison.

They determined that the “Strong” detailing was unnecessary, as the eight foot “Workhorse” wall performed better than the eight foot “Strong” wall (9.8 kips as compared to 7.7 kips, respectively). Their 4’ long wall with “Strong” details resisted a shear of 1.9 kips, approximately a quarter of the 7.7 kip capacity reached by the corresponding 8’ wall.

Their wire tests cited Heckert and Looney’s work, describing an average ultimate strength for 14-gauge 2”x4” welded wire mesh to be 323 lbs. They recommended further research into plaster adhesion and required boundary detailing. As a result of these tests, all subsequent walls have expanded diamond metal lath on the cap and base of each wall, as well as a 10” band of wire mesh at the base, which they used in their workhorse wall. The detailing used in the present research is a modified “Workhorse” detail, per the recommendations in Heckert, Summers, and Swartchick’s paper, in that expanded diamond lath is used at the cap and base and 2”x2” mesh was used rather than 2”x4” mesh.

3.0 Materials

Every wall was constructed using the seven materials described in this section. For each wall, the boundary conditions used consistent materials sourced from the same provider (Material Items 4-7). The same plaster mix for the cement-lime plaster (Material Item 3) was used for every wall in these tests, but the strength of each batch varied from batch to batch. The straw bales and wire mesh (Material Items 1 and 2, respectively) differed in size and type from wall to wall, and these details are discussed below.

3.1 Straw Bales

The straw bales were sourced from Dennis LaGrande of DLG Farms in Williams, CA, and were comprised of rice straw. The nominal 18” wide x 16” tall x 48” long standard bales, used in the Welded-18 and Woven-18 walls, were made using standard baling equipment. LaGrande had to overhaul an older baling machine to make smaller bales, however, and due to limited functionality was forced to tie the 14” x 16” x 48” bales by hand. These 14” wide bales were used in the Welded-14 wall. The hand-tied bales had significant irregularities with regard to length, lack of planar surfaces, and variability of cross-section along the length of the bales.

To obtain the 9” x 16” x 24” bales used in the Welded-9 wall, a standard 18” x 16” x 48” bale was re-tied, chain-sawed in half, and pulled apart lengthwise to create four (4) bales in total. Each bale had two strings. Further, because the bales were now shorter, a running bond was used for the Welded-9 wall, requiring that two 9” x 16” x 24” bales be re-tied and pulled apart to create four 9” x 16” x 12” bales.

3.2 Wire Mesh

The Welded-9, Welded-14, and Welded-18 walls used 14-gauge 2” x 2” electro-galvanized welded steel wire mesh manufactured by Flynn & Enslow, Inc., imported from China. This wire mesh was purchased in 2013 and used in prior SCU tests. Its ultimate strength based on these prior tests is approximately 70 ksi (Heckert and Looney, 2014; Heckert, et al., 2015).

The Woven-18 wall used 17-gauge electro-galvanized steel woven wire mesh with 1-½ inch hexagonal pattern manufactured by Deacero Buildmaster in the U.S. This mesh was also used in Heckert and Looney’s work at SCU. This mesh provides a cross sectional area per unit length that is 59.4% of that of the 14-gauge mesh (Table 4). Based on “Allowable Shears for

Plastered Straw-Bale Walls” (Aschheim, et al., 2014), the woven wire mesh was calculated to reach 73% of the welded wire mesh capacity using the same area per unit length method, but it used a 1.25” spacing for the woven wire mesh’s hexagonal pattern (Parker, et al., 2006). Included are the American Steel and Wire (AS&W) standards for 14-gauge and 17-gauge wire.

Table 4: Dimensions of reinforcing mesh based on the actual measurements of ten specimens. Italicized are the AS&W nominal values.

Wire Type	*Nominal Diameter (in)*	Average Measured Diameter (in)	*Nominal Area (in^2)*	Average Measured Area (in^2)	Nominal Spacing (in)	Area per unit length (in^2/in)
17-gauge woven, 1-1/2” hexagons	*0.0540*	0.050	*0.00229*	0.0020	1.5”	0.0013
14-gauge welded, 2”x2” grid	*0.0800*	0.075	*0.00503*	0.0044	2.0”	0.0022

The tensile strengths of the wire mesh types were determined using the 10-kip machine in the SCU civil lab. The 14-gauge welded wire mesh reached an average peak load of 301 lbs for wires sampled in the transverse direction, and 314 lbs for wires sampled in the longitudinal direction. The 17-gauge woven wire mesh reached an average peak load of 151 lbs, based on specimens with single-strand failures. These loads were averaged from five sample wires each. Their respective strengths can be found in Table 5 below, calculated by dividing their average peak load by their average diameter (Table 4). ‘Transverse’ refers to wires running vertically from top to bottom of the wall, and ‘longitudinal’ refers to wires running horizontally across the wall width.

Table 5: The average strength of the two wire mesh types used in this test series.

Average Wire Mesh Ultimate Strength (ksi)			
	Transverse	Longitudinal	Average
14g welded wire	68.8	69.8	69.3
17g woven wire	77.4	N/A	77.4

The wire strength results are consistent with those of Heckert and Looney. (2014). The welded wire and woven wire they used came from the same rolls as the wire used in this test series. The welded wire used in this test series was, on average, 3% weaker in the transverse direction

(70.7 ksi) and 2% weaker in the longitudinal direction (71.1 ksi) than the wire in the Heckert and Looney test series. The woven wire in this test series, however, was 17.3% stronger in this test series than the 66.0 ksi woven wire they tested (this value was based on the average wire diameter found in the present testing, as the average wire diameter was not reported by Heckert and Looney for the woven wire mesh). The Parker, et al. (2006) test series tested 14-gauge 2x2 welded mesh but did not test 17-gauge woven wire mesh. They sourced their wire from Bekaert Corporation in Van Buren, Arkansas, and their welded wire mesh was 12% stronger in the transverse direction (77.2 ksi). It was concluded that there is variability between wire meshes ordered from different manufacturers, which adds to the complexity of studying strawbale walls.

3.3 Cement-Lime Plaster

The walls were plastered using the cement-lime plaster mix design developed by Avon and Swartchick, as provided in Table 6 by relative volume, weight per batch, and percentage of mix by weight. Avon and Swartchick report the mix as satisfying Type CL cement-lime plaster as specified by the 2003 Portland Cement Association Plaster/Stucco Manual, as adapted from ASTM C926. The relative volumes of sand to cement and lime conform to this Type. The amount of sand is limited to no more than four (4) parts in terms of relative volume, and the sand to cementitious materials (lime and cement) in this mix design is 3:1 by volume.

Table 6: Mix proportions for hand-applied plaster.

Material	Relative Volume	Weight per Batch, lbs (kg)	Approximate Percentage of Mix by Weight
Type II Portland Cement	1.00	13.3 (6.0)	12%
Type S Hydrated Lime	0.75	4.9 (2.2)	5%
Plaster Sand (Quail Hollow No. 2, provided by Graniterock from Felton quarry)	5.25	76.4 (34.7)	71%
Water	1.25*	12.8 (5.8)	12%

*with additional water as needed for workability

Plaster cubes at 2” x 2” x 2” were created to test the plaster strength for each wall. Four to seven batches of plaster were required per wall, and the cubes were taken from a sampling of batches, always three cubes from the same batch. The cubes were tested using the Tinius Olsen machine in the Santa Clara University Structural Lab. The strength of the air-cured plaster for the Welded-18 and Woven-18 walls averaged 2300 psi, from six (6) separate 2” x 2” x 2” cubes (Table 7). The cubes cured alongside the wall until the walls were tested. The strength of the plaster for the Welded-14 wall averaged 580 psi, averaged from nine (9) separate 2” x 2” x 2” air-cured cubes (Table 7). These cubes were tested several weeks after they were made, whereas the cubes of the other two walls were tested several months after they were formed.

Table 7: Mean strength test results of 2” cube samples of plaster mixes described in Table 1.

CUBE SAMPLES (2” sides)				
Test	Cure Type	Time of Test	Mean Strength (psi)	COV
Wire Series	Air	Same Day as Wall Test	2300	7.7%
Welded-14 Wall	Air	Same Day as Wall Test	581	24.8%
Welded-9 Wall	Air	7 days	589	39.5%
Welded-9 Wall	Lime Bath	7 days	562	30.6%

The samples for the Welded-9 wall were tested at seven days after mixing. Five cubes air-cured near the wall, and five cubes cured in a lime-saturated water bath. Another five lime-cured cubes will be tested at 28-day strength, as three cubes were taken from each of five batches of plaster that constituted the first layer of plaster on both sides of the wall. Their results are detailed in Table 8.

Table 8: Results of Welded-9 Wall plaster batches 1-5, constituting the first ½ layer on either side of the wall. The 7-day compressive strengths of the 2” cube samples are listed by cure type.

	Compressive Strength (psi)		
Batch	Air-Cured	Lime-Saturated Water Bath	Percent Difference Between Air- and Lime-Cured
1	570	636	11.56%
2	924	661	-28.5%
3	323	315	-2.7%
4	437	460	5.2%
5	692	738	6.7%
Average	**589**	**562**	**-4.7%**

When tested, the cubes from Batches 2 and 3 were stronger air-cured than lime-cured, which skewed the average to show a higher cube strength when air-cured as opposed to cured in a lime-saturated water bath (Table 8). This unexpected result could be due to non-uniform tamping when consolidating the plaster in the molds, such that interior voids propagated cracks. The lime and cement used in the plaster mix sometimes had chunks of lime or plaster that did not get crushed in the mixer, and it is possible these chunks (<0.5in diameter) influenced the strength of the plaster cubes if present within them. It could also have been caused by other unidentified issues. Though the data of Batch 3 was significantly lower than the other four batches and the Batch 2 data differs from expected results, the average of all five tests is included in Table 7 for comparison. Note that the coefficient of variation for the Welded-9 plaster was 39.5% for the air-cured cubes and 30.6% for the lime-cured cubes.

The IRC required strength of cement-lime plaster mixes is 1000 psi, which is a lime–saturated water bath cured strength based on cube tests (Appendix S, Table AS 106.12). The plaster on the first two walls, in the Wire Series, exceeded the requirements even when air-cured. The plaster of the Welded-14 wall was less than 60% of required strength when air-cured (Table 7), so it must be adjusted for lime-cured strength to determine if it met the IRC requirements.

According to the results of Avon and Swartchick, the strength of their lime-cured plaster (with the same mix design) was 1.59 times as large as its air-cured counterpart (2015). Even given this estimated adjustment to the air-cured cube sample strengths, the adjusted strength of the Welded-14 plaster is estimated to be 921 psi, which is somewhat below the 1000 psi IRC requirement. Given that the 7-day strengths of the Welded-9 plaster cubes showed a decrease in strength from air-cured to lime-cured, the 1.59 adjustment factor was a generous increase, especially given the relatively high variability in the cube strengths. The use of this high factor was meant to show that the Welded-14 plaster still falls short of the 1000 psi requirement, even when generously adjusted for increased lime-cured strength.

At seven days, the average lime-cured Welded-9 wall plaster strength was 562 psi. Using the rule of thumb regarding plaster strength, the cubes would have reached 70% of their 28-day strength at seven days. Therefore their capacity after more than 28 days of curing is estimated to be 562/0.70 = 803 psi. This strength is also somewhat below the 1000 psi minimum required by the IRC. For comparison, the cubes made from the University of Illinois tests averaged a 7-day strength of 1850 psi, which is 84% of the 2200 psi strength at the time of testing (Ash, et al., 2013).

The highly variable plaster strength results make it difficult to determine accurate plaster strengths, especially when trying to predict lime-saturated strengths to compare to IRC requirements. Despite the variability, it appeared that the plaster on the Welded-14 and Welded-9 walls did not meet the 1000 psi minimum strength provided in the 2015 IRC.

3.4 Timber

Various timber beams and plywood sheets were used in this test series. To construct the caps and bases, 4x4 beams and ½" thick plywood (or ¾" thick plywood for the Welded-9 wall) were used. The cap and base simulate a typical floor and ceiling connection. For each wall, the base was bolted to a structural support (known informally as a "sled"), which provided stability during transportation from the structural lab to the hydraulic testing frame. The sled also allowed the wall to be bolted to the floor to prevent lateral slippage during seismic testing. All timber was Douglas-Fir, sourced from a local lumberyard.

3.5 Staples

The asphalt paper, metal lath, and wire mesh were connected to the base and cap using galvanized steel staples. The pneumatically driven staples are 16-gauge x 1.75 inch (44mm) leg, medium crown chisel point, (SENCO N19BAB) heavy wire staples.

3.6 Expanded Metal Lath

A 6" wide strip of expanded diamond metal lath manufactured by CEMCO, California Expanded Metal Products Company was pneumatically stapled to each 4-foot side of the base and headers. The product came in 8-foot long strips labeled as "galvanized Stripite."

3.7 Asphalt Paper

A 4" wide strip of asphalt paper was stapled along the full outside length of each base and cap, covering the 4x4 member and the edge of the plywood exposed on the side. In practice, strawbale walls typically use two strips of asphalt paper stapled on top of each other to provide adequate moisture protection. This flashing acts as a barrier between the wood and plaster so that the wood does not absorb moisture when the plaster is applied. The product conforms to

Federal Spec. UU-B-790A, Type 1, Style 4, Grades A-B-C and was manufactured by Salinas Valley Wax Paper Co., Inc.

4.0 Experimental Design

To address the two objectives, regarding wire mesh type and slenderness, four wall specimens were constructed. One wall specimen, Welded-18, was the reference wall for all tests. A second wall, Woven-18, was built to examine the effect of woven wire mesh as compared to welded wire mesh. These two walls constituted the “Wire Mesh Series” described in the section Wire Mesh Series. The final two walls, when compared to the Welded-18 wall, constituted the “Slenderness Series” outlined in the section of the same name. The third wall was constructed using small, 14”-wide bales (Welded-14) and the fourth wall (Welded-9) used the ¼-size bales with 16” x 9” x 24” dimensions. Table 9 clarifies the wall names and which test they were used in.

Table 9: Clarification of test series name and walls used to make conclusions about the test series objective.

Test Series Name	Walls Compared
Wire Mesh Series	1. Welded-18 2. Woven-18
Slenderness Series	1. Welded-18 2. Welded-14 3. Welded-9

All four wall specimens were constructed in the Civil Structural Lab at Santa Clara University, and three of the four walls have already been tested under a reversed cyclic loading protocol in SCU’s 3-DOF lateral test frame. The specimens were of the style termed “load-bearing” due to the absence of post-and-beam wood framing. The standard bales for the Welded-18 and Woven-18 walls were placed in stacked bond rather than running bond because the nominal length of the wall (in plan) was only 4 ft (1.2 m). To simplify construction and use resources efficiently, the walls were approximately 8 ft (2.4 m) high, including the wooden base and cap. All bales were made of rice straw, and they are depicted in Appendix B.

Before stapling the wire mesh to the base and cap, the bales were compressed using a series of straps tightened around the perimeter of the wall. The wire mesh reinforcement was fixed in position at approximately 12” intervals (horizontally and vertically) to the bales using 12-gauge wire “hairpins,” normally used for hanging T-track for suspended ceilings.

Plaster was applied to both sides of the wall in two nominally ½ inch layers, encasing the wire mesh. The same cement-lime plaster mix was used for all walls and all plaster batches (Table 6). Typically the scratch coat came up to the mesh (approximately ½”), and the brown coat then continued ½” beyond the mesh to create a nominal 1” thick shell of plaster in total. The hand-applied scratch coats were scratched using a standard 12” scarifier. Both coats were finished with a hand trowel. The total plaster thickness was controlled with depth guides provided by plastic standoffs held in place with the 12-gauge hair pins. No finish coat was applied.

To further maintain consistency, the same detailing was used in constructing each of the four walls (Figure 1). For the base connection, the Woven-18 specimen had expanded lath and a 10-inch band of 2”x2” welded wire mesh sandwiching the main woven wire mesh that ran full height. These are referred to as “Heckert details” since they were recommended by Heckert, et al. (2015). The bales were stacked vertically and encased in wire mesh, which was stapled to the wooden header (“cap”) and base to secure the system. Each base and cap were protected by a layer of asphalt paper as would be done in practice, to prevent moisture leaching out of the plaster and to avoid moisture-related damage to the wood.

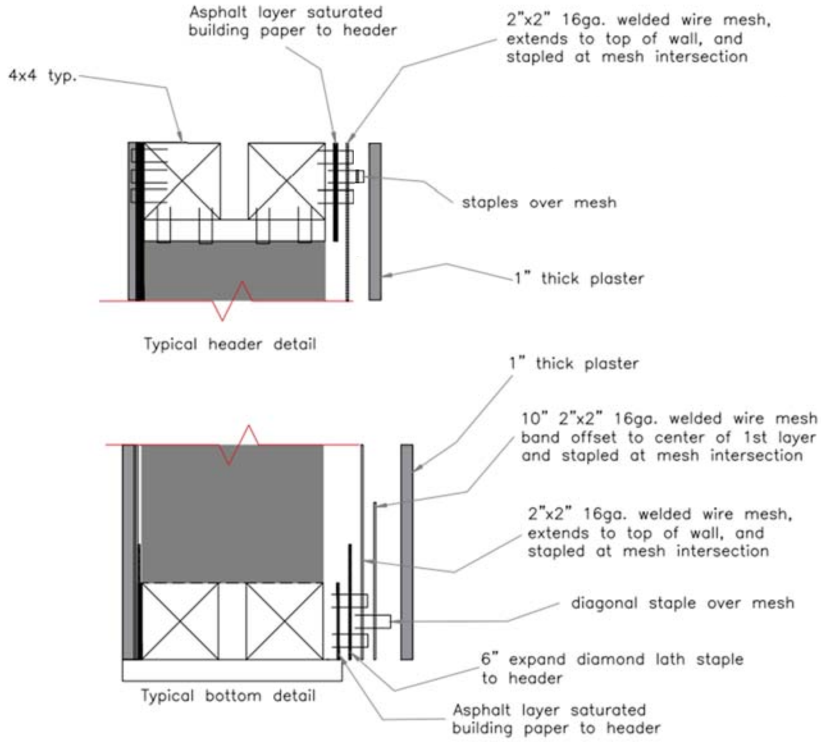

Figure 1: Header (cap) and base detail for welded wire walls shown in exploded view on the right of each image. The only difference for the woven wire wall details was the type of wire used for the mesh.

An additional 10” of welded wire mesh band was stapled to the base to strengthen the connection region. Due to the fixed connection at the base of the wall and pin connection at the top, more damage is expected to occur at the base. The entire wall acts like a cantilever, so the bending moment is highest at the base. Staples were used to secure the layers of wire mesh and asphalt paper to the cap and base, as shown in Figure 1. The staples were applied diagonally across each of the mesh intersections.

4.1 Wire Mesh Series

Fourteen-gauge 2” x 2” electro-galvanized welded steel wire mesh was selected for the first wall, Welded-18 (Figure 2), which used nominally 18” thick bales stacked flat. The comparison wall, Woven-18, used 17-gauge woven wire mesh and also used 18” thick bales (Figure 3). Both walls used the bales stacked flat, so that their 18” side governed the width of the wall, which was nominally 20” wide including the 1” plaster layer on either side. Both walls were nominally four feet (4’) long.

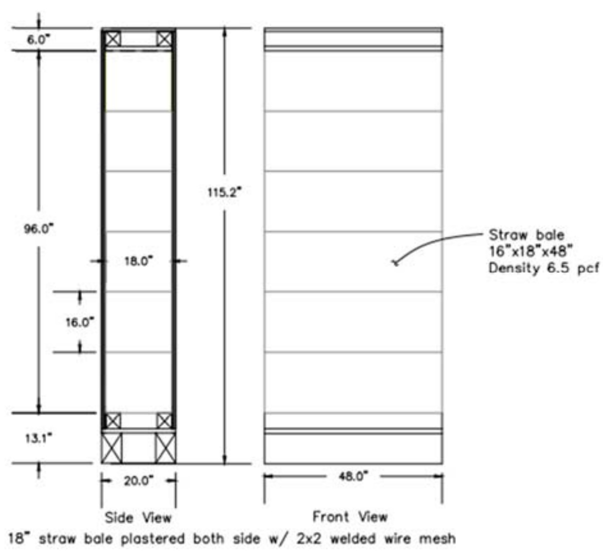

Figure 2: Welded-18 wall

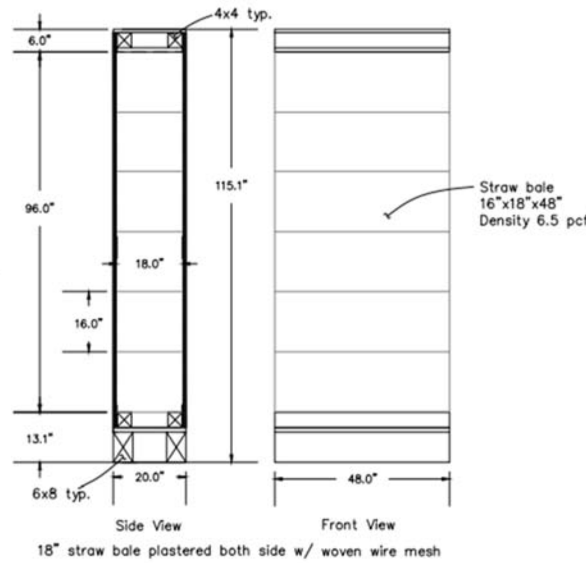

Figure 3: Woven-18 wall.

4.2 Slenderness Series

The three walls tested in this series compare the strength of a standard wall (18" wide bales), a thin wall (14" wide bales), and a limit wall (9" wide bales). Their respective names are Welded-18, Welded-14, and Welded-9. As described in "Background on Objective 2" the Welded-9 wall represents the very limit of the building code (Figure 5), while the Welded-18 acts once more as a comparison wall (Figure 2). The purpose of the Welded-14 wall is to examine the performance of slightly smaller bales (Figure 4).

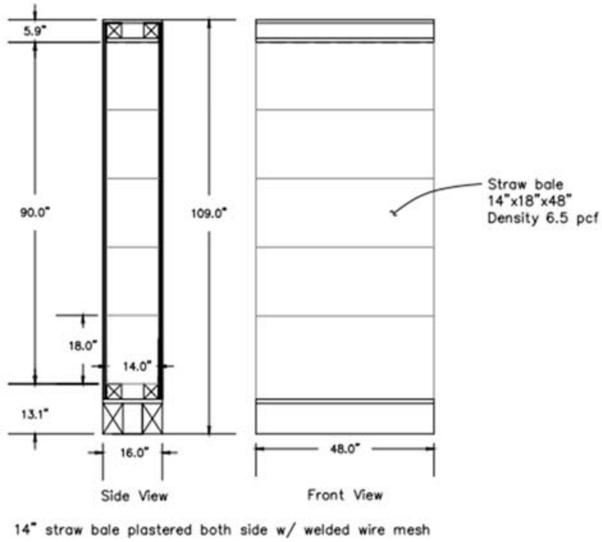

Figure 4: Welded-14 wall.

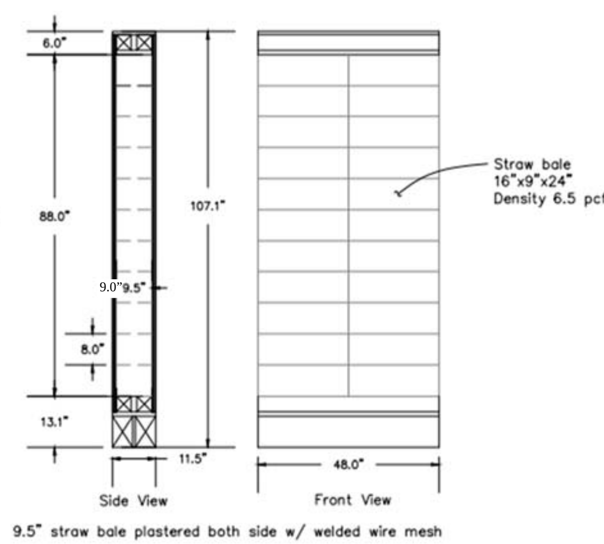

Figure 5: Welded-9 wall.

A standard straw bale is 16” x 18” x 48” and can be oriented on its 16” side (“on edge”) or 18” side (“laid flat”) when stacked to form a wall. Welded-18 from the Wire Series, at 18” wide because the bales were laid flat, was used as the basis for comparison in the Slenderness Series. Some balers produce 14” wide bales, and the question remains as to whether these thin bales significantly impact wall strength. A new wall, Welded-14, was constructed out of 14” x 16” x 48” bales, oriented on edge so that the nominal wall thickness was 14 inches. Therefore, the performance of a thin wall could be quantified.

The final wall of this series is the limit wall, Welded-9. Using 9” wide bales and having a nominal 8 feet height, the Welded-9 dimensions reach the slenderness limit set by the building code. A standard 18” bale was quartered using a chainsaw, and the strawbale “bricks” (9” x 16” x 24”) were placed in a running bond for ease of construction. All other details matched the other two welded wire mesh walls.

5.0 Testing Procedure

The same standard test protocol was followed for each strawbale wall in this seismic test series. The wall was pinned at the top, and two structural T-shapes served as stability guides, located at the top on either end of the wall to prevent possible twisting during the test (Figure 6).

Figure 6: T-Plate slotted into the cap of the Welded-18 wall.

The testing protocol was a modified CUREE Caltech protocol found in ASTM E 2126-02A (Krawinkler, 2009; Heckert, et al, 2015). The reversed cyclic loading provided one leading cycle with two trailing cycles at 75% of the leading cycle (Figure 7). A "leading cycle" is defined as a cycle that reaches a new peak displacement within the test, whereas a "trailing cycle" follows a leading cycle, and does not reach a new peak displacement. The reference drift limit is approximately 1% of the wall's height, taken as 0.96" for all four walls, even though the Welded-14 and Welded-9 walls were respectively 6" and 8" shorter than the Welded-18 and Woven-18 walls. Individual tests were terminated at arbitrary displacements based on the condition of the wall.

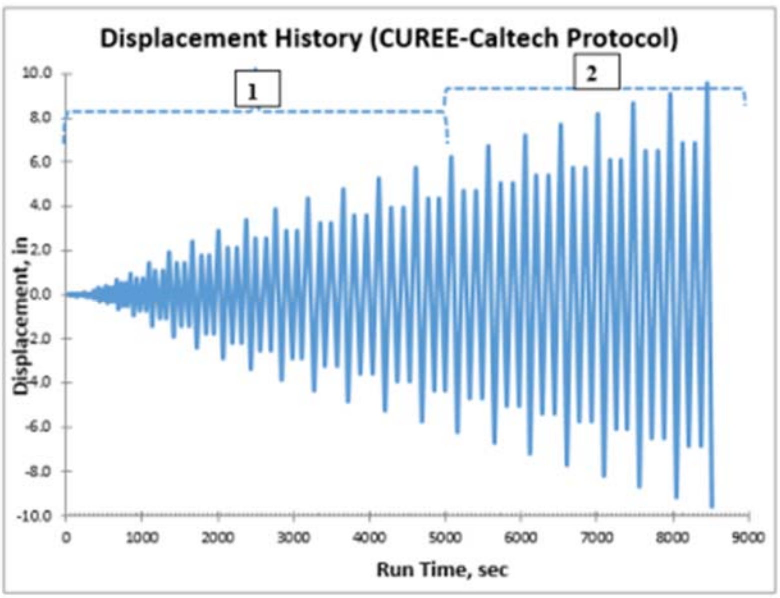

Figure 7: Applied reversed cyclic lateral displacement history, based on CUREE-Caltech Protocol (Section 1 of the graph) with an extension (Section 2 of the graph). Taken from Heckert, et al. (2015).

The actuators supporting the test beam contained spherical bearings. There was friction in the system as well as a second-order (P-Delta) effect associated with the weight of the loading beam, wherein the lateral actuator resists the tendency of the frame to collapse to either side under self weight. The bare frame response must be removed from the results of the test via formula, specific to the SCU test frame.

To address the question sometimes asked about how well a strawbale wall can resist gravity load after seismic shaking, all four walls were subjected to a vertical loading test after the completion of lateral testing. The wall was displaced to 3% drift (approximately 3 inches) prior to application of vertical displacements. The vertical test continued in increments of vertical displacement until either bowing of the wall (for Welded-9) or shifting wall plaster encroached on unintended restraints.

6.0 Results

Individual wall results are presented in this section, and the comparison between walls can be found in the Discussion section. For each lateral test, a procedure was followed to account for the response of the bare frame described in the Testing Procedure section. Presented here are the results corrected according to the April 16, 2017 version of the "Correction of Test Data" document outlining the protocol, written by Mark Aschheim.

6.1 Experimental Results

6.1.1 Welded-18 Wall

The first wall tested, Welded-18, was the reference wall for evaluating other tests. This wall was made of 18" wide rice straw bales encased in 14-gauge 2" x 2" welded wire mesh. It reached an average peak lateral load of 2338 pounds at a lateral displacement of 1.64 inches, which resulted in a drift of 1.5 % when considering the wall's height as the nominal height of the plastered area, 106 inches (Table 10).

Table 10: Peak values from Welded-18 lateral test, including lateral loads at specified displacements. Given for both the positive and negative direction.

Welded-18 Lateral Test Results				
	Peak Lateral Load (lbs)	Displacement at Peak Load (in)	Load at 1% drift (lbs)	Load at 3% drift (lbs)
Positive	2456	1.65	1626	905
Negative	2220	1.62	1455	1433
Average	2338	1.64	1541	1169

The graph of the lateral load-lateral displacement response from the seismic test shows a significant reduction in carried load after the peak was reached at 1.6% drift (Figure 8). Its failure, like all of the walls, was governed by flexure due to the walls 2:1 aspect ratio (8' tall to 4' length).

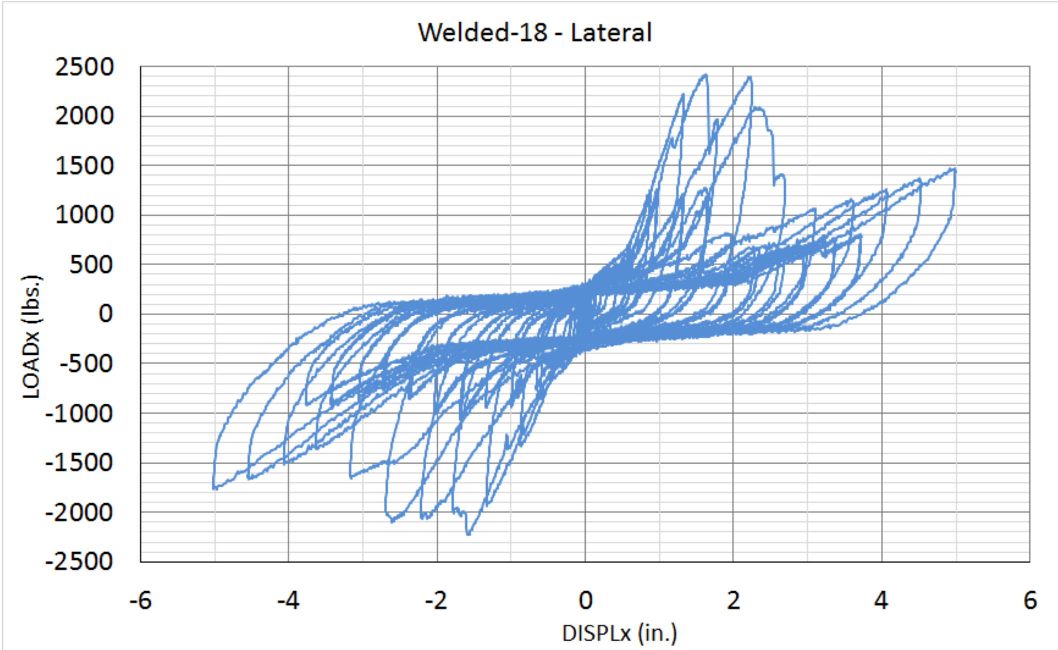

Figure 8: Load versus displacement for lateral test of Welded-18 wall.

During the Welded-18 lateral test, the plaster delaminated where the first applied coat met the second applied coat, i.e. the two layers encasing the wire mesh. This was especially evident on the back side of the wall, where chunks of the outer plaster layer spalled off the inner layer at the bottom left and right corners. The second layer of plaster spalled off the wooden base at the bottom of the wall, though the first layer remained intact, protecting the staples (Figure 9).

Figure 9: Outer layer of plaster delaminated from the inner layer of plaster at the base of the wall. The stapled connections of mesh to the sill plate seem to still be intact.

During the later cycles, when the top of the wall was subjected to lateral displacements of 5” or more, the base (or sled) of the wall slid relative to the test floor a maximum of ½” laterally at the peak of each leading cycle. Figure 10 shows the two 1-inch diameter threaded rods restraining a 6x6 member across the base of the wall. The bearing between the 6x6 and wall base was not strong enough to prevent slippage as the test progressed (Figure 10). Subsequent tests employed plates affixed to either side of the sled, shimmed tight to the sled and bolted to the test floor, to prevent sliding.

Figure 10: Connection of the base of the wall to the floor of the test frame.

The bottom half of the plaster skin buckled, especially near the connection at the base (Figure 11). Except where the plaster buckled outward and delaminated, most of it remained bonded to the wire and straw. Cracks in the plaster mainly proliferated near the bottom of the wall, and those that formed higher up did not grow much after initial formation.

Figure 11: Wire mesh buckling at the base of the wall near the end of the lateral test, after peak displacement had been reached. Each side was repetitively subjected to compression and then tension.

The following images present the wall at approximately 1%, 3%, and maximum displacement. These values will be compared across all walls in the “Discussion” section.

Figure 12: Welded-18 wall at (a) 1.28” lateral displacement with lateral load 998 lbs and (b) 2.88” lateral displacement with lateral load 1567 lbs. These photos were taken during a leading cycle.

Figure 13: Welded-18 wall at 4.3” lateral displacement with lateral load 521 lbs. Taken during a leading cycle.

6.1.2 Woven-18 Wall

The second wall tested was the Woven-18 wall, constructed using the rice straw bales laid flat (18” wide) and 17-gauge woven wire mesh. It reached an average peak lateral load of 1,702 lbs at 1.33 inches, which corresponds to 1.3% drift given a nominal height of 106 inches (Table 11). In the positive and negative direction, peak lateral loads deviated from the mean by only 1.5%.

Table 11: Peak values from Woven-18 lateral test, including loads at specified displacements.

Woven-18 Lateral Test Results				
	Peak Lateral Load (lbs)	Displacement at Peak Load (in)	Load at 1% drift (lbs)	Load at 3% drift (lbs)
Positive	1676	1.33	1454	650
Negative	1727	1.32	1569	989
Average	1702	1.33	1512	820

Its graph shows a significant reduction in lateral load (Figure 14). It reached 73% of the capacity of the Welded-18 wall, and it also lost approximately 50% of its lateral resisting strength after reaching a peak.

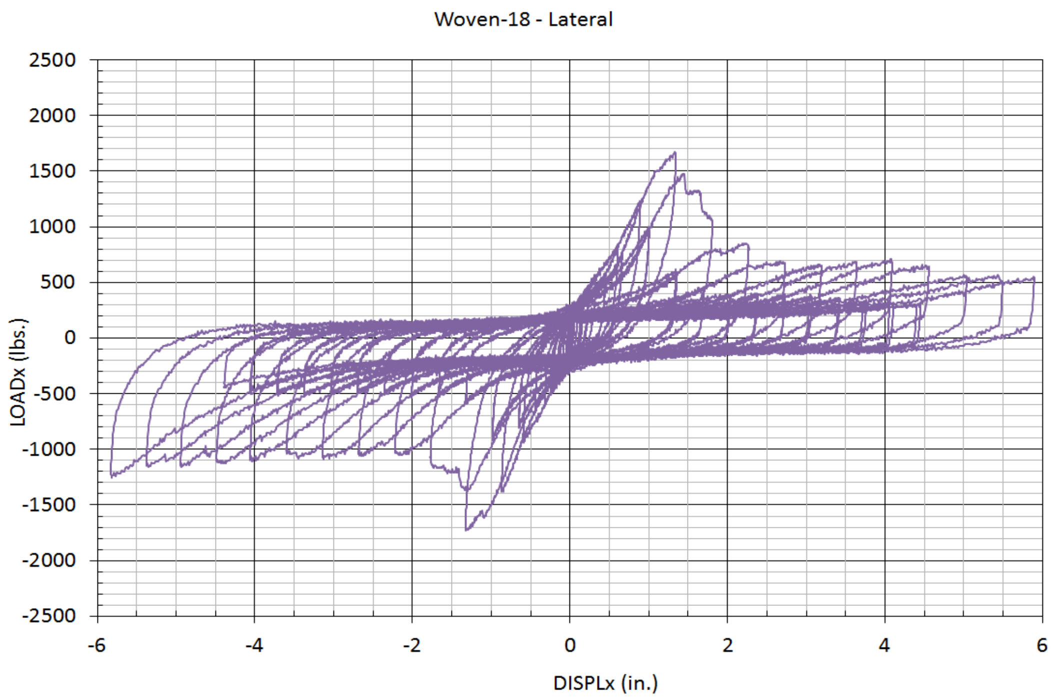

Figure 14: Lateral test results of the Woven-18 wall.

Prior to the Woven-18 lateral test, the test frame crushed the top right corner of the wall, damaging the plaster (Figure 15). The top one (1) foot of plaster was removed from the wall and replaced with new plaster made to the same specifications (Figure 16). A small crack formed at the interface between old and new plaster, but it didn't propagate during testing. The demarcation of darker old plaster and lighter new plaster is visible in the images below.

Figure 15: Crushing damage done to the top right corner of the wall prior to the lateral test.

Figure 16: The new plaster can be seen here as a band of lighter plaster at the top of the wall.

During the Woven-18 lateral test, the damage was concentrated at the bottom. The exterior layer of plaster delaminated from the interior layer of plaster and popped off due to bending of the wire mesh. The mesh elongated in flexural tension and bent out of plane when loaded in compression. Most of the interior layer of plaster remained attached to the straw bales. Cracks in the plaster followed the “diamond” pattern of the woven wire mesh, and propagated diagonally from the bottom corners and edges of the “diamonds” (Figure 17). This is attributed to the net plaster section being reduced at the location of a wire, thus leading to higher tensile stresses at the location of a wire.

Figure 17: Taken post-lateral test. Cracks in the plaster following the “diamond” pattern of the woven mesh. Delamination is evident here where the outer layer popped off to expose the woven wire, while the inner layer remained in place.

Figure 18: The damage was concentrated in a line approximately 10” above the base, due to the extra 10” band of wire mesh added to the modified workhorse connection detail, also termed the “Heckert detail.”

For this test, a new restraint system used 8x10 Douglas-Fir beams to resist overturning moment and to keep the base of the wall in place. The combination of the 8x10 Dougla-Fir beams and plates bolted to the test floor and shimmed to block sliding at end of the wall successfully kept the base of the wall in place and prevented the slippage that had occurred during the Welded-18 test. See the Welded-14 wall results for an image of the restraint system (Figure 23), as the same system was implemented for the Woven-18 and Welded-9 as well.

The following images present the wall at approximately 1% and 3% displacement (Figures 19a and 19b). These values will be compared across all walls in the “Discussion” section. Figure 20 shows damage of the wall at the end of the lateral testing.

Figure 19: Woven-18 wall at (a) 1.06” lateral displacement with lateral load 1402 lbs and (b) approx. 3” lateral displacement at an unspecified lateral load. Figure 19(a) was taken during a leading cycle, and Figure 19(b) was taken during a trailing cycle.

Figure 20: Backside of the Woven-18 wall, post-lateral test. Extensive delamination of the outer plaster layer has revealed the inner plaster layer. Plaster at the base remains intact and seemingly protects the stapling at the connection underneath.

6.1.3 Welded-14 Wall

The third wall tested was the Welded-14 wall, which was constructed of nominally 14”-wide rice straw bales stacked on edge. During the lateral test, the Welded-14 wall had a tendency to buckle, though the modified connection details kept the staple-wire connections at the base and the top undamaged. The bales did not stack true and hence the plaster undulated. During testing the plaster moved laterally when subjected to compression or tension as part of the induced bending moment—that is, an “S-shaped” undulation would become more S-shaped under compression and less S-shaped under tension.

The wall reached an average peak lateral load of 1591 lbs, but the load carried by its left and right sides varied from the average by 21.4% relative to the mean (Table 12). When the wall was displaced in the positive direction (to the left in most photographs), its ability to carry lateral load was higher. Even the displacement at the peaks differed, showing that the peak in the negative direction was reached at a lateral displacement of 1.30” while the peak in the positive direction was reached at 1.64” (Table 12).

Table 12: Peak values from Welded-14 lateral test, including loads at specified displacements.

Welded-14 Lateral Test Results				
	Peak Lateral Force (lbs)	Lateral Displacement at Peak (in)	Lateral Force at 1% drift (lbs)	Lateral Force at 3% drift (lbs)
Positive	1931	1.64	1690	821
Negative	1250	1.30	1089	670
Average	1591	1.47	1390	746

From the graph (Figure 21) it appears that the peak lateral forces in both directions were reached in the same leading cycle. Both peaks occurred at approximately 1.5% drift.

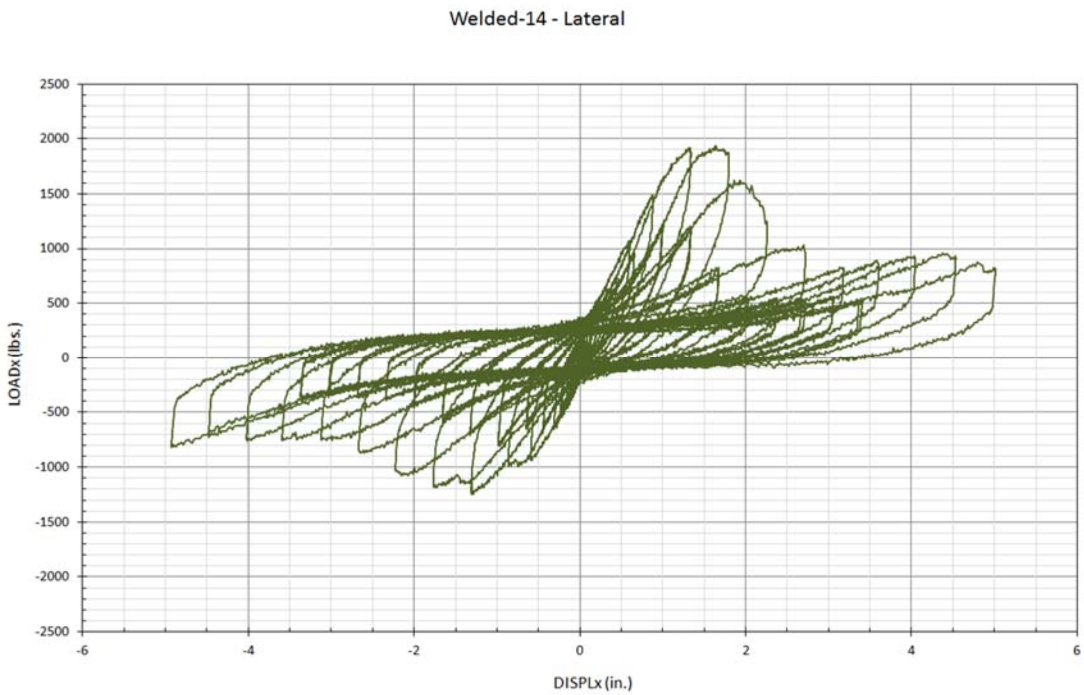

Figure 21: Welded-14 lateral test hysteresis test.

During the Welded-14 lateral test, the wall's irregular, wavy shape contributed to a higher tendency to buckle in compression, even during the lateral test (Figure 22). The lack of tension in the welded wire caused the wall to "breathe" during the test, where both sides of the plaster buckled outward.

Figure 22: A picture of the Welded-14 wall prior to testing. The wall was not uniform and bore a distinct vertical warp.

The base, however, remained secure and did not slide laterally due to improved overturning and sliding restraints (Figure 23). The modified “workhorse” connection details at the base of the wall performed well, concentrating the damage in a line 10” above the base, protecting the staples at the base connections (Figure 24).

Figure 23: The new restraint system implemented in the Woven-18 and Welded-14 walls.

Figure 24: Damage at the base of the Welded-14 wall post-lateral test was concentrated approximately 10” above the base of the wall, due to the extra 10” band of wire added to the modified workhorse connection detail. The outer layer of plaster delaminated from the left side, but the inner layer remained intact, protecting the staples underneath.

The following images present the wall at approximately 1%, 3%, and near maximum displacement (Figures 25a, 25b, and 26). These values will be compared across all walls in the “Discussion” section.

Figure 25 Welded-14 at (a) 1.17” lateral displacement and load 1704 lbs and (b) 3.26” lateral displacement with load 520 lbs. Both images were taken during a leading cycle.

Figure 26: Welded-14 at 3.70” lateral displacement with load 229 lbs. Taken during a trailing cycle.

6.1.4 Vertical Tests

Though all walls were subjected to a vertical test after completing the lateral test, the results of only two walls were successfully recorded—Woven-18 and Welded-14. The data from the Welded-18 wall was overwritten during subsequent testing before it had been properly saved. Thus, the results from the Woven-18 wall were taken as representative of a vertical test on an 18” wide wall. All vertical tests occurred with the walls at 3% lateral drift.

The Woven-18 wall reached a peak vertical load of 7100 lbs (Figure 27). Over the duration that a displacement was held, the vertical load decreased, suggesting creep (Figure 28). The expected load was 1200 lbs (see Discussion for details). The woven wire wall was tested to a maximum vertical displacement of 6.8 inches. The Woven-18 wall was able to maintain a load of 6900 lbs even as it was compressed almost seven (7) inches; this load is called the “Sustained Vertical Load” in Table 5.

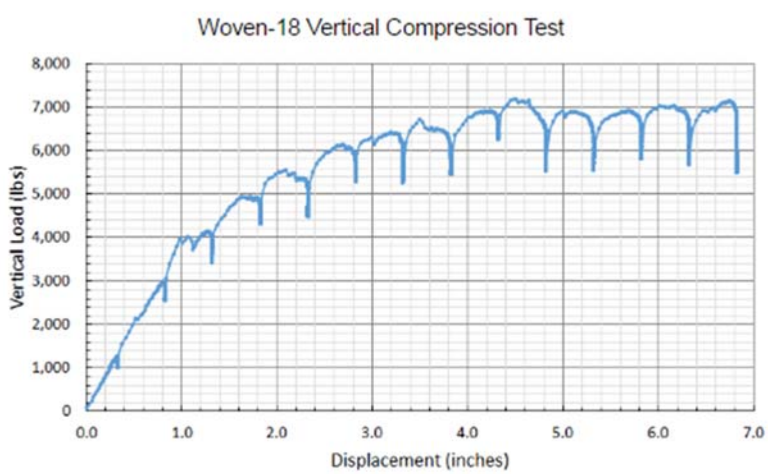

Figure 27: Vertical test of Woven-18 wall in load vs. displacement.

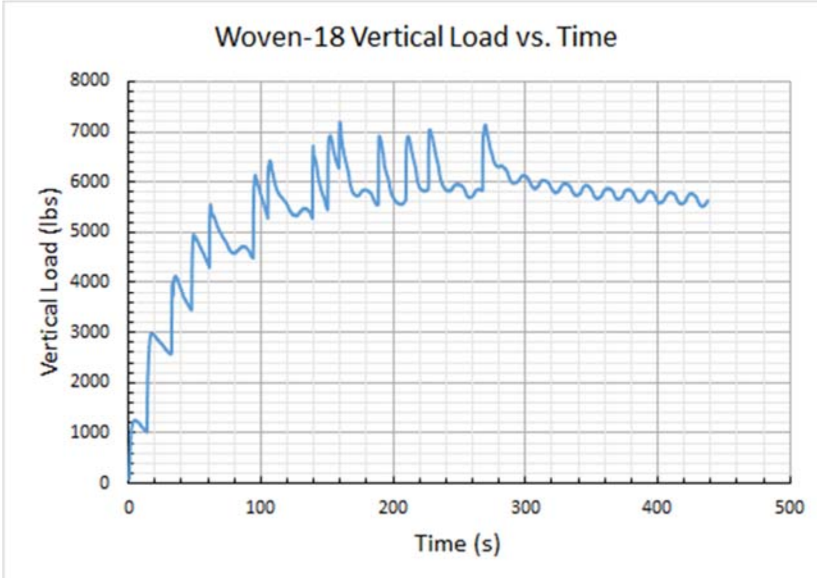

Figure 28: Vertical test of Woven-18 in load vs. time.

The Welded-14 wall's maximum compressive load was 2710 lbs, but the maximum sustained load was only 1,600 lbs (Figure 29). "Sustained" was defined as the load the wall was bearing even after it was vertically displaced by 4 - 6 inches. The wall failed by global out of plane buckling. The unusual degree to which this wall deviated from true vertical suggests that even poorly constructed walls will have adequate capacity to hold roof loads (expected to be approximately 1,200 lbs) except in cases of unusually high vertical loads; attention to vertical trueness will allow much higher capacities to be counted on.

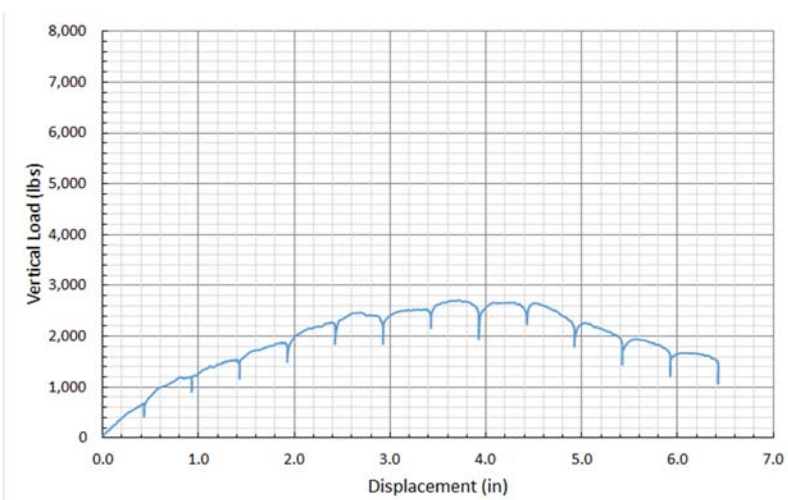

Figure 29: Vertical test of Welded-14 wall in load vs. displacement.

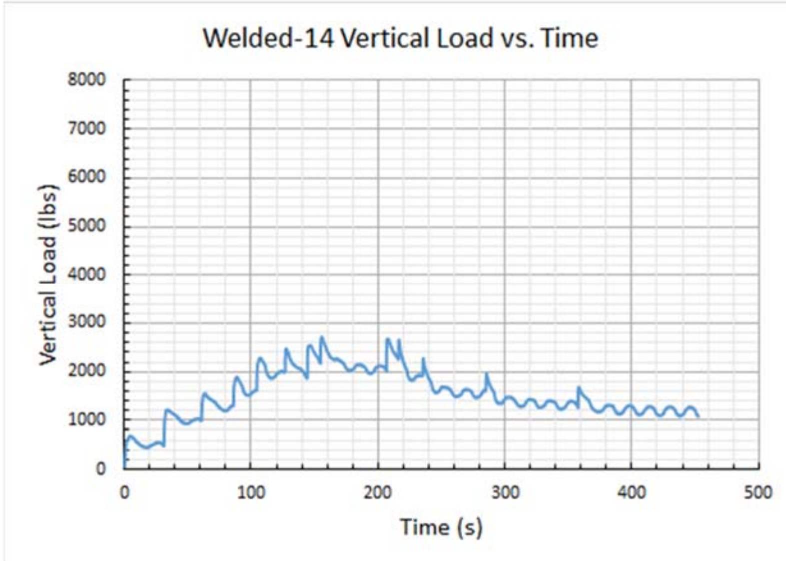

Figure 30: Vertical test of Welded-14 wall in load vs. time.

As shown in the vertical load versus time graphs, the slope after each peak decreased more quickly as the load got higher. If given enough time, the vertical load may have returned to zero.

Due to its tendency to buckle (Figure 31), the Welded-14 wall reached a lower peak vertical compressive load than the 18" thick Woven-18 wall (Table 13). The Welded-14 Wall also maintained about 25% of the vertical load that the Woven-18 Wall sustained when compressed to the approximately the same displacement.

Table 13: Maximum vertical loads and continuous sustained loads.

	Woven-18	Welded-14
Max Vertical Load (lbs)	7194	2710
Sustained Vertical Load (lbs)	6900	1600

In both cases, damage was concentrated at the bottom of the wall, as well as the middle sections where flexure took place. The modified workhorse details from Heckert et al. moved the damage up from the base, which was observed in the lateral test, and concentrated the damage in a line approximately 10" above the very bottom of the wall (Figure 32).

Figure 31: Side view of the Welded-9 wall buckling during the vertical test.

Figure 32: Damage sustained approximately 10” above the base of the Woven-14 wall after the vertical (and lateral) tests, after returning the top of the wall to its pre-test position.

6.2 Analytical Results

6.2.1 Expected Strengths Based on Material Properties

Simple models were created by Heckert, et al. to estimate flexural and shear strengths of walls based on approximations of conventional reinforced concrete flexural strength models and shear strength models (2015). The models consider the material properties of the plaster and wire, the dimensions of the wall, and the strength of the stapled connection. Heckert, et al. applied the models to estimate flexural strengths for each of their walls. On average, the actual strengths were within 31.4% of the model predictions (75.9% COV). As seen in Table 14, their 4' wall experimental strength was within 23.3% of the strength determined for a 4' wall (Heckert, et al., 2015). All walls, except the 11' long wall, were predicted to fail in flexure.

Table 14: Data from Heckert, et al.'s lateral test of four walls as compared to their predicted strength using the simple model based on material properties. Apparent modes of failure taken from narrative (2015).

Wall Length (ft)	Detailing ("Strong" or "Workhorse")	Apparent Mode of Failure (Shear or Flexure)	Predicted Lateral Strength (kips)	Experimental Lateral Strength (kips)
8	Strong	Flexure	7.8	7.7
8	Workhorse	Neither: premature staple failure	4.9	9.8
4	Strong	Neither: post and sill disconnection	3.3	4.3
11	Strong	Neither: individual panel shear friction failure	23.0	11.3

The walls in the current test series were governed by flexure, so the model was adjusted for new detailing and applied to these walls. The height of the wall was taken as the distance from the top pin of the actuator in the 3DOF Structural Test Frame (approximately 3" above the cap) to the height of the mesh band (10" above the base), assuming this to be the location of the critical section. The area per linear inch of mesh reinforcing was taken from Table 4, and the strengths of the woven and welded wire meshes were taken from Table 5. Yield strengths were estimated to be 1/1.10 times the ultimate strengths of Table 5. The average cross-sectional areas of Table 4 were used, rather than the AS&W nominal areas. The model gave strengths based on only one plastered face of the wall, so the values in

Table 15 had been multiplied by two (2) to account for the strength of the two plastered faces. In this way the model was modified to determine the expected strengths of the walls in these test series, and their predicted strengths can be found alongside the actual strengths in Table 15 below.

Table 15: Comparison of wall strengths (plastered on both sides) based on analysis and determined from tests.

Wall	**Shear Associated with Flexure at Base (lbs)**	**Shear Associated with Shear Failure (lbs)**	**Calculated Strength at Top (lbs)**	**Actual Average Strength (lbs)**	**Percentage of Expected**
Welded-18	*3,218*	17,276	3,218	2,338	72.7%
Woven-18	*1,569*	10,126	1,569	1,702	108.5%
Welded-14	*2,785*	14,241	2,785	1,591	57.1%
Welded-9	*2,851*	14,580	2,851	N/A	N/A

*Calculated for both faces plastered, acting together; governing calculated failure load shown in italics.

Though the models give an estimate of the strength, their predictions deviate from the actual strength by an average 26.2% with two out of the three estimates falling short of the actual wall strengths (COV 65.7%). This disparity was comparable to the variance that Heckert, et al. found, which predicted actual strength within 31.4% (75.9% COV).

To determine the impact of plaster strength, wire strength, and height of the walls, a simple analysis of two iterations of the model was conducted. Variables were adjusted in the Welded-18 and Welded-14 iterations of the model, as seen in Table 16.

Table 16: Model analysis of variables' impact. This table shows what impact, in percent, each variable had on wall strength.

Wall	Variable	Change	Impact on Wall Strength (+/- %)
Welded-18	Steel Ultimate Strength	+/- 10ksi	12%
	Height	+/- 2 inches	2%
	Plaster	+/- 300 psi	1%
Welded-14	Steel Ultimate Strength	+/- 10ksi	9%
	Height	+/- 2 inches	2%
	Plaster	+/- 300 psi	19%

Height had minimal impact in both cases, and the impact of plaster appeared to depend on original plaster strength which was 2300 psi for Welded-18 and 581 psi for Welded-14. The greatest impact on wall strength occurred when adding and subtracting 300psi from the strength of the Welded-14 plaster, but changing the steel ultimate strength contributed an approximately 10% change for both wall strength models.

6.2.2 Expected Strengths based on Past Studies

Based on the testing of full-scale walls at the University of Illinois, expected strengths were estimated for walls with 17-gauge woven wire mesh and 14-gauge 2” x 2” welded wire mesh (Ash, et al., 2003). The shear strength per linear foot of their Wall D, reinforced with woven wire mesh, was 0.8 kips/ft (6.4 kips total as seen in Table 1). Their medium-detailed cement-lime plaster wall, Wall E, had a strength of 2.4 kips/ft (19.0 kips total) and their heavy-detailed cement-lime plaster wall, Wall F, had a strength of 2.2 kips/ft (17.9 kips total); both of these walls used welded wire mesh (Table 1).

For other variables held constant, flexural strength is expected to be a function of the square of the plan length of the wall (simply, the tensile force developed by the mesh is proportional to length, and the lever arm associated with bending moment is proportional to length). Thus, based on these previous experimental values, four-foot long walls with 2x2 14-gauge mesh would be expected to have a strength of $17.9/(2^2) = 4.48$ to $19.0/(2^2) = 4.75$ kips (averaging 4.6 kips), while the four-foot walls with woven wire mesh would be expected to have a strength of $6.4/(2^2) = 1.6$ kips. Differences in mesh and plaster strength should be but are not included in these simple estimates. These walls had a wooden post-and-beam frame to take load as well. These strengths are compared with the average wall strengths (Table 17).

In an ASCE paper entitled “Allowable Shears for Plastered Straw-Bale Walls,” allowable shears were estimated for walls with different plaster and wire mesh combinations (Aschheim, et al., 2014). The estimation took an unadjusted shear strength based on the six full-scale wall tests done by Ash, et al. (2003), and then multiplied this strength by three factors based on plaster mix, wire mesh type, and safety. For cement-lime plastered walls, the suggested Allowable Shear (V_{all}) with 17-gauge woven wire mesh was 380 lb/ft and the allowable shear with 14-gauge 2”x2” welded wire mesh was 520 lb/ft. When multiplying these values by the 4’ length of the wall, the respective strengths of the woven and welded wire walls are 1,520 lbs and 2,080 lbs (Table 17).

Table 17: Expected shear strength of walls based on wire mesh type (Aschheim, et al.) and on the Ash, et al. paper, where a post-and-beam system was used.

Wire Mesh Type	Expected from Ash, et al. (2003)	Expected from Aschheim, et al. (2014)	Actual Lateral Strength
Woven	1.6 kips	1.5 kips	1.7 kips
Welded	4.6 kips	2.1 kips	2.3 kips

Based on the simple geometric relationship (data from Ash, et al.), the strength of the Woven-18 wall was estimated well. In contrast, the strength of the Welded-18 wall was only 50% of its expected value.

Using the values from the ASCE paper (Aschheim, et al.), both walls were estimated well, within 12% for the Woven-18 wall and within 9% for the Welded-18 wall. The ASCE paper also includes a safety factor of 8.2 - 9.2 to calculate Shear Strength (V_n) which results in shears of 13.9 kips for four-foot woven wire walls and 17.0 kips for four-foot welded wire walls. The paper allows these values to be decreased based on the aspect ratio (1/2 for these walls), which means the shear strengths become 6.9 kips for woven wire walls and 8.5 kips for welded wire walls. Given this reduction, the actual strengths reached on average 26% of this V_n shear value, a significant shortfall.

7.0 Discussion

7.1 Wire Series

The welded wire mesh wall reached a larger peak load than the woven wire mesh wall, as expected. The Woven-18 wall reached an average of 73% of the Welded-18 wall’s peak load capacity, conforming to the ratio of strengths discussed in the 2014 ASCE paper (Aschheim et al.); they determined the difference in wire mesh reinforcement per unit length (area per length) was 73% and incorporated this value into their shear strength predictions. As such, these tests provide supporting evidence for the ASCE paper’s conclusion regarding welded versus woven wire strengths, and the tests also indicate that the results are internally consistent. That said, the ratio of wire reinforcement determined from this present series was 59.4% which did not predict the wall strength ratio. Perhaps the measured distance between the woven wires could be used instead of the nominal value if this phenomenon were to be explored further.

The Woven-18 wall maintained a fairly consistent strength after failure, proving to have a more consistent ductile response than the Welded-18 wall (Table 15). Although the ductile response occurred at a relatively low strength, the ductility would provide robustness to intense seismic shaking. The initial strength exhibited by the Welded-18 wall would be useful for resisting (without significant damage) the more frequent, less intense shaking of wind loads.

Both walls lost 50% capacity, on average, after reaching peak lateral load at about 1.5% drift. The Welded-18 and Welded-14 walls had almost the exact same lateral loads at 1% drift, whereas the Woven-18 had a markedly decreased initial lateral force resisted at 1% drift (Table 15). The Welded-14 wall may have had a high peak load due to the presence of the same 14-gauge welded wire mesh that was in the strongest wall, Welded-18.

Table 15: Lateral displacement and lateral force experienced by each wall at 1%, 3%, and final drifts. Height was taken as the height of the plastered, encompassing the stapled region at the wooden cap and base.

		1% Drift		**3% Drift**		**Final Drift**	
Wall	**Height (in)**	Lateral Displacement, x (in)	Lateral Force, Fx (lbs)	Lateral Displacement, x (in)	Lateral Force, Fx (lbs)	Lateral Displacement, x (in)	Lateral Force, Fx (lbs)
Welded-18	106	1.06	1654	3.18	984	3.57	816
Woven-18	106	1.07	1455	3.19	976	5.77	1242
Welded-14	100	1.04	1657	3.00	791	4.88	815

Due to their 2:1 aspect ratio, at 4’ long and 8’ tall, the Welded-18 and Woven-18 walls failed mainly in flexure. They suffered the most damage at the bottom corners, where they experienced the greatest compression and tension. During the lateral test, both the Welded-18 and Woven-18 walls also exhibited a line of damaged plaster laterally crossing approximately 10” above the bottom of the wall. All of the walls exhibited some degree of delamination between the plaster layers at the base of the wall, but the Welded-18 wall exhibited the most severe delamination (Figures 11-13). The welded wire, however, had numerous wire failures near the bottom, which developed beginning after the 3% drift cycle and continued as the test continued (Figures 17-20). The woven wire only had one wire failure, potentially because the woven wire's hexagonal shape allowed it to lengthen and narrow under tension.

7.2 Slenderness Series

The Welded-14 wall achieved lower peak loads than the thicker walls, both in the lateral seismic test and the vertical compressive test. Both the irregularities of the bales themselves and the slenderness of the wall may have contributed to the wall's weakness relative to the Welded-18 wall. This initial distortion of the reinforced plaster changed the local loading from a pure tensile or compressive case (under flexure) to a combination of axial force and flexure. Additionally, based on the air-cured cubes, the Welded-14 wall had much weaker plaster than the Welded-18 and Woven-18 walls, which reduced its compressive plaster strength, and thus its overall flexural strength.

Due to the irregularities in the Welded-14 wall, the determination of the validity of the existing slenderness limits will depend heavily on the performance of the Welded-9 Wall. Because it was not tested by the time this thesis was submitted, its results could not be incorporated in this document for comparison to the other walls. At time of submittal, the quality of construction was visibly superior to the Welded-14 wall, which bodes well for both the lateral and vertical tests.

7.3 Vertical Series

The two walls with vertical test results could adequately support a typical roof load. Even for the irregular Welded-14 wall, typical roof loads could be supported, at least long enough for building occupants to evacuate. This typical load was calculated as such: using a combined dead and live roof load of 30 psf and a maximum span of 20 feet (tributary length of 10 feet), the linear load on the roof is 300 pounds per foot. This math results in a total vertical load of 1200 lbs for a four-foot wall, and both walls were sufficient to carry that load as they were compressed more than 6 inches.

7.4 Complexities in Understanding Strawbale Wall Performance

Strawbale construction utilizes a number of materials, each prone to some variability. The observed behavior is also a function of many variables, which define locations where damage takes place. Furthermore, construction practices and workmanship of even code-compliant walls will vary with the tradespeople doing the work. Interpretation of test results thus requires some care.

Standards followed within our lab include use of the same recommended "workhorse" connection (Heckert and Looney, 2014) and plaster mix design (Avon and Swartchick, 2014) for all four walls. The walls of this test series were built using the same materials: cement-lime plaster (hand-applied), steel welded or woven wire mesh, wooden bases and caps, and rice straw bales. Their elevation dimensions were nominally the same for the Welded-18 and Woven-18 walls: 4' long by 8' tall. The Welded-14 wall was 4' long and 7'-6" tall, and the Welded-9 wall was 4' long and 7'-4" tall. The tests were performed in the same hydraulically driven 3DOF Structural Test Frame, with the same displacement loading protocol.

Nevertheless, some irregularity is inherent. There is some minor variability in the 18-inch bales (and significantly more in the 14-inch bales), and apparently significant variability in the strengths of the plasters. Further, different people assisted with the construction of the walls, potentially leading to some variability in the plasters applied to the walls. The time between construction, plastering, and testing varied for each wall.

Due to the scale of these walls, only one specimen is made to examine a particular detail or variable. Thus, the potential influence of other random variables on performance is not readily discerned, since the system behavior may be a function of more than just the explicit parameter that is varied. If it were possible to make two or even three of each wall, factors relating to individual wall irregularities could be diminished and the results could be more reliably interpreted.

8.0 Sources of Error

8.1 Incongruous bales

The farmer who supplied the bales for the Welded-14 wall had to hand-tie them, since his machinery was only calibrated for 18” thick bales. This led to numerous imperfections in the bales, particularly regarding shape and length, and made them difficult to stack. The performance of the Welded-14 wall cannot be considered representative of walls made with regular 14-inch bales. Even machine-baled straw bales have minor imperfections and inconsistencies.

8.2 Test machine malfunctions

The Hydraulic Frame experienced a few issues throughout the course of the year, including control failures and hydraulic fluid leaks. These issues delayed the schedule at times and caused damage to one wall that was subsequently repaired, without obvious detriment to the wall performance.

8.3 Imperfect construction

Though the provided details and mix designs were adhered to, there is always some potential for imperfections or unintentional variances between the quality of construction to occur.

Straw bale construction is variable by nature, and no two walls are exactly the same because no two bales are exactly the same, and no two plaster batches are exactly the same, even if the builder adheres to the same mix design.

8.4 Inconsistent time between construction & testing

The welded wire and woven wire walls were built and plastered by August 2016, while the Welded-14 wall was completed by January 2017. As such, the first two walls had ample time for their plaster to cure, while the Welded-14 wall only had two months to cure. Effects of this on strength are thought to be minor.

9.0 Recommendations for Future Work

As a result of this research, there are several recommendations for continued research on examining strawbale shear wall construction:

1. Complete the Welded-9 Wall test to determine recommendations for slenderness limit.
2. Build and test a second Welded-18 wall to understand if the unexpected low strength observed for this wall was representative.
3. Monitor wire strength variability for a given wire type for different manufacturer's batches.
4. Investigate plaster strength variability for a given mix, comparing samples cured in air and those cured in a lime-saturated water bath.
5. Compare the behavior of a 4' wall with IRC details and welded wire mesh to one with Heckert details.
6. Compare the behavior of woven and welded wire mesh for shear-dominated walls (e.g. 8 feet long and 8 feet tall).

10.0 Summary and Conclusions

The results of this test series provided further confirmation that welded wire offers more strength than woven wire when used in the construction of strawbale walls. The four-foot walls with welded wire mesh fell significantly short of predicted strengths. Walls with a higher slenderness ratio are more prone to buckling, and in combination with an aspect ratio greater than 1:1, such walls should be heavily analyzed regarding their flexural strength. Further testing, especially of the slender limit wall, will be required to analyze these results in more detail.

The vertical tests confirmed that strawbale walls can withstand an estimated typical roof dead load of 300 plf after an earthquake event, even when constructed using non-compliant 14" bales. This implies that strawbale buildings offer occupants enough time to safely exit the structure after a large earthquake.

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Appendix A: Cost Estimate Calculation

Given the material prices as purchased in bulk from distributors, each wall cost approximately $170 to build (see Table A-1). This number applies to a four-foot wall, and it does not include labor, which, for inexperienced builders, reached upwards of 50 hours per wall.

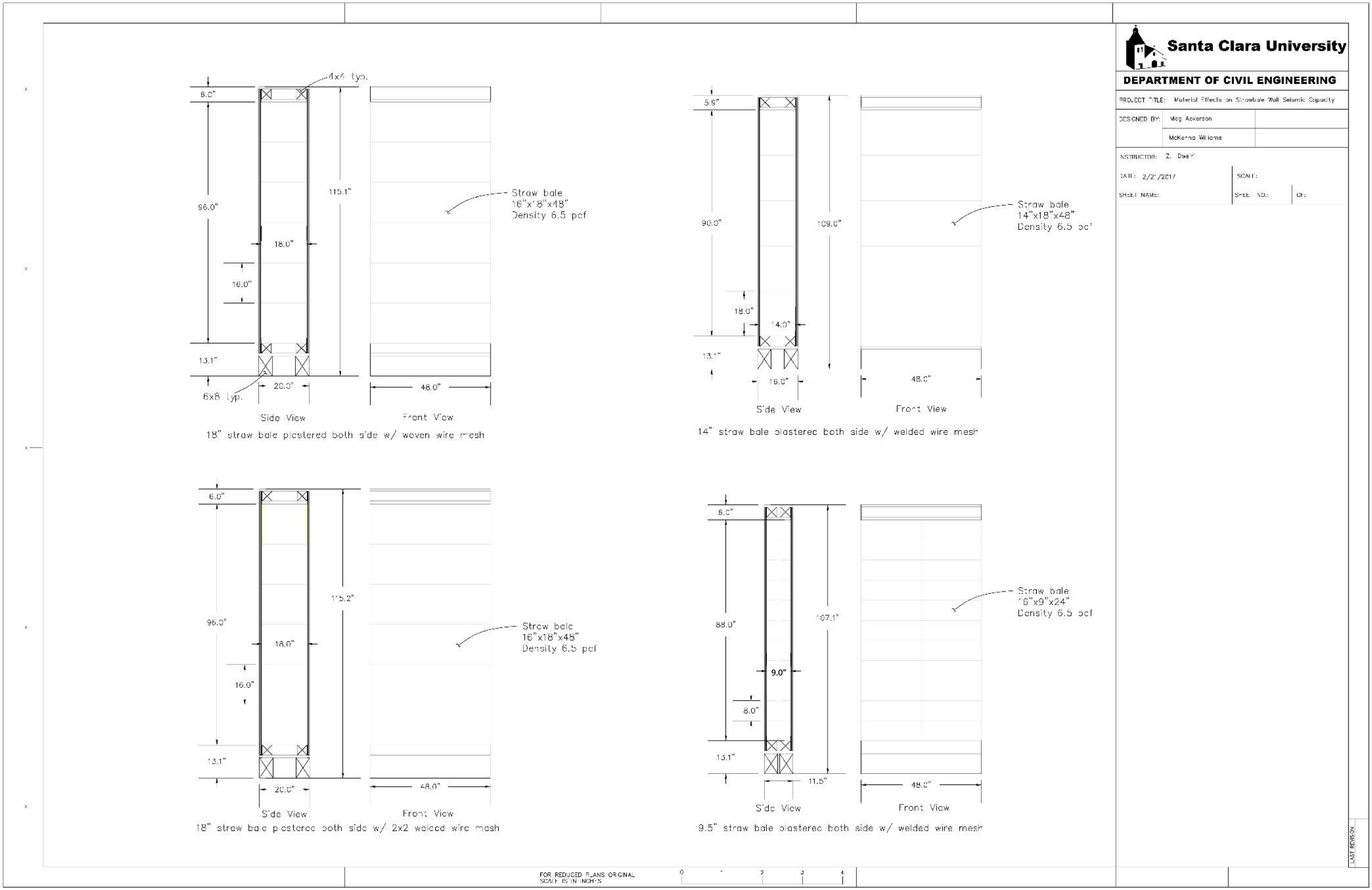
Table A-1: Cost estimation for a four-foot strawbale wall constructed in the SCU civil lab.

Item	**Quantity**	**Unit Price**	**Total Price**
Straw Bale	5	$10.00/bale	$50.00
Plaster Batch	4	$5.00/batch	$20.00
Timber	2	$35.00/box beam	$70.00
Wire	16’	$1.75/ft	$28.00
TOTAL		**$42.00/ft**	**$168.00**

Summers and Zaleski determined their total cost to be $7,000 for four walls of varying length (“Straw Bale Seismic Design Capacities III,” 2014). When dividing the total by the walls in feet, their unit cost of a strawbale wall is $225/foot. Their high value can be attributed to the heavy “strong” detailing used in each of the walls; they cited $1000 as the cost of steel plates at the base of each of their walls. Their walls were also constructed using post-and-beam methods, which require more lumber than was used in this series’ walls.

Appendix B: Detailed Design Drawings and Standard Details

Please see drawings on next page.


4x4 typ.
6.0"
96.0"
115.1"
18.0"
16.0"
13.1"
6x8 typ.
20.0"
48.0"
Straw bale
16"x18"x48"
Density 6.5 pcf
Side View
Front View
18" straw bale plastered both side w/ woven wire mesh
5.9"
90.0"
109.0"
18.0"
14.0"
13.1"
16.0"
48.0"
Straw bale
14"x18"x48"
Density 6.5 pcf
Side View
Front View
14" straw bale plastered both side w/ welded wire mesh
6.0"
96.0"
115.2"
18.0"
16.0"
13.1"
20.0"
48.0"
Straw bale
16"x18"x48"
Density 6.5 pcf
Side View
Front View
18" straw bale plastered both side w/ 2x2 welded wire mesh
6.0"
88.0"
107.1"
9.0"
8.0"
13.1"
11.5"
48.0"
Straw bale
16"x9"x24"
Density 6.5 pcf
Side View
Front View
9.5" straw bale plastered both side w/ welded wire mesh
Santa Clara University
DEPARTMENT OF CIVIL ENGINEERING
PROJECT TITLE: Material Effects on Strawbale Wall Seismic Capacity
DESIGNED BY: Meg Ackerson
McKenna Williams
INSTRUCTOR: Z. Dweiri
DATE: 2/21/2017
SCALE:
SHEET NAME:
SHEET NO.:
OF:
LAST REVISION
FOR REDUCED PLANS ORIGINAL SCALE IS IN INCHES
0 1 2 3 4

Appendix C: Welded-18 Expected Shear Calculation

Please see spreadsheet on next page.

Plastered Straw Bale Panel Proportioning — User Input

4-Jun-17 — Spreadsheet Output

Meg changed it

4' STANDARD WALL (Welded-18)

Materials

Bales		
Unit Weight of Straw Bales (@ equil. MC)	8.5	pcf
Bale width (tributary to plaster skin)	9.00	in.

2x2 mesh 14 ga				Stucco Lath: 2x2 mesh, 14 gauge	2x2 mesh, 16 gauge	W1.4 6"	2x4 mesh 14 ga
fy (mean fy/1.15)	55	ksi		60.0	66	60.0	50
fye (nominal * 1.15) = (ult / 1.1)	63.0	ksi	ult/1.1		75.9	69	57.5
Spacing of vertical wires (plan dimension)	2.0	in		2.0	2.0	6.0	4.0
Spacing of horizontal wires (vertical dimension)	2.0	in		2.0	2.0	6.0	2.0
Wire diameter	0.0750	in		0.0800	0.0625	0.1350	0.0800
Area	0.00442	in2					
Horizontal reinforcing ratio	0.221%						
Area of vertical wires/unit length horizontally	0.00221	in2/in					
Vertical reinforcing ratio	0.221%		neglects laps				
Mesh roll width	48.0	in		72.0	28.0	72.0	72.0
Lap width	0.0	in		6.0	3.0	9.0	6.0
Effective vertical reinforcing ratio (smeared)	0.221%		without laps				
rho.fye	0.139	ksi	use vertical reinforcing ratio				

cement-lime (stucco)			
Cube strength (code min)	1000	psi	cubes cured in lime saturated water bath
Measured cylinder strength	1840	psi	cube/1.25 to get cylinder (NOT times 1.3 to get expected due to hydration)
Unit weight of plaster	138	pcf	
Plaster Thickness (of single skin)	1.00	in	specified as 7/8

For reference, from IBC and IRC proposal development	A	B	C	D	E
	clay	soil cement	lime	cement-lime	cement
unit wt	110	130	130	138	142
min cube	100	1000	600	1000	1400

16-gauge staples (7/16" crown x 1-3/4" leg)			
Strength from Parker (= 1.2 kN)	270	lbs	monotonic wire anchorage tests
Shear strength in plastic hinge region	97	lbs	cyclic wall tests, based on number of wires ignoring laps
If protected from yielding/inelasticity			
take Vn at 2/3 of 270	180	lbs/staple	
If part of plastic hinge region			
base on 97	100	lbs/staple	

mesh in Illinois tests, 8 ft	quantity each side
uniform mesh wires	49
laps (3 at 6 in)	12
band at base (not stapled)	49
total at base	110
recalibrate ignoring laps	98

Wall Segment (capacity of one reinforced plastered face)

Wall Segment Dimensions			
Height, hi	8.21	ft	above 10" mesh band
Plan Length, Li	4.00	ft	

Dead Load at Top of Wall Segment (tributary to single skin)			
Height of plastered wall above segment	7.96	ft	
Tributary length of straw above segment	4.00	ft	assumes jambs carry horizontal wall segment weight
Weight of straw (tributary to plaster skin)	203	lbs	
Weight of plaster	366	lbs	
Superimposed dead load	0	lbs	zero for post and beam
Axial force at top of segment	569	lbs	

Dead Load at Base of Wall Segment (tributary to single skin)		
Weight of straw within segment	209	lbs
Weight of plaster within segment	378	lbs
Weight of wall segment	587	lbs
Axial force at base of segment	1156	lbs

Shear Strength of Wall Segment (plastered one side)			
Expected, based on expected material strengths			
Vc,e	3.29	kips	
Vs,e	5.34	kips	
Vn,e	8.6	kips	

Shear Required to Cause Plastic Hinging (of single skin)			
Beta1	0.85		
Mp at top of wall segment (reinforced plaster one side)			
c	0.41	ft	
Mp, at top of wall segment	158.5	k-in	zero actually
Mp at base of wall segment (reinforced plaster one side)			
c	0.44	ft	
Mp, at base of wall segment	170.0	k-in	
Vmp (shear associated with Mp)	1.6	k	cantilever value

Design Failure Mode			
Vmp/Vn	0.19		>1 = shear, except shear strengths may be a lot higher than calculated
Failure mode (assumed)	Flexure Governs		
Shear corresponding to failure	1.6	kips	

Stapled Boundary Shear Strength			
If shear or plastic hinging at boundary, provide	0.3	layers of mesh/skin	heavy mesh boundaries for both shear and plastic hinging? Maybe just for plastic hinging...
Then V at boundary equals	1.61	kips	
If boundary protected from inelastic action:			
Length of boundary	4	ft	
Provide	0.2	layers of mesh/skin	
Then V at boundary equals	2	kips	

Performance Indices			
Plan Length, Li	4.00	ft	
Height, hi	8.21	ft	
Height of plastered wall above segment	7.96	ft	
Tributary length of straw above segment	4.00	ft	
Vn, (shear associated with shear failure)	8.638	kips	
Vmp (shear associated with Mp)	1.6	kips	
Vmp/Vn	0.19		
Failure mode (assumed)	Flexure Governs		
Shear corresponding to failure	1.61	kips	
Within wall segment: Vmp/Vn	0.19		
If boundary yielding: Vboundary/Vsegment	1.00		
Number of boundary mesh layers	0.34		
If boundary protected: Vboundary/Vsegment	1.00		
Number of boundary mesh layers	0.19		

Wall Segment (capacity of two reinforced plastered faces)			
Vmp (shear associated with Mp)	3.218	kips	(Vmp for one plaster face times 2)
Vn (shear associated with shear failure)	17.27639515	kips	(Vn for one plaster face times 2)

pier hinging, Vmp/Vn around 0.7 or 0.8

pier shear failure, Vmp/Vn high

plastic hinging in pier at boundary

shear failure in pier at boundary

Appendix D: Woven-18 Expected Shear Calculation

Please see spreadsheet on next page.

Plastered Straw Bale Panel Proportioning		User Input
	4-Jun-17	Spreadsheet Output
		Meg changed it
4' WOVEN WIRE WALL (Woven-18)		

Materials

Bales									
Unit Weight of Straw Bales (@ equil. MC)	8.5	pcf							
Bale width (tributary to plaster skin)	9.00	in.							
						Stucco Lath			
2x2 mesh 14 ga				2x2 mesh, 14 gauge	2x2 mesh, 16 gauge	W1.4 6"	2x4 mesh 14	Woven wire mesh, 17 gauge
fy (mean fy/1.15)	61	ksi		60.0	66	60.0	50	50
fye (nominal * 1.15) = (ult / 1.1)	70.4	ksi			75.9	69	57.5	57.5
Spacing of vertical wires (plan dimension)	3.0	in	<nominal?	2.0	2.0	6.0	4.0	3.2
Spacing of horizontal wires (vertical dimension)	1.5	in	for two wires	2.0	2.0	6.0	2.0	1.5
Wire diameter	0.0500	in		0.0800	0.0625	0.1350	0.0800	0.0490
Area	0.00196	in2						
Horizontal reinforcing ratio	0.262%		for two wires every 1.5"					
Area of vertical wires/unit length horizontally	0.00065	in2/in						
Vertical reinforcing ratio	0.065%		neglects laps					
Mesh roll width	48.0	in		72.0	28.0	72.0	72.0	
Lap width	0.0	in		6.0	3.0	9.0	6.0	
Effective vertical reinforcing ratio (smeared)	0.065%		no laps					
rho.fye	0.046	ksi						

cement-lime (stucco)			
Cube strength (code min)	1000	psi	cubes cured in lime saturated water bath
Measured cylinder strength	1840	psi	cube/1.25 to get cylinder (NOT times 1.3 to get expected due to hydration)
Unit weight of plaster	138	pcf	
Plaster Thickness (of single skin)	1.00	in	specified as 7/8

For reference, from IBC and IRC proposal development

	A	B	C	D	E
	clay	soil cement	lime	cement-lime	cement
unit wt	110	130	130	138	142
min cube	100	1000	600	1000	1400

16-gauge staples (7/16" crown x 1-3/4" leg)			
Strength from Parker (= 1.2 kN)	270	lbs	monotonic wire anchorage tests
Shear strength in plastic hinge region	97	lbs	cyclic wall tests, based on number of wires ignoring laps
If protected from yielding/inelasticity			
take Vn at 2/3 of 270	180	lbs/staple	
If part of plastic hinge region			
base on 97	100	lbs/staple	

mesh in Illinois tests, 8 ft	quantity each side
uniform mesh wires	49
laps (3 at 6 in)	12
band at base (not stapled)	49
total at base	110
recalibrate ignoring laps	98

Wall Segment (capacity of one reinforced plastered face)

Wall Segment Dimensions			
Height, hi	8.21	ft	above 10" mesh band
Plan Length, Li	4.00	ft	

Dead Load at Top of Wall Segment (tributary to single skin)			
Height of plastered wall above segment	7.96	ft	
Tributary length of straw above segment	4.00	ft	assumes jambs carry horizontal wall segment weight
Weight of straw (tributary to plaster skin)	203	lbs	
Weight of plaster	366	lbs	
Superimposed dead load	0	lbs	zero for post and beam
Axial force at top of segment	569	lbs	

Dead Load at Base of Wall Segment (tributary to single skin)		
Weight of straw within segment	209	lbs
Weight of plaster within segment	378	lbs
Weight of wall segment	587	lbs
Axial force at base of segment	1156	lbs

Shear Strength of Wall Segment (plastered one side)			
Expected, based on expected material strengths			
Vc,e	3.29	kips	
Vs,e	1.77	kips	
Vn,e	5.1	kips	

Shear Required to Cause Plastic Hinging (of single skin)			
Beta1	0.85		
Mp at top of wall segment (reinforced plaster one side)			
c	0.17	ft	
Mp, at top of wall segment	64.3	k-in	zero actually
Mp at base of wall segment (reinforced plaster one side)			
c	0.20	ft	
Mp, at base of wall segment	77.3	k-in	
Vmp (shear associated with Mp)	0.8	k	cantilever value

Design Failure Mode			
Vmp/Vn	0.15		>1 = shear, except shear strengths may be a lot higher than calculated
Failure mode (assumed)	Flexure Governs		
Shear corresponding to failure	0.8	kips	

Stapled Boundary Shear Strength			
If shear or plastic hinging at boundary, provide	0.2	layers of mesh/skin	heavy mesh boundaries for both shear and plastic hinging? Maybe just for plastic hinging...
Then V at boundary equals	0.78	kips	
If boundary protected from inelastic action:			
Length of boundary	4	ft	
Provide	0.1	layers of mesh/skin	
Then V at boundary equals	1	kips	

Performance Indices		
Plan Length, Li	4.00	ft
Height, hi	8.21	ft
Height of plastered wall above segment	0.00	ft
Tributary length of straw above segment	0.00	ft
Vn, (shear associated with shear failure)	5.063	kips
Vmp (shear associated with Mp)	0.8	kips
Vmp/Vn	0.15	
Failure mode (assumed)	Flexure Governs	
Shear corresponding to failure	0.78	kips
Within wall segment: Vmp/Vn	0.15	
If boundary yielding: Vboundary/Vsegment	1.00	
Number of boundary mesh layers	0.16	
If boundary protected: Vboundary/Vsegment	1.00	
Number of boundary mesh layers	0.09	

Wall Segment (capacity of two reinforced plastered faces)			
Vmp (shear associated with Mp)	1.569	kips	(Vmp for one plaster face times 2)
Vn (shear associated with shear failure)	10.12556511	kips	(Vn for one plaster face times 2)

pier hinging, Vmp/Vn around 0.7 or 0.8
pier shear failure, Vmp/Vn high
plastic hinging in pier at boundary
shear failure in pier at boundary

Appendix E: Welded-14 Expected Shear Calculation

Please see spreadsheet on next page.

Plastered Straw Bale Panel Proportioning	User Input	
4-Jun-17	Spreadsheet Output	
	Meg changed it	

4' THIN 14" WALL (Welded-14)

Materials

Bales			
Unit Weight of Straw Bales (@ equil. MC)	8.5	pcf	
Bale width (tributary to plaster skin)	7.00	in.	

2x2 mesh 14 ga				2x2 mesh, 14 gauge	Stucco Lath 2x2 mesh, 16 gauge	W1.4 6"	2x4 mesh 14 ga	Woven wire mesh, 17 gauge
fy (mean fy/1.15)	55	ksi		60.0	66	60.0	50	50
fye (nominal * 1.15) = (ult / 1.1)	63.0	ksi			75.9	69	57.5	57.5
Spacing of vertical wires (plan dimension)	2.0	in		2.0	2.0	6.0	4.0	3.2
Spacing of horizontal wires (vertical dimension)	2.0	in		2.0	2.0	6.0	2.0	1.5
Wire diameter	0.0750	in		0.0800	0.0625	0.1350	0.0800	0.0490
Area	0.00442	in2						
Horizontal reinforcing ratio	0.221%							
Area of vertical wires/unit length horizontally	0.00221	in2/in						
Vertical reinforcing ratio	0.221%		neglects laps					
Mesh roll width	48.0	in		72.0	28.0	72.0	72.0	
Lap width	0.0	in		6.0	3.0	9.0	6.0	
Effective vertical reinforcing ratio (smeared)	0.221%		no laps					
rho.fye	0.139	ksi						

cement-lime (stucco)			
Cube strength (code min)	1000	psi	cubes cured in lime saturated water bath
Measured cylinder strength	465	psi	cube/1.25 to get cylinder (NOT times 1.3 to get expected due to hydration)
Unit weight of plaster	138	pcf	
Plaster Thickness (of single skin)	1.00	in	specified as 7/8

For reference, from IBC and IRC proposal development					
	A	B	C	D	E
	clay	soil cement	lime	cement-lime	cement
unit wt	110	130	130	138	142
min cube	100	1000	600	1000	1400

16-gauge staples (7/16" crown x 1-3/4" leg)			
Strength from Parker (= 1.2 kN)	270	lbs	monotonic wire anchorage tests
Shear strength in plastic hinge region	97	lbs	cyclic wall tests, based on number of wires ignoring laps
If protected from yielding/inelasticity			
take Vn at 2/3 of 270	180	lbs/staple	
If part of plastic hinge region			
base on 97	100	lbs/staple	

mesh in Illinois tests, 8 ft	quantity each side
uniform mesh wires	49
laps (3 at 6 in)	12
band at base (not stapled)	49
total at base	110
recalibrate ignoring laps	98

Wall Segment (capacity of one reinforced plastered face)

Wall Segment Dimensions			
Height, hi	7.71	ft	above 10" mesh band
Plan Length, Li	4.00	ft	

Dead Load at Top of Wall Segment (tributary to single skin)			
Height of plastered wall above segment	7.46	ft	
Tributary length of straw above segment	4.00	ft	assumes jambs carry horizontal wall segment weight
Weight of straw (tributary to plaster skin)	148	lbs	
Weight of plaster	343	lbs	
Superimposed dead load	0	lbs	zero for post and beam
Axial force at top of segment	491	lbs	

Dead Load at Base of Wall Segment (tributary to single skin)		
Weight of straw within segment	153	lbs
Weight of plaster within segment	355	lbs
Weight of wall segment	507	lbs
Axial force at base of segment	998	lbs

Shear Strength of Wall Segment (plastered one side)		
Expected, based on expected material strengths		
Vc,e	1.78	kips
Vs,e	5.34	kips
Vn,e	7.1	kips

Shear Required to Cause Plastic Hinging (of single skin)

Beta1	0.85		
Mp at top of wall segment (reinforced plaster one side)			
c	1.26	ft	
Mp, at top of wall segment	123.7	k-in	zero actually
Mp at base of wall segment (reinforced plaster one side)			
c	1.35	ft	
Mp, at base of wall segment	128.8	k-in	
Vmp (shear associated with Mp)	1.4	k	cantilever value

Design Failure Mode			
Vmp/Vn	0.20		>1 = shear, except shear strengths may be a lot higher than calculated
Failure mode (assumed)	Flexure Governs		
Shear corresponding to failure	1.4	kips	

Stapled Boundary Shear Strength			
If shear or plastic hinging at boundary, provide	0.3	layers of mesh/skin	heavy mesh boundaries for both shear and plastic hinging? Maybe just for plastic hinging...
Then V at boundary equals	1.39	kips	
If boundary protected from inelastic action:			
Length of boundary	4	ft	
Provide	0.2	layers of mesh/skin	
Then V at boundary equals	1	kips	

Performance Indices		
Plan Length, Li	4.00	ft
Height, hi	7.71	ft
Height of plastered wall above segment	0.00	ft
Tributary length of straw above segment	0.00	ft
Vn, (shear associated with shear failure)	7.120	kips
Vmp (shear associated with Mp)	1.4	kips
Vmp/Vn	0.20	
Failure mode (assumed)	Flexure Governs	
Shear corresponding to failure	1	kips
Within wall segment: Vmp/Vn	0.20	
If boundary yielding: Vboundary/Vsegment	1.00	
Number of boundary mesh layers	0.29	
If boundary protected: Vboundary/Vsegment	1.00	
Number of boundary mesh layers	0.16	

Wall Segment (capacity of two reinforced plastered faces)			
Vmp (shear associated with Mp)	2.785	kips	(Vmp for one plaster face times 2)
Vn (shear failure)	14.24064876	kips	(Vn for one plaster face times 2)

pier hinging, Vmp/Vn around 0.7 or 0.8
pier shear failure, Vmp/Vn high
plastic hinging in pier at boundary
shear failure in pier at boundary

Appendix F: Welded-9 Expected Shear Calculation

Please see spreadsheet on next page.

Plastered Straw Bale Panel Proportioning	User Input	
4-Jun-17	Spreadsheet Output	
	Meg changed it	
4' SLENDER 9" WALL (Welded-9)		

Materials

						Stucco Lath		
Bales								
Unit Weight of Straw Bales (@ equil. MC)	8.5	pcf						
Bale width (tributary to plaster skin)	4.50	in.						
2x2 mesh 14 ga				2x2 mesh, 14 gauge	2x2 mesh, 16 gauge	W1.4 6"	2x4 mesh 14	Woven wire mesh, 17 gauge
fy (mean fy/1.15)	55	ksi		60.0	66	60.0	50	50
fye (nominal * 1.15) = (ult/ 1.1)	63.0	ksi	ult/1.1		75.9	69	57.5	57.5
Spacing of vertical wires (plan dimension)	2.0	in		2.0	2.0	6.0	4.0	3.2
Spacing of horizontal wires (vertical dimension)	2.0	in		2.0	2.0	6.0	2.0	1.5
Wire diameter	0.0750	in		0.0800	0.0625	0.1350	0.0800	0.0490
Area	0.00442	in2						
Horizontal reinforcing ratio	0.221%							
Area of vertical wires/unit length horizontally	0.00221	in2/in						
Vertical reinforcing ratio	0.221%		neglects laps					
Mesh roll width	48.0	in		72.0	28.0	72.0	72.0	
Lap width	0.0	in		6.0	3.0	9.0	6.0	
Effective vertical reinforcing ratio (smeared)	0.221%		no laps					
rho.fye	0.139	ksi						

cement-lime (stucco)			
Cube strength (code min)	1000	psi	cubes cured in lime saturated water bath
Measured cylinder strength	642	psi	cube/1.25 to get cylinder (NOT times 1.3 to get expected due to hydration)
Unit weight of plaster	138	pcf	
Plaster Thickness (of single skin)	1.00	in	specified as 7/8

For reference, from IBC and IRC proposal development					
	A	B	C	D	E
	clay	soil cement	lime	cement-lime	cement
unit wt	110	130	130	138	142
min cube	100	1000	600	1000	1400

16-gauge staples (7/16" crown x 1-3/4" leg)			
Strength from Parker (= 1.2 kN)	270	lbs	monotonic wire anchorage tests
Shear strength in plastic hinge region	97	lbs	cyclic wall tests, based on number of wires ignoring laps
If protected from yielding/inelasticity			
take Vn at 2/3 of 270	180	lbs/staple	
If part of plastic hinge region			
base on 97	100	lbs/staple	

mesh in Illinois tests, 8 ft	quantity each side
uniform mesh wires	49
laps (3 at 6 in)	12
band at base (not stapled)	49
total at base	110
recalibrate ignoring laps	98

Wall Segment (capacity of one reinforced plastered face)

Wall Segment Dimensions			
Height, hi	8.21	ft	above 10" mesh band (85.5" measured height)
Plan Length, Li	4.00	ft	
Dead Load at Top of Wall Segment (tributary to single skin)			
Height of plastered wall above segment	7.96	ft	
Tributary length of straw above segment	4.00	ft	assumes jambs carry horizontal wall segment weight
Weight of straw (tributary to plaster skin)	101	lbs	
Weight of plaster	366	lbs	
Superimposed dead load	0	lbs	zero for post and beam
Axial force at top of segment	468	lbs	
Dead Load at Base of Wall Segment (tributary to single skin)			
Weight of straw within segment	105	lbs	
Weight of plaster within segment	378	lbs	
Weight of wall segment	482	lbs	
Axial force at base of segment	950	lbs	

Shear Strength of Wall Segment (plastered one side)			
Expected, based on expected material strengths			
Vc,e	1.95	kips	
Vs,e	5.34	kips	
Vn,e	7.3	kips	

Shear Required to Cause Plastic Hinging (of single skin)			
Beta1	0.85		
Mp at top of wall segment (reinforced plaster one side)			
c	0.99	ft	
Mp, at top of wall segment	134.1	k-in	zero actually
Mp at base of wall segment (reinforced plaster one side)			
c	1.05	ft	
Mp, at base of wall segment	140.4	k-in	
Vmp (shear associated with Mp)	1.4	k	cantilever value

Design Failure Mode			
Vmp/Vn	0.20		>1 = shear, except shear strengths may be a lot higher than calculated
Failure mode (assumed)	Flexure Governs		
Shear corresponding to failure	1.4	kips	

Stapled Boundary Shear Strength			
If shear or plastic hinging at boundary, provide	0.3	layers of mesh/skin	heavy mesh boundaries for both shear and plastic hinging? Maybe just for plastic hinging...
Then V at boundary equals	1.43	kips	
If boundary protected from inelastic action:			
Length of boundary	4	ft	
Provide	0.2	layers of mesh/skin	
Then V at boundary equals	1	kips	

Performance Indices			
Plan Length, Li	4.00	ft	
Height, hi	8.21	ft	
Height of plastered wall above segment	0.00	ft	
Tributary length of straw above segment	0.00	ft	
Vn, (shear associated with shear failure)	7.290	kips	
Vmp (shear associated with Mp)	1.4	kips	
Vmp/Vn	0.20		
Failure mode (assumed)	Flexure Governs		
Shear corresponding to failure	1.4	kips	
Within wall segment: Vmp/Vn	0.20		
If boundary yielding: Vboundary/Vsegment	1.00		
Number of boundary mesh layers	0.30		
If boundary protected: Vboundary/Vsegment	1.00		
Number of boundary mesh layers	0.17		

Wall Segment (capacity of two reinforced plastered faces)			
Vmp (shear associated with Mp)	2.851	kips	(Vmp for one plaster face times 2)
Vn (shear associated with shear failure)	14.58042873	kips	(Vn for one plaster face times 2)

pier hinging, Vmp/Vn around 0.7 or 0.8
pier shear failure, Vmp/Vn high
plastic hinging in pier at boundary
shear failure in pier at boundary