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REVERSED CYCLIC IN-PLANE TESTS OF LOAD-BEARING PLASTERED STRAW BALE WALLS

C. Ash[1], M. Aschheim[2], and D. Mar[3], and B. King

SUMMARY

The results of an experimental study on the lateral load resisting characteristics of 6 full-scale straw bale wall assemblies are presented. Walls having earth plaster and cement stucco skins and varied levels of detailing were subjected to reversed cyclic lateral loads. Preliminary recommendations are made for wall detailing, allowable design strengths, and R factors that may be used for seismic design, for walls that are nominally 8 ft. high by 8 ft long.

INTRODUCTION

Low-rise buildings have been constructed using straw bale walls for more than a century (King [1]). While decades of experience have demonstrated that plastered straw bale construction can perform well, the variety of materials, details, and construction procedures used in the past is quite varied. The lack of standardization has encouraged innovation, facilitating improvements in construction or to overcome problems that occasionally have been encountered. However, the continued evolution of this sustainable (green) building technology into the mainstream requires the insight and guidance of experts aided with solid research and engineering data. A comprehensive, multi-faceted research program was recently initiated to systematically develop the data required to support this goal. Of particular interest are the development of structural details consistent with a philosophy for the design of the wall assemblages, experimental testing to obtain data on the structural and nonstructural performance of plastered straw bale wall assemblies, and the development of engineering design procedures that are consistent with and appropriate for the seismic-resisting characteristics of plastered straw bale wall construction.

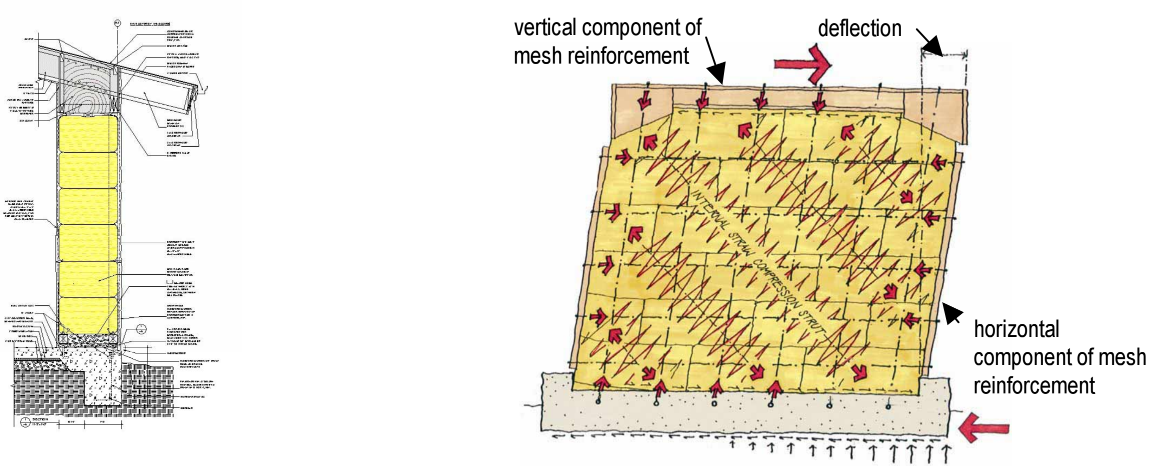
This coordinated research, conducted by the Ecological Building Network, is the first of its kind to collect multi-faceted data on a broad range of performance issues relevant to the development of recommended practices, the characterization of engineering properties, and the preparation of model code provisions for straw bale construction. Experts in varied disciplines familiar with straw bale construction established the materials and details used across the test program. Consequently, the results obtained provide a comprehensive description of the behavior of the assemblies of interest. The test program included structural and non-structural tests of straw bale wall assemblies and components as well as reviews that

[1] Designer, Degenkolb Engineers, Seattle, WA 98104; cash@degenkolb.com
[2] Associate Professor, Department of Civil Engineering, Santa Clara University, 500 El Camino Real, Santa Clara, CA 95050; maschheim@scu.edu.
[3] Principal, Tipping-Mar Associates, Berkeley, CA 94704; mar@tippingmar.com
[4] Director, Ecological Building Network, Sausalito, CA 94965; bruce@ecobuildnetwork.org

summarize the development of straw bale wall construction and previous laboratory test results, the results of previously unpublished fire safety tests, and the results of previous studies on thermal characteristics of straw bale wall assemblies. These are available on line from the Ecological Building Network at http://www.ecobuildnetwork.org/. Structural testing conducted under this research program includes tests to determine properties of earth, lime, and lime-cement plasters, tests to assess bearing capacities and creep deflections of plastered bales, tests to determine development lengths required for mesh for different types of plasters, tests to determine shear and tension capacities at the connection between the mesh and various wood sill plates, tests on the out-of-plane response of full-scale plastered straw bale wall assemblies, and tests on the in-plane response of full-scale straw bale wall assemblies. Moisture-related tests conducted as part of this research program include tests on straw and plastered straw bales, the monitoring of bale moisture in a variety of recently-constructed walls at the Ridge Winery in Healdsburg, California, and doctoral research at UC Davis on the degradation and decay of rice and wheat straw due to biological agents at various moisture levels and temperatures. Only a subset of this work is reported in this paper, which focuses on the in-plane performance of straw bale wall assemblies under reversed cyclic lateral loads, which pertains to the resistance of these wall assemblies to wind and seismic actions. Brief summaries of the results of other experimental investigations are reported by King [2].

OBJECTIVES AND SCOPE OF TEST PROGRAM

The test program sought to establish the behavior and engineering design basis for several types of plastered straw bale wall systems. A representative wall system and hypothesized lateral force resisting mechanism are shown in Figure 1. In order to proportion the full-scale wall assemblies, a series of small and medium scale tests were conducted to determine fundamental information about the behavior of various components required for a complete load path in the wall assembly. The main objective of the test series was to establish a rational basis for the design of the wall assemblies using capacity design principles to establish and isolate locations of intended inelastic behavior. Of particular importance was to establish the relationship between various levels of detailing and the ductility and strength that could be obtained. Thus, the in-plane tests functioned both as proof tests in relation to the adequacy of component details in the full-size wall assemblies and as fundamental experiments to establish the hysteretic characteristics of these full-size wall assemblies. The tests have validated the use of particular details and design philosophies for securing ductile behavior and help to establish the seismic-resisting characteristics of the plastered straw bale wall assemblies.



(a) (b)

Figure 1. (a) Straw bale wall assembly and (b) postulated load path.

Five of the six wall assemblies were tested under reversed cyclic displacements of generally increasing amplitude, while a monotonically increasing displacement was applied to one of the walls. All walls maintained their integrity and continued to support gravity load through imposed drifts of 7.5%, the capacity of the test setup. Three of the walls had earthen skins, which represent a relatively soft and environmentally-neutral material. The remaining three walls had cement stucco skins, a relatively stiff and strong but more costly material often associated with negative environmental impacts. The skins were applied to both sides of the wall and were reinforced with different types of mesh reinforcement using different levels of detailing. The resulting modes of behavior and hysteretic responses allow recommendations to be made for design and detailing to obtain walls that can serve as effective components of a lateral force resisting system. Comparison of the hysteretic behavior with that obtainable by plywood walls allows a preliminary recommendation to be made on the R factors that may be used for the design of plastered straw bale wall assemblies

SMALL AND MEDIUM SCALE TESTS

The small and medium scale tests were relied upon to establish the component details used in the designs of six full-size walls. Those of particular relevance to this paper are the mesh lap splice tests, mesh-sill plate connection tests, and shear and anchorage tests.

Mesh Lap Splice Tests

Of fundamental importance in the detailing of straw bale wall assemblies is the lap splice length required to develop the strength of the mesh, in both earth plaster and cement stucco plasters. Figure 2(a) illustrates the tension lap splice tests that were conducted. A 6-inch lap splice was found to be adequate for developing the strength of 2 in. x 2 in. x 14 gauge mesh in three cement stucco specimens. A 12-inch lap splice was adequate for developing the strength of a plastic mesh (Cintoflex, 1.75 in. x 2 in. x 1/16 in diameter) in three earth plaster specimens. The steel and plastic meshes had strengths of 78,000 psi and 26,000 psi, respectively.

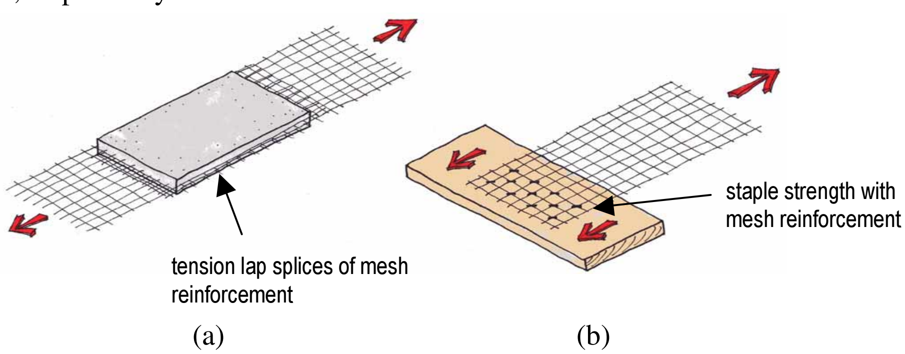


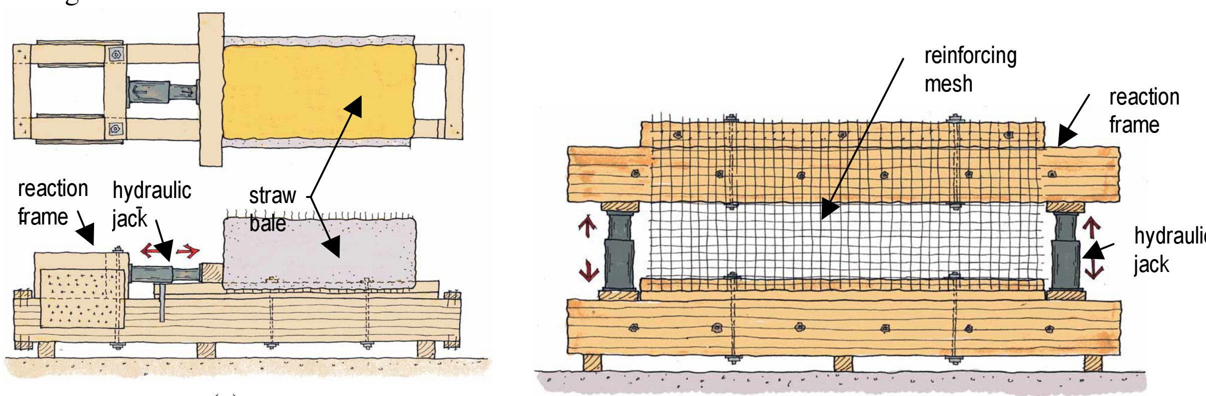
Figure 2: (a) Mesh lap splice tests; (b) mesh-sill plate connection tests.

Mesh-Sill Plate Connection Tests

To develop a ductile mechanism in the wall under lateral loading requires that the mesh be adequately anchored to transmit tension and shear to the sill plate. An economical means was sought to develop the strength of the mesh, using the setup illustrated in Figure 2b. It was found that the 2 in. x 2 in. x 14 gauge wire mesh could be fully developed using a 14 gauge x 1.75 inch pneumatically driven staple (7/16 in. crown) along a single line of mesh intersections (at 2 in. centers). Similarly, the same staples were adequate to develop the Cintoflex plastic mesh. However, a 2 in. x 2 in. x 16 gauge mesh anchored in this way was found to be brittle, due to fractures that took place at the welds at the mesh-wire intersections above the line of staples, leading to a tentative recommendation to avoid the 16-gauge mesh. Seventeen-gauge stucco lath (often referred to as Chicken wire) was found to be much too flexible due to its shape and should not be used as plaster reinforcement in structural applications. An alternate connection, consisting of gluing and clamping the mesh using a plywood cleat nailed over the mesh into the sill plate is not recommended, because the mesh pulled through the construction glue. Epoxy glue was found to be capable of holding the mesh, but is considered to be too costly.

Shear Anchorage Tests

Figure 3a illustrates the test setup used to determine the shear strength of the mesh-sill plate connection. Bales with mechanically connected 2 in. x 2 in. x 14 gauge wire mesh were jacked against a reaction frame that was connected to heavy wood sill plates. The mechanical mesh connection simulated a plastered bale condition. Based on these tests, the need for a shear band at the base of the wall was established. This band is provided by the vertical legs of the "U" shaped band of mesh at the base of the wall of Figure 4a.



(a)

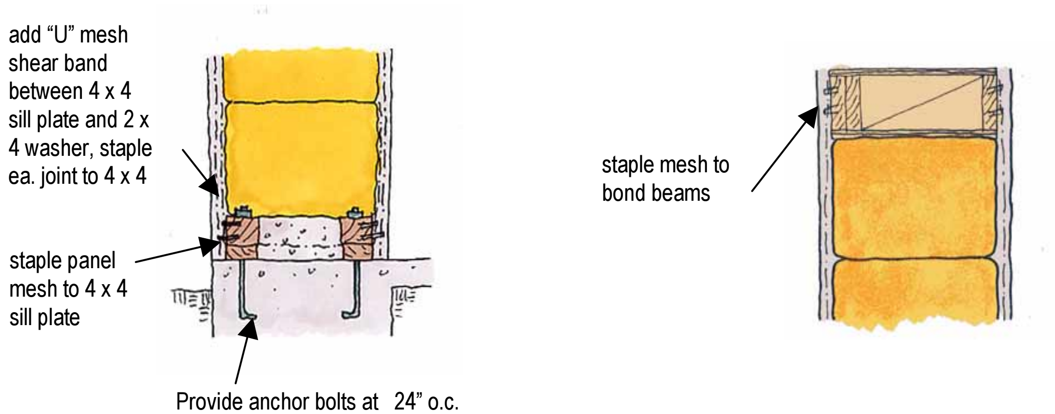
Figure 3. (a) Shear anchorage test setup, and (b) tension anchorage test setup.

Tension Anchorage Tests

Figure 3b illustrates the tension anchorage tests, which tests the load path at the base of the wall assembly, that consists of the mesh, staples, sill plate, and anchor bolts. Many varied details were tried in order to identify the most feasible means to anchor the tension mesh. It was found that single and doubled 2 x 4 sill plates were not sufficient for developing the mesh, as the plates were too flexible and were prone to cross-grain tension failures. When plates distorted due to bending and twisting, tension loads in the mesh were concentrated near the bolts, leading to premature local yielding and fracture. The recommended detail (Figure 4a) uses a "U" shaped band of mesh at the base of the wall, sandwiched between a 4 x 4 plate over a 2 x 4 sill plate and lapped over the wall mesh, with each layer of mesh stapled (at every intersection along a line) to the 4 x 4. Anchor bolts were 5/8 in. diameter with plate washers and spaced at 24 inches on center. The stiff 4 x 4 plate with tightly spaced bolts allowed essentially uniform tension on the mesh and higher overall tension loads for the entire mesh panel. It should be noted that the potential strength of the mesh is highly dependent on the stiffness of the anchorage. Based on these tests, anchorage of the mesh at the top of the wall is as recommended in Figure 4b.

Using the test setup of Figure 3b, the Cintoflex was determined to have an ultimate strength of 26,076 psi, the 2 x 2 x 14 gauge mesh was determined to have a strength of 78,476 psi, and the polytwine was determined to have a strength of 515 pounds per string.[4] These strengths are "in-place" strengths and

[4] The test setup utilized 2 jacks, each having an area of 5.15 in^2. The breaking pressure of 700 psi for the Cintoflex corresponds to 7210 pounds. The two curtains of 79-inch wide mesh (with vertical wires at 1.75-inch centers and horizontal wires at 2-in. centers) developed 80 pounds per wire, which corresponds



(a) (b)

Figure 4. Recommended anchorage details (a) at the base of the wall and (b) at the top of the wall.

account to a limited degree for the flexibility of the boundary conditions expected in realistic applications.

THE LARGE-SCALE IN-PLANE WALL TESTS

The large-scale in-plane tests were conducted to illustrate the load-deformation response of several different wall assemblies. The full-size walls were nominally 8-ft in height and 8-ft in width, constructed of three-string bales of rice straw. An earthen plaster was used for three walls and a cement stucco plaster was used for the remaining three walls. Varied details were used in the walls in order to relate the level of detailing with the measured load-deformation response.

Design Philosophy

Straw bale wall systems are composite systems that require the cooperation of a number of components to develop a complete load path for carrying lateral loads from the roof to the foundation. Consistent with the philosophy of capacity design, ductile behavior can be obtained when these components are proportioned to have sufficient strength to force inelasticity to develop in those components in which inelastic action can be sustained to relatively large deformations with little loss of carrying capacity. The earth plaster and stucco skins are much stiffer than the straw bales themselves, with the straw acting to brace the relatively slender skin materials. If compression struts in the bales are to be fully mobilized, the skins would have to degrade significantly under previous load reversals. Prior to this degradation, stress will develop throughout the field of each face, and these stresses will redistribute as the skins crack or fail locally due to the development of principal tension and principal compression stresses due to the applied lateral and gravity loads. The best prospects for obtaining ductile seismic response are associated with several alternative possibilities: (1) rocking of the wall, involving the opening of gaps at the base of the wall, with gravity loads providing a restoring moment; (2) flexural yielding of the wall, involving yielding of the mesh, (3) development of compression struts in the bales. Yielding of the mesh in flexure corresponds to an under-reinforced reinforced concrete member and is achievable with cement stucco skins. The compressive strength of earth plasters is low enough that even with relatively light plastic mesh (e.g. Cintoplex), crushing of the skin may occur, corresponding to an over-reinforced

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to 26,076 psi for a cross-sectional area of 0.0031 in^2 per wire. The breaking pressure of 3200 psi for the 2 x 2 x 14 gauge mesh corresponds to 32,960 lbs. The two curtains of 84-inch wide mesh (with both vertical and horizontal wires spaced at 2-inch centers) developed 392 pounds per wire, which corresponds to 78,476 psi for a cross-sectional area of 0.005 in^2 per wire. The breaking pressure of 500 psi for the polytwine corresponds to 5150 pounds. The 5 loops of polytwine (10 strings) developed 515 pounds per string.

reinforced concrete member. However, loss of the flexural resistance associated with failure of the mesh or skin is not tantamount to failure if a stable rocking mechanism can be achieved. The walls exhibited substantial toughness and did not lose their integrity or their capacity to carry gravity loads through drifts of 7.5% (the limit of the test setup), and it seems unlikely that earthquake displacement demands would be sufficient to cause collapse in light single-story residential construction having a relatively dense network of interconnected straw bale walls. Nevertheless, for purposes of code design, sufficient lateral strength must be demonstrated by analysis and provided through adequately detailed walls. Three levels of detailing were developed for each set of three walls, aiming for a flexural yielding mechanism in the case of the stucco walls and a flexural crushing or rocking mechanism in the case of the earth plaster walls.

Wall Details and Materials

Walls A, B, and C were constructed with earth plaster skins. A detailed as-built drawing of Wall B, having intermediate details, is provided in Figure 5; design drawings of all walls are provided in Ash et al. [3]. Details of the walls are as follows:

- Wall A has details intended for non-seismic applications, consisting of five poly-twine loops running continuously over the header beam and under the 2×4 sill. The ploy-twine used was Farmland 350 lb. Baling Twine and is marketed as a replacement for bailing wire. Poly-twine cross ties connecting the vertical loops are provided at alternating courses of bales. No other special details were provided.
- Wall B has a "moderate" level of detailing, consisting of a layer of Cintoflex plastic mesh (a black plastic mesh consisting of 0.05" nominal diameter legs spaced at 1-7/8" centers in both directions). This mesh was provided on a 96" wide roll. The mesh was stapled to the header beam and the 4x4 sill. A band of mesh running along the top and bottom portions of the wall was used to enhance anchorage at these locations, with the intent of forcing inelastic action away from the edges and into the field of the wall. Cross ties consisting of poly twine were provided at 24 inches on center at alternate courses of bales using 12" bamboo dowels to provide out-of-plane support to the earth plaster skin. Load transfer at the top of the wall was assisted by 3 rows of 20d nails, spaced at 8" along the length of the header beam.
- Wall C has a "high" level of detailing, consisting of 2 in. x 2 in. x 14 gauge welded wire mesh, with a "U"-shaped band providing enhanced anchorage to the 4 x 4 sill at the base and additional bands providing enhanced anchorage to the header beam. Confinement at the base of the wall was provided by clamping the lowest course to the 4 x 4 sills using plywood plates and threaded rods.

The earth plaster was comprised of earth, sand, water and straw fibers. Compression tests of 2-in. cube samples resulted in average compressive strengths of 290 psi after curing for 44 days and 160 psi after curing 94 days. This variability in compressive strengths could reflect varied constitutions, the heterogeneous properties of the materials and perhaps the need to use larger cubes to obtain more uniform samples, or might possibly reflect changes in moisture content over this time

Walls D, E, and F were constructed with cement stucco skins. As-built details for wall E are provided in Figure 6. Details of the walls are as follows:

- Wall D has relatively poor details, consisting of 17-gauge chicken wire mesh, provided both for reinforcement and to assist with stucco application. The roll width was 36 in.; the mesh was laid vertically with 6-inch laps between runs. The mesh was stapled at the base to a 2 x 4 sill and at the top to the header beam.

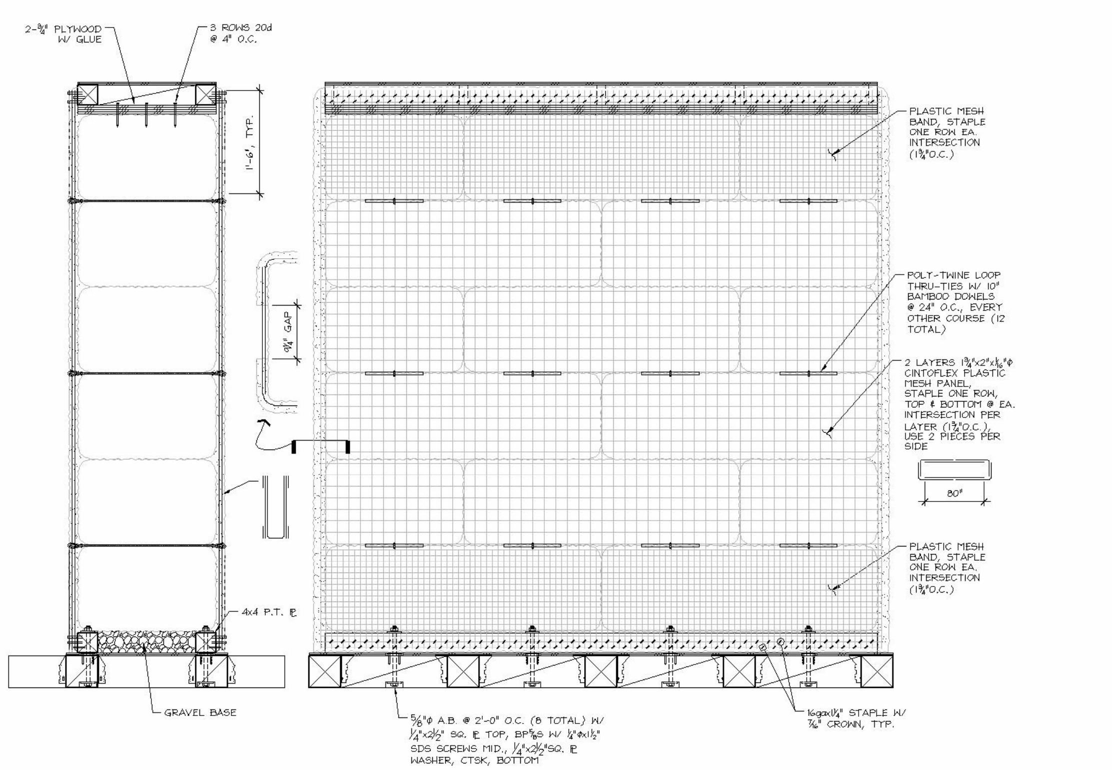


Figure 5. Details of Wall B—Earth plaster with intermediate detailing

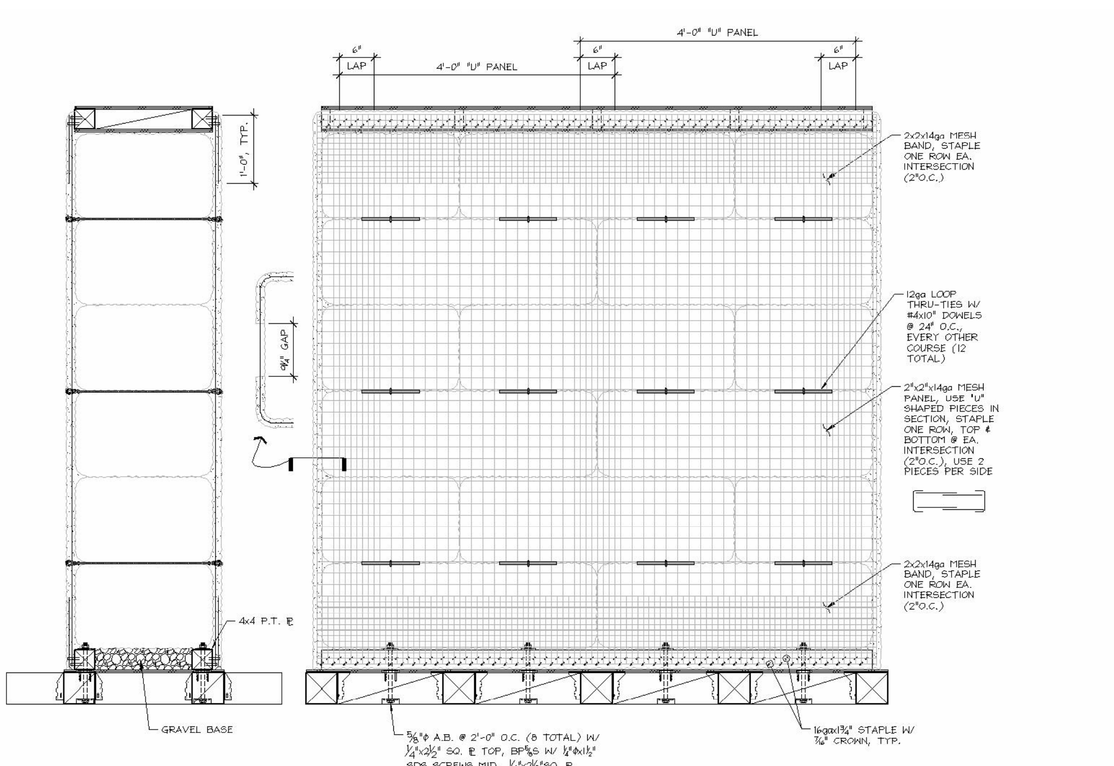


Figure 6. Details of Wall E (Cement stucco plaster with intermediate detailing

- Wall E has "intermediate" details, consisting of a 2 in. x 2 in. x 14 gauge welded wire mesh, stapled at every wire intersection along a line at the at the bottom into a 4x4 sill and at the top into the header beam. An extra layer of mesh reinforcement was provided at the top and bottom bale courses to assist in load transfer through the wall anchorages. Through ties consisting of 12-gauge wires at 24 in. centers at every other course of bales were provided, and were anchored by segments of rebar located in the body of the stucco. The through ties were provided to support the plaster against out-of-plane instability.
- Wall F has a "high" level of detailing, consisting of the details of Wall E supplemented with additional anchorage of the lowest course of bales, additional cross ties, and spikes added to anchor the header beam into the top course of straw. One layer of wire mesh was clamped under the 4x4 sill plate and bent up to engage with the skin reinforcement. The lowest bale course was confined in a manner similar to Wall C.

The stucco used Portland cement and lime as a binder. Batches were mixed in a mortar mixer. Quantities were measured using liquid measure containers in the following amounts: 30 gallons of sand, 8 gallons of cement, 2 gallons of slaked lime, and 6-1/2 gallons of water. The lime had been hydrated for five days prior to mixing the stucco, by mixing 6 gallons of water into a 50-pound bag of slaked finish lime. Average compressive strengths of 2-inch cubes were as follows: 1850 psi at 7 days, 2210 psi at 36 days, and 2200 psi at 95 days. The cubes were cured in the open air of the testing lab.

Staples used to anchor the mesh were pneumatically driven 14-gauge staples having a 1.75-inch legs and a 7/16 in. crown. The staples were not galvanized, creating the potential for corrosion over the long term. To avoid this, a U-shaped band could be used to provide anchorage at the base; epoxied-cleats may be useful at the to anchor the mesh at the top of the wall.

The specimens were constructed in the laboratory and tested after the plaster had cured for at least 1 month for the cement stucco specimens and 2-months for the earth plaster specimens. The temperature in the laboratory was maintained at approximately 70^{o}F and the relative humidity ranged between approximately 30% and 50% during this period.

Experimental Setup and Loading

The test setup (Figure 7) allowed gravity loads of approximately 200 pounds per linear foot of wall (1600 pounds over the length of the wall) to be applied using a system of counterweights and pulleys. Lateral displacements (monotonic for Wall A and reversed cyclic with generally increasing amplitude for the other walls, illustrated in Figure 8) then were applied using a servo-controlled hydraulic actuator. At the conclusion of each test, two complete cycles of 7.5% drift (7.2 inches at the top of the 8-ft wall) were imposed on Walls B through F while a single half cycle of this amplitude was imposed on Wall A. The 7.5% limit represents the limit of the test setup rather than the capacity of the walls. The top of the wall was free to rotate as it displaced; the beam attached to the wall was guided between Teflon pads to provide bracing against out-of-plane movement.

Observed Behavior

Photographs illustrating observed damage at various points in the testing are provided in Figure 9. The recorded lateral load versus displacement at the top of the walls is plotted in Figure 10 for each of the walls. After the completion of these cycles, each wall was also subjected to a manually-controlled reversed cyclic displacement of 7.5% (7.2" inches at the top of the wall). Damage occurred to the skins and to the 2x4 sills during these drifts, resulting in a reduction of lateral stiffness and strength, but each wall maintained its integrity and continued to support gravity loads throughout the tests. Key observations of the wall response are as follows:

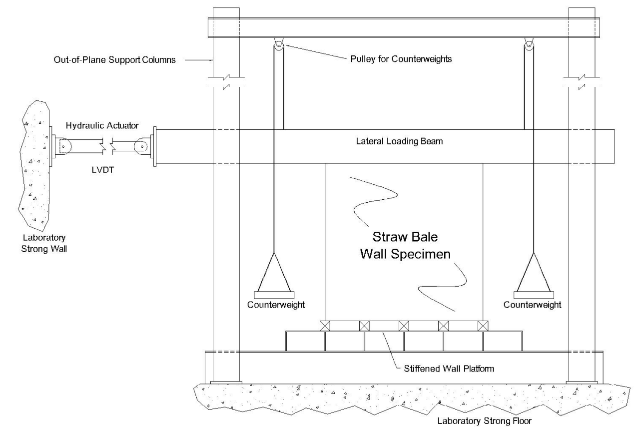


Figure 7. Test setup

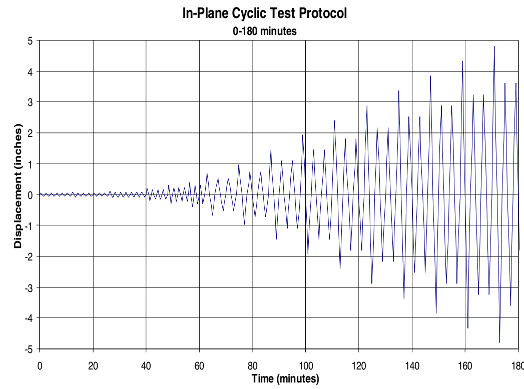


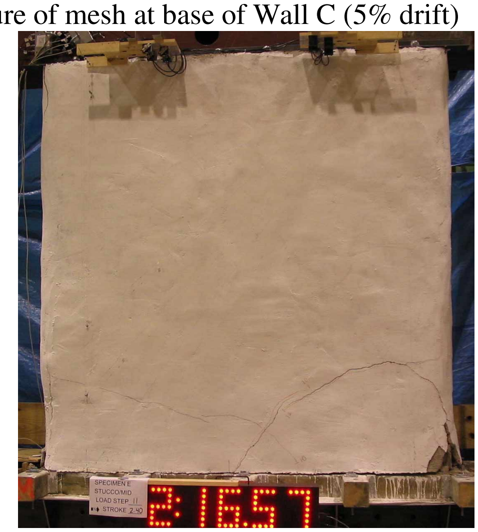
Figure 8. Imposed reversed cyclic displacement history

- Wall A had a peak strength of 3.2 kips (400 plf), which was reached at 4% drift (3.84 inches) and was maintained for higher displacements. The lack of mesh to anchor the earth plaster to the header beam led to slip of the header beam relative to the top course of bales Figure 9a. For drifts less than 2.5%, the failure mode appeared to be dominated by this slip. At drifts greater than 2.5%, crushing of the earth plaster in the compression zone began as the lateral resistance increased to its maximum of 3.2 kips. This change in behavior suggests that the poly-twine loops became engaged after the box beam slipped sufficiently. There was no evidence to suggest the poly-twine failed at any point during the tests, as the poly-twine was found to be taught when inspected at the gaps that opened at the base of the wall during testing.
- Wall B had a peak strength of 4.7 kips (590 plf), which was reached at a drift level of 1% (0.96 inches). The predominant failure mode was crushing of the earth plaster in the compression zone and base sliding, with the base sliding becoming more pronounced at later stages of the test. The change in the shape of the hysteretic loops in Figure 10 suggests that sliding of the wall became more significant at higher displacements. At 4% drift, measured sliding at the base of ¾ inch (peak-to-peak) accounted for 20% of the lateral displacement. At 2.5% drift, the repeated cycling had begun to degrade the earth plaster to its sand, clay, and straw constituents, and caused a reduction in the height of the wall, with the debris acting as a wedge to push the skin away from the bales. At this drift level, the earth plaster began interlocking with the 6 x 6 cross-ties of the test setup. Flexural cracking developed in the wall at large drifts.
- Wall C had a peak strength of 6.1 kips (760 plf), which was reached at 1.5% drift (1.44 inches). At 1% drift, the 6.0 kip resistance was an increase of nearly 30% over the peak resistance of Wall B, which was reached at this drift level. Failure occurred by crushing of the earth plaster and sliding of the wall at its base. The steel mesh reduced the measured slip at the base to a 1/2" amplitude at a drift of 4%. Fracture of the mesh at a drift of 5% is shown in Figure 9b; the fracture is attributed to repeated cycling during the progressive crushing of the earth plaster in compression, leading to low-cycle fatigue. No flexural tension cracks were observed during testing of Wall C, indicating the mismatch of plaster compressive strength to wire mesh tensile strength. As the earth plaster crushed, it began to interlock with the 6 x 6 cross-ties of the test setup at drifts above 3%.
- Wall D had a peak strength of 6.4 kips (800 plf), which was reached at 1% drift (0.96 inches). Pinching of the hysteresis loops began prior to this peak (Figure 10) and is indicative of the rocking behavior that was visually observed at larger displacements. After the peak was reached,

(a) Slip of the box beam at the top of Wall A

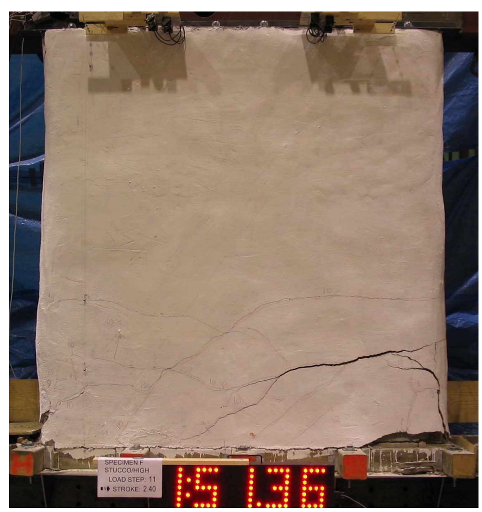
(b) Fracture of mesh at base of Wall C (5% drift)

(c) Cross grain tension failure of 2 x 4 sill at base of Wall D (5% drift)



(d) Flexural cracks in Wall E (2.5% drift)

(e) Fracture of mesh and failure of staples at base of Wall E (7.5% drift)



(g) Flexural cracks in Wall F (2.5% drift)

Figure 9: Behavior of specimens (photographs)

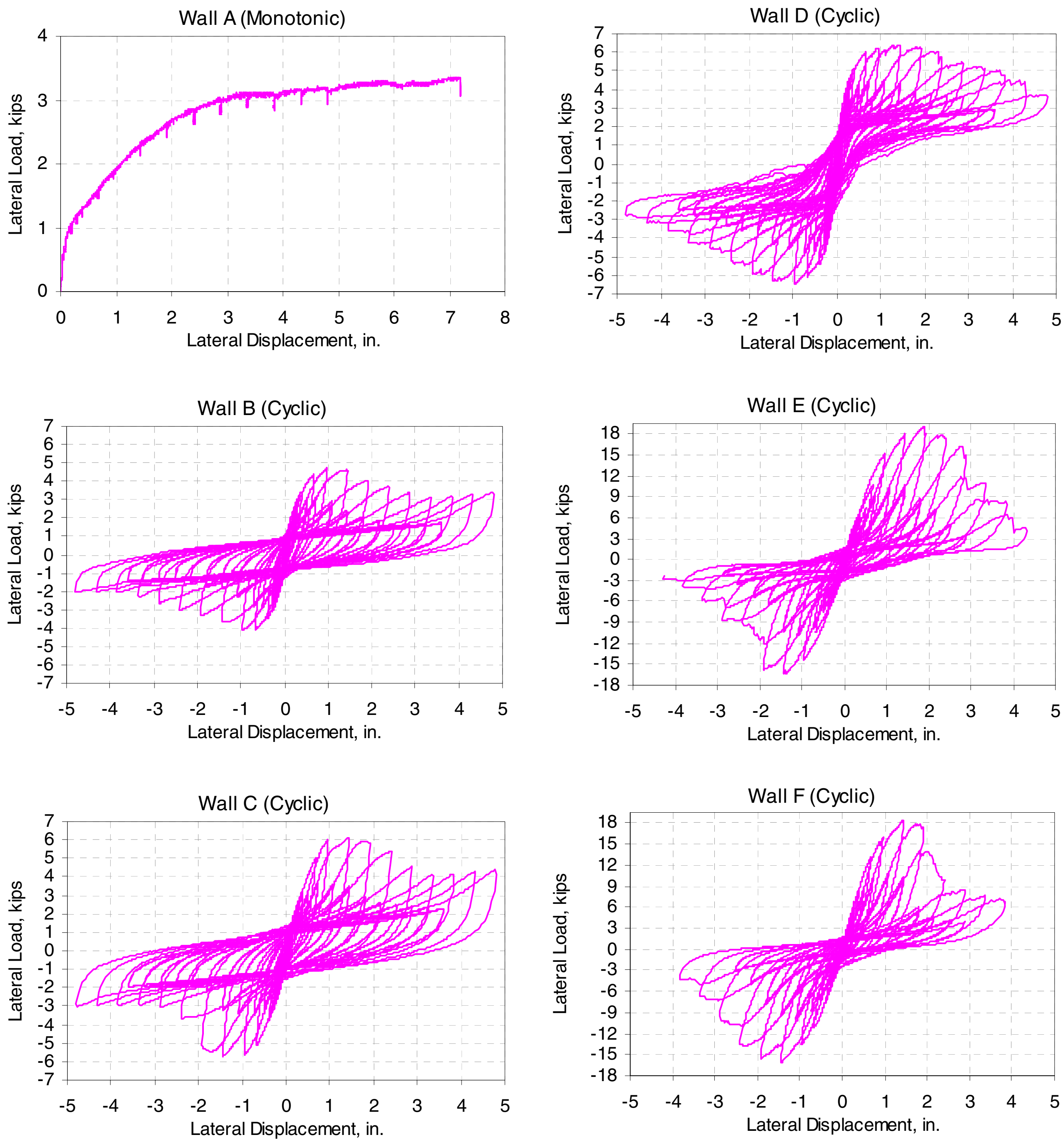


Figure 10: Lateral load-displacement response of the six wall specimens.

two distinct failure modes were observed at the base of the wall. In some locations the chicken wire mesh was distorted and elongated, and had fractured, while at other locations the 2 × 4 sill plate receiving the mesh staples had failed in cross-grain bending as shown in Figure 9c. By the end of the test, the combination of mesh fractures and sill failure had progressed along the full length of the wall, allowing the observed rocking behavior to occur unimpeded. None of the staples anchoring the mesh were observed to fail or pull out.

- Wall E had a peak strength of 19 kips (2380 plf), which was reached at 2% drift (1.92 inches). Several flexural cracks developed within the bottom third of the wall height as shown in Figure 9d. The mode of failure was the loss of tensile capacity of the reinforcing mesh, from both mesh fracture and staple pull out, as shown in Figure 9e. The mesh fracture was attributed to a combination of tensile elongation and low-cycle fatigue associated with the load reversals, which appeared to work the vertical wires of the mesh near the staples, which also tended to push the stucco out, away from the straw. The failures predominately occurred at the staple locations with some failures also occurring at the intersections of the horizontal and vertical wires, where the wires are spot welded together in the manufacturing of the mesh
- Wall F had a peak strength of 18.2 kips (2280 plf), which was reached at 1.5% drift (1.44 inches). A comparison of the load-displacement plots in Figure 10 for Walls E and F, respectively, shows the limited benefit gained from the additional detailing provided in this wall. The confinement of the first bale course was intended to shift the failure above this level while distributing the yielding of the reinforcing mesh over more of the wall height. This was effective, as shown in Figure 9f, but even so, there was not an appreciable difference in the ductility or strength of this wall compared with that of Wall E.

Discussion

Figure 11 compares the envelopes of the load-deformation curves of the individual specimens. The walls are seen to develop similar strengths at large displacements, associated with the restoring moment provided by the dead load when the walls rock about their bases. The stiffnesses and strengths at smaller displacements are seen to be a function of the plaster material and reinforcing. The stiffest walls had the cement stucco skin; the strongest walls had wire mesh reinforcing the cement stucco skin (Walls E and F). The nearly unreinforced earth-plaster wall (Wall A) had the least strength and stiffness. Although the earth plaster walls (Walls B and C) failed by crushing of the earth plaster, the earth plaster was sufficient to maintain the integrity of the stacked straw bales for carrying gravity loads. Rocking of the walls at large drifts mobilized a gravity load restoring moment that provided sufficient lateral strength to give the overall straw bale wall system a relatively ductile behavior.

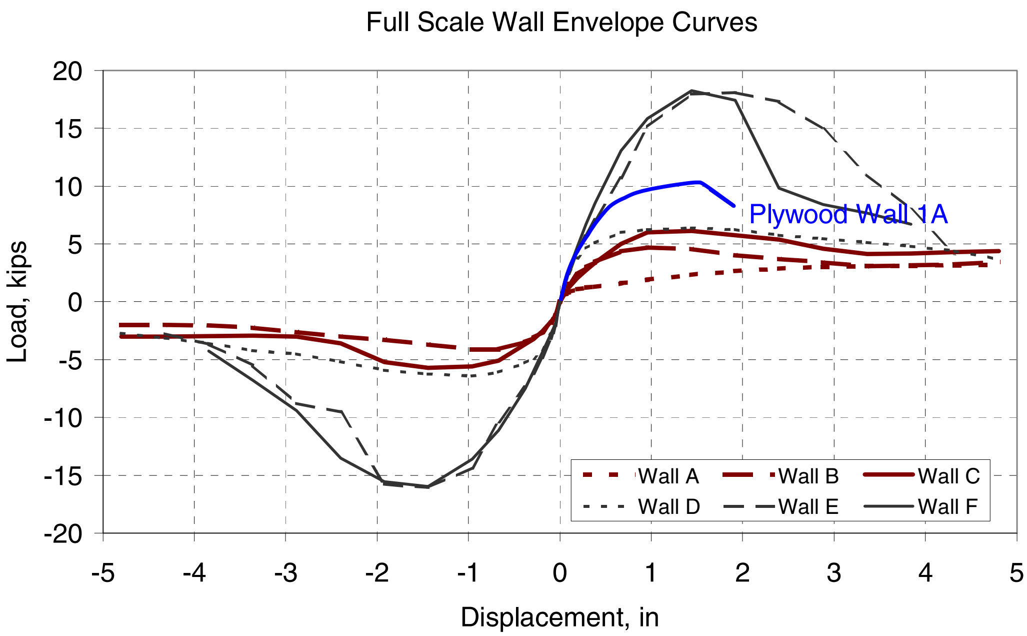


Figure 11. Envelope lateral load-displacement curves for the six wall specimens.

Walls E and F developed similar load deformation responses, indicating that the clamping of the lowest course of bales with threaded rods and plywood, the spikes, the additional cross ties, and the confinement of the first course of Wall F provided little benefit. Comparison of Walls E and D shows the substantial increase in strength obtained when welded wire mesh anchored to 4x4 sills is used in place of chicken wire anchored to 2 x 4 sills.

COMPARISON WITH PLYWOOD SHEARWALLS

A series of reversed cyclic tests of plywood shear panels having the same nominal dimensions as the full-scale plastered straw bale wall tests was completed in 1998 (Rose [4]). According to Rose, the load-displacement curves had similar shape characteristics for all tests specimens. An example is illustrated in Figure 12, for a shear wall tested under a similar reversed cyclic loading protocol. This wall was constructed of 15/32-inch Structural 1 Plywood, with 10d common nails at 4-inch centers along the edges. Holddowns at either end of the wall were attached to 4x4 end posts. Intermediate studs consisted of 2x4s at 16-in. centers. A 3x4 post was used for the center stud, where a butt joint was formed by the adjoining plywood panels, and for the bottom plate. Only reversed cyclic loads were applied; gravity loads bearing on the walls from above were not reproduced in this test. The top of the wall was free to displace and rotate, just as in the straw bale wall tests. A blue curve representing the envelope of response is superimposed on the measured response. A peak load of approximately 10,150 pounds was reported, at a drift of 2.0 inches.

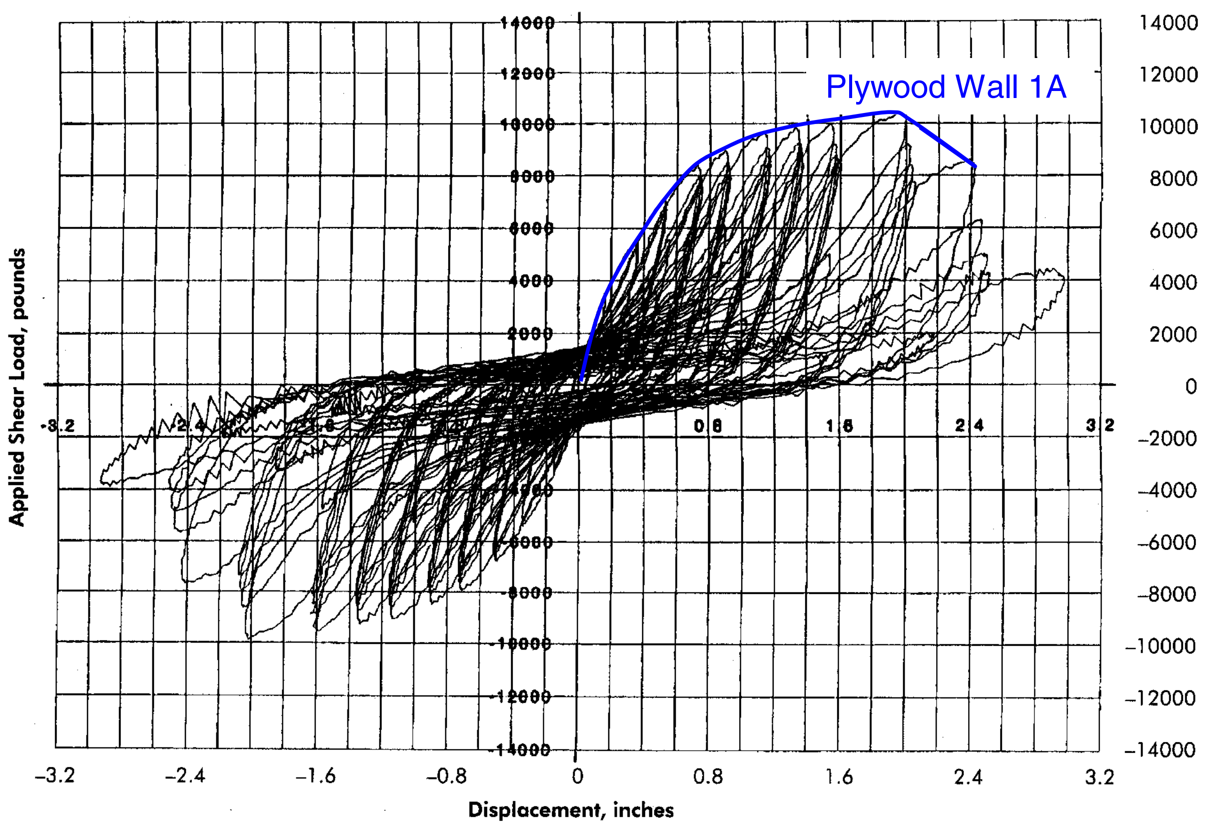


Figure 12. Lateral load–displacement response of an 8 ft. x 8 ft. plywood wall (Wall 1A) reported by Rose, adapted from [4]

The blue curve representing the envelope of response of the plywood wall was superimposed on the envelopes of the individual plastered straw bale wall assembly responses (Figure 11), adjusted for the scale of the plot. A preliminary determination of design values for Wall E (Stucco with intermediate detailing) will be established by comparison with accepted design values for plywood shear walls, as follows:

1. According to Rose [4], the allowable design load for plywood walls is about 1/1.7 times the actual strength of the wall at a drift of 0.5% (0.48 inches). Because Wall E has a strength of about 8.3 kips (1040 plf) at this drift level, the corresponding design strength is 1040 plf / 1.7 = 610 plf.
2. The actual load factor (peak strength divided by allowable design strength) is (18.1 kip / 8 ft) / 0.610 klf = 3.70. This is well in excess of the load factor provided by plywood walls, which ranged between 2.0 and 2.7 for the plywood walls reported by Rose [4].
3. Because of the large load factor and the large deformation capacity of plastered straw bale walls (Wall E) relative to that of plywood shear walls, the R factor of 5.5 that is used for the seismic resistant design of bearing walls with light-framed walls sheathed with structural wood panels is a conservative value for the design of plastered straw bale wall assemblies that resemble Wall E with regard to aspect ratio and dominant mode of behavior.

As the understanding of the engineering characteristics of straw bale wall assemblies develops further, a more refined evaluation of R factors suitable for their design can be developed.

CONCLUSIONS

Based on the full-scale in-plane tests of plastered straw bale wall assemblies supporting imposed nominal dead loads of 200 plf:

(1) The details used in Walls B, C, E, and F were shown to provide good control over the mechanism of inelastic deformation that resulted in the walls.
(2) The compressive strength of the earth plaster walls was critical for Walls B and C, and resulted in a crushing failure that has parallels to an over-reinforced reinforced concrete member.
(3) The welded wire mesh used in Walls E and F yielded successfully as part of the development of a ductile flexural mode of response in these walls.
(4) All walls maintained their integrity and continued to carry gravity loads through drifts of 7.5% of their height, with no sign of impending failure. Larger drifts could not be imposed because of limitations in the test setup.
(5) The details used in Walls B, C, and E can be recommended for future use for the construction of walls having a nominal 1:1 aspect ratio (8-ft high by 8-ft in length). Further analysis and review of test data is required to make recommendations for walls having a broader range of aspect ratio.
(6) A preliminary comparison with the measured response of plywood walls indicates that an R factor of 5.5 would be conservative for the design of walls having geometry and details corresponding to Wall E. An allowable shear of 610 plf (for straw bale walls with mesh-reinforced stucco on both sides of the wall) would be consistent with the allowable shear values commonly used for the design of plywood shear walls. Measured peak strengths (overstrength) were approximately 3.7 times this allowable value.

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