DESIGN AND IMPLEMENTATION OF A RAMMED INFILL ADOBE AND PLASTIC BOTTLE WALL SYSTEM IN HONDURAS

By
Garry Alvin Enns

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Department of Biosystems Engineering
Faculty of Engineering
University of Manitoba
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ABSTRACT

A new wall construction technique utilizing concrete filled PVC tubes, adobe, polyethylene terephthalate (PET) plastic bottles and cabling was designed, analyzed and implemented. The system was designed as an alternative to traditional earthen buildings in Honduras that are vulnerable to hurricane winds and seismic activity. Six prototype panels were constructed at the Alternative Village located at the University of Manitoba to evaluate the wall system for racking and transverse load capacity. A kitchen was then constructed at an elementary school in western Honduras.

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1. Introduction

1.1. Purpose of the Research

The purpose of this study was to assess the performance of a formed in place earth wall system constructed of concrete filled PVC tube columns, rammed adobe, empty polyethylene terephthalate (PET) drinking bottles, aircraft cable and barbed wire. The two main aspects evaluated were the system's resistance to racking and transverse loads.

This research was conducted to support the work of *Students for Sustainability (S4S),* a group under the auspices of the Alternative Village at the University of Manitoba. A link was established in 2012 with a community in western Honduras that provided students, both graduate and undergraduate, the opportunity to conduct research and implement designs to actual projects in the developing country. Nearly 30% of the 8.1 million people living in Honduras make less than $2 per day (United Nations Development Programme 2013) and the country was declared one of the 25 'countries of focus' by the Government of Canada's efforts for international development in 2014 (Government of Canada 2015). S4S collaborates with local community leaders and organizations in Honduras to identify research needs and applicable locations and methods for implementation.

1.2. Nature and Scope of the Problem

Earthen buildings often fail in a brittle fashion without warning due to low tensile strength, however implementing alterations to traditional earth construction can be culturally challenging. Honduras is situated in an area of seismic and hurricane activity so there is added incentive to improve structural performance of earthen structures in the region. Using alternative techniques and materials can be construed by locals as interference or rejected because of higher costs from extra materials or from the time required to learn the new system (Blondet and Aguilar 2007).

Working in the rural community of Guanteque in western Honduras required cultural sensitivity to available construction materials, methods and technical skills. The majority of housing built in Guanteque is adobe and generally constructed by the residents themselves. These homes lack any additional reinforcing materials so they are vulnerable to the effects of seismic activity and high wind speeds. While technical solutions do exist to improve the safety of earthen buildings, there is a need for cheaper and simpler methods (Blondet and Aguilar 2007). Introducing a system that local residents could adopt or adapt had to be cost and time effective. As many residents in Guanteque lack higher levels of formal education or access to information outside of the community, an interactive process for introducing and learning new techniques is necessary. Building a kitchen and dining room facility at an elementary school utilizing the wall system seen in Figure 1, allowed for greater community exposure and interaction in the construction process.

Figure 1: Kitchen Built in Guanteque, Honduras

Buildings in Latin America are often designed to withstand seismic load instead of wind, though wind may be the governing load (Compañy 2002). An overview of the seismic codes in Central and South America reveals Honduras is the only Latin America country located within a seismic zone that has never had a seismic code to characterize local hazards (Chavez 2012). Design values for seismic activity obtained from the U.S. Geological Survey were used to evaluate seismic loads as seen in Appendix H. Design wind load values are not available for Honduras but wind speed data from hurricane events is available at various locations in the country. Hurricane Mitch, which struck Central America in October 1998, was the deadliest hurricane in the Western Hemisphere in over 200 years (History.com 2009). The storm that eventually grew into a Category 5 hurricane was classified as a Tropical Storm as it past over the town of Belen (13 kms from Guanteque) with wind speeds of 40.3 mph. The hurricane tracked over a considerable amount of land prior to Honduras as it went through Mexico and Guatemala before increasing intensity as it moved through Honduras tracking north off the coast (National Oceanic and Atmospheric Administration n.d.). A future hurricane could track differently through Honduras bringing higher wind speeds to Guanteque which is located roughly 81 miles from the northern coast. A design wind speed of 111 mph was chosen as a middle ground between the Tropical Storm rating experienced by Gaunteque during Hurricane Mitch, and the Category 5 wind speed levels (over 157 mph winds) achieved by Mitch off the northern coast of Honduras. This wind speed represents the low end of a Category 3 Hurricane (National Weather Service 2013). The design wind load of 111 mph evaluated in Appendix G shows that wind loads do govern over seismic loads in the community of Guanteque.

1.3. Research Objectives

The objectives of this study were to determine the feasibility of using PET bottles as filler in a rammed infill adobe wall system and successfully implementing the system in Honduras. Specific research objectives of this study were to:

1. Evaluate the axial capacity of 102 mm (4”) diameter PVC concrete filled plastic tubes;
2. Determine the racking capacity of the wall system;
3. Establish the transverse resistance of the wall system.

1.4. General Description of Research Method

1.4.1. Initial Concept Development

More than 50 percent of the earth's population dwell in buildings constructed from earthen materials (Schroder and Ogletree 2010) as many underprivileged people in developing countries take advantage of it's availability and little or no cost. Honduras is certainly no exception to this as many homes are built from adobe block as previously mentioned in section 1.2. Adobe construction is explained in further detail in section 2.4 but its main disadvantages are being very labor-intensive, time consuming and being susceptible to seismic and high-wind loads. As a research project under the auspices of the Alternative Village and the student group *Students for Sustainability* at the University of Manitoba, the project needed to incorporate alternative and sustainable building techniques and materials to overcome the disadvantages of adobe construction. So the initial phase of research was to select materials and conduct a variety of load-deformation tests on the individual components of the wall system to optimize their performance. The following factors were considered when selecting materials and construction methods to increase the likelihood of successfully implementing the wall system in Guanteque:

- Culturally acceptable, understandable and replicable by locals
- Material costs and availability
- Tool costs, acquisition and ease of use
- Time required for construction

Figure 2 shows the wall system configuration and the materials selected for prototyping, and ultimately implementation, to overcome the disadvantages of adobe block construction.

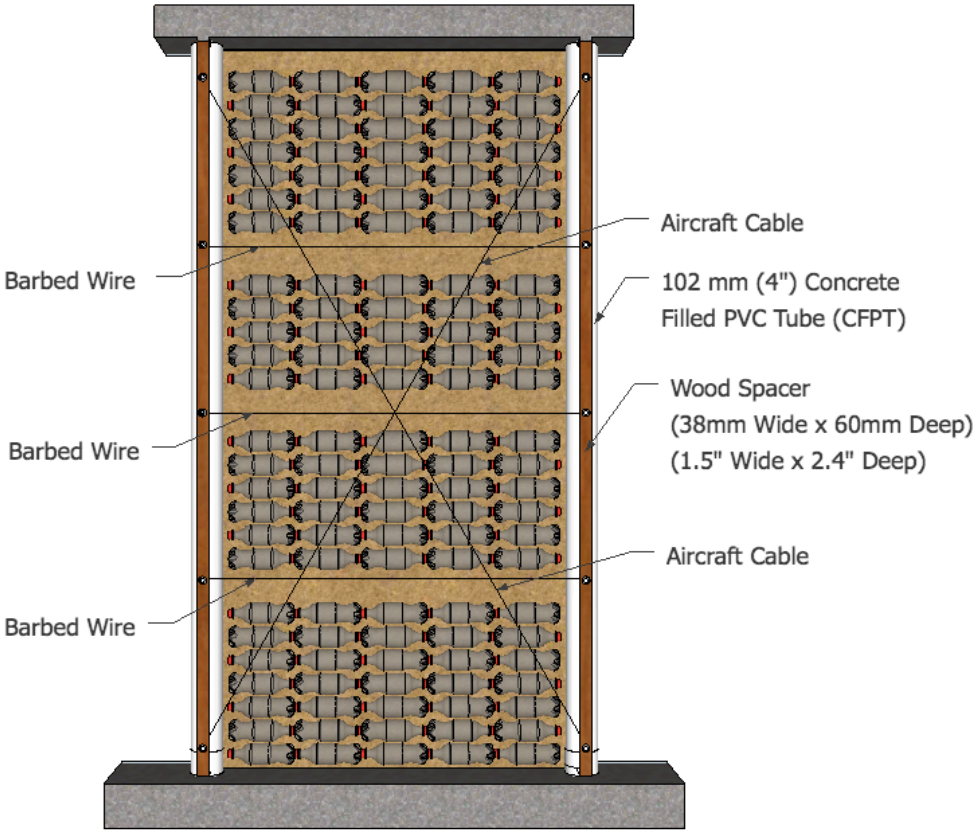

Figure 2: Prototype Schematic

1.4.2. Wall System Component Details

Five main components worked together to complete the wall system. The complete wall system implemented in Honduras was essentially 1.2 m x 2.4 m (4’ x 8’) panels as seen in Figure 2 acting in series. Individual components, their purpose and function are:

1. **Concrete Filled PVC Tubes (CFPTs)**

CFPTs had been introduced to local residents of Honduras in previous work completed by *Students for Sustainability* at the University of Manitoba so they had some familiarity with the technique. Adobe block construction, like other earthen wall systems, results in heavy walls that are essentially loadbearing walls due to the weight of the earth itself. CFPTs were chosen to carry the axial load from the roof system to the footings to eliminate additional load acting upon the infill of the wall system. Threaded rod pushed through and cast into the CFPTs provided connection points for the aircraft cable and barbed wire, and ability to secure wood spacers onto the PVC upon which formwork could be attached for placing infill materials.

2. Aircraft Cabling

Cabling was chosen for it's high tensile strength and elasticity to resist racking loads. Using turnbuckles to complete the loops created by the aircraft cable provided the means to consistently tension the cables.

3. Barbed Wire

Barbed wire aided in resisting transverse loads by carrying the loads through to the CFPTs. It was chosen over aircraft cable because it was cheaper and more likely to be used by local Honduran residents, many of whom are farmers and use barbed wire for constructing fences.

4. PET Bottles

Recyclable PET bottles were the 'sustainable' feature of the wall system. Their ability to absorb shock loads was the prime reason for their selection. Placing air-filled bottles in the wall was a free and lightweight material.

5. Rammed Adobe

Adobe is common to Honduras so locals would have no problem replicating the mix. Utilizing the adobe in a rammed earth formwork system demonstrated a very quick technique for constructing walls as the labor and time intensity of making blocks, letting them dry and then placing them was eliminated. The potentially devastating effect of rainfall on the quality of adobe blocks during any phase of construction was also eliminated as the rammed adobe could withstand rainfall events at any point during construction.

1.4.3. Prototype Construction & Implementation

Prototypes were constructed and tested at the Alternative Village located at the University of Manitoba in Winnipeg, Manitoba, Canada, prior to implementation of the system in the community of Guanteque in western Honduras. Adobe block walls or other earthen building walls were not constructed for comparative testing to the wall system developed in this thesis. Brittle failures and low-tensile strength are widely accepted deficiencies of earthen building techniques so the focus of this research was on producing an earthen wall system that could withstand high impact wind and seismic loads without succumbing to brittle failure.

Six 1.2 m x 2.4 m (4’ x 8’) prototypes complete with infill were constructed to allow for three cyclic racking and three airbag transverse tests to be conducted. The desire was to build smaller models using similitude but given the unpredictability of how the various components would behave together in the system, 1.2 m x 2.4 m (4’ x 8’) models were used. Building models of this dimension also allowed for comparison to standard 1.2 m x 2.4 m x 0.14 m (4’ x 8’ x 5.5”) wood studded wall sections sheeted with 13 mm (½”) drywall and 13 mm (½”) oriented strand board (OSB) typically found in construction in Canada and previously tested at the Alternative Village. Two additional 1.2 m x 2.4 m (4’ x 8’) prototypes were constructed solely of the cast PVC tube columns, concrete bond beam and aircraft cabling and used for cyclic racking trials to avoid damaging the complete prototypes as cyclic loading had not been attempted previously with the equipment to be used.

The prototypes were all constructed inside of the research facility building to protect them from potential rain and snow during the fall of 2014. Temperatures inside the building ranged from roughly 10°C to 20°C during the phase of infill placement and curing. The wall systems were allowed to cure for a minimum 28 days prior to testing.

Successful implementation of the wall system in Honduras was measured simply by the completion of the kitchen building using locally available materials and informal feedback from community members as they observed and interacted with the construction process. Community members in Guanteque contributed to the project by collecting used plastic bottles and supplying the earth and straw necessary for the earth mix.

2. Literature Review

Currently employed methods and materials for improving the tensile capacity of earthen building techniques are examined in this literature review. An overview of concrete filled tubes and PET bottle use in construction, followed by a general summary of common earthen building techniques complete review of literature.

2.1. Reinforcement Techniques for Earthen Buildings

2.1.1. Internal Reinforcement

Internal reinforcement materials vary from biodegradable materials to steel. The amount of the material used depends on the type of earthen structure being built. For example, many homes in Central America are built in the style of *bahareque* (wattle and daub). *Bahareque* is variation of frame construction where regularly spaced horizontal branches are tied to wooden poles that are set firmly in the ground at corners and intermediate panel points. Smaller branches are then woven through the horizontal members before the wall is finished with mud plaster. The condition of structural wood and the weight of the roof are susceptible factors in *bahareque* construction. The wood needs to be treated from insects and rodents, and for moisture where wood contacts the soil to maintain integrity. (López, Bommer and Méndez 2004)

Adobe and rammed earth wall systems can be reinforced with vertical and horizontal reinforcement bars. Specially made O-bricks and U-bricks (O and U shaped holes in the blocks) for adobe allow for vertical bars to be grouted into the wall, while horizontal members can be placed in mortar layers between blocks and tied to the vertical bars. (Schroder and Ogletree 2010) While rebar rods are the conventional reinforcement, vertical cane rods and horizontal layers of crushed canes have also shown excellent response to seismic motion. Availability can be a limiting factor when considering the use of cane. (Blondet and Aguilar 2007)

2.1.2. External Reinforcement

Wire mesh nailed into adobe walls and covered with a cement mortar has been used to provide additional strength. The mesh is laid in horizontal and vertical strips similar to beam and column layouts. The mode of failure with this system under earthquake simulations tests was found to be brittle, but houses built with this style of reinforcement didn't suffer any damage during a moderate earthquake in Peru while surrounding houses suffered extensive damage. The cost of wire mesh and cement can be a major deterrent for this style of reinforcement. (Blondet and Aguilar 2007)

Wooden elements making a frame entrenched into the inside and outside faces of adobe or rammed earth walls is another alternative to external reinforcement. Embedding the wood into the rammed earth or trenched into adobe blocks allows the wall surface to be mortared smooth to maintain the original shape of the wall. A comparative study between a wooden element frame and wire mesh reinforcement showed these alternatives improved seismic behavior but they did not prevent, they simply delayed the collapse of the tested models. Of the two, the wooden elements had a better resistance to earthquake input showing better ductile behavior. (Yamin, et al. 2004)

2.2. Concrete Filled Tube Columns

Concrete filled tube (CFT) columns can offer both structural and ease of construction advantages to reinforced concrete members. The tubes provide tension, bending moment and shear resistance while concrete delivers stiffness and compressive strength that reduce buckling potential. (Roeder, Cameron and Brown 1999) Several factors make CFTs an economical and time efficient option. The tubes are the formwork and internal reinforcement is generally unnecessary in the concrete fill (Moon, et al. 2013). The tubes for CFTs have historically been steel or fibre-reinforced-polymer (FRP). There is a need to explore alternative material options for the tubes as the corrosion of steel and the cost of FRP production can be prohibitive (Gathimba, Oyawa and Mang'uriu 2014).

Plastics can be used for a variety of structural applications as they have high resistance to environmental degradation and high strength to weight ratios. Little research has been done to provide a database on the ultimate strength of plastic columns with concrete core and more research is needed to evaluate various tube sizes, slenderness ratio and concrete properties. Compressive axial testing on six 102 mm (4”) concrete diameter specimens at Ryerson University found 3 mm (0.1”) thick PVC tubes provide significant confinement to concrete increasing the ultimate compressive strength of the columns (Marzouck and Sennah 2002). Tests were conducted on plain concrete specimens to analyze the gain from PVC tube containment. Specimens of 270 mm (10.5”) and 416 mm (16”) in height had increases in compressive axial resistance of 11% and 17% respectively when constrained. This shows that containing concrete has a greater impact on structural integrity as height increases. The compressive axial resistance of the PVC contained concrete decreased as height of specimen increased from

318 kN at 270 mm (10.5”) to 287 kN at 758 mm (30”). The plain concrete failed in a brittle manner whereas the concrete-filled PVC had significant axial plastic strain before failure. The high level of ductility by concrete-filled PVC tubes would provide warning of potential failure and collapse.

2.3. PET Bottles as a Sustainable Material

Plastic bottles are a sustainable material because the environmental degradation costs of disposing or recycling the bottles is greater than reusing the bottles for other purposes. Recycling plastic requires energy and produces wastewater and air pollutants. The bottles are also socially equitable as they are available to both rich and poor. As industrial building materials replace indigenous and traditional materials, the cost of housing increases and the ability of homeowners to design and build their own homes decreases. Plastic bottles are not only a sustainable alternative to industrial materials, they offer several other advantages including being light weight, low in cost, have suitable thermal behavior, are non-brittle and able to absorb shock loads. (Shoubi, Shoubi and Barough 2013)

A common technique for building with plastic bottles is turning the bottles into bricks by filling and packing them with a variety of materials. This idea, first developed by Andreas Froese and called the ECO-TEC technique, has been used in eight countries across Latin America, Africa and Asia (Eco Tecnologia n.d.). The basic idea of creating bottle bricks is to fill the bottles with rubble, soil, plastic bags, debris or other available materials. These bricks are laid horizontally on top of each other or mortared together in rows, and tied together with string or barbed wire.

The compressive strength at failure for individual bottle bricks packed with waste plastic bags or other plastic waste was found to be 2.72 MPa with Poisson's ratios ranging from 0.27-0.35 (Taaffe, et al. 2014). Another study evaluated the strength of masonry plastic bottle blocks with three variations on bottle infill. The infill variations were dry sand, saturated sand, and air filled. Eight bottles were mortared into blocks 300 x 300 x 300 mm (12" x 12" x 12") using cement mortar (1:2:0.54 sand: cement: water) and subjected to unconfined compressive loading. The air filled bottle block showed the highest compressive strength of 0.67 MPa, with dry sand and saturated sand bottles testing at roughly 0.62 and 0.61 MPa respectively. This compares to 3.67 MPa strength for traditional 390 x 190 x 190 mm (15" x 7.5" x 7.5") hollow concrete block. (Mansour and Ali 2015)

Mansour and Ali also found that air filled bottle blocks had a higher thermal resistance than blocks containing filled bottles and traditional masonry block. Their simulations showed the thermal resistance values for air, dry sand, traditional masonry, and saturated sand were 18.967, 9.9093, 4.63 and 1.932 °C/W respectively.

The construction technique of laying and tying individual bottle bricks is quite different than the reliance on cement mortar used for placing masonry plastic bottle blocks tested by Mansour and Ali. The variation in the two systems reveals a knowledge gap exists in the use of plastic bottles for construction to achieve optimal physical, mechanical and thermal properties.

2.4. Earthen Wall Systems

A variety of earthen wall systems exist that are influenced by climate, culture and availability of materials. Variations are found both in the materials and in the processes used to construct the walls.

Adobe is the most common and oldest earth-based material and it’s used primarily in drier areas. The basic mix for *adobe* is clay and sand mixed with water to create mud. The mud is often mixed with straw to improve the tensile strength. Frequently the mix is poured into forms to create *adobe blocks* that are sun dried before being mortared into place. Making, curing, and then placing the blocks is labor-intensive and time consuming but inexpensive. *Cob* is a variation to adobe that uses a similar mix to *adobe block*, but uses a little more straw, and skips the step of making individual blocks. *Cob* walls are formed by adding lumps of adobe directly to the wall and then working them in to create a seamless wall. *Cob* walls work best when a new course can be laid every day so new layers can be more easily integrated into the previous layer. (Nunan 2010)

Rammed earth is the process of tamping earth into formwork in layers. The consistency of the mix and moisture levels is more important for *rammed earth* systems than adobe. Too much moisture leads to shrinkage in the walls as they dry causing cracks, while too little moistures creates weak walls as the soil does not bond during the tamping process. *Rammed earth* can be tamped with hand rammers or pneumatic tampers. Forms are removed immediately after the molds have been filled and lifted in preparation for the next course. (Nunan 2010) *Stabilized rammed earth* is a mix that relies on the addition of cement to strengthen the walls and increase erosion resistance. For example, the use of 10 percent cement in the SIREWALL *stabilized rammed earth system* showed a 610 mm

(24”) SIREWALL is 13.5 times stronger than a standard 203 mm (8”) concrete wall (SIREWALL 2015).

Compressed Earth Block (*CEB*) is a system designed to increase the compression strength of soil blocks. Their uniform shape is advantageous for construction and they can be shaped to interlock with each other. Unlike *adobe block, CEBs* can be used a day after being formed and contain lower moisture content than *adobe or cob*. Machines for the creation of *CEBs* are manually operated or automated and simple to use. (Nunan 2010)

3. Materials and Methods

3.1. Individual Component Preparation

The wall system relied on the properties of several materials and components to work together. Hence the evaluation of individual components promoted their optimization when combined into the system.

3.1.1. Concrete Filled Plastic Tube Columns (CFPTs)

3.1.1.1 CFPT Construction

To limit the variables and loads placed upon the earth wall system, the decision was made to utilize CFPTs to transfer the axial roof loads to the foundation. The CFPTs also provided additional racking and transverse resistance but the testing was limited to axial compressive testing. Initial design concepts for the earth wall system included insertion of 13 mm (½”) PVC tubes through 102 mm (4”) PVC to create tie-through locations for cabling or threaded rod as necessary. Therefore the columns tested included 13 mm (½”) PVC inserts at 127 mm (5”) spacing aligned 90° to each other as shown in Figure 3, and a single piece of longitudinal 9.5 mm (3/8”) steel rebar.


Figure 3: CFPT with 13 mm (½") PVC Inserts

Three rectangular boxes made of nominal 51 mm x 102 mm (2" x 4") lumber with inside diameter equal to the outside diameter of the 102 mm (4") PVC were placed upon a flat working surface to keep the 13 mm (½") PVC inserts in line and 90° to each other. One box was secured in place at each end while the third box with a 19 mm (¾") pilot hole was free to move along the pipe to the locations where holes were to be drilled for the inserts as shown in Figure 4.

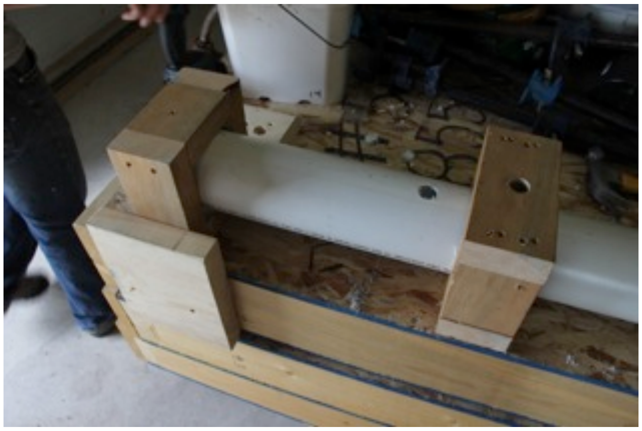
Figure 4: Jig for Aligning and Drilling Holes in PVC Tube

Four 2.4 m (8') CFPTs were cast for axial compressive tests. To study the effects of different mixes on ability to pour and on capacity, two CFPTs were cast with typical A-base gravel and two with clean aggregate and sand. The availability of clean aggregate or the ability to clean it if necessary can be limited in Honduras hence the decision to test different mixes. Both mixtures had the same aggregate to cement ratio. The straight A-base was mixed 3:1 (A-base: cement) and the clean aggregate 2:1:1 (aggregate: sand: cement). Type 10 general use hydraulic cement was used for the mix.

Table 1: A-Base Gravel & Clean Aggregate for Concrete Proportions

Passing Standard Sieves	**Typical A-Base Gravel Proportion Ranges (%) (1)**	**Course Aggregate for Portland Cement Concrete (%) (2)**
25 mm		100
19 mm	100	90 – 100
16 mm	80 – 100	
9.5 mm		20 – 55
4.75 mm	40 – 70	0 – 10
2 mm	25 – 55	
425 um	15 – 30	
75 um	8 - 15	0 - 2

Note 1: Source: Manitoba Infrastructure and Transportation 2002
Note 2: Source: Manitoba Infrastructure and Transportation 1986

A funnel was attached to a standard 102 mm (4") PVC coupler set on top of each column to facilitate easier pouring from 20L pails. Consolidation of the concrete was done by lightly tapping the columns with a rubber mallet as it was being poured. The columns were poured in a vertical position with a 4" PVC end cap placed on the bottom end. An electric blade mixer was used to mix the concrete as shown in Figure 5.

Figure 5: Mixing Concrete with Electric Blade Mixer

3.1.1.2 CFPT Axial Test Setup

The four CFPTs were tested in axial compression after seven days of curing. Hinge connections at the two ends were manufactured using 51 mm (2”) hitch balls held in position by a steel ring on the bottom and a bolted clip assembly at the top as seen in Figure 6.

Figure 6: CFPT Axial Test Bottom (L) & Top (R) Hinge Connections

An initial test was completed on an empty 8’ long 4” PVC tube with four linear potentiometers spaced at 90° around the mid-span of the column (4’ height) to monitor displacement and one linear potentiometer secured to the load cell to monitor crosshead movement (graph found in Appendix F). The linear potentiometers at mid-span were extended to touch the column prior to testing but not attached. Column failure did not occur parallel to any of the linear potentiometers so data was not available for mid-span deflection.

The four CFPTs were restrained to movement parallel to the orientation of the test frame so mid-span deflection data could be collected safely without damaging linear potentiometers as seen in Figure 7. Two linear potentiometers were tied to the mid-span of the column using tie wire, one linear potentiometer at each ¼ point from the top and bottom, and a linear potentiometer on the crosshead.

Figure 7: CFPT Axial Test Restrained Movement

3.1.2. PET Bottle Orientation Compression Tests

PET bottles were tested in axial compression with the bottles oriented vertically and horizontally to aid in determining if the bottles should be stacked vertically or horizontally. The axial load on each bottle is minimal in the wall system as the lightweight bottles are stacked on top of each other. A variety of bottles were tested in both unrestrained and restrained conditions in horizontal orientation as shown in Figure 8. The restrained condition was a wood box to hold the sides of the bottles to simulate the packed earth on either side of the bottle in the wall system. All bottles were capped for the tests by first removing the caps to allow the bottles to refill with air if necessary and then hand tightened. The PET bottles tested in the vertical orientation were all tested without sidewall restraint as seen in Figure 8.

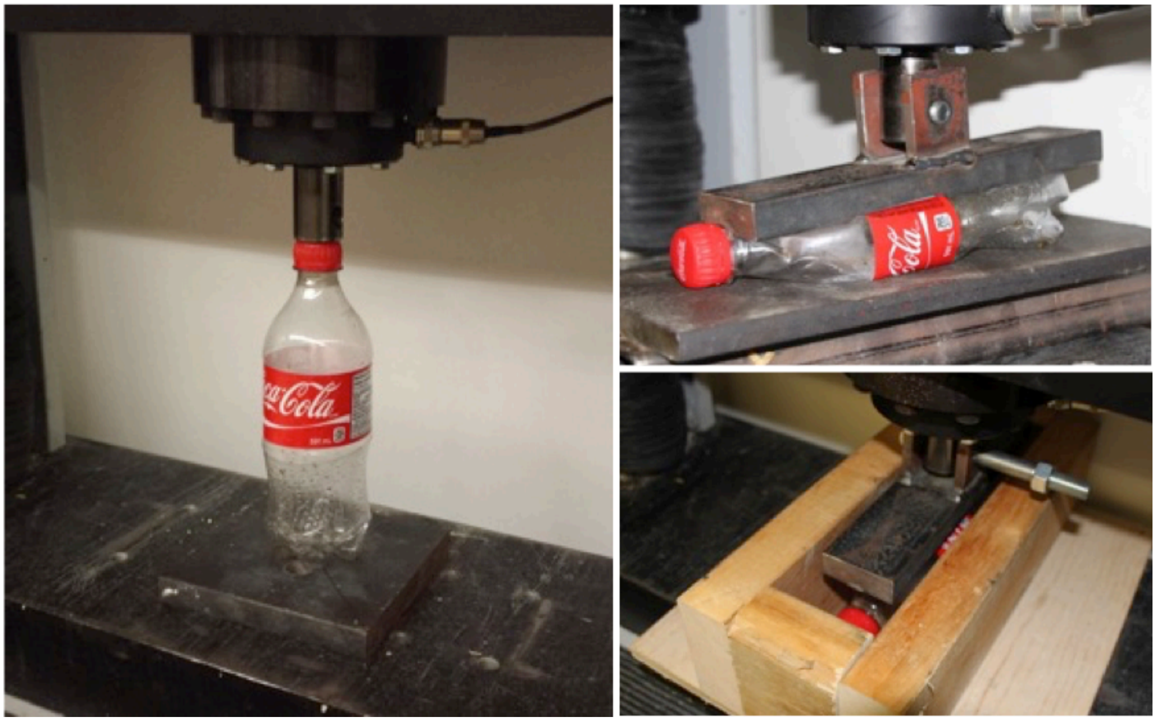

Figure 8: Vertical (L) and Horizontal (R) PET Bottle Compression Tests

3.1.3. Earth Mix

The soil for the project was from Anola, MB and contained 45% sand, 35% silt and 20% clay (Griffiths 2007). This soil falls in the recommended proportional ranges for earth masonry as seen in Table 2.

Table 2: Earth Mix Proportions

Particle	Recommended Proportion Range (%) [(1)]	Anola Soil Proportion (%)
Clay	5 - 20	20
Silt	20 - 40	35
Sand	25 - 50	45

Note 1: (Morton 2008)

The soil was stored in a greenhouse to dry and shelter it from rain. It was sifted for large aggregate and other organics using a 25 mm (1") square mesh screen as shown in Figure 9.

Figure 9: Sieving Earth With 1" Square Mesh Screen

Wheat straw was added to the earth to promote bonding in the earth mix. The straw was cut into lengths roughly 152 mm (6”) and smaller by placing straw into an aluminum garbage can and chopped using a weed trimmer as shown in Figure 10.

Figure 10: Chopping Wheat Straw with Weed Trimmer

Six earth block samples were constructed to test the axial capacity of 228 mm (9”) and 254 mm (10”) wide walls after 1, 3 and 5 days of drying for implications on the overall width of the wall and allowable rate of placing additional earth lifts. The minimum width of the walls was a function of space for 76 mm (3”) diameter PET bottles, cabling on either side of the bottles, and 51 mm on both exterior faces for ramming adobe. 228 mm wide walls were accepted as minimum width but block samples of 254 mm wide were also conducted for strength comparison. The samples were built according to the illustration seen in Figure 11. The earth and straw for these samples were dry mixed in a metal container using a hoe. Water was added until small lumps of earth roughly 25 mm (1”) in diameter started to form while mixing. The earth to straw ratio was roughly 14:1 with 10% water content.

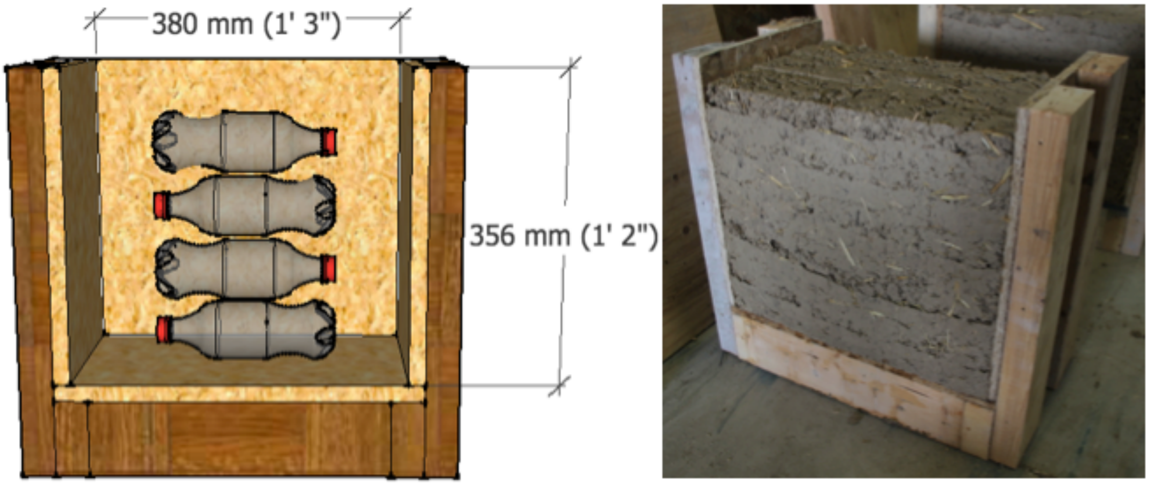

Figure 11: Infill Block Configuration (L) and Prototype Sample (R)

The infill block samples were tested axially using a hydraulic cylinder and loaded at a rate of 1.5 mm/min. A 6 mm (1/4") thick piece of rubber was placed between the earth and the steel plate attached to the push cylinder to distribute the load more evenly. The steel plate was 229 mm (9”) in length, to limit testing to the length of the pop bottles in the block sample. Crosshead movement was measured using a linear potentiometer. Sidewall displacement was manually measured by pausing the test at various intervals determined during the tests by observing the behavior of the samples under load as indicated in Table 3. Twelve measurements were taken across each face of the block for sidewall displacement. The blocks were constrained in the longitudinal length mimicking in-wall behavior as shown in Figure 12.

Table 3: Infill Block Axial Test Load Increments

Test ID	Load Increments (N)					
	1	2	3	4	5	6
1 Day - 9"	1557	2002	2224	-	-	-
1 Day - 10"	2224	2669	2745	-	-	-
3 Day - 9"	1557	2224	3336	4448	5560	-
3 Day - 10"	1557	2224	3336	4448	5560	6339
5 Day - 9"	2358	4448	6672	8896	11121	12442
5 Day - 10"	4448	6672	8896	11121	13345	13878

Figure 12: Infill Block Axial Test

3.1.4. Aircraft Cable Connection & Tensioning

3.1.4.1 Cable & Aluminum Cable Sleeve Strength

Racking resistance in the wall was provided by 3 mm (1/8”) (7x7) galvanized aircraft cable of a rated 7562 N (1,700 lb) breaking strength. The cables were clamped using aluminum cable sleeves and the connection was tested to determine if one sleeve was sufficient or if two were required as seen in Figure 13. Plotting tension vs. displacement during the tension test provided the basis for determining an appropriate tension of the cables in the wall system.

Figure 13: Single (L) and Double (R) Aluminum Cable Sleeve Connection Test

3.1.4.2 Aircraft Cable Tensioning using Spoke Tension Meter

Aircraft cable was fed through 16 mm (5/8”) chain link hung on the 9.5 mm (3/8”) threaded rod placed through and cast into the CFPTs, then tightened using 13 mm (1/2”) galvanized turnbuckles. The tension in the cables was measured using a bicycle spoke tension meter as shown in Figure 14. For the development of the calibration curve used for the spoke tension meter see Appendix A.

Figure 14: Tensioning Aircraft Cable with Bicycle Spoke Tension Meter

3.2. Wall Panel Construction

3.2.1. Footing

Eight footings were poured using a 3:1 (A-base gravel: cement) ratio, 1.8 m x 0.3 m x 0.1 m (6' x 1' x 5.5") (l x w x h) as seen in Figure 15. Concrete was consolidated using the end of a trowel. Rebar dowels protruded a foot out of the concrete at column locations. 152 mm (6") long PVC unions cut from the connection end of 102 mm (4") pressurized PVC tubes were placed into the concrete footings at 1.2 m (4') on center before the concrete set as seen in Figure 15. The unions protruded 76 mm (3") above the footing and leveled using a 1.2 m (4') level.

Figure 15: Empty Concrete Footing (L) & Poured With 102 mm (4") PVC Unions (R)

3.2.2. Columns

The 102 mm (4") PVC columns were preassembled with the 9.5 mm (3/8") threaded rod, 8 mm (5/16") chain and wood spacers prior to placement in the unions cast into the footings. The PVC was cut 2.36 m (93") long to allow for 25 mm (1") protrusion into the 102 mm (4") concrete bond beam resulting in a 2.4 m (8') high wall. 19 mm (3/4") diameter holes were drilled in the PVC for the threaded rod using the jig seen in Figure 16 to ensure holes were 180° to each other and at the spacing indicated in Figure 17.

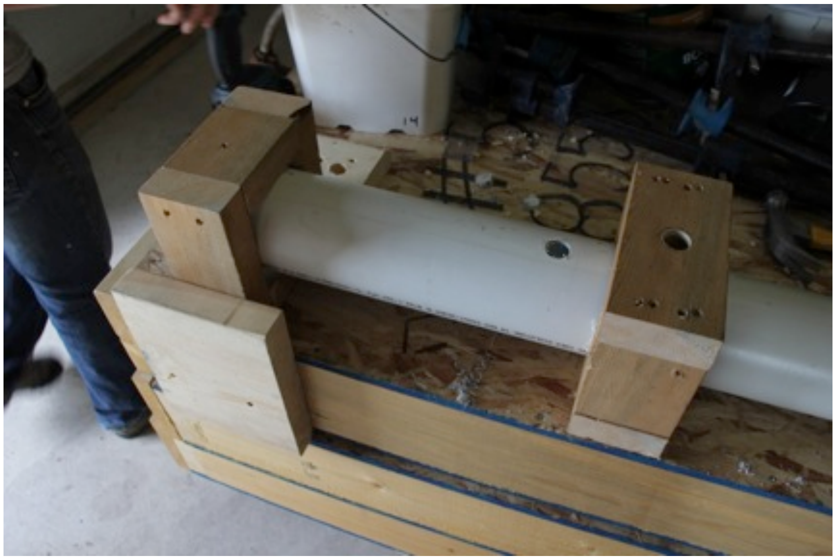

Figure 16: Jig for Accurate Drilling of Holes in PVC Tubes

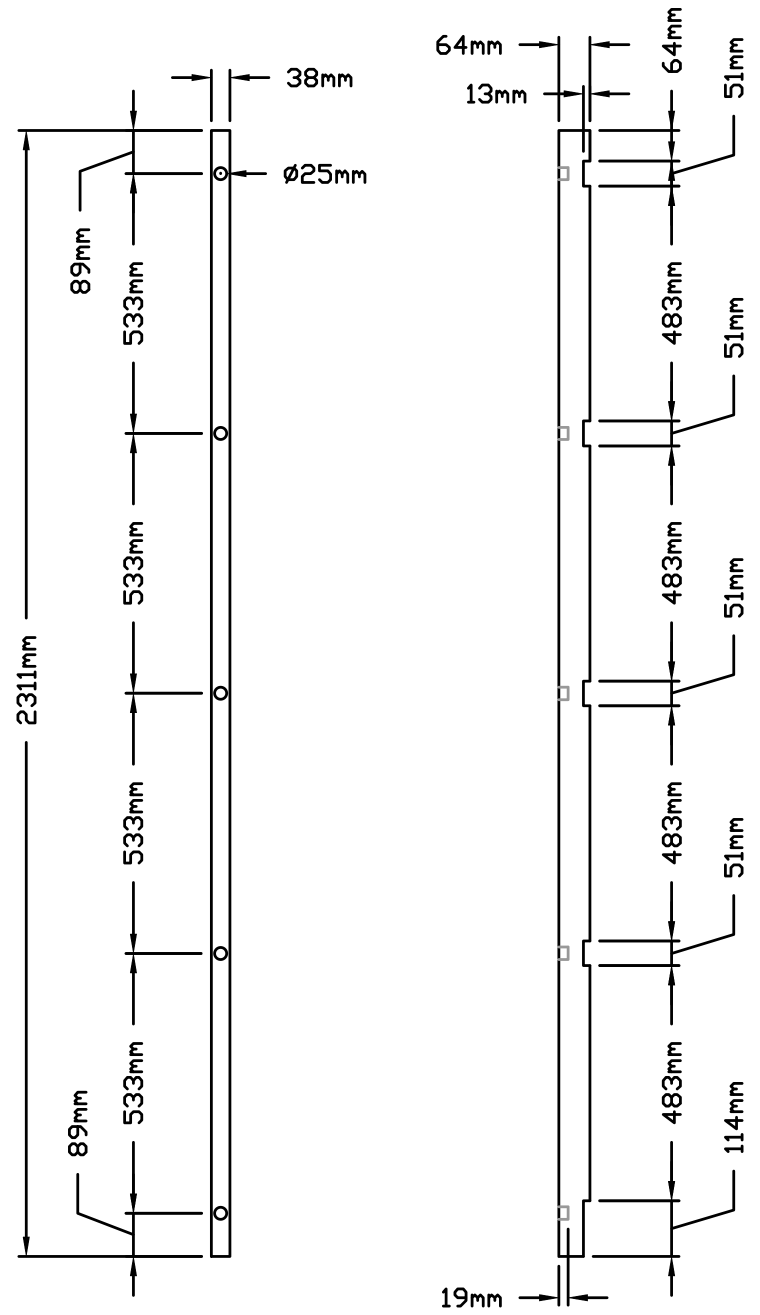

Figure 17: Wood Cutout Detail
(Note: 1 mm = 0.03937 in)

The wood was notched for the chain, washers and nuts needed for securing the wood on the threaded rod as shown in Figure 18. The wood was cut to 2.31 m (91") in length leaving a 13 mm (1/2") space between the wood and footing and the wood and bond beam.

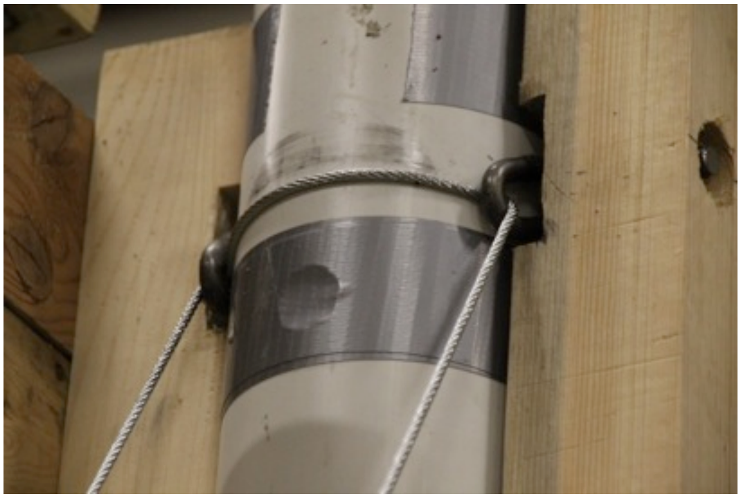

Figure 18: Wood Detail Allowing for Cable Connections

A 610 mm (2') dowel was tied to each 305 mm (1') dowel previously cast in the footing. Assembled columns were then placed in the unions cast into the bond beam, plumbed and braced before being cast with 4:1 (A-base: cement) concrete. Concrete was vibrated using a rubber mallet on the exterior of the columns. The columns were filled to within 25 mm (1") of the top and a 9.5 mm (3/8") rebar dowel was then inserted 610 mm (2') into the top of the column.

3.2.3. Aircraft Cable

Aircraft cable was run through the single chain links at the top and bottom of the columns in a continuous loop as shown in Figure 18. The cable was tied together using 16 mm (5/8") turnbuckles and one aluminum cable sleeve on each connection that allowed the cable to be tensioned to 445 N (100 lbs) using the spoke tension meter.

3.2.4. Infill Mixing, Layering & Compaction

The earth, straw and water were mixed using an electric mortar mixer to create an adobe mix. Earth and straw were dry mixed prior to adding the desired amount of water. The adobe and air-filled PET bottles (all capped) were layered and compacted according to the schematic seen in Figure 2 so section 1.4.1. Soil was compacted with a nominal 51 mm x 102 mm (2” x 4”) thickened using a piece of 13 mm (1/2”) OSB to compact the outer 51 mm (2”) of the wall system on both faces. A nominal 51 mm x 102 mm (2” x 4”) to compact the outside edges 38 mm (1 1/2") was used at the locations of the barbed wire. The earth layer penetrating through the wall at the barbed wire locations was lightly packed by laying a nominal 51 mm x 102 mm (2” x 4”) on it’s face and pressing firmly resulting in a flat surface from which to restart layers of pop bottles. The following steps were used to fill a typical 230 mm (9”) lift, not containing barbed wire: (note: mix refers to the mixture of earth, straw and water)

1. Set 305 mm (12”) wide forms so bottom is overlapping previous lift by 13 mm (1/2") (Note: forms worked best when left dry).
2. Placed 51 mm (2”) of loose mix evenly in the form (by hand or poured with a 5 gallon pail).
3. Pulled mix away from the center of the wall to the edges creating a V.
4. Placed bottles horizontally, end to end in the V.
5. Filled the form level with the top of the bottle layer (resulting in a 76 mm (3”) lift of earth).
6. Packed the outside edges of the earth by firmly tamping with the wood block.
7. Refilled the form with mix above the existing bottles and repeated steps 2-6 for the 2nd and 3rd layers of bottles.
8. Removed the forms immediately after completing the 3rd layer of bottles and compaction that resulted in a 9” lift.

The top 230 mm (9”) lift was compacted flat using the same method used at the barbed wire locations in preparation for the concrete bond beam. Figure 19 shows stages of the infill layering and compaction technique.

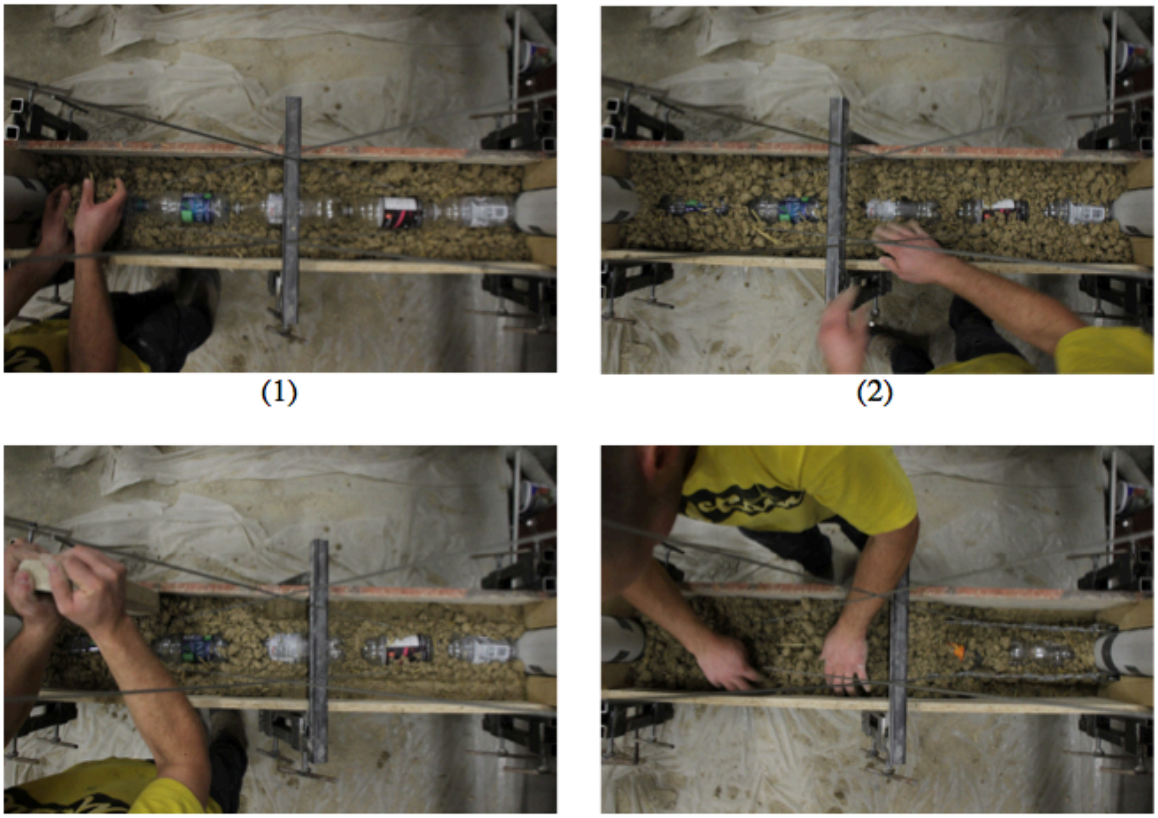
(1) (2) (3) (4)

Figure 19: Infill Construction - Bottle Placement (1), Earth Fill Level With Bottles (2), 51 mm (2”) Edge Compaction (3), Covering Barbed Wire (4)

3.2.5. Bond Beam

The 102 mm (4”) thick bond beam was cast using a 4:1 (A-base: cement) ratio the width of the 230 mm (9”) wall and 1.5 m (5’) long to fully encase the columns in the bond beam as seen in the formwork for the bond beam in Figure 20. Concrete was consolidated using the end of a trowel.

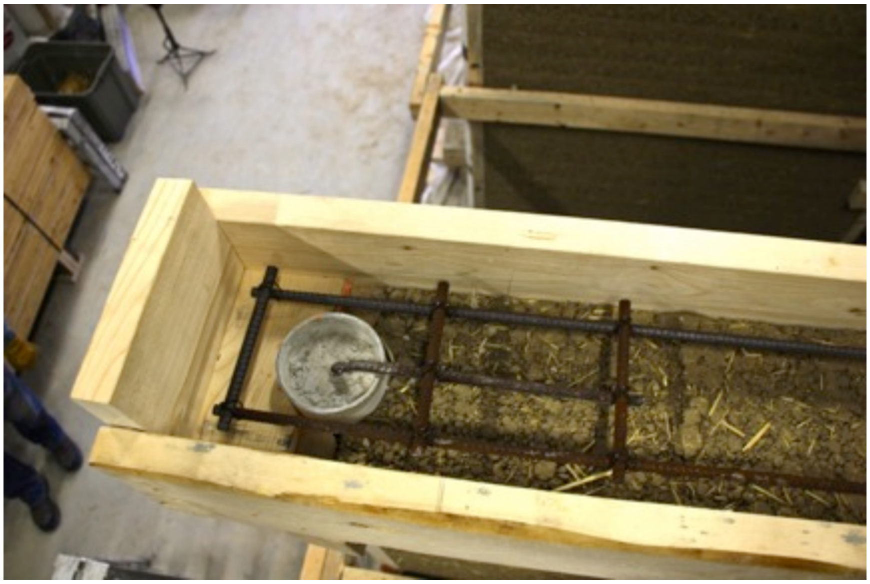

Figure 20: Bond Beam Formwork and Rebar

3.3. Cyclic Racking Test

A total of four cyclic racking tests were completed inside of a steel test frame designed for testing panels. The first test was performed on a wall system without the earth infill and horizontal barbed wire reinforcement. The other components (footing, columns, aircraft cable and bond beam) were identical in construction to the wall systems with infill. The remaining three tests were performed on the complete infill wall system.

Cyclic racking tests were conducted by adapting ASTM E2126−11 (ASTM International 2014) cyclic load tests. The rate of displacement was 1.1 mm/s in the push and 5.1 mm/s in the pull. The valve assembly controlling the rate of displacement used during the test was only manufactured to control the hydraulics in the push direction hence the disparity in the push and pull rate. Cycles were phased in displacement increments of ±12.7 mm (1/2”) with 3 cycles at each increment as shown in Figure 21. The wall system without infill and the first complete wall system were tested to ±51 mm (2”) because pull back distance on the hydraulic cylinder used for inducing the load was limited to 63 mm (2.5”). Modifications to the test setup increased the maximum pull back distance to 102 mm (4”) allowing for the last two earth walls to reach ultimate failure. All racking tests experienced the first 12 cycles, but only the last two tests were stopped after ultimate failure was observed as demonstrated in the line change seen in Figure 21.

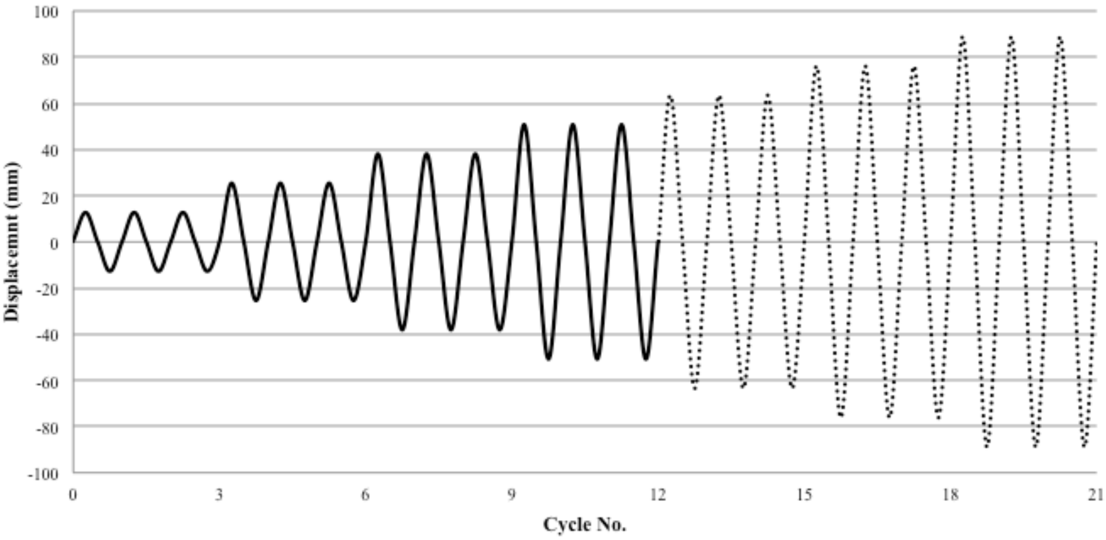

Figure 21: Cyclic Racking Cycles & Displacement Increments

The footings were fixed in place to prevent any uplift or lateral movement during the test as seen in Figure 22. Linear potentiometers were attached to the bottom of the columns to monitor any uplift separation of the columns from the footing as seen in Figure 22. Steel supports placed loosely along the length of the bond beam protected the walls from lateral movement perpendicular to the applied load as seen in Figure 23.

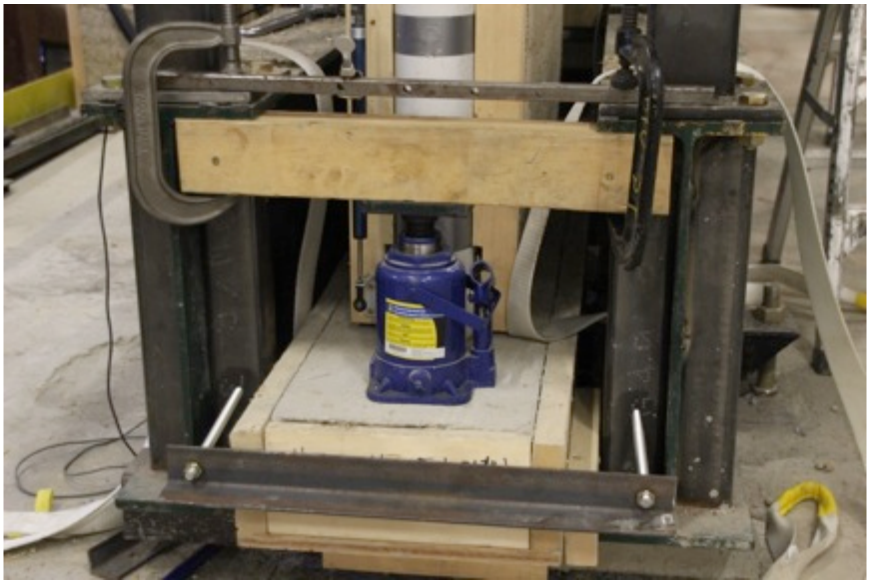

Figure 22: Cyclic Loading Footing Restraint and Linear Pot

Figure 23: Lateral Support on Bond Beam During Cyclic Racking

The cylinder was attached to the bond beam using pin connections to allow for possible rotation from uplift on the wall during cyclic loading. A linear potentiometer was attached to the load cell on the cylinder to record the rate of displacement. Two 152 mm (6”) I-beam sections were bolted together to sandwich the bond beam using 13 mm (1/2”) threaded rod allowing for cyclic loading to occur. Figure 24 shows the securement of the bond beam to the cylinder and the linear potentiometer to the load cell.

Figure 24: Cyclic Loading I-Beam Connection to Bond Beam

3.4. Airbag Transverse Test

The impact of wind loads on the wall system was completed using an airbag pressure system. The tests were conducted by adapting ASTM E72-13a (ASTM International 2014) for conducting strength tests of panels for building construction. A wooden box frame was built to contain the pressure bag made from 6 mil vapor barrier and secured to the wall system using pipe clamps to the bond beam and footing as seen in Figure 25. Air pressure inside of the pressure bag chamber was measured using a pressure gauge with a range of 0-5 psi and recorded using a data acquisition system. A 20 gallon air compressor was used to fill the pressure bag chamber and flow rate was controlled using a standard air compressor regulator valve set at 30 psi. Five linear potentiometers were used to measure the displacement of the earth wall on the opposite face of the loaded surface. The wall systems were not loaded to ultimate failure to avoid damaging the pressure bag system. The test was stopped when the pressure inside the chamber reached 257 psf (well above the design load for the wall system of 32 psf) equaling half the designed capacity of the wood frame containing the pressure bag as calculated in APPENDIX C.

Figure 25: Airbag Assembly Pressurized (L) and Secured to Wall System (R)

4. Results

4.1. PET Bottle Strength

Figure 26 contains the results of testing bottle strength as described in Section 3.1.2. The graph ends at a crosshead displacement of 35 mm (1.4”) as results varied greatly beyond that measure and is nearly 50% of the bottles initial height of 76 mm (3”) (when bottles lay horizontal).

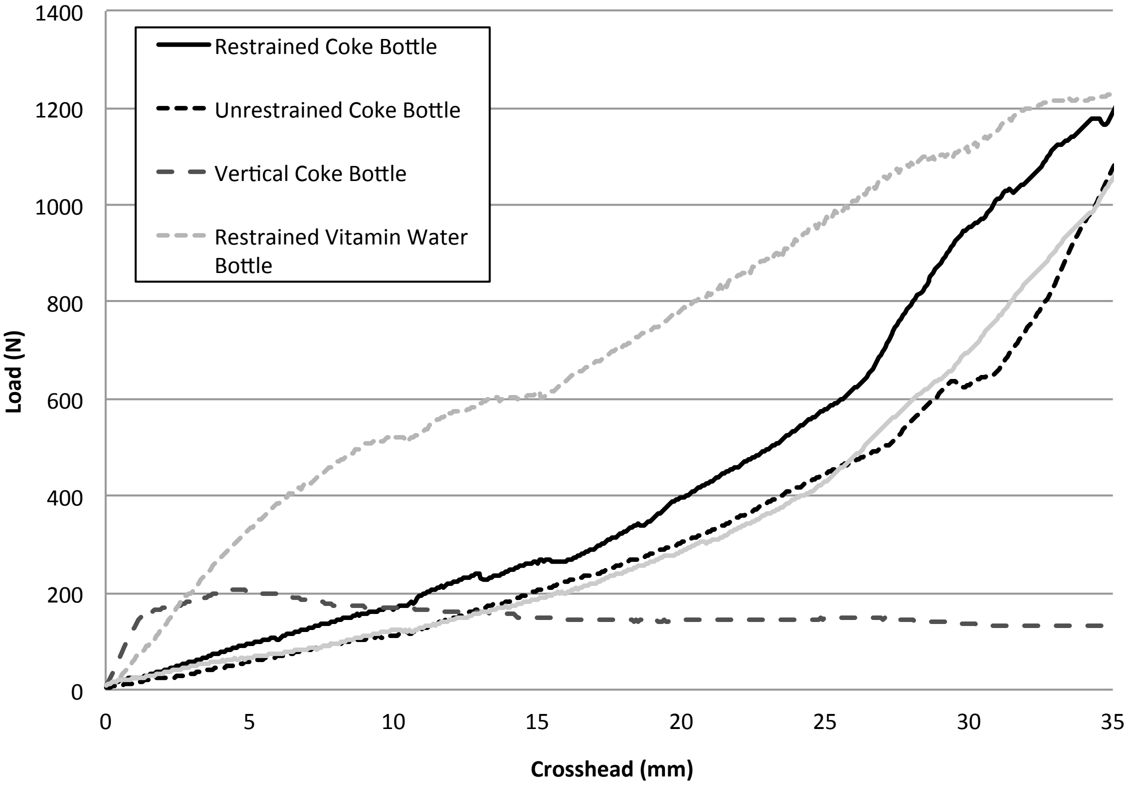

Figure 26: PET Bottle Compressive Strength

4.2. CFPT Axial Capacity

The axial capacity of the four CFPTs tested according to Section 3.1.1.2 are shown in Figures 27 & 28. Two serviceability limits of L/180 and L/360 (L = 2.4 m or 8') are included in the graphs.

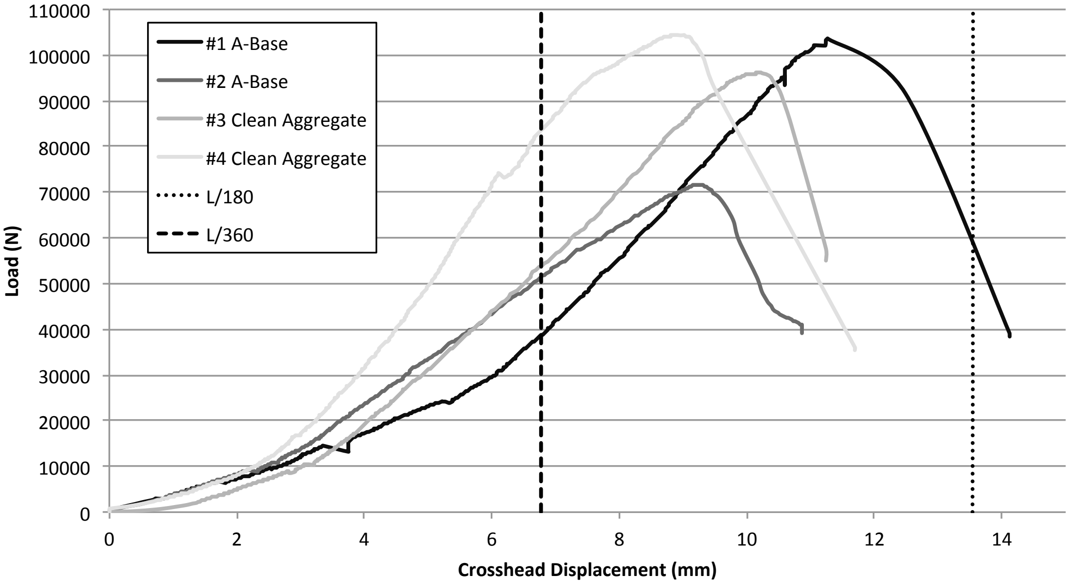

Figure 27: CFPT Column Axial Tests – Crosshead Displacement

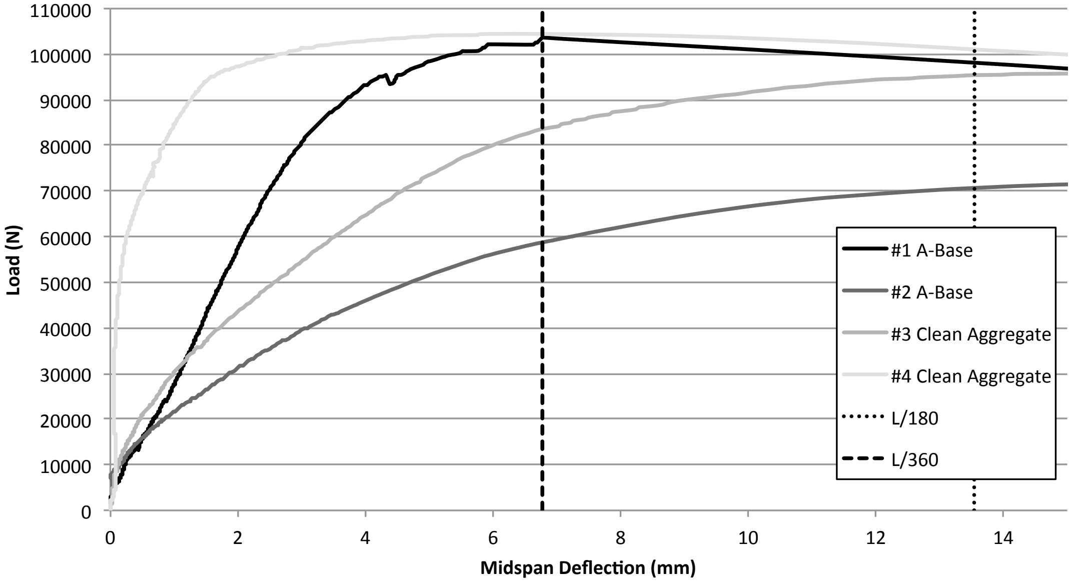

Figure 28: CFPT Column Axial Tests – Mid-span Deflection

4.3. Infill Block Axial Capacity

Figures 29 & 30 show axial capacities of the infill block samples described in Section 3.1.3. The dips in the plots resulted from times when loading was paused to allow for manual displacement measurements on the sides of the samples.

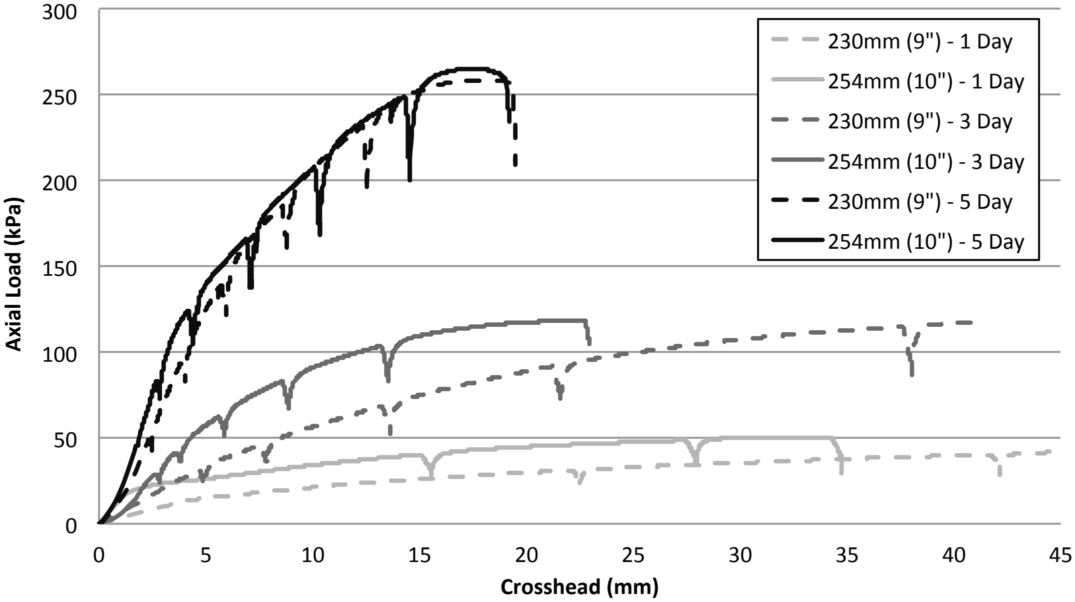

Figure 29: Infill Block Axial Tests: 1 Day, 3 Day, & 5 Day Curing

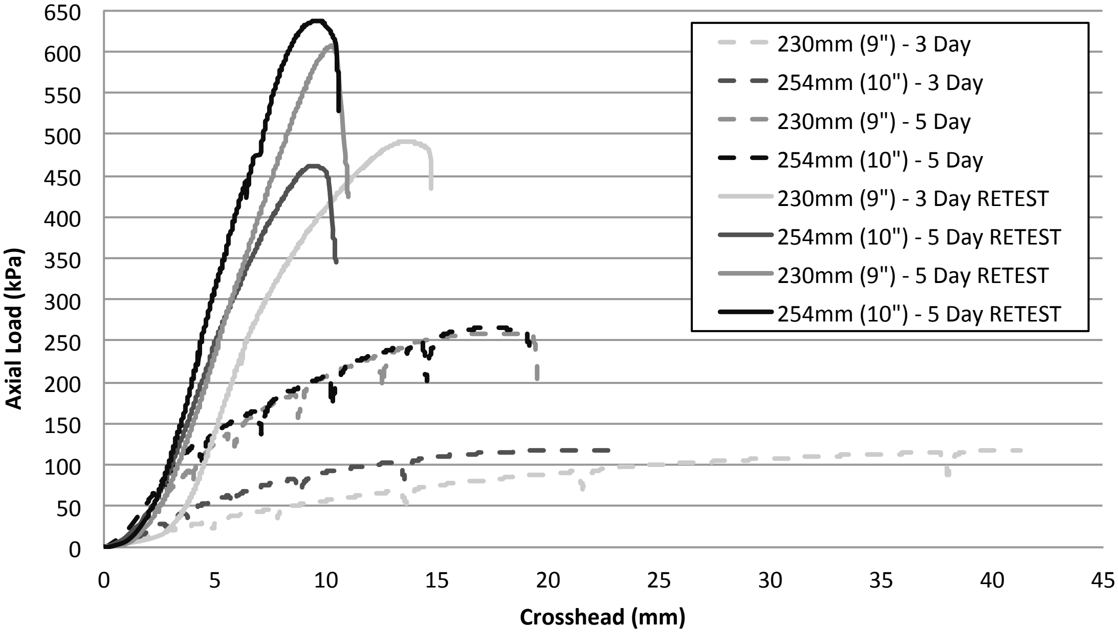

Figure 30: Infill Block Axial 3 & 5 Day Tests with Retests on Day 10

The manual measurements of sidewall displacement during the axial loading allowed Poisson's ratio to be calculated at various loads as seen in Figure 31. Transverse displacement versus axial load is shown in Figure 32. The values used for Figures 31 & 32 are the maximum displacement recorded for each load increment.

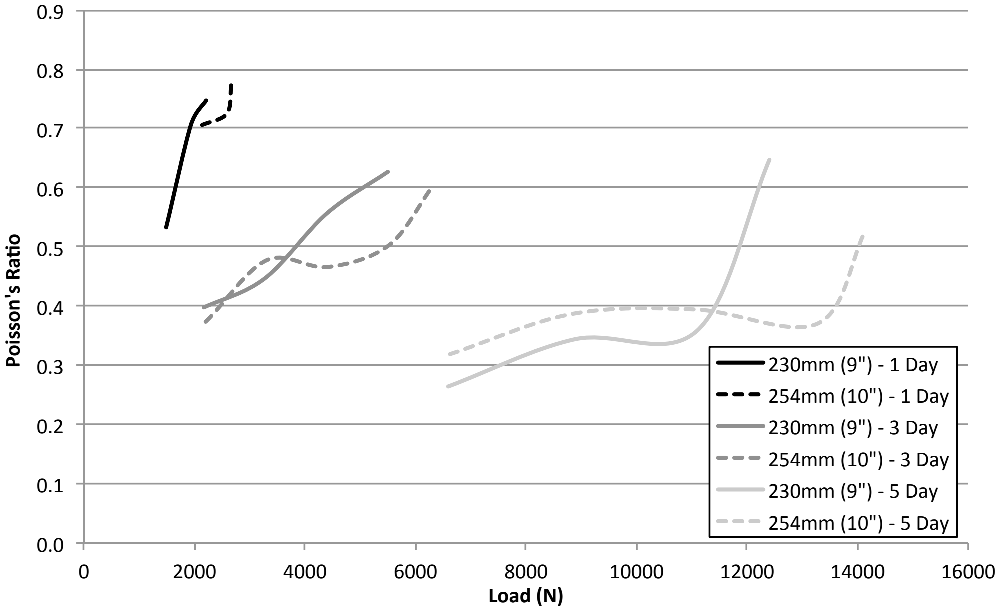

Figure 31: Infill Block Axial - Poisson's Ratio

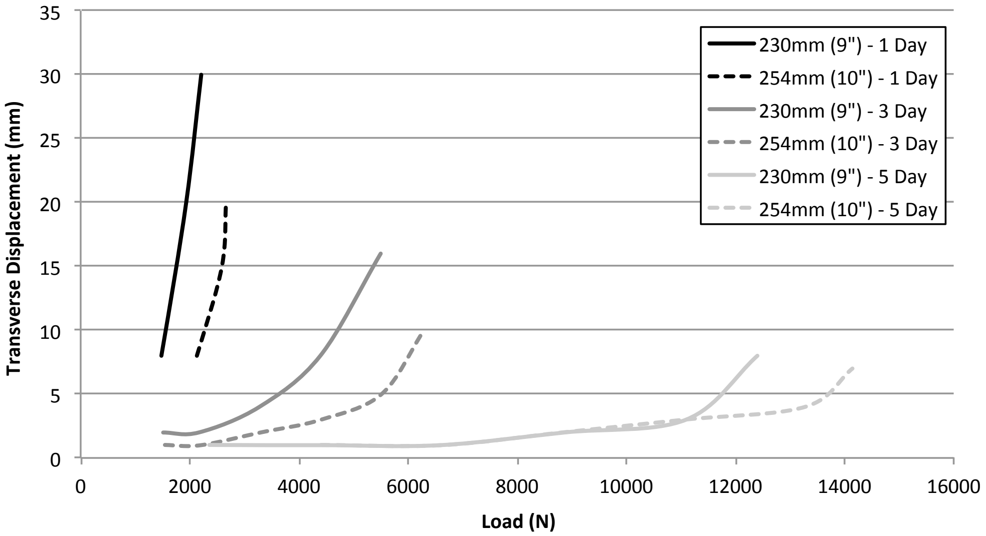

Figure 32: Infill Block Transverse Displacement Under Axial Load

4.4. Aircraft Cable Tensile Capacity

Figure 33 shows the ultimate capacity of the aircraft cable tested with single or double aluminum sleeve connectors as described in Section 3.1.4.1. The graph also indicates the rated breaking strength of the cable.

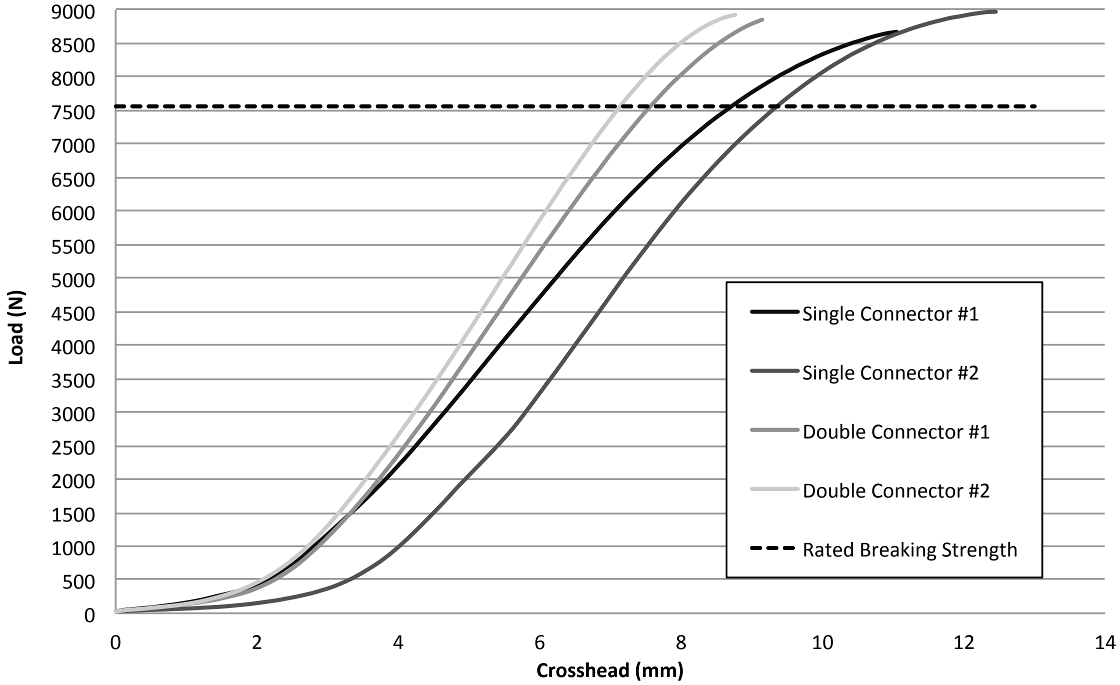

Figure 33: Aircraft Cable Tension Tests for Single & Double Connectors

4.5. Cyclic Racking Strength

Envelope curves from the cyclic racking tests as described in Section 3.3 are shown in Figure 34. The figure also includes the racking capacity of three 165 mm (6.5") stud wall assemblies constructed and tested at the Alternative Village. The stud walls were constructed in accordance with Canadian building standards using studs spaced at 610 mm o.c. (24") and finished with 13 mm (1/2") oriented strand board (OSB) and 13 mm (1/2") standard gypsum drywall. They were racked according to ASTM E72 (ASTM

International 2014), (Delijani and Dick 2013). Figure 35 shows the detailed cyclic loading of the second complete wall system that was racked. The detailed first and third wall system cyclic racking graphs are found in APPENDIX D.

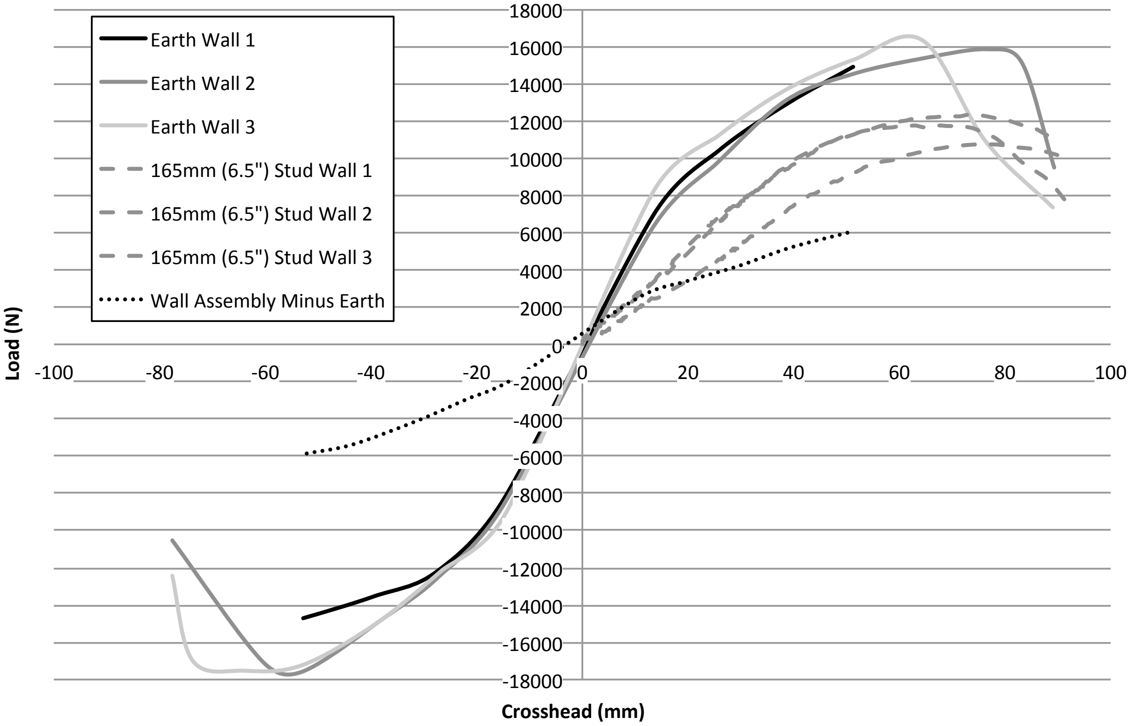

Figure 34: Racking on Various Wall Assemblies

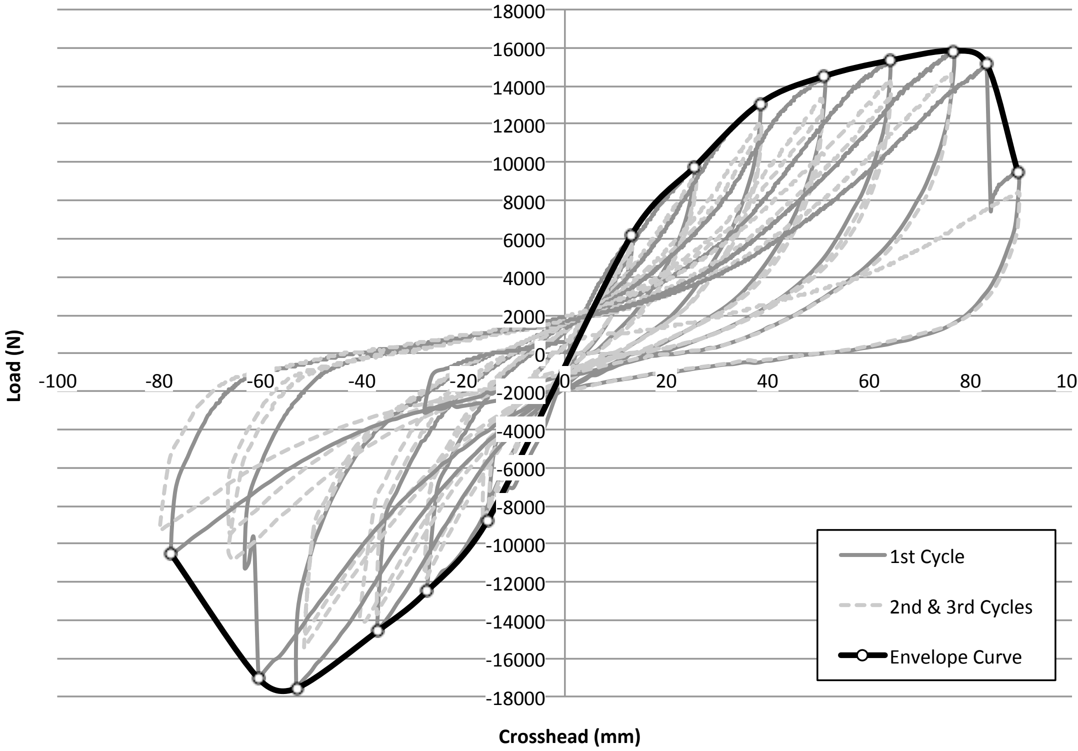

Figure 35: Detailed Cyclic Racking – Wall System 2

4.6. Transverse Strength

Figure 36 shows the results of the airbag transverse tests completed on three wall sections as described in Section 3.4. The figure includes transverse loading of three 165 mm (6.5”) stud wall assemblies constructed and tested at the Alternative Village as described in Section 4.5. The stud walls were subjected to transverse load according to ASTM E72 (ASTM International 2014), (Delijani and Dick 2013). As air pressure was released from the airbag, regression of the wall system was measured by manually placing the potentiometers against the rebounding wall at various intervals. Figure 37 includes data from all five linear potentiometer locations acting on the first wall system tested. APPENDIX E contains the detailed transverse results of the other two wall systems. The linear potentiometer located 305 mm (12”) from the top in wall 1 and 2 was moved from the horizontal center of the wall to a column for wall 3.

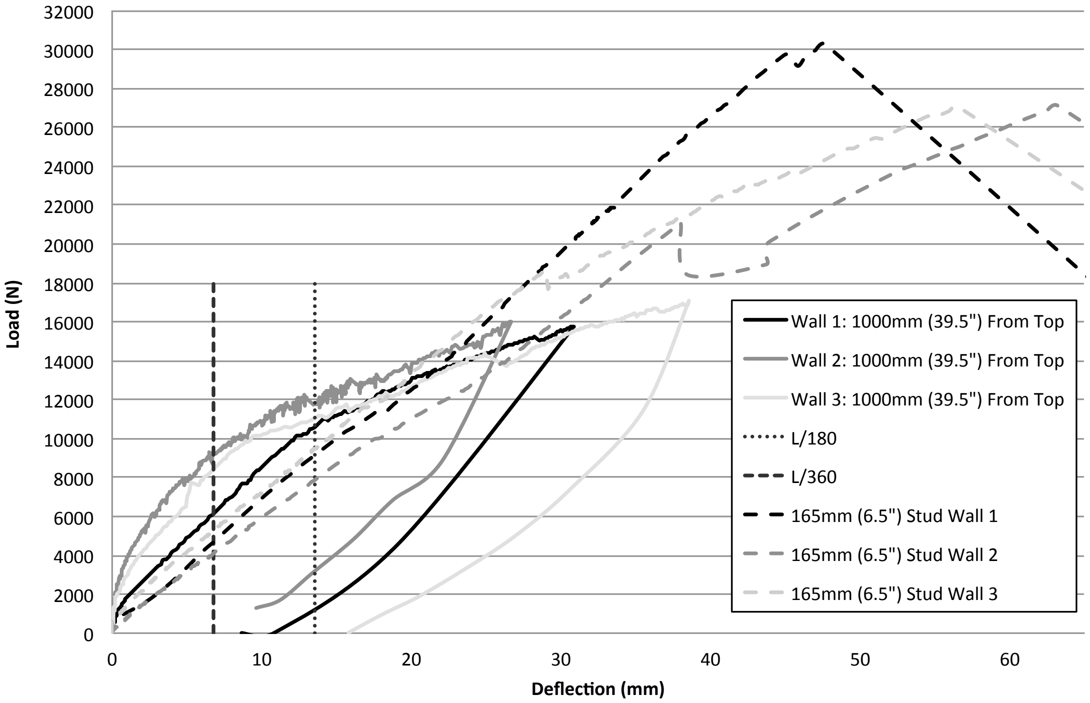

Figure 36: Transverse on Various Wall Assemblies

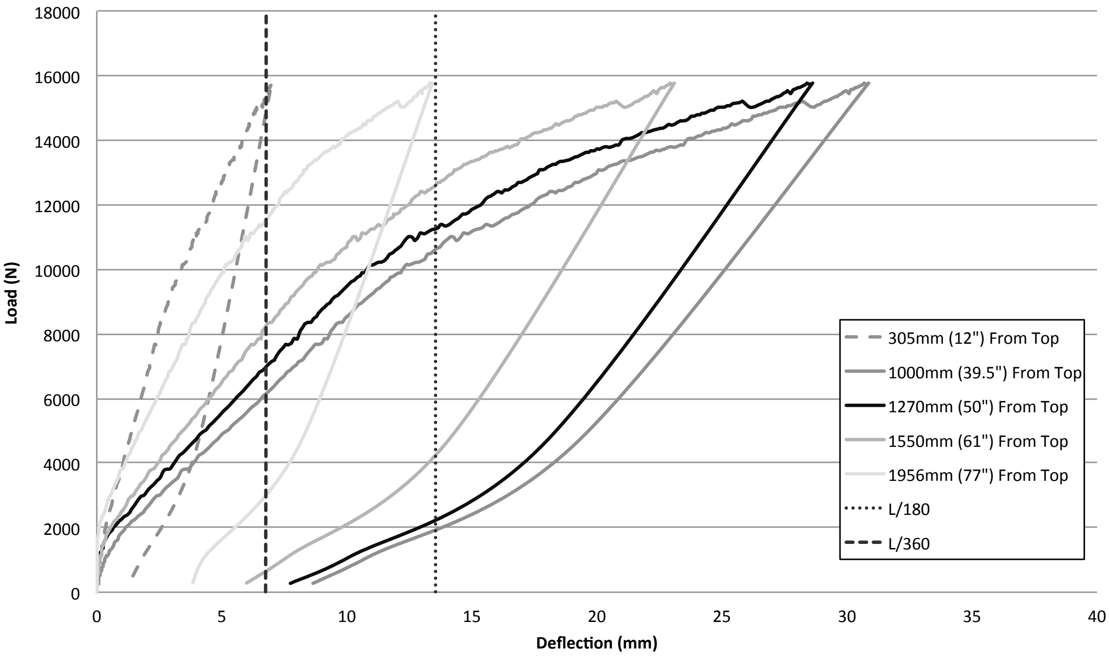

Figure 37: Transverse Test Wall 1

4.7. Summary of Test Results

Table 4 summarizes the results pertinent to the wall systems capacity to handle the axial, racking and transverse design loads. Numbers in bold are the values used to calculate the serviceability limit factors and the ultimate factors of safety relative to the design loads. The design loads are factored but the tested values are direct results of the testing and do not contain capacity reduction factors. Therefore the factors of serviceability and safety are multiplication factors showing allowable variation in construction of the wall system's ability to handle the design loads. Calculations for the design loads are found in Appendix G.

Table 4: Infill Wall System Design Loads & Capacities Summary

TEST ORIENTATION	DESIGN LOAD (kN)	AVERAGE MAX. LOAD (kN)	LOAD @ L/180 (kN)			LOAD @ L/360 (kN)			SERVICE-ABILITY FACTOR	FACTOR OF SAFETY
			Min	Avg	Max	Min	Avg	Max		
AXIAL										
102mm (4") PVC Column	22.9	**93.92**	-	-	-	**30.28**	48.16	73.43	**1.32**	**4.10**
165mm (6.5") Stud Wall (1) (3)		157.40								
RACKING										
Earth Wall System	4.47	**17.05**	**5.69**	7.56	8.63	3.64	4.75	5.33	**1.27**	**3.82**
165mm (6.5") Stud Wall (1) (3)		11.61	2.47	3.08	3.45	1.18	1.58	1.84		
Wind Speed (mph) (2)		268	155			124				
TRANSVERSE										
Earth Wall System	4.55	N/A	**10.61**	**11.15**	11.77	6.15	7.86	9.22	**2.33**	**2.45**
165mm (6.5") Stud Wall (1) (3)		28.1	7.88	8.83	9.47	4.09	4.68	5.26		
Wind Speed (mph) (2)			241	247		184				

Note 1: Nominal 51mm x 152mm (2" x 6") @ 610mm (24") o.c. sheeted with 13mm (1/2") Oriented Strand Board & 13mm (1/2") Drywall

Note 2: Wind speed calculated for earth wall system loads

Note 3: (Delijani and Dick 2013)

5. Discussion

5.1. PET Bottle Strength & Orientation

As a result of the compressive strength testing of bottles seen in Figure 26, the decision was made to place the bottles horizontally in the earth. The vertically oriented bottles took high load in the initial few mm of loading, but after 4 – 5 mm of crosshead movement, they reached ultimate strength and slowly lost the ability to handle load. The horizontally laid bottles continually took more load until the bottles were less than half their original width. Neither the vertically or horizontally oriented bottles failed suddenly. The failure consistently occurred when the bottle cap lost its seal and air began to escape. Significant axial load was not placed upon the bottles as the bottles are placed on top of each other with adobe loosely packed around them so either orientation would have theoretically worked.

Horizontally laid bottles in the adobe were easier for constructability making the orientation decision the logical choice. Placing bottles vertically would have been a challenge to hold them in place temporarily while packing adobe around them. One disadvantage to using horizontal layers is the fact that vertical bottles could have potentially created a 'key' if staggered between layers to create a stronger connection between lifts.

5.2. CFPTs Axial Load Capacity

The serviceability factor on the CFPTs to carry the axial load is L/360 (2438/360 = 6.8 mm) at a load of 38.63 kN using the minimum strength sample tested. It is important to note that the axial capacity was determined after only 7 days of concrete curing. As the 7-day strength is roughly 70% of strength found after 28 days of curing, the actual capacity of the column will be significantly higher. The crosshead movement of 6.8 mm is the limiting serviceability factor as the load at L/360 at mid-span deflection is roughly 58.5 kN, nearly double the load of the crosshead movement serviceability limit. There is sufficient capacity for the design load of 22.9 kN calculated in Appendix G. The #2 column (A-base mix) had the lowest ultimate strength and was expected to be as the mix was noted to be drier than the other columns and was more difficult to pour into the column. The drier mix did not produce noticeable voids in the concrete as compared to the other columns as seen in Figure 38.

Figure 38: CFPTs After Testing @ Mid-Span (ordered #1 closest to # 4 farthest)

Upon removal of the PVC tubing from the cast concrete after testing, hairline cracks were observed primarily at the locations of the 13 mm (½") PVC inserts as shown in Figure 39 as an example from the #3 column (clean aggregate mix). The columns were very elastic and rebounded back to original shape. The PVC showed no visible signs of damage.

Figure 39: CFPT Hairline Crack @ 13 mm (½") PVC Insert Location

The CFPTs consistently failed in the direction of the rebar locations in the columns which were off-centered due to the 13 mm (½") PVC inserts as seen in Figure 40 (#1, #2 & #3 all failed to the south, #4 to the north). This demonstrates that load was attracted to the stiffness of the rebar. As the columns were tested with 13 mm (½") PVC inserts that were not used for the final design and the columns took significant load before mid-span deflection reached L/360, the decision was made to save several feet of rebar in each column and only use a 610 mm (24") dowel extending from the footing and the bond beam into the column for shear resistance. Mid-span deflection was also restrained in the wall system by bolting on the wood spacers to achieve 229 mm (9") wall thickness, by the earth infill, and the barbed wire connecting the columns at quarter points and mid-span, further reducing the need for longitudinal rebar through the entire CFPT. The A-base mixed concrete did have lower stiffness than the clean aggregate and sand mix but still had adequate strength for the design loads.

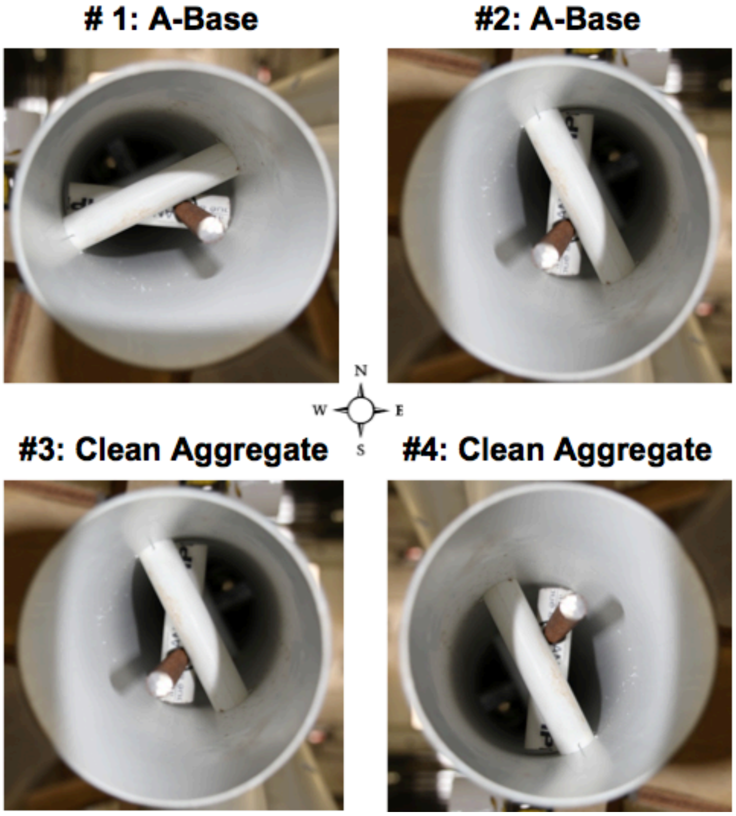

Figure 40: Rebar Orientation in CFPTs

5.3. Infill Block Axial Strength

The weight of each infill lift was approximately 100 kg (220 lbs) so the axial capacity of each lift needed to accommodate the weight of the layer above it. This equates to each lift exerting roughly 3.7 kPa onto the layer below it. After one day of curing, the 229 mm (9”) thick wall experiences roughly 1.1 mm of displacement from 3.7 kPa according to the axial test results. Upon 3 days of curing, if the weight of 3 lifts was added simultaneously to the primary lift, a displacement of 1.63 mm would occur on the first lift and after 5 days and 5 lifts added simultaneously, 1.23 mm of movement. Since these vertical displacements were minimal and did not have any measureable effect on horizontal displacements, 229 mm (9”) thick walls were deemed acceptable to carry the weight of the infill above.

Poisson’s ratio and the transverse displacement upon axial loading both showed that the 229 mm (9”) thick walls start to perform better than 254 mm (10”) thick walls after 5 days of curing (below loading of 11,500 N). This confirmed that 229 mm (9”) thick walls were acceptable for use.

The ability of the infill adobe to recover from stress during the construction process was clear from Figure 30 as the 3 and 5 day test samples were retested. All samples took approximately 350 kPa more when retested on Day 10. The ultimate failure occurred at 8 – 10 mm of crosshead displacement upon retest at Day 10 instead of 18 – 23 mm on Day 5 and Day 3 respectively. It’s obvious that the drier the samples the more brittle they are. The highest axial load would be exerted on the primary layer. So the addition of 9 lifts on top of the primary equates to roughly 33.3 kPa of dead load. Using L/360 as a limitation of displacement on the primary layer for serviceability (6.8 mm), the layer is able to withstand 161 kPa upon 5 days of curing and 383 kPa after 10 days of curing (using the 5 day sample that was retested on Day 10 so 383 kPa could be considered conservative).

5.4. Aircraft Cable Tensile Strength

There was no difference in the ultimate tension capacity of the cable between using one or two aluminum sleeve connectors. The cable consistently failed at the location of the connector as seen in Figure 41. The lowest breaking strength of 8674 N (1950 lbs) was 15% higher than the rated breaking strength of 7562 N (1700 lbs).

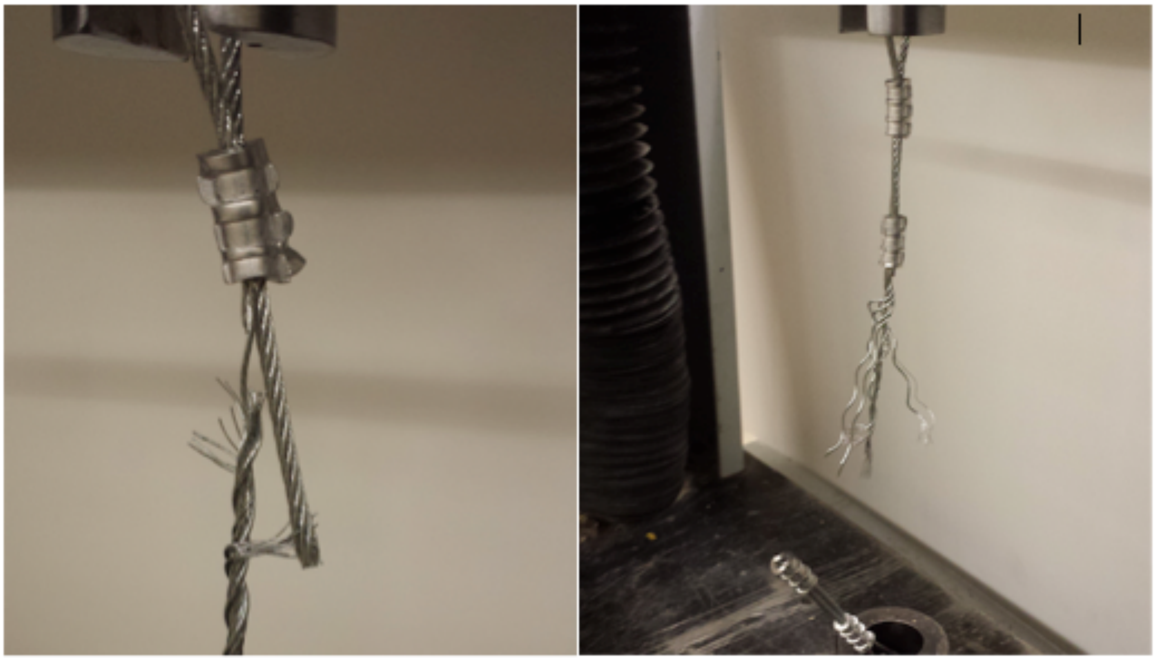

Figure 41: Aircraft Cable Tension Test Failure

To maximize the effectiveness of the cable to aid in resisting racking forces, the cable was tensioned using the turnbuckles to approximately 445 N (100 lbs). At 445 N, Figure 33 shows an increase in stiffness that remains almost linear until ultimate tensile strength is reached. The cables showed no signs of slackness when tensioned to 445 N in the wall system so tensile forces in the cable during racking should have exerted minimal force on the infill surrounding the cable.

5.5. Cyclic Racking Strength

The earth wall system exhibited much higher stiffness than typical stick-frame construction to resist racking. Addition of the adobe to the system significantly increased the capacity indicating that the columns and aircraft cabling provide minimal contribution to resist racking under service load conditions (L/180 = 13.5 mm displacement) as seen in Figure 34. However, the aircraft cable must carry an increasing portion of load as the system nears ultimate strength as failure of the system occurred when the cable snapped as seen in Figure 35 at roughly 83 mm (3.25”) in the push direction and 60 mm (2.4”) in the pull direction.

The building constructed in Honduras should theoretically withstand minimum 155 mph winds as seen in Table 4 under serviceability limits of L/180 and fail at wind speeds of 268 mph. Hence, the building constructed should be able to withstand the winds of a Category 5 hurricane with wind speeds over 157 mph.

Failure was consistently observed at two main locations. The critical failure occurred roughly 1800 mm (70”) from the top as the compacted adobe separated from the wall system as seen in Figure 42. The compacted adobe separated from the wall as layered sections with diagonal cracking near the locations of the aircraft cable as observed in Figure 43. The addition of dimensional 51 mm x 102 mm (2” x 6”) lumber in Honduras to cover the wood spacers on the columns may serve to increase the racking capacity as it would restrain the compacted adobe from separating. Figure 43 also shows the second location of failure – the vertical separation between the top two infill layers.

Figure 42: Racking Wall 1 - Compacted Adobe Separation @ 1800 mm (70") From Top

Figure 43: Racking Failure Wall 3

5.6. Transverse Strength

In the serviceability limit state (L/180), the infill wall system performs better than stud wall construction as seen in Figure 36. Upon 22 mm (0.9”) of deflection, the stud wall accepts a higher load as the stiffness is linear compared to the logarithmic behavior of the adobe wall. Logarithmic behavior results from the ability of the adobe and PET bottles to deform under loading before transferring the load through the wall to the backside where the linear potentiometers were monitoring movement. The elasticity of these components was evident when the face of the wall being loaded exhibited no signs of being stressed following the test.

The adobe wall section displaying the highest deflection under load was consistently located 1000 mm (39.5”) from the top of the bond beam seen in Figure 44 (linear potentiometer in the center of the wall near the bottom of the picture). This location was centrally located between barbed wire reinforcement. Also seen in the figure is a crack developing between the lifts as the lower lift has displaced more than the lift above which has barbed wire to aid in lateral resistance (wire location evident by the threaded rod).

The middle of the earth wall (1270 mm or 50” from top of bond beam) showed the second highest amount of displacement. There was a line of barbed wire located at this mid-height location so this would suggest that the barbed wire is effective in carrying tension load through to the threaded rods embedded in the CFPTs. However, horizontal displacement also decreases as the height from the top of bond beam increases meaning the weight of the adobe above may also be resisting lateral movement, so the effect of barbed wire at 1270 mm (50”) may be negligible.

Figure 44: Airbag Transverse Displacement @ 1000 mm (39.5”) From Top

The decision to monitor the deflection at the location between the two final lifts at 305 mm (12”) from the top of bond beam was a result of the separation failure during cyclic racking. Minimal movement was observed at this location during the first and second transverse tests. This may be from the proximity of the concrete bond beam attracting the transverse loading as load is attracted to stiffness. Moving the location of the linear potentiometer from 305 mm (12”) to the mid-point of a column for the third and final test showed the elasticity of the CFTP after the pressure was released. Figure E - 2 of Appendix E shows the column retracting from a deflection of 21 mm to 4 mm after loading with Figure 45 showing the deflection during and after loading by observing the gap between the wall system and the airbag containment.

Figure 45: Airbag Transverse Deflection (L) & Retraction (R)

5.7. Implementation

Construction of the kitchen and dining room in Honduras took place in the months of January and February of 2015. Those months are considered the dry season and construction projects abound during this time. Community members engaged with the project by providing the earth, straw and PET bottles for the wall system. They also provided several volunteers to aid in concrete work and one individual was hired on a casual basis. Upon successful erection of the walls and roof, the community desired to finish the earth walls with plaster and build the concrete tables for the dining area.

The first layer of infill was altered for implementation to include 5% (by volume) cement in order to strengthen the bond to the concrete footing and provide extra protection from heavy rainfall.

Two small challenges arose the in process of construction. The first was the challenge of finding turnbuckles with capacity matching or exceeding the aircraft cable. Contacting multiple stores and individuals throughout the country was necessary in order to locate acceptable turnbuckles. The other material that had to be ordered in was the 5/16” chain for the cable connections and the aircraft cable. The chain and cable were common items that were not in stock at the time. The second challenge was the use of the concrete mixer for the earth mix. Mortar mixers were not available for rent or purchase. The rolling action of the concrete mixer with paddles made controlling the distribution of moisture through the mix difficult. The earth would stick on the sidewalls of the mixer and large lumps of earth mix would easily form making the material harder to work with than the desired pebble type soil. Trial and error fixed the formation of lumps by adding water in portions to the mix. Adobe near the opening of the mixer would absorb a bulk of the water being added so the mixer was emptied in smaller batches rather than mixing all of the water into the whole batch of adobe in the mixer. The issue of earth sticking to the walls of the mixer was not resolved and required half an hour of cleaning on a daily basis.

The overall material costs for building the kitchen walls (not including the footing, slabs, columns for dining area, roof, doors and windows) was $1328 CDN. This translates into a cost of $79 per meter ($24 per foot). The two most expensive items were the dimensional wood used to cover the wood spacers on the columns ($248.82) and the PVC tubes ($231.88). The cost for renting the mixer and the diesel needed to run it for the earth wall portion of construction was $102.78.

Information including the layout, cost breakdown and images of construction in Guanteque are found in Appendix I.

6. Conclusions and Recommendations

The focus of this research was to assess the performance of a rammed infill adobe and plastic bottle wall system.

The system was found to overcome the low tensile strength and brittle failure of common earthen building techniques. Using the composition of materials with ductile behavior, namely CFPTs of PVC for axial load, aircraft cable for racking load and PET bottles to aid in dissipation of transverse loads proved to be effective.

Axial strength of CFPTs was tested after only 7 days of concrete curing time. At this early stage of curing, the columns had adequate strength of 38.63 kN under serviceability limit states of L/360 to hand the design load of 22.9 kN. After failure, no visible signs were apparent on the exterior of the columns. Upon removal of the PVC casing, minimal cracking was discovered in the concrete. The ability of the columns to retract near to their original shape after compression failure make them an attractive alternative to standard reinforced concrete columns. The ease of constructing CFPTs as the PVC simply needs to be cut to height and braced plumb make them a time-efficient option for columns.

Six prototype panels measuring 1.2 m x 2.4 m (4' x 8') were constructed, tested and analyzed to determine their capacity to resist racking and lateral loads. The analysis showed the system is able to dissipate considerable wind induced loads.

Under cyclic racking, the panels could withstand over 60 mm (2") of travel before ultimate failure loads of 17.05 kN. While individual lifts of earth did separate from the wall system, the system as a whole did not succumb to the racking load. The separation of the earth lifts perpendicular to the wall system were partially restrained in implementation of the system in Honduras as the wood spacers attached to the columns were covered with dimensional 51 mm x 152 mm (2” x 6”) lumber. The aircraft cable played an important role in racking load capacity as failure of the cables was the point of ultimate tensile failure. Under serviceability limits of L/180, the system carried loads over 5.69 kN compared to 2.47 kN of standard 6.5” stud wall assemblies.

The transverse capacity of the system could not be tested to ultimate limit states as the strength of the airbag testing apparatus was a limiting factor, but serviceability limits at deflection of L/180 showed capacities more than double the design loads. The absorption of transverse load with minimal effect on the appearance of this wall system is a highly appealing characteristic.

Successful implementation of the wall system in Honduras demonstrated to residents in the community of Guanteque that alterations to commonly used earthen building techniques can result in reduced construction time while increasing the structural properties of the building. The additional material cost of $1328 to implement this system over traditional adobe block construction in the region could be a limiting factor for local residents.

This research resulted in the successful implementation of a new technique for earth wall systems that achieved favorable structural properties. The concept of this building technique provides a number of future research streams that include:

- The effect of PVC diameter and wall thickness on axial capacity of CFPTs;
- The effect of slenderness ratio on CFPTs of various lengths;
- Evaluating the axial capacity of PVC tubes filled with aggregate or other materials without use of cement;

- Exploring alternative materials to aircraft cabling and barbed wire including caña brava or wood;
- Changes to thermal behavior of earthen wall systems when using PET bottles in various configurations;
- The effect of seismic loading on the wall system.

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Appendix A – Park Tool TM-1 Tension Meter Calibration

A Park Tool TM-1 Bicycle Spoke Tension Meter was calibrated to measure the tension in 1/8” Aircraft Cable. The tests were conducted by securing the cable at one end and tensioning the cable using a 2 ton chain hoist (not pictured) with a weigh scale hooked in series as seen in Figure A - 1. The chain hoist allowed for incremental changes to the tension load placed on the cable and the readings recorded are found in Table A - 1.

Figure A - 1: Park Tool TM-1 Calibration Setup

Table A - 1: Park Tool TM-1 Tensions Meter Calibration - Raw Data

TEST 1		**TEST 2**		**TEST 3**		**TEST 4**	
Load (lbs)	TM-1 Reading	Load (lbs)	TM-1 Reading	Load (lbs)	TM-1 Reading	Load (lbs)	TM-1 Reading
28	3	20	3	23	5	25	4
30	4	28	7	30	7	35	8
34	6	41	10	37	9	50	12
41	9	52	14	61	15	68	17
46	10	71	17	75	17	88	20
53	12	85	20	90	20	103	22
61	15	100	22	108	23	121	24
72	17	111	23	130	25	154	27
82	19	160	27	153	27	177	29
93	21	189	30	175	29	200	31
103	22	227	32	200	31	233	32
115	23			222	32		
127	25			243	32		
138	26			254	33		
153	27			271	34		
167	28			286	34		
180	29						
192	30						
200	30						
214	31						
221	32						
226	32						
236	32						
256	33						
285	34						

The raw data was graphed as seen in Figure A - 2 and the appropriate trendlines added. Plugging 5 lb increments the trendline equations as the x-value yielded four theoretical Park Tool readings shown in Figure A - 2. The average of the four theoretical readings was then calculated and plotted against the 5 lb increments as seen in Figure A - 3.

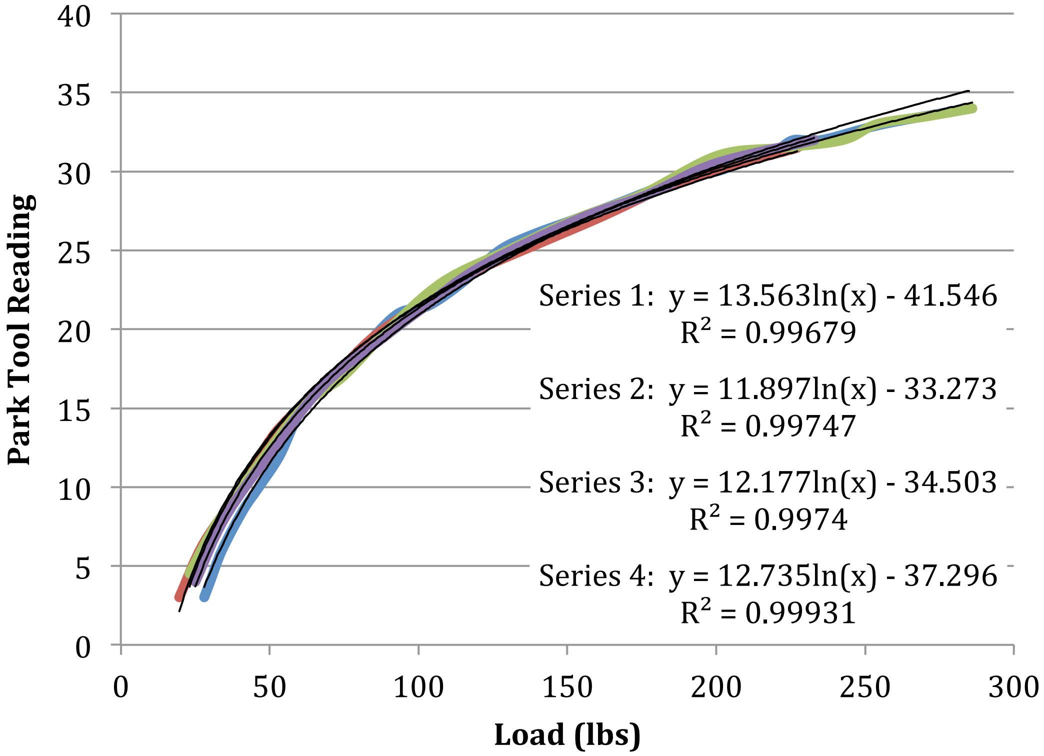

Figure A - 2: Park Tool TM-1 Tension Meter Individual Calibration Tests

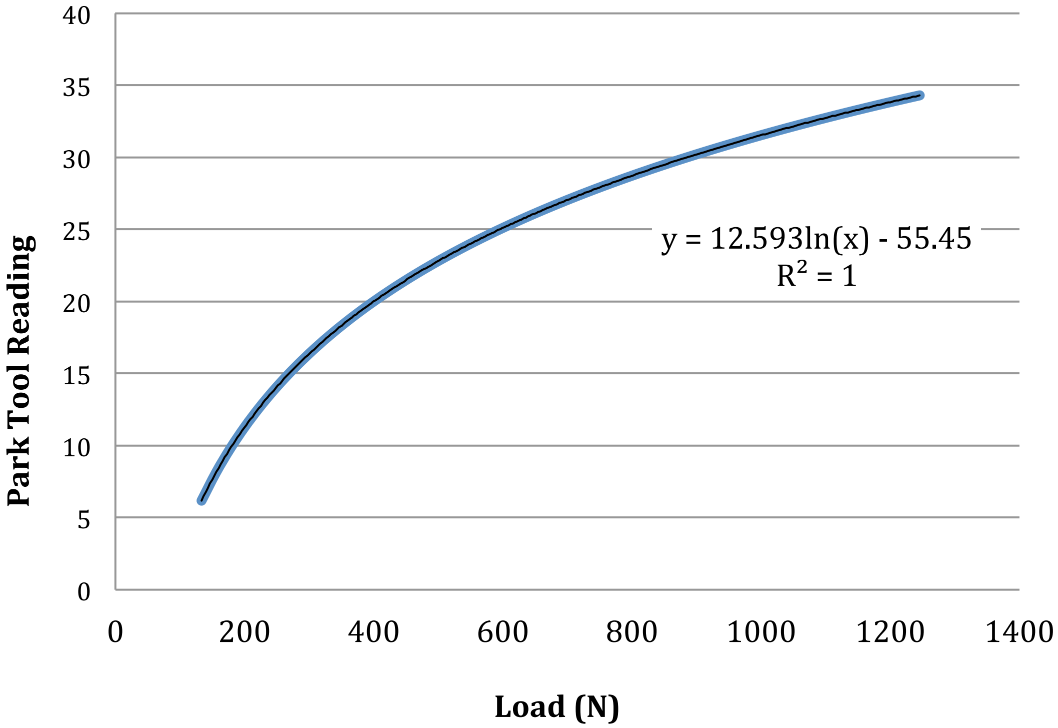

Figure A - 3: Park Tool TM-1 Final Calibration Curve

Table A - 2: Park Tool TM-1 Final Calibration Calculations

Load (lbs)	Average	Test 1	Test 2	Test 3	Test 4	Load (N)
30	6	5	7	7	6	133
35	8	7	9	9	8	156
40	10	8	11	10	10	178
45	11	10	12	12	11	200
50	13	12	13	13	13	222
55	14	13	14	14	14	245
60	15	14	15	15	15	267
65	16	15	16	16	16	289
70	17	16	17	17	17	311
75	18	17	18	18	18	334
80	19	18	19	19	19	356
85	19	19	20	20	19	378
90	20	19	20	20	20	400
95	21	20	21	21	21	423
100	21	21	22	22	21	445
105	22	22	22	22	22	467
110	23	22	23	23	23	489
115	23	23	23	23	23	512
120	24	23	24	24	24	534
125	24	24	24	24	24	556
130	25	24	25	25	25	578
135	25	25	25	25	25	601
140	26	25	26	26	26	623
145	26	26	26	26	26	645
150	26	26	26	27	27	667
155	27	27	27	27	27	689
160	27	27	27	27	27	712
165	28	28	27	28	28	734
170	28	28	28	28	28	756
175	28	29	28	28	28	778
180	29	29	29	29	29	801
185	29	29	29	29	29	823
190	29	30	29	29	30	845
195	30	30	29	30	30	867
200	30	30	30	30	30	890
205	30	31	30	30	30	912
210	31	31	30	31	31	934
215	31	31	31	31	31	956
220	31	32	31	31	31	979
225	32	32	31	31	32	1001
230	32	32	31	32	32	1023
235	32	33	32	32	32	1045
240	32	33	32	32	33	1068
245	33	33	32	32	33	1090
250	33	33	32	33	33	1112
255	33	34	33	33	33	1134
260	33	34	33	33	34	1157
265	34	34	33	33	34	1179
270	34	34	33	34	34	1201
275	34	35	34	34	34	1223
280	34	35	34	34	34	1246

Appendix B – Pressure Gauge Calibration with Water Column

The pressure gauge used for the airbag transverse tests was calibrated using a water column as shown in Figure B - 2. The equation taken from the trendline graphed in Figure B - 1 was used to convert the voltage recorded during the test into pressure. The raw data is found in Table B - 1.

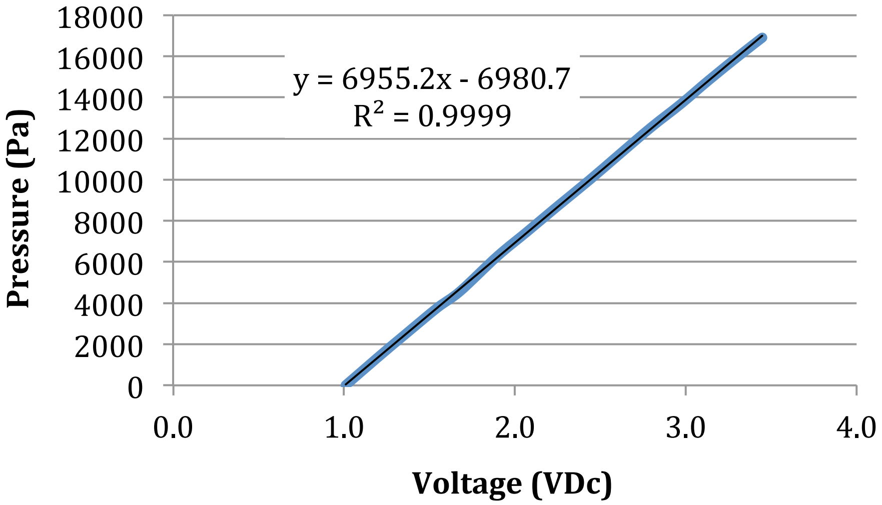

Figure B - 1: Pressure Gauge Calibration Curve

Table B - 1: Pressure Gauge Calibration Raw Data

Inches Water	VDc	Psi	Pa
0.000	1.010	0.000	0.00
5.313	1.193	0.192	1323.28
10.250	1.369	0.370	2553.16
15.188	1.548	0.549	3783.04
18.375	1.680	0.664	4577.01
25.188	1.897	0.910	6273.93
30.000	2.072	1.084	7472.67
35.000	2.251	1.264	8718.11
40.000	2.434	1.445	9963.56
45.125	2.613	1.630	11240.14
50.375	2.798	1.820	12547.85
54.938	2.972	1.985	13684.32
59.500	3.133	2.150	14820.79
64.500	3.318	2.330	16066.23
67.875	3.448	2.452	16906.91

Figure B - 2: Water Column

Appendix C – Wood Frame Design for Airbag Transverse Test

NOTE: All equations and design wood capacities from the *Wood Design Manual 2010* (Canadian Wood Council 2010)

The limiting factor for the airbag wood containment chamber is the backside 2”x4” frame seen in Figure C - 1.

Figure C - 1: Backside of Airbage Chamber

Joist spacing: 300 mm
Joist span: 1.2 m

Capacity of S-P-F No.1/No.2, 38 x 89: $M_r = 0.906$ kN-m * 1.10 = 0.997 kN-m
$V_r = 5.17$ kN

Calculate w_f from M_f & V_f using M_r & V_r:

$V_f = V_r = w_f L/2$: $w_f = 5.17*2/1.2 = 8.62$ kN-m

$M_f = w_f L^2/8$: $w_f = 0.997*8/(1.2^2) =$ **5.54 kN-m governs**

Calculate load capacity: (consider the airbag pressure to be live load factored 1.5 times)

Factored Load: 5.54/0.3 = **18.5 kPa**
Live Load: 18.5 kPa/1.5 = **12.3 kPa = 257 psf**

Appendix D – Cyclic Racking Graphs for Walls 1 & 3

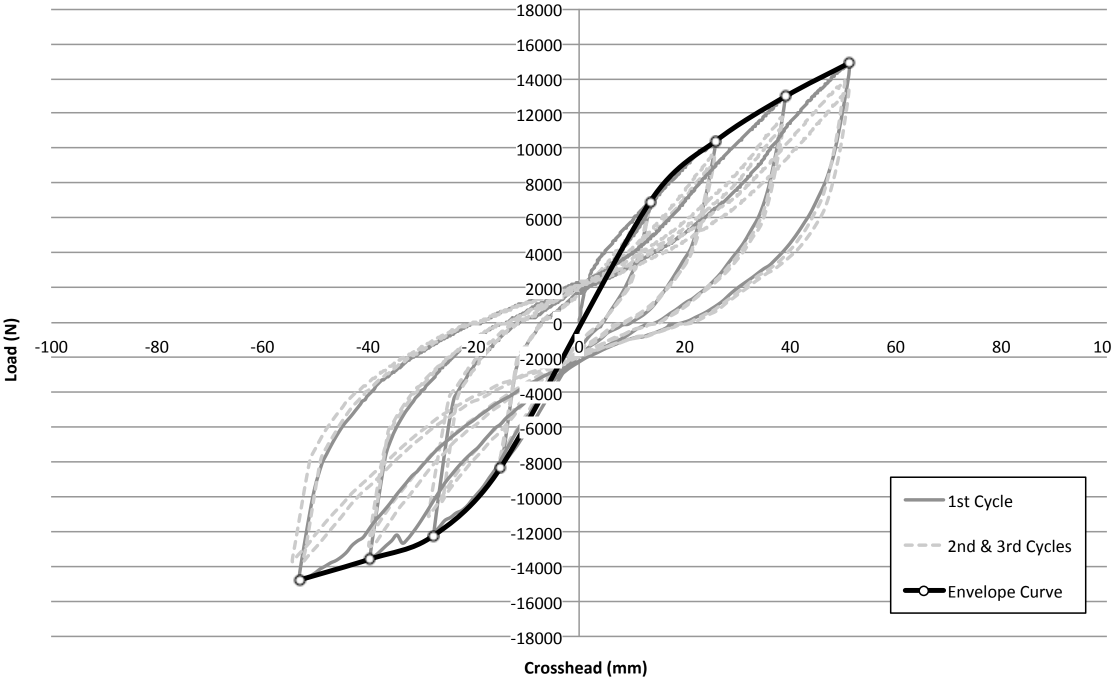

Figure D - 1: Detailed Cyclic Racking Wall 1

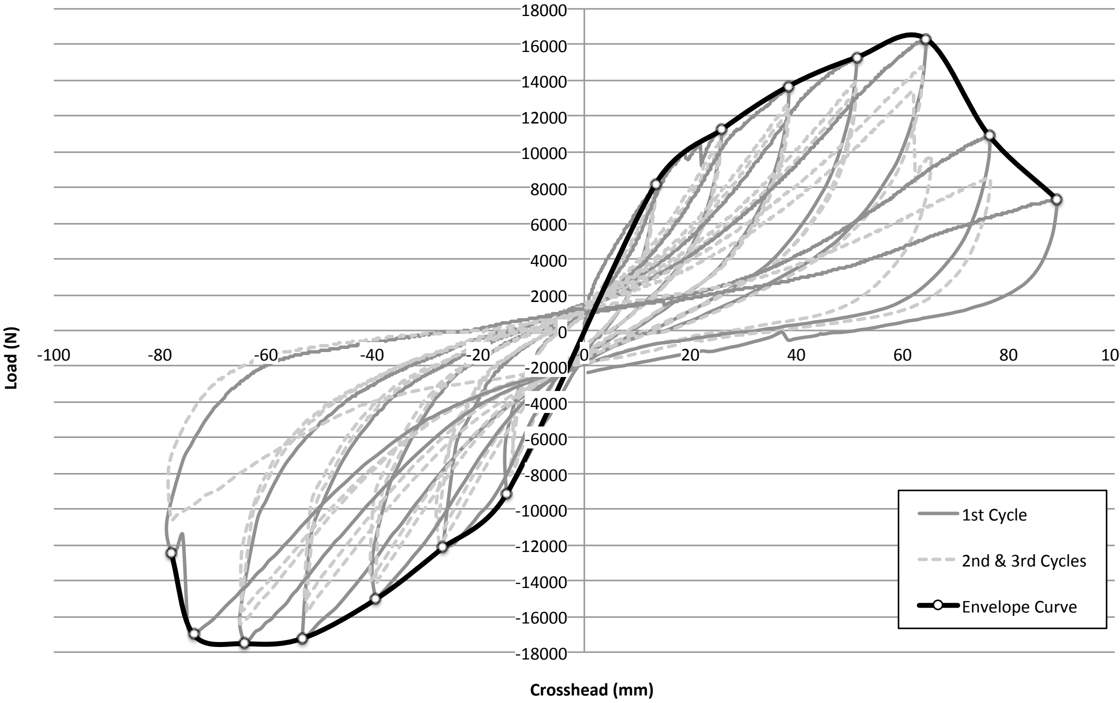

Figure D - 2: Detailed Cyclic Racking Wall 3

Appendix E – Transverse Graphs for Walls 2 & 3

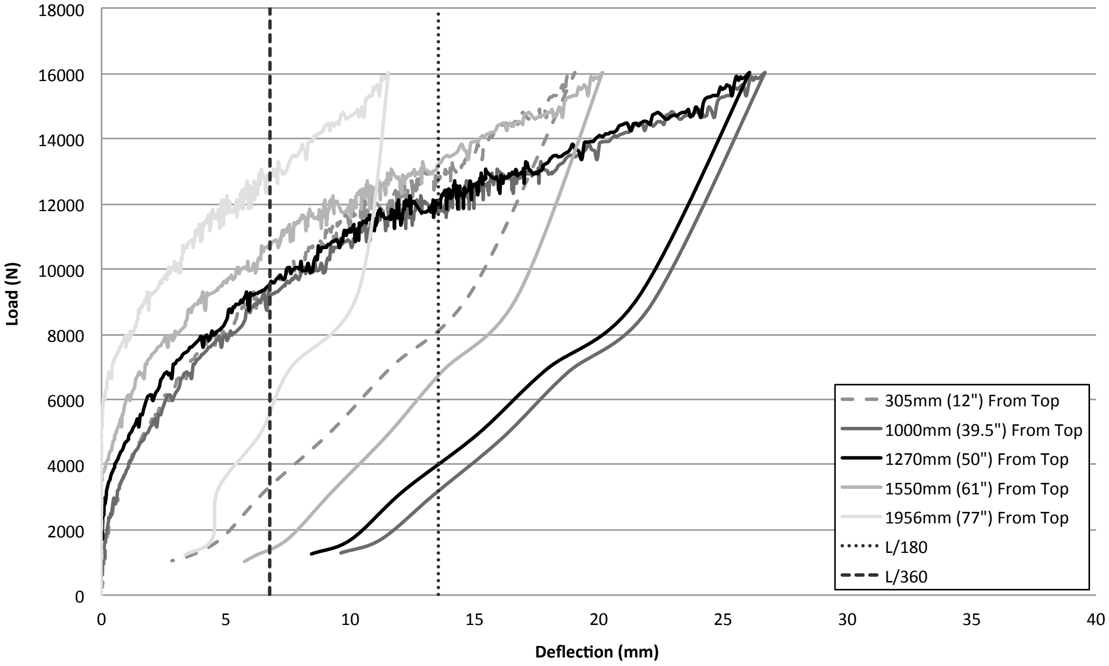

Figure E - 1: Detailed Transverse Wall 2

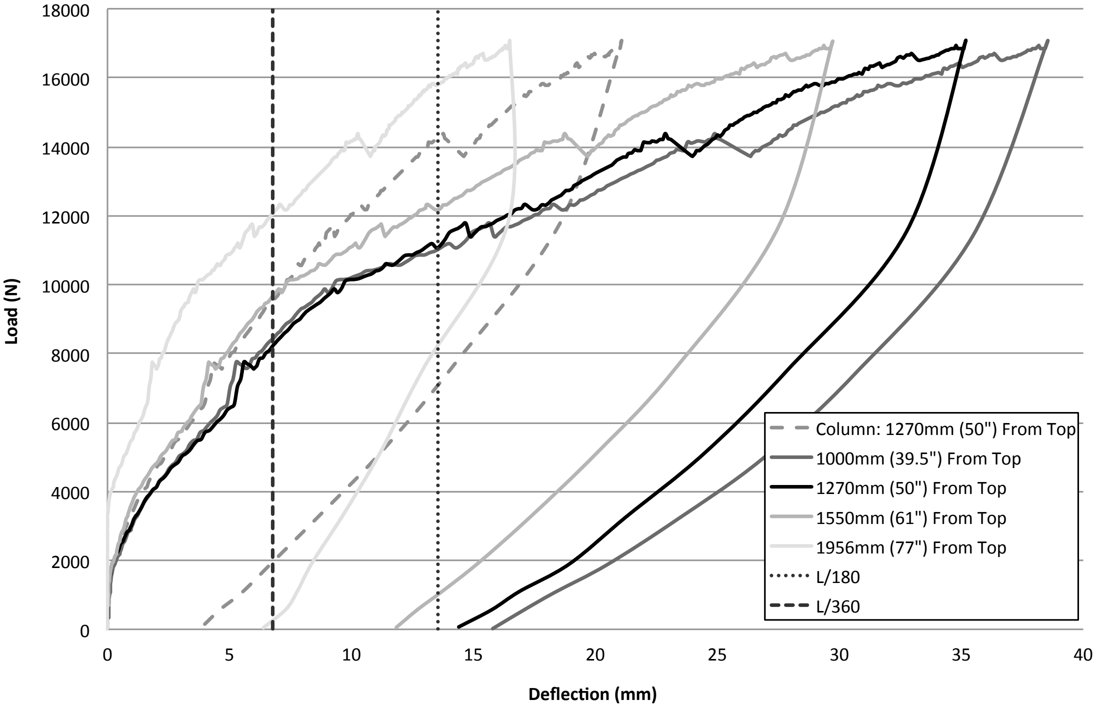

Figure E - 2: Detailed Transverse Wall 3

Appendix F – Axial Graph for Empty 102mm (4”) PVC

A 2.4 m (8’) long 102 mm (4”) PVC tube was tested axially to observe the functionality of the test apparatus. After first cycle failure and releasing the load, the tube appeared to retract to it’s original shape, so the tube was immediately retested to failure in a second cycle and the results are shown below in Figure F - 1. The tubes were insufficient to handle the design axial load of 22.90 kN. The stiffness of the empty does raise the question whether filling the tubes with aggregate and/or sand could raise the bearing capacity sufficiently to avoid using cement.

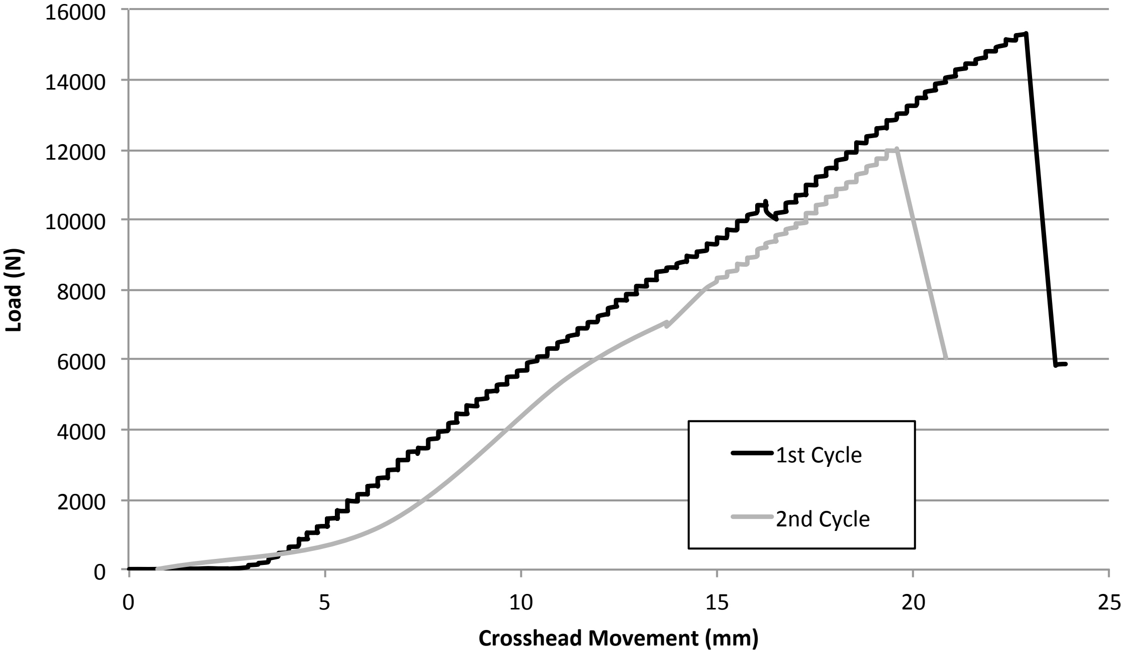

Figure F - 1: Empty 102 mm (4”) PVC Tube Axial Test Results

Appendix G – Design Load Calculations

AXIAL DESIGN LOAD:

Wind Pressure: Roof = 195 kg/m^2
(Based on Table 2, wind speed of 160 km/h, *Guidelines For Prevention Against Wind In Hospitals And Health Centers* (Compañy 2002)):

Roof Tributary Area: 9’9” * 9’ = 87.75 ft^2 **= 8.15 m^2**

Wind Load: 195 kg/m^2 * 8.15 m^2 = **1589.25 kg**
Dead Load: 88 kg

AXIAL DESIGN LOAD: (1.25*88kg) + (1.4*1589.25kg) = **2335 kg = 22.90 kN**
(Equation from Table 1.2, Case 4: 1.25D + 1.4W, *Wood Design Manual 2010* (Canadian Wood Council 2010))

RACKING DESIGN LOAD:

Wall Area = 18’9” * 8’ = 150 ft^2
Wind Pressure = 32 psf (111.8 mph wind)

Force Exerted on Wall = 150 ft^2 * 32 lbs/ft^2 = **4800 lbs**
% of Force Transferred into Foundation Assumed Conservatively 25%
Force Transferred to One Short Wall: (4800 – 25%)/2 = **1800 lbs**

Force Acting Per Foot on Length of Short Wall – Opening:
1800lbs/**(**10’2”-3’) = 251.2 lbs/ft

RACKING DESIGN LOAD: 251.2 lbs/ft * 4’ = 1004.8 lbs = **4.47 kN**
(4’ for the length of a prototype section)

TRANSVERSE DESIGN LOAD:

Design Area: 32 ft^2
Wind Pressure: 32 psf (111.8 mph wind)

TRANSVERSE DESIGN LOAD: = 32 ft^2 * 32 lbs/ft^2 = 1024 lbs = **4.55 kN**

Appendix H – Seismic Load Calculations

NOTE: Equations and values from *ASCE 7: Minimum Design Loads for Buildings and Other Structures* (American Society of Civil Engineers 2003) unless otherwise noted.

S_s = 1.33g, S_1 = 0.53g, where g = 9.81 m/s^2

- Values for S_S and S_1 found for Guanteque, Honduras from *U.S. Geological Survey* (U.S. Geological Survey 2014)

Site Classification (Table 9.4.1.1): Site Class D – Stiff Soil

- Gives **F_a = 1.0, F_v = 1.5**

S_{MS} = F_aS_s = 1.0*1.33g = 1.33g
S_{M1} = F_vS_1 = 1.5*0.53g = 0.795g

S_{DS} = (2/3)*S_{MS} = (2/3)*1.33g = 0.887g
S_{D1} = (2/3)*S_{M1} = (2/3)*0.53g = 0.353g

Design Category D (from Tables 9.4.2.1a & 9.4.2.1b based on S_{DS} & S_{D1})

Response Modification Coefficient R = 3 (from Table 9.5.2.2, using 'Ordinary Reinforced Concrete Moment Frames' building frame as the design system in the absence of anything related to this system)

W = weight of structure: 2432.5 kg (weight of wall) + 225 kg (weight of roof)
= 2657.5 kg ≈ 2700 kg

V = seismic base shear = 1.2*S_{DS}*W/R
= 1.2*(0.887g)*2700kg/3
= 1.2*0.887*9.81m/s^2*2700kg/3
= **9398 N**

F = vertical distribution of seismic forces = V (since the building is one floor)
= 9398 N

Appendix I - Implementation Layout, Costs & Images

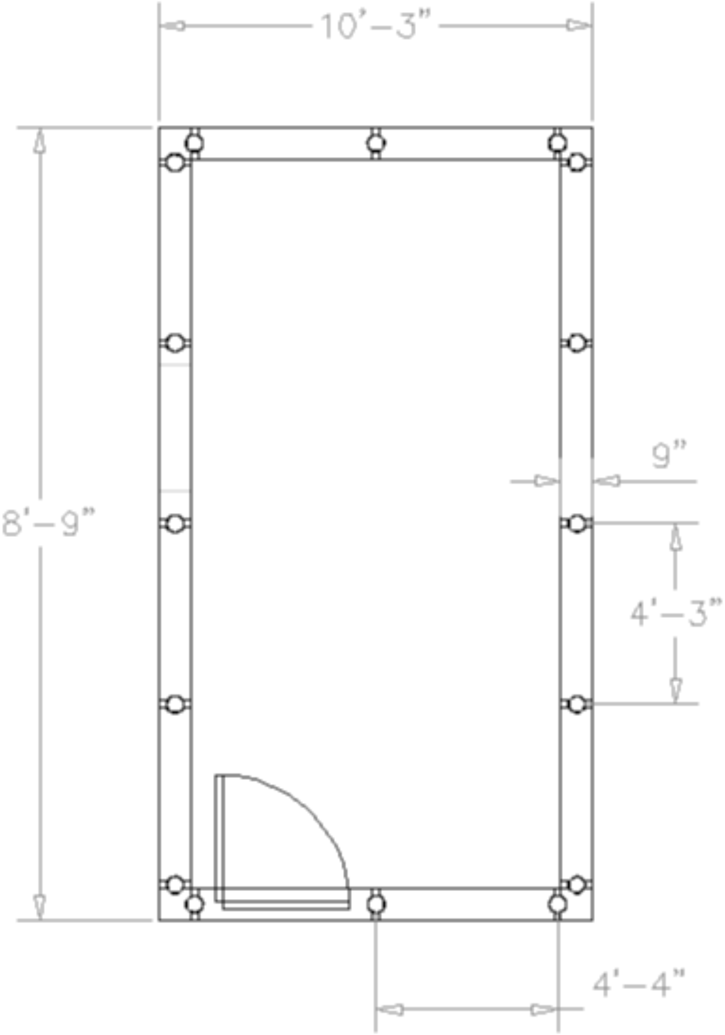

Figure I - 1: Kitchen Layout Dimensions in Guanteque

The costs for implementation shown in Table I - 1 are exclusive to the construction of the walls themselves including the columns, bond beam, cabling, and wood (sans footings, cost of window & door, roof, and floor slab). The cost per meter and foot are divided over the entire perimeter length of the wall system.

Table I - 1: Cost Breakdown of Implementation in Honduras

Item	Cost Per Unit (HNL)	Quantity	Length Or Unit	Cost (HNL)	Cost (CDN)
4" PVC	521.74	8	20'	4173.92	$231.88
2" x 6" x 8' Wood	240.00	30	-	3936.00	$218.67
1/2" Turnbuckles	180.00	20	-	3600.00	$200.00
5/16" Chain	2225.22	1	-	2225.22	$123.62
Wood For Spacers	60.13	32	-	1924.00	$106.89
Mixer	100.00	12	Days	1200.00	$66.67
3/8" Rebar	96.00	12	6m	1152.00	$64.00
Cement	150.00	7	Bag	1050.00	$58.33
Aircraft Cable	417.39	2	-	834.78	$46.38
Diesel for Mixer	65.00	10	Days	650.00	$36.11
2" x 4" x 8' Wood	32.00	6	-	542.80	$30.16
4" Union	65.00	8	-	520.00	$28.89
1/8" Cable Clamps	6.00	80	-	480.00	$26.67
3/8" Rod	60.87	7	Meter	426.09	$23.67
3" Screws	140.00	2	-	280.00	$15.56
Gravel	370.00	0.71	Cubic Meter	262.70	$14.59
Barbed Wire	456.52	0.5	Roll	228.26	$12.68
3/8" Nut	1.30	160	-	208.00	$11.56
3/8" Washer	0.87	160	-	139.20	$7.73
5/16" Washers	140.00	0.5	-	70.00	$3.89

TOTAL COST: **23902.97** HNL **$1,327.94** CDN

Perimeter Length of Wall (Meters) 16.8
Cost Per Meter: **1422.80** HNL **$79.04** CDN

Perimeter Length of Wall (Feet): 55
Cost Per Foot: **434.60** HNL **$24.14** CDN

Figure I - 2: Construction Site

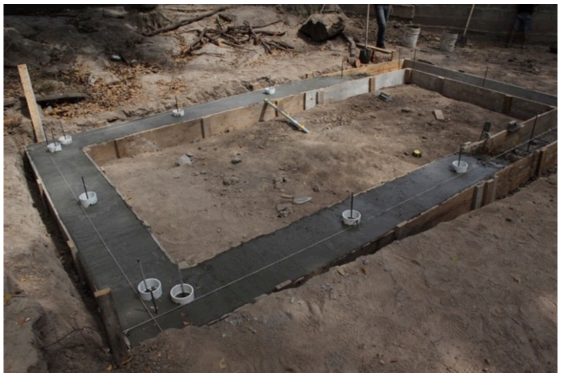

Figure I - 3: Concrete Footing

Figure I - 4: Setting the Columns

Figure I - 5: Bracing the Columns

Figure I - 6: Concrete Filled PVC Tubes

Figure I - 7: Getting a Load of Clay

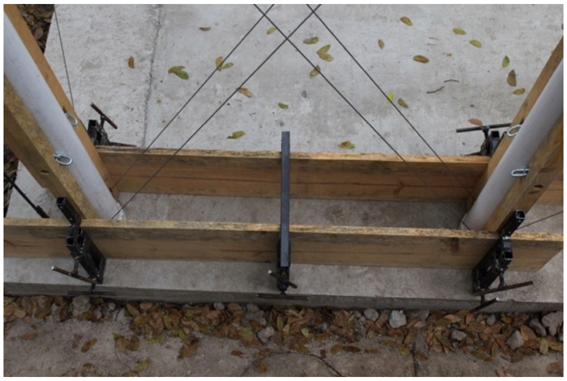

Figure I - 8: Infill Formwork Set

Figure I - 9: Rammed Adobe in Corner

Figure I - 10: PVC Drain Pipe Embedment

Figure I - 11: Threaded PVC Connection

Figure I - 12: Start of Second Layer & Barbed Wire

Figure I - 13: Door Formwork & Rebar

Figure I - 14: Last Infill Layer

Figure I - 15: Concrete Bond Beam

Figure I - 16: Dining Side of Kitchen

Figure I - 17: Back Side of Kitchen

Figure I - 18: Nursing Students with S4S Teaching First Aid Skills

Figure I - 19: Meeting with Community Members to Discuss the Project

Figure I - 20: S4S Team of 2014-15