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**EVALUATION OF POROTON MASONRY AND
SUGGESTION FOR QUICK DESIGN PROCEDURE**

A Thesis
Presented for the
Master of Science
Degree
The University of Tennessee, Knoxville

Michael Richard Radcliffe
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DEDICATION

This thesis is dedicated to my mother, Mrs. Carol Radcliffe, my father, Mr. Richard Radcliffe, and the rest of my extended family. Without their love, support, and many sacrifices none of this would have been possible.

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ABSTRACT

Poroton masonry units are highly cored extruded clay units with numerous thin webs. They have ground surfaces and are laid on a thin fluid mortar bed, with no mortar in the head joints. They offer the advantages of quick construction, which can be performed by a general labor force.

This thesis describes the results of component and prism testing of Poroton masonry. Compressive strength tests occurred on individual units and on prisms constructed with two units in accordance with the manufacturer's directions. The average unit compressive strength was approximately 2900 psi. The average prism compressive strength was about 900 psi. In addition to compressive testing on the two unit prisms, flexural strength testing was conducted on the prisms. During flexural strength testing, modifications to the flexural testing device, also known as the "Bond Wrench," were made. These changes allowed a pure moment to be applied during flexural tension testing. The average flexural tensile strength for Poroton masonry was 65 psi.

With the strength properties now defined, a review of unreinforced and reinforced design methods applicable to

the Poroton units have been made. For unreinforced design, conventional procedures can be used for Poroton construction. However, section properties can be approximately determined based on net moment of inertia of 60% of the gross moment of inertia. For reinforced design, an approximate method was developed. The approximate method is based on a simplified "T-beam" design that is commonly used for partially grouted walls. The distance from extreme compression fiber to the neutral axis was slightly modified to account for the unusual cross-sectional geometry. This approximate reinforced design method tended to be slightly conservative when compared to an exact design method.

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Chapter 1

Introduction

1.1 Background

With the recent opening of the global market, foreign technology and products typically unavailable to consumers in the United States have been introduced. One such product is the German designed Poroton structural clay tile, developed by Wienerberger Industries; a parent company of United States based General Shale Brick. The Poroton unit does not meet any of the current ASTM specifications for either hollow brick or structural clay tile, and construction is different from conventional masonry. Thus, there is the need to conduct an evaluation of strength and to develop a design philosophy that would be simple for designers to use day-to-day. With this information, Poroton masonry could be used here in the United States.

1.2 Purpose

The purpose of this thesis is to describe the procedure for the preparation and subsequent testing of the Poroton units in a number of configurations. The ultimate compressive strengths of both individual units and two unit prisms were measured. In addition, ultimate flexural

tension strength tests of the prisms were conducted. After the testing phase, a design procedure was established for both unreinforced and reinforced masonry construction.

1.3 Poroton Description

The typical Poroton unit is a large highly cored extruded clay unit. Due to its highly cored cross-section, the Poroton has a very good thermal rating. It's top and bottom surfaces have been ground to assist in leveling of the unit during construction.

The Poroton product line includes many different types of units, both in size and shape. There are specialized units made for corners, jambs, bond beams, and vertical reinforcing. See Figure 1.1 for a sample of the types available.

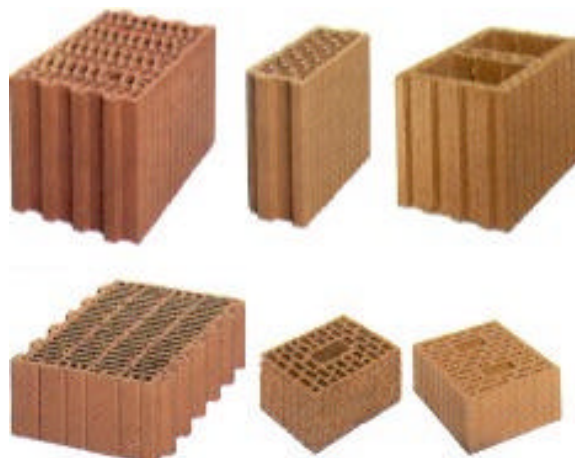


FIGURE 1.1 - Examples of Different Unit Configurations

1.4 Poroton Construction

A major advantage of the Poroton product line is its quick construction applications. The majority of the construction process does not need skilled masons. The first course of units will need to be placed in a mortar bed by a skilled mason in typical fashion (see Figure 1.2).

From then on, the units can be stacked one row at a time using laborers or unskilled masons. Coarse adhesion is achieved by placing a very fluid mortar between courses. There are two methods for placing the fluid mortar. The first way is to use a roller over the previous course (see Figure 1.3).

The other method allows the dipping of the unit to be placed (Figure 1.4). After the mortar has been applied, the next course is stacked in a running bond fashion if possible. Either method does not include the placement of mortar in the head joints. Therefore, there is a need for weatherproofing of the structure. Typically, some type of plaster or stucco coat will be used.



FIGURE 1.2 - First Course Construction



FIGURE 1.3 - Roller Application of Mortar



FIGURE 1.4 – Dipping Application of Mortar

1.5 Format

This thesis is divided into three major sections. The first section details the procedure for preparation and testing of components used in Poroton construction and lists the results of such tests. The second section describes the procedure for preparation and testing of the two Poroton unit prisms and tabulates the subsequent results for these tests. The third section presents the established design considerations/procedures for Poroton construction.

Chapter 2

Constituent Properties

2.1 Unit Mass Properties

Three specimens for each size of the blocks under investigation were used to obtain the corresponding geometric properties. The overall dimensions (height, length, and width) were measured with dial calipers as seen in Figures 2.1 & 2.2. The average of the three specimens for each size of the units is shown in Table 1. Actual observations of each specimen can be found in Appendix A.

Since there are no applicable standards for the testing of Poroton units, the procedures of ASTM C140 – 97 *Standard Test Method of Sampling and Testing Concrete Masonry Units* were followed to obtain the weight and area properties also listed in Table 1. Four values of weight were measured to determine density, absorption, volume and moisture content. Received weight was recorded upon arrival of units from overseas. Determination of immersed weight (W_i) occurred after the test specimens were immersed for a period of 24 hours in a temperature range of 60 to 80 degrees Fahrenheit. After removal of the specimen from the tank, the visible surface water was removed with a damp cloth and the weight was recorded as the saturated weight

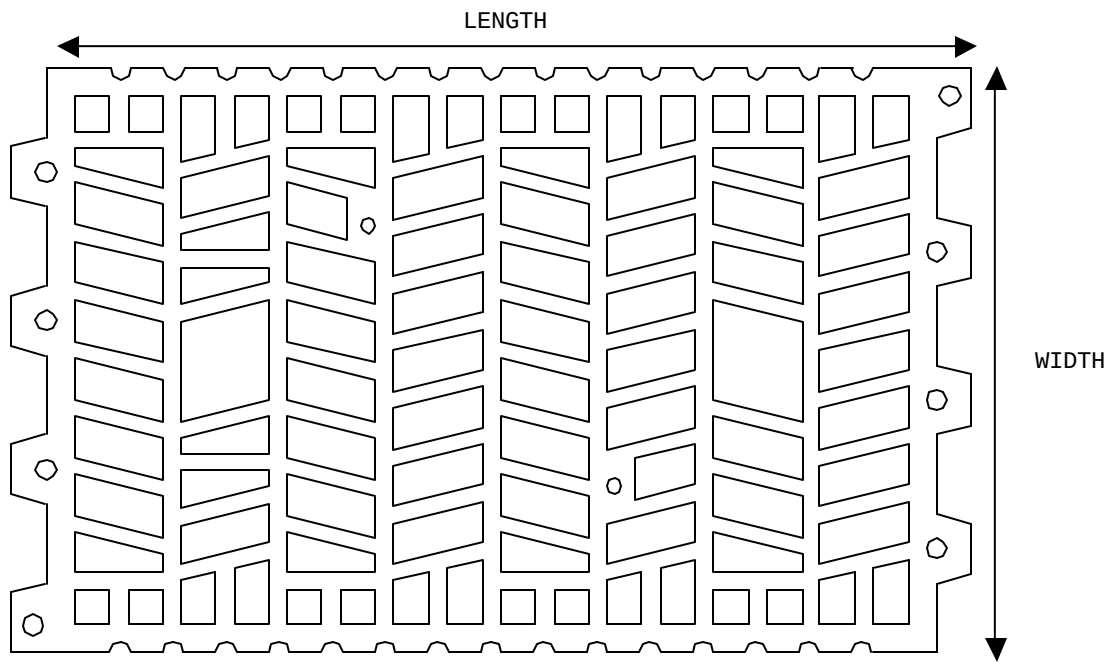


FIGURE 2.1 - Large Poroton Unit Cross Section

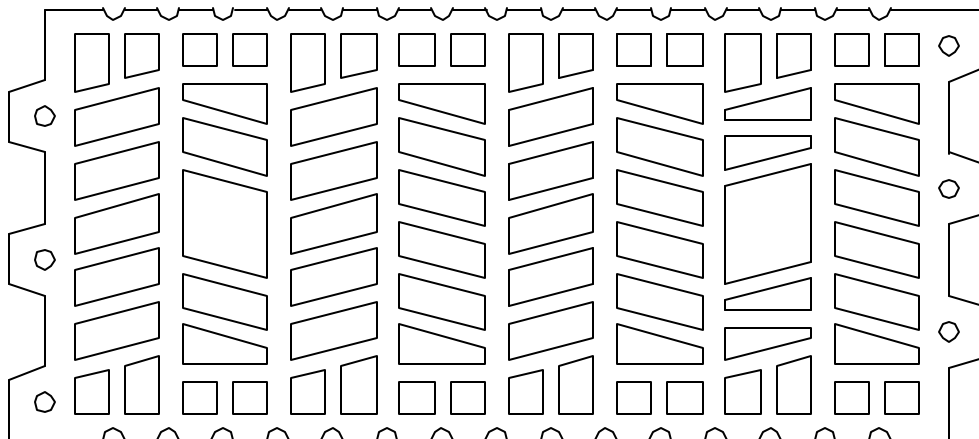


FIGURE 2.2 - Small Poroton Unit Cross Section

TABLE 1. Geometric Properties

Property	Small Block	Large Block
<u><i>Overall Dimensions</i></u>		
Height (in)	9.84	9.80
Thickness (in)	6.60	9.29
Length (in)	14.50	14.63
<u><i>Mass Properties</i></u>		
Received Weight (W_r)	30.30	39.06
Immersed Weight (W_i)	17.88	23.12
Saturated Weight (W_s)	36.31	46.64
Oven Dried Weight (W_d)	30.13	38.86
Absorption (%)	20.51	20.01
Moisture Content (%)	2.77	2.58
Density (lb/ft ³)	102	103.1
Net Volume (ft ³)	0.295	0.377
Weight of Unit (lb)	30.1	38.9
Wall Weight (psf)	30	39
<u><i>Area</i></u>		
Gross Area (in ²)	95.8	135.9
Net Area (in ²)	51.8	66.4
% Coring (%)	45.9	51.1
Net Area per Ft (in²/ft)	42.9	54.5

(W_s). Upon completion of the saturated weight determination, the specimens were dried in ventilated ovens at 100 degrees Celsius for at least 24 hours. Then a weight was recorded as oven-dry weight (W_d).

Important design properties include the wall weight in pounds per square foot (psf), and the net area per ft length of wall. These are shown in bold in Table 1.

2.2 Unit Cross Sectional Properties

Due too extremely complex cross sectional geometry, the moments of inertia for the units' cross-sections were not determined by conventional methods. Instead, the moments of inertia were determined by making AutoCAD® drawings (see Figures 2.1 & 2.2) for each cross-section of the units, and using AutoCAD's moment of inertia capabilities. The results from AutoCAD® are given in Table 2. Based on these results, it appears that 60% of the gross moment of inertia is a reasonable estimate of the net moment of inertia for both sizes of the Poroton units (See Table 2.) Given the same basic cross section, this approximation will avoid the need for additional AutoCAD® drawings for other unit sizes.

For the design considerations that follow, the section modulus is generally used. These are included in Table 2.

TABLE 2 - Moment of Inertia & Section Modulus

Property	Small Block	Large Block
Per unit		
Gross I (in ⁴)	347.4	977.5
Net I (in ⁴)	233.1	578.3
% Net of Gross (%)	67.1	59.2
Net S (in ³)	70.6	124.5
Per Foot of Wall Length		
Net I (in ⁴ /ft)	192.9	474.3
Net S (in ³ /ft)	58.4	102.1

2.3 Unit Preparation for Compressive Strength Tests

The Poroton units used as specimens for the compressive strength tests were capped with HydroCal®, a gypsum cement. The capping mold consisted of a 22" x 18" sheet of glass under a 12" x 18" x 0.5" rectangular steel frame. The mold was lightly coated with oil prior to the placement of the HydroCal®. The gypsum cement compound was mixed by hand until a consistency was achieved so that the compound could not flow freely between the steel frame and the sheet of glass. The specimen was then placed in the mold so that its axes were at right angles to the capping surface. The units were allowed to stand for about thirty minutes before they were removed from the mold to assure

that the cap would remain relatively undisturbed. Both ends of the specimen were capped. One specimen required recapping due to an imperfect initial cap.

2.4 Unit Compressive Strength Testing Apparatus

The Poroton units' compressive strength was determined in accordance with the procedures of ASTM C140 - 97 *Standard Test Method of Sampling and Testing Concrete Masonry Units*. The testing apparatus was a Forney Compression Testing Machine, with a capacity of 300,000 pounds. Since the area of the upper bearing block was not sufficient to cover the bearing area of the capped Poroton unit, a 1.5" thick steel bearing plate, conforming to the requirements of section 6.1.2 of ASTM C140 - 97, was used. This setup is depicted in Figure 2.3.

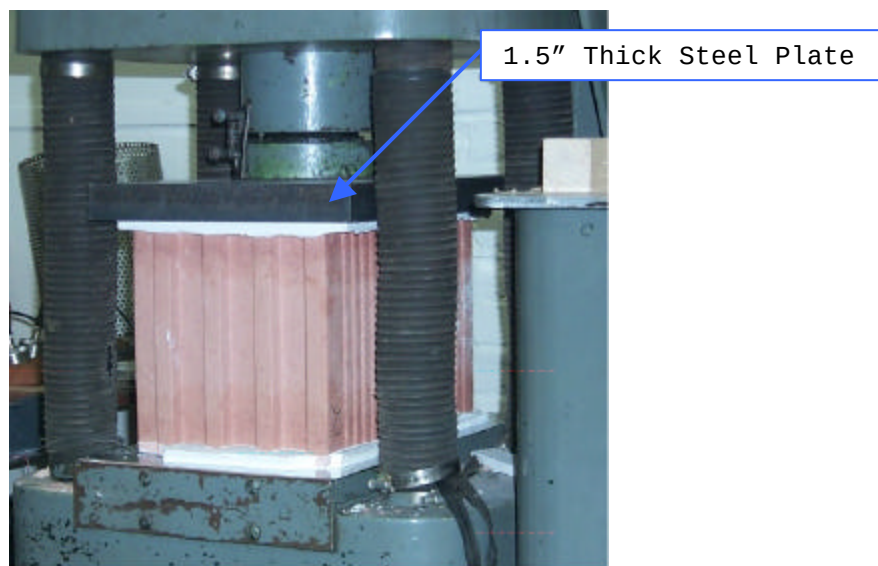


FIGURE 2.3 - Unit Compression Testing Setup

2.5 Unit Compressive Strength Testing and Results

The test specimens were placed such that their geometric centroids were collinear with the centroid of the bearing blocks. Then the steel plate was added and also centered on the unit. The units were free from visible moisture and dampness. A uniform loading rate was applied so that the maximum load was achieved in not less than 3 minutes or more than 5 minutes after loading began.

During the loading of the units, failure began to occur with the units splitting along the vertical axis. Rather large pieces of the units peeled off. Photos were taken of three large units after compression tests were completed. Figure 2.4 depicts a large unit after the ultimate compressive load has been applied. Additional photos can be found in Appendix B.

From the maximum compressive strength test results listed in Table 3, one can observe that there is no significant difference between the average net strengths of both the small and large units. However, the strength of the Poroton units was observed to be much lower than the strength of typical clay units, and about the same as a standard concrete masonry unit.

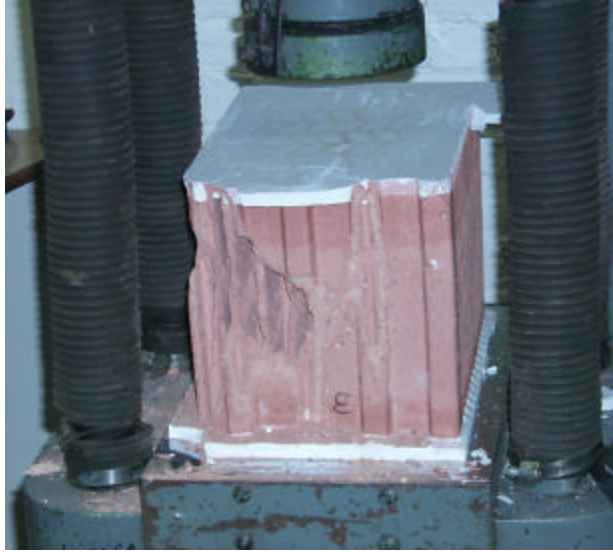


FIGURE 2.4 - Large Unit After Compressive Tests

TABLE 3 - Unit Compression Test Results

Large Block	Area (in ²)	Compressive Load (lbs.)	Compressive Stress (psi)
1*	68.73	172000	2503
2*	68.73	189000	2750
3	66.43	192000	2890
4	66.41	187000	2816
5	66.48	210000	3159
Average	67.356	190000	2823
Small Block	Area (in ²)	Compressive Load (lbs.)	Compressive Stress (psi)
1*	53.07	154000	2902
2*	53.07	155000	2921
3	52.06	158000	3035
4	51.39	158000	3075
5	52.06	148500	2852
Average	52.33	154700	2957
* Area was estimated by AutoCAD Drawings; other Areas determined in Section 2.1			
Combined	Area (in ²)	Compressive Load (lbs.)	Compressive Stress (psi)
Average	59.843	172350	2880

2.6 Mortar Strength Properties

The Poroton manufacturer supplied the mortar that was used during prism construction. There was not a mortar ingredient list on the packaging. Three 2" x 2" x 2" cubic and three 2x4 inch cylindrical mortar specimens were made. It was observed that there was considerable shrinkage of the mortar after curing. The cubic specimens were used to determine the compressive strength of the mortar, with the average being 1090 psi. Figure 2.5 shows the mortar cubes during compression testing. The cylinders were tested flatwise to determine a split cylinder tensile strength, Figure 2.6. The average tensile strength was 102 psi. All the mortar test results are listed in Table 4.

TABLE 4 - Mortar Test Results

Specimen	Compressive Strength from 2x2x2 inch cube (psi)	Split Tensile Strength from 2x4 inch cylinder (psi)
1	920	80
2	1310	99
3	1040	126
Average	1090	102



FIGURE 2.5 - Mortar Cube Testing

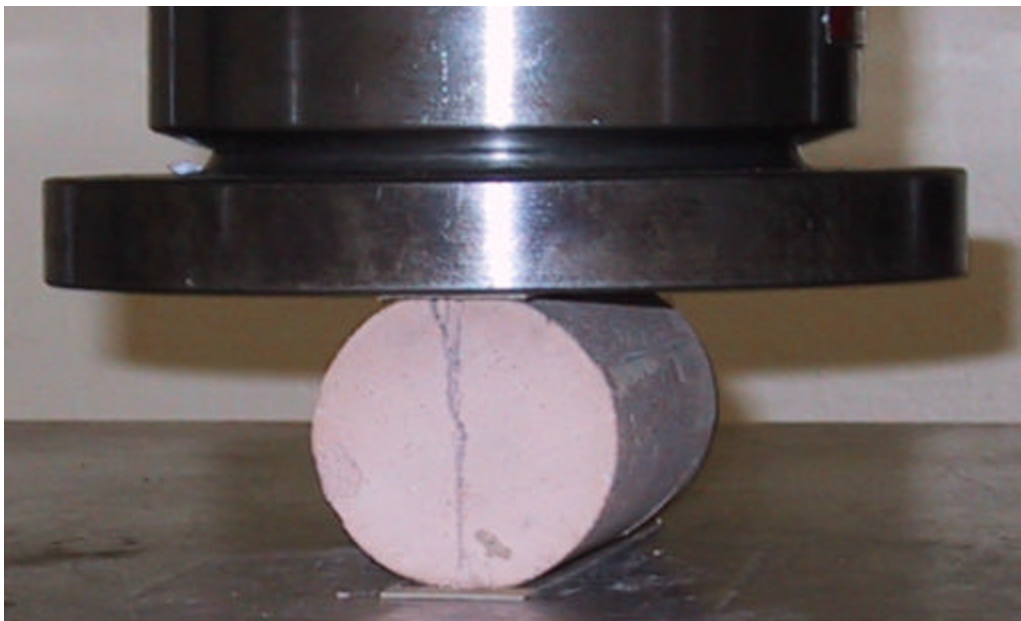


FIGURE 2.6 - Cylinder Tensile Strength Test

Chapter 3

Prism Strength Properties

3.1 Prism Construction

There were two phases of prism construction. The first phase included the construction of seven prisms for each size unit that would be used for the flexural bond strength tests. Only after the flexural bond strength tests were complete could the second phase of prism construction begin. The second phase included the construction of six prisms for each size unit that would be used for the prism compressive strength determination.

3.2 Phase I of Prism Construction

Unit preparation for prism construction began with the visual inspection of the units. This visual inspection checked for any major abnormalities in the unit's cross-section, i.e. cracks or missing pieces. Next, the units were cleaned of any dust or loose debris. The units were then placed in groups of two. The groups of two would then become a single prism.

In order to adhere units together to make up a prism, the manufacturer's mortar was used. Preparation of mortar was guided by the mixing instructions. The mixing instructions gave information about proportion of

ingredients and desired consistency of mortar after mixing. The specified proportion was 10 liters of water for every 25 kilograms of mortar mix, resulting in a 2.5:1 mortar mix to water ratio. Since the mortar mix was packaged in 25 kg bags, it was decided not to use the entire package of mortar mix. Instead, 10 kilograms of mortar mix and 4 liters of water were used. The instructions called for a clean mortar bucket to be filled with the appropriate amount of water. Then, under vigorous stirring, add the mortar mixture. Stirring and tamping continued until mortar was free from clumps and globs. The mortar was then placed into a rectangular pan for dipping of units (see Figure 3.1)



FIGURE 3.1 – Pan of Mortar

Upon completion of mortar preparation, the first unit, which would become the bottom unit in the prism, was placed on a flat, level surface. The second unit was dipped into the mortar pan and carefully placed on top of the first unit (see Figure 3.2 & 3.3 respectively). The total time of prism construction was approximately 2 hours, which was about half of the allowable process time recommended by the manufacturer. The units went undisturbed for 3-4 days. Figure 3.4 depicts units after the 3-4 day period. After that, they were gently moved to a location that would allow them to cure in controlled climate conditions for the rest of the recommended 28 days of curing. Ponce et al. (1999) suggest that the units should be stored in air to achieve a moisture content equal to about 20% of total absorption.



FIGURE 3.2 – Unit Dipping



FIGURE 3.3 - Second Unit Placement



FIGURE 3.4 - Prisms 3-4 Days After Construction

3.3 – Flexural Bond Strength Testing Apparatus

Many flexural bond strength tests have been conducted on masonry specimens in the past. The method prescribed in ASTM 1072 – 92 *Measurement of Masonry Flexural Bond Strength* was used for the testing of flexural bond strength. However, the testing apparatus was modified so that a “pure” couple could be applied to the mortar joint. The testing apparatus, also known as a “bond wrench”, prescribed in the ASTM Specification consisted of a steel frame and lower clamping bracket that could support prisms of 2 or more units. An upper clamping bracket was secured to the top unit in the prism. This upper bracket was connected to a single loading arm where the flexural load was applied. In theory, the application of this single bending load causes an unbalanced stress distribution across the prism cross-section (Figure 3.5 a). This distribution is composed of two components, a uniform axial compressive stress distribution and a linear flexural stress distribution. These are shown in Figure 3.5 b & c respectively. The rate at which the compressive load affects the unbalanced stress distribution is inversely proportional to the length of the loading arm. The longer arm reduces the load required to produce the same flexural

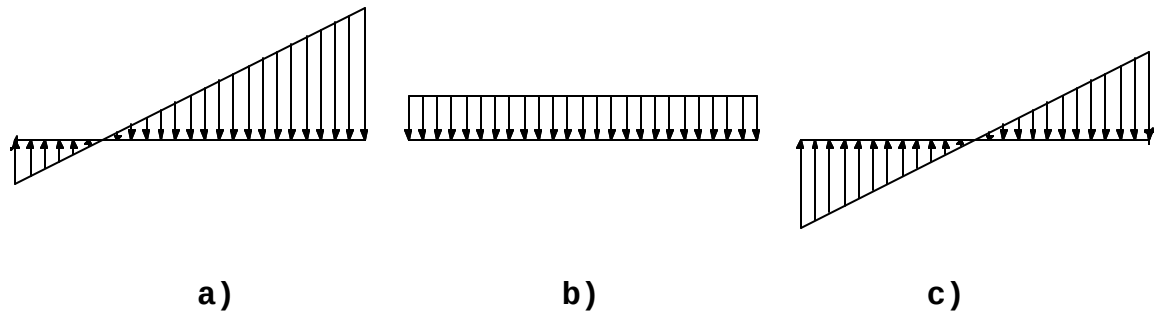


FIGURE 3.5 - Stress Distributions

stress. Therefore, the longer the arm the smaller the influence the compressive load has on the unbalanced stress distribution. McGinley (1996) states that the discrepancy of actual strain distributions, compared to linear distributions assumed by theory, appears to be more pronounced as the percentage of axial stress, relative to the peak flexural stress, is increased.

To get a true measure of flexural strength, one needs to minimize the effect of the compressive load on the prism. McGinley suggests that the load should be placed as far away from the prism as possible. Instead of creating an arm with a substantially longer length, the new design of the “bond wrench” incorporates two loading arms on opposite sides of the upper clamping bracket. Figure 3.6 on the next page represents this new design of the bond wrench.

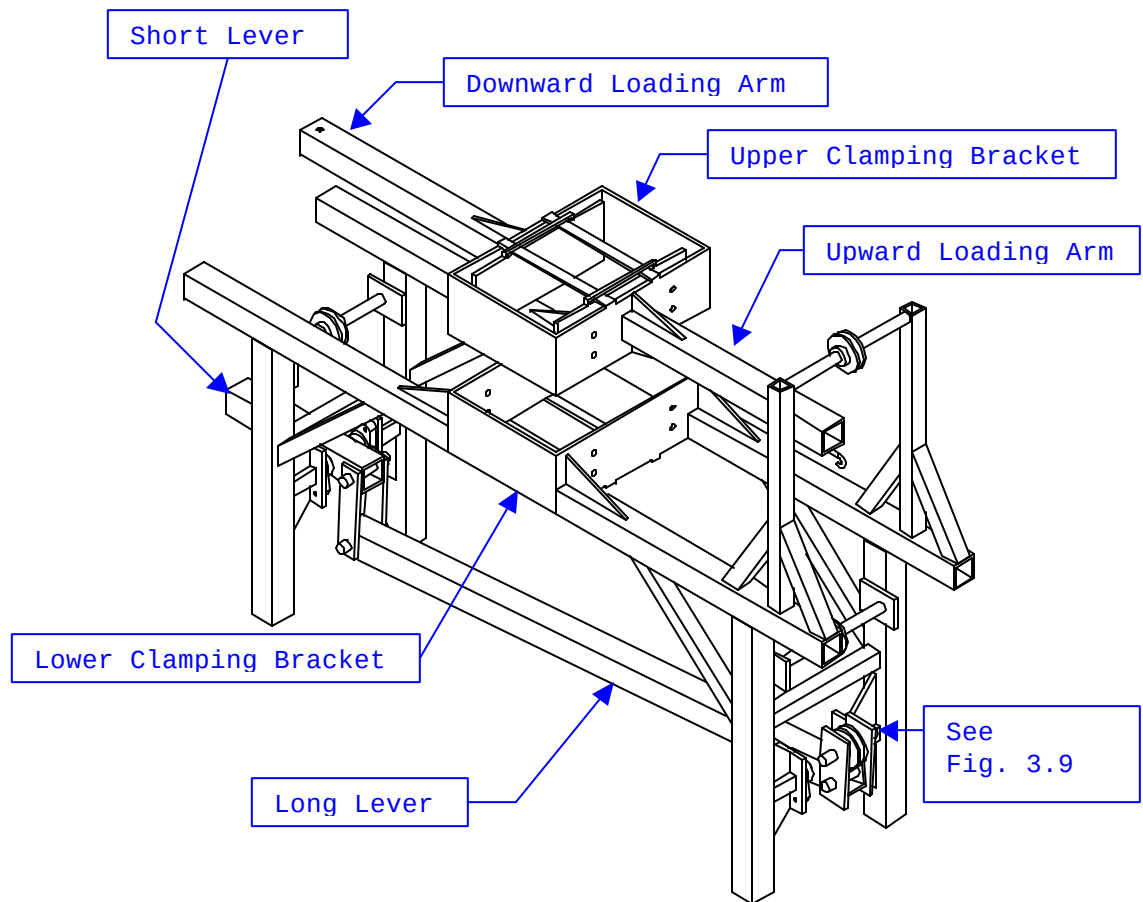


FIGURE 3.6 – The New Bond Wrench

If one arm is loaded with a force that is equal in magnitude but opposite in direction from the other, then the compressive stress distribution will be minimal with respect to the applied magnitude of loading. Summing forces in the vertical direction at the mortar joint being tested, one can see that the downward load negates the upward load. This only leaves the weight of the upper clamping bracket being applied to the joint. The stress associated with the upper clamping bracket's weight was minimal in comparison to the loadings that were required to

produce the ultimate flexural stress. McGinley also demonstrates that loading direction can have an effect on the strain distribution, since placing an upward force on a single loading arm "may move the point of application of the bearing stresses on the tension side of the prism, add to the tension strains at the mortar interface, and most importantly, may change the strain distribution in the prism from that produced by a downward loading."

In order to achieve the equal and opposite forces applied to the two arms, a lever system, in conjunction with a system of near frictionless pulleys, was used (see Figure 3.7 for system schematic). Figure 3.8 shows the upper left pulley. The new bond wrench applies both loadings on their respective sides at a distance of 2'-6" from the centroid of the upper clamping bracket. The lever system was designed to apply sufficient tension into the steel wire cable, Figure 3.9, that would eventually apply the equal and opposite loads to their respective arms. The lever system applied 60 lbs for every one pound applied to the loading end of the short lever.

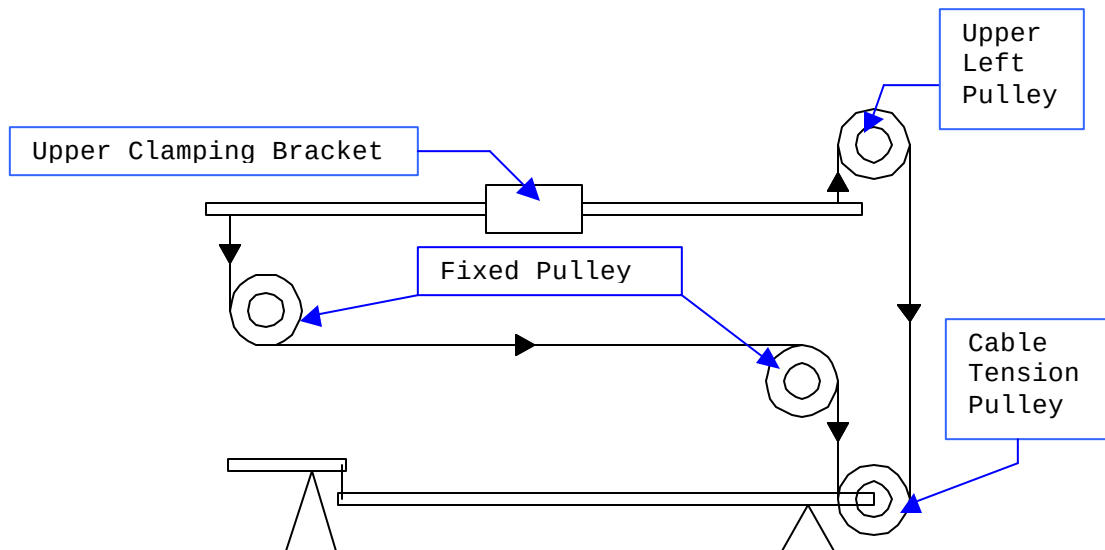


FIGURE 3.7 - Lever/Pulley System Schematic



FIGURE 3.8 - Frictionless Pulley

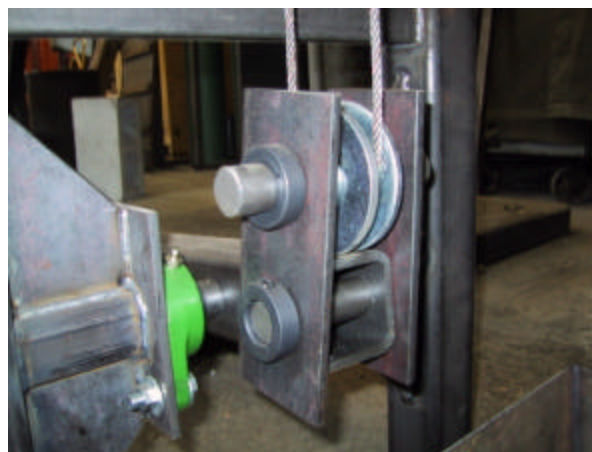


FIGURE 3.9 - Steel Cable Tensioning Device

3.4 Flexural Bond Strength Testing and Results

Once built, the new bond wrench was used with the following procedure. A prism was loaded into the lower clamping bracket, positioned so that its centroid was collinear with the centroid of the lower clamping bracket (see Figure 3.10). Then the prism was secured in the lower bracket by placing the two clamping plates along the long sides of the prism and tightening the eight bolts on each side. Next, the upper clamping bracket was placed so that it too was geometrically centered on the prism. The upper bracket has two plates already attached by the horizontal bars on top. These bars act as a guide for the plates. In addition, they provided temporary support for the upper clamping bracket before the tightening of the clamping plates.

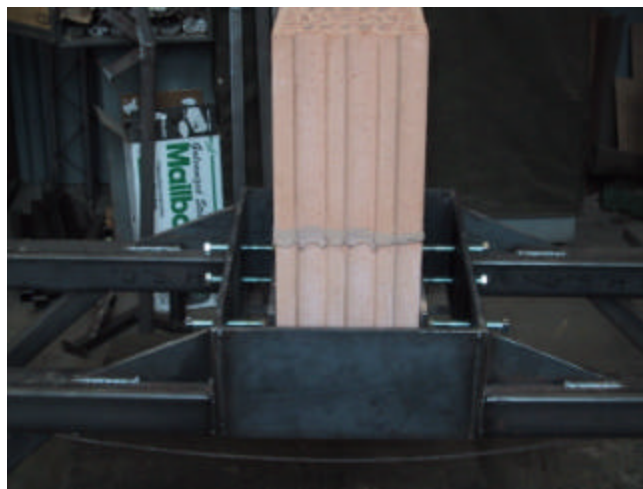


FIGURE 3.10 - Prism Placement in Lower Clamping Bracket

The two brackets were then clamped snug to the prism exposing the joint to be tested, Figure 3.11. The two thin plates welded to the top clamp bracket allow the upper clamping bracket to rest on the prism and to act as support for the two clamping plates during placement. This configuration minimizes the work involved in tightening the upper clamping bracket. The steel cable was attached to the arms and relieved of all slack. The lever system was then used to add additional tension to the cable thus resulting in the ultimate flexural load. The load was applied at a uniform rate such that the total load was applied in not less than 1 minute nor more than 3 minutes. Bond strength is not significantly affected by the rate of testing, within the low and high rates currently permitted by ASTM C1072 (Ponce et al. 1999). Therefore, the possible effects of varied loading rates between specimens have been ignored. The cable tension was measured with an inline load cell. The placement of the load cell is depicted in Figure 3.12.

Unfortunately, one of the small prisms broke during handling, leaving six small prisms and seven large prisms to be tested. Figure 3.13 depicts a prism after flexural failure.

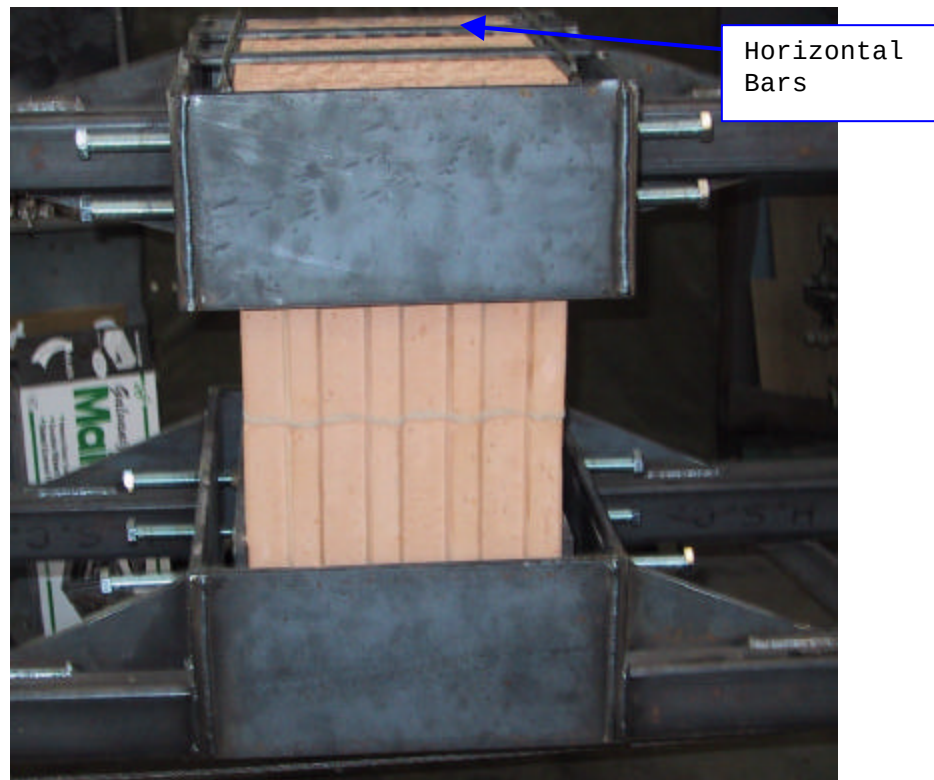


FIGURE 3.11 - Upper and Lower Clamping Brackets in Place



FIGURE 3.12 - LOAD CELL



FIGURE 3.13 - Prism After Flexural Failure

Additional photos of flexural testing can be found in Appendix C. A summary of test results and subsequent stress calculation results are listed in Tables 5, 6, and 7.

Based on a factor of safety of 3 for flexural tension, the allowable tensile stress for unreinforced Poroton construction will be 21 psi. This allowable is comparable to the allowable flexural tensile stress of 19 psi for hollow ungrouted units normal to the bed joint for Type N Portland cement/lime mortar in the ACI 530-99/ASCE 5-99/TMS 402/99 Building Code Requirements for Masonry Structures.

TABLE 5 - Summary of Flexural Strength Tests

Property	Small Block	Large Block	Combined
Number of Tests	6	7	13
Mean Flexural Tension Strength (psi)	70.4	60	64.8
Coefficient of Variation (%)	24.6	32.3	28.5

TABLE 6 - Large Prism Flexural Strength Test Results

Large Block Prism Number	Area (in ²)	Applied Comp. Load (lbs)	Applied Comp. Stress (psi)	Moment of Inertia (in ⁴)	"C" (in)	Cable Tension (lbs)	Applied Moment (lb-in)	Applied Flexural Stress (psi)	Applied Stress @ Tensile Face (psi)	Applied Stress @ Comp. Face (psi)
1	68.73	130.0	1.89	578.29	4.656	102.1	6215.3	50.04	48.15	51.94
2	68.73	130.0	1.89	578.29	4.656	178.5	10866.2	87.49	85.60	89.38
3	68.73	130.0	1.89	578.29	4.656	142.3	8662.5	69.75	67.86	71.64
4	68.73	130.0	1.89	578.29	4.656	167.0	10166.1	81.86	79.96	83.75
5 (n)	68.73	130.0	1.89	578.29	4.656	124.5	7578.9	61.02	59.13	62.92
6 (n)	68.73	130.0	1.89	578.29	4.656	105.4	6416.2	51.66	49.77	53.55
7 (n)	68.73	130.0	1.89	578.29	4.656	65.3	3975.1	32.01	30.12	33.90
Average w/low out	68.73	130.0	1.89	578.29	4.656	136.6	8317.6	66.97	65.08	68.86
Average	68.73	130.0	1.89	578.29	4.656	126.4	7697.2	61.98	60.08	63.87

(n) - Incorrect Bond Orientation

TABLE 7 - Small Prism Flexural Strength Test Results

Small Block Prism Number	Area (in ²)	Applied Comp. Load (lbs)	Applied Comp. Stress (psi)	Moment of Inertia (in ⁴)	"C" (in)	Cable Tension (lbs)	Applied Moment (lb-in)	Applied Flexural Stress (psi)	Applied Stress @ Tensile Face (psi)	Applied Stress @ Comp. Face (psi)
1	54.39	130.0	2.39	233.08	3.313	75.7	4608.2	65.49	63.10	67.88
2	54.39	130.0	2.39	233.08	3.313	112.7	6860.6	97.50	95.11	99.89
3	54.39	130.0	2.39	233.08	3.313	104.3	6349.3	90.23	87.84	92.62
4	54.39	130.0	2.39	233.08	3.313	70.7	4303.9	61.17	58.78	63.56
5 (n)	54.39	130.0	2.39	233.08	3.313	79.8	4857.8	69.04	66.65	71.43
6 (n)	54.39	130.0	2.39	233.08	3.313	61.5	3743.8	53.21	50.82	55.60
7 (n)	54.39	130.0	2.39	233.08	3.313	8.2	499.2	7.09	4.70	9.48
Average w/low out	54.39	130.0	2.39	233.08	3.313	84.1	5120.6	72.77	70.38	75.16
Average	54.39	130.0	2.39	233.08	3.313	73.3	4460.4	63.39	61.00	65.78

(n) - Incorrect Bond Orientation

3.5 - Phase II of Prism Construction

Due to the lack of a sufficient number of specimens, individual units that remained structurally intact after flexural bond strength testing were used for the prism compressive strength tests. After a visual inspection, enough units to make six prisms of each size remained.

Before compressive prisms would be constructed, it was deemed appropriate to bond the previously unbonded end of the units. Like the compression testing of individual units, a smooth flat surface was required for compression testing of the prisms. The placement of a HydroCAL® cap over the previously bonded end could only begin after any excess mortar was removed from the units. Figure 3.14 shows the typical amount of excess mortar on the units after flexural strength testing.

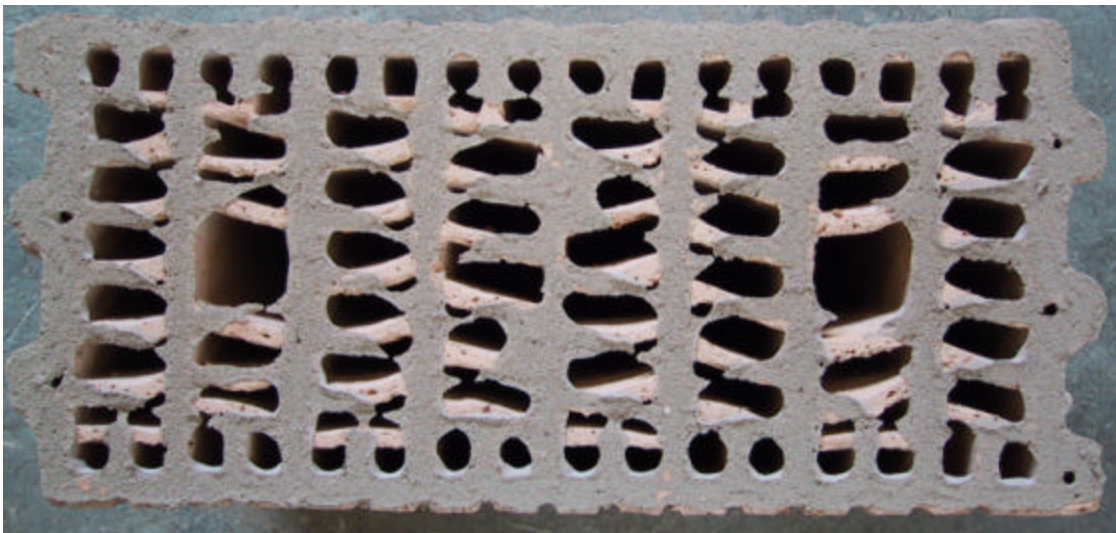


FIGURE 3.14 - Excess Mortar on Unit

The removal of excess mortar was accomplished by utilizing a belt sander with a semi-course grit belt. The excess mortar was removed to the extent of not damaging the units themselves.

The procedure for the placement of the HydroCAL® caps was the same as when they were placed for the individual unit compression tests. However, the mortar that was not removed, for fear of destroying the integrity of the units, acted like a sponge and absorbed most the moisture of the HydroCAL® mixture. The sponge like behavior prevented the caps from gaining sufficient strength to remain bonded to the unit. Instead of using HydroCAL® caps, it was deemed necessary to use Neoprene rubber pads with a durometer of 70 to act as the caps for prism compression testing. According to ASTM, the rubber pads are a viable option instead of the gypsum based HydroCAL® caps.

After the revision in the cap, prism construction began. Again, prism construction followed the recommended directions of the manufacturer. These directions were previously described in "Phase One" of prism construction.

3.6 – Prism Compressive Strength Testing and Results

The prisms were tested on a Tinius-Olsen machine. Conforming to the procedure specified in ASTM C 67, the

loading was applied at a constant rate such that the ultimate load was again achieved within the one to three minute timeframe. The results of the twelve specimens are listed in Table 8. The actual compressive stress (f'_m) for the Poroton, regardless of size, is approximately two-thirds of that for a standard concrete block. The value of compressive stress for CMU's is given in ACI 513-99 as 1350 psi.

Just like the unit compressive testing, the prisms' ductile failure began to occur when the units themselves began to split along their vertical axes before the ultimate load had been reached, Figure 3.15. Additionally, significant spalling of extreme face shells occurred in all tests. Ultimate failure is depicted in Figure 3.16. Other pictures of prism compressive testing are located in Appendix D.

TABLE 8 - Compressive Prism Strength Test Results

Large Block	Area* (in ²)	Compressive Load (lbs.)	Compressive Stress (psi)
1	68.73	70000	1018.5
2	68.73	72200	1050.5
3	68.73	64700	941.4
4	68.73	55600	809.0
5	68.73	55800	811.9
6	68.73	61200	890.4
Average	68.73	63250	920.3
		COV (%)	8.4
Small Block	Area* (in ²)	Compressive Load (lbs.)	Compressive Stress (psi)
1	53.07	43500	819.7
2	53.07	42000	791.4
3	53.07	47800	900.7
4	53.07	49200	927.1
5	53.07	52400	987.4
6	53.07	49300	929.0
Average	53.07	47367	892.5
		COV (%)	4.0
Combined	Area* (in ²)	Compressive Load (lbs.)	Compressive Stress (psi)
Average	60.90	55308	908.2
		COV (%)	6.4

* Area was estimated by AutoCAD Drawings

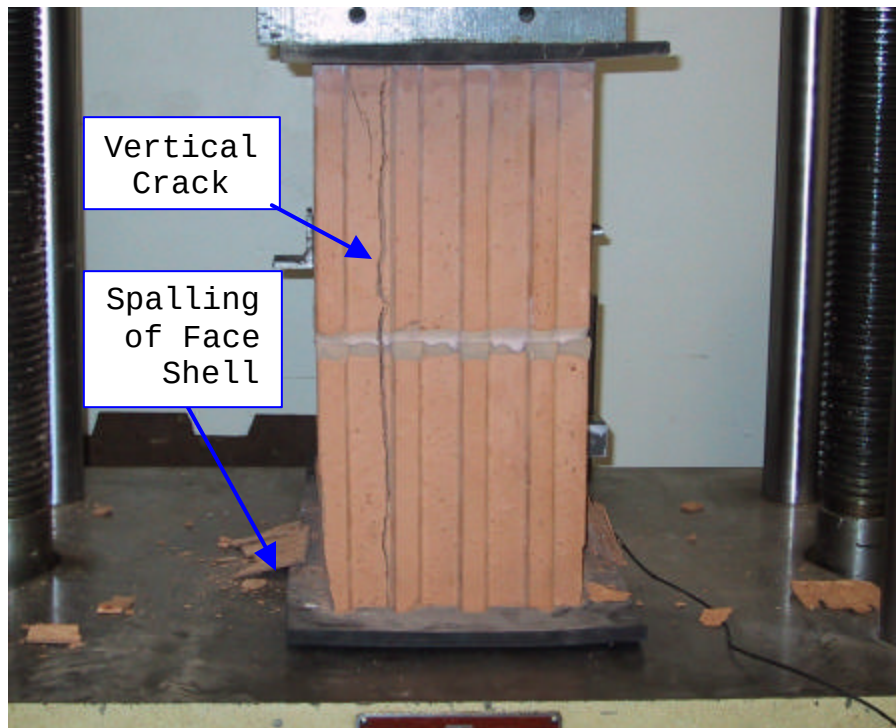


FIGURE 3.15 - Splitting of Prism During Compression Testing

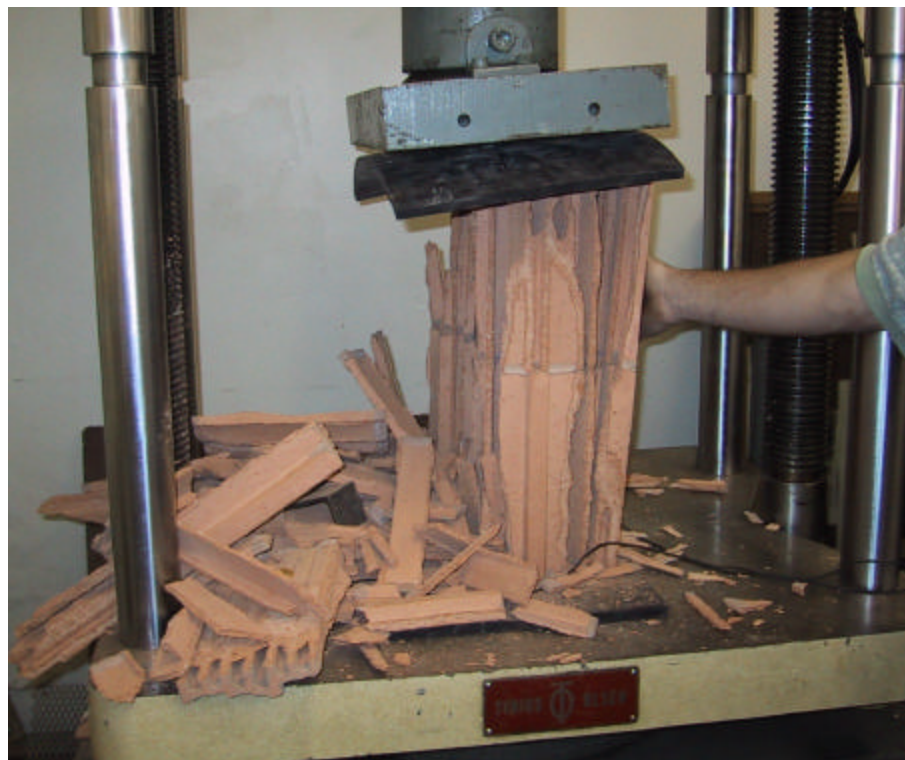


FIGURE 3.16 - Ultimate Compressive Prism Failure

Chapter 4

Design Considerations

4.1 Unreinforced Masonry

Conventional unreinforced masonry design procedures can be used when designing with Poroton masonry. The only difficulty is determining cross-sectional properties, especially the moment of inertia. The moment of inertia can either be determined directly using an AutoCAD drawing of the cross-section of the unit, or can be estimated as 60% of the gross moment of inertia (see Section 2.2).

In order to design unreinforced Poroton walls, one may utilize the following properties in Table 9.

TABLE 9 - Summary of Unreinforced Design Properties

Design Property	Block Size	
	Large	Small
Net Area (A_N) =	54.5in ² /ft	42.9in ² /ft
Net Section Modulus (S_N) =	102.1in ³ /ft	58.4in ³ /ft
Net Moment of Inertia (I_N) =	474.3in ⁴ /ft	192.9in ⁴ /ft
Radius of Gyration (r) =	2.95in	2.12in
Ultimate Tensile Stress =	63psi	63psi
Allowable Tensile Stress (f_t) =	21psi	21psi
Ultimate Compressive Stress (f'_m) =	900psi	900psi
Allowable Compressive Stress ($1/3 \times f'_m$) =	300psi	300psi
Modulus of Elasticity ($700 \times f'_m$) =	630ksi	630ksi

The geometric properties listed in Table 9 were obtained via processes described in Chapter 2. The allowable tensile stress and ultimate compressive stress are approximate averages of both the large and small Poroton unit tests. The allowable compressive stress and modulus of elasticity are based on the formulas given for clay masonry units in the Masonry Standards Joint Committee Report. These properties provide the flexibility to design unreinforced shear, bearing, and non-bearing Poroton walls.

Table 10 gives the allowable lateral wind loads for various heights of non-bearing Poroton walls. Since some structural design codes permit a 1/3 increase in allowable stress for wind loadings and some do not, wind loads both considering and not considering the 1/3 stress increase are included. Specifically, the allowable loads tables are based two key assumptions.

- 1) The wall is simply supported at the top and at the base.
- 2) The maximum flexural tension due to wind loadings occurs at mid-span.

See Appendix E for a sample calculation of tabulated values.

Typically there would be an approximate 3/8" plaster coat on at least one side. The effect of the plaster coating has not been included in Table 10. Revised cross-sectional properties including the plaster coat on one and both sides are given in Table 11. Tables 12 and 13 give the allowable wind loads for Poroton masonry including the plaster coatings. In the development of Tables 12 and 13, it was also assumed that the plaster will act compositely with the Poroton unit, the plaster has the same modulus of elasticity as the Poroton masonry, and the allowable flexural tensile stress remains 21 psi. The addition of one plaster coating will increase the allowable loadings on a wall constructed with small units by about 16%. Two coatings of plaster will increase the allowable loading for the wall with small units by about 41%. The increases in allowable loadings for walls constructed with the large units are 13% and 33% respectively.

TABLE 10 - Allowable wind loads for Poroton non-bearing walls

Wall Height (ft)	Allowable Wind Pressure (psf)			
	1/3 stress increase		No 1/3 stress increase	
	Small Block	Large Block	Small Block	Large Block
5	46.3	81.1	35.4	62.1
6	32.6	57.0	25.0	43.8
7	24.2	42.4	18.6	32.7
8	18.7	32.8	14.5	25.4
9	15.0	26.2	11.6	20.4
10	12.3	21.5	9.5	16.7
11	10.2	18.0	8.0	14.0
12	8.7	15.3	6.8	12.0
13	7.5	13.2	5.9	10.3
14	6.5	11.5	5.1	9.0

TABLE 11 - Cross-sectional properties including a 3/8 inch plaster coat

Plaster Coating	Small block			Large block		
	I_n (in ⁴)	S_n (in ³)	A_n (in ²)	I_n (in ⁴)	S_n (in ³)	A_n (in ²)
One side	243.2	71.6, 67.9	56.3	572.3	121.4, 115.6	70.9
Two sides	302.4	82.3	60.8	684.5	136.4	75.4

Note: two values are given for the section modulus for plaster coating on one side since the cross-section is no longer symmetrical.

TABLE 12 - Allowable wind loads for Poroton non-bearing walls with a 3/8" plaster coat on one side

Wall Height (ft)	Allowable Wind Pressure (psf)			
	1/3 stress increase		No 1/3 stress increase	
	Small Block	Large Block	Small Block	Large Block
5	53.9	91.8	41.2	70.3
6	37.8	64.4	29.0	49.6
7	28.1	47.9	21.7	37.0
8	21.8	37.1	16.8	28.7
9	17.4	29.6	13.5	23.0
10	14.3	24.3	11.1	18.9
11	11.9	20.3	9.3	15.9
12	10.1	17.2	7.9	13.5
13	8.7	14.8	6.8	11.7
14	7.6	12.9	6.0	10.2

TABLE 13 - Allowable wind loads for Poroton non-bearing walls with a 3/8" plaster coat on both sides

Wall Height (ft)	Allowable Wind Pressure (psf)			
	1/3 stress increase		No 1/3 stress increase	
	Small Block	Large Block	Small Block	Large Block
5	65.3	108.4	49.9	82.9
6	45.9	76.0	35.2	58.5
7	34.1	56.5	26.3	43.6
8	26.4	43.8	20.4	33.9
9	21.1	35.0	16.4	27.2
10	17.3	28.6	13.4	22.3
11	14.4	23.9	11.3	18.7
12	12.3	20.3	9.6	16.0
13	10.6	17.5	8.3	13.8
14	9.2	15.3	7.2	12.1

4.2 Reinforced Masonry - Exact Method

An “exact” method was developed for analyzing reinforced Poroton walls under flexural loading. The method was similar to a “T-section” analysis, except that there were multiple “stem” widths or sections in the analysis. The cross-section of the Poroton wall was modeled as a series of rectangles with varying widths. The steel reinforcement was assumed to be located at the mid-depth of the wall. See Figure 4.1 for an example. An Excel spreadsheet was used to determine the location of the neutral axis and then the moment capacity. The first step was to assume a distance “ kd ” from the extreme compression fibers to the neutral axis. Then a comparison of the moment of compression area versus the moment of tension area, which are supposed to be equal in an uncracked cross section, was done. Adjustments were made to “ kd ” until the moments of areas were indeed equal.

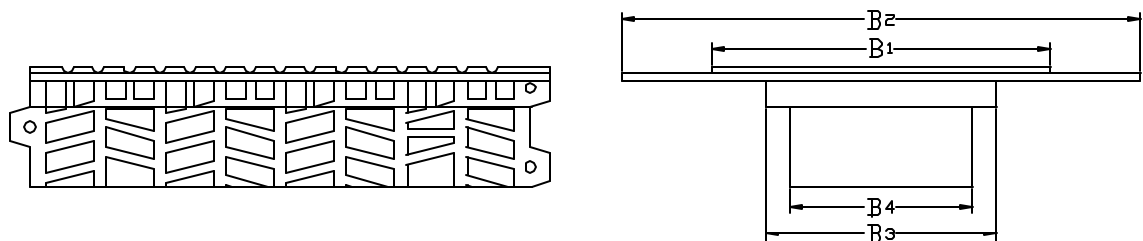


FIGURE 4.1 - Model of Small Unit's Cross-section for Exact Method

The first rectangle considered was that containing the flutes on the outer surface of the units. The thickness of this rectangle was the depth of the flutes, and the width was the solid portion of the Poroton flutes. The second rectangle was the solid face shell of the unit. The third rectangle accounted for the webs of the unit. There are additional webs near the face, so there was a rectangle that accounted for the additional web area. Finally, there was a fourth rectangle that accounted for the web area near the center of the unit and the proportional width of the grouted cell. Depending on how the unit is laid, the cross-webs, which typically were not parallel to the face but at an angle, may not line up from unit to unit and were not considered.

What was thought to be the most common cases were analyzed. These cases included the use of #4, #5, and #6 reinforcing bars. In addition, special units with two cells large enough for reinforcing would be considered. These units look similar to a standard CMU in cross-section. The face shells of the reinforceable units are approximately equal to the Poroton units investigated earlier. Reinforcing spacing considered for the small block were 7.25, 14.5, 29, and 43.5 inches. For the large

block, reinforcing spacing of 7.4, 14.8, 29.6, 44.4, and 59.2 inches were considered. These odd spacing were based on the length of the Poroton units that were tested. For the small units, the spacing of 7.25 inches would be a bar in every cell, and hence solidly grouted. The spacing of 14.5 inches would be a bar in every other cell, with only the cell containing the bar being grouted. Both of these walls would only use the special reinforceable Poroton units. The spacing of 29 inches corresponds to a wall constructed with a standard Poroton unit and a reinforceable Poroton unit being alternately used in the wall. Note that a spacing of 21.75 inches is not considered, as it would require using half units at every bar. The maximum spacing investigated was 43.5 inches. It consists of two regular Porotons and one reinforceable Poroton unit. This spacing approximately corresponds to the MSJC code limit of six times the wall thickness for the effective compression width per bar (actual limit is 39.6 inches for the small block and 55.8 inches for the large units). If one wished to use a larger spacing, that could be done by limiting the effective compression width to six times the wall thickness.

Table 14 gives the allowable moment capacities for the small block using the “exact” method, and Table 15 gives the allowable moment capacities for the large block using the “exact” method.

4.3 Reinforced Masonry - Approximate Method

An approximate method was also developed for analysis. This was a T-beam analysis, similar to that commonly used for partially grouted walls (Schneider & Dickey). A flange as described below is considered along with a web that only consists of the grouted cell. These are shown in Figure 4.2 for the small unit. The distance from the neutral axis to the extreme compression fiber, d , was taken as half the thickness of the unit minus half the flute depth. In other words, “ d ” was the distance from the center of the block to the center of the flutes. A face shell thickness was taken as the width of the face shell including the flutes.

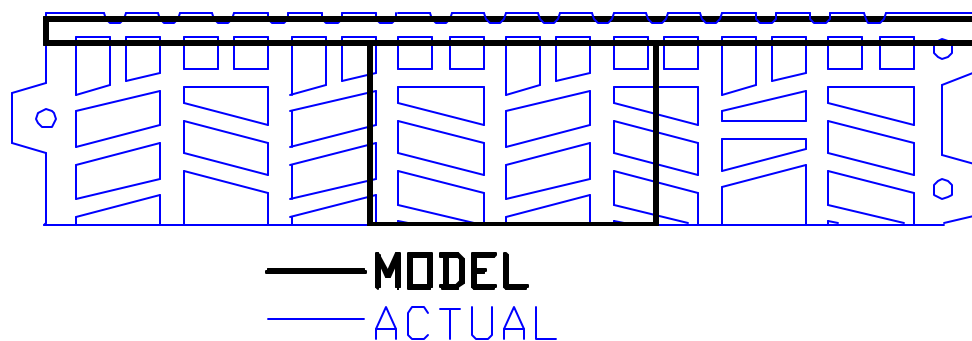


FIGURE 4.2 - Simplified Model of Small Unit's Cross-section for Approximate Method

This is an approximation, as it assumes that ignoring the voids in the flutes on the face shell can approximately include the neglected web area of the Poroton units. The results from the approximate analysis are also included in Tables 14 and 15 for the small and large units respectively. Note that the approximate method tends to be slightly conservative. Tables 16 and 17 show a typical relationship between allowable wind loads for unreinforced masonry and reinforced masonry of both small and large Poroton units. In formulating Tables 16 and 17, only the reinforced spacings of #4 bars were compared. Figures 4.3 and 4.4 represent this same relationship, only graphically.

TABLE 14 - Allowable Moment Capacities per Foot of Wall for Reinforced Walls with Small Poroton Units (kip-ft/ft)

Bar Spacing	#4			#5			#6		
	Exact	T-Beam	% Diff.	Exact	T-Beam	% Diff.	Exact	T-Beam	% Diff.
7.25"	0.86	0.87	(1.2)	0.94	0.96	(2.1)	1.01	1.01	0.0
14.5"	0.56	0.59	(5.1)	0.61	0.64	(4.7)	0.65	0.68	(4.4)
29"	0.46	0.45	2.2	0.51	0.48	6.3	0.55	0.51	7.8
43.5"	0.39	0.39	0.0	0.46	0.43	7.0	0.50	0.45	11.1

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TABLE 15 - Allowable Moment Capacities per Foot of Wall for Reinforced Walls with Large Poroton Units (kip-ft/ft)

Bar Spacing	#4			#5			#6		
	Exact	T-Beam	% Diff.	Exact	T-Beam	% Diff.	Exact	T-Beam	% Diff.
7.4"	1.66	1.68	(1.2)	1.85	1.87	(1.1)	1.99	2.00	(0.5)
14.8"	1.06	1.10	(3.6)	1.17	1.21	(3.3)	1.25	1.29	(3.1)
29.6"	0.84	0.81	3.7	0.95	0.88	8.0	1.02	0.93	9.7
44.4"	0.57	0.57	0.0	0.85	0.77	10.4	0.92	0.81	13.6
59.2"	0.43	0.43	0.0	0.66	0.66	0.0	0.86	0.75	14.7

TABLE 16 - Comparison of Small Unreinforced Poroton to Small Reinforced Poroton using #4 Bars

Wall Height (feet)	Allowable Wind Pressure (lb/ft ²)				
	Unreinforced	Reinforced			
		Spacing (in)			
		7.25	14.5	29.0	43.5
		M _{ALLOWABLE} (k-ft/ft)			
		0.86	0.56	0.46	0.39
5	35.4	275.2	179.2	147.2	124.8
6	25.0	191.1	124.4	102.2	86.7
7	18.6	140.4	91.4	75.1	63.7
8	14.5	107.5	70.0	57.5	48.8
9	11.6	84.9	55.3	45.4	38.5
10	9.5	68.8	44.8	36.8	31.2
11	8.0	56.9	37.0	30.4	25.8
12	6.8	47.8	31.1	25.6	21.7
13	5.9	40.7	26.5	21.8	18.5
14	5.1	35.1	22.9	18.8	15.9

TABLE 17 - Comparison of Large Unreinforced Poroton to Large Reinforced Poroton using #4 Bars

Wall Height (feet)	Allowable Wind Pressure (lb/ft ²)					
	Unreinforced	Reinforced				
		Spacing (in)				
		7.4	14.8	29.6	44.4	59.2
		M _{ALLOWABLE} (k-ft/ft)				
		1.66	1.06	0.84	0.57	0.43
5	62.0	531.2	339.2	268.8	182.4	137.6
6	43.8	368.9	235.6	186.7	126.7	95.6
7	32.7	271.0	173.1	137.1	93.1	70.2
8	25.4	207.5	132.5	105.0	71.3	53.8
9	20.4	164.0	104.7	83.0	56.3	42.5
10	16.7	132.8	84.8	67.2	45.6	34.4
11	14.0	109.8	70.1	55.5	37.7	28.4
12	12.0	92.2	58.9	46.7	31.7	23.9
13	10.3	78.6	50.2	39.8	27.0	20.4
14	9.0	67.8	43.3	34.3	23.3	17.6

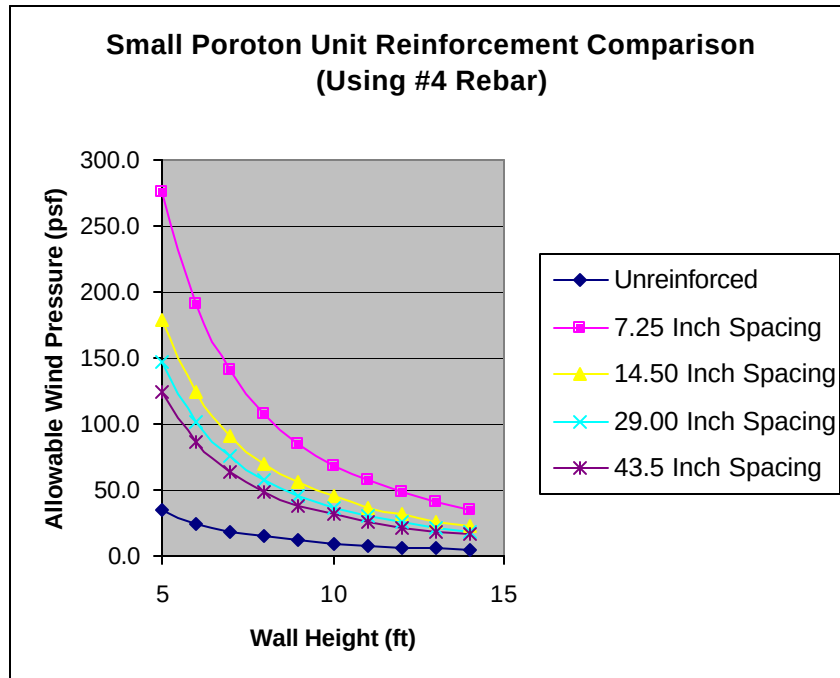


FIGURE 4.3 – Comparison of Small Poroton Reinforcing

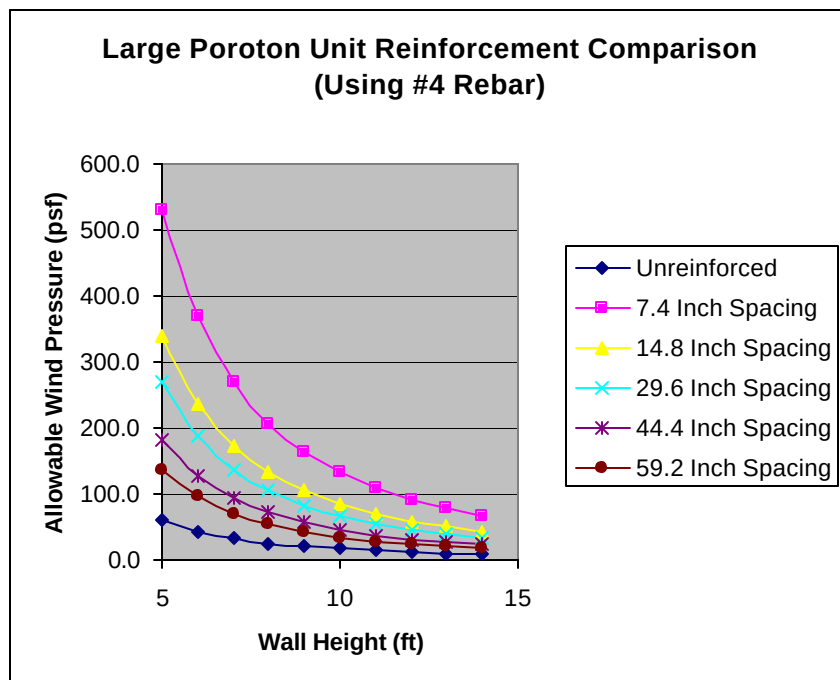


FIGURE 4.4 – Comparison of Large Poroton Reinforcing

Chapter 5

Summary and Recommendations

5.1 Summary of Constituent Properties

The size and shape of the Poroton units posed some difficulty in the measurement of volumetric properties. However, all the desired properties were obtained, some by unconventional methods. From a design standpoint, the most important properties of the Poroton would include a net density of approximately 102 lbs/ft³, and a net ultimate compressive strength of 2900 psi regardless of the size of the unit. Specific cross-sectional properties were determined using AutoCAD's ability to solve for the moment of inertia of extremely complex cross-sections (again, refer to Table 2.) Mortar properties were observed according to the applicable ASTM standards. Mortar compressive strength was approximately 1090 psi and tensile strength was 102 psi.

5.2 Summary of Prismatic Properties

The recommended construction procedures of Poroton masonry are very simple in nature. An unskilled layperson could construct a wall with minimal help from a skilled mason. This was also the case in prism construction. Once constructed, the Poroton prisms were subjected to flexural

loading. The design of the testing device prescribed by the ASTM standards for flexural testing of clay fired structural units, has been investigated many times. The conclusion of past investigations has determined that the larger the ratio of axial stress to flexural stress applied to the specimen, the further away from a linear stress distribution one gets. This desired linear stress distribution is the result of simple beam theory. The proposed solution was to redesign the testing device to have two moment arms that would be loaded with the same magnitude but opposite directions. Thus, resulting in a "pure" moment loading. The associated allowable flexural stress of 21 psi for Poroton masonry compares well with the 19 psi of standard concrete masonry units with Type N Portland Cement-Lime Mortar. Also investigated was ultimate compressive strength of the prisms. An ultimate compressive stress of 900 psi was observed. This too was relatively the same regardless of the size of the unit.

5.3 Recommendations for Design Procedures

Since there is very complex cross-sectional geometry, which makes determining design properties like the moment of inertia and section modulus extremely difficult and time consuming, this investigation leads to the recommendation

that the manufacturer produce design tables for the project designer to use during the structural design process.

Tables for both unreinforced and reinforced Poroton masonry should be made available. Samples of these design tables can be found in Chapter 4.

5.4 Other Recommendations for the Future of Poroton

Construction in the United States

Should Poroton units be adopted for construction in the United States, it is conjectured that the geometry of the unit would be modified to correspond to a standard 8-inch modular construction. In choosing the thickness of the unit, consideration should be given to the code requirement limiting the effective compression width per bar to six times the wall thickness. The largest spacing looked at, 43.5 and 59.2 inches for the small and large Poroton respectively, just exceed the code's limit, and would thus make design more complicated. The approximate method for reinforced masonry design suggested in this report would be an alternate means of analysis.

List Of References

List of References

- American Society for Testing and Materials, 2000 Annual Book of ASTM Standards. Section 4, Volume 4.05
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APPENDICES

APPENDIX A
(Actual Unit Mass Properties Observations)

TABLE A.1 – Actual Observations of Large Block #1

Actual Measurements			Large #1	
W_r	17614g	38.83Lbs	Height	9.844inches
W_s	21084g	46.48Lbs	Width	9.250inches
W_i	10365g	22.85Lbs	Length	14.656inches
W_d	17558g	38.71Lbs		

Absorption	
A	20.53lbs/ft ³ 20.08%

Moisture Content	
M.C.	1.588%

Density	
D	102.2lbs/ft ³

Net Volume	
V_n	0.379ft ³

Average Net Area	
A_n	66.48in ²

Gross Area	
A_g	135.6in ²

TABLE A.2 – Actual Observations of Large Block #2

Actual Measurements			Large #2	
W_r	17754g	39.14Lbs	Height	9.781inches
W_s	21182g	46.70Lbs	Width	9.250inches
W_i	10542g	23.24Lbs	Length	14.625inches
W_d	17640g	38.89Lbs		

Absorption	
A	20.77lbs/ft ³ 20.08%

Moisture Content	
M.C.	3.204%

Density	
D	103.5lbs/ft ³

Net Volume	
V_n	0.376ft ³

Average Net Area	
A_n	66.41in ²

Gross Area	
A_g	135.3in ²

TABLE A.3 – Actual Observations for Large Block #3

Actual Measurements			Large #3	
W_r	17788g	39.22lbs	Height	9.781inches
W_s	21199g	46.74lbs	Width	9.375inches
W_i	10555g	23.27lbs	Length	14.594inches
W_d	17684g	38.99lbs		

Absorption	
A	20.61lbs/ft ³ 19.88%

Moisture Content	
M.C.	2.959%

Density	
D	103.7 lbs/ft ³

Net Volume	
V_n	0.376ft ³

Average Net Area	
A_n	66.44in ²

Gross Area	
A_g	136.8in ²

TABLE A.4 – Actual Observations for Small Block #1

Actual Measurements			Small #1	
W_r	13701g	30.21lbs	Height	9.813inches
W_s	16453g	36.27lbs	Width	6.625inches
W_i	8086g	17.83lbs	Length	14.563inches
W_d	13652g	30.10lbs		

Absorption

A 20.888lbs/ft³
 20.517%

Moisture Content

M.C. 1.7494%

Density

D 101.81lbs/ft³

Net Volume

V_n 0.2956ft³

Average Net Area

A_n 52.061in²

Gross Area

A_g 96.477in²

TABLE A.5 – Actual Observations for Small Block #2

Actual Measurements			Small #2	
W_r	13722g	30.25lbs	Height	9.906inches
W_s	16445g	36.26lbs	Width	6.594inches
W_i	8107g	17.87lbs	Length	14.469inches
W_d	13628g	30.04lbs		

Absorption	
A	21.081lbs/ft ³ 20.671%

Moisture Content	
M.C.	3.3369%

Density	
D	101.98lbs/ft ³

Net Volume	
V_n	0.2946ft ³

Average Net Area	
A_n	51.389in ²

Gross Area	
A_g	95.403in ²

TABLE A.6 – Actual Observations for Small Block #3

Actual Measurements			Small #3	
W_r	13808g	30.44lbs	Height	9.813inches
W_s	16510g	36.40lbs	Width	6.594inches
W_i	8143g	17.95lbs	Length	14.469inches
W_d	13718g	30.24lbs		

Absorption

A 20.821lbs/ft³
 20.353%

Moisture Content

M.C. 3.2235%

Density

D 102.3lbs/ft³

Net Volume

V_n 0.2956ft³

Average Net Area

A_n 52.061in²

Gross Area

A_g 95.403in²

APPENDIX B
(Additional Photos of Unit Compressive Tests)

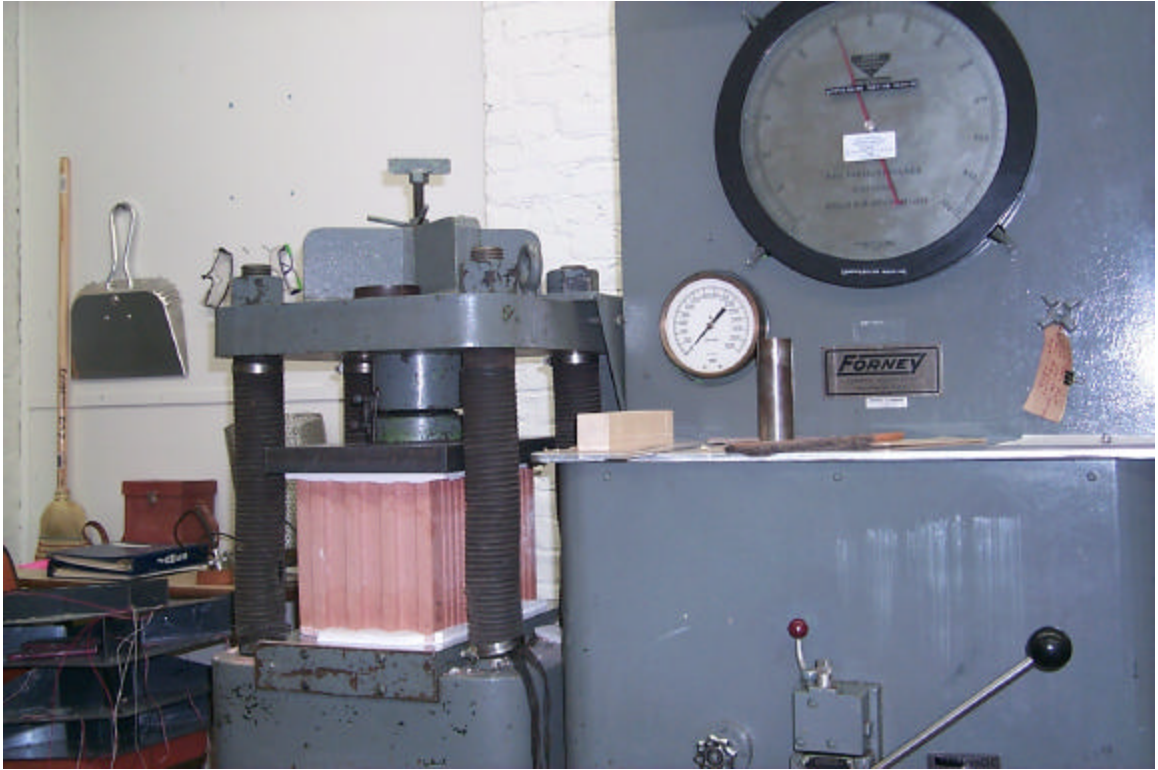


FIGURE B.1 - Unit Compression Testing Setup



FIGURE B.2 – Close-up of Large Block After Test Completed



FIGURE B.3 - Close-up of Large #3 After Test Completed

APPENDIX C
(Additional Bond Wrench Photos)



FIGURE C.1 - Front View of Bond Wrench

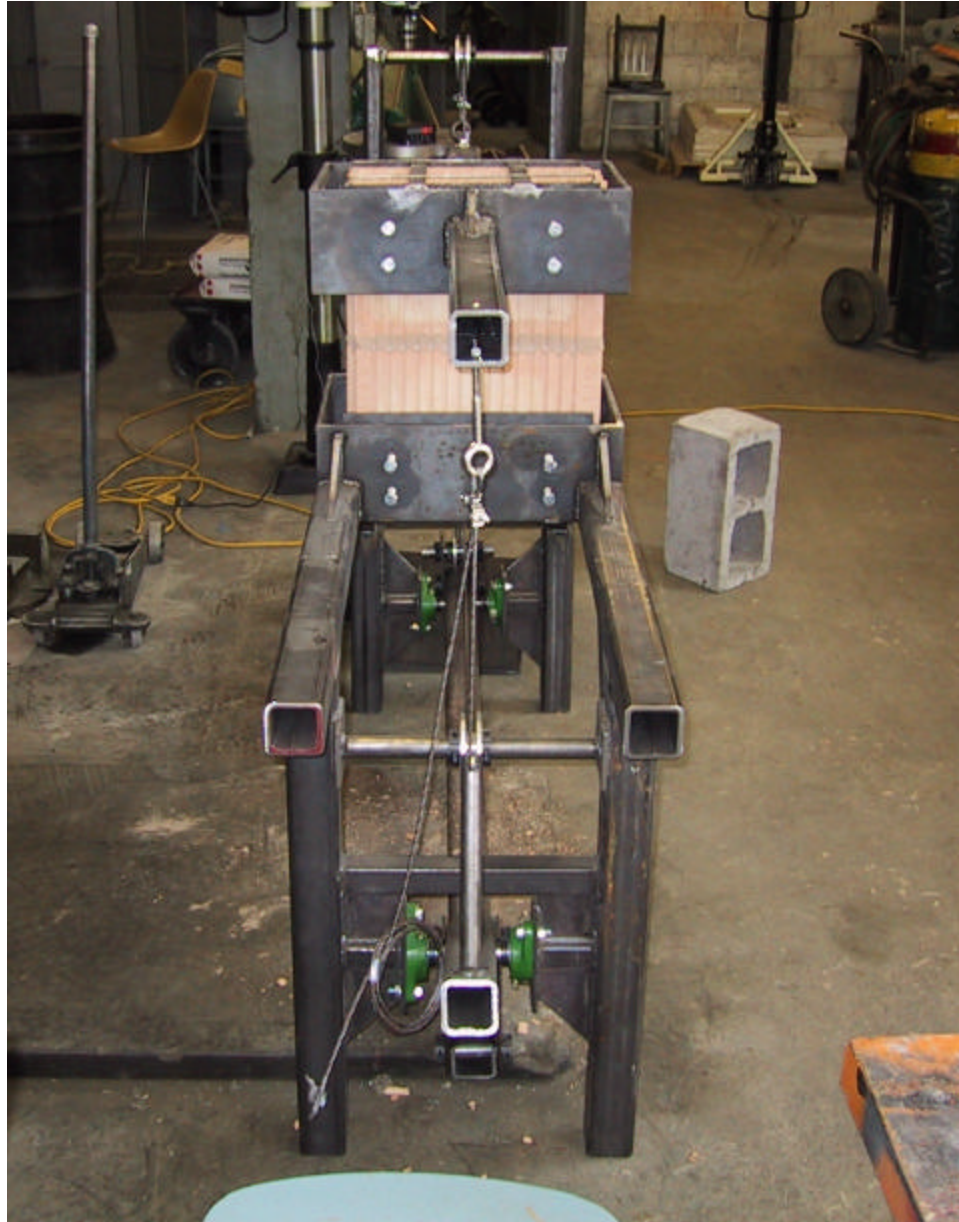


FIGURE C.2 – Downward Force Side of Bond Wrench



FIGURE C.3 – Upward Force Side of Bond Wrench

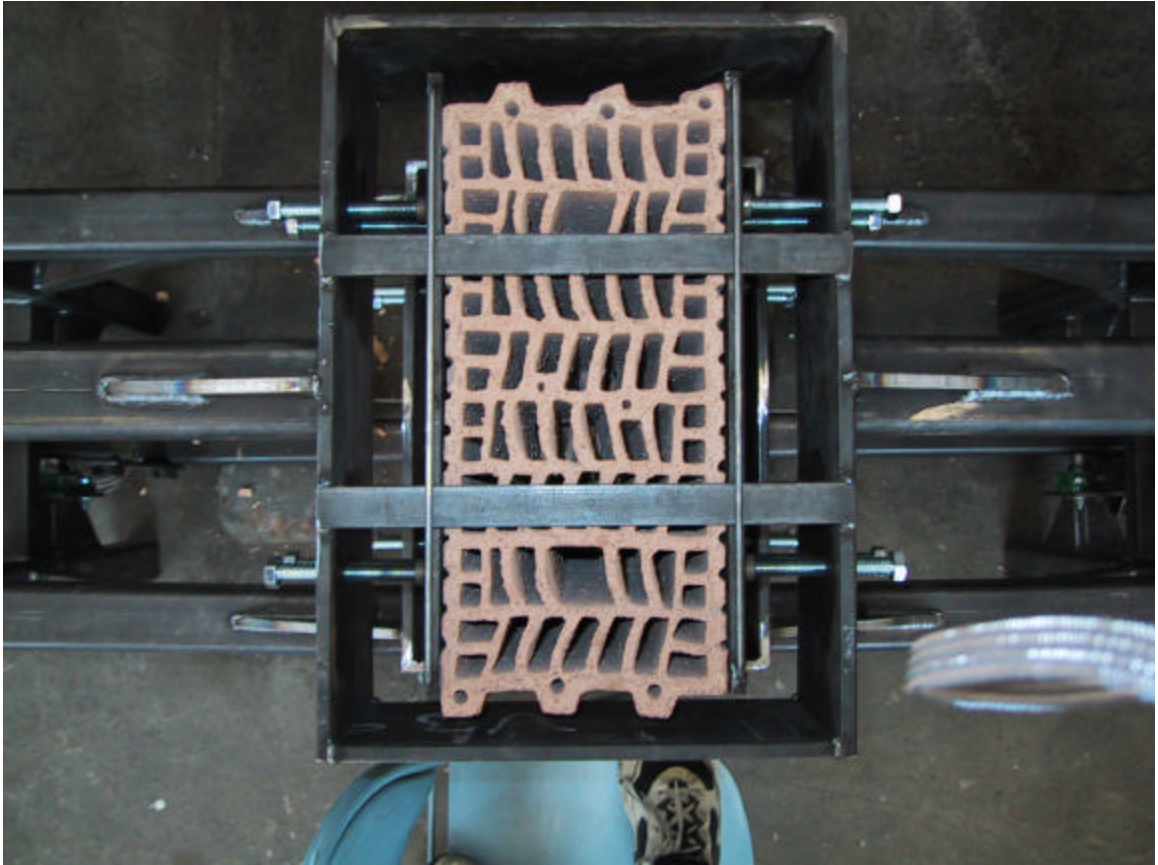


FIGURE C.4 – Top View of Prism Loaded into Bond Wrench

APPENDIX D
(Additional Prism Compression Photos)



FIGURE D.1 - Large Unit Prism Prior to Compression Testing



FIGURE D.2 - Spalling/Failure of End Face of Large Prism



FIGURE D.3 - Face Shell Failure of Large Prism



FIGURE D.4 - Significant Spalling of Both Face and End Shells of Large Prism



FIGURE D.5 - Spalling of Small Prism Face Shell



FIGURE D.6 - Cracking of Small Prism



FIGURE D.7 - Failure of End of Small Prism



FIGURE D.8 - Remains of Small Prism Specimen

APPENDIX E
(Sample Calculations)

UNREINFORCED POROTON MASONRY

Determining Allowable Wind Pressure

INPUT:

Wall Height 5.00ft
Allowable Stress Factor 1.00
Allowable Tension 21.00psi
Section Modulus 102.1in³/ft
Net Area 54.5in²/ft
Wall Weight 39.0psf

KEY ASSUMPTIONS:

- 1) Wall is simply supported at the top and at the base of the wall.
- 2) Wind load is uniform over the entire wall height.

CALCULATIONS:

$$\text{Axial Load (P)} = \frac{H_{\text{wall}} \times W_{\text{wall}}}{2}$$

$$P = 97.5 \text{ lb/ft}$$

$$F_t = -\frac{P}{A} + \frac{?L^2}{8S} \Rightarrow ? = \frac{8\left(\frac{P}{A} + F_t\right)S}{L^2}$$

Allowable Wind Pressure	62.0psf
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VITA

Michael Radcliffe was born in Charleston, West Virginia on January 1, 1975. He attended schools in the Kanawha County School System, where he graduated from Herbert Hoover High School in June 1993. He entered Marshall University in Huntington, West Virginia during August 1993, where after three years he transferred to West Virginia Institute of Technology in Montgomery, West Virginia. In December 1998, he was awarded, with Cum Laude honors, a Bachelor of Science in Civil Engineering. After working for a while, he entered the Master's program in Civil Engineering at the University of Tennessee, Knoxville. He received the Master's Degree in December 2001. He remains an active member of Tau Beta Pi, Chi Epsilon, and Phi Kappa Phi honor societies.

He is presently working toward a Doctorate of Philosophy in Civil Engineering at the University of Tennessee, Knoxville.