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DEVELOPMENT OF DRY STACKED CLAY MASONRY WALL SYSTEMS

W. Mark McGinley[1]

ABSTRACT

In the past few years there has been increased interest in dry stacked masonry wall systems since, potentially, dry stacked systems offer a significant cost savings over conventional masonry systems. An experimental investigation designed to evaluate the viability of producing an extruded clay masonry unit for use in dry stacked masonry wall systems was conducted.

The goal of this investigation was to determine the lateral uniform load carrying capacity of a dry stacked wall system fabricated with a 143 mm (6" nominal) hollow clay unit.

A total of twelve wall specimens were tested under out-of-plane loading using an ASTM E -72 Air Bag Test. Four different reinforcing configurations were evaluated and the wall systems were shown to perform well with respect to the loads predicted using working stress and ultimate strength analysis techniques.

INTRODUCTION

In the past few years there has been increased interest in dry stacked masonry wall systems. In particular, a number of concrete masonry units have been developed for dry stacked wall systems for use in buildings and retaining wall structures (Pardo 1992, Harris and Hamid 1993, Harris and Hamid 1992, Hines 1993, Dawe et al. 1989, Valsangkar et al. 1991). Potentially, dry stacked systems offer a significant cost savings over conventional masonry systems. As a result, a significant number of dry stacked concrete masonry wall systems are being marketed in the US. and Canada.

[1] Associate Professor of Architectural Engineering, North Carolina A & T State University, Greensboro, NC USA 27411, USA.

The following report describes an experimental investigation designed to evaluate the viability of producing an extruded clay masonry unit for use in dry stacked masonry wall systems. This is a continuation of a previous investigation which showed that dry stacked clay masonry units can be successfully used in a Geogrid stabilized retaining wall system (McGinley 1992). On the basis of the performance of the dry stack retaining wall unit, it appears that this, or a similar unit can be used for other applications such as fencing systems, noise barrier walls, and possibly load-bearing and nonload-bearing walls for buildings.

The goal of this investigation was to:

1. determine the lateral uniform load carrying capacity of a dry stacked wall system fabricated with a 143 mm (6", nominal) hollow clay unit; and

2. evaluate the dry stacked wall construction procedures.

EXPERIMENTAL PROGRAM

In co-operation with a local brick plant, a dry stack hollow clay masonry unit was developed and produced. As shown in Fig. 1, the 89 mm x 143 mm x 292 mm (3.5" x 5 5/8" x 11.5") unit was formed with a tongue and groove shape and hollow cores. The unit was designed to be stacked dry on its bed joint and interlock horizontally via the 19 mm (3/4") tongue and groove at the head joint. This 143 mm (6") unit was similar to the 203 mm (8") unit used for the previous retaining wall investigation (McGinley 1992), but was shorter on one side to facilitate laying this unit in curved wall systems.

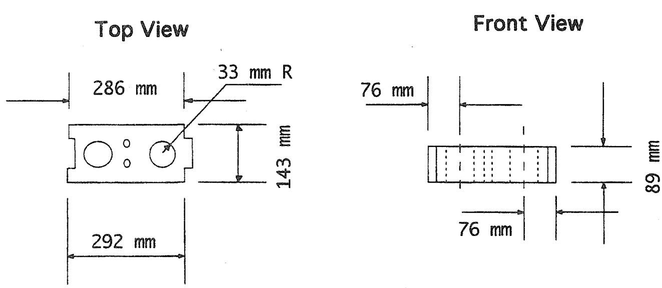
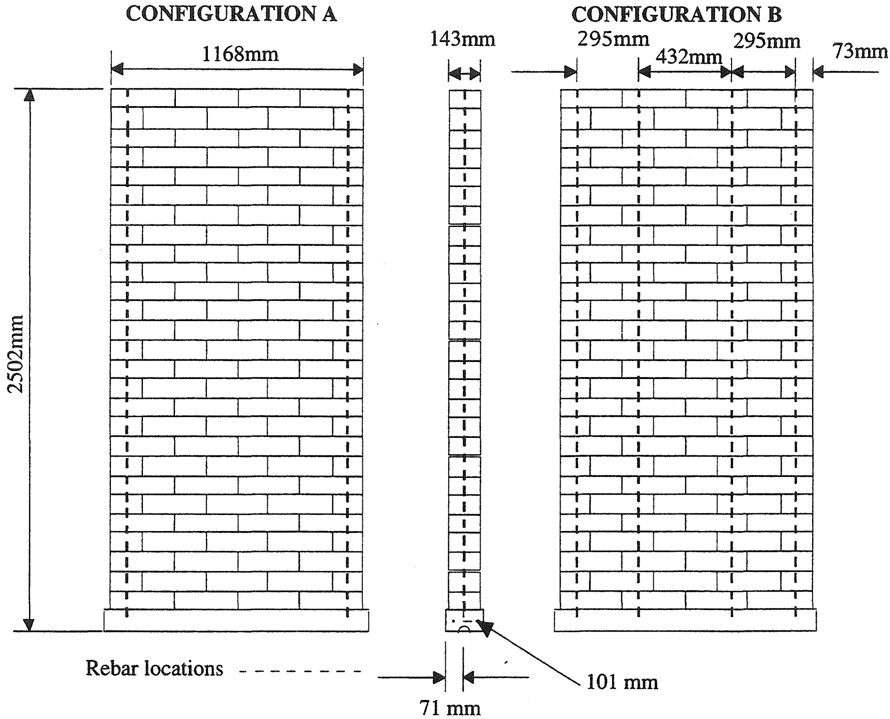


Fig. 1 Dry Stack Hollow Clay Masonry Unit

WALL SPECIMEN CONSTRUCTION

Once sufficient numbers of the clay units had been produced, these units were used to fabricate twelve, 1168 mm x 2502 mm x 143 mm (4' x 8' x 6", nominal dimensions) partially reinforced single wythe wall specimens. Two basic reinforcing configurations were used, one that placed a reinforcing bar in the outer cores, one each side of the specimen, and one which used four reinforcing bars spaced along the width of the specimen. The wall specimen configurations are shown in Fig. 2. Two sizes of 415 MPa (60 ksi) steel reinforcing bars were used, a 20 M (#3) bar and a 25 M (#4) bar. Core holes that did not contain reinforcing bars were left ungrouted. A total of four different wall configurations were constructed.



CONFIGURATION A - One Rebar in center of each of the exterior cores, either 20 M rebar or 25 M Rebar

CONFIGURATION B - One Rebar in center of each of the exterior cores, and two in the more central cores either 20 M rebar or 25 M Rebar

Fig. 2 Masonry Wall Specimen Configurations

Each wall was constructed using the following procedure.

1. The reinforced concrete footings were constructed and allowed to cure for a minimum of 28 days. These footings had sections left out to act as cleanouts for the grouted cores and to tie the vertical wall reinforcing to the horizontal reinforcing of the footing. See Fig. 3.

2. The first fourteen courses were laid up in running bond by stacking the units dry on their bed surfaces.

3. The wall was braced and the hooked reinforcing bars were inserted through the wall cores into the footing. This reinforcement was cut to extend above the fourteenth course by the lap splice length

required by the ACI 530/ASCE 5/TMS 402 - 92 code. The 20 M bars had a lap length of 457 mm (18") and the 25 M bars had a lap length of 610 mm (24").

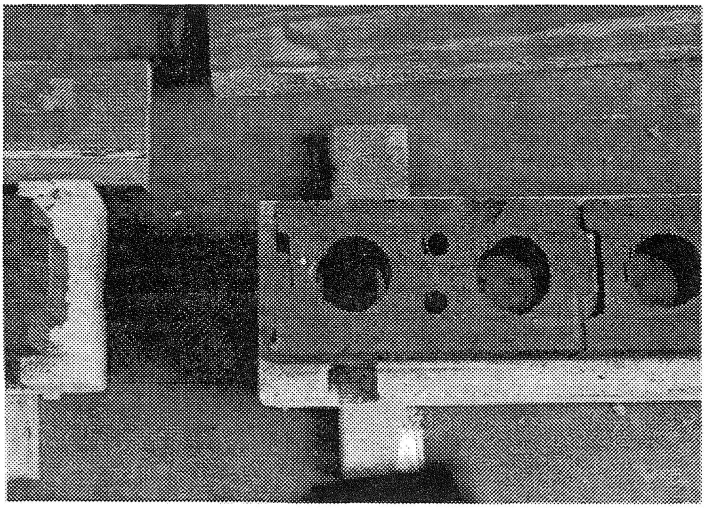


Fig. 3 Cleanouts in Footing for Two Bar Wall Specimens

4. The cores containing rebar were grouted using a 1 part cement and 3 parts sand. The grout was initially consolidated by agitating the bar and reconsolidated using a mechanical vibrator.

5. The remaining courses of the wall were laid up within one hour of the first grout pour.

6. The top section of the wall was braced and straight sections of reinforcing bar were inserted in the cores. These cores were then grouted. A mechanical vibrator was again used to consolidate the grout.

During the construction of the wall specimens, it was apparent the bracing would be required for construction of these of wall systems in the field. The relatively large variation in the surfaces of the bed joints caused the walls to be unstable at heights over about 1.5 m (5'). However, it may also be possible use higher lift grouting if sufficient bracing is used. The construction of the wall system was quick and was carried out by students who had no masonry construction experience. This low skill requirement for the majority of the wall construction should further enhance the economic viability of the wall system.

Three compression strength cylinders were fabricated for each grout mix. The final grout mix was also used to fabricate four rectangular grout specimens using the units as forms, as described in ASTM Standard C1019 - 84

Standard Method of Sampling and Testing Grout. All grout specimens were cured in a curing tank and tested for compressive strength after 28 days.

For each wall configuration, three specimens were fabricated to ensure a statistically valid evaluation of the wall strength. The wall specimens were cured in the lab environment a total of 28 days and then tested using an air bag system, as described in ASTM Standard E 72 -80 Standard Methods of Conducting Strength Tests of Panels for Building Construction. The pressure in the air bag was increased uniformly until the specimen failed. In addition, the loading was paused at approximately 2.39 kN/m^2 (50 psf), 4.79 kN/m^2 (100 psf) and 7.19 kN/m^2 (150 psf). Throughout the test, the air bag pressure and deflections at six locations over the height of the specimen were measured. Figure 4 shows the testing configuration and location of the deflection measurements.

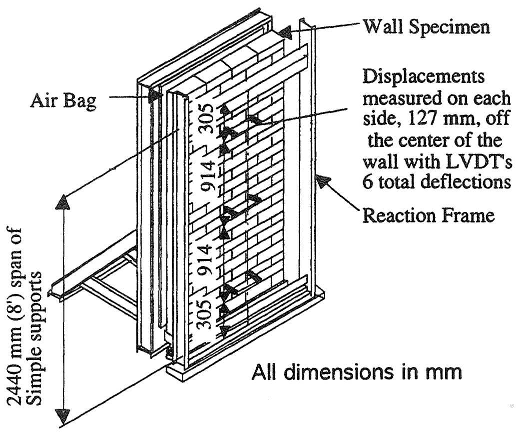


Fig. 4 Location of Deflection Measurements

TEST RESULTS AND DISCUSSION

Grout Compressive Strength Tests

The results of the compression tests on the grout specimens are summarized in the following section. The average batch compressive strengths ranged from 8.94 MPa (1261 psi) to 22.87 MPa (3317 psi), and the overall average compressive strength for the grout cylinders was 19.66 MPa (2851 psi) (COV 20.2 %). The average compressive strength of the square grout specimens was 26.25 MPa (3807 psi) (COV 7.6 %).

Wall Test Results

The maximum pressures resisted by each of the wall specimens are listed in Table 1. Table 1 also lists the average maximum pressure resisted by each of the four wall specimen configurations and the respective coefficients of variation.

Two distinct types of load-deflection behavior were observed during testing. With the exception of Specimen #5, wall specimens that were grouted only in their outer cores (Configuration A) exhibited the following behavior:

1. The walls showed what appeared to be linear elastic behavior up to the load at which the grouted cores cracked.

2. After cracking, the apparent stiffness of the wall system decreased significantly and linear load-deflection behavior continued up to the load at which apparent yielding of the rebar occurred.

3. After yielding of the steel bars, a further significant decrease in the stiffness of the wall occurred. Small increases in load produced large deflections. In addition, there were large horizontal rotations about the bed joints and large vertical rotations about the head joints near the center span of the wall specimen. (see Fig. 5)

Table 1 Wall Test Results

Wall	Reinforc.	Max. Wall Pressure (kN/m^2)	Max. Wall Pressure (psf)	Ave. Wall Pressure (kN/m^2)	COV (%)
1	25 M, Config A	9.20	192.1		
2	25M, Config A	8.62	180		
3	25M, Config A	8.38	175.1	8.73	4.80
4	20M, Config A	7.98	166.7		
5	20M, Config A	7.48	156.2		
6	20M, Config A	7.11	148.5	7.52	5.81
7	20M, Config B	14.16	295.7		
8	20M, Config B	16.38	342.1		
9	20M, Config B	15.45	322.7	15.33	7.28
10	25M, Config B	17.22	359.7		
11	25M, Config B	18.76	391.8		
12	25M, Config B	19.45	406.3	18.48	6.18

4. Load popping noises were heard when the wall load ranged between approximately 3.35 kN/m^2 and 4.79 kN/m^2 (70 and 100 psf). During subsequent loading, vertical cracks were observed in the masonry units near the outer edge of the wall specimens, at approximately mid-height.

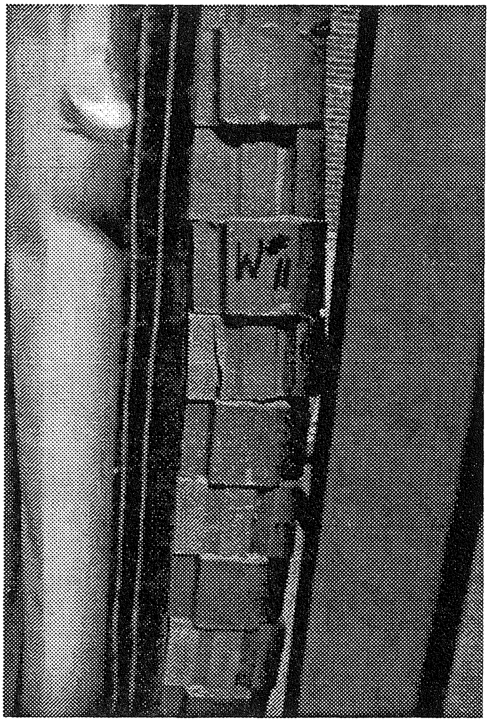


Fig. 5 Horizontal Rotation of Units About Bed Joints

5. With the exception of Wall Specimen # 5, all Configuration A wall specimens failed when units in the courses near mid-height slipped horizontally across the bed joints. This slipping was accompanied by large rotations about the head joints near the center of the wall (see Fig. 6). Wall Specimen #5 exhibited the same behavior as Configuration B specimens.

6. After the maximum wall load was reached, significant cracking of the compression face of the brick units near the center span was observed on all wall specimens (see Fig. 7). Because of safety concerns, loading was stopped after the load started to drop off, even though the wall specimens continued to resist load.

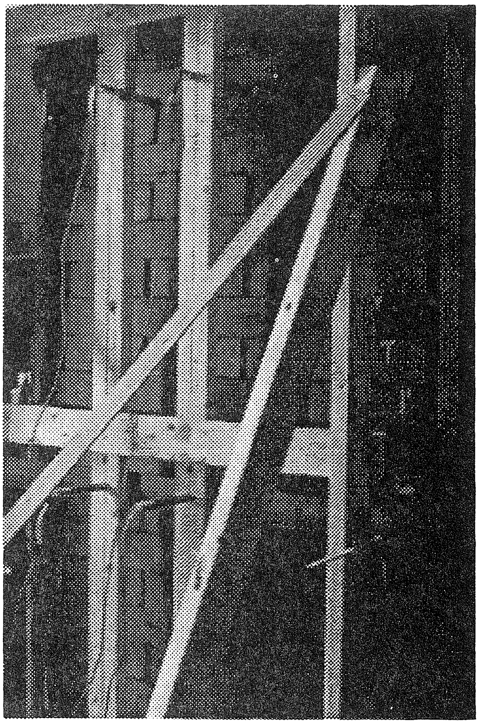


Fig. 6 Typical Failure of Configuration A Wall Specimens

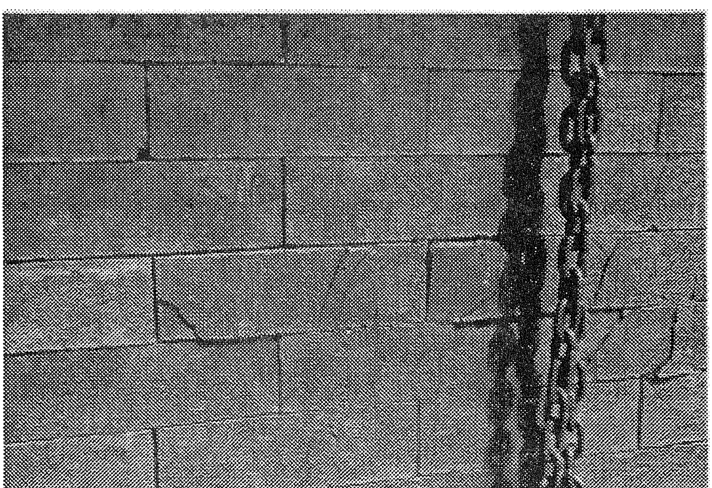


Fig. 7 Cracking of Compression Face of the Brick Units

The wall specimens constructed with Configuration B (4 of the cores reinforced and grouted) exhibited essentially the same load-deflection behavior as described above, except near failure. At failure, significant cracking of the compression face of the brick units occurred near mid-span. In addition, little head joint rotation was observed during the loading of these wall specimens. The lack of slip failure on Specimen # 5 indicates that the resistance to slip on the bed surfaces may be highly variable.

The typical load deflection curves for all four specimen configurations are shown in Fig. 8.

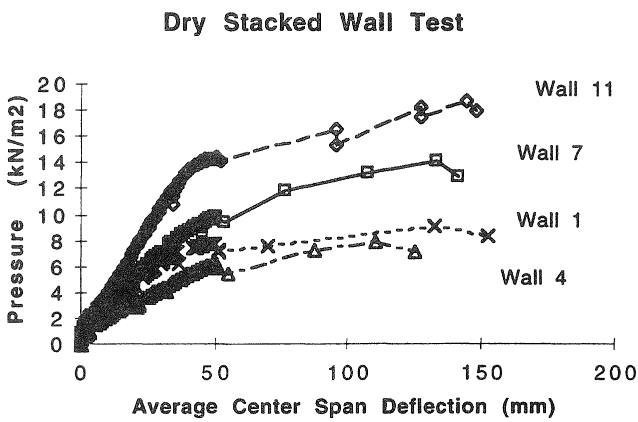


Fig. 8 Typical Load-Deflection Behavior of Dry Stacked Wall Specimens

ANALYSIS

To evaluate the performance of the wall specimens, two flexural strength analyses were performed. The first analysis predicted the wall specimen maximum load resistance using the ACI 530/ASCE 5/TMS 402 -92 Building Requirements for Masonry Structures and the second predicted the maximum load resistance using an ultimate strength approach.

The ACI 530/ASCE 5/TMS 402 -92 is a working stress design code and assumes linear-elastic material behavior. For the analysis, both the strength of the masonry assembly (f'm) and elastic modulus of the assembly (Em) must be determined. Typically, these values are obtained using data obtained from compression tests on prisms. However, no prism tests were conducted on the dry stacked system since the uneven surfaces of the dry bed joint would make determination of these values problematic. The wall tests did indicate that the dry stacked clay masonry walls appeared to be more flexible than conventional mortared walls. It was assumed for the analysis that the f'm of the masonry was equal to 20.68 MPa (3000 psi) and the Elastic modulus, E was assumed to be 750 x f'm. These values compare well with the grout tests and strength of the brick units (>50 MPa). To accurately determine these material properties, further investigation of the compression behavior of the dry stacked assembly is required.

The analysis of the wall system also requires the effective compression width of the masonry to be determined. Again, the uneven bearing of the dry bed surfaces made determination of this parameter difficult. However, since no mortar was present at the head joints, and therefore no significant shear stresses were transferred across these interfaces, it was assumed that the effective compression width of the masonry was limited to the length of a masonry unit, 292 mm (11.5").

Using the standard working stress formulas for flexural moment capacities shown below, the moment capacities for a given reinforced core were calculated and are summarized in Table 2. For the calculations, the allowable stress for 415 MPa (60 ksi) steel was assumed to be 165.5 MPa (24.0 ksi) and the modular ratio, n, was assumed to be 12.9.

$$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n \qquad \rho = \frac{A_s}{bd} \qquad n = \frac{E_s}{E_m} \qquad [1]$$

Ms = Moment capacity with steel stress governing

$$Ms = A_s\, j\, d\, f_s \qquad j = 1 - k/3 \qquad [2]$$

Mm = Moment capacity with masonry stress governing

$$Mm = 0.5\, f_m\, j\, k\, b\, d^2 \qquad [3]$$

It should be noted that the values selected for f'm, Em and the effective compression width of the masonry forced the steel moment capacity of the wall system to govern in all cases.

The predicted wall capacity for each of the four wall configurations was obtained by multiplying the corresponding bar capacity by the number of bars in the wall, and converting this capacity into an equivalent wall load (Wall load = Moment capacity x 8 /(wall width x (wall span)^2). This calculation assumes simple supports and a uniform wall load.

The ratios of average measured maximum wall load to predicted (design) wall load for each of the four wall configurations are also listed in Table 2. These ratios are essentially the factor of safety for the wall system and range for 2.79 to 4.34. While these values are relatively high for this ductile mode of failure, they are well within the 3 to 5 range of safety factors normally associated with working stress design of masonry systems. It appears that the ACI 530/ASCE 5/TMS 402 code can be used to provide conservative designs for the dry stacked clay masonry wall systems, at least within the bounds of the wall configurations evaluated during this testing program.

Table 2 Working Stress Design Values

b (mm)	d (mm)	As (mm2)	r	k	j	Fb (MPa)	Ms Bar (N.m)	Mm Bar (N.m)	Wall Load (Kn/m2)	Ratio M/Pred
(2) 20M Rebar										
292	71.4	71	.003404	0.255	0.915	6.90	767	1199	1.77	4.26
(2) 25M Rebar										
292	71.4	129	.006189	0.327	0.891	6.90	1357	1497	3.13	2.79
(4) 20M Rebar										
292	71.4	71	.003404	0.255	0.915	6.90	767	1199	3.53	4.34
(4) 25M Rebar										
292	71.4	129	.006189	0.327	0.891	6.90	1357	1497	6.25	2.96

If lower values for f'm, Em, and the effective compression width of the masonry were assumed, these ratios would be even higher. The assumed values of f'm, Em. and effective width appear to predict the observed wall performance reasonably well.

It appears that if this wall system was to be used for a free standing wall system that was cantilevered off a suitably designed footing system and subjected to 20 psf wind load, the ACI 530 design code would allow the four wall configurations to safely extend to heights that range from 1.58 m to 3.11 m (5.4 to 10.2 ft).

If an ultimate strength design approach is used to predict the wall strength, it appears that a somewhat more consistent prediction of the actual wall strength can be made. Table 3 shows a summary of wall strength using the typical reinforced concrete design equations and a resistance factor, ϕ, of 1.0. For the calculations, f'c was assumed to equal f'm, β was taken as 0.85, and the effective width and depth were assumed to be the same as the values used for the working stress calculations.

Note that the values of capacities in the table must be modified by an appropriate load and resistance factor for design purposes. However, better and a more consistent prediction of the wall strength may be possible using an ultimate strength approach for these wall systems.

Table 3 Ultimate Strength Design Values

b (mm)	d (mm)	As (mm2)	fy (MPa)	f'c (psi)	a (mm)	Mn (N.m)	Wall Load (Kn/m2)	Ratio M/Pred
	2 bars							
292	71.4	71	414	20.68	5.71	2012	4.63	1.62
292	71.4	129	414	20.68	10.39	3532	8.13	1.07
	4 bars							
292	71.4	129	414	20.68	5.71	2012	9.26	1.65
292	71.4	129	414	20.68	10.39	3532	16.27	1.14

In summary, it appears that dry stacked clay masonry wall systems can be designed to form a sufficiently strong wall system to resist out-of-plane wind and seismic loads. Further investigation of the effects of pilasters in the wall system and construction detailing of capping/bond beams will be necessary for use of this system for heights exceeding 1.5 m (10 ft).

In light of the test results, the dry stacked clay masonry wall system evaluated in this investigation appears to have great potential as a wall system for building construction. However, if it is to be used as an exterior wall system in buildings, further investigation of the water permeance and thermal resistance of this wall system must be performed.

SUMMARY AND CONCLUSIONS

The viability of producing an extruded clay masonry unit for use in dry stacked masonry wall systems was evaluated through an experimental investigation. The goals of this investigations were to determine the lateral uniform load carrying capacity of a dry stacked wall system fabricated with a 143 mm (6", nominal) hollow clay unit and evaluate wall construction procedures.

From the results of the tests, it appears that the dry stacked clay masonry wall system resists a simulated out-of-plane wind loading quite well. The limited results of this testing program also indicate that the design procedures in ACI 530/ASCE 5/TMS 402 - 92 can be used to provide conservative designs for this type of wall system, at least within the bounds of the wall system configurations tested.

It also appears that ultimate strength design procedures may give more consistent and accurate prediction of the wall system flexural strength than the working stress procedures of in ACI 530/ASCE 5/TMS 402.

The dry stacked wall system appears to be quite easy and quick to construct, even using workers with little or no masonry construction skills. Due to the instability of these wall systems before grouting, however, it is likely that they will need to be braced in the field until the grouting is completed and the wall system has developed sufficient strength.

ACKNOWLEDGMENTS

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