Flexural and In-Plane Shear Testing on Flexlock Concrete Masonry Panels

for

Cercorp Initiatives Incorporated

Conducted by:

NATIONAL CONCRETE MASONRY ASSOCIATION

RESEARCH AND DEVELOPMENT LABORATORY

Project No. 01-391

NATIONAL CONCRETE MASONRY ASSOCIATION

The National Concrete Masonry Association (NCMA) is a non-profit organization whose mission is to support and advance the common interests of its members in the manufacture, marketing, research, and application of concrete masonry products. The Association is an industry leader in providing technical assistance and education, marketing, research and development, and product and system innovation to its members and to the industry.

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The Research and Development Laboratory is devoted to the scientific research and testing of concrete masonry products and systems. The Laboratory is staffed by professional engineers and technicians with many years of experience in the concrete masonry industry. The Laboratory is equipped to perform nearly any physical research or testing of concrete masonry units and assemblages. The Laboratory performs research and development work for both the Association and individual companies.

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1.0—INTRODUCTION

This report describes the results of flexural strength and in-plane shear strength testing of Flexlock concrete masonry panels by the National Concrete Masonry Association's Research and Development Laboratory. The testing was performed for, and funded by, Cercorp Initiatives Inc.

The primary objective of this research was to evaluate the flexural capacity and in-plane shear capacity of Flexlock concrete masonry panels that are dry-stacked and then post-tensioned. The CMU faceshells are calibrated so that a smooth bearing surface is available. The wall system was post-tensioned using Dur-O-Wal post-tensioning tendons and hardware. Six wall panels were constructed and tested in accordance with ASTM E 72, *Standard Test Method of Conducting Strength Tests of Panels for Building Construction*. Also, six wall panels were constructed and tested in accordance with the procedures for in-plane shear testing described in this report.

Additional tests were performed on the concrete masonry units. The results of these tests and other material descriptions are included in Appendix A.

2.0—MATERIALS

2.1—Concrete Masonry Units

All of the concrete masonry units used in the research program were hollow 8 × 8 × 16-inch Flexlock concrete masonry units (Figure 1). All of these units were manufactured at the same time to reduce any possible variations due to batching, mixing, or molding of the Flexlock units. The units were delivered to the laboratory in ready-to-build condition.

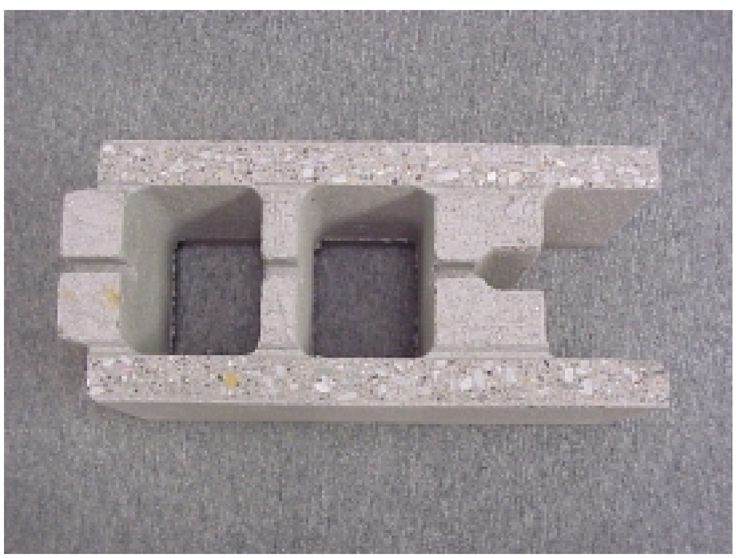


Figure 1—Flexlock CMU

The units were tested for compression strength and absorption in accordance with ASTM C 140, *Standard Methods of Sampling and Testing Concrete Masonry Units*. Results of the unit tests are summarized in Table 1. The detailed results are shown in Appendix A.

Table 1—Physical Properties of Concrete Masonry Units

Physical property	Test no. 1	Test no. 2	Test no. 3
Width, in.	7.59	7.60	7.59
Height, in.	7.99	7.99	7.99
Length, in.	15.95	15.91	15.94
Minimum face shell thickness, in.	1.33	1.32	1.32
Density, pcf	99.0	97.0	97.4
Net compressive strength of unit, psi	5810	6170	4440

Notes: 1) Reported values are based on the properties of saw-cut absorption, density, and compression specimens
2) Reported values for physical dimensions are based on full sized units

The CMU has three different web thicknesses and the outer most web protrudes out from the unit. At the client's request, the unit was tested to determine the shear resistance of this web as it was punched through the CMU. The CMU was placed vertically into the compression machine and loaded to failure. The shear area was found to be 6.1 square inches. Table 2 presents the results of this testing.

Table 2—Punch-Through Web Test

Test No.	Total load (lb)	Shear resistance (psi)
Test no. 1	4680	767
Test no. 2	4120	675
Test no. 3	4860	797

2.2—Concrete Masonry Prisms

Concrete masonry prisms were constructed to determine the dry stack compressive strength. Two units were gypsum capped, according to ASTM C 140, and then placed one atop another to form a dry stack prism. This prism was then placed into the compression machine and loaded to failure. The test procedure followed ASTM C 1314 – *Test Method for Compressive Strength of Masonry Prisms*. Figure 2 shows the dry stack prism.

Figure 2—Dry stack prism

The average gross area compressive strength was tested to be 940 psi. Only the faceshells carry load, so the net area compressive strength can be determined using the average faceshell thickness determined from the ASTM C 140 testing. The average net area was determined to be 42.2 in^2 and the average masonry prism strength was 2,655 psi. Figure 3 shows the typical failure mode of the prism and Table 3 lists the individual prism strength tests. Figure 3 shows a uniform distribution of load across the face shell. It was noted that at approximately 30,000 lbs. a crack appeared through the webs of all the units tested and Figure 4 shows a typical crack in the web.

Figure 3—Prism failure

Table 3—Prism Test Results

Test no.	Maximum Load (lbs)	Net Area Compressive Strength (psi) (1)
Test no. 1	108,620	2574
Test no. 2	131,460	3115
Test no. 3	96,080	2277

1) based on net area = 42.2 in^2

Figure 4—Web crack at 30,000 lbs, test no. 2

3.0—WALL PANEL CONSTRUCTION AND TESTING PROCEDURES

3.1.1—Panel Construction – Flexural Strength Testing

All panels were constructed using good construction techniques in accordance with ACI 530.1/ASCE 6/TMS 602 *Specification for Masonry Structures*. All panels were constructed by a journeyman mason with a minimum of 15 years experience in concrete masonry construction.

The overall nominal dimensions of the finished panels were 104 inches high, 56 inches wide and 8 inches thick. The panels were constructed by dry stacking CMU using a running bond pattern on 10-inch bottom channel sections. The bottom and top courses were grouted to provide load support for when the walls were loaded in flexure. Once the specified height was attained, a top 10-inch channel section was placed onto the top units.

Two 7/16 inch steel rod tendons were placed in the wall assembly spaced at 32 inches center to center. According to the manufacture's specification the ultimate and yield stresses of the tendons are: f_u = 122 ksi and f_y = 100 ksi. The top and bottom channels had a hole to receive the tendon and the channels were used as a bearing surface for the tendon washers. A load indicator washer was used on the top channel and the tendons were stressed until the nibs on the washers were flat, indicating a tendon tensile load of between 12 to 14 kips. But, during the installation and stressing of the tendons in all the walls, the following was observed: 1) tendon rotated as the wrench was turning until a point where nut moved relative to the tendon, and 2) the load indicator washer was turning with the nut such that the indicator washer nibs were grinding down. Consequently, the turning of the tendon nut was halted, when the tendon was turning within the wall to a point where a tendon failure would occur. At this point a torque wrench was used and the tendon torque was measured to be 125 ft-lbs.

Additionally, the tendon elongation was measured as ¼ inch and met the manufacturer specification of ¼ inch elongation for an 8 feet wall height. Five of the six wall panels were assembled with the tendons spaced 32 inches apart. The sixth wall panel had a third tendon and the center-to-center tendon spacing in this case was 16 inches. Figure 5 shows the bottom channel, bottom course, and the two steel rod tendons placed in the CMU.

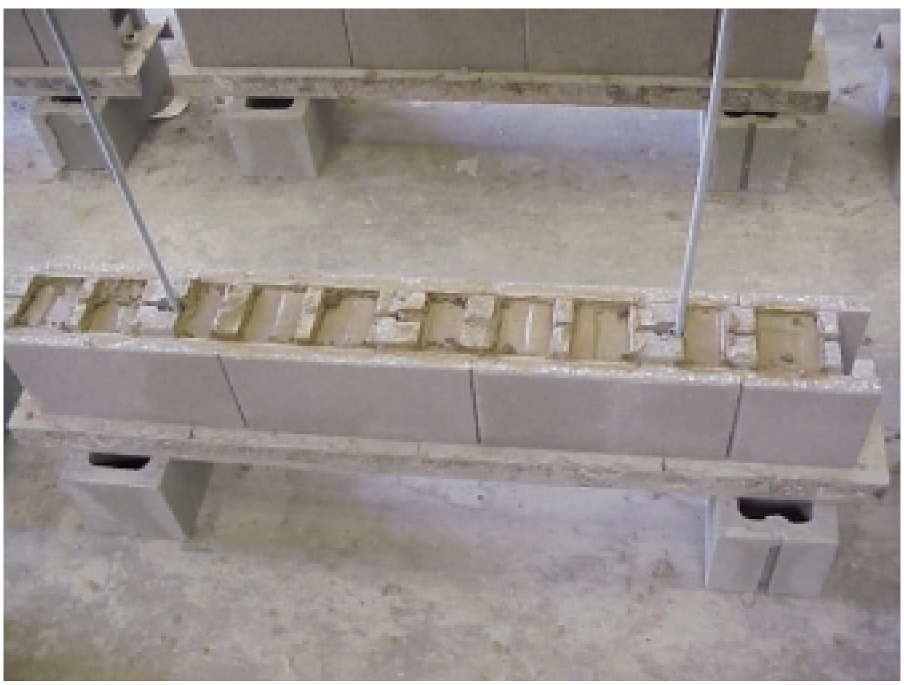


Figure 5—Wall panel construction – first course grouted

Subsequent testing was conducted to measure the tension in the tendons by using load cells. In this test two walls – 104 inches high and 56 inches wide – each with two tendons were constructed exactly as the flexure walls described above. Each tendon had a load cell placed atop the top channel and then under the tendon bearing plate as shown in Figure 6. For each tendon, elongation was measured, tensile load was monitored using the load cell, and torque was measured. The tendons were tighten as previously done and the same observations regarding the twisting of the tendon and the nut grinding the indictor washer were noted. The tendon was tightened to a point where tendon failure would occur and the torque was measured to be 125 ft-lbs. in both walls. The load cell measurements were recorded and the average tensile load was measured as approximately 5000 lbs. The individual tendon loads were: 5296 lbs., 5160 lbs., 5333 lbs., 4029 lbs.

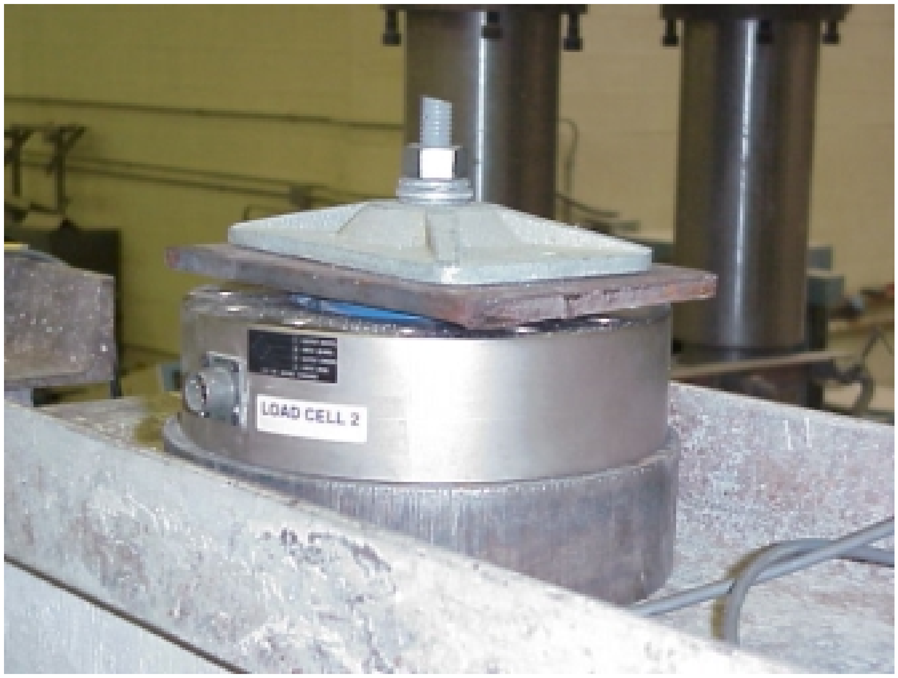


Figure 6—Load cell setup for tensile load measurement

Tendons for the testing were used in the as-delivered condition. Each full size tendon consisted of a 7-0' tendon coupled with an all-thread rod to make the full height of each test panel. The 7-0' tendon, coupler and all-thread are all standard components of the as-delivered post tensioning system. For all the test panels, the 7-0' tendon was attached to the bottom channel, and then the coupler was placed to attach the all-thread rod. In this configuration, the coupler was located approximately 7 feet above the bottom channel.

3.1.2—Test Procedures – Flexural Strength Testing

All panels were tested in accordance with ASTM E 72, *Standard Test Method of Conducting Strength Tests of Panels for Building Construction*. The test was conducted with the wall in a horizontal position using third point loading as shown in Figure 7. The load was applied using a hydraulic ram and load was measured using a precision load cell with 50,000 lb capacity and deflection was measured using two linear displacement sensors (LVDT). Additionally, joint expansion was measured at two locations using dashpots.

Load and deflection data were recorded via a data acquisition system at a sampling rate of 1 Hz. Two LVDTs were used to measure midpoint deflection on the left and right side of the panel.

During the testing, load and deflection were measured and recorded over the test duration until failure.

Figure 7—Test Setup for Third-Point Loading

3.2.1—Panel Construction – In-plane Shear Strength Testing

Six additional wall panels were constructed using the procedure described above. The walls were constructed on steel channels where the tendons were attached to the bottom and top channels. For the shear tests, the bottom and top courses of the wall panels were not grouted since these courses were not loading points. Tendon couplers were also used in these panels located approximately 7 feet above the bottom course of the panel. Additionally, the wall was white washed to facilitate the observation of any developed crack pattern. The wall panel was then placed on a structural steel wide flange beam and the bottom channel of the wall panel was secured to the beam using high strength bolts. Figure 8 shows a schematic of a wall panel set into the shear test frame.

3.2.2—Test Procedures – In-plane Shear Strength Testing

The in-plane test involved the application of horizontal load to the top of the panel in its own plane while the base was held rigid. The test frame for this test is shown in Figure 8 and the test frame consisted of three W12 $\times$ 96. The Research and Development Laboratory is equipped with a strong floor that has connection plates spaced at 8 feet on center. Consequently, the horizontal floor beam and the diagonal support were attached to connection plates. The vertical load column was connected to the horizontal floor beam.

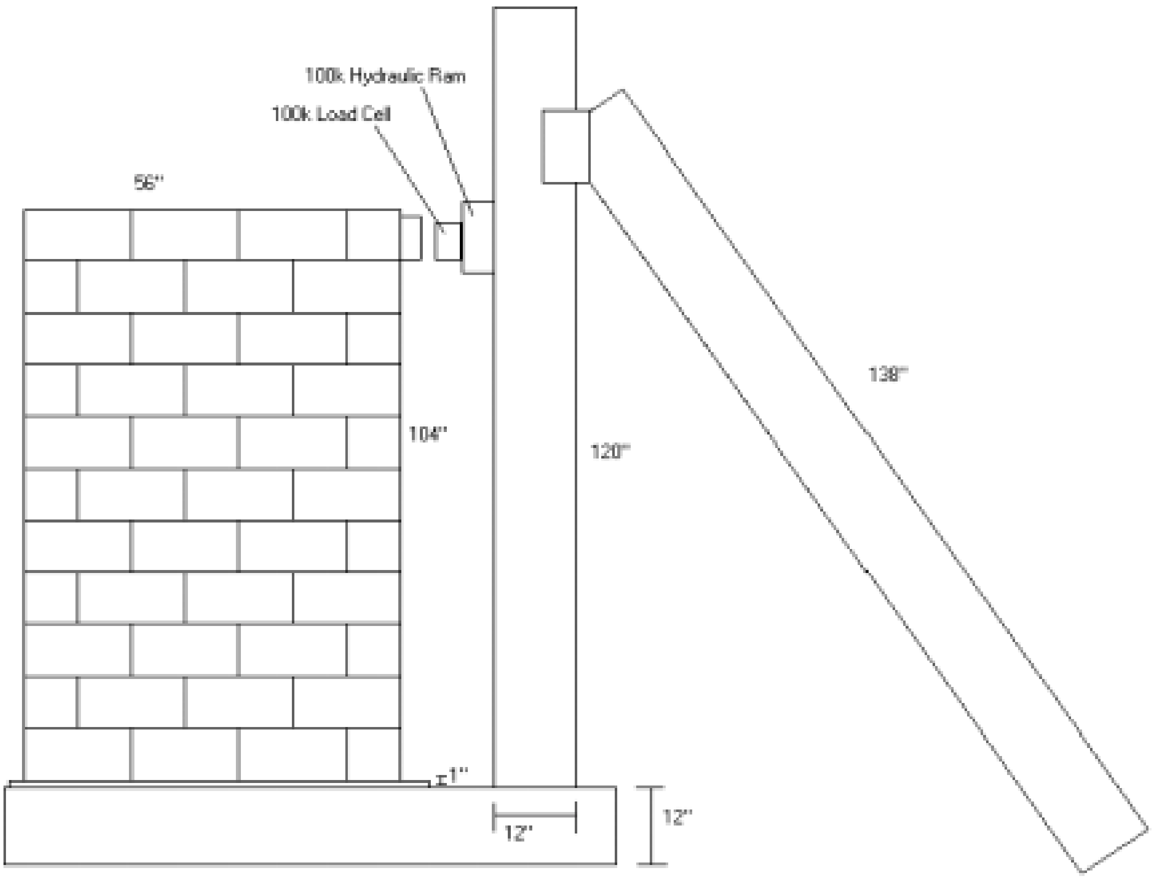


Figure 8—In-plane Shear Testing Frame

Lateral load was applied to the wall specimen by means of a hydraulic ram of 100-kip capacity. The magnitude of the applied load was measured with a 50-kip load cell that was placed between the hydraulic ram and the wall test panel. Dashpots were placed in two locations to measure the displacement and drift of the wall specimens. Figure 9 shows the placement of the dashpots to measure the displacement (dashpot #1) and drift (dashpot #2).

Both lateral load and dashpot measurements were made using the data acquisition previously described. Lateral load was applied in 500 lb. increments and held to observe a cracking in the wall panels. The wall panels were loaded to failure. The testing was also photographed and video recorded.

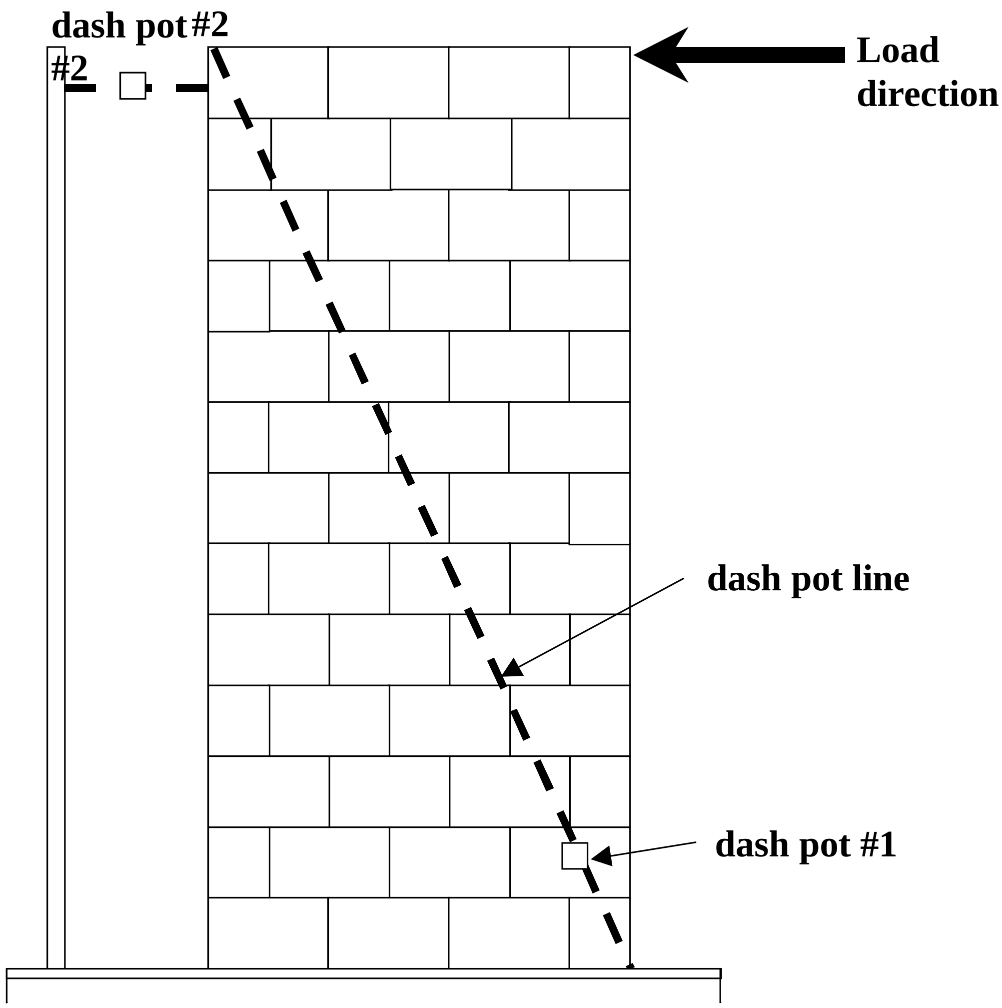


Figure 9—Location of Dashpot #1 and Dashpot #2

4.0—TEST RESULTS AND OBSERVATIONS

4.1—Flexure Strength Testing

Six Flexlock wall panels were tested in flexure and load and deflection data were recorded and analyzed. The six wall panels were divided into three groups:

- Walls # 1, 2, & 3 were tested individually until failure – each wall had two tendons
- Walls # 4 and 5 were cycle tested applying load, then released for 4 cycles – each wall had two tendons
- Wall # 6 was tested to failure – wall had three tendons

The data was collected from the beginning of the testing until wall failure was observed. The load was applied in 500 lb. increments held for approximately one minute and failure was determined when the wall panel could not sustain additional load. A smoothing routine – moving average with a unit interval equal to 100 – was used on the load-deflection data to eliminate any noise in the data caused by the laboratory environment. Figure 10 shows a comparison between the raw load-deflection data and the moving average of the data. It was determined that the signal interference in the data shown in the figure was on the order of 100 Hz and could be filtered out during the data acquisition. This was done for test panels # 4, 5, & 6, although the moving average filter was applied to all the data to get smooth curves.

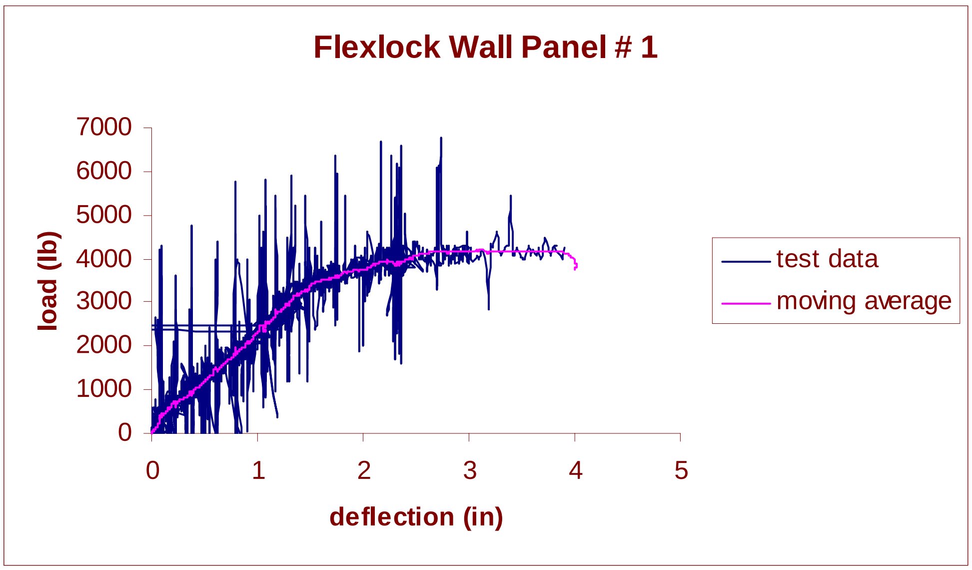


Figure 10—Comparison Between Test Data and Moving Average Filter

The load-deflection curves for wall panels 1, 2, & 3 are shown in Figure 11. This shows linear behavior for the three walls up to approximately 3500 lbs, with 1.5 inches deflection. After this point the walls were observed to exhibit ductile behavior with very little or no breakage of the concrete masonry units near the point of maximum flexural compression. Figure 12 shows the extent of the damage to the CMU for wall # 2. No CMU was damaged for walls 1 & 3.

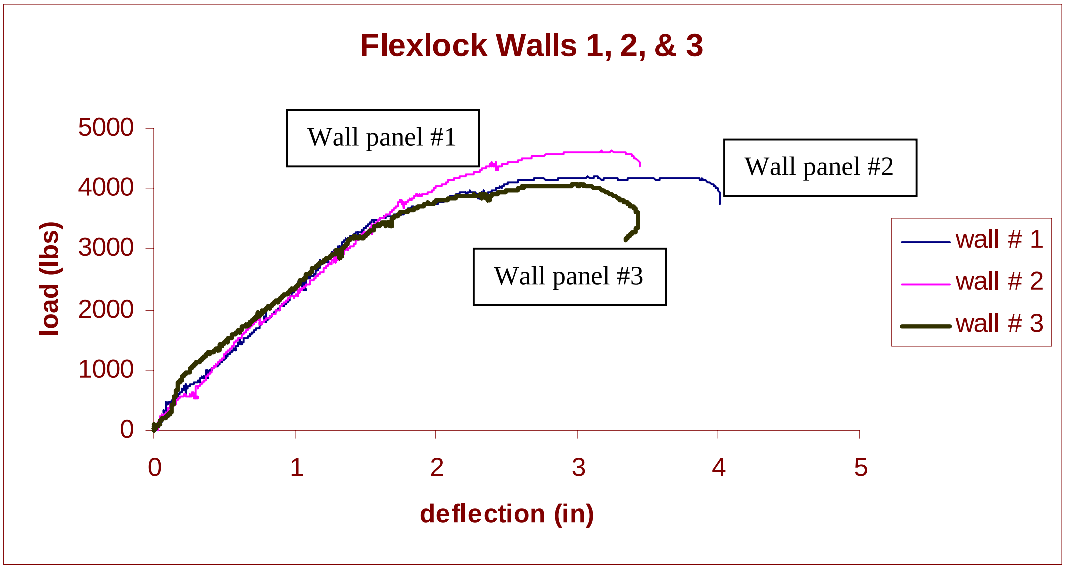


Figure 11—Comparison Between Wall Panels #1, #2, and #3

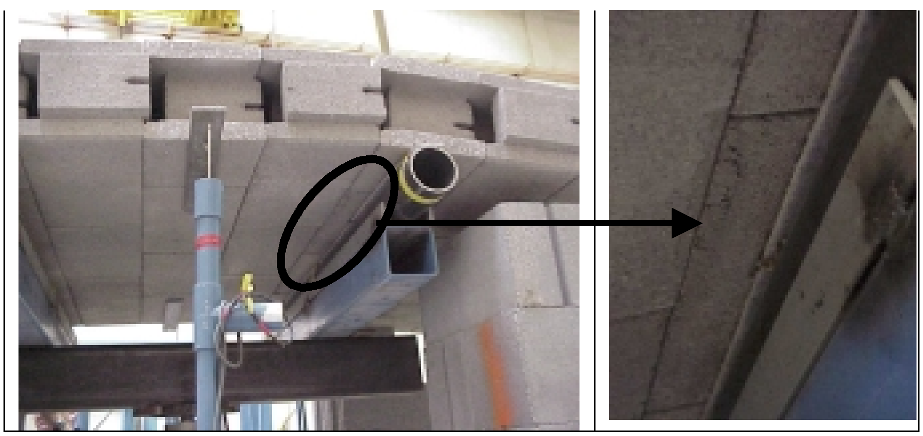


Figure 12—Damage to CMU During Flexure Testing

The next two wall panels – panels 4 & 5 - were cycle tested to see if this loading affected the performance of the wall panels. Wall panel # 4 was loaded at 500 lb increments to 2500 lbs and then relaxed and reloaded four times. On the fifth cycle, the wall panel was loaded to failure. Figure 13 shows the results of this cycle test, which shows no degradation of wall capacity over the five cycles. The results for wall # 4 - cycle 5 - were compared to the first three walls and are shown in Figure 14. Finally, Figure 15 shows the cycle testing for wall # 5, showing the 4 cycles of loading to 2500 lb.

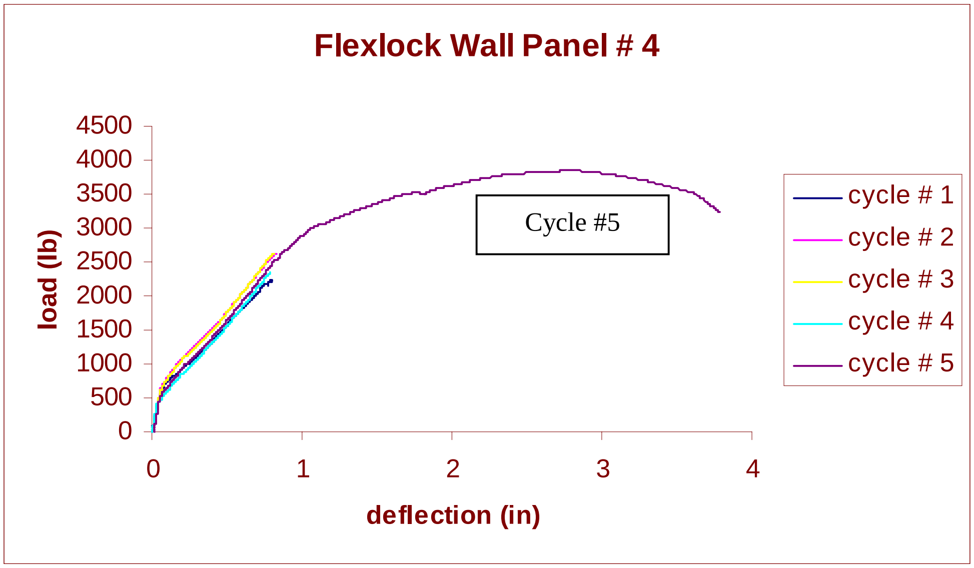


Figure 13—Wall Panel #4 Showing 5 Load Cycles

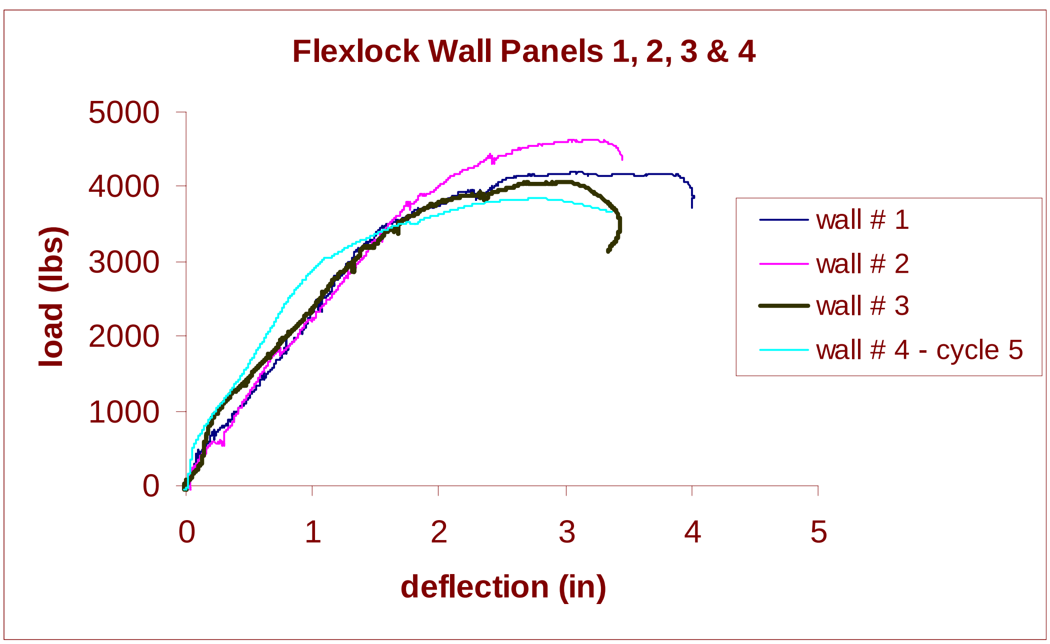


Figure 14—Comparison of Wall Panels #1, #2, #3, and #4 – Cycle 5

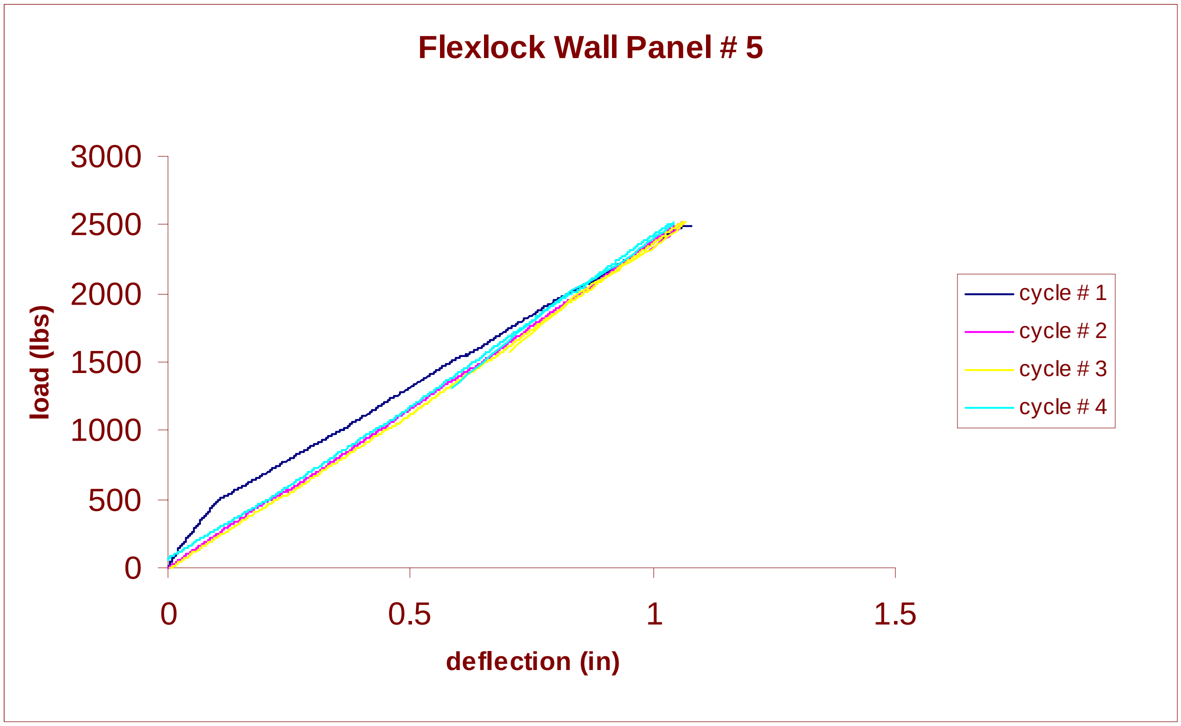


Figure 15—Comparison of Wall Panel #5 Cycle

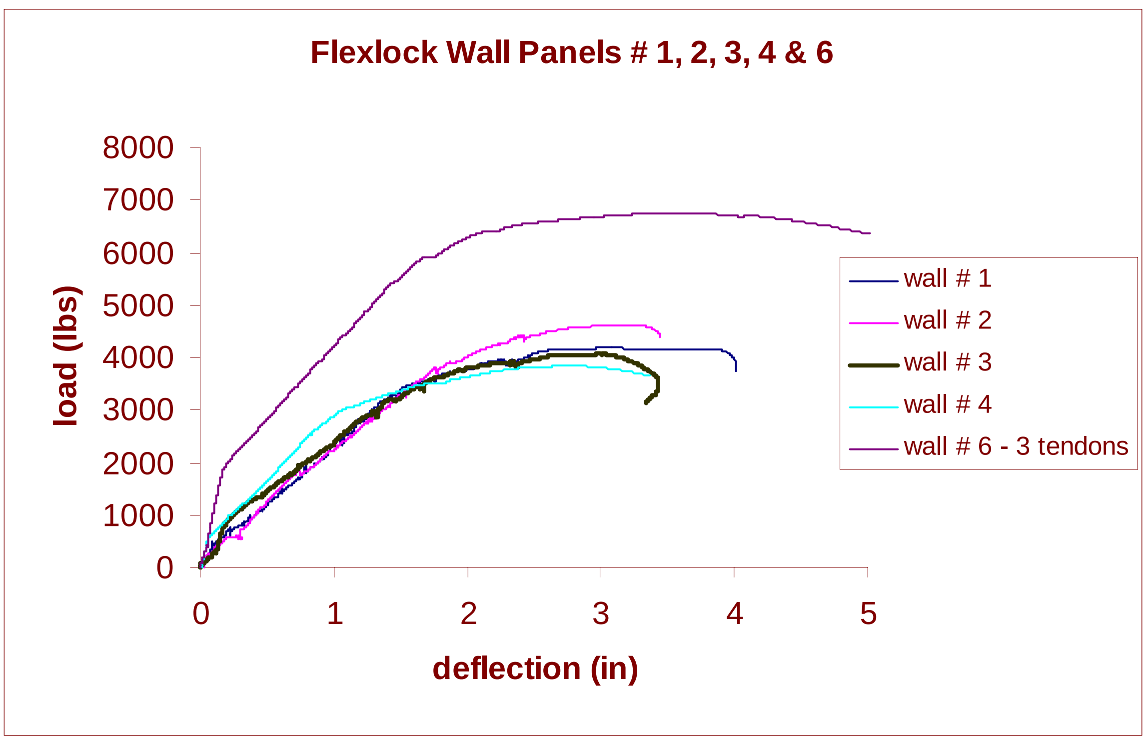


Figure 16—Flexlock Wall Panel #6 Compared to Wall Panels 1, 2, 3, and 4

Wall panel # 6 was assembled with three tendons spaced 16 inches apart and tested to failure by loading the wall panel at 500 lb increments. Figure 16 shows the load-deflection curve for this test along with the tests for the first four panels.

4.2—In-plane Shear Tests

Six Flexlock walls were tested by applying a lateral load as described above. Table 4 lists the wall panel configurations with the tendon spacing that was tested. All of the shear wall panels were 8 feet by 8 inches high. These walls were tested by loading in 500-lb increments and observations were made with regard to crack pattern, drift and displacement measurement. The results of the tests are summarized in Table 5.

Table 4—Wall Panel Configuration for In-plane Tests

Panel Designation	Wall Length (in)	Aspect Ratio	Tendon Spacing
Shear wall #1	104	1.0	2@ 80 inches c/c
Shear wall #2	104	1.0	2@ 80 inches c/c 1@ 44 inches from right
Shear wall #3	72	1.4	2@ 48 inches c/c
Shear wall #4	72	1.4	2@ 48 inches c/c 1@ 28 inches from right
Shear wall #5	56	1.9	2@ 32 inches c/c
Shear wall #6	56	1.9	2@ 32 inches c/c 1@ 28 inches from right

Table 5—Test Results of In-plane Walls

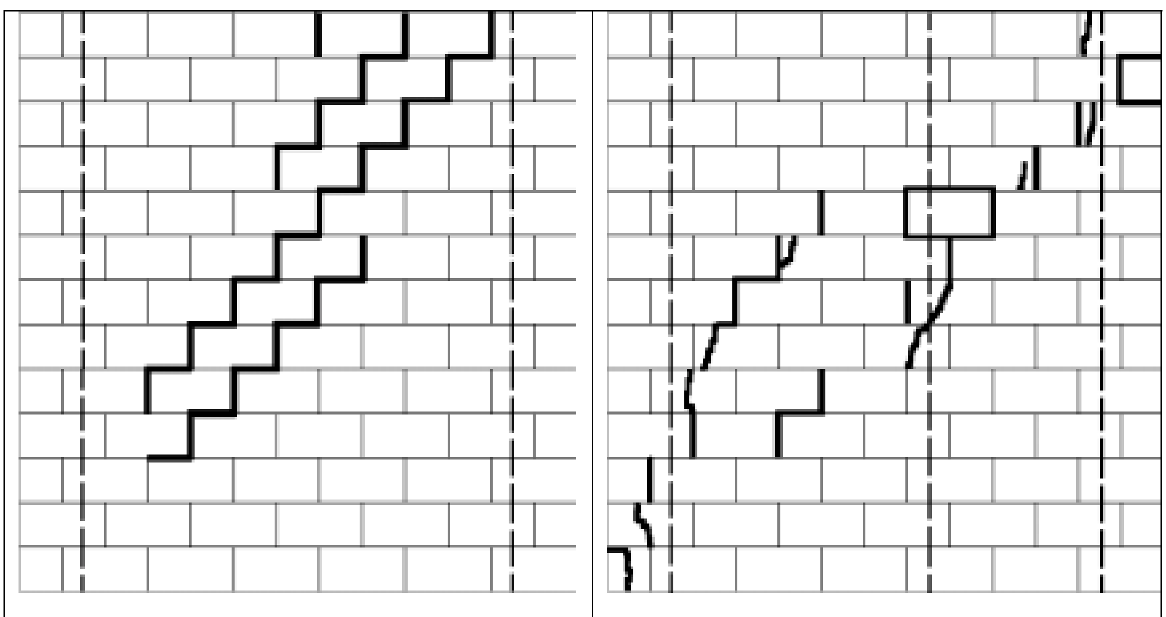
Panel designation	Lateral load (lbs)	Displacement dashpot #1 (in)	Drift, Δ dashpot #2 (in)	Drift ratio Δ/h (%)
Shear wall #1	4904	0.49	0.906	0.9
Shear wall #2	8378	0.44	1.420	1.4
Shear wall #3	4581	0.44	0.807	0.8
Shear wall #4	6390	0.26	0.556	0.5
Shear wall #5	4584	0.45	1.053	1.0
Shear wall #6	5982	0.41	0.907	0.9

As the walls were subject to lateral force, rotation about the corner of the wall panel was observed and this drift was measured as the lateral movement resulting from this rotation. In effect, these wall panels all behaved as cantilevers with shear slip. As more lateral load was applied, the crack pattern was observed and photographed. These photos were used to sketch the crack patterns that are shown in Figures 17, 18, and 19. In the figures, tendons are shown as dotted vertical lines. Finally, Figures 20, 21, and 22 are the load-drift curves for the 104 inch, 72 inch, and 56 inch wall panel, respectively.

Crushing of the toe of the wall was observed in 5 out of the 6 wall panels tested and the shorter panels generally exhibited more extensive toe crushing. For the longest wall panels – 104 inches – adding a third tendon introduced more extensive block crushing. For the shorter wall panels, adding a third tendon introduced more extensive block crushing and more random cracking not associated with typical stair-step cracking associated with shear failures. Table 6 summarizes the cracking observed in the shear wall test panels and estimates the area of toe crushing. The toe crushing area was estimated by taking the height of the unit and multiplying by the faceshell thickness. In all the shear wall panels, the units were cracked through the wall, so the toe crushing area was estimated using both faceshells.

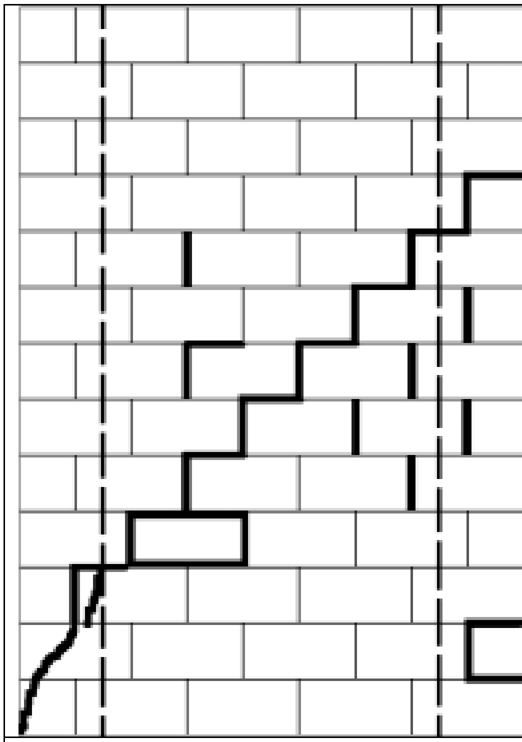
Table 6—Summary of Shear Wall Cracking and Unit Crushing

Panel designation	Number of blocks cracked	Approximate area of toe crushing (in^2)	Comments
Shear wall #1	None	0.0	No crushing or cracks observed
Shear wall #2	10	42.3	Cracking throughout the wall panel. Two units crushed at toe of wall
Shear wall #3	3	72.1	Three units crushed at toe of wall
Shear wall #4	4	29.9	One unit crushed at toe of wall
Shear wall #5	6	105.6	Five units crushed at or near toe of wall
Shear wall #6	6	63.4	Four units crushed at or near toe of wall

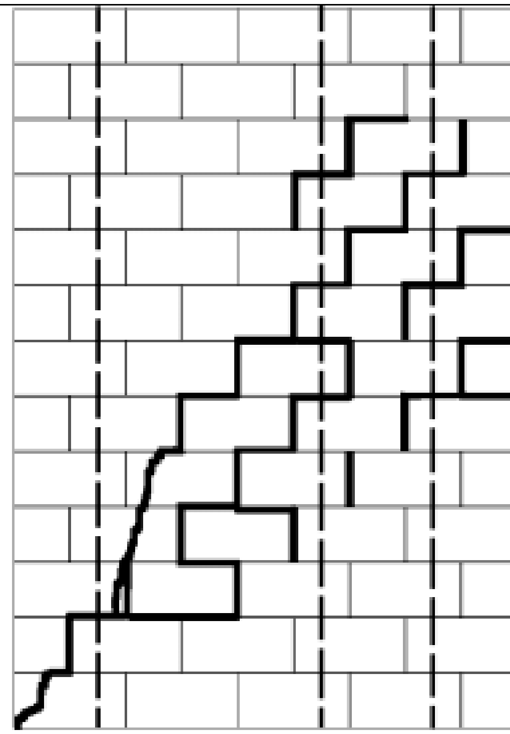


Shear wall #1 | **Shear wall #2**

Figure 17—Crack Pattern – 104-inch Wide Panels

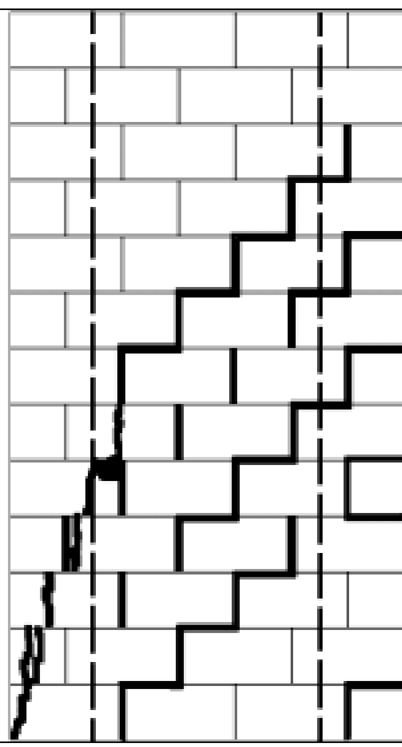


Shear wall #3

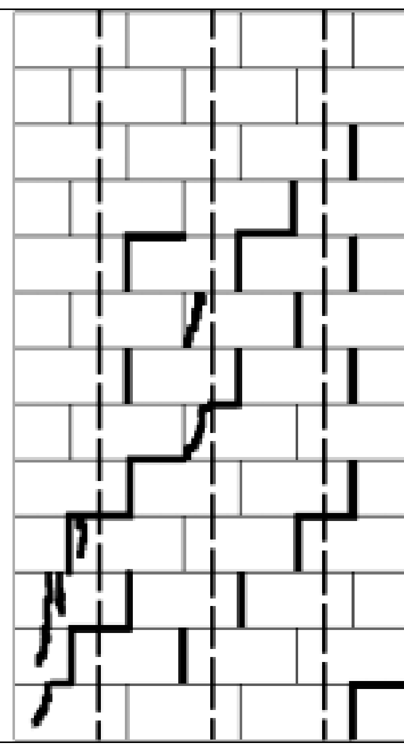


Shear wall #4

Figure 18—Crack Pattern – 72-inch Wide Panels



Shear wall #5



Shear wall #6

Figure 19—Crack Pattern – 56-inch Wide Panels

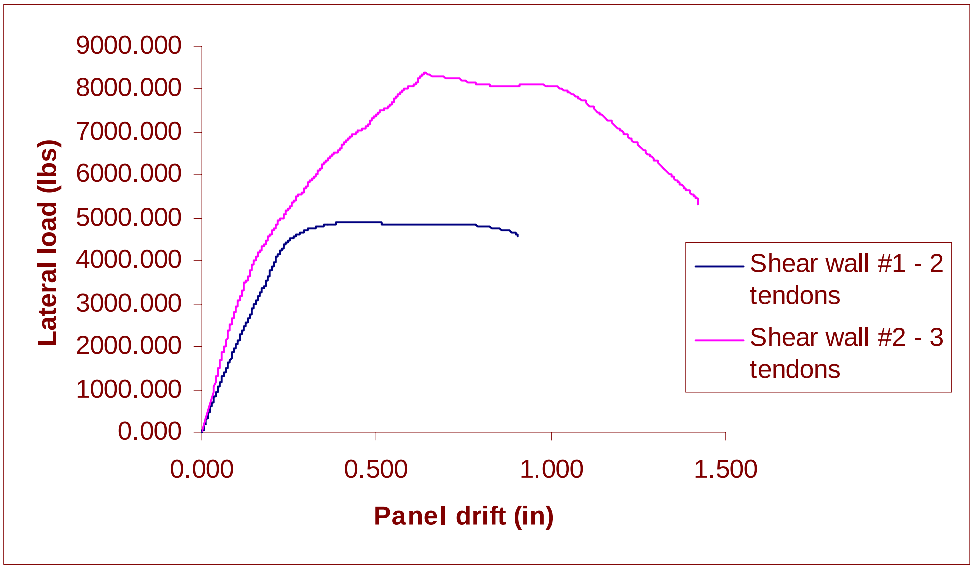


Figure 20. Comparison between Wall Panels #1 – 2 tendons and #2 – 3 tendons

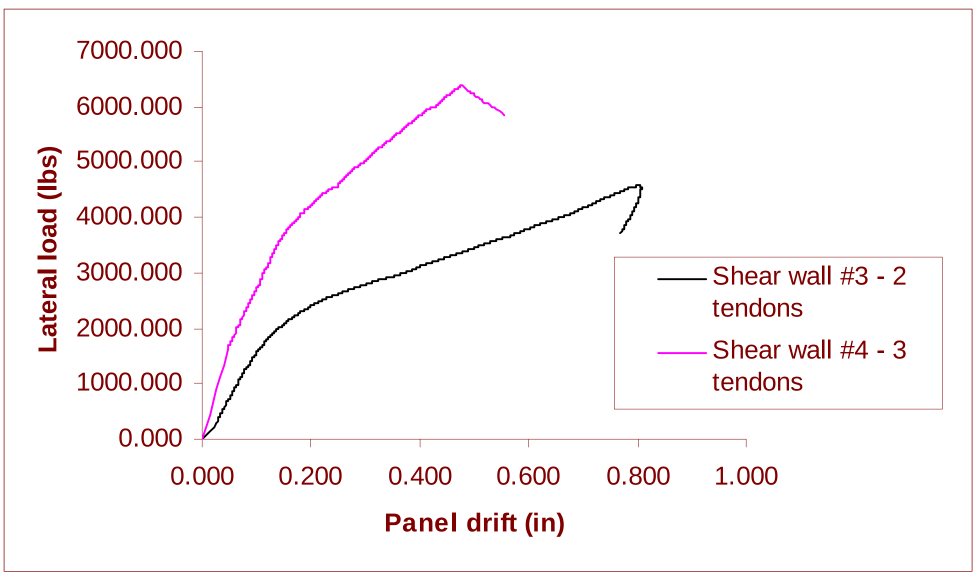


Figure 21—Comparison Between Wall Panels #3 – 2 Tendons and #4 – 3 Tendons

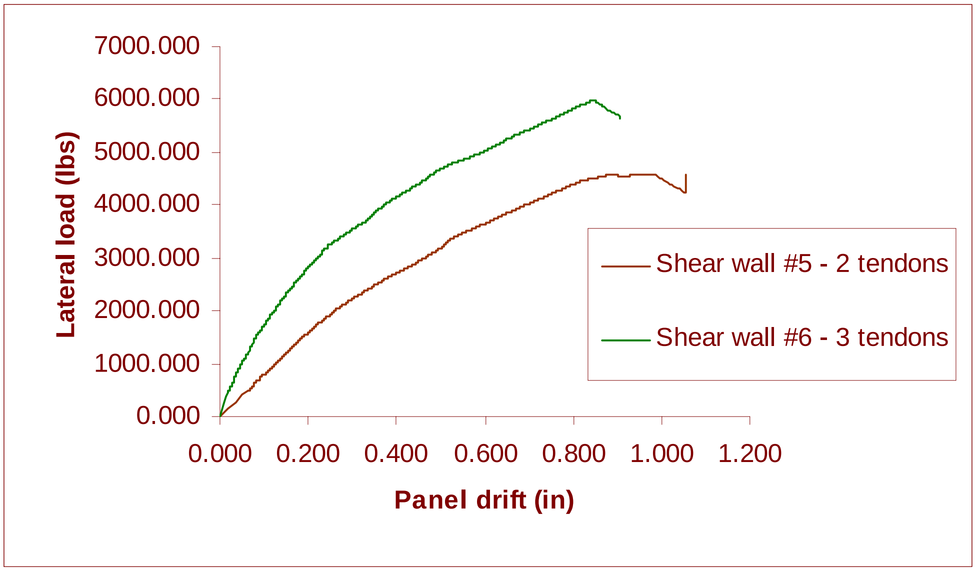


Figure 22—Comparison Between Wall Panels #5 – 2 Tendons and #6 – 3 Tendons

APPENDIX A
Lab Reports in units and prism testing

NCMA Research and Development Laboratory
ASTM C 426-99
Standard Test Method for Drying Shrinkage of Concrete Masonry Units

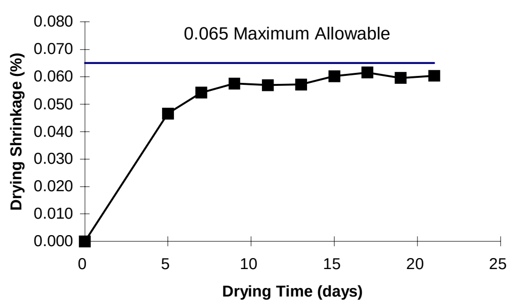
Client: Cercorp Initiatives Inc.
Address: 162 Township Highway 202
Bloomingdale, OH 43910

Job Number: 01-391
Date Rec'd: 10/11/01
Report Date: 01/02/02

Unit Size and Description: 8x8x16" Hollow, Post Tension CMU
4x16" Half face-shell Specimens

Sides A & B

	Unit #1		Unit #2		Unit #3		Average	
	Weight (lbs)	Total Linear Drying Shrinkage (%)	Weight (lbs)	Total Linear Drying Shrinkage (%)	Weight (lbs)	Total Linear Drying Shrinkage (%)	Weight (lbs)	Total Linear Drying Shrinkage (%)
As Received	4.80	---	5.00	---	5.03	---	4.94	---
Saturated	5.09	---	5.23	---	5.32	---	5.21	---
5 Days	4.67	0.053	4.93	0.041	4.91	0.047	4.84	0.047
7 Days	4.63	0.059	4.90	0.047	4.88	0.056	4.80	0.054
9 Days	4.61	0.065	4.88	0.049	4.85	0.060	4.78	0.058
11 Days	4.60	0.065	4.87	0.047	4.83	0.058	4.77	0.057
13 Days	4.59	0.066	4.86	0.048	4.83	0.057	4.76	0.057
15 Days	4.59	0.068	4.86	0.051	4.82	0.062	4.75	0.060
17 Days	4.58	0.069	4.85	0.052	4.81	0.063	4.75	0.062
19 Days	4.58	0.069	4.85	0.050	4.81	0.061	4.74	0.060
21 Days	4.57	0.069	4.84	0.050	4.80	0.062	4.73	0.060



Average Total Linear
Drying Shrinkage
at Equilibrium = 0.060 %

Jeffrey H. Greenwald, P.E.
Director of Research and Development

ASTM C 1314 Test Report:
Constructing and Testing Masonry Prisms Used to Determine Compliance with Specified Compressive Strength of Masonry

Project No.: 01-391-02
Report Date: 11/02/01

Client: Cercorp Initiatives Inc.
Address: 162 Township Highway 202
Bloomingdale, OH 43910

Testing Lab: National Concrete Masonry Association
Research and Development Laboratory
2302 Horse Pen Road
Herndon, VA 20171

Project Identification: ________________
Prism Identification: 8x16x16", Hollow, Dry Stack Bond, Concrete Masonry Prism

Specified Compressive Strength of Masonry: Not specified psi

Prism Details:
Number of Mortar Bed Joints: 1
Number of Masonry Units Used: 2
Date Constructed: 10/30/01
Date Grouted: N/A
Date Retrieved from Site: N/A
Date Delivered to Lab: N/A
Date Tested: 10/31/2001

Masonry Unit Information:
Unit Supplier: Cercorp Initiatives Inc.
Unit Dimensions: 8x8x16
Unit Net Area (hollow units): 66.9 in2

Grout Information
Grout Supplier / Preparer: N/A
Grout Type / Description N/A
Grout Slump (ASTM C 143): N/A
Method of Consolidation: N/A

Mortar Information
Mortar Supplier / Preparer: N/A
Mortar Type / Description: N/A

Compression Test Machine Information
Diameter of Spherical Seat: 10 in.
Required Upper Bearing Plate Thickness: 3.8 in.
Required Lower Bearing Plate Thickness: 1.0 in.
Provided Upper Bearing Plate Thickness: 5.1 in.
Provided Lower Bearing Plate Thickness: 2.5 in.

Tested Prism Properties:

Prism No.	Age at Test (days)	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Gross Area (in^2)	Max Load (lb.)	Gross Compr. Strength (psi)	h/t Ratio	h/t CF*	Corrected Gross Strength (psi)
1	N/A	7.59	16.05	15.93	120.87	108620	899	2.11	1.01	910
2	N/A	7.59	16.05	15.93	120.81	131460	1088	2.12	1.01	1100
3	N/A	7.59	16.08	15.93	120.83	96080	795	2.12	1.01	800
									Average	940

* Height to thickness correction factor from Table 1 of ASTM C 1314. Values have been linearly interpolated as necessary.

Compressive strength of masonry (average for the three prisms): **940** **psi**

Jeffrey H. Greenwald, P.E.
Director of Research and Development

ASTM C 140 Test Report
From Saw-Cut Specimens

Job No.: 01-391-01
Report Date: 11/02/01

Client: Cercorp Initiatives Inc.
Address: 162 Township Highway 202
Bloomingdale, OH 43910

Testing Agency: National Concrete Masonry Association
Research and Development Laboratory
Address: 2302 Horse Pen Road
Herndon, VA 20171

Unit Specification: ASTM C90

Sampling Party: Cercorp Initiatives Inc.

Unit Designation/Description:
8x8x16"
Hollow, Light Weight Post Tension CMU

Note: Specimens have been saw cut from a face shell of each unit in accordance with the provisions of 6.2.4 of ASTM C 140-01a

Summary of Test Results

Physical Property	Required Values	Tested Values[1]	
Net Compressive Strength	1900 min	5470	psi

Physical Property	Required Values	Tested Values[2]	
Min. Faceshell Thickness (t_{fs})	1.25 min	1.32	in.
Min. Web Thickness (t_w)	1.00 min	1.12	in.
Equivalent Web Thickness	2.25 min	2.54	in.
Equivalent Thickness	****	4.20	in.
Max. Var. from Spec. Dimensions	.125 max	0.37	in.
Net Cross-Sectional Area	****	66.9	in^2
Gross Cross-Sectional Area	****	121.0	in^2
Percent Solid	****	55.3	%
Density	****	97.8	
Absorption	18 max	8.8	pcf

[1] Reported values are based on the properties of saw cut compression specimens.
[2] Reported values are based on the properties of full sized units.

Individual Unit Test Results

Properties of Full-Size Units	Avg Width in.	Avg Height in.	Avg Length in.	Avg./Min. t_{fs}[3] in.	Min. t_w in.
Unit #4	7.59	7.99	15.95	1.33	1.12
Unit #5	7.60	7.99	15.91	1.32	1.12
Unit #6	7.59	7.99	15.94	1.32	1.13
Average	7.59	7.99	15.93	1.32	1.12

[3] Where the thinnest point of opposite face shells differ in thickness by less than 0.125 inches, their measurements are averaged.

Properties of Saw-Cut Compression Specimens	Received Wt, W_R lb	Avg Width in.	Avg Height in.	Avg Length in.	Net Area[4] in^2	Max. Load lb	Net Compressive Strength psi
Unit #1	1.00	1.27	2.52	5.05	6.39	37120	5810
Unit #2	1.02	1.27	2.52	5.04	6.40	39480	6170
Unit #3	0.97	1.26	2.52	5.03	6.36	28220	4440
Average	1.00	1.27	2.52	5.04	6.38	34940	5470

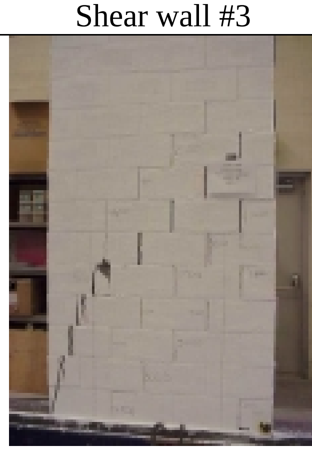
[4] Unit areas determined as the product of the width and length of coupons tested in compression.

Properties of Full-Size Absorption Units	Received Wt, W_R lb	Immersed Wt, W_I lb	Saturated Wt, W_S lb	Oven-Dry Wt, W_D lb	Absorp pcf	Density pcf	Net Volume ft^3	Percent Solid %
Unit #4	32.24	13.73	33.10	30.74	7.6	99.0	0.3104	55.45
Unit #5	31.78	13.76	33.04	29.98	9.9	97.0	0.3090	55.30
Unit #6	31.58	13.55	32.84	30.12	8.8	97.4	0.3091	55.26
Average	31.87	13.68	32.99	30.28	8.8	97.8	0.3095	55.34

Comments: These tested properties meet or exceed the applicable requirements of ASTM C 90.

Jeffrey H. Greenwald, P.E.
Director of Research and Development

APPENDIX B
In-plane Shear Stress
Photos of Wall Panel Failures

Wall length	2 tendons	3 tendons
104 inches	 Shear wall #1	 Shear wall #2
72 inches	 Shear wall #3	 Shear wall #4
56 inches	 Shear wall #5	 Shear wall #6