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Dry Stacked Surface Bonded Masonry - Structural Testing and Evaluation

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DRY STACKED SURFACE BONDED MASONRY – STRUCTURAL

TESTING AND EVALUATION

by

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GRADUATE COMMITTEE APPROVAL

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ABSTRACT

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The ENDURA block system is a dry-stack surface-bonded masonry system. Typical masonry construction uses thin-set mortar in the bed joints to provide a bearing surface for the blocks while the ENDURA system typically relies on shims and a surface bonding coat to ensure that the wall is level and plumb and to provide stability. Typical ENDURA block walls are built with the reinforcement placed eccentrically in the walls. Testing was performed on ten walls in order to determine axial capacity. The walls were ten feet high by eight feet wide. Each of the walls was built using a different configuration of block type, reinforcement spacing, and amount of grout. A steel frame with two hydraulic jacks was used to apply vertical load to the top of the walls. Three conclusions were drawn from the axial testing performed. First, typical ENDURA block walls built without thin set mortar in the bed joints have similar axial capacity to walls built with the thin set mortar. Second, walls built with un-reinforced cells grouted resisted

significantly more load than walls built with only the reinforced cells grouted. Third, more research is required in order to establish a control and to determine whether the eccentrically placed rebar has a significant effect on the axial capacity of the walls.

ACKNOWLEDGMENTS

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1 Introduction

In June of 2007, testing was performed on ten dry-stacked surface-bonded masonry walls, ENDURA walls, at Brigham Young University to determine their axial capacity. ENDURA walls are typically dry-stacked concrete masonry units with eccentrically placed reinforcement and a surface bonding cement applied to the face of the walls. The surface coat provides a physical bond between the blocks, and is also applied for aesthetic purposes. Polystyrene insulation inserts are placed at all ungrouted hollow block cells.

The fundamental goal of this research was to measure the ultimate axial capacity of the walls and determine the difference, in any, in capacity between typically built ENDURA walls and ENDURA walls built with thin-set mortar between the courses of blocks. The tested walls were 10’ high by 8’ feet wide. The size of the walls was chosen to represent a typical wall that may be built. . All walls were tested to failure by applying an approximately uniformly distributed axial load at the top of the walls.

There are several differences between the ENDURA block wall system and typical concrete block construction. Two of the most significant differences are that the blocks are typically dry-stacked or placed without mortar in the bed joints and the reinforcement is placed eccentrically in the wall. In order to compare and contrast these variations from typical CMU construction testing was performed on several ENDURA walls with mortar between the blocks or with centered reinforcement.

A discussion of current concrete masonry construction and available dry-stack systems is provided in Chapter 2 of this thesis. A literature review of relevant articles to the subject of this thesis is also presented in Chapter 2. Chapter 3 presents the test matrix and summarizes the individual walls which were tested. The methods used to construct the ENDURA block walls, the testing, and data collection procedures are given in Chapter 4. In Chapter 5 the results of the testing performed on the ENDURA block walls are given. Chapter 6 presents a discussion of the quantitative results as well as a summary of some significant visual observations. Chapter 7 lists the conclusions from the tests conducted.

2 Dry-Stack Masonry Systems

This section presents a discussion of traditional CMU construction as well as dry-stack masonry systems. A literature review of several peer reviewed articles relating to dry-stack systems is also contained in this chapter.

2.1 Traditional Concrete Masonry Construction

There are several disadvantages to using concrete masonry units in construction when compared with other methods such as tilt up concrete or steel framing. CMU construction often requires more time spent by more highly skilled laborers which means added cost to the project. This problem is now being magnified by the fact that the number of qualified masons is shrinking quickly in the United States. Another disadvantage to typical CMU construction is the amount of mortar that is required to be mixed on-site. This makes construction time consuming and also makes construction during inclement weather very difficult. Shrinkage cracking is another significant issue that faces CMU construction in harsh climates. (Beall, C., 2000)

One method that has been suggested to alleviate some of these problems is to dry-stack the blocks. The dry-stack method makes construction significantly easier and thus reduces the need for skilled labor, and makes year round construction more feasible. However, dry-stack systems are not without their disadvantages. One of the chief issues is that without the mortar between courses of block there is no easy way to deal with irregularities in the individual blocks. There is no economical method of producing concrete masonry blocks with little or no variation in height. Dry-stack masonry contractors have come up with several methods to address this issue including using metal shims between blocks, using small amounts of mortar where required, or placing the blocks in a stack bond.

2.2 Dry-Stack Systems in the United States

In the United States there are six companies which have dry-stack masonry systems available on the market:

- Haener Block
- Azar Block
- Sparlock
- Durisol
- Faswall
- Endura Block

Each system has its advantages and disadvantages. The Azar block (Report NER-683), Durisol (Report ER-5472), and Endura (Report ER-4997) block systems each have ICC Legacy reports available. Each system is constructed without mortar between courses of blocks and the individual blocks are generally the same size as a typical 8”x8”x16” CMU block, but each system is unique.

Haener block has been on the market longer than any other dry-stack system. It is an interlocking system; the individual blocks have raised lugs that align with the block above. The system requires the same amount of grout as conventional CMU construction. Azar block walls are a similar dry-stack interlocking system. The bed and head joints are manufactured to interlock with adjacent blocks. Azar block, however, requires that all walls be solid grouted.

The Sparlock system uses unique shaped blocks that slide together. The blocks are placed in a stack bond arrangement. Sparlock is not typically used in bearing wall situations but has been employed heavily on the firewall market. Vertical reinforcement and grout may be used but typically are not required for non-bearing situations.

The Durisol and Faswall systems are made of composite materials consisting of soft wood fibers and Portland cement. Because of their decreased compressive strength, these systems also require all walls to be solid grouted. The advantage of these composite block systems is that they are light weight and therefore construction is easier and faster. (Vanderwerf, P., 1999)

2.3 The ENDURA Block System

The focus of this thesis and testing has been on the ENDURA block system. The system has three main components:

1. The patented block
2. The patented poly-styrene insulation inserts
3. The fiber reinforced surface bonding cement

The exterior shape of the individual block is quite similar to a conventional 8”x8”x16” CMU block, but the interior configuration of the block is significantly different. The two rows of openings allow room for the structural components of grout and reinforcement as well as for the insulation inserts. There are five different block configurations in the ENDURA wall system:

- Stretcher (Figure 2.1)
- Right corner (Figure 2.2)
- Left corner (Figure 2.3)
- Half stretcher (Figure 2.4)
- Half Square (Figure 2.5)

In a typical ENDURA block wall, the stretcher block will be the most commonly used. The purpose of the stretcher block is to span between corners. The right and left corner blocks are used at corners in the wall depending on which way the wall is turning. The half stretcher block is used in case specific situations which may arise on a job. The half square blocks are typically used where the wall ends without continuing around a corner.

The polystyrene inserts serve two important functions in the ENDURA block wall system. First, they provide insulation for the building and this is one of the major selling points of the system. The other important function of the insulation inserts is that since they are slightly taller than the blocks themselves they help alleviate the problems created because of block irregularities. Along with using metal shims, the insulation inserts help overcome this major drawback to dry-stack masonry systems.

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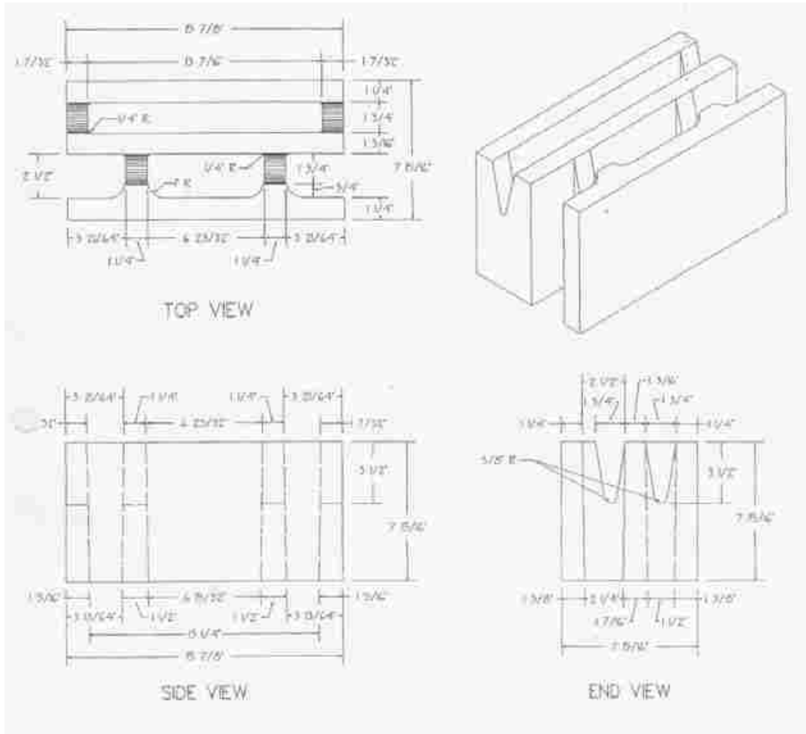

Figure 2.1 Stretcher Block

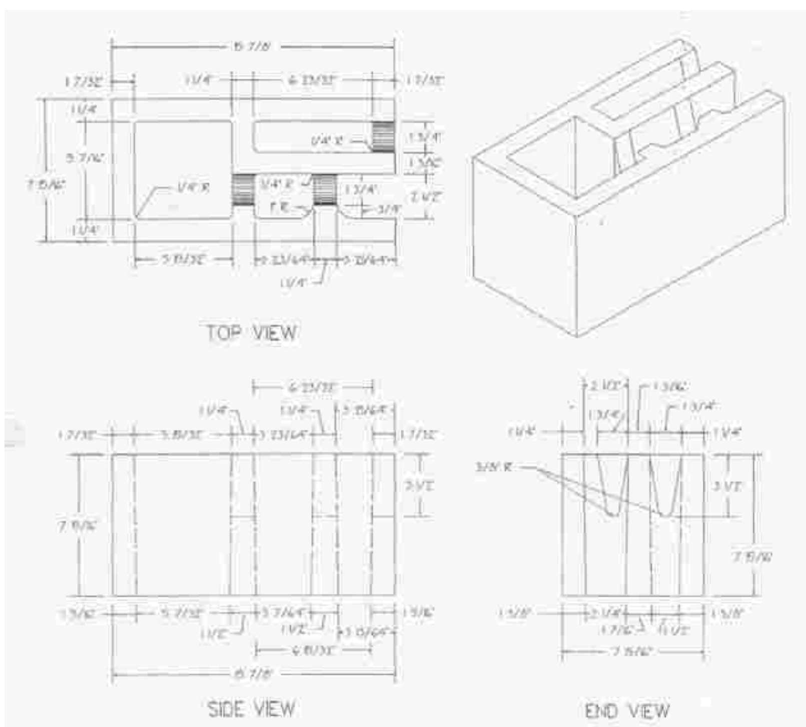

Figure 2.2 Right Corner Block

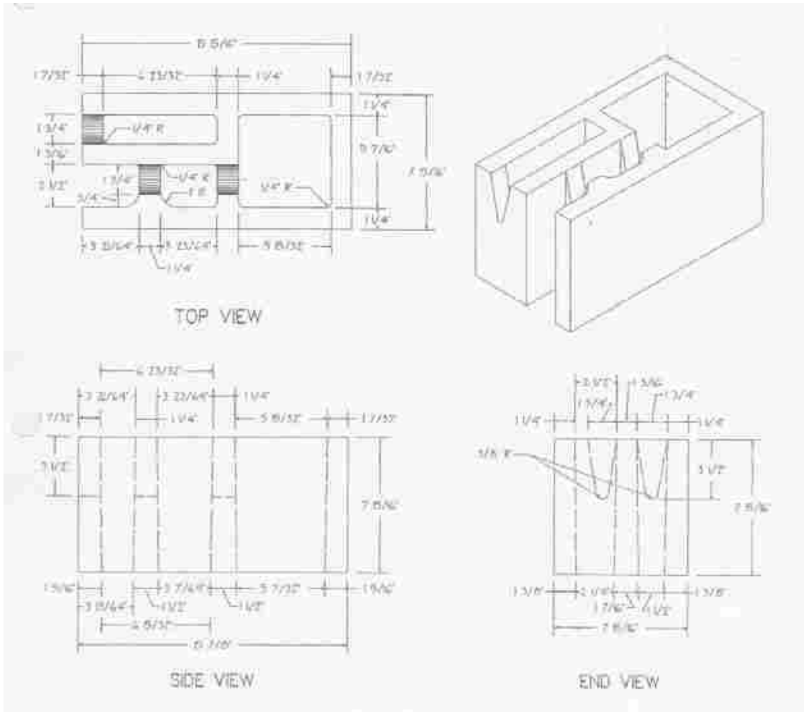

Figure 2.3 Left Corner Block

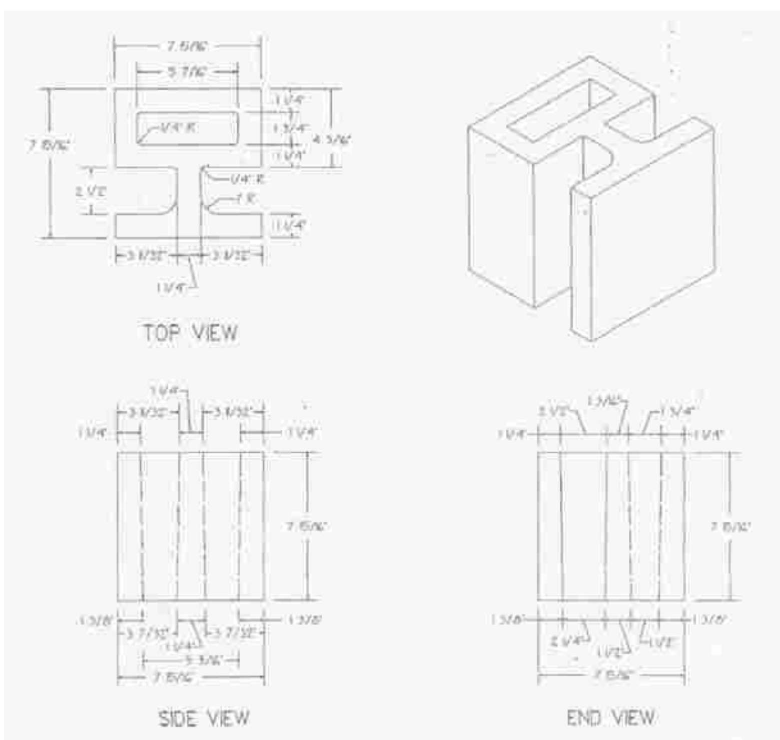

Figure 2.4 Half Stretcher Block

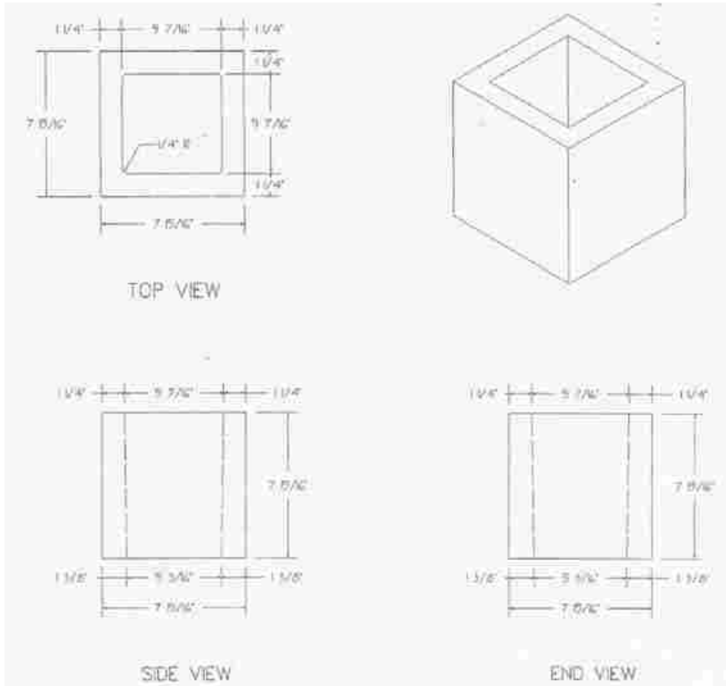

Figure 2.5 Half Square Block

individual blocks. Along with using metal shims, the insulation inserts shown in Figure 2.6 can help overcome this major drawback to dry-stack masonry systems. The two shapes of inserts available are long inserts, which fit in the narrow exterior cavity, and short inserts, which fit in the interior cavities which are not grouted.

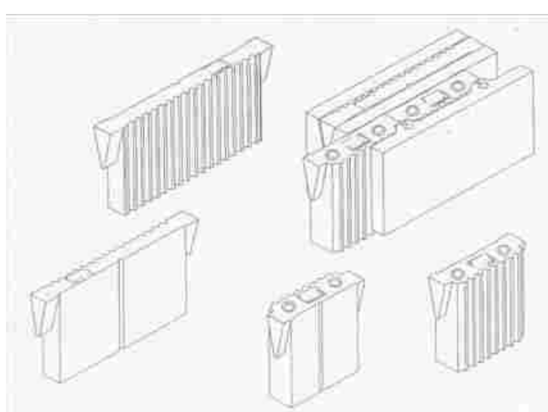

Figure 2.6 Insulation Inserts

2.4 Relevant Literature

The ENDURA block product is a proprietary system; therefore little technical literature is available specifically relating to the ENDURA system. However, a literature review was conducted on research and testing relating to dry-stack masonry systems. While many of these studies focused on other proprietary dry-stack systems, they were still valuable in understanding the issues and concepts concerning dry stacked masonry walls. The following provides a summary of several articles that have valuable information with respect to this study.

2.4.1 Laboratory-Based Productivity Study on Alternative Masonry Systems (Anand, K.B., and Ramamurthy, K. (2003))

In this paper, Anand and Ramamurthy discuss the productivity of Dry stacked masonry systems versus conventional mortar jointed masonry construction. The structural performance of the walls was not tested. The efficiency and productivity were studied, since increased efficiency is said to be one of the chief advantages of dry-stack systems. In this study the output of a mason's crew were evaluated as different types of masonry walls were constructed. Anand and Ramamurthy concluded that a crew could output approximately 60% more using interlocking dry-stacked blocks than with traditional hollow block masonry.

2.4.2 Dry-Stacked Masonry in Comparison with Mortar Jointed Masonry (Marzahn, G. (1997))

Gero Marzahn lists several advantages of dry stack masonry in comparison with traditional solid masonry. The lack of mortar in dry stacked systems leads to a much lower moisture content in a masonry wall. Such lower moisture content may help decrease moisture damages as well as shrinkage cracking. With traditional concrete block construction, a wall cannot be loaded immediately after construction because the uncured mortar has not achieved its final strength. The individual blocks may also tend to “swim” in the mortar leading to decreased stability during consruction. These issues, however, do not affect dry stack systems. A dry stacked masonry wall achieves its full compressive strength immediately after construction is complete.

Marzahn suggests that the thickness of a mortar layer in masonry wall has a direct influence on compressive strength. The thicker the mortar layer, the lower the compressive strength. He also asserts that the compressive strength of dry stacked masonry may be higher because there is no interaction between the concrete block and the mortar layer. However, the lack of mortar may also be a disadvantage because of slight variations in individual block heights.

Testing was performed on dry stacked samples which were five units high and one unit wide. Testing was also performed on similar traditionally built masonry walls, which were tested both with and without the use of mortar. Marzahn compared stress versus strain for the walls built with and without mortar. The stress strain curve for the mortar jointed walls appeared to be nearly linear until failure suggesting that there was nearly a constant modulus of elasticity. However, the dry-stacked wall’s stress strain curve showed large initial deflections and a modulus of elasticity that increased as vertical load increased. Marzahn attributes this to “the fact that the dry-stacked units had to settle down in order to balance uneven surfaces and notches before they were able to carry loads.” One suggestion to minimize such an effect is to prestress the masonry walls.

2.4.3 Investigation on the Initial Settlement of Dry-Stacked Masonry under Compression (Marzahn, G. (1999))

There has been relatively little research and testing done to evaluate the performance of dry-stacked masonry based on failure behavior. Compression tests performed by Marzahn have shown "that a strength up to 85-95 percent of the compressive strength of thin mortar layered masonry is possible." In earlier work (Marzahn, 1997), differential settlement as axial compressive load was applied to dry stack masonry walls had been observed. More testing was performed to analyze the failure behavior of dry-stacked walls.

Short term and long term tests were performed on dry stacked masonry walls built using varying individual block strengths. Walls using a thin mortar layer were also tested as a means of comparison.

The results of the testing showed that while the ultimate capacities of the dry stacked and mortar jointed walls were similar; the failure or deformation behavior was quite different. The walls built using mortar tended to have a nearly linear relationship between stress and strain from the beginning of the test up to failure. However, the stress-strain behavior of dry masonry was somewhat bi-linear. The first part of the curve, which extended to approximately one third of the failure load, was linear which resulted from the initial settlement of uneven surfaces and block inaccuracies. The second part of the curve depended more on the deformation of the bricks. An understanding of this initial settlement is important in the study of dry stacked masonry systems. Dry stacked walls tended to have large initial deformations. Marzahn suggests that prestressing the walls could be a viable method of limiting these large initial deflections.

2.4.4 Load Capacity of Dry-Stack Masonry Walls (Uzoegbo H. C., Senthivel R., and Ngowi J. V. (2007))

At the University of Witwatersrand in Johannesburg, South Africa testing was performed on a dry-stack masonry system. The system chosen for this testing was the Hydraform block system which is not currently available in the United States. The Hydraform system is an interlocking system that uses the interlocking ability of the individual blocks to provide stability and strength lost when mortar is not used. The authors of this article sought to research the structural behavior of dry stack systems which they claim to be more cost and time effective when compared to traditional concrete block construction. They also claim significant cost savings “due to savings in cost of mortar, the block units, and construction time.”

Axial compression tests were conducted on walls which were 3 meters wide and 2.5 meters tall. The first course of block was laid in mortar in order to provide a level bearing surface. A 3 meter long steel beam was used to distribute the applied load evenly on top of the wall. Dial gauges were used to monitor displacement of the walls. Walls to be tested were built using blocks with varying unit strengths.

The general failure mode of the tested walls was vertical cracks at the mid section of the walls. A loud snap often accompanied the appearance of the vertical cracks. When some of the lower unit strength walls were tested the top courses of block were crushed at failure loads. Crushing usually occurred as the ratio of unit strength to overall wall panel strength decreases.

A control wall built using traditional concrete block techniques was also tested. The control wall result was used to normalize the testing results of the dry-stack walls. The failure load of the dry-stack walls was divided by the conventional wall’s capacity.

The results of the testing were used to develop a proprietary “relationship between the unit strength and the masonry panel strength.” Results of these tests showed a 65% increase in axial strength when mortar was used in the bed joints. The difference in strength was attributed to a difference in failure mode. The dry stack walls tended to fail in shear and splitting of the head joints. When mortar was used in the bed joints, the mortar resisted the shear, which slightly increased the axial capacity of those walls.

2.4.5 Development and Evaluation of Hollow Concrete Interlocking Block Masonry System (Anand, K. B., and Ramamurthy K. (2005))

Anand and Ramamurthy, led by a desire to have a more efficient masonry building system, developed a dry-stack ineterlocking masonry system called the IITM-HILBLOCK system. This system was developed and tested in India and is not available in the United States. These researchers also claim that the dry-stack method of masonry construction has several advantages over traditional masonry construction. These advantages include “simplicity in block laying, reduction in mortar consumption, and general independence of workmanship variations.” A labor cost reduction of up to 80%” is reported.

The HILBLOCK system is made up of vertically and horizontally interlocking dry stack concrete masonry blocks. This system also includes open-ended units to simplify the placement of vertical reinforcement. When masonry blocks are required to be lifted over vertical reinforcement, there tend to be more splice points in the wall. The open-ended units can be slid horizontally into place which decreases the number of splice points used.

Both concentric and eccentric axial testing was performed on the HILBLOCK system. Test specimens were comprised of three units stacked vertically. Testing was conducted on both dry-stacked prisms as well as units stacked with a thin layer of mortar between blocks. Grouted as well as ungrouted specimens were also tested.

The results of the axial strength tests showed a 20-30% increase in prism strength when the thin layer of mortar was used. During some of the tests, the grout column actually remained intact as the face of the block shells fell away. These results indicate that the amount of grout used directly influences the axial capacity and failure modes of the prisms. Test results from the eccentric load testing showed "as expected, decreasing capacities associated with increasing eccentricities."

2.4.6 Devolopment and Performance Evaluation of Interlocking-Block Masonry (Anand K. B., and Ramamurthy K. (2000))

In this article, Anand and Ramamurthy summarize the testing conducted on the SILBLOCK system, a system used primarily in India. A comparison is made between this proprietary dry-stack interlocking block system and traditional mortar bonded masonry.

Ten masonry wallettes were constructed and tested under a concentric axial load. The results of the testing showed that "the allowable axial compressive stress for interlocking block masonry is higher than that of conventional masonry."

Fifteen masonry wallettes were also constructed and tested under an eccentric axial load. Walls were tested with eccentricities of 0, t/3, and t/6, where t is the width of the block. The results of the test also showed a significant decrease in strength when the eccentricity was increased. However, this testing showed that the reduction in capacity due to eccentricity was smaller than that for a typical CMU wall. At an eccentricity of t/6, the capacity reduction was only around 10% for the SILBLOCK masonry as compared to approximately 30% for the conventional system. The observed increase in strength under eccentric load was attributed mostly to the interlocking features of the SILBLOCK system.

2.4.7 Behaviour of Interlocking Mortarless Block Masonry (Jaafar, M. S., Alwathaf, A. H., Thanoon, W. A., Noorzaei, J., and Abdulkadir, M. R. (2006))

The main disadvantage of dry stacked masonry systems is that geometric imperfections in individual blocks and varying individual block heights play an increased role in the performance of a system. In traditional concrete block construction, these imperfections are compensated for by the mortar in the bed joints. The testing summarized in this article sought to investigate two types of geometric imperfections. The first type is caused by the variation of regularity and roughness of the block bed interfaces, while the second is caused by variations in individual block height. When neighboring blocks are slightly different heights, a gap can form when blocks are placed in a running bond.

Two different tests were performed. The first test was a “Single Joint” test. The single joint test was comprised of two blocks stacked on top of each other. Small mechanical gauge Demec Points (DPs) were placed near the block interfaces. These DPs were placed near the interface to measure only the deflection caused by the first type of geometric irregularities. The results of the single joint test showed that there was a change in stiffness during testing. The initial stiffness was attributed to settling of the blocks and the closing of block irregularities. As more of the block areas came into contact the stiffness increased slightly.

The second test was a “Multiple Joint” test. In this test, blocks were placed in a running bond. The test was to simulate both types of irregularities in a dry stack wall. The results of this test showed large differences in displacement between tested walls. These large differences were attributed to the varying block heights which caused small gaps between interfaces. The sized of these gaps varied from walls to wall, thus the differences in deflection.

2.4.8 Structural Behavior of an Interlocking Masonry Block (Hatzinikolas, M., Elwi, A. E., and Lee, R. (1986))

Hatzinikolas, Elwi, and Lee suggested three important parameters that are required for a dry-stack masonry system to be successful. First, a dry-stacked system “is effective only if its performance is at least equal or superior to that of normal blocks.” Second, the system must provide adequate bending strength in both the vertical and horizontal directions. Third, the system must provide adequate resistance to water penetration and have good insulation properties. In this article, the researcher tested and analyzed the G. R. dry stacked masonry system, which is a proprietary system similar to the Sparlock system. The individual blocks interlock and are laid in a stack bond.

Fifteen five course tall by three course wide walls were tested in axial compression. Walls were tested under compression with load applied normal to the bed joints. The load was applied at the top of the walls and distributed by a 130 mm deep steel channel. A layer of compressible fibreboard was used at the top and bottom of the walls in order to ensure that the load was applied evenly on the walls. As ultimate loads were approached, vertical face shell cracking developed. This vertical cracking was attributed to the lateral tensile stresses that develop in the block as the interior grout expands under compression. Numerical values were obtained for the axial capacities of the tested walls. However these values are only applicable to the G. R. system.

2.4.9 Surface Bonding Cement: A New Technology for Masonry (Klausmeier, R. D. (1978))

The use of a surface bonding cement is one method that has been suggested to compensate for the lack of mortar in dry-stacked masonry walls. The surface bonding cement is applied in a 1/8" thick layer at each face of a wall and is comprised of fiberglass reinforced concrete. Test results presented in this article show mixed results pertaining to the effectiveness of dry stacked surface bonded masonry walls. Tests showed a 64% increase in flexural strength when compared with a traditionally built masonry wall. However, the testing also showed a 43% decrease in axial compressive strength for dry stacked surface bonded walls. The tests also showed that the surface bonded walls had excellent resistance to water penetration.

2.4.10 Effect of Grouting on the Strength Characteristics of Concrete Block Masonry (Hamid, A. A., Drysdale, R. G., and Heidebrecht, A. C. (1978))

The recognized procedure for estimating the strength of a grouted masonry wall is based on the gross cross sectional bearing area of the wall. The gross area includes the area of masonry and grout. The grout is required to have at least the same strength as the concrete block. Thus, the strength of the gross cross section may be conservatively based on the strength of the weaker concrete block. This article sought to review the validity of this method through several tests performed on both grouted and ungrouted masonry prisms. Through testing, the researchers also sought to determine the influence of grout strength on the overall prism strength. The prisms tested consisted of three half block courses with varying mortar and grout characteristics.

The results of these tests indicated that only small increases in prism strength were achieved with large increases in grout strength. The tests also showed that a superposition of grout strength multiplied by grout area and block strength multiplied by block area greatly overestimates the strength of the prisms. The researchers concluded that mortar strength was not the most significant parameter in determining axial strength.

2.4.11 Summary of Literature Review

After a review of the technical literature available, a number of conclusions can be drawn concerning dry-stacked masonry systems:

- Dry stack masonry has several advantages over conventional mortar jointed masonry including: faster construction time, lower construction costs, and smaller amounts of required wet material.
- There are numerous proprietary systems that each require individual testing.
- There are two predominant methods of compensating for some of the attributes lost when mortar is not used. First, the individual blocks can be designed to interlock with adjacent blocks. Second, a surface bonding cement can be applied to the surface of the walls.
- Dry stacked walls tend to have large initial deflections as blocks settle and as gaps caused by block height irregularities are closed. After the large early displacements, the stiffness tends to increase.

- The amount of grout has a large influence on the overall axial capacity of a wall. However, increasing the strength of the grout may not be as influential.
- A superposition of bearing areas multiplied by individual component strengths may overestimate the capacity of a wall.
- When walls are loaded eccentrically, the capacity is decreased. However, the effect of eccentricity may be slightly smaller for dry stacked walls than for traditional block walls.
- Failure generally tends to be preceded by vertical cracking caused by lateral stresses developed when the grout column expands under compression. When test prisms were solid grouted, or when the contribution of grout strength to overall strength increases, failure was typified by crushing at the upper section of test prisms.
- A layer of thin set mortar tends to have a minor, yet measurable influence on axial capacity. The mortar layer tends to slightly increase axial strength.

2.5 Preliminary Calculations

Calculations were made prior to testing to estimate the axial capacity of a typical wall. Two methods of estimating the axial capacity on the wall were used. The first was the strength design method outlined in the Building Code Requirements for Masonry Structures (ACI 530, 2005) and given in Equation 2-1.

The strength design equation was developed to calculate the axial capacity of traditional concrete block walls, and is therefore not directly applicable to ENDURA

$$P_n = 0.80\left\{0.80 A_n f'_m \left[1-\left(\frac{h}{140r}\right)^2\right]\right\} \quad \textbf{2-1}$$

where

P_n = Axial Compressive Strength

A_n = Net Cross Sectional Area

f'_m = Masonry Unit Strength

h = Height of Wall

r = Radius of Gyration

block walls. The estimated capacity was used to establish an upper bound and determine if the testing equipment would be capable of testing the walls to failure.

The second method used to estimate the axial capacity of a typical wall was the strength design method from the ICC Evaluation Service Legacy Report for the ENDURA block system (ICC-ER-4997, 2001). The method uses Equation 2-2.

$$P_n = (0.8)[0.85 f'_m (A_e - A_s) + f_y A_s] \quad \textbf{2-2}$$

where

f'_m = Masonry Unit Strength

A_e = Effective Area

A_s = Area of Steel

f_y = Steel Yield Strength

Wall #1 was the solid grouted wall and thus had the largest structural bearing area and was expected to have the greatest axial capacity. Preliminary calculations for wall #1 were used to estimate the adequacy of the testing apparatus. Table 2-1 shows the estimated ultimate axial capacity of wall #1 using each of the methods discussed above.

Table 2-1 Estimated Axial Capacity

	Building Code equation (kip)	ENDURA block ICC Report (kip)
Wall #1 Pn=	626	807
Wall #1 φPn=	563	194

3 Test Matrix

The set of ENDURA BLOCK walls tested in June 2007 included ten different configurations. The variables between the walls included type of block, grout and reinforcement spacing, the use of a thin set mortar, and the vertical spacing of the bond beam. All steel reinforcement used was #4 rebar. Table 3-1 provides a summary of the Test Matrix for this set of walls.

Table 3-1 Test Matrix Summary

Wall #	BLOCK TYPE	REBAR LOCATION	VERTICAL REBAR SIZE	VERTICAL REBAR SPACING	HORIZONTAL BOND BEAM SPACING	THIN SET USED	LARGE CELLS GROUTED	SMALL CELLS GROUTED
1	CORNER	CENTER	#4	16" o.c.	48"	-	X	X
2	CORNER	CENTER	#4	16" o.c.	48"	-	-	X
3	CORNER	CENTER	#4	16" o.c.	48"	-	-	-
4	CORNER	CENTER	#4	16" o.c.	48"	X	-	-
5	CORNER & STRETCHER	CENTER	#4	24" o.c.	48"	-	-	X
6	CORNER & STRETCHER	CENTER	#4	24" o.c.	48"	-	-	-
7	CORNER & STRETCHER	CENTER	#4	24" o.c.	48"	X	-	-
8	STRETCHER	EDGE	#4	24" o.c.	48"	X	-	-
9	STRETCHER	EDGE	#4	24" o.c.	48"	-	-	-
10	STRETCHER	EDGE	#4	48" o.c.	60"	-	-	-

Wall #1 was built using corner blocks only. All cells including the corner square cells and both large and small cells were grouted solid. Vertical reinforcement was placed at 16” o.c. in the grouted corner square cells. Horizontal reinforcement was placed at 4’, 8’, and 10’. Since all cells in wall #1 were grouted solid, no insulation inserts were used.

Wall #2 was also built using corner blocks only. Small cells were grouted while large cells were filled with insulation inserts. Vertical reinforcement was placed at 16” o.c. in the grouted corner square cells.

Wall #3 was also built using corner blocks only. All small and large cells were filled with insulation inserts. Vertical reinforcement was placed at 16” o.c. in the grouted corner square cells.

Wall #4 was built the same as wall #3 except that a layer of thin set mortar was used between all block courses.

Wall #5 was built using alternating corner and stretcher blocks. Small cells were all grouted while the large cells were filled with insulation inserts. Vertical reinforcement was placed at 24” o.c. in the grouted corner square cells.

Wall #6 was also built using alternating corner and stretcher blocks. All small and large cells were filled with insulation inserts. Vertical reinforcement was placed at 24” o.c. in the grouted corner square cells.

Wall #7 was built the same as wall #6 except that a layer of thin set mortar was used between all block courses.

Wall #8 was built using stretcher blocks only. Vertical reinforcement was placed at 24” o.c. vertical in a grouted small cell. All remaining large and small cells were filled with insulation inserts. Inserting rebar in the small cells of the ENDURA blocks means the reinforcement is eccentrically placed within the wall.

Wall #9 was built the same as wall #8 except that a layer of thin set mortar was used between all block courses.

Wall #10 was built using stretcher blocks only. Vertical reinforcement was placed at 48” o.c. in a grouted small cell. All remaining large and small cells were filled with insulation inserts. Similar to walls #8 and #9, the reinforcement was not centered in the wall.

Walls #1 through #9 had the sixth, twelfth, and fifteenth course of blocks grouted solid with a horizontal reinforcing bar extending the full length of the wall. This created horizontal bond beams at 4’, 8’, and 10’. Wall #10 had the seventh and fifteenth course of blocks grouted solid with a horizontal reinforcing bar extending the full length of the wall creating horizontal bond beams at 5’ and 10’.

4 Testing Means and Methods

Ten walls built using the ENDURA block system were tested during this research. A summary of the construction process and the testing methods is contained in this section.

4.1 The Construction Process

All block walls to be tested were built on C10x15.3 steel channels. The purpose of this channel was to provide support as the walls were moved into the test frame. Vertical rebars used in the first lift were welded to the channel before placement of the block. This was done to provide stability and strength as the walls were moved. The welded rebars also simulate the fixity of the reinforcement that would extend out of a concrete foundation or footing in a field application. Then a layer of mortar was placed in the steel channel so that the block would have a level bearing surface. The initial setup of the steel channel and welded reinforcement is shown in Figure 4.1.

The ENDURA block system is traditionally a dry stacked surface bonded masonry system. Dry stacked means that no mortar is used between the blocks. As the blocks were stacked, levels were used to check that the wall was plumb. Small metal shims were used where variations in block height caused the block to not be level. A string line was used at each 4’ or 5’ lift to make sure that the wall was not cupping or

Figure 4.1 Rebar and Steel Channel

bowing. The metal shims could also be used to correct cupping problems as the walls were stacked. Screwdrivers were used to pry block courses apart so the metal shims could be inserted. The additional space created by the shims between courses of blocks was usually enough to correct a problem. As each course of block was stacked prefabricated insulation panels were inserted as per the test matrix and grouting patterns.

The walls constructed with the thin set mortar between courses were stacked in a similar manner as the dry stacked walls. The mortar compensated for variations in block height and created a level bearing surface for the block. One of the chief advantages of using mortar between the courses of blocks is that the gaps created by blocks that are not uniform heights are filled. When dry-stacking the blocks, gaps such as the one shown in Figure 4.2 can form in the wall.

Figure 4.2 Gap in Wall Created by Non-Uniform Block Heights

After the first lift of blocks was stacked, grout was poured and the reinforcement was inserted. Each wall had varying grout and reinforcement spacing. The height of the lift and bond beam also varied. At the bond beam height, an entire course of block was solid grouted and one horizontal #4 bar was inserted. The grout used was an ASTMC-387 mix with a compressive strength that was expected to exceed 3500 psi. At walls where the end block of the bond beam course was open, such as with a stretcher block, forms were built and the grout was poured against the forms to create the solid bond beam. When vertical reinforcement was inserted, the builder lifted the rebar up and down several times in order to help the grout consolidate and settle into place. Stingers or other mechanical vibrators are not generally used in construction of ENDURA block walls.

For the purpose of the testing to be performed a grout cap approximately 3 inches thick was poured on top of each wall to insure that there were no high or low spots. After the walls were stacked to full height and all grout and reinforcement had been placed, the surface bond or structure coat was applied to both faces of each wall. The surface bond is a high strength cement mix with glass fiber additives. The surface bond used with the

ENDURA Block system is a proprietary mix that basically consists of sand, cement, lime, and water with glass fibers mixed in. Before the structural coat was applied, the walls were pre-moistened in order to prevent premature drying and crumbling of the surface bond. A four foot long “darby” tool was used to apply the surface bond material to the face of the wall. The darby tool also helped insure that an even coat approximately 1/8” thick was applied. A masonry trowel was used next in order to smooth the finish left by the darby tool. After the surface bond was in place the walls were kept moist for the following 24 hours so that moisture would be available for the surface bond to cure properly. A finished wall after the structural coating had been applied is shown in Figure 4.3.

Figure 4.3 Finished Wall

4.2 Steel Frame

A steel reaction frame was assembled on the strong floor of the BYU structural laboratory. DYWIDAG bars which connected the steel frame to the strong floor were post-tensioned to the strong floor in order to minimize frame movement. The reaction frame and wall #1 are shown in Figure 4.4.

Figure 4.4 Steel Frame and Specimen 1

The frame was estimated to have a vertical testing capacity of 2000 kips. Two hydraulic jacks, each with a capacity of approximately 1000 kips were attached to the horizontal beam of the frame. A W12x79 deep steel beam, with web stiffeners added to prevent web buckling and localized flange failure, was attached to the bottom of the hydraulic jacks. The beam served to spread the load evenly over the length of the walls. A neoprene pad was placed between the steel beam and the top of the wall in order to account for minor irregularities in the top of the wall.

4.3 Data Collection

Three types of data were collected during the testing: applied vertical load, vertical deflection of the specimen, and horizontal deflection of the specimen at mid-height. The applied load was obtained from loads cells that were mounted between the hydraulic jacks and the steel beam. The hydraulic jack and load cell assembly is shown in Figure 4.5.

Figure 4.5 Hydraulic Jacks and Load Cells

The vertical wall deflection was measured using string pots. Three string pots were mounted on a wood frame which was independent from the steel frame and masonry walls. The strings were then attached to the top of the steel beam. During testing, the east side string pot was damaged and was replaced by an LVDT. The assembly used to measure vertical deflection is shown in Figure 4.6.

Figure 4.6 String Pot Attachment

The horizontal deflection at mid-height of the wall was measured using three Linear Variable Displacement Transformers (LVDTs). The LVDTs were also mounted on the wood frame which was independent from the steel frame and masonry walls. The setup of the LVDT's used to measure horizontal deflection is shown in Figure 4.7.

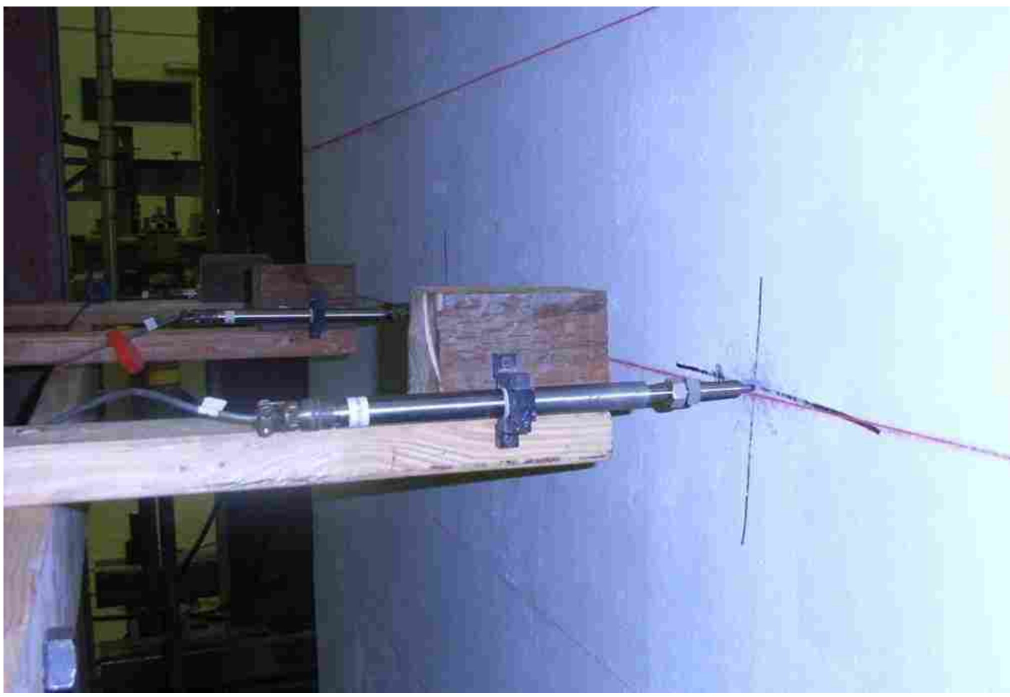

Figure 4.7 LVDT Placement

5 Results

The goal of this testing was to find the axial capacities of walls built using the ENDURA block system. The ultimate capacity of each wall is shown in Figure 5.1.

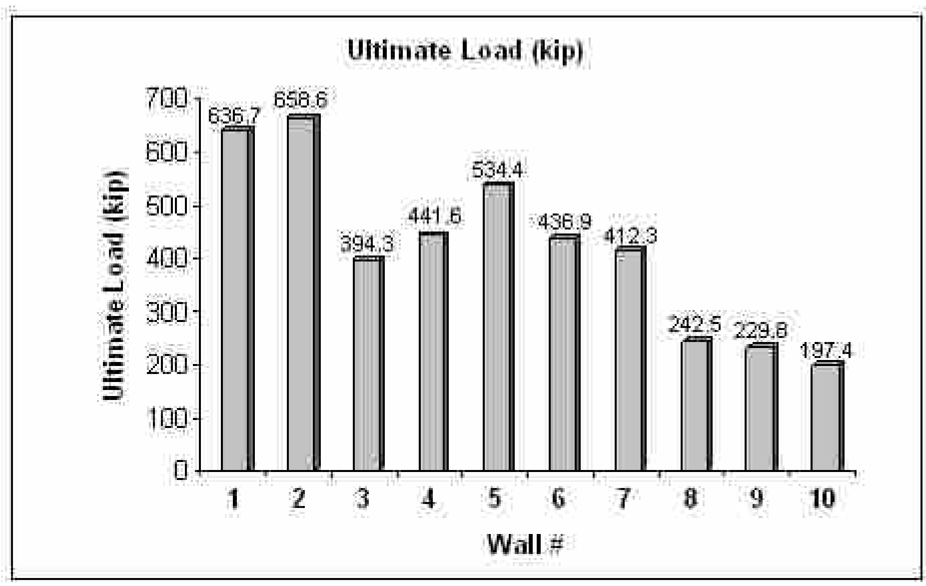

Figure 5.1 Wall Capacity

The ultimate axial capacities of the walls were normalized in two different ways. First, the axial capacities were divided by the structural bearing area. Structural bearing area is being defined as the sum of the bearing area of grout, concrete block, and reinforcement. Second, the axial capacities were normalized by only the grout bearing area, meaning bearing area not including void area, concrete block area, or area of reinforcement. The grout bearing area and structural bearing area for each wall are presented in Table 5-1. These normalized values for axial capacity based on structural area and grout area are shown in Table 5-2.

Table 5-1 Wall Bearing Areas

Wall #	Structural Bearing Area (in^2)	Grout Bearing Area (in^2)
1	756	348
2	685	277
3	586	177
4	586	177
5	664	252
6	531	118
7	531	118
8	481	66
9	481	66
10	448	33

Table 5-2 Normalized Axial Capacities

Wall #	Ultimate Axial Capacity (kip)	Normalized Axial Capacity Based on Structural Area (ksi)	Normalized Axial Capacity Based on Grout Area (ksi)
1	636.7	0.84	1.83
2	658.6	0.96	2.38
3	394.3	0.67	2.22
4	441.6	0.75	2.49
5	534.4	0.80	2.12
6	436.9	0.82	3.70
7	412.3	0.78	3.49
8	242.5	0.50	3.68
9	229.8	0.48	3.49
10	197.4	0.44	5.99

The deflection at the ultimate load was also recorded. Such a measurement is important when determining the overall performance of the walls. The ultimate loads of each of the walls prior to failure and the corresponding deflections are shown in Table 5-3.

Some of the walls tested did not reach their ultimate capacities until after they began to crumble and break apart. In order to protect the instruments used to record horizontal deflection, they were removed when deemed necessary.

Table 5-3 Wall Deflection at Ultimate Load

Wall #	Ultimate Vertical Load (kip)	Vertical Deflection at Ultimate Load (in.)	Horizontal Deflection at Ultimate Load (in.)[a]
1	636.7	0.232	-0.563
2	658.6	0.320	-0.776
3	394.3	0.100	-0.400
4	441.6	0.336	not recorded[b]
5	534.4	0.393	not recorded[b]
6	436.9	0.325	-0.589
7	412.3	0.249	-0.534
8	242.5	0.469	not recorded[b]
9	229.8	0.424	-0.723
10	197.4	0.223	-0.067

[a] (-) refers to deflection away from steel frame
[b] data not recorded in order to protect LVDT's

Load versus deflection graphs also provide a means of evaluating the performance of the walls tested. Plots of load versus deflection for walls using corner blocks only are shown in Figure 5.2 and Figure 5.5. Plots of load versus deflection for walls using corner and stretcher blocks are shown in Figure 5.3 and Figure 5.6. Plots of load vs. deflection for walls built using stretcher blocks only are shown in Figure 5.4 and Figure 5.7.

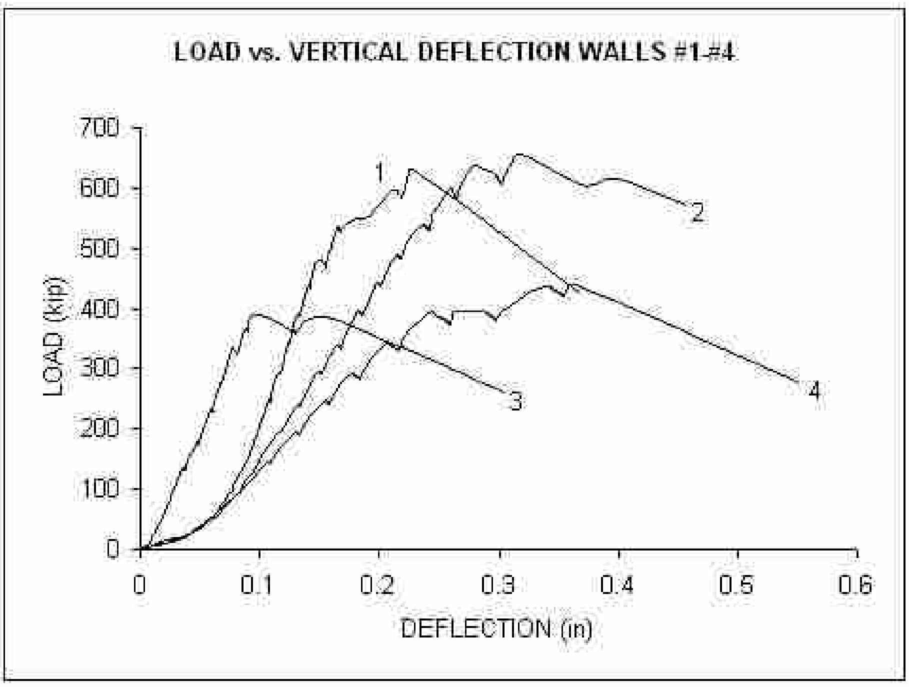

Figure 5.2 Vertical Wall Deflection #1-#4

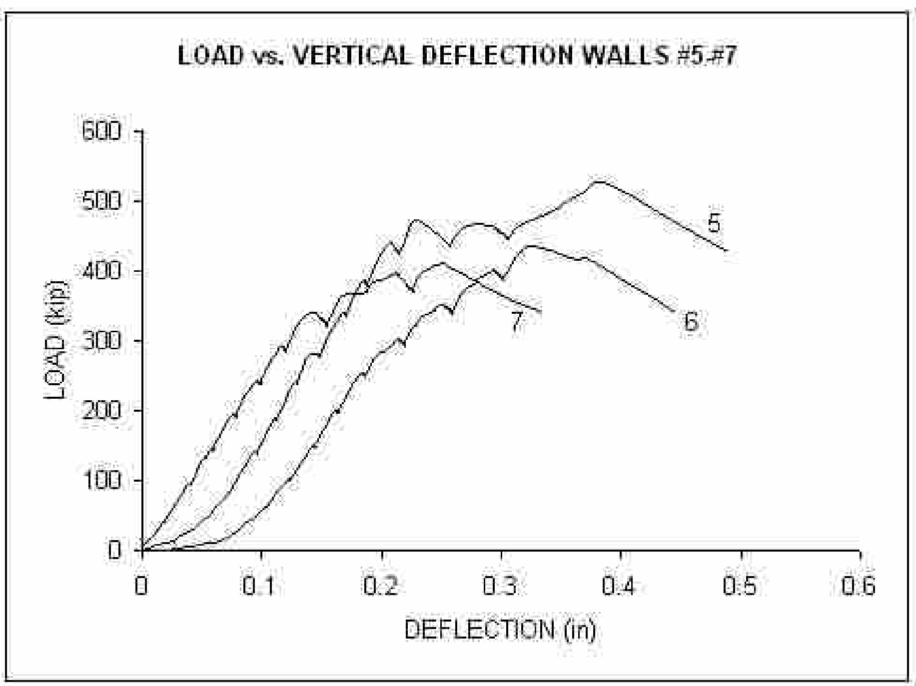

Figure 5.3 Vertical Wall Deflection #5-#7

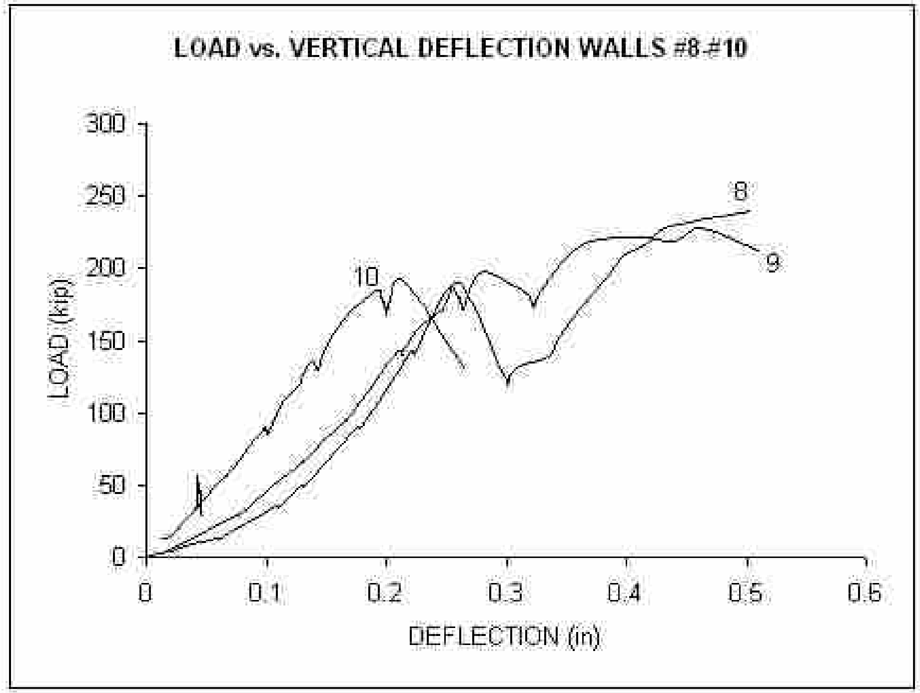

Figure 5.4 Vertical Wall Deflection #8-#10

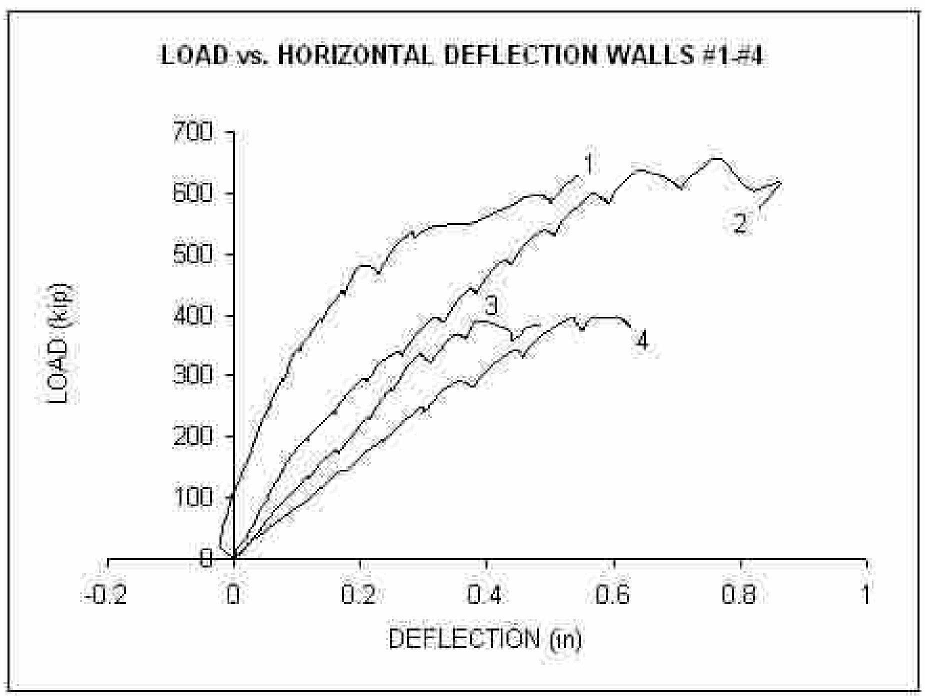

Figure 5.5 Horizontal Wall Deflection #1-#4

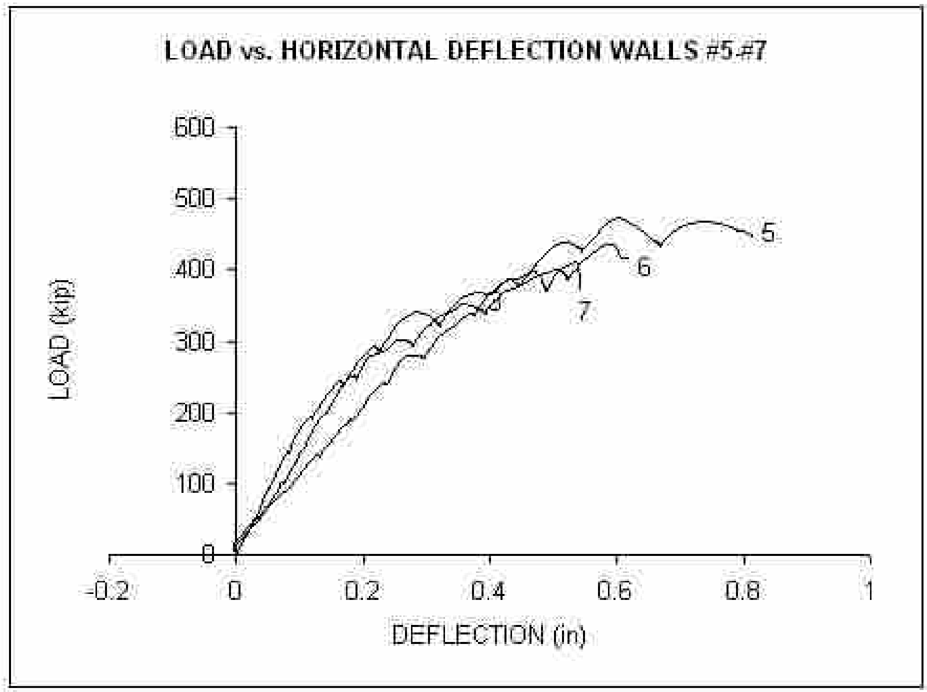

Figure 5.6 Horizontal Wall Deflection #5-#7

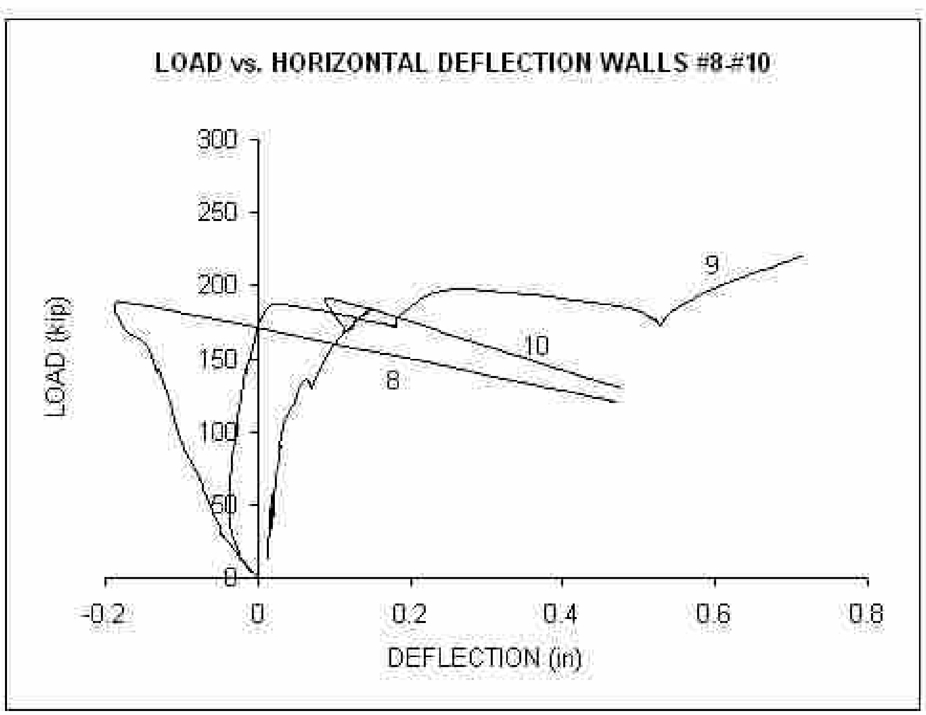

Figure 5.7 Horizontal Wall Deflection #8-#10

6 Observations and Discussion of Results

In addition to quantitative data, qualitative data were also obtained. The ENDURA block walls performed generally as was expected. There were three variables between the walls which were compared. First, the effect of the spacing of grout and reinforcement including whether or not un-reinforced cells were grouted was investigated. Second, the capacities of the walls with eccentrically placed rebar and those with rebar in the center were compared. Third, researchers examined whether the thin set layer of mortar used on some of the walls had a significant effect on the performance of the walls.

6.1 Wall #1

Wall #1 was solid grouted and heavily reinforced. Researchers therefore expected that this wall would have the greatest axial capacity; however this was not the case. At failure, Wall #1 experienced crushing in the top two to three courses of block while the rest of the wall was still entirely intact. This wall had a high ratio of grout strength to overall wall strength. As has been suggested in previous research this high ratio is often accompanied by crushing at the top of the wall. (Uzoegbo, 2007) A photograph of wall #1 at failure is shown in Figure 6.1.

Figure 6.1 Wall #1 at Failure

6.2 Wall #2

Wall #2 performed similarly to wall #1. At failure there was extensive cracking and crushing in the top two to three courses of block. However, some delamination of the structural coating occurred at the bottom course. After testing the coating was removed revealing that the bottom course had also experienced extensive cracking. This suggests that the coating bubbling out was not a controlling factor since both the top and bottom of the walls crushed. Wall #2 resisted the largest vertical load of any of the walls tested.

6.3 Wall #3

As wall #3 was loaded to failure vertical cracking started to develop at approximately 16” o.c. suggesting that cracking was occurring between ungrouted blocks. Reinforcement, which was also placed at 16” o.c., may have played a role in this vertical cracking. When failure occurred, the top west corner of the wall gave way first. There a couple of possible explanations for such occurrence. First, the blocks used to build the wall were manufactured in different plants and the height of the blocks may have varied more than more than typically expected, making it difficult to level the top of the wall. Thus, if the west side of the wall was slightly taller than the east side it could crush first. The second possibility is that the hydraulic jack may have loaded the wall unevenly, applying slightly more load on the west side. There were also significant portions at the lower front and upper rear of the wall where the structural coating bubbled out and separated from the wall as shown in Figure 6.2. The separation of the structural coating was a common occurrence in almost all of the walls as ultimate loads were approached.

Figure 6.2 Bubbling of Structural Coating

6.4 Wall #4

Wall #4 differed from wall #3 in that a thin set mortar was used horizontally between all courses of blocks. The wall achieved a reasonably similar ultimate load to that of wall #3; however it did experience slightly more deflection before it failed. The additional deflection may be a result of the more compressible mortar layer deflecting before failure. After failure some reinforcing bars were exposed and S-shaped rebars were discovered. The final rebar shape suggests that some bars buckled and that those bars contributed to the ultimate capacity of the wall. The behavior of the wall was controlled not only by crushing of the blocks but also by buckling of the reinforcement.

6.5 Wall #5

Wall #5 resisted the third highest load of any of the walls, which may be explained by the fact that this wall along with walls #1 and #2 were the only walls with solid grouted small cells. These three walls are the top 3 performers suggesting that the amount of the grout in the walls has a more significant effect on the axial capacity of the walls than the spacing of reinforcement.

6.6 Wall #6

Wall #6 experienced fairly uniform crushing across the length and height of the wall. The east side of the wall appeared to fail first but the west side was very close to failure as well. At the top half of the wall the blocks split and the face of the block separated and fell forward as shown in Figure 6.3. There also may have been some slight buckling towards the top half of the wall.

Figure 6.3 Wall #6 at Failure

6.7 Wall #7

Wall #7 was the last wall tested. This wall performed quite similarly to wall #6 experiencing fairly uniform crushing throughout the wall and an ultimate load capacity similar to that of wall #6. Such response was expected since the only design difference between wall #6 and wall #7 was that wall #7 had the thin set mortar between all courses of blocks. Figure 6.4 shows wall #7 at failure.

6.8 Wall #8

Wall #8 was built with the reinforcement in the inside or small cell of the stretcher blocks. Such placement of the reinforcement created an inherent eccentricity in the wall because the side with the rebar tends to attract more load even though the load is applied at the center of the wall. Wall #8 buckled as would be expected in an

Figure 6.4 Wall #7 at Failure

eccentrically loaded wall. The first cracks that developed were horizontal cracks directly above and below the bond beam in the center of the wall, which suggests that the wall was experiencing some out of plane bending. At failure the ungrouted face of the block separated completely from the grouted cells and fell forward as shown in Figure 6.5.

6.9 Wall #9

Wall #9 performed similarly to wall #8 as was expected since the only design difference between the two walls was that wall #9 did not have the thin set mortar between courses. At failure the ungrouted face of the block separated and fell forward in a similar manner as that of wall #8. Vertical cracks, separating the grouted and ungrouted faces of the blocks developed just prior to failure as shown in Figure 6.6.

Figure 6.5 Wall #8 at Failure

Figure 6.6 Cracks Separating Grouted and Ungrouted Faces of the Blocks

6.10 Wall #10

Wall #10 was the first wall tested. This wall was the lightest reinforced. The only cells grouted were the cells which contained rebar. As expected, the wall had the smallest load capacity. The wall had reinforcement and grout in the inside or small cells of the stretcher blocks again creating an eccentricity in the wall. As the load was applied, the wall experienced out of plane bending. Wall #10 behaved in a similar manner as walls #8 and #9. These three walls had significantly lower axial capacities than the remaining walls suggesting that the eccentrically placed rebar has a large effect on the axial performance of the walls.

6.11 Influence of Additional Grouting on Axial Capacity

Figure 6.7 shows a comparison of walls in which only the reinforced cells were grouted and those in which the un-reinforced small or large cells were grouted.

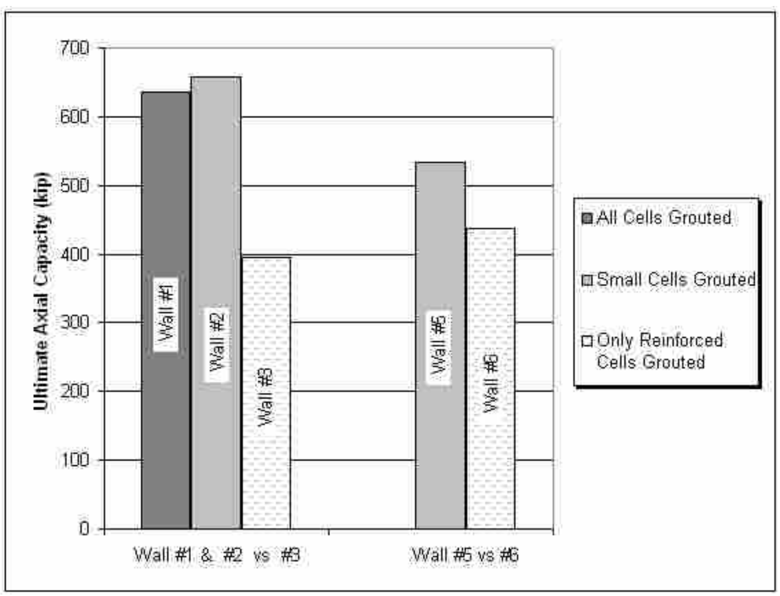

Figure 6.7 Effect of Grout on Axial Capacity

Walls #1, #2, and #3 were designed to be similar with the only variable being the amount of grout used in the wall. Walls #1 and #2 had the small cells of all blocks grouted solid meaning there was significantly more grout in these walls than in wall #3. As is illustrated in Figure 6.7, there is nearly a 40 percent decrease in strength between heavily grouted walls #1 and #2 versus the minimally grouted wall #3. A comparison of wall #5 against wall #6 shows similar results. There is nearly a 20 percent decrease in strength when the small cells are not grouted.

A plot of percent grout area versus axial capacity is shown in Figure 6.8. Percent grout area was calculated as a ratio of the area of grouted cells over the total bearing area of the wall. The linear relationship shown in Figure 6.8 indicates that additional grouting directly influences the axial compressive strength of the walls.

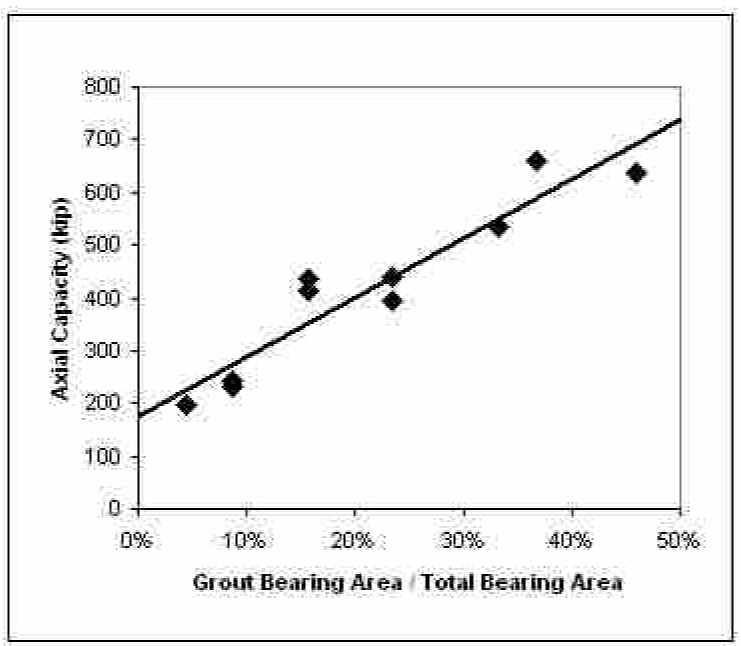

Figure 6.8 Percent Grout Area vs. Axial Capacity

6.12 Influence of Eccentrically Placed Rebar on Axial Capacity

There were two wall pairs in the test matrix that could be used to investigate the effect of eccentrically placed rebars on the axial capacity of the ENDURA block walls. The variables between the wall pairs were the type of block used, the amount of grout used, and the position of grout and reinforcement within the walls. Walls #6 and #7 were constructed using corner and stretcher blocks. The large square corner cells were grouted with the reinforcement centered in the walls. Walls #8 and #9 were constructed using only stretcher blocks. The interior small cells were grouted where reinforced. The stretcher blocks require that rebar and grout be placed eccentrically in the walls.

A comparison of the normalized axial capacities of walls #6 and #7 versus walls #8 and #9 could be used to determine whether or not eccentrically placed grout and rebar have an effect on axial capacity. Figure 6.9 shows the axial capacities normalized based on the grout bearing area. Figure 6.10 shows the axial capacities normalized based on structural bearing area as was defined in Chapter 5.

Figure 6.9 suggest that there is a considerable decrease in axial capacity when the rebar and grout are placed eccentrically in the walls. However, in Figure 6.10, when only the bearing area of grout is considered; the performance of the walls does not appear to vary based on the placement of the reinforcement and grout. Figure 6.10 suggests that grout bearing area may have a more significant effect on capacity than does the location of reinforcement. There appear to be too many variables between these wall pairs to make an accurate assessment of whether eccentricity has an impact on axial capacity.

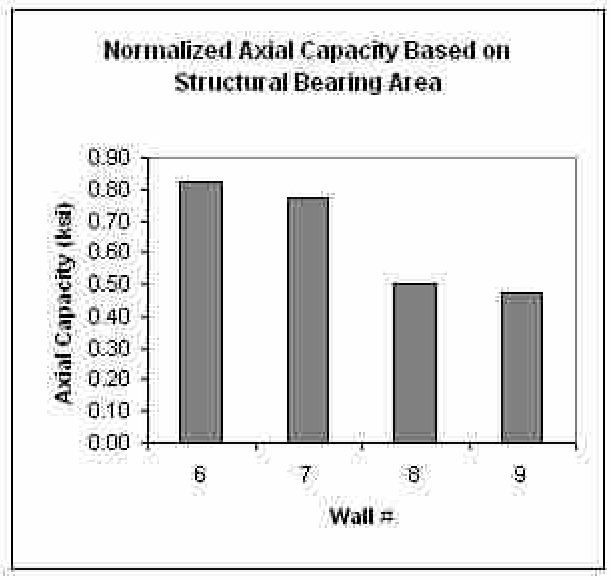

Figure 6.9 Wall #6, #7 vs. #8, #9

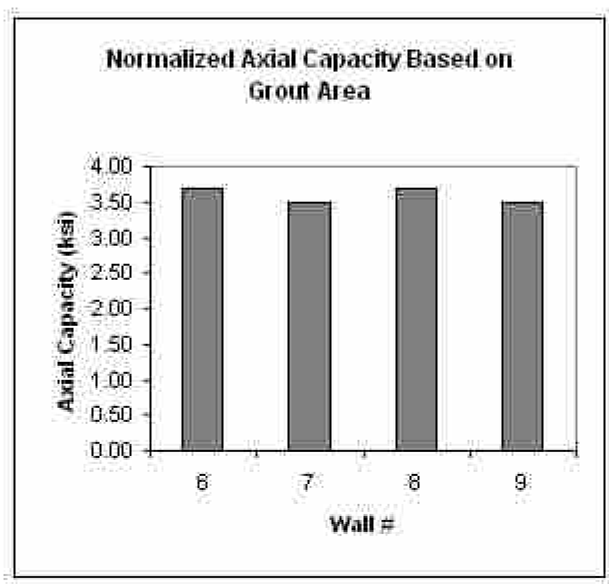

Figure 6.10 Walls #6, #7 vs. #8, #9

6.13 Influence of Thin Set Mortar on Axial Capacity

There were three pairs of walls built for which the only variable was whether or not a layer of thin set mortar was used between courses of blocks. These pairs and their respective axial capacities are shown in Figure 6.11 which shows that the performance doesn't seem to be correlated with the use of mortar. The variation in axial capacities is less than 10 percent when wall pairs constructed with and without thin set mortar are compared. These small differences in axial capacity may be explained by factors other than the mortar such as imperfections generated during construction or the application of load not being exactly uniform from one wall to the next.

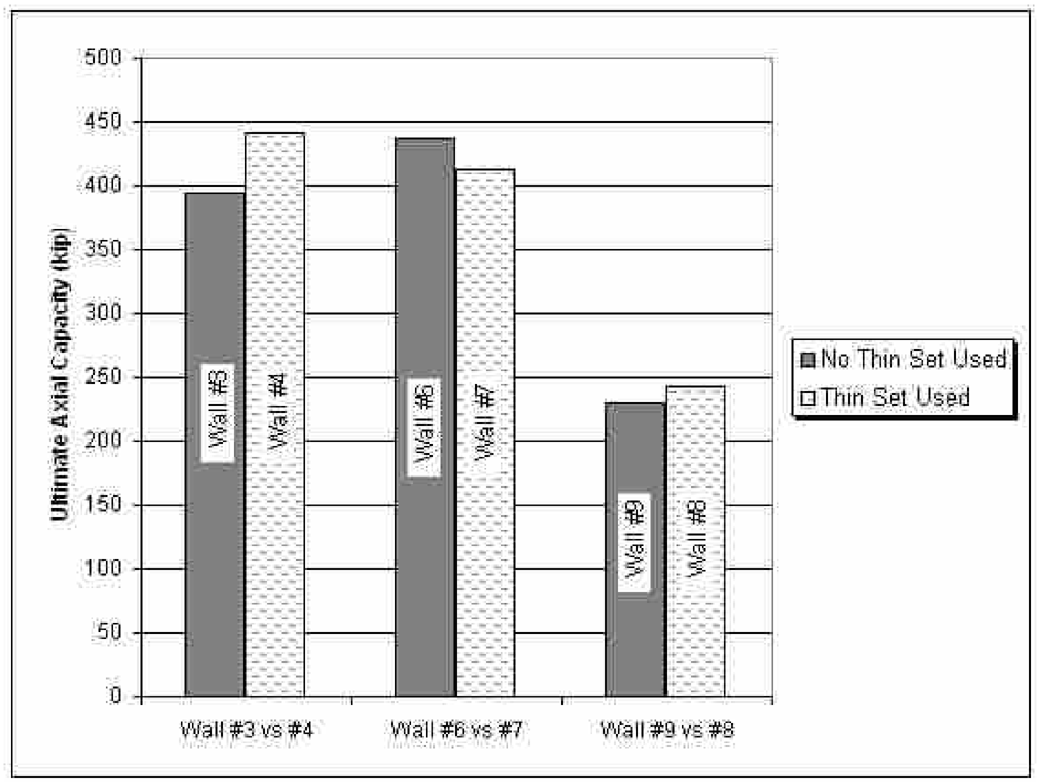

Figure 6.11 Effect of Thin Set Mortar on Axial Capacity

The ENDURA block walls performed similar to other dry stacked masonry systems. The majority of the ENDURA walls experienced large initial deflections with increasing stiffness as the testing proceeded. However, some of the walls did not experience this effect. Current theories suggest two causes of the change in stiffness that occurs in dry-stacked systems. The first is that the roughness at the dry block interface settles under initial loading. The second is that the gaps caused by slight variations in block height must close before the maximum stiffness is achieved. (Marzahn, 1999) (Jaafar, 2006) Through additional testing, the cause of the change in stiffness could be further investigated. Also, it may be possible through extensive testing to develop an analytical model for calculating the axial capacity of ENDURA block walls.

7 Conclusion

After testing only 10 different wall configurations it is apparent that more research is required to develop an analytical procedure to determine the design capacitites of ENDURA walls. However, three significant conclusions can be drawn from the testing performed. First, the amount of grouted cells in the walls directly influences the overall strength of the wall. Second, the ultimate capacity of typical built ENDURA Walls is approximately the same as the ENDURA Walls built with the thin set mortar. Additional small scale testing could be performed in order to better understand the effects of the dry interface between blocks. In a small scale test the contact points between blocks could be more easily isolated. Testing with and without the surface bonding cement could also be performed in order to determine whether this component of the system has an effect on axial capacity. Third, the walls reinforced with rebars placed in the center of the corner blocks appeared to have greater ultimate capacities than the walls with eccentrically placed rebars. However, it is unclear whether this difference is due to reinforcing eccentricity, the amount of grout used in the walls, or the type of block used. More testing should be carried out in order to ascertain the effects of eccentricity on ENDURA block walls.

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Appendix A. Wall #1

The following is a list of notes and pictures taken during the testing of wall #1:

- At 200 kip some measurable horizontal deflection began to occur.
- At 400 kip some popping within the wall could be heard.
- At 550 kip popping sounds continued but there are still no visible cracks.
- At 600 kip the structure coat began to buckle at the top course and horizontal deflection is up to about 0.5 in.
- At 630 kip failure occurred. The top two courses crushed leaving the rest of the wall almost completely intact.

Prior to Testing (wall #1)

Top Course Crushing (wall #1)

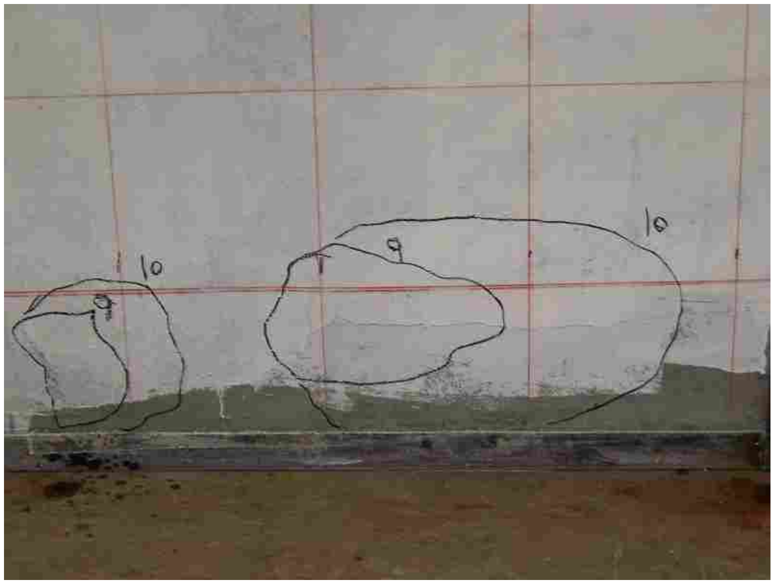
Failure (wall #1)

Close-up of Failed Section (wall #1)

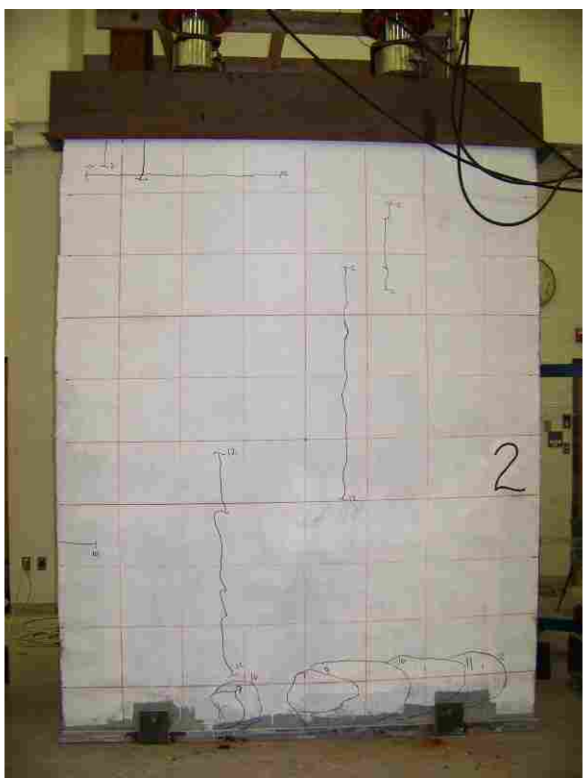
Appendix B. Wall #2

The following is a list of notes and pictures taken during the testing of wall #2:

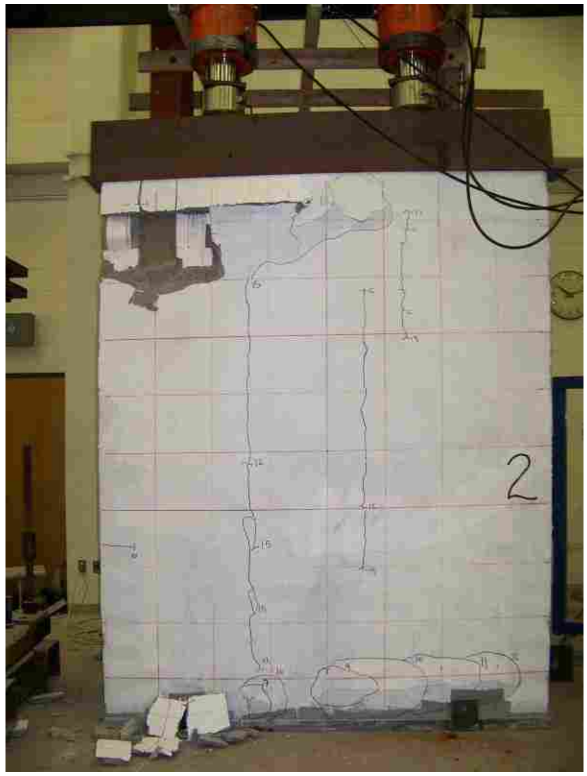
- At 250 kip small cracks develop at top course possibly due to a high spot in wall.
- At350 kip the structure coat at the bottom of the wall buckled. Small vertical cracks began to develop at the middle to top of the wall approximately 2-3' long.
- At 400 kip the structure coat continued to buckle and separate from the block. Vertical cracks continue to propagate.
- At 550 kip long vertical cracks on interior and exterior sides developed. The structure coat continued to buckle at the top and bottom of the wall. A horizontal crack developed between the top two courses.
- At 600 kip the top of the wall began to crush and crumble. More small cracks developed throughout the wall.
- At 650 kip failure occurred. The top two to three courses crushed. The failure occurred almost entirely on the east end of the wall. The bottom course was also extensively cracked.



Buckling of Structure Coat (wall #2)



During Testing (wall #2)



Failure (wall #2)

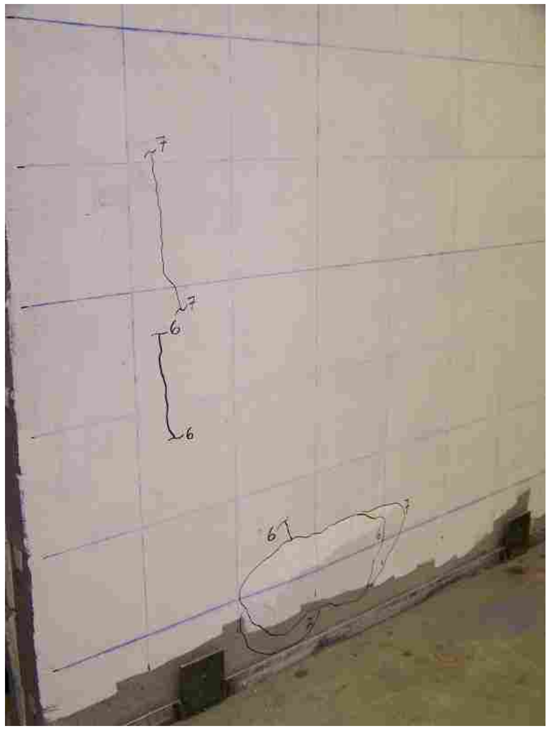
Close-up View of Failed Section (wall #2)

Separation of Structure Coat at Failure (wall #2)

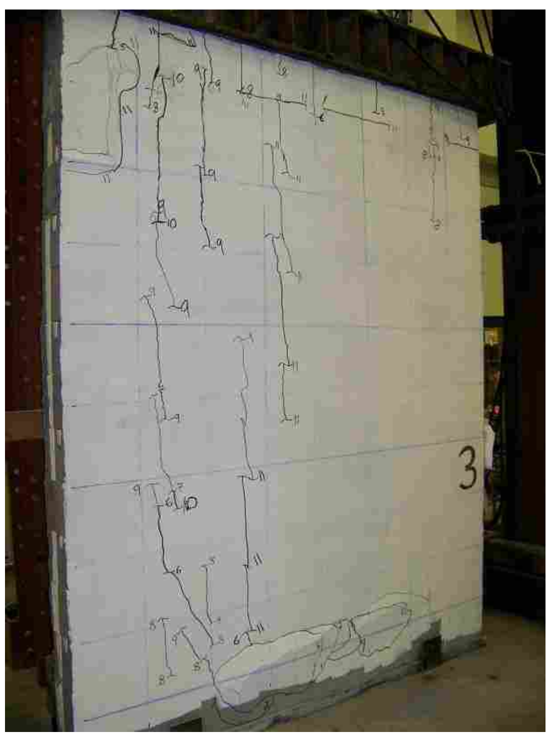
Appendix C. Wall #3

The following is a list of notes and pictures taken during the testing of wall #3:

- At 200 kip small vertical cracks began to develop at the top course of the wall possibly due to high spots in the wall. The cracks were directly below the hydraulic jacks suggesting that load distribution may not be uniform.
- At 250 kip a vertical crack became visible extending approximately 3-4' up from the bottom of the wall.
- At 300 kip Vertical cracking continues at both interior and exterior surfaces. The vertical cracks appeared to be at approximately 16" o.c. This suggests that cracks appeared between blocks and that the blocks may have been moving independently from each other.
- At 350 kip the first horizontal cracking becomes visible at the bottom exterior side of the wall. At the interior side, the structure coat began to buckle towards the bottom of the wall.
- At just below 400 kip there was extensive cracking at both the top and bottom of the wall. There was severe buckling of the structure coat as the wall began to fail. There was extensive cracking, mostly vertical, throughout the wall at failure.

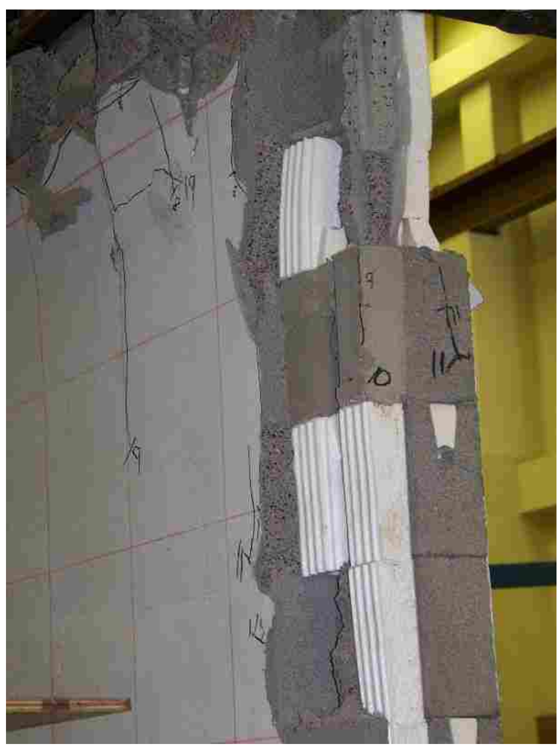


Buckling of Structure Coat at Bottom of Wall (wall #3)



During Testing (wall #3)

Failure (wall #3)



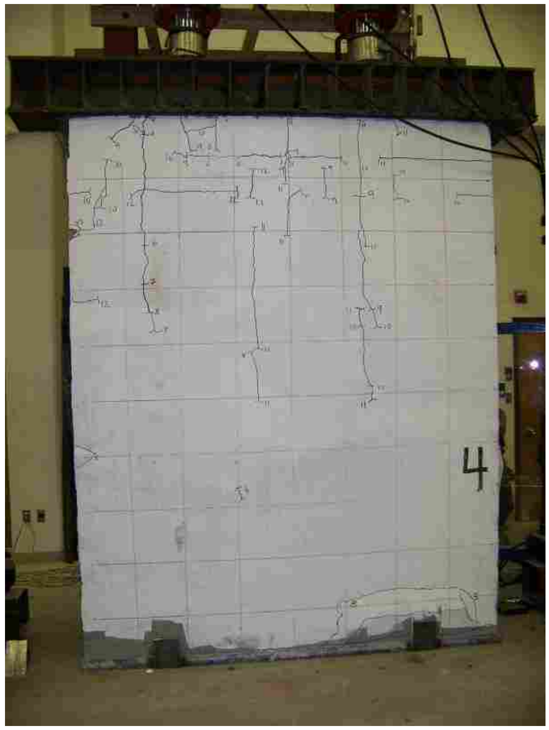
Close-up View of Failed Section (wall #3)

Top Course Crushing (wall #3)

Appendix D. Wall #4

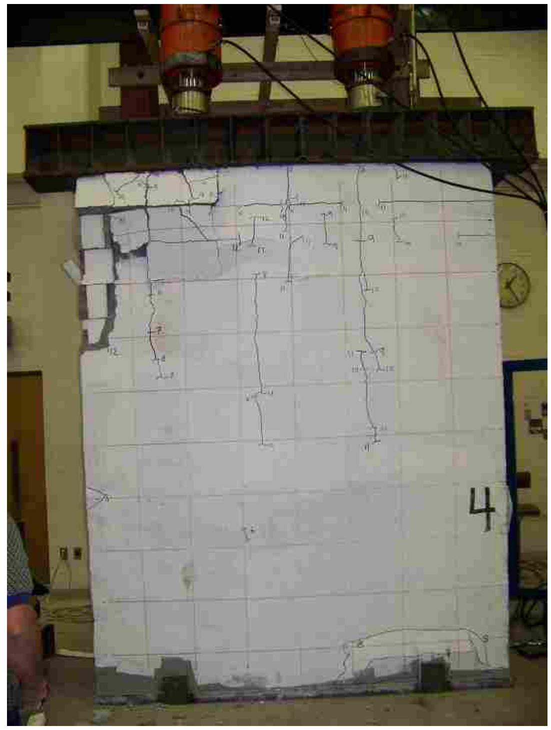
The following is a list of notes and pictures taken during the testing of wall #4:

- At 150 kip the first small cracks at the top course developed.
- At 200 kip the small cracks continue to propagate down from the top of the wall.
- At 300 kip the structure coat at the bottom interior side of the wall buckled. The vertical cracks continue to get longer.
- At 400 kip new long cracks developed on both sides of the wall in the center. Some horizontal cracking began at the top of the wall. Several small vertical cracks became visible at the top of the side of the wall.
- At just before 450 kip failure occurred. Failure was fairly uniform across the length of the wall. Most of the cracking occurred at the top half of the wall.



During Testing (wall #4)

Side View at Failure (wall #4)



Failure (wall #4)

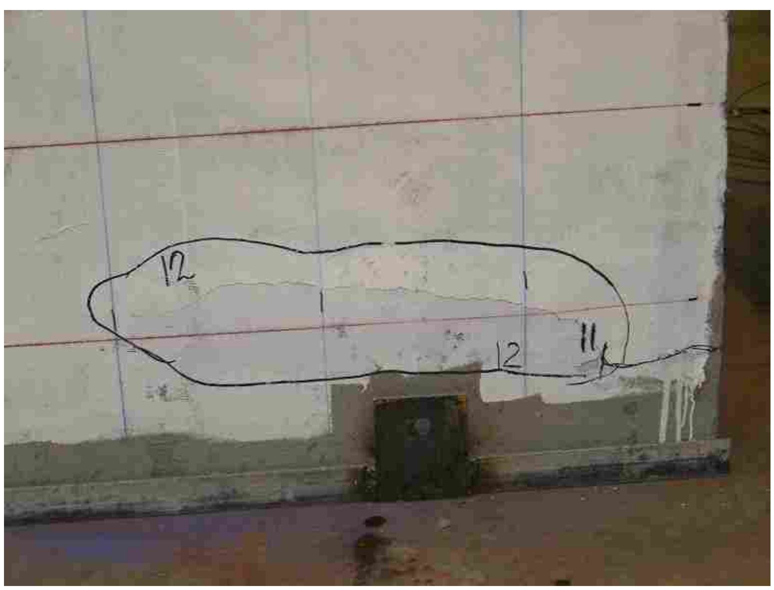
Close-up View of Failed Section (wall #4)

Appendix E. Wall #5

The following is a list of notes and pictures taken during the testing of wall #5:

- At 150 kip the first small horizontal cracks became visible at the center.
- At 300 kip the structure coat on the interior face at the top of the wall buckled.
- At 350 kip small vertical cracks began to appear. Popping sounds could be heard but cracking was still minimal.
- At 450 kip a horizontal crack at the bottom course appeared. Horizontal cracks at the top courses also appeared. The structure coat at the top of the interior face began to buckle.
- At 500 kip the bubbling of the structure coat continued. Long vertical cracks appeared on both sides of the walls. Horizontal cracking continued to propagate.
- At 530 kip failure occurred. The top course of block was completely crushed. There was extensive cracking throughout the wall as well as continued buckling of the structure coat.

Cracking at Side of Wall (wall #5)



Buckling of Structure Coat (wall #5)

Cracking at Side of Wall (wall #5)

Failure (wall #5)

Failure at Top Course of Wall (wall #5)

Appendix F. Wall #6

The following is a list of notes and pictures taken during the testing of wall #6:

- Between 100 kip and 200 kip some small pieces at the top of the wall began to chip off possibly due to a high spot in the wall.
- At 250 kip some vertical cracking appeared on both the interior and exterior sides. The vertical cracks extended from the top of the wall to about mid-height.
- At 300 kip a large bubble in the structure coat appeared at the interior lower east corner of the wall. More vertical cracking and some buckling of the structure coat also appeared at the top of the wall on the exterior side between the hydraulic jacks.
- At 350 kip existing cracks continue to propagate. Crushing and buckling occurred at the top west interior side of the wall. More buckling of the structure coat occurred. The bottom corner block at the west end of the wall has some damage.
- At 400 kip the cracking continued to increase mostly in the top half of the wall.
- The wall failed at 430 kip. Large pieces of the structure coat came free from the wall.

During Testing (wall #6)

Damage at Lower West Corner (wall #6)

Damage at Lower East Corner (wall #6)

Exterior Side of Wall during Testing (wall #6)

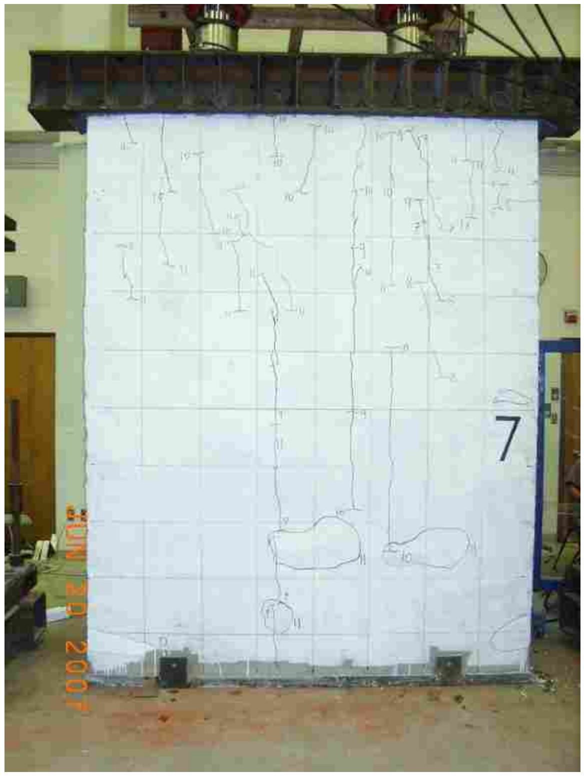
Failure (wall #6)

Close-up of Failed Exterior Section (wall #6)

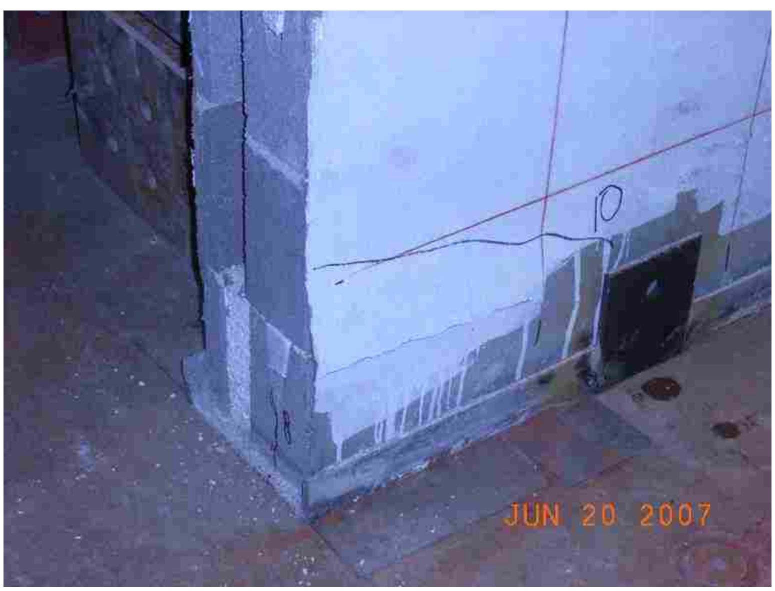
Appendix G. Wall #7

The following is a list of notes and pictures taken during the testing of wall #7:

- At 60 kip some damage occurs at the top of the wall possibly due to a high spot in the wall.
- At 250 kip the first significant cracking develops. This cracking is vertical directly under the east hydraulic jack.
- At 300 kip more vertical cracking appeared. Existing crack continued to propagate and new vertical cracks appeared under the west hydraulic jack. There was also some minor buckling of the structure coat on the exterior side of the wall.
- At 350 kip more vertical cracking appeared mostly at the top half of the wall.
- At 400 kip the top half of the wall on both the interior and exterior sides has experienced severe and extensive cracking.
- At 410 kip failure occurred. The top west corner of the wall failed first however it appeared that the east side was also extensively damaged.



During Testing (wall #7)



Damage at East Corner Possibly Due to Low Spot in Floor (wall #7)

Failure (wall #7)

Failure at Exterior Side of Wall (wall #7)

Close-up of Failure at West Side of Wall (wall #7)

Appendix H. Wall #8

The following is a list of notes and pictures taken during the testing of wall #8:

- At 100 kip the first cracking appeared. Cracking was horizontal at the interior face of the wall at approximately mid-height.
- At 150 kip there was more horizontal cracking at mid-height. At this load the cracking had extended the full width of the wall.
- Between 150 kip and 200 kip extensive cracking developed in the sides of the walls, this was a precursor to the failure mechanism.
- At 240 kip catastrophic failure occurred. Failure was very sudden. The interior face of the blocks completely fell off onto the floor. Some buckling of the wall could be seen. The wall bowed out toward the interior face of the wall.

Cracking in the Sides of the Wall (wall #8)

Horizontal Cracking (wall #8)

Failure (wall #8)

Close-up of East end of Wall at Failure (wall #8)

Buckling of Wall at Failure (wall #8)

Appendix I. Wall #9

The following is a list of notes and pictures taken during the testing of wall #9:

- At 100 kip the first small crack appeared. This crack was a short vertical crack at the bottom west side of the wall.
- At 150 kip some diagonal cracking appeared at the lower east side of the interior face of the wall.
- At 200 kip horizontal cracking developed at the first bond beam level of the wall at the interior face. There was also some buckling of the structure coat at the top section of the exterior face of the wall. There were large vertical cracks in the sides of the wall.
- At 230 kip catastrophic failure occurred. The interior face of almost all of the blocks fell to the floor. The failure was very sudden.

Diagonal Cracking at East Side of Wall (wall #9)

Vertical Cracking in Sides of Wall (wall #9)

Bubbling of Structure Coat (wall #9)

Failure (wall #9)

Buckling of Wall at Failure (wall #9)

Rebar Buckling at Failure (wall #9)

Appendix J. Wall #10

The following is a list of notes and pictures taken during the testing of wall #8:

- At 80 kip some small vertical cracks appeared at the top of the wall just below the hydraulic jacks.
- At 100 kip the structure coat began to bubble at the upper west portion of the interior face of the wall. There was also some bubbling of the structure coat at the low middle portion of the exterior face of the wall.
- At 150 kip more cracking appeared on both sides of the upper portion of the wall. Vertical cracks in the sides of the wall also appeared.
- At 200 kip failure occurred. The interior face of the blocks sheared away from the remainder of the wall. The lower half of the wall buckled out against the LVDTs which were set up to record horizontal deflection. The failure was sudden.

Bubbling of Structure Coat (wall #10)

Vertical Cracks in Sides of Wall (wall #10)

Failure (wall #10)