Evaluation of
Minimum Reinforcing Bar Splice Criteria for Hollow Clay Brick and Hollow Concrete Block Masonry

for

The Council for Masonry Research
Brick Industry Association
National Concrete Masonry Association
Western States Clay Products Association

conducted by:

NATIONAL
CONCRETE MASONRY
ASSOCIATION
RESEARCH AND DEVELOPMENT LABORATORY

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EVALUATION OF MINIMUM REINFORCING BAR SPLICE CRITERIA FOR HOLLOW CLAY BRICK AND HOLLOW CONCRETE BLOCK MASONRY

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CMR TECHNICAL COMMITTEE

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NCMA SPLICE LENGTH RESEARCH TASK GROUP

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WSCPA TECHNICAL COMMITTEE

James E. Amrhein	Jeffrey Elder	John Tawressey	Walter Dickey
Jim Anderegg	R. Dale Yarbrough	Steve Dill	Don Wakefield

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NCMA RESEARCH AND DEVELOPMENT LABORATORY AND ENGINEERING STAFF

Robert D. Thomas, *Director of Engineering and Research*
H. Sali Gunasekara, *Research Engineer*
Phillip J. Samblanet, *Director of Engineering*
Jason J. Thompson, *Structural Engineer*
Donald L. Breeding, *Senior Laboratory Technician*
Douglas H. Ross, *Laboratory Technician*

NATIONAL CONCRETE MASONRY ASSOCIATION
2302 Horse Pen Road
Herndon, Virginia 20171
(703) 713-1900
www.ncma.org

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NOTATIONS, ABBREVIATIONS AND ACRONYMS

Notations

A_b	=	area of reinforcing bar, in^2;
c_{cl}	=	clear cover of reinforcement, in.;
d_b	=	diameter of the reinforcing bar, in.;
F_s	=	allowable stress in reinforcing steel, psi;
f_y	=	tensile yield stress of reinforcement, psi;
f'_m	=	specified compressive strength of masonry at the age of 28 days;
f'_{mt}	=	tested compressive strength of masonry, psi;
K	=	reinforcement cover;
l_d	=	minimum lap splice length of reinforcement;
l_{de}	=	basic development length of reinforcement;
l_r	=	basic development length based on regression analysis, in.;
l_s	=	tested lap length of splice, in.;
T_r	=	predicted load capacity of the splice, lb;
(	=	reinforcement size factor;
N	=	strength reduction factor;
%	=	percent

Abbreviations

Avg	average
CMU	Concrete Masonry Unit
g	gram or grams
in.	inch or inches
in.^{2}	square inches
lb.	pound or pounds
LVDT	Linear Variable Differential Transducer
m	meter or meters
mm	millimeter or millimeters
max	maximum
min	minimum
N.A.	not applicable
N.M.	not measured
No.	number
pcf	pounds per cubic foot
psi	pounds per square inch
typ	typical

Acronyms

ACI	American Concrete Institute
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BIA	Brick Industry Association
CMR	Council for Masonry Research
HBX	Hollow brick type designation from ASTM C 652
NCMA	National Concrete Masonry Association
SW	Hollow brick grade designation from ASTM C 652. Severe Weathering.
TMS	The Masonry Society
WSCPA	Western States Clay Products Association

INCH-POUND TO METRIC CONVERSIONS

This research report uses English units as the standard for the purpose of calculations and generation of plots and figures. Metric units are provided in the text and figures for informational purposes only and may be approximate. Listed below are applicable English to metric conversions for this report.

Quantity	To Convert These ENGLISH UNITS	To These METRIC UNITS	Multiply ENGLISH UNITS by.......
Length	foot (ft)	meter (m)	0.305
	inch (in.)	millimeter (mm)	25.4
Area	square foot (ft^2)	square meter (m^2)	0.0929
	square inch ($in.^2$)	square millimeter (mm^2)	645
Mass	pound (lb)	kilogram (kg)	0.454
Mass Density	pounds/cubic foot (pcf or lb/ft^3)	kilogram/cubic meter (kg/m^3)	16.02
Force	pound (lb)	newton (N)	4.448
Force/Unit Length	pound/foot (lb/ft)	newton/meter (N/m)	14.59
Force/Unit Area	pound/square inch (psi or $lb/in.^2$)	megapascal (MPa)	0.00689
	pound/square foot (psf or lb/ft^2)	megapascal (MPa)	47.88
Temperature	degrees Fahrenheit (°F)	degrees Celsius (°C)	°C = (°F-32)/1.8

EVALUATION OF MINIMUM REINFORCING BAR SPLICE CRITERIA FOR HOLLOW CLAY BRICK AND HOLLOW CONCRETE BLOCK MASONRY

1.0 INTRODUCTION

1.1 Background

Hollow concrete masonry units, clay brick units, mortar, and grout are used in reinforced masonry construction to resist compressive stresses, while the steel reinforcement resists tensile stresses. The grout bonds the reinforcing steel and the masonry units into a structural system so that stresses can be transferred between the materials. Being two very different materials, the reinforcing steel and the masonry perform and deform differently under a given stress condition. As reinforcing steel is stressed, the potential for cracking and spalling of the masonry around the bars increases. Sufficient structural cover is required to prevent premature cracking and spalling of the masonry and to provide sufficient bond to develop the tensile capacity of the reinforcing steel. In areas where continuous lengths of reinforcing bars can not or are not used, bars must be overlapped, welded, or mechanically spliced. Due to their associated cost, welded and mechanical splices are typically cost prohibitive. To provide adequate performance, sufficient lap splice lengths are needed to ensure transfer of stresses between these two reinforcing bars.

1.2 Purpose

The purpose of the test program was to investigate the effect of different combinations of masonry material strength, splice length, structural cover depth, and diameter of reinforcement in both concrete and clay masonry panels. The results of the test program could then be used to re-evaluate current building code provisions for minimum splice criteria.

1.3 Scope of Research

The testing of clay masonry panels using 4-inch and 6-inch hollow clay brick was conducted in a single phase. The testing of concrete masonry panels using 8-inch and 12-inch units was conducted in four phases.

Thirty-three clay masonry panels were constructed using 4-inch and 6-inch hollow brick. (As with the concrete masonry specimens, three panels were constructed for each set, resulting in 11 total sets of clay masonry specimens.) For the nine 4-inch clay brick masonry panels, No. 4 reinforcing bars were placed at cover depths of 1 in. and 1¼ in. Splice lengths of 30 and 36 bar diameters were used for the twenty-four 6-inch clay brick masonry panels, using reinforcing bars of sizes No. 4 through No. 7 placed in the panels at cover depths of 1¼ in., 1½ in., 2 in., and 2.4 in. from the outside face of the panels. Splice lengths for the reinforcing bars in the 6-inch panels were 30, 36, 40 and 48 bar diameters.

For the third phase of research investigating the performance of lap spliced reinforcement in concrete masonry, eighteen panels (six sets of panels were constructed with three specimens per set) were constructed using 8-inch concrete masonry units and a tested concrete masonry strength of approximately 4,000 psi as determined from grouted prism test results. Reinforcing bar sizes of No. 4 through No. 9 were placed at a structural cover depth of 2 inches. The No. 4 bars were spliced 18 in. (36 bar diameters), while all other panels were constructed using splice lengths of 48 bar diameters.

In the second phase of concrete masonry research, all of the thirty panels (again with three panels per set) were constructed using 8-inch concrete masonry units and a tested concrete masonry strength of approximately 1,700 psi as determined from grouted prism test results. Reinforcing bars of sizes ranging from No. 4 through No. 9 were placed at various combinations of structural cover depth (2 in. and 3 in.) and lap splice length (36 bar diameters for No. 4 bar, 48 bar diameters for No. 5 through No. 9 bars, and 64 bar diameters for No. 8 and No. 9 bars).

In the third phase of concrete masonry research, ten sets (each set containing three identical specimens) were constructed. In addition to varying bar diameter, splice length, and cover, the compressive strength of the grout was also varied to document its influence on the performance of these splices. Based on previously completed research, the test matrix of the Phase III investigation was established utilizing lap lengths incorporating No. 7 through No. 9 bars centered in 8-inch concrete masonry units having tested prism strengths ranging from 3,180 psi to 3,290 psi (depending on the compressive strength of the grout) utilizing the following criteria:

1. For each of the bar diameters tested, some specimens were constructed with lap lengths of 48 d_b. These specimens provided a reference point for comparison to previously completed tests in Phases I and II.
2. Lap lengths for some specimens were selected based on the diameter of reinforcement and the targeted compressive strength of the masonry to achieve a capacity of 125% of the yield strength of the reinforcing steel. (This selection was made for reasons outlined in Sections 5 and 6).
3. Lap lengths for other specimens were selected based on the diameter of reinforcement and the targeted compressive strength of the masonry to achieve a capacity 140% that of the yield strength of the reinforcing steel. (This selection was made in an attempt to reach the ultimate capacity of the reinforcing steel).

For the fourth phase of research, specimens were constructed incorporating splices of No. 5 through No. 8 reinforcing bars in 8 and 12-inch concrete masonry. The primary purpose of the Phase IV research was to document the added benefit (or extent of benefit) of the structural cover.

As stated, three panels were constructed for each bar size, splice length, and cover depth tested in each set. Table 1.1 outlines the combinations of bar sizes, splice lengths, and cover depths that were used in each of the masonry panels. Test specimens are identified with the system indicated below. Each panel included two sets of spliced bars to reduce eccentric moments induced when testing a specimen containing a single lap splice. In each splice, one bar protruded from the top and bottom of the panel. Each bar was pulled in direct tension (Figure 1.1) to determine the maximum capacity of the splice. Details of the test results are reported in Appendix 1.

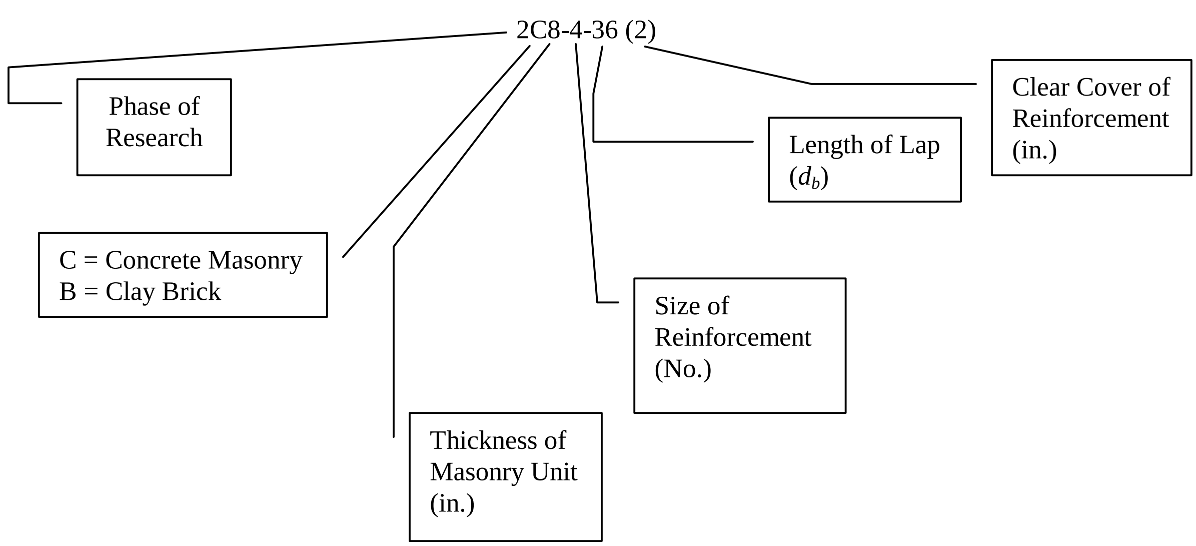

Table 1.1 Test Matrix

Panel Designation	Reinforcing Bar Size	Splice Length	Cover Depth	Panel Dimensions (W × H × L)
(a) 4 inch Clay Brick Masonry Panels				
B4-4-30 (1.3)	No. 4	$30d_b$ = 15 in.	2.5 d_b = 1.3 in.	4 in. × 20 in. × 40 in.
B4-4-36 (1)	No. 4	$36d_b$ = 18 in.	2.0 d_b = 1.0 in.	4 in. × 20 in. × 40 in.
B4-4-36 (1.3)	No. 4	$36d_b$ = 18 in.	2.5 d_b = 1.3 in.	4 in. × 20 in. × 40 in.
(b) 6 inch Clay Brick Masonry Panels				
B6-4-30 (1.5)	No. 4	$30d_b$ = 15 in.	3.0 d_b = 1.5 in.	6 in. × 20 in. × 40 in.
B6-4-36 (1.5)	No. 4	$36d_b$ = 18 in.	3.0 d_b = 1.5 in.	6 in. × 20 in. × 40 in.
B6-5-40 (2)	No. 5	$40d_b$ = 25 in.	3.2 d_b = 2.0 in.	6 in. × 28 in. × 40 in.
B6-5-48 (1.5)	No. 5	$48d_b$ = 30 in.	2.4 d_b = 1.5 in.	6 in. × 32 in. × 40 in.
B6-6-48 (2)	No. 6	$48d_b$ = 36 in.	2.7 d_b = 2.0 in.	6 in. × 40 in. × 40 in.
B6-6-48 (2.4)	No. 6	$48d_b$ = 36 in.	3.2 d_b = 2.4 in.	6 in. × 40 in. × 40 in.
B6-7-48 (2)	No. 7	$48d_b$ = 42 in.	2.3 d_b = 2.0 in.	6 in. × 44 in. × 40 in.
B6-7-48 (2.4)	No. 7	$48d_b$ = 42 in.	2.7 d_b = 2.4 in.	6 in. × 44 in. × 40 in.
(c) Phase I: 8 inch Concrete Block Masonry Panels				
1C8-4-36 (2)	No. 4	$36d_b$ = 18 in.	4.0 d_b = 2.0 in.	8 in. × 24 in. × 40 in.
1C8-5-48 (2)	No. 5	$48d_b$ = 30 in.	3.2 d_b = 2.0 in.	8 in. × 32 in. × 40 in.
1C8-6-48 (2)	No. 6	$48d_b$ = 36 in.	2.7 d_b = 2.0 in.	8 in. × 40 in. × 40 in.
1C8-7-48 (2)	No. 7	$48d_b$ = 42 in.	2.3 d_b = 2.0 in.	8 in. × 48 in. × 40 in.
1C8-8-48 (2)	No. 8	$48d_b$ = 48 in.	2.0 d_b = 2.0 in.	8 in. × 56 in. × 40 in.
1C8-9-48 (2)	No. 9	$48d_b$ = 54 in.	1.8 d_b = 2.0 in.	8 in. × 64 in. × 40 in.
(d) Phase II: 8 inch Concrete Block Masonry Panels				
2C8-4-36 (2)	No. 4	$36d_b$ = 18 in.	4.0 d_b = 2.0 in.	8 in. × 24 in. × 40 in.
2C8-5-48 (2)	No. 5	$48d_b$ = 30 in.	3.2 d_b = 2.0 in.	8 in. × 32 in. × 40 in.
2C8-6-48 (2)	No. 6	$48d_b$ = 36 in.	2.7 d_b = 2.0 in.	8 in. × 40 in. × 40 in.
2C8-6-48 (3)	No. 6	$48d_b$ = 36 in.	4.0 d_b = 3.0 in.	8 in. × 40 in. × 40 in.
2C8-7-48 (2)	No. 7	$48d_b$ = 42 in.	2.3 d_b = 2.0 in.	8 in. × 48 in. × 40 in.
2C8-7-48 (3)	No. 7	$48d_b$ = 42 in.	3.4 d_b = 3.0 in.	8 in. × 48 in. × 40 in.
2C8-8-48 (3)	No. 8	$48d_b$ = 48 in.	3.0 d_b = 3.0 in.	8 in. × 56 in. × 40 in.
2C8-8-64 (2)	No. 8	$64d_b$ = 64 in.	2.0 d_b = 2.0 in.	8 in. × 72 in. × 40 in.
2C8-9-48 (3)	No. 9	$48d_b$ = 54 in.	2.7 d_b = 3.0 in.	8 in. × 56 in. × 40 in.
2C8-9-64 (2)	No. 9	$64d_b$ = 72 in.	1.8 d_b = 2.0 in.	8 in. × 80 in. × 40 in.
(e) Phase III: 8 inch Concrete Block Masonry Panels				
3C8-7-48A (C)	No. 7	$48d_b$ = 42 in.	3.9 d_b = 3.4 in.	8 in. × 48 in. × 40 in.
3C8-7-76 (C)	No. 7	$77d_b$ = 67 in.	3.9 d_b = 3.4 in.	8 in. × 72 in. × 40 in.
3C8-7-85 (C)	No. 7	$85d_b$ = 74 in.	3.9 d_b = 3.4 in.	8 in. × 80 in. × 40 in.
3C8-8-48 (C)	No. 8	$48d_b$ = 48 in.	3.3 d_b = 3.3 in.	8 in. × 56 in. × 40 in.
3C8-8-88 (C)	No. 8	$88d_b$ = 88 in.	3.3 d_b = 3.3 in.	8 in. × 96 in. × 40 in.
3C8-8-98 (C)	No. 8	$98d_b$ = 98 in.	3.3 d_b = 3.3 in.	8 in. × 104 in. × 40 in.
3C8-9-48 (C)	No. 9	$48d_b$ = 54 in.	2.9 d_b = 3.2 in.	8 in. × 64 in. × 40 in.
3C8-9-101 (C)	No. 9	$101d_b$ = 114 in.	2.9 d_b = 3.2 in.	8 in. × 116 in. × 40 in.
3C8-9-113 (C)	No. 9	$113d_b$ = 127 in.	2.9 d_b = 3.2 in.	8 in. × 132 in. × 40 in.
3C8-7-48B (C)	No. 7	$48d_b$ = 42 in.	3.9 d_b = 3.4 in.	8 in. × 48 in. × 40 in.

(f) Phase IV – 12 in. Concrete Masonry Panels				
4C12-5-32 (C-2")	No. 5	$32d_b$ = 20 in.	8.8 d_b = 5.5 in.	12 in. × 24 in. × 40 in.
4C12-5-32 (C)	No. 5	$32d_b$ = 20 in.	8.8 d_b = 5.5 in.	12 in. × 24 in. × 40 in.
4C8-5-32 (C)	No. 5	$32d_b$ = 20 in.	5.6 d_b = 3.5 in.	8 in. × 24 in. × 40 in.
4C12-6-31 (C)	No. 6	$31d_b$ = 23 in.	7.3 d_b = 5.4 in.	12 in. × 32 in. × 40 in.
4C12-6-48 (C)	No. 6	$48d_b$ = 36 in.	7.3 d_b = 5.4 in.	12 in. × 40 in. × 40 in.
4C12-7-37 (C)	No. 7	$37d_b$ = 32 in.	6.1 d_b = 5.4 in.	12 in. × 40 in. × 40 in.
4C12-7-57 (C)	No. 7	$57d_b$ = 50 in.	6.1 d_b = 5.4 in.	12 in. × 56 in. × 40 in.
4C12-7-76 (C)	No. 7	$76d_b$ = 67 in.	6.1 d_b = 5.4 in.	12 in. × 72 in. × 40 in.
4C12-8-45 (C)	No. 8	$45d_b$ = 45 in.	5.3 d_b = 5.3 in.	12 in. × 48 in. × 40 in.
4C12-8-71 (C)	No. 8	$71d_b$ = 71 in.	5.3 d_b = 5.3 in.	12 in. × 72 in. × 40 in.

Ancillary tests performed to document the properties of the materials in the research include:

- mortar tests: consistency by cone penetration, compressive strength
- grout tests: slump, compressive strength
- concrete masonry unit tests: compressive strength, absorption, density
- clay masonry unit tests: compression strength, absorption, saturation coefficient, initial rate of absorption
- clay masonry prism tests: compressive strength of ungrouted prisms
- concrete masonry prism tests: compressive strength of grouted
- reinforcing bar tests: yield strength, breaking strength, total elongation

Detailed information on each of these items can be found in Appendices A.2 through A.6.

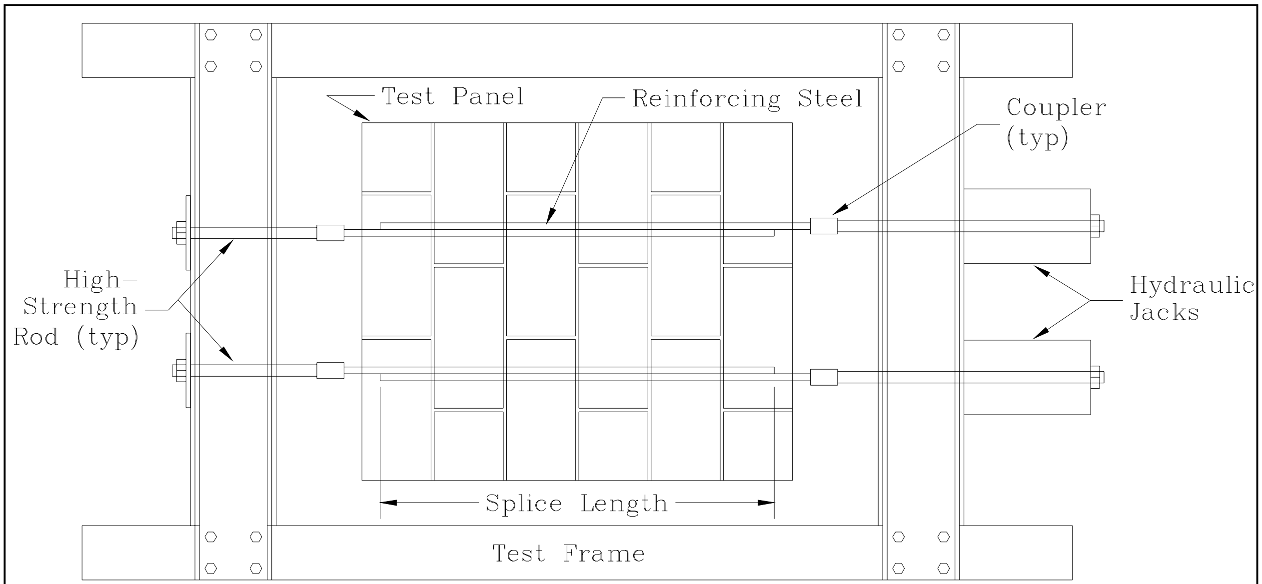

Figure 1.1: Test Configuration Used to Evaluate Splice Capacities

2.0 MATERIALS AND MATERIAL PROPERTIES

2.1 Masonry Units

2.1.1 Hollow Clay Bricks

Hollow clay bricks of 4-inch and 6-inch nominal thickness were used to construct clay brick panels. The specified dimensions of these bricks were 3½ x 3½ x 15½ in. (4 x 4 x 16 in. nominal dimensions) and 5½ x 3½ x 15½ in. (6 x 4 x 16 in. nominal dimensions) respectively. The average measured face shell thickness of the 4-inch brick was 0.81 inches and that of the 6-inch brick was 1.28 inches. The hollow clay bricks complied with the applicable requirements of ASTM C 652, *Standard Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale*[1k], Type HBX and Grade SW.

One set of 4-inch brick and one set of 6-inch brick were tested in compression using half-length specimens. One set each of 4-inch and 6-inch brick were tested for absorption. The brick were tested in accordance with ASTM C 67, *Standard Methods of Sampling and Testing Brick and Structural Clay Tile*[1a]. The test results are summarized in Table 2.1 with a detail of the clay brick shown in Figure 2.1. More detailed results are included in Appendix A.5.

Table 2.1: Average Tested Properties of Hollow Clay Brick

Unit Property	4 x 4 x 16 in. Hollow Clay Brick	6 x 4 x 16 in. Hollow Clay Brick
Net Area Compressive Strength	11,980 psi	17,950 psi
Gross Area Compressive Strength	7,750 psi	11,810 psi
Cold Water Absorption	6.1 %	5.3 %
5-Hour Boil Water Absorption	9.1 %	7.4 %
Saturation Coefficient	0.66	0.72
Initial Rate of Absorption	3.9 g/30 in.^{2}/minute	2.5 g/30 in.^{2}/minute
Dimensions • Width • Height • Length • Minimum Face Shell Thickness • Minimum End Shell Thickness • Minimum Web Thickness	 3.47 in. 3.52 in. 15.57 in. 0.81 in. 0.95 in. 0.65 in.	 5.45 in. 3.54 in. 15.42 in. 1.28 in. 1.35 in. 1.14 in.
Void Area	35.3 %	36.1 %

2.1.2 Concrete Masonry Units

The specified dimensions for the concrete masonry units used in all four phases of research were 7ε x 7ε x 15ε in. (8 x 8 x 16 in. nominal dimensions). In addition, the specified dimensions of the concrete masonry units used in the fourth phase of research were 11ε x 7ε x 15ε in. (12 x 8 x 16 in. nominal dimensions). All of the units used had square corners on both ends and square cores as shown in Figure 2.2. The face shell thickness at the top of the 8-inch units as laid was approximately 1½-inch and decreased to approximately 1¼-inch in a straight taper to the bottom of the unit as laid. Similarly, the face shell thickness at the top of the 12-inch units as laid was approximately 1¾-inch and decreased to approximately 1½-inch in a straight taper to the bottom of the unit as laid.

The concrete masonry units were tested in accordance with ASTM C 140, *Standard Methods for Sampling and Testing Concrete Masonry Units*[1c]. Unit test results are summarized in Table 2.2. The concrete masonry units complied with applicable requirements of ASTM C 90, *Specification for Load-Bearing Concrete Masonry Units*[1b], although drying shrinkage was not evaluated. More detailed results are included in Appendix A.5.

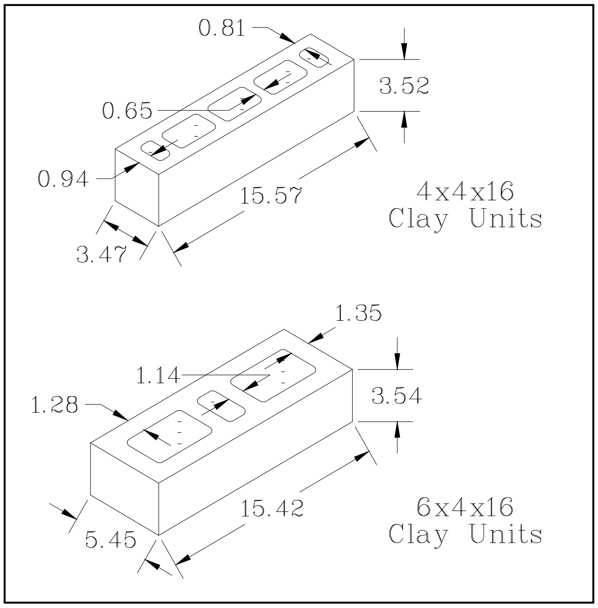

Figure 2.1: Configuration and Dimensions of Hollow Clay Bricks Used in Research
(Dimensions shown are average measured values in inches.)

Table 2.2: Average Tested Properties of CMU

Unit Property	Phase I 8 x 8 x 16 in. Hollow CMU	Phase II 8 x 8 x 16 in. Hollow CMU	Phase III / IV 8 x 8 x 16 in. Hollow CMU	Phase IV 12 x 8 x 16 in. Hollow CMU
Net Area Compressive Strength	3,070 psi	2,170 psi	2,670 psi	2,640 psi
Oven-Dry Density	97.8 pcf	95.7 pcf	102.0 pcf	101.6 pcf
Absorption	11.9 pcf	13.4 pcf	13.7 pcf	11.0 pcf
Dimensions				
• Width	7.66 in.	7.62 in.	7.61 in.	11.62 in.
• Height	7.64 in.	7.65 in.	7.59 in.	7.61 in.
• Length	15.58 in.	15.61 in.	15.57 in.	15.58 in.
• Minimum Face shell Thickness	1.30 in.	1.29 in.	1.28 in.	1.51 in.
• Minimum Web Thickness	1.22 in.	1.21 in.	1.20 in.	1.37 in.
Percent Solid	52.3 %	52.2 %	52.0 %	49.1 %

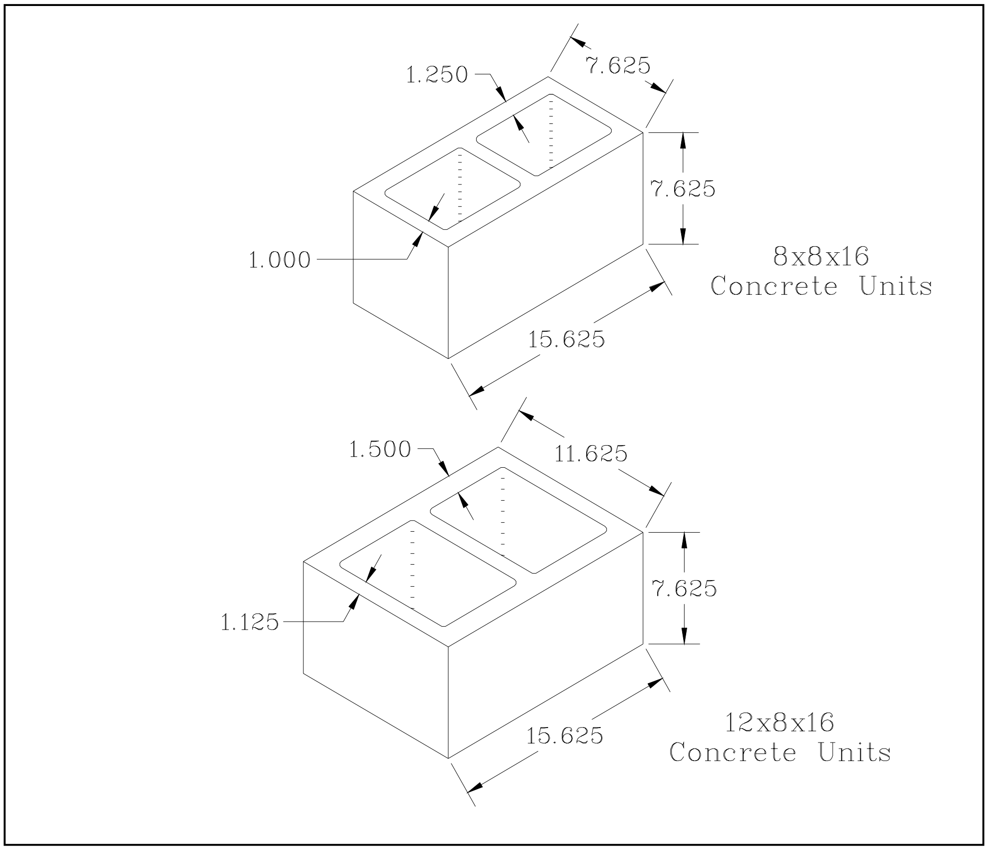

Figure 2.2: Configuration and Dimensions of Concrete Masonry Units Used in Research
(Dimensions shown are specified dimensions for ASTM C 90 CMU in inches. Actual dimensions for CMU used in Phases I through IV are shown in Table 2.2).

2.2 Mortar

2.2.1 Mortar Proportioning and Mixing

Type S portland cement and lime mortar was used to construct all panels and prisms. The mortar was mixed by volume in accordance with ASTM C 270, *Standard Specification for Mortar for Unit Masonry* [1h], as follows:

Parts by Volume	Mortar Constituent
1	Portland Cement
½	Hydrated Lime
4-½	Masonry Sand

Type I portland cement conforming to ASTM C 150, *Standard Specification for Portland Cement* [1f] and Type S hydrated lime conforming to ASTM C 207, *Standard Specification for Hydrated Lime for Masonry Purposes* [1g] were purchased in bags from local suppliers. Masonry sand conforming to ASTM C 144, *Standard Specification for Aggregates for Masonry Mortar* [1e] was purchased in bulk quantities.

Potable water was added to the mortar during the mixing at the discretion of the mason to produce a workable consistency. All mortar was mechanically mixed for 3 to 10 minutes and any mortar unused 1-½ hours after initial mixing was discarded. Retempering of the mortar was permitted once, but stiff or hard mortar due to hydration was not used.

2.2.2 Mortar Testing

Mortar was sampled on each day of specimen construction for the purpose of documenting mortar properties. The following mortar tests were performed:

Mortar Property	Test Method	Reference Standard
Consistency	Cone Penetration	ASTM C 780 [1l]
Compressive Strength	28-day, 2-in. cubes	ASTM C 780 [1l]

Table 2.3 summarizes the average properties of the mortar used to construct the concrete and clay masonry specimens of all phases of research. More detailed test results are included in Appendix A.3.

Table 2.3: Average Tested Properties of Mortar

Construction Mortar	Average Cone Penetration (mm)	Average 28-day Compressive Strength
Clay Masonry Specimens	53	3,130
CMU – Phase I	48	2,380
CMU – Phase II	51	3,200
CMU – Phase III	57	3,110
CMU – Phase IV	53	3,990

2.3 Grout

A local ready-mix concrete/grout supplier furnished the grout used in all phases of research. For each delivery of grout, the slump was measured in accordance with ASTM C 143, *Standard Test Method for Slump of Hydraulic Cement Concrete*[1d]. Water was added to the grout on-site to produce a slump of approximately 10-½ inches prior to placement. It should be noted that the mix designs for the grouts used in the different phases of research were intentionally varied to obtain a desired compressive strength. The fine and coarse aggregates used complied with the requirements of ASTM C 404, *Standard Specification for Aggregates for Masonry Grout*[1i].

The proportions listed in the following Subsections for the concrete masonry specimens were not intended to be in compliance with the proportion specification of ASTM C 476, *Standard Specification for Grout for Unit Masonry*[1j]. Instead, the purpose of the reported proportions was to target a desired 28-day compressive strength. Compliance with ASTM C 476 was documented using the property specifications of that standard. Tested grout strengths, determined in accordance with ASTM C 1019, *Standard Test Method for Sampling and Testing Grout*[1m], were documented for each mix design. More detailed test results are included in Appendix A.4.

2.3.1 Clay Brick Masonry

Fine grout was used for all clay masonry panels. The proportions used were in compliance with the proportion specifications of ASTM C 476 as detailed in Table 2.4a. These proportions produced tested compressive strengths averaging 6,310 psi for the 4-inch brick masonry specimens and 6,170 psi for use in the 6-inch brick masonry specimens.

Table 2.4a: Clay Brick Fine Grout Proportions

Parts by Volume	Grout Constituent
1	Portland Cement
2-½	Fine Aggregate

2.3.2 Phase I Grout

The coarse grout supplied for the Phase I concrete masonry test specimens had the proportions listed in Table 2.4b as reported by the ready-mix supplier. The mix design had a higher aggregate to cement ratio than the C 476 proportion specifications in an attempt to reduce the compressive strength of the grout to be more similar to that of the concrete masonry units. Tested grout strengths, determined in accordance with ASTM C 1019 were still higher than those of the concrete masonry units (3,070 psi) averaging 5,130 psi.

Table 2.4b: Phase I Coarse Grout Proportions

Parts by Volume	Grout Constituent
1	Portland Cement/Slag Cement
2	Coarse Aggregate
2-¾	Fine Aggregate

2.3.3 Phase II Grout

The coarse grout used in the Phase II concrete masonry panels was proportioned to obtain a lower cement ratio with respect to the grout used for the Phase I test specimens, again in an attempt to match the documented unit compressive strength of the units (2,240 psi) used in this phase. Preliminary grout testing was performed to obtain the mix design shown in Table 2.4c. It was noted during grout placement, the proportions used resulted in a rather non-uniform grout consistency. Although the tested grout strengths averaged 2,050 psi (exceeding the minimum required strength in ASTM C 476 of 2,000 psi), material segregation was observed during placement.

Table 2.4c: Phase II Coarse Grout Proportions

Parts by Volume	Grout Constituent
1	Portland Cement/Slag Cement
2-½	Coarse Aggregate
3-½	Fine Aggregate

2.3.4 Phase III Grout

The coarse grout used in the third phase of concrete masonry splice length research consisted of two different mix designs as shown in Tables 2.4d-1 and 2.4d-2. The first mix design targeted a 3,000 psi grout compressive strength and was supplied by a local ready-mix supplier. The second mix design targeted a 5,000 psi grout compressive strength and was proportioned by the laboratory. The measured compressive strength for the 3,000 psi target strength was determined to be 4,680 psi. The measured compressive strength for the 5,000 psi target strength grout was determined to be 5,530 psi. With closer inspection of the two respective mix designs, one would expect the measured compressive strengths to vary by more than 810 psi due to the significantly different proportions. Because the mix design detailed in Table 2.4d-2 was manufactured in the laboratory with strict adherence to the intended proportions, it was generally felt that the grout proportions supplied by the ready-mix supplier may not have been consistent with those reported in Table 2.4d-1.

Table 2.4d-1: Phase III Coarse Grout Proportions – 3,000 psi Target

Parts by Volume	Grout Constituent
1	Portland Cement/Slag Cement
2-¼	Coarse Aggregate
3-½	Fine Aggregate

Table 2.4d-2: Phase III Coarse Grout Proportions – 5,000 psi Target

Parts by Volume	Grout Constituent
1	Portland Cement/Slag Cement
1-½	Coarse Aggregate
2-¾	Fine Aggregate

2.3.5 Phase IV Grout

The coarse grout used in the fourth phase of concrete masonry splice length research consisted of the proportions detailed in Table 2.4e as reported by the ready-mix supplier. The target compressive strength was 3,000 psi. The measured compressive strength was determined to be 3,560 psi.

Table 2.4e: Phase IV Coarse Grout Proportions

Parts by Volume	Grout Constituent
1	Portland Cement/Slag Cement
2-½	Coarse Aggregate
3-α	Fine Aggregate

2.4 Construction and Curing of Compression Prisms

At the time the panels were constructed for each of the four phases, three masonry prisms were constructed in accordance with ASTM C 1314, *Constructing and Testing Masonry Prisms Used to Determine Compliance with Specified Compressive Strength of Masonry*[1n].

For the brick masonry specimens, one set of three prisms was constructed for each of the two unit sizes used in the research. Both sets were constructed using three full-size units laid stack bond in a full mortar bed. The resulting nominal prism dimensions for the 4-inch units were 4 x 12 x 16 in., and for the 6-inch units were 6 x 12 x 16 in. These prisms were not grouted.

For the concrete masonry prisms, two half length units (saw-cut from full size units) were laid in a stack bond configuration in a full mortar bed using the same mortar used in the panel construction. The resulting outside dimensions of these prisms were approximately 8 inches (or 12 inches depending on the respective size of the units) wide by 16 inches high by 8 inches long. The concrete masonry prisms were filled in one lift with grout from the same truck used for grouting the panels. The grout was consolidated using the same consolidation and reconsolidation procedures as was used for the corresponding panels. Enough grout was added to bring the grout level with the top surface of the prism.

Immediately following grouting procedures, the prisms were sealed within plastic bags. Approximately 48 hours prior to testing at an age of 28 days, the prisms were removed from the plastic bags and allowed to equalize with the temperature and moisture conditions of the laboratory environment. Average prism strengths are reported in Table 2.5. More detailed results are included in Appendix A.2.

Table 2.5: Compressive Strength of Masonry Prisms

Prisms	Average Compressive Strength (psi)
4 in. Brick Masonry - Ungrouted	4,310
6 in. Brick Masonry - Ungrouted	6,410
8 in. Concrete Masonry - Grouted-Phase I	4,070
8 in. Concrete Masonry - Grouted-Phase II	1,700
8 in. Concrete Masonry - Grouted-Phase III (3,000 psi target strength)	3,180
8 in. Concrete Masonry - Grouted-Phase III (5,000 psi target strength)	3,290
8 in. Concrete Masonry - Grouted-Phase IV	2,850
12 in. Concrete Masonry - Grouted-Phase IV	2,180

Although grouted brick masonry prisms were not tested, the following masonry strengths are used in this report for evaluation and discussion purposes based on the measured compressive strengths of ungrouted masonry prisms and grout compression specimens:

4-inch Brick Masonry: 4,500 psi
6-inch Brick Masonry: 6,300 psi

2.5 Reinforcing Steel

Grade 60 deformed reinforcing bars were used in all phases of this research. Stock bar lengths were delivered to the laboratory and were cut to specified lengths in the laboratory. Upset threads were milled onto one end of each of the reinforcing bars to provide a means of applying tensile loads during panel testing. The intent of the upset threads was to prevent the possibility of a weakened cross section at this location.

Tension tests were performed on bars from the same production heat in conformance to ASTM A 370, *Test Methods and Definitions for Mechanical Testing of Steel Products*[1o] and were verified to meet the requirements with ASTM

A 615, *Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*[1p]. A summary of these test results is included in Table 2.6. Each value represents a set of tests on three bars and calculated strengths are based on the nominal bar area. More detailed results are included in Appendix A.6.

Table 2.6: Tension Tests on Reinforcing Bars Used in Specimen Construction[1]

Reinforcing Bar Size	**Average Yield Strength (psi)**	**Average Breaking Strength (psi)**	**Average Total Elongation**
Bars used for Phase I Concrete Masonry Panels			
No. 4 Bars	73,830	98,830	17.7%
No. 5 Bars	68,000	97,670	17.9%
No. 6 Bars	72,000	96,830	17.7%
No. 7 Bars	71,830	101,170	18.4%
No. 8 Bars	70,170	98,170	18.4%
No. 9 Bars	70,000	98,000	18.5%
Bars used for Phase II Concrete Masonry Panels and Clay Brick Masonry Panels			
No. 4 Bars	67,000	94,000	17.5%
No. 5 Bars	69,780	105,160	16.6%
No. 6 Bars	64,620	102,120	15.6%
No. 7 Bars	72,440	100,500	18.0%
No. 8 Bars	65,860	97,600	19.3%
No. 9 Bars	64,970	97,900	19.1%
Bars used for Phase III Concrete Masonry Panels			
No. 7 Bars	68,670	96,110	18.2%
No. 8 Bars	67,930	94,680	18.7%
No. 9 Bars	67,800	96,070	18.0%
Bars used for Phase IV Concrete Masonry Panels			
No. 5 Bars	74,620	103,340	17.2%
No. 6 Bars	64,850	92,420	16.5%
No. 7 Bars	67,060	102,450	17.8%
No. 8 Bars	64,900	95,360	15.7%

[1]Material testing of the reinforcing steel was conducted by an independent laboratory.

3.0 CONSTRUCTION AND CURING OF TEST SPECIMENS

3.1 Workmanship

Two journeyman masons with over 30 years combined experience in masonry construction constructed the test specimens using good construction techniques in accordance with ACI 530.1/ASCE 6/TMS 602[12].

3.2 Masonry Panel Construction

The clay brick panels were constructed on tar paper overlaying a smooth level concrete floor. Conversely, the concrete masonry panels were constructed on a loose course of concrete masonry units for easier removal of any mortar that protruded into the cores as the construction of the panels progressed. No initial leveling bed of mortar was used for either the clay or concrete panels. Panels were laid in a running bond configuration using face shell bedding except at the ends of the panels where the end web was mortared. Mortar joint thickness was δ ± χ inch. Mortar joints on the faces of the panels were struck and tooled with a concave jointer after they became thumbprint hard. Joints at the ends of the panel were struck flush. When feasible, mortar that protruded into the cores of the units more than ½-inch was removed with a hand trowel as the construction of the panels progressed. Hardened mortar that protruded more than ½-inch into the grout space was removed by rodding prior to placing the grout.

Each set of panels was built to different heights based on the amount of lap length required for the spliced reinforcing bars. The panels were built high enough to completely embed the lap lengths while still providing the minimum embedment permitted for the set's modular dimensions. Due to fixed coursing heights, the distance between the end of the splice to the ends of the panel varied between specimen sets. This distance ranged from 1-1/2 inches to 2-1/2 inches in the brick masonry panels and from 1 inch to 5 inches in the concrete masonry panels.

Approximately one week after construction, each panel was carefully raised from the bottom using an overhead crane. The bottom course was cleared of any debris or obstructions that may have collected during the construction process. Each panel was then placed on a raised plywood platform that served as a mold for the grout. Two circular holes, large enough to accommodate the respective reinforcing bars used in each test panel and small enough to hold the bars securely in position and to limit grout and water seepage around the bars, were drilled in the plywood platform.

Reinforcing steel was secured in position at the top of the panel using either small rigid plastic templates or 9-gage steel wire. The purpose of these anchors was to maintain the specified cover depth and splice location within the panel. Reinforcing bars were lapped using contact splices and loosely tied at two or three locations (depending on the length of lap and the diameter of the reinforcement) along the length of the lap with wire prior to their placement in the panel. Typical cross sections of brick masonry panels and concrete masonry panels are shown in Figures 3.1-a and 3.1-b.

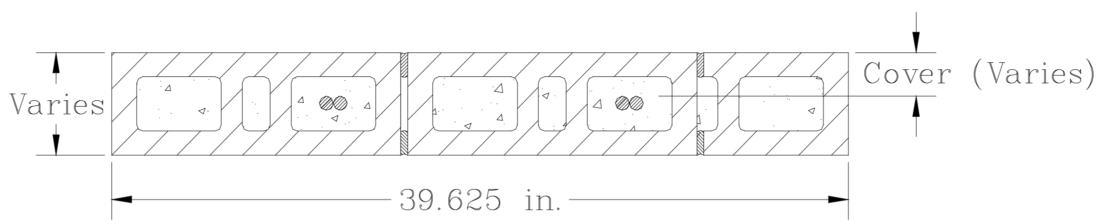

Figure 3.1-a: Typical Clay Brick Masonry Panel Cross-Section

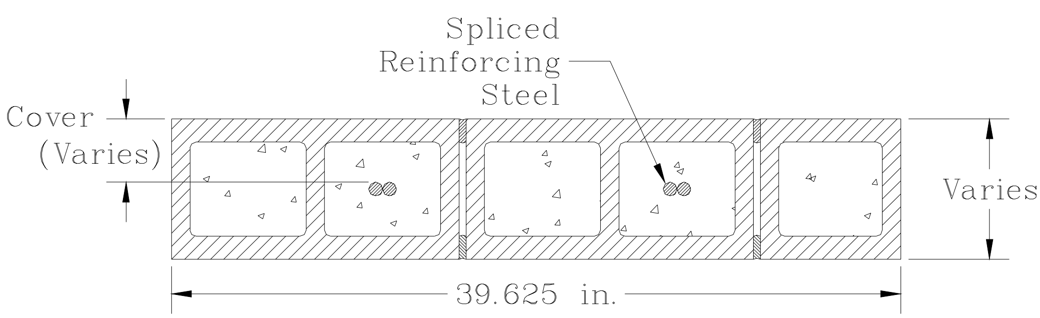

Figure 3.1-b: Typical Concrete Masonry Panel Cross-Section

For the clay brick masonry panels, the grout was placed into the cores of the test panels by hand. For the concrete masonry panels, grout was placed by two different methods. For the concrete test specimens of Phases I and II, the grout was discharged from the ready-mix truck directly into a three-quarter yard bucket which was hoisted to the tops of the panels using an overhead crane. A chute was used to place the grout into each core of each panel from the concrete bucket. For the concrete test specimens of Phases III and IV, the grout was placed using a grout pump.

The grout was placed in a single lift for all panels of all phases of research. The grout in each core was consolidated and reconsolidated using a ¾-inch diameter vibrator. Reconsolidation times averaged 10 minutes after initial consolidation for the concrete masonry panels, 15 minutes after for the 4-inch brick masonry panels, and 45 minutes after for the 6-inch brick masonry panels. After reconsolidation, additional grout was added as necessary to compensate for the water lost from the grout. The panels cured in laboratory air until time of testing.

4.0 TEST PROCEDURES

A test frame was constructed consisting of four structural steel members bolted together to form a rectangular perimeter around the test panel (refer to Figure 1.1). To alleviate any need for bracing or shoring of the testing equipment or test specimens, the structural frame was laid horizontally on the laboratory floor. Prior to testing, the height of the test frame was adjusted to accommodate the specific height of the reinforcing bars protruding from each panel.

Next, each panel was individually rotated into a horizontal position within the testing frame to rest flat on steel pipes. The pipes were used to support the panel in position while limiting any frictional resistance that would prevent panel movement during testing. The face of the panel having the least depth of cover to the reinforcement was facing up during testing. (For those test specimens where the spliced reinforcement was centered within the cells, the choice of the exposed testing face was arbitrary.) A typical specimen is shown in Figure 4.1.

Once a panel was positioned into the frame, high-strength steel couplers were attached to each of the four steel reinforcing bars protruding from the panel. To the opposite end of each coupler, another reinforcing bar (threaded on both ends) having a diameter greater than those used in the panel was attached. At the top of the panels (as constructed), these connector bars extended through holes in the steel frame and through the center bore-hole rams bearing against the steel frame. Steel washers and threaded nuts were used on the threaded ends of the bars so that these bars could be pulled in direct tension. Similarly, at the bottom of the panel (as constructed), the connector bars extended through the steel frame. Steel washers and threaded nuts were fastened to the ends of these threaded bars as well.

A hydraulic pump was used to supply pressure to the rams. The rams were connected in parallel using a “T” joint for the hydraulic line coming from the pump. This method was used to ensure that both rams would be under the same hydraulic pressure and therefore exert as near as possible the same force to each of the two bars to which they were connected. Based on an evaluation of the forces acting on the test panel, the forces in the bars at the bottom of the panel were the same as the forces in the bars at the top of the panel.

For the specimens tested in Phases I and II, the forces exerted by the rams were measured using 100 kip capacity load cells. For the brick masonry panels and the Phase II concrete masonry panels, linear variable differential transducers (LVDTs) were used to measure the displacements between gage points A and B of the splice bars as shown in Figure 4.2. The load cells and the LVDTs were connected to a data acquisition system which recorded the applied load and the splice displacements throughout the testing of a panel. For the concrete masonry test specimens of Phases III and IV, the applied load was determined by calibration to the applied hydraulic pressure. For the panels of Phases III and IV, no displacement measurements were obtained.

During testing, load was applied to each specimen at a constant rate until failure occurred (defined by rupture of the reinforcing steel, pullout of the reinforcing steel, or the longitudinal splitting of the masonry), at which time the test was stopped. Panel distress, in the form of cracking of bed joints and masonry units, was continuously recorded during each test.

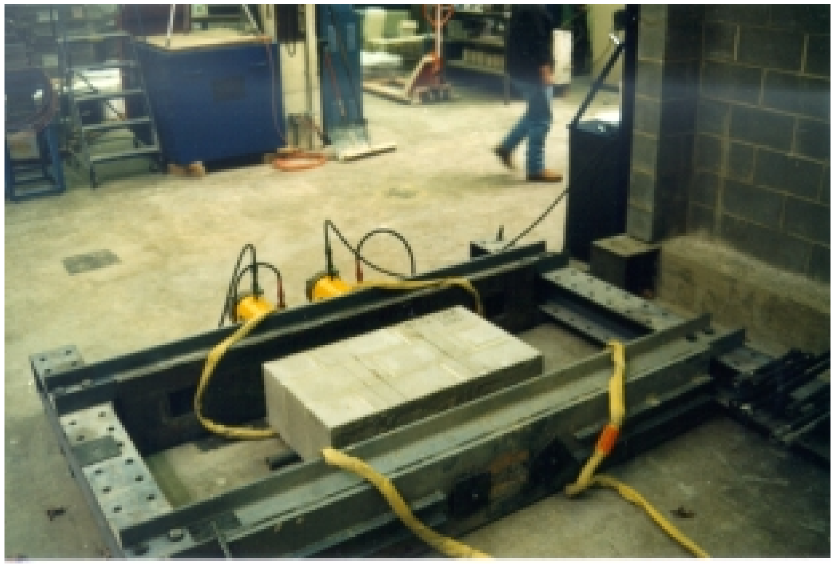

Figure 4.1: Testing Configuration

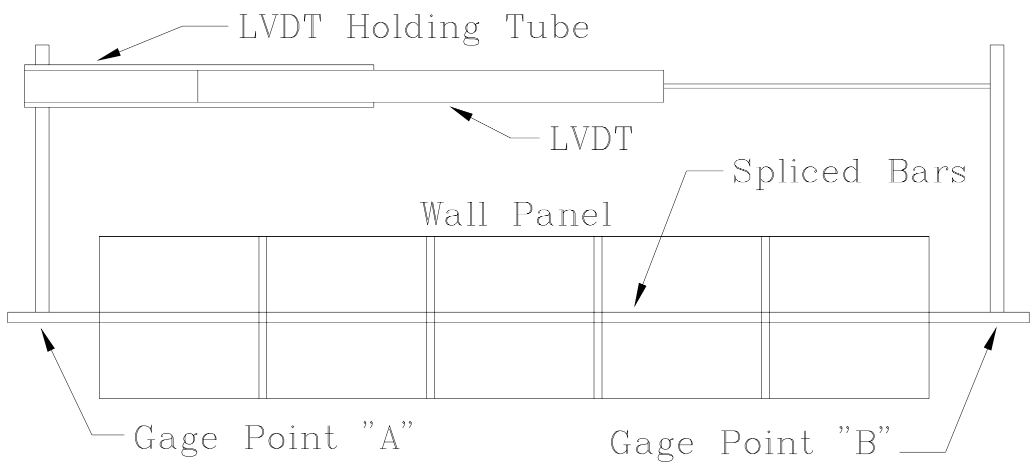

Figure 4.2 – LVDT Placement

5.0 TEST RESULTS AND OBSERVATIONS

5.1 Summary of Test Results

Table 5.1 provides a summary of the results of the testing program. Plots showing the relationship between tensile load and splice displacement for the 4-inch and 6-inch clay brick test specimens and the concrete masonry specimens of Phase II are shown in Figures 5.1, 5.2, and 5.3, respectively. Each of these figures includes the plots of only one panel from the set. The panel selected for plotting was that which exhibited the median maximum splice displacement for the set of panels. Plots and test results for individual panels are located in Appendix 1.

Table 5.1: Summary of Test Results

Panel Designation (set of three specimens)	Length of Lap Tested (in.)	Average Maximum Load (lb.)	Average Maximum Stress in Reinforcement At Failure (psi)	Ratio of Maximum Stress in Reinforcement to Specified Yield Stress	Average Maximum Splice Displacement[1] (in.)	Estimated Average Splice Displacement[1] at Yield (in.)
4-inch Clay Masonry Panels						
B4-4-30 (1.3)	15	15,184	75,922	1.27	0.202	0.012
B4-4-36 (1.0)	18	15,469	77,345	1.29	0.236	0.034
B4-4-36 (1.3)	18	16,646	83,228	1.39	0.343	0.018
6-inch Clay Masonry Panels						
B6-4-30 (1.5)	15	16,369	81,843	1.36	0.300	0.015
B6-4-36 (1.5)	18	18,225	91,123	1.52	0.539	0.017
B6-5-40 (2)	25	25,777	83,151	1.39	0.465	0.036
B6-5-48 (1.5)	30	24,278	78,316	1.31	0.310	0.027
B6-6-48 (2)	36	38,044	86,463	1.44	0.648	0.059
B6-6-48 (2.4)	36	37,006	84,105	1.40	0.716	0.066
B6-7-48 (2)	42	45,690	76,150	1.27	0.248	0.063
B6-7-48 (2.4)	42	45,819	76,364	1.27	0.316	0.061
Phase I – 8-inch Concrete Masonry Panels						
1C8-4-36 (2)	18	20,124	100,620	1.68	N.M.	N.M.
1C8-5-48 (2)	30	28,843	93,041	1.55	N.M.	N.M.
1C8-6-48 (2)	36	38,448	87,383	1.46	N.M.	N.M.
1C8-7-48 (2)	42	48,303	80,504	1.34	N.M.	N.M.
1C8-8-48 (2)	48	58,103	73,549	1.23	N.M.	N.M.
1C8-9-48 (2)	54	65,570	65,570	1.09	N.M.	N.M.
Phase II – 8-inch Concrete Masonry Panels						
2C8-4-36 (2)	18	16,809	84,045	1.40	0.512	0.028
2C8-5-48 (2)	30	22,709	73,253	1.22	0.261	0.054
2C8-6-48 (2)	36	30,299	68,861	1.15	0.153	0.049
2C8-6-48 (3)	36	33,162	75,368	1.26	0.337	0.051
2C8-7-48 (2)	42	37,056	61,760	1.03	0.080	N.A.
2C8-7-48 (3)	42	44,237	73,728	1.23	0.214	0.066
2C8-8-48 (3)	48	38,398	48,605	0.81	0.059	N.A.
2C8-8-64 (2)	64	51,521	65,216	1.09	0.166	0.126
2C8-9-48 (3)	54	47,493	47,493	0.79	0.068	N.A.
2C8-9-64 (2)	72	56,321	56,321	0.94	0.130	N.A.
Phase III – 8-inch Concrete Masonry Panels						
3C8-7-48A (C)	42	44,948	74,912	1.25	N.M.	N.M.
3C8-7-76 (C)	67	52,716	87,860	1.46	N.M.	N.M.
3C8-7-85 (C)	74	53,445	89,074	1.48	N.M.	N.M.
3C8-8-48 (C)	48	55,629	70,417	1.17	N.M.	N.M.
3C8-8-88 (C)	88	62,739	79,417	1.32	N.M.	N.M.
3C8-8-98 (C)	98	63,606	80,514	1.34	N.M.	N.M.
3C8-9-48 (C)	54	67,733	67,733	1.13	N.M.	N.M.
3C8-9-101 (C)	114	76,473	76,473	1.27	N.M.	N.M.
3C8-9-113 (C)	127	75,675	75,675	1.26	N.M.	N.M.
3C8-7-48B (C)	42	50,288	83,814	1.40	N.M.	N.M.

Phase IV – 12-inch Concrete Masonry Panels						
4C12-5-32 (C)	20	29,436	94,955	1.58	N.M.	N.M.
4C8-5-32 (C)	20	29,810	96,160	1.60	N.M.	N.M.
4C12-6-31 (C)	23	41,150	93,522	1.56	N.M.	N.M.
4C12-6-48 (C)	36	41,455	94,216	1.57	N.M.	N.M.
4C12-7-37 (C)	32	50,452	84,087	1.40	N.M.	N.M.
4C12-7-57 (C)	50	53,644	89,406	1.49	N.M.	N.M.
4C12-7-76 (C)	66.5	56,631	94,386	1.57	N.M.	N.M.
4C12-8-45 (C)	44.5	56,597	71,642	1.19	N.M.	N.M.
4C12-8-71 (C)	71	61,588	77,960	1.30	N.M.	N.M.

[1]The displacements are the movement between the gage points A and B as shown in Figure 4.2. The measured displacements include deformations of the bars and the slips between the bars and the surrounding grout.
N.M. – Not Measured (Splice displacements were measured for the specimens of Phase II and the clay brick specimens only.)
N.A. – Not Applicable (Reinforcement did not exceed yield, therefore these values could not be calculated.)

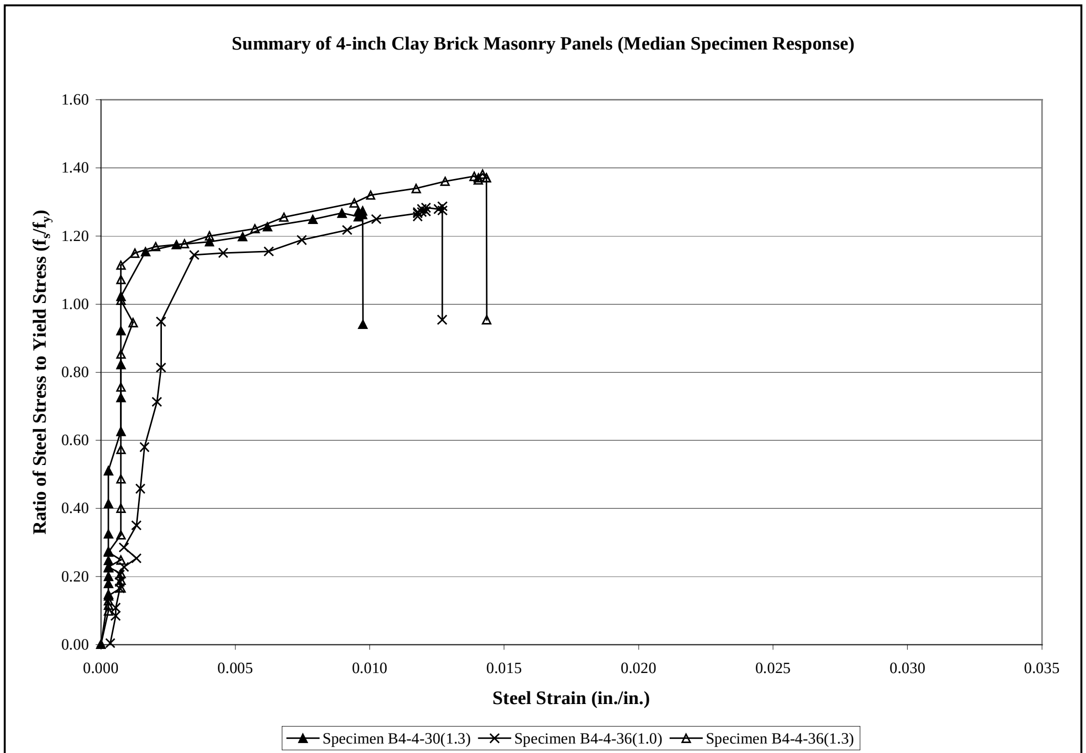

Figure 5.1: Yield Stress Ratio-Strain Plots for One Panel from Each Set of 4 inch Clay Specimens

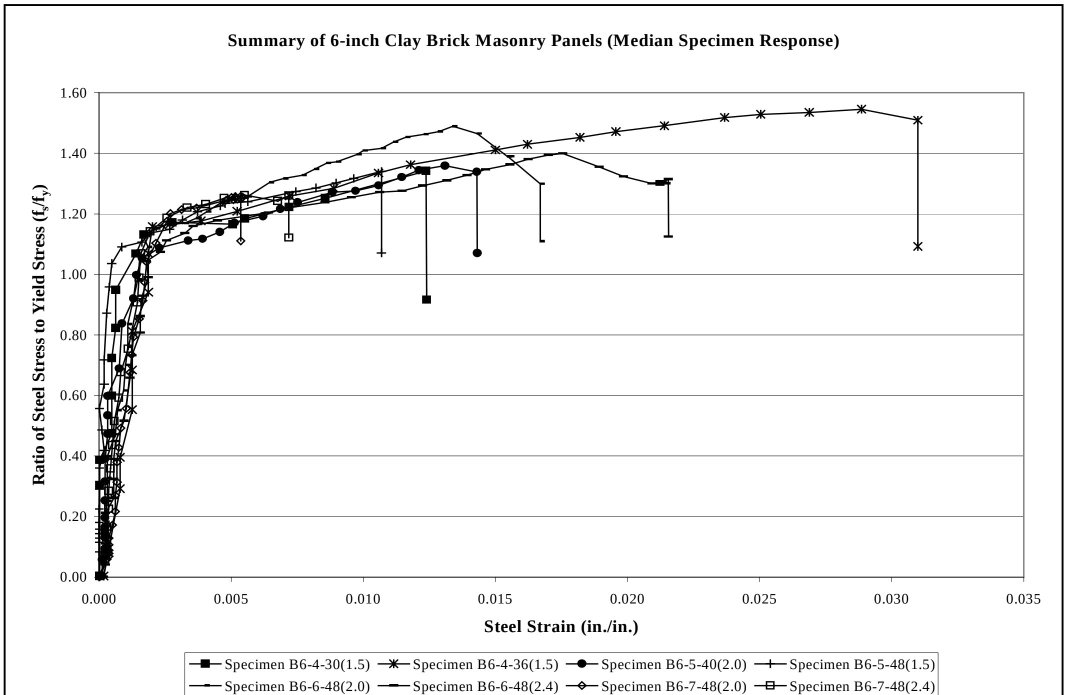

Figure 5.2: Yield Stress Ratio-Strain Plots for One Panel from Each Set of 6 inch Clay Specimens

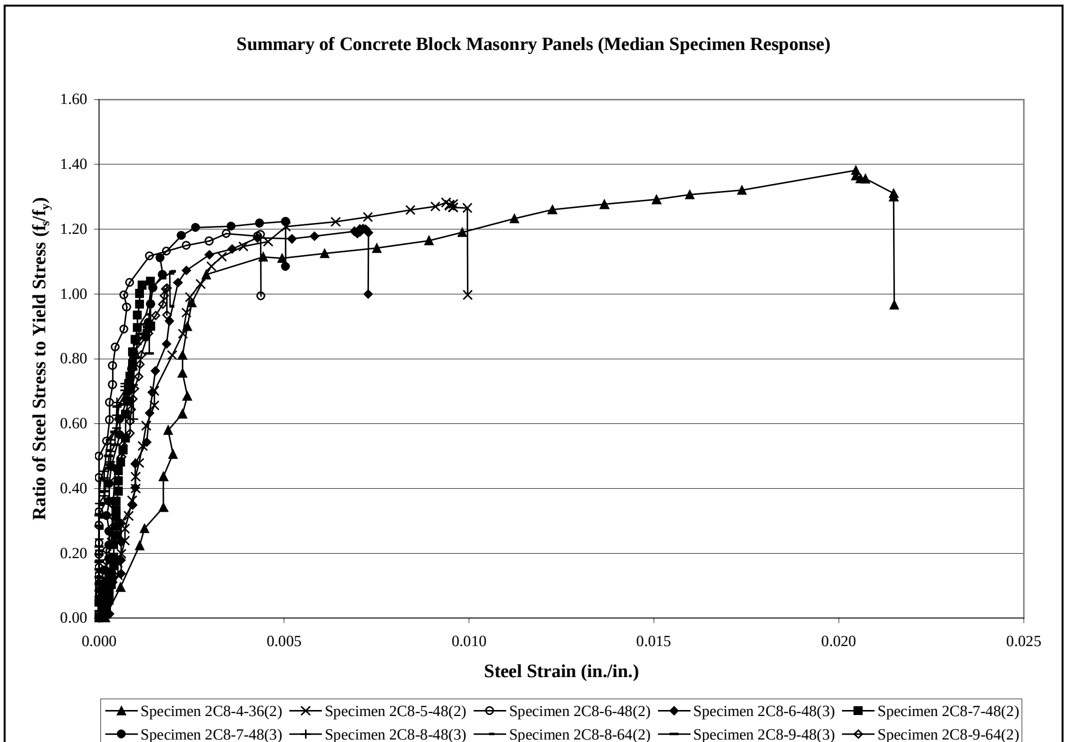

Figure 5.3: Yield Stress Ratio-Strain Plots for One Panel from Each Set of Phase II Concrete Specimens

5.2 Typical Failure Modes

Throughout all phases of testing, three basic modes of failure were observed in the test panels:

- Rupture of the reinforcing steel;
- Longitudinal splitting of the masonry; and
- Pullout of the reinforcement.

Of the 141 individual specimens tested, 16 failed by rupture of the reinforcement, 1 failed by pullout of the reinforcement, and the remaining 124 panels failed by splitting of the masonry.

5.2.1 Rupture of the Reinforcing Steel

Rupture of the reinforcing steel occurred only for wall panels listed in Table 5.2. Typically, only minor hairline cracking was observed on the faces of the panels prior to failure of the reinforcement. Occasionally, in conjunction with rupture of the reinforcing steel, either radial cracking or cone-shaped spalling (approximately 1 in. deep by 3 in. diameter) was observed at the top and bottom of the walls around the ends of the reinforcing bars.

Table 5.2: Test Specimens With Rupture of Reinforcing Steel

Panel Designation	Length of Lap (in.)
1C8-4-36(2)-1	18.0
1C8-4-36(2)-2	18.0
1C8-4-36(2)-3	18.0
4C8-5-32(C)-1	20.0
4C8-5-32(C)-3	20.0
4C12-5-32(C)-1	20.0
4C12-5-32(C)-2	20.0
4C12-5-32(C)-3	20.0
4C12-6-31(C)-1	23.0
4C12-6-31(C)-2	23.0
4C12-6-31(C)-3	23.0
4C12-6-48(C)-2	36.0
4C12-6-48(C)-3	36.0
4C12-7-76(C)-1	66.5
4C12-7-76(C)-2	66.5
4C12-7-76(C)-3	66.5

5.2.2 Longitudinal Splitting of the Masonry

The mode of failure frequently associated with the testing of lap spliced reinforcement in the prescribed direct tension configuration is the longitudinal splitting of the masonry assemblage. The failure surface that formed followed a path defined by the least cover dimension. Two mechanisms that have a significant contribution to this splitting phenomenon are as follows[11]:

1. The first contribution is a direct result of the bond stresses anchoring the reinforcement to the masonry. Since the resultant of the bond stresses acts at an angle, although it varies depending on the extent and type of deformations, it is often adequately approximated at 45° with respect to the axis of the reinforcing bar. The stresses can be resolved into their respective axial and radial components. While the axial portion of the bond stresses resist the motion of the reinforcement, the radial stresses create an internal dilation pressure within the masonry cell. It is this radial pressure that incites the potential of longitudinal splitting of the masonry.
2. The second primary contribution to the splitting of the masonry assemblage stems from the reinforcement splice itself. As two spliced reinforcing bars are stressed, they tend to slip past one another while simultaneously "riding-up" the opposing bar, slightly increasing their separation. Although the majority of slippage occurs subsequent to the formation of cracks in the grout, some small slippage may occur as the adhesion between the grout and the steel is lost. This increase in separation creates additional internal pressures, further increasing the potential for longitudinal splitting of the masonry.

It has been theorized and reported[11] that the distribution of bond stresses along the length of an embedded reinforcing bar is highly nonlinear, even prior to inelastic deformations. This nonlinear distribution of bond stresses creates a region of high bond stresses near the loaded end of the reinforcement. This region of high bond stresses, when combined with the above two described splitting contributions, becomes a catalyst for the splitting of the masonry assemblage.

5.2.3 Pullout of Reinforcement

As mentioned, only one panel (4C12 6-48-1) of the 141 tested failed by pullout of the reinforcement. This mode of failure can be described as a loss of capacity due to the loss of bond between the grout and steel without splitting of the masonry or rupture of the reinforcement. Since this failure was observed in a test panel in which the other two panels of the set failed due to rupture of the reinforcing steel, it was first theorized that the pullout failure was due to poor consolidation of the grout. However, after testing, the panel that exhibited pullout failure was broken apart to inspect the quality of grout consolidation. The grout within this panel did not exhibit any signs (i.e., poor consolidation, segregation of materials, etc.) that may be contributed to a detrimental performance of the splice. Additionally, no radial cracks were observed that might have lead to the loss of confinement of the splice, resulting in the reinforcement slippage. Due to the limited number of test specimens that exhibited this performance and the lack of evidence that could have contributed to such a performance, any definitive conclusions based on these observations would be speculative.

5.3 Typical Splice Performance

As a general overview of the distress observed in the test specimens prior to any failure, the appearance of the first cracks on the surface of the panels (typically in the masonry bed joints) were noted at the approximate loads listed below.

- 10,000 lb. force for the 4 in. clay masonry
- 14,000 lb. force for the 6 in. clay masonry
- 22,000 lb. force for the Phase I 8 in. concrete masonry
- 10,000 lb. force for the Phase II 8 in. concrete masonry
- 22,000 lb. force for the Phase III 8 in. concrete masonry
- 37,000 lb. force for the Phase IV 12 in. concrete masonry
- 24,000 lb. force for the Phase IV 8 in. concrete masonry with contact splices

The panel distress that followed the initial onset of hairline cracks appeared to be dependent on two primary test variables: A) the mode of failure and B) the tested length of lap. When the rupture of the reinforcement was the mode of failure, little to no other distress was witnessed in the test specimens after the onset of the initial hairline cracks. Conversely, when longitudinal splitting of the masonry was the mode of failure, the hairline cracks would continue to propagate and widen until failure of the test specimen would occur.

In a similar fashion, the length of lap of the spliced reinforcement had a significant impact on the performance of the splice prior to failure. In panels containing relatively short lap lengths, little to no distress was observed in the test specimens prior to failure – regardless of the mode of failure. Similarly, where rupture of the reinforcement was the mode of failure and the lap tested was relatively long, little panel distress was observed prior to failure. Conversely, test specimens containing relatively long lap lengths and where the mode of failure was due to longitudinal splitting of the masonry, extreme cracking and spalling was observed prior to the panels' failure. Refer to Figure 5.4 for a summary of the influence of the testing variables on the performance of the tested splices. Since only one pullout failure was observed throughout all phases of testing, no conclusive results could be drawn on this type of failure mode.

An additional dissimilarity between the performance of the various test specimens included the location of the initial hairline cracks. For the panels in which the mode of failure was due to steel fracture, any panel cracking witnessed would typically be focused around the ends, where the reinforcing steel protruded from the panels. This observation is attributed to the stress concentrations described in Section 5.2.2. For those test specimens that failed by longitudinal splitting of the masonry, hairline cracks in the bed joints near the mid-height of the wall panels typically occurred first, with cracking occurring in other bed joints soon afterwards. As the load was increased, cracks present became more pronounced and new cracks formed near the ends of the panels. Once loads reached those approximately corresponding to yield strength of the reinforcement, the propagation and width of the cracks became much more pronounced.

5.4 Analysis of Results

With the goal of quantifying the contributions of various testing variables beyond the scope of this research program, the results of this study were combined with the results from other programs investigating the performance of lap splices in masonry. These independent investigations include research completed by Thompson[11] and Hammons, et al.[4].

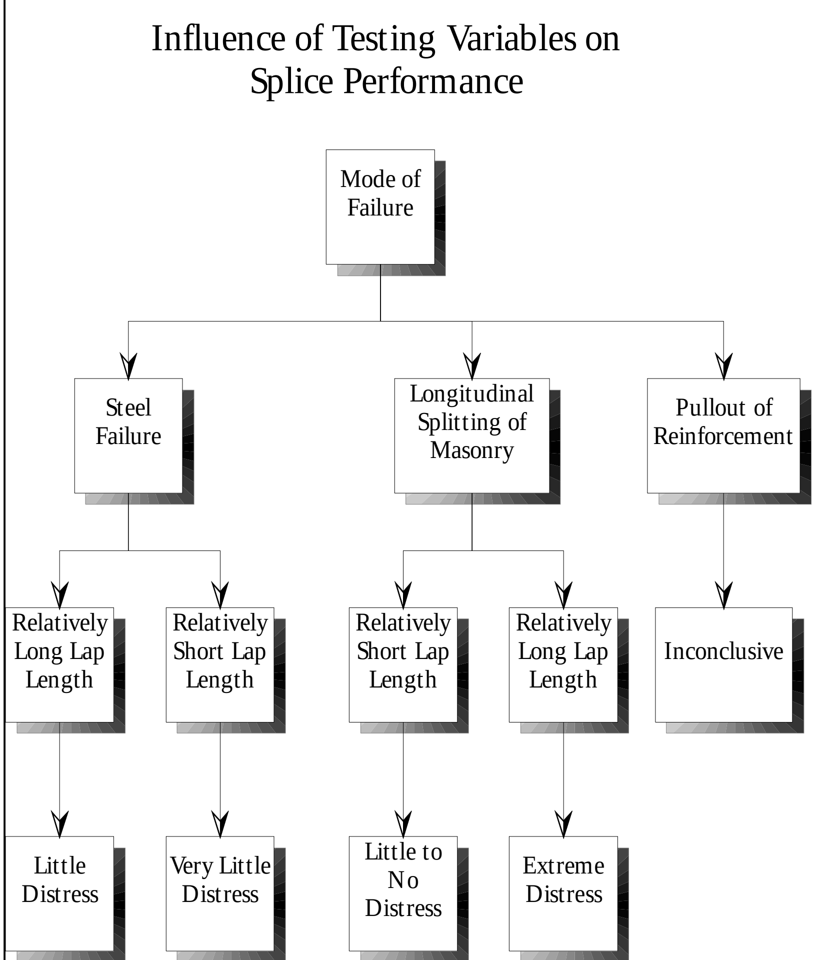

Figure 5.4: Influence of Testing Variables on Splice Performance

Of the data compiled from these three different programs, the clay masonry specimens, the specimens that contained transverse steel (in the form of bed joint steel), the specimens that failed by fracture of the reinforcement, and the specimens that failed by pullout of the reinforcement were removed for the analyses, leaving only specimens in which the primary mode of failure was due to longitudinal splitting of the panel. As a note, the only specimens that failed by pullout of the reinforcement either contained transverse steel or were part of this investigation. The reasons for the removal of these specimens are as follows. First, regarding the fractured reinforcement, the additional load capacity of these splices, above that which is required to fracture the reinforcement, is unknown. Second, selecting only the specimens with longitudinal splitting allows the analyses to be focused on a particular mode of failure. Having this uniform datum established, more generalized conclusions can be drawn.

The resulting data set used for the analyses consisted of 177 individual specimens (after removing those specimens outside of the scope of analysis as outlined above) reinforced with Grade 60 spliced steel ranging from No. 4 bars to No. 11 bars and with lap lengths varying from 12 inches to nearly 130 inches. Clear covers ranged from 1.44 inches to 5.5 inches in concrete masonry units of nominal thickness from 4 inches to 12 inches.

Although numerous correlations were attempted, only the findings believed to provide insight into the performance of splice lengths in masonry are presented here. Refer to Thompson[11] for more information.

5.4.1 Multiple Linear Regression Analyses

Targeting the parameters that historically have been shown to have a significant effect on the capacity of a splice, i.e., compressive strength of the masonry, diameter of the reinforcing bar, length of lap of the splice, and cover, several multiple linear regression analyses were performed in an attempt to determine the contribution of each of these parameters to the performance of a splice. (As a note, these analyses included looking at the effect of the compressive strength of the unit and the compressive strength of the grout as well as the influence of using either the clear cover or the cover to the center of the reinforcement. Although good correlation to the square root of the compressive strength of the unit and the grout was obtained, the square root of the compressive strength of the masonry was chosen as it has historically served as a basis for the determination of length of lap for splices. Additionally, the correlation obtained by using either the unit or grout strengths showed no added benefit by changing this historical basis. An added benefit of using the compressive strength of the masonry assemblage is that it provides a composite measure of the grout strength and the unit strength. Correlation to the clear cover was chosen over the cover to the center of the reinforcement as it was felt that clear cover measurements more accurately represented the affect of increasing the diameter of the reinforcement. For example, for a given wythe thickness, if the reinforcing steel were centered in the masonry cell, the cover would remain constant for any bar diameter if the cover were measured to the center of the steel. However, if the cover were measured to the outside diameter of the reinforcement, the cover would decrease as the diameter of the reinforcement increased.)

The resulting multiple linear regression equation that gave the best prediction of the observed capacities of splices in reinforced concrete masonry is as follows:

$$T_r = -17624 + 305\, l_s + 25204\, {d_b}^2 + 322\sqrt{f_{mt}} + 3332 c_{cl} \qquad \text{[Eq. 1]}$$

Where:
- T_r = Predicted load capacity of the splice, lb.;
- l_s = Tested lap length of splice, in.;
- d_b = Diameter of reinforcement, in.;
- f_{mt} = Tested compressive strength of masonry, psi; and
- c_{cl} = Clear cover of reinforcement, in.

Equation 1 results in an r^2 value of 0.93 and an adjusted r^2 value of 0.93. A plot of the measured load capacity of the splices tested versus the predicted load capacity given by Equation 1 is shown in Figure 5.5.

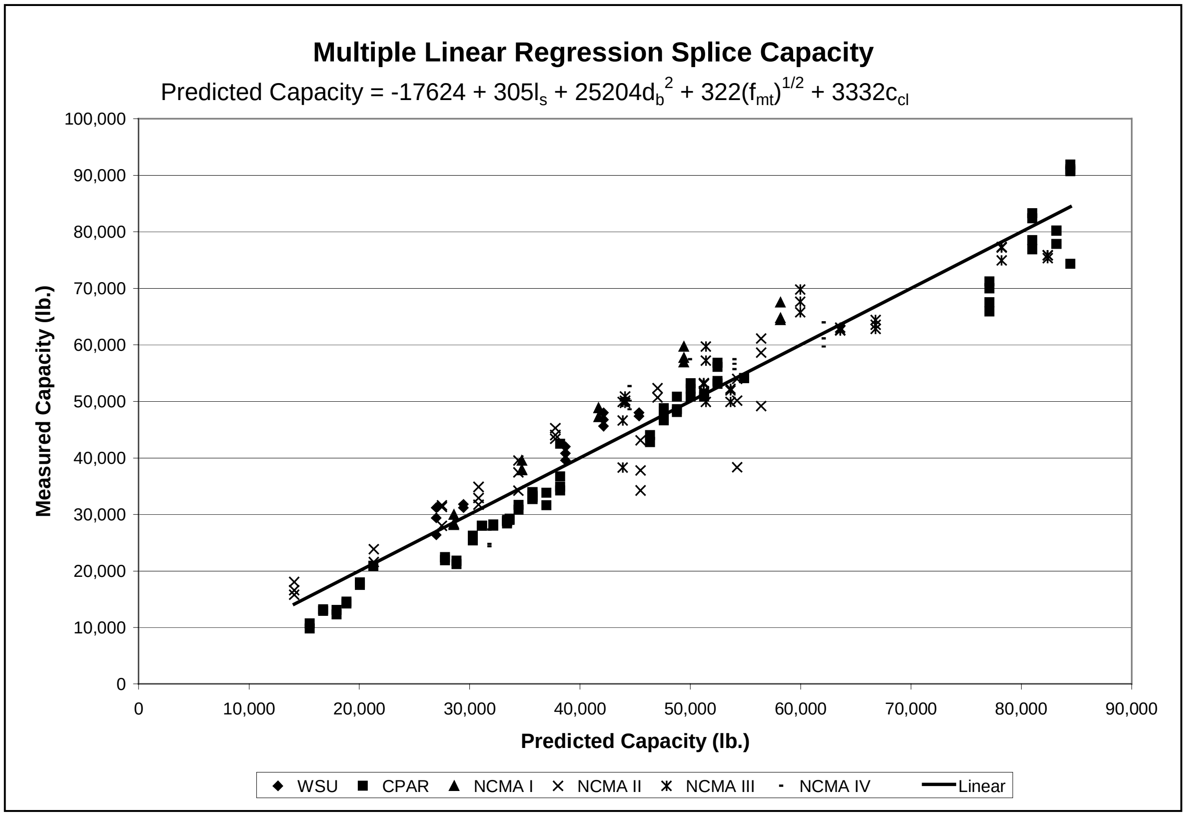

Figure 5.5: Predicted Splice Capacity

From Equation 1, the predicted load capacity of the splice can be replaced by a specified nominal strength required of the splice. Based on code requirements outlined in Section 6, the predicted load capacity of the splice is replaced with 1.25 times the area of the reinforcing bar times the specified yield strength of the reinforcing bar, i.e., $T_r = 1.25A_bf_y$. The basic length required to develop this capacity l_{de} was then solved for:

$$l_r = \frac{1.25\,A_b\,f_y + 17624 - 25204 d_b^{\,2} - 322\sqrt{f'_m} - 3332\,c_{cl}}{305} \qquad \text{[Eq. 2]}$$

Where: l_r = Basic development length based on regression analysis, in.;
A_b = Area of reinforcing bar, in^2;
f_y = Yield strength of reinforcing steel, psi;
d_b = Diameter of the reinforcing bar, in.;
fN_m = Specified compressive strength of the masonry, psi; and
c_{cl} = Clear cover to reinforcement, in.

5.4.2 Simplification of Regression Equation

Since Equation 2 is in an impractical form for every day design applications, an attempt was made to simplify the equation. After an inspection and review of currently available design equations, the form of the current Uniform Building Code equation (Equation 6) for calculating the development length was chosen as:

1) It is familiar to design professionals; and
2) From analysis, the length of lap was determined to be proportional to the square of the diameter of the reinforcing bar and the yield strength of the reinforcement and inversely proportional to the square root of the compressive strength of the masonry assemblage and the cover as is presented in the UBC equation.

Since the influence of each of these variables is straightforward except for the cover, the next step was to determine the limiting contribution of the clear cover.

As was previously noted, there was one observed pullout failure (for specimens that did not contain horizontal reinforcement) included in the testing program completed by NCMA. This single panel that failed by pullout was reinforced with a No. 6 bar containing a 36-inch ($48d_b$) lap length. The other two panels of this set failed by fracture of the reinforcement. It is also of interest to note that three other panels tested with a No. 6 bar containing a 23-inch ($31d_b$) lap length in NCMA's fourth phase of research failed by fracture of the reinforcement. Combine this with the observation that the No. 5 bars tested during the Phase IV research program had significantly more clear cover and yet did not fail by pullout, the observed pullout failure may simply be attributed to being a fluke. (Because of this isolated pullout failure, it was initially thought that this performance could be attributed to poor grout consolidation or segregation of the grout. However, after testing, the panel was torn apart with no observed indication that these factors were present or could have contributed to a detrimental performance.)

Historically, the rationale for including a maximum cover limit in the codes for developing reinforcing steel was originally intended to preclude any pullout type failures. However, this may simply be a remnant of past days when the design equations for development were based on a limiting value of the bond stress. Equation 2 is not based on bond stress, but instead establishes a requirement of attaining a minimum capacity of $1.25f_y$ while taking into account other types of failure such as the longitudinal splitting of the masonry. As a result, the limitation on the clear cover may not be as significant as it once was (however – still important). Nonetheless, erring on the side of conservatism and choosing a limit of $7d_b$ provides a benefit in light of this questionable performance.

If it were not for the single incidence of pullout failure, one could easily rationalize an increase in the maximum allowable cover up to the maximum tested to date ($8.8d_b$). However, because of the numerous questions raised with the pullout failure, increasing the allowable maximum cover beyond $7d_b$ (or $7.25d_b$ at which the pullout failure was observed) cannot be rationally justified until these questions have answers.

Taking the above into consideration, the following simplified design equation for the basic development length is presented:

$$l_{de} = \frac{0.13\, d_b{}^2\, f_y\, \gamma}{K \sqrt{f'_m}} \qquad \text{[Eq. 3]}$$

Where: l_{de} = basic development length, in., not to be taken less than 12 inches;
d_b = diameter of reinforcement, in.;
f_y = specified yield strength of reinforcement, psi;
(= reinforcement size factor;
= 1.0 for No. 3 through No. 5 reinforcing bars;
= 1.4 for No. 6 through No. 7 reinforcing bars;
= 1.5 for No. 8 through No. 11 reinforcing bars;
K = minimum clear cover to reinforcement, in., shall not be taken more than $7d_b$; and
fN_m = specified compressive strength of masonry; psi.

Because no tests were completed using reinforcement larger than No. 11 bars, the application of Equation 3 to reinforcement larger than No. 11 bars is not recommended.

Next, applying a strength reduction factor as is currently used in the model building codes for lap spliced reinforcement, the following equation for the length of lap required for spliced reinforcement in masonry is presented:

$$l_d = \frac{l_{de}}{\phi} \qquad \text{[Eq. 4]}$$

Where:

l_d = minimum lap splice length of reinforcement, in.; and

ϕ = strength reduction factor; equal to 0.80.

Equations 2, 3, and 4 are plotted in Figure 5.6 for the following design parameters:

- 1,500 psi masonry compressive strength;
- 8-inch masonry units;
- Reinforcement centered within the cells of the units (thereby providing maximum cover); and
- Grade 60 reinforcement.

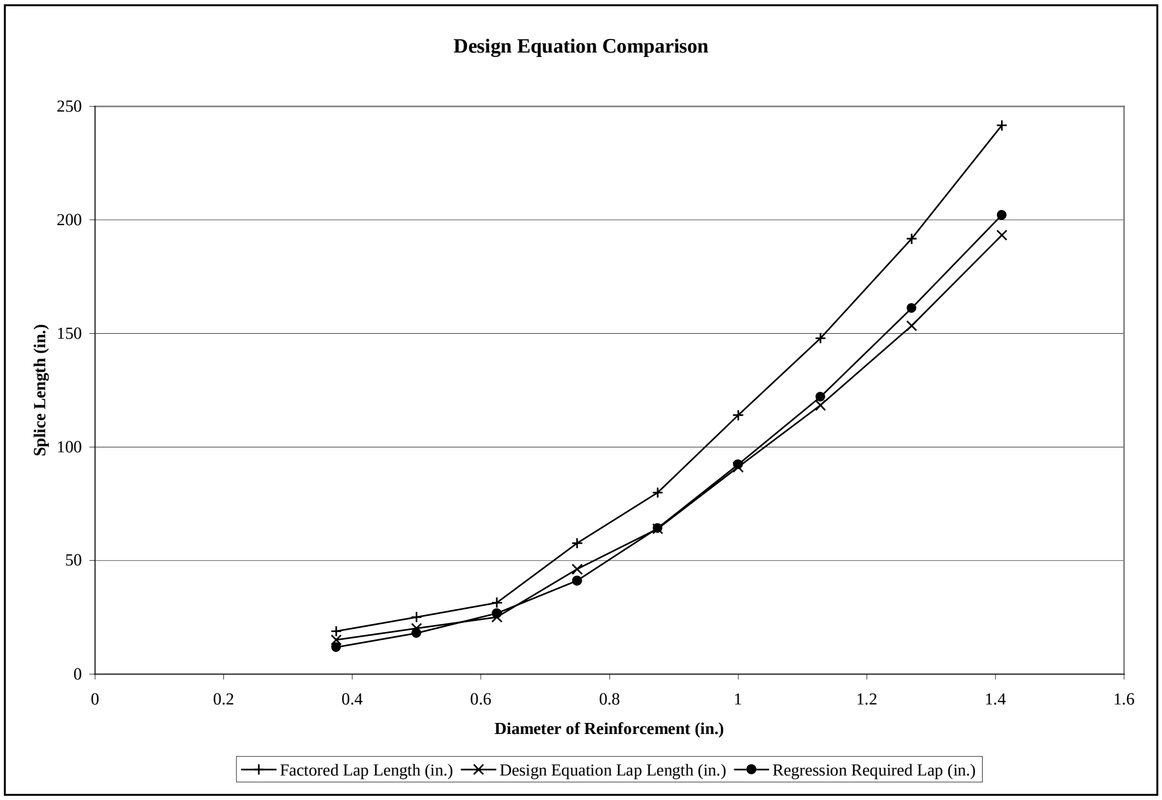

Figure 5.6: Design Equation Comparison

As can be seen in Figure 5.6, Equation 3, without the strength reduction factor, approximately matches the regression based Equation 2. The plot of Equation 4 requires slightly higher lap splice lengths due to the addition of the strength reduction factor.

5.5 Effect of Design Variables

5.5.1 Masonry Compressive Strength

Splice capacity of concrete masonry panels increased approximately 27% by increasing the compressive strength of the masonry 140%. This effect is demonstrated by comparing the tests for No. 4 through No. 7 bars in Phase I using a masonry strength (based on prism tests) of 4,070 psi, compared to tests of similar panels in Phase II using masonry strengths of 1,700 psi.

As noted in Section 2.3.3, segregation of grout materials (due to the low cement content of the grout) was observed during the grouting of the Phase II concrete masonry panels. This segregation may have influenced the integrity of the grout in these panels. Although the tested grout strengths exceeded the minimum requirements of ASTM C 476, these observed characteristics may have reduced the grout's capacity to resist the splitting forces induced by the tensile loads and displacements of the spliced bars during panel testing.

The splice capacities of the 6-inch clay masonry panels having masonry strengths of approximately 6,410 psi compared well with capacities of the Phase I concrete masonry panels when similar lap lengths and cover depths were used. One set of brick masonry panels, B6-6-48(2), had the same splice and cover as one set of concrete masonry panels, 1C8-6-48(2). Despite having a compressive strength that was approximately 50% greater than the concrete masonry panels, the splice capacity of the brick masonry panels was not significantly increased. However, the possible effects of a smaller panel thickness and smaller grout core in comparison to the concrete masonry panels may have offset the greater material strengths for the brick masonry panels.

5.5.2 Cover Depth

The results of the research confirmed that increasing cover depth results in greater splice capacities and more signs of panel distress prior to failure. This relationship is best seen in the 4-inch clay masonry test results. Increasing the cover depth for a No. 4 bar by ¼-inch from 1 to 1.25 inches increased the splice capacity by 8% and the maximum measured displacement of the splice by 45%. The Phase II concrete masonry research included tests on No. 6 and No. 7 bars at cover depths of 2 inches and 3 inches. The specimens having the 50% larger cover depths achieved 8.5% greater tensile loads for the No. 6 bars and 18% greater tensile loads for the No. 7 bars. The increased cover depths for these concrete masonry panels also resulted in approximately 120 and 170% increases in maximum measured splice displacement for the No. 6 and No. 7 bars, respectively.

5.5.3 Lap Length

Increasing lap length had a somewhat greater influence on splice capacity than cover depth. Increasing the lap length of No. 4 bars in the 4 inch and 6 inch clay masonry walls by 20% from 30 to 36 d_b raised the splice capacity approximately 10 and 11%, respectively, and the maximum measured splice displacement improved 80 and 70%. Tests conducted on No. 8 and No. 9 bars in the Phase II concrete masonry panels showed that specimens constructed using 64 d_b splices at a cover depth of 2 inches had a 6% greater capacity and approximately twice the measured splice displacement than specimens with a shorter 48 d_b splices at a greater cover depth of 3 inches. However, testing indicated that there is a point of diminishing benefit to increasing the lap length. For example, comparing the Phase III specimens where the cover and material strengths were maintained constant for a given bar diameter (excluding specimen set 3C8-7-48B (C) which is not considered for this discussion) shows this trend as indicated in Figure 5.7. This phenomenon may also help explain the exponential increase in lap length required as the diameter of the reinforcement increases as given by Eq. 3 and shown in Figure 5.6.

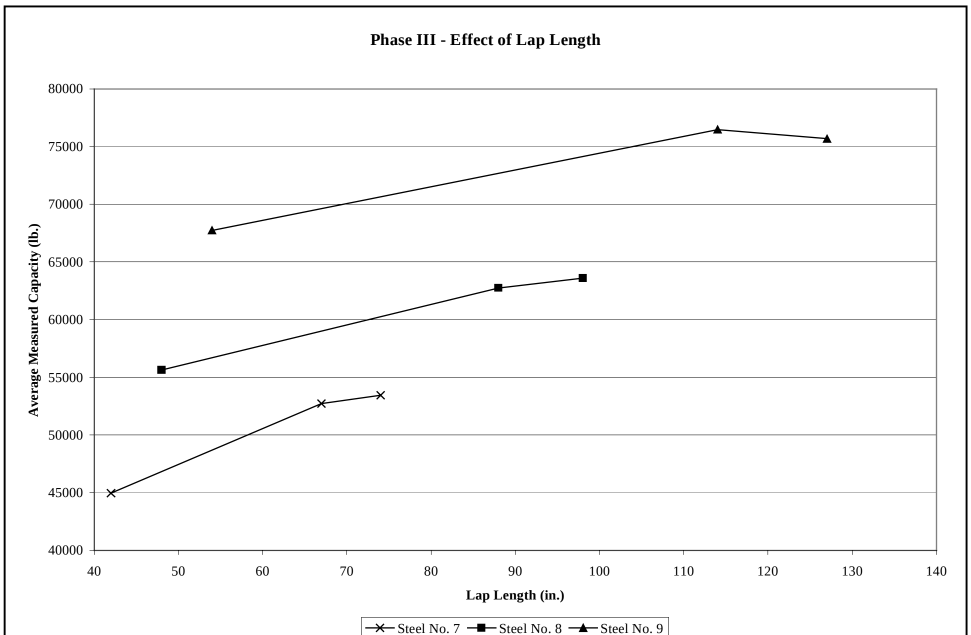

Figure 5.7: Influence of Splice Lap Length on Nominal Strength

6.0 REVIEW OF CURRENT CODE SPLICE CRITERIA

Several criteria currently exist in the United States to determine the required splice length for reinforcing bars in masonry. ACI 530/ASCE 5/TMS 402, *Building Code Requirements for Masonry Structures*[12] requires:

$$l_d = 0.002\, d_b\, F_s \qquad \text{[Eq. 5]}$$

Where:

F_s	=	allowable stress in reinforcing steel, psi;
d_b	=	diameter of reinforcing bar, in.

Since an allowable steel stress of 24,000 psi is typically used in design (neglecting increases for wind or seismic loads), this equation requires l_d to be 48 d_b or larger. When allowable stresses are increased by one-third for wind and seismic loads (as permitted by ACI 530/ASCE 5/TMS 402, Section 5.3.2), the allowable steel stress is 32,000 psi and l_d would be required to equal or exceed to 64 d_b.

The ACI 530/ASCE 5/TMS 402 also requires that mechanical and welded connections be capable of developing a force equal to 1.25 times the specified yield strength of the reinforcement (1.25 f_y). (This criterion was imposed within Section 5 for developing the required capacity for conventionally spliced reinforcement.)

While the Uniform Building Code[13] contains both of the above criteria, it also contains the following strength design criteria:

$$l_{de} = \frac{0.15\, d_b^{\,2}\, f_y}{K\sqrt{f'_m}} \qquad \text{[Eq. 6]}$$

Where:

l_{de}	=	basic development length of reinforcement, in.;
d_b	=	diameter of reinforcing bar, in.;
f_y	=	tensile yield stress of reinforcement, psi;
K	=	minimum reinforcement cover, not to be taken greater than 3 d_b, in.; and
f'_m	=	specified compressive strength of masonry at age of 28 days, psi.

The development length determined by Equation 6 is permitted to be reduced to $52d_b$. This permitted maximum development length allows lap splice lengths to be much shorter in many cases than would be required by the formula directly. Table 6.1 shows the effect of this permitted reduction in development length. For design in accordance with the strength design provisions of the UBC, the splice length must equal or exceed that given by Eq. 7.

$$l_d = \frac{l_{de}}{\phi} \qquad \text{[Eq. 7]}$$

Where:

l_d	=	minimum lap splice length of reinforcement, in.; and
ϕ	=	strength reduction factor; equal to 0.80.

Table 6.1 shows required embedment lengths (l_{de}) and required lap lengths (l_d) based on Equation 6 and 7 using the masonry strengths from Phase I and Phase II of the research.

Table 6.1a: Minimum Basic Development Lengths, l_{de}, and Splice Lengths, l_d, Required by Eq. 6, 7, and 4 for f_y = 60,000 psi and Properties of Panels Tested in Phases I, II and Clay Masonry Specimens

Bar Size	52 d_b Embed. Length (in.)	**Embedment length, l_{de}, as required by Eq. 6**							
		f'_m = 1,700 psi		f'_m = 4,070 psi	f'_m = 4,310 psi		f'_m = 6,410 psi		
		K = 2 in.	K = 3 in.	K = 2 in.	K = 1 in.	K = 1.25 in.	K = 1.5 in.	K = 2 in.	K = 2.4 in.
No. 4	26.0	36.4	36.4	23.5	34.3	27.4	18.7	18.7	18.7
No. 5	32.5	45.5	45.5	29.4	53.6	42.8	29.3	23.4	23.4
No. 6	39.0	61.4	54.6	39.7	77.1	61.7	42.2	31.6	28.1
No. 7	45.5	83.6	63.7	54.0	105.0	84.0	57.4	43.0	36.6
No. 8	52.0	109.1	72.8	70.5	137.1	109.7	74.9	56.2	47.8
No. 9	58.7	138.1	92.1	89.7	174.4	139.5	95.4	71.5	60.9
Bar Size	52 d_b/ϕ Embed. Length (in.)	**Splice Length, l_d as required by Eq. 7**							
		f'_m = 1,700 psi		f'_m = 4,070 psi	f'_m = 4,310 psi		f'_m = 6,410 psi		
		K = 2 in.	K = 3 in.	K = 2 in.	K = 1 in.	K = 1.25 in.	K = 1.5 in.	K = 2 in.	K = 2.4 in.
No. 4	32.5	45.5	45.5	29.4	42.8	34.3	23.4	23.4	23.4
No. 5	40.6	56.8	56.8	36.7	66.9	53.6	36.6	29.3	29.3
No. 6	48.8	76.7	68.2	49.6	96.4	77.1	52.7	39.5	35.1
No. 7	56.9	104.5	79.6	67.5	131.2	105.0	71.7	53.8	45.8
No. 8	65.0	136.4	91.0	88.2	171.4	137.1	93.7	70.3	59.8
No. 9	73.3	173.6	115.7	112.2	218.0	174.4	119.2	89.4	76.1
Bar Size		**Splice Length, l_d as required by Eq. 4**							
		f'_m = 1,700 psi		f'_m = 4,070 psi	f'_m = 4,310 psi		f'_m = 6,410 psi		
		K = 2 in.	K = 3 in.	K = 2 in.	K = 1 in.	K = 1.25 in.	K = 1.5 in.	K = 2 in.	K = 2.4 in.
No. 4		29.6	19.7	19.1	37.1	29.7	20.3	15.2	13.0
No. 5		46.2	30.8	29.8	58.0	46.4	31.7	23.8	20.2
No. 6		93.1	62.1	60.2	117.0	93.6	63.9	48.0	40.8
No. 7		126.7	84.5	81.9	159.2	127.4	87.0	65.3	55.5
No. 8		177.4	118.2	114.6	222.8	178.2	121.8	91.3	77.7
No. 9		225.7	150.4	145.8	283.4	226.8	155.0	116.2	98.9

As is shown in Tables 6.1a and 6.1b, Equation 6 contains a very restricted limitation on the benefit associated with increasing depth of cover. This observation is exhibited by comparing the splice length required by Equation 6 using smaller diameter bars. In other words, by holding all variables constant except for the clear cover, Equation 6 requires the same length of lap for lap splices having both 2 inches and 3 inches of cover.

Table 6.1b: Minimum Basic Development Lengths, l_{de}, and Splice Lengths, l_d, Required by Eq. 6, 7, and 4 for f_y = 60,000 psi and Properties of Panels Tested in Phases III and IV

Bar Size	52 d_b Embed. Length (in.)	**Embedment length, l_{de}, as required by Eq. 6**		
		f'_m = 3,120 psi	f'_m = 3,290 psi	f'_m = 2,180 psi
		K = Centered in Cell	K = Centered in Cell	K = Centered in Cell
No. 4	26.0	26.9	26.2	32.1
No. 5	32.5	33.6	32.7	40.2
No. 6	39.0	40.3	39.2	48.2
No. 7	45.5	47.0	45.8	56.2
No. 8	52.0	53.7	52.3	64.3
No. 9	58.7	63.1	61.5	72.5
Bar Size	52 d_b/ϕ Embed. Length (in.)	**Splice Length, l_d as required by Eq. 7**		
		f'_m = 3,120 psi	f'_m = 3,290 psi	f'_m = 2,180 psi
		K = Centered in Cell	K = Centered in Cell	K = Centered in Cell
No. 4	32.5	33.6	32.7	40.2
No. 5	40.6	42.0	40.9	50.2
No. 6	48.8	50.4	49.0	60.2
No. 7	56.9	58.7	57.2	70.3
No. 8	65.0	67.1	65.4	80.3
No. 9	73.3	78.9	76.8	90.6
Bar Size		**Splice Length, l_d as required by Eq. 4**		
		f'_m = 3,120 psi	f'_m = 3,290 psi	f'_m = 2,180 psi
		K = Centered in Cell	K = Centered in Cell	K = Centered in Cell
No. 4		15.0	15.0	15.0
No. 5		19.5	19.0	18.6
No. 6		40.0	38.9	31.3
No. 7		55.4	54.0	41.6
No. 8		79.0	77.0	59.0
No. 9		102.6	99.9	75.9

6.1 Comparison of Test Results to Code Criteria

Properly designed masonry should be capable of carrying and distributing or transferring any design loads subjected to it. The capacity of all of the clay masonry panels tested and all of the Phase I concrete masonry panels tested exceeded the specified yield strength of the reinforcing bars. These panels were constructed using masonry having a compressive strength in excess of 4,000 psi. All of the Phase II concrete masonry panels containing No. 4 through No. 7 bars were also able to develop the yield strength of the bars. However, the 1,700 psi masonry strength of these panels did not permit the reinforcement in 3 of the 4 panel sets containing No. 8 and No. 9 bars (each lapped $48d_b$ and $64d_b$) to reach their yield strength.

As previously noted, many codes require mechanical and welded splices to be capable of resisting loads of at least 1.25 times the specified yield strength of reinforcing steel (1.25 f_y). If this criterion is used to evaluate acceptable performance of the tested splices:

- *For No. 4 Bars*: Splice capacities exceeding 1.25 f_y were achieved using $36d_b$ lap splices at minimum cover depths with low masonry strengths. Smaller laps of $30d_b$ would likely have achieved this capacity if higher masonry strengths were used.

- *For No. 5 and No. 6 Bars*: When these bar sizes were spliced for 40d_b or more in clay masonry panels having strengths greater than 4,500 psi, acceptable performance was achieved with cover depths as small as 1.0 inch.

  When these bar sizes were spliced for 48d_b or more in concrete masonry panels having strengths greater than 4,000 psi, acceptable performance was achieved with cover depths as small as 2 inches. Stresses in excess of 1.25 f_y were also achieved with No. 6 bars for splice lengths of 48d_b in masonry of relatively low compressive strength with cover depths of 3 inches. Similar capacities would be expected for No. 5 bars placed in similar conditions.

- *For No. 7 Bars*: Acceptable performance was achieved at minimum cover depths using 4,070 psi concrete masonry. However, with concrete masonry strengths of 1,700 psi, stresses in excess of 1.25 f_y were not attained for a splice length of 48d_b and a cover depth of 3 inches. Splice lengths of 48d_b in clay masonry with masonry strengths of 6,300 psi and cover depths of 2-δ inches provided acceptable performance.

- *For No. 8 and No. 9 Bars*: The only acceptable performance observed using No. 8 and No. 9 bars was with the long lap lengths of Phase III or using 12 inch concrete masonry units in Phase IV.

While these tests were not designed to evaluate the Table 6.1 splice requirements, the following observations can be made:

- The lap lengths used for the clay masonry panels were less than the minimum splice lengths required by Table 6.1a (based on Equation 7) with the exception of B6-6-48(2.4). All of the splices in the clay masonry panels achieved capacities in excess of the specified yield strength of the reinforcing steel (f_y = 60,000 psi) indicating that Equation 6 (without the strength reduction factor) may be overly conservative, particularly for small and mid-size reinforcing bars. The research shows that for splices shorter than those required by Equations 6 and 7, splice capacities in excess of the yield strength of the reinforcing steel can be obtained.

- The lap lengths used for the Phase I concrete masonry panels were less than the minimum lap splice lengths required by Table 6.1a (based on Equation 7). Additionally, most panels (except panels 1C8-4-36(2) and 1C8-5-48(2)) had lap lengths less than the embedment lengths required by Table 6.1a (based on Equation 6). All of the splices in the Phase I concrete masonry panels achieved capacities in excess of the specified yield strength of the reinforcing steel (f_y = 60,000 psi), again indicating that Equation 6 may be overly conservative for large f'_m values, particularly for small and mid-size reinforcing bars. The research shows that for splices shorter than those required by Equations 6 and 7, splice capacities in excess of the yield strength of the reinforcing steel can be obtained.

- The lap lengths used for all Phase II concrete masonry panels were less than the minimum splice lengths required by Table 6.1a (based on Equation 6 using f'_m = 1,700 psi) except for specimen set 2C8-5-48 (2). All of the splices evaluated for No. 4 to No. 7 bars achieved capacities in excess of the specified yield strength of the reinforcing steel, indicating that Equation 6 may be overly conservative for small and mid-size reinforcing bars placed in low strength masonry.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the research, the following conclusions can be drawn:

1. The research confirmed the effect of the parameters included in Eq. 6 on splice capacities, i.e. increasing splice length, structural cover depth, and masonry strength improves performance and increases the nominal capacity of a splice. Also increasing bar diameter necessitates longer lap lengths. The research does not, however, validate the splice requirements resulting from the use of Eq. 6.

2. Criteria based on a linear relationship between lap length and bar diameter do not account for the effects of other variables on splice performance. Therefore, such criteria (i.e. $l_d = 48\ d_b$) is conservative for some cases and unconservative for others.

   - Splice length criteria included in Eq. 5 and 6 overestimate lap lengths required to develop the yield stress of small reinforcing bars.

   - Splice length criteria included in Eq. 5 and 6 underestimate lap lengths required to develop the yield stress of larger reinforcing bars. Similarly, the upper limit of 52 d_b/ϕ permitted by Eq. 6 underestimates lap lengths required to develop the yield stress of larger reinforcing bars (No. 8 and No. 9 bars). The design of reinforced masonry using small cover depths further compounds this issue.

3. Consideration should be given to increasing minimum permitted grout compressive strengths from 2,000 psi to at least 3,000 psi. The cementitious materials used to achieve 2,000 psi grout strength were not considered adequate to obtain uniform grout consistency without material separation. This increased requirement will not likely adversely impact the economy of grouted masonry construction since few projects are believed to use grout having compressive strengths below 3,000 psi. This increased requirement would reduce the possibility of grout segregation, and the higher minimum grout strengths would likely decrease required lap lengths.

4. The development length given by Eq. 3 and 4 accurately predicts necessary splice (or embedment) length required to develop $1.25A_sf_y$.

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[1b]	C 90-96	*Specification for Load-Bearing Concrete Masonry Units*
[1c]	C 140-96	*Test Methods for Sampling and Testing Concrete Masonry Units*
[1d]	C 143-90a	*Test Methods for Slump of Hydraulic Cement Mortar*
[1e]	C 144-93	*Specification for Aggregates for Masonry Mortar*
[1f]	C 150-95	*Specification for Portland Cement*
[1g]	C 207-91(92)	*Specification for Hydrated Lime for Mortar Purposes*
[1h]	C 270-96	*Specification for Mortar for Unit Masonry*
[1i]	C 404-97	*Specification for Aggregates for Masonry Grout*
[1j]	C 476-95	*Specification for Grout for Unit Masonry*
[1k]	C 652-95a	*Specification for Hollow Brick (Hollow Masonry Units Made from Clay or Shale)*
[1l]	C 780-94	*Test Method for Construction and Preconstruction Evaluation of Mortars for Plain and Reinforced Masonry*
[1m]	C 1019-89a(93)	*Test Method for Sampling and Testing Grout*
[1n]	C 1314-95	*Constructing and Testing of Masonry Prisms Used to Determine Compliance with Specified Compressive Strength of Masonry*
[1o]	A 370-94	*Test Methods and Definitions for Mechanical Testing of Steel Products*
[1p]	A 615/A 615M-95[93]	*Specifications for Deformed and Plain Billet Bars for Concrete Reinforcement*

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[8] Suter, G. T., Fenton, G. A., “Splice Length Tests of Reinforced Concrete Masonry Walls.” The Masonry Society, June 1985, p. 14.

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[11] Thompson, J. J., “Behavior and Design of Tension Lap Splices in Reinforced Concrete Masonry”, M. Eng. Thesis, Washington State University, Pullman, Washington, August, 1997.

[12] Building Code Requirements for Masonry Structures (ACI 530-95/ASCE 5-95/TMS 402-95) and Specification for Masonry Structures (ACI 530.1-95/ASCE 6-95/TMS 602-95), Reported by the Masonry Standards Joint Committee.

[13] Uniform Building Code. Whittier, CA: International Conference of Building Officials (ICBO), 1997.

APPENDIX

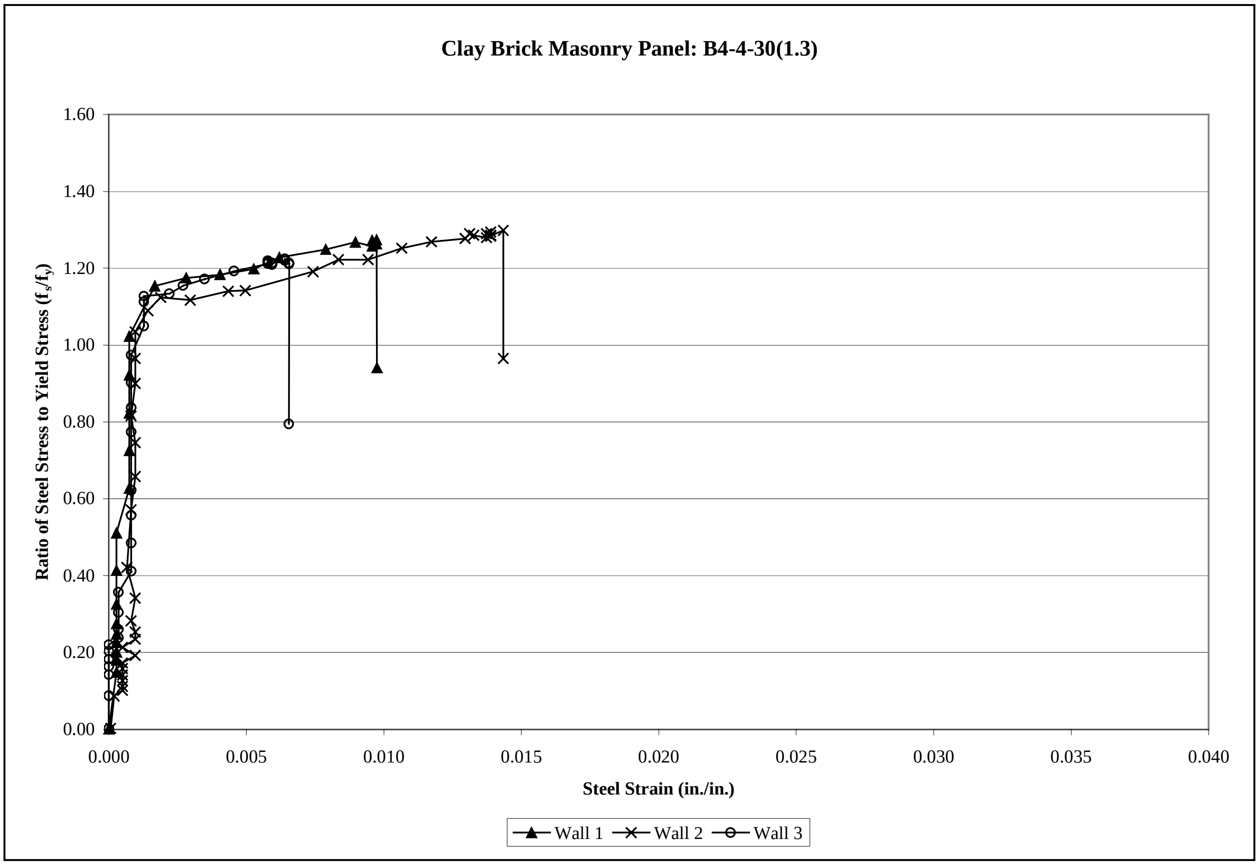
A.1 Results for Splice Length Research Panel Tests

A.1.1 4 inch Clay Masonry Panels

Panel Number	Average[1] Maximum Tension Force in Splice (lb)	Average[1] Maximum Stress in Reinforcement (psi)	Ratio of Average Maximum Stress in Reinforcement to f_y	Average[1] Maximum Steel Strain (in./in.)	Average[1,2] Maximum Displacement of Splice (in.)
B4-4-30 (1.3)-1	15,285	76,424	1.274	0.0098	0.1950
B4-4-30 (1.3)-2	15,580	77,901	1.298	0.0144	0.2870
B4-4-30 (1.3)-3	14,688	73,440	1.224	0.0066	0.1311
Average	15,184	75,922	1.265	0.0102	0.2044
B4-4-36 (1.0)-1	15,581	77,903	1.298	0.0113	0.2263
B4-4-36 (1.0)-2	15,376	76,881	1.281	0.0144	0.2879
B4-4-36 (1.0)-3	15,451	77,255	1.288	0.0127	0.2541
Average	15,469	77,347	1.289	0.0128	0.2561
B4-4-36 (1.3)-1	16,692	83,458	1.391	0.0194	0.3883
B4-4-36 (1.3)-2	16,742	83,711	1.395	0.0194	0.3884
B4-4-36 (1.3)-3	16,580	82,901	1.382	0.0144	0.2870
Average	16,671	83,356	1.389	0.0177	0.3546

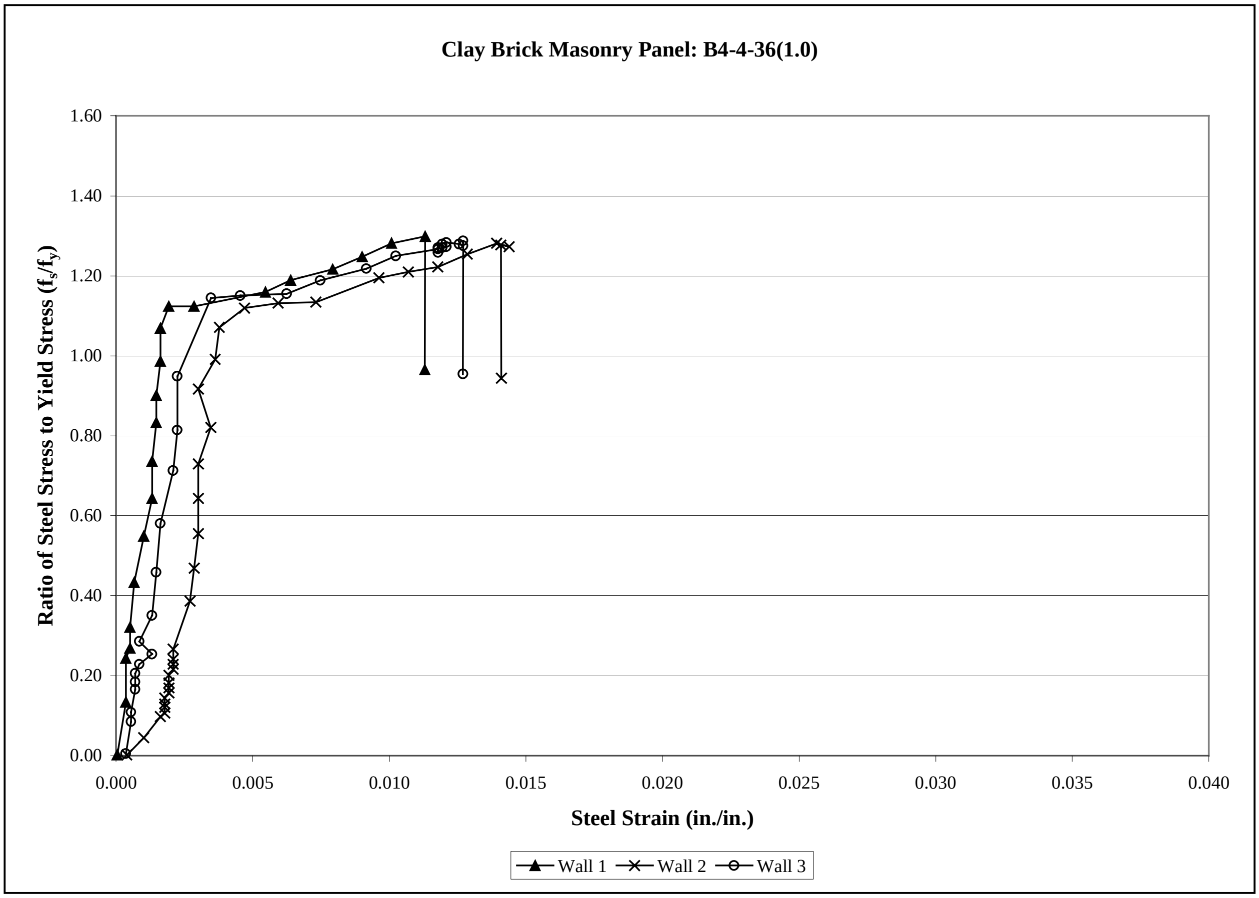
[1]Average value of two lap splices per panel.
[2]The displacements are the movement between the established gage points. The measured displacements include deformations of the bars and any slippage between the bars and the surrounding grout.

A.1.1.1 – 4 inch Brick Panels B4-4-30 (1.3)-1, 2, and 3



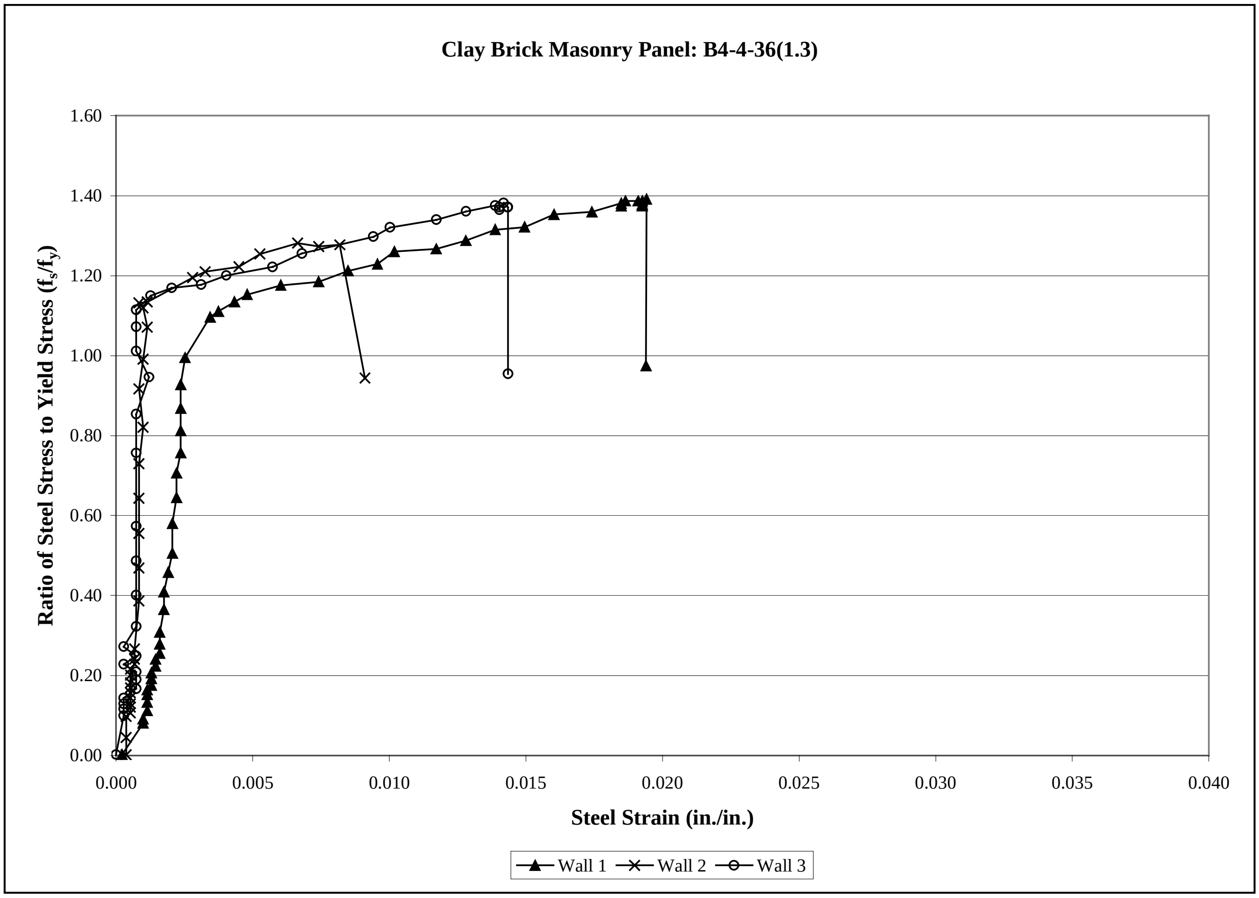
Specimen B4-4-30(1.3)	**Wall 1**	**Wall 2**	**Wall 3**	**Average**	
Average Maximum Load (lb.)	15,285	15,580	14,688	15,184	lb.
Average Maximum Stress in Reinforcement (psi)	76,424	77,901	73,440	75,922	psi
Average Ratio of Maximum Steel Stress to f_y	1.274	1.298	1.224	1.265	
Average Maximum Steel Strain (in./in.)	0.0098	0.0144	0.0066	0.0102	in./in.
Average Maximum Splice Displacement (in.)	0.1950	0.2870	0.1311	0.2044	in.

A.1.1.2 – 4 inch Brick Panels B4-4-36 (1.0)-1, 2, and 3



Specimen B4-4-36(1.0)	**Wall 1**	**Wall 2**	**Wall 3**	**Average**	
Average Maximum Load (lb.)	15,581	15,376	15,451	15,469	lb.
Average Maximum Stress in Reinforcement (psi)	77,903	76,881	77,255	77,347	psi
Average Ratio of Maximum Steel Stress to f_y	1.298	1.281	1.288	1.289	
Average Maximum Steel Strain (in./in.)	0.0113	0.0144	0.0127	0.0128	in./in.
Average Maximum Splice Displacement (in.)	0.2263	0.2879	0.2541	0.2561	in.

A.1.1.3 – 4 inch Brick Panels B4-4-36 (1.3)-1, 2, and 3



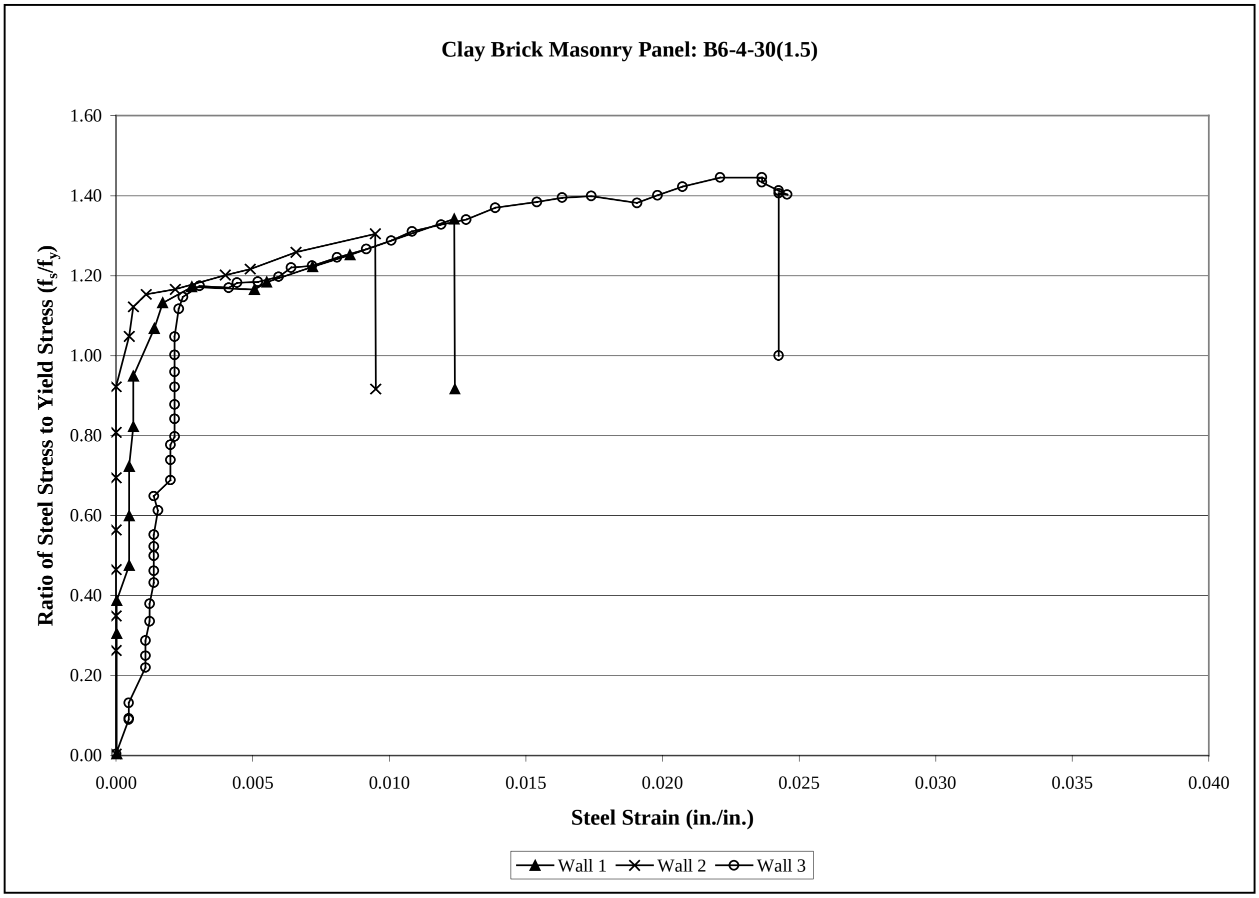
Specimen B4-4-36(1.3)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	16,692	16,742	16,580	16,671	lb.
Average Maximum Stress in Reinforcement (psi)	83,458	82,711	82,901	83,356	psi
Average Ratio of Maximum Steel Stress to f_y	1.391	1.395	1.382	1.389	
Average Maximum Steel Strain (in./in.)	0.0194	0.0194	0.0144	0.0177	in./in.
Average Maximum Splice Displacement (in.)	0.3883	0.3884	0.2870	0.3546	in.

A.1.2 – 6 inch Brick Masonry Panels

Panel Number	Average[1] Maximum Tension Force in Splice (lb)	Average[1] Maximum Stress in Reinforcement (psi)	Ratio of Average Maximum Stress in Reinforcement to f_y	Average[1] Maximum Steel Strain (in./in.)	Average[1,2] Maximum Displacement of Splice (in.)
B6-4-30 (1.5)-1	16,105	80,524	1.342	0.0124	0.2480
B6-4-30 (1.5)-2	15,658	78,288	1.305	0.0095	0.1900
B6-4-30 (1.5)-3	17,343	86,716	1.445	0.0246	0.4914
Average	16,369	81,843	1.364	0.0155	0.398
B6-4-36 (1.5)-1	18,550	92,749	1.546	0.0310	0.6200
B6-4-36 (1.5)-2	18,995	94,975	1.583	0.0324	0.6480
B6-4-36 (1.5)-3	17,129	85,645	1.427	0.0187	0.3743
Average	18,225	91,123	1.519	0.0274	0.5474
B6-5-40 (2)-1	25,238	81,414	1.357	0.0125	0.3510
B6-5-40 (2)-2	27,088	87,381	1.456	0.0234	0.6556
B6-5-40 (2)-3	25,281	81,551	1.359	0.0143	0.4010
Average	25,869	83,449	1.391	0.0168	0.4692
B6-5-48 (1.5)-1	24,746	79,827	1.330	0.0118	0.3791
B6-5-48 (1.5)-2	23,164	74,722	1.245	0.0065	0.2081
B6-5-48 (1.5)-3	24,924	80,402	1.340	0.0107	0.3421
Average	24,278	78,317	1.305	0.0097	0.3097
B6-6-48 (2)-1	39,237	89,175	1.486	0.0179	0.7170
B6-6-48 (2)-2	35,582	80,868	1.348	0.0156	0.6250
B6-6-48 (2)-3	39,312	89,346	1.489	0.0167	0.6681
Average	38,044	86,463	1.441	0.0168	0.6700
B6-6-48 (2.4)-1	36,386	82,695	1.378	0.0129	0.5170
B6-6-48 (2.4)-2	37,839	85,997	1.433	0.0195	0.7792
B6-6-48 (2.4)-3	36,945	83,965	1.4399	0.0216	0.8520
Average	37,056	84,219	1.404	0.0180	0.7194
B6-7-48 (2)-1	44,921	74,868	1.248	0.0050	0.2210
B6-7-48 (2)-2	46,852	78,086	1.301	0.0072	0.3162
B6-7-48 (2)-3	45,297	75,496	1.258	0.0054	0.2391
Average	45,690	76,150	1.269	0.0059	0.2588
B6-7-48 (2.4)-1	45,431	75,718	1.262	0.0072	0.3160
B6-7-48 (2.4)-2	48,511	80,852	1.348	0.0103	0.4550
B6-7-48 (2.4)-3	43,514	72,524	1.209	0.0045	0.1992
Average	45,819	76,365	1.273	0.0074	0.3234

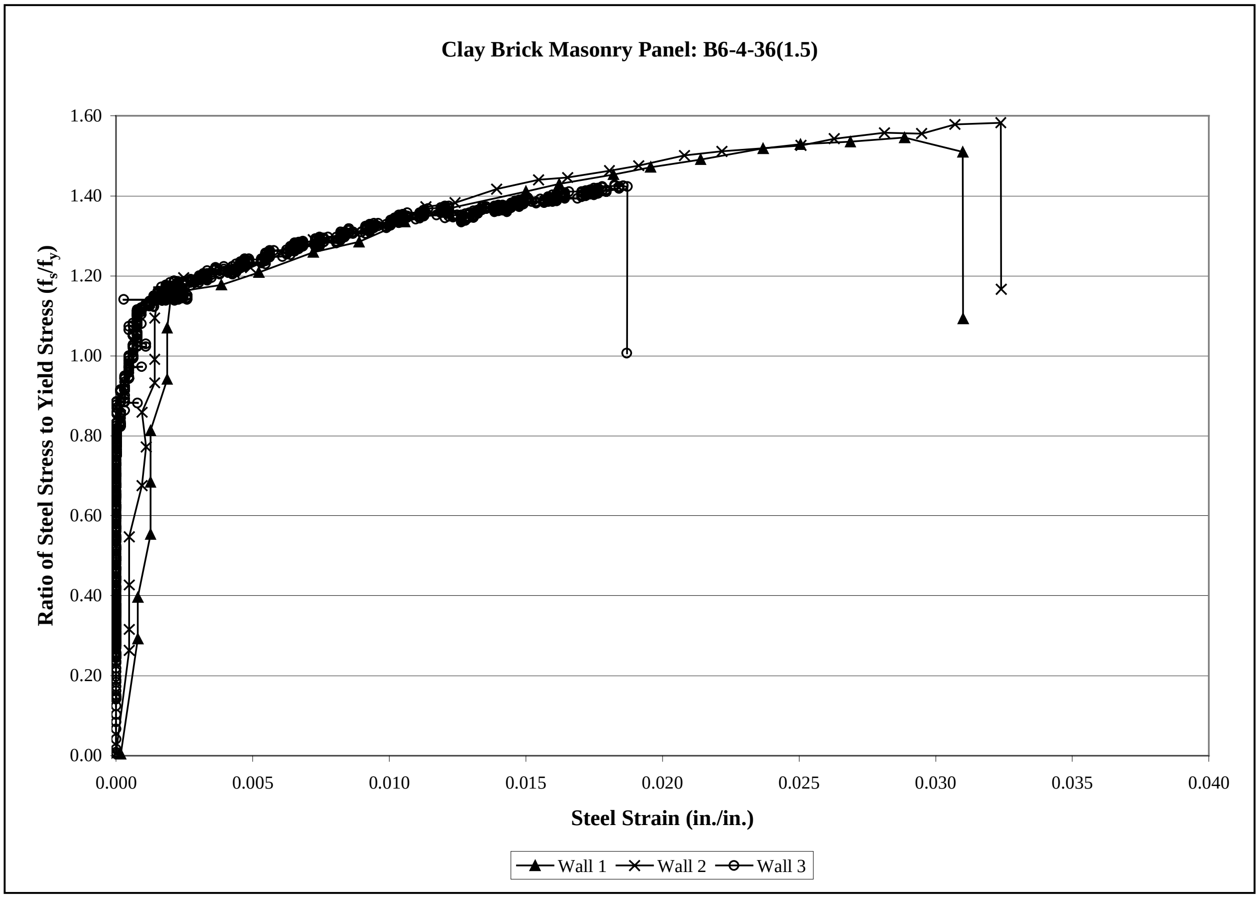
[1]Average value of two lap splices per panel.
[2]The displacements are the movement between the established gage points. The measured displacements include deformations of the bars and any slippage between the bars and the surrounding grout.

A.1.2.1 – 6 inch Brick Panels B6-4-30 (1.5)-1, 2, and 3



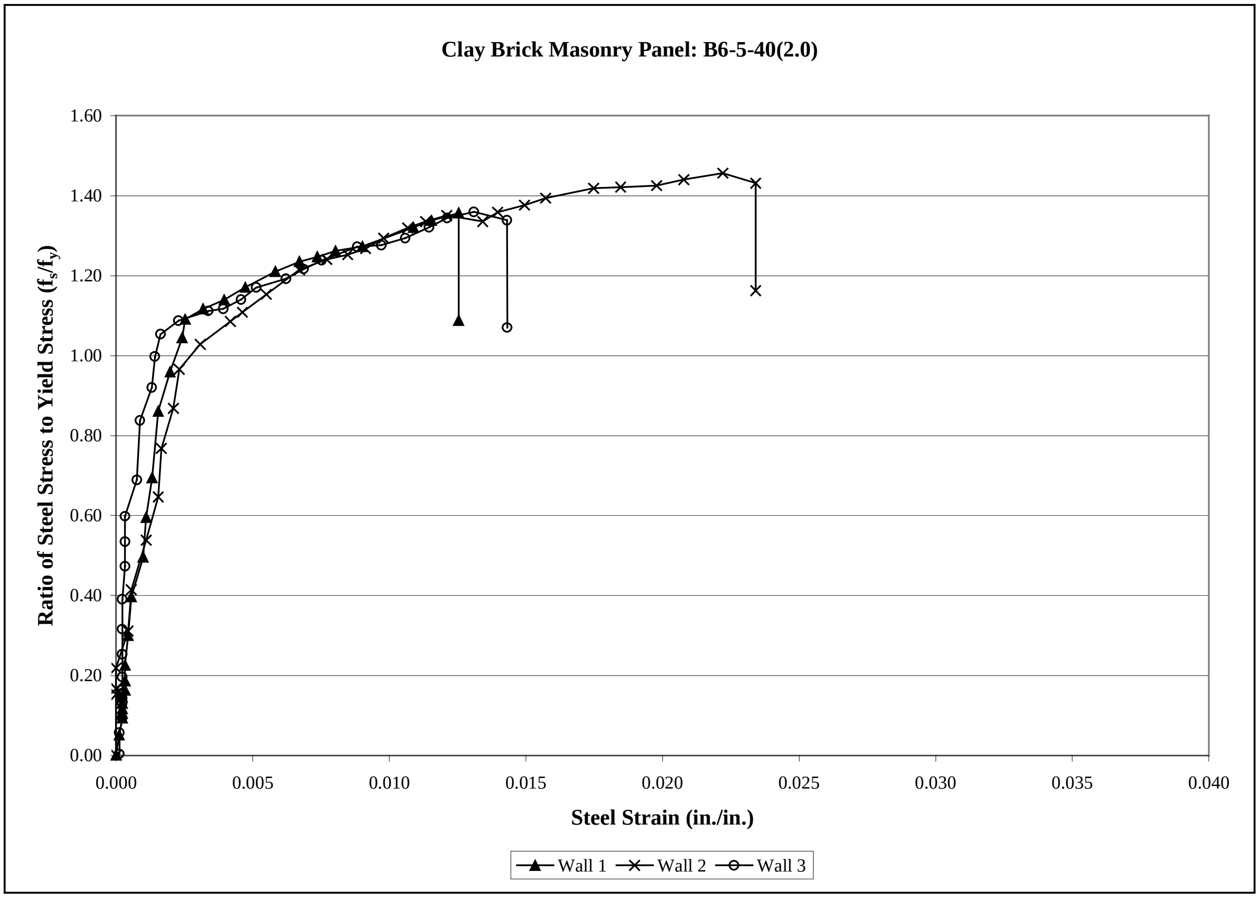
Specimen B6-4-30(1.5)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	16,105	15,658	17,343	16,369	lb.
Average Maximum Stress in Reinforcement (psi)	80,524	78,288	86,716	81,843	psi
Average Ratio of Maximum Steel Stress to f_y	1.342	1.305	1.445	1.364	
Average Maximum Steel Strain (in./in.)	0.0124	0.0095	0.0246	0.0155	in./in.
Average Maximum Splice Displacement (in.)	0.2480	0.1900	0.4914	0.3098	in.

A.1.2.2 – 6 inch Brick Panels B6-4-36 (1.5)-1, 2, and 3



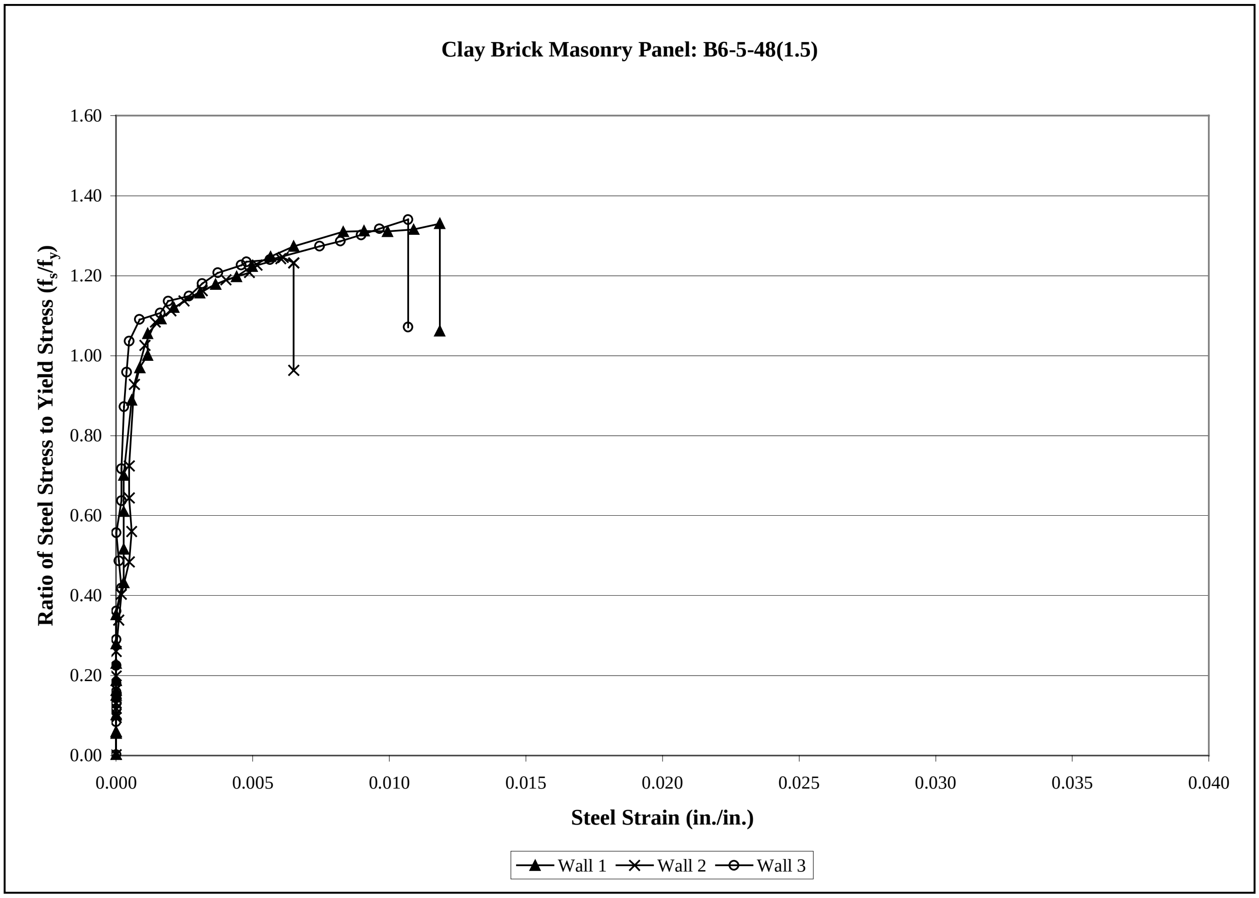
Specimen B6-4-36(1.5)	**Wall 1**	**Wall 2**	**Wall 3**	**Average**	
Average Maximum Load (lb.)	18,550	18,995	17,129	18,225	lb.
Average Maximum Stress in Reinforcement (psi)	92,749	94,975	85,645	91,123	psi
Average Ratio of Maximum Steel Stress to f_y	1.546	1.583	1.427	1.519	
Average Maximum Steel Strain (in./in.)	0.0310	0.0324	0.0187	0.0274	in./in.
Average Maximum Splice Displacement (in.)	0.6200	0.6480	0.3743	0.5474	in.

A.1.2.3 – 6 inch Brick Panels B6-5-40 (2.0)-1, 2, and 3



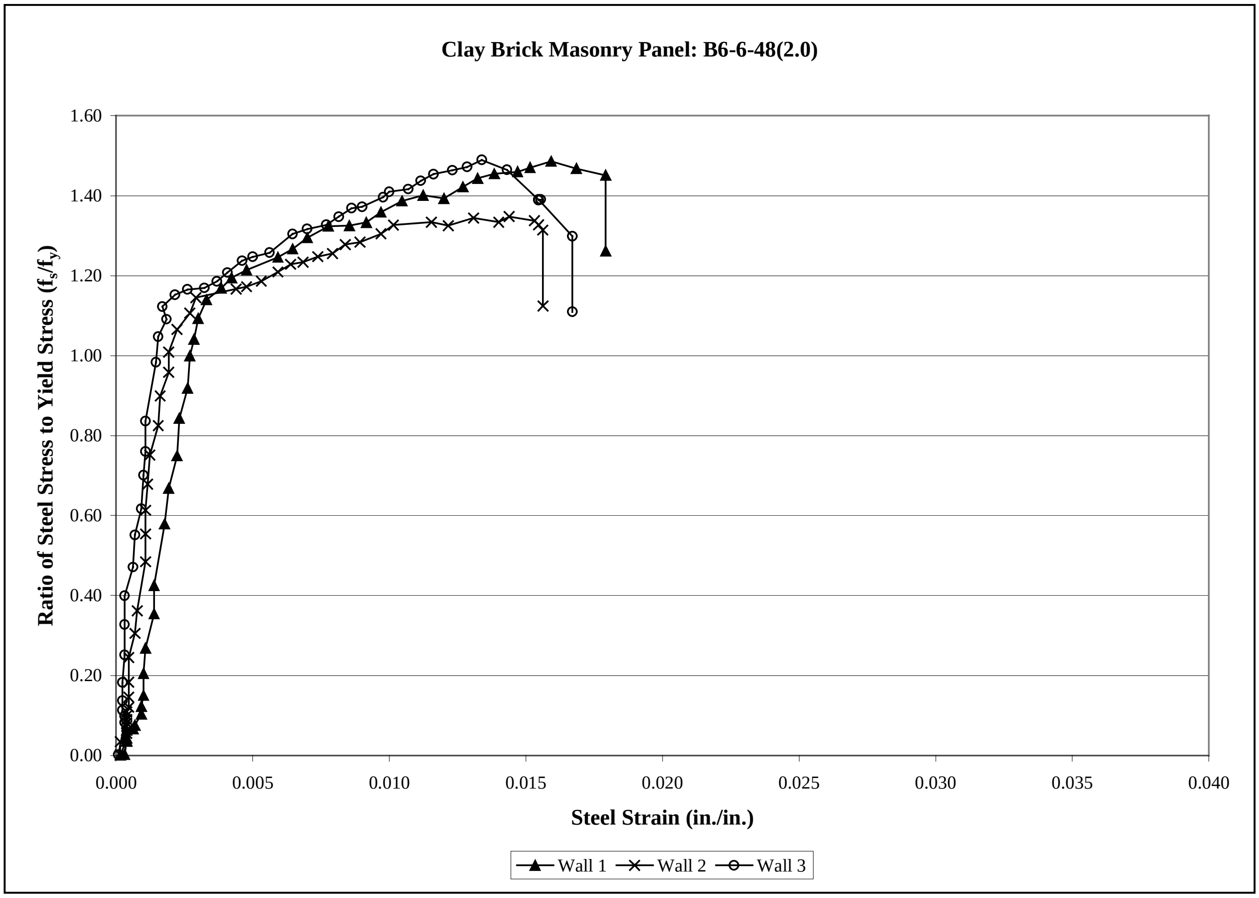
Specimen B6-5-40(2.0)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	25,238	27,088	25,281	25,869	lb.
Average Maximum Stress in Reinforcement (psi)	81,414	87,381	81,551	83,449	psi
Average Ratio of Maximum Steel Stress to f_y	1.357	1.456	1.359	1.391	
Average Maximum Steel Strain (in./in.)	0.0125	0.0234	0.0143	0.0168	in./in.
Average Maximum Splice Displacement (in.)	0.3510	0.6556	0.4010	0.4692	in.

A.1.2.4 – 6 inch Brick Panels B6-5-48 (1.5)-1, 2, and 3



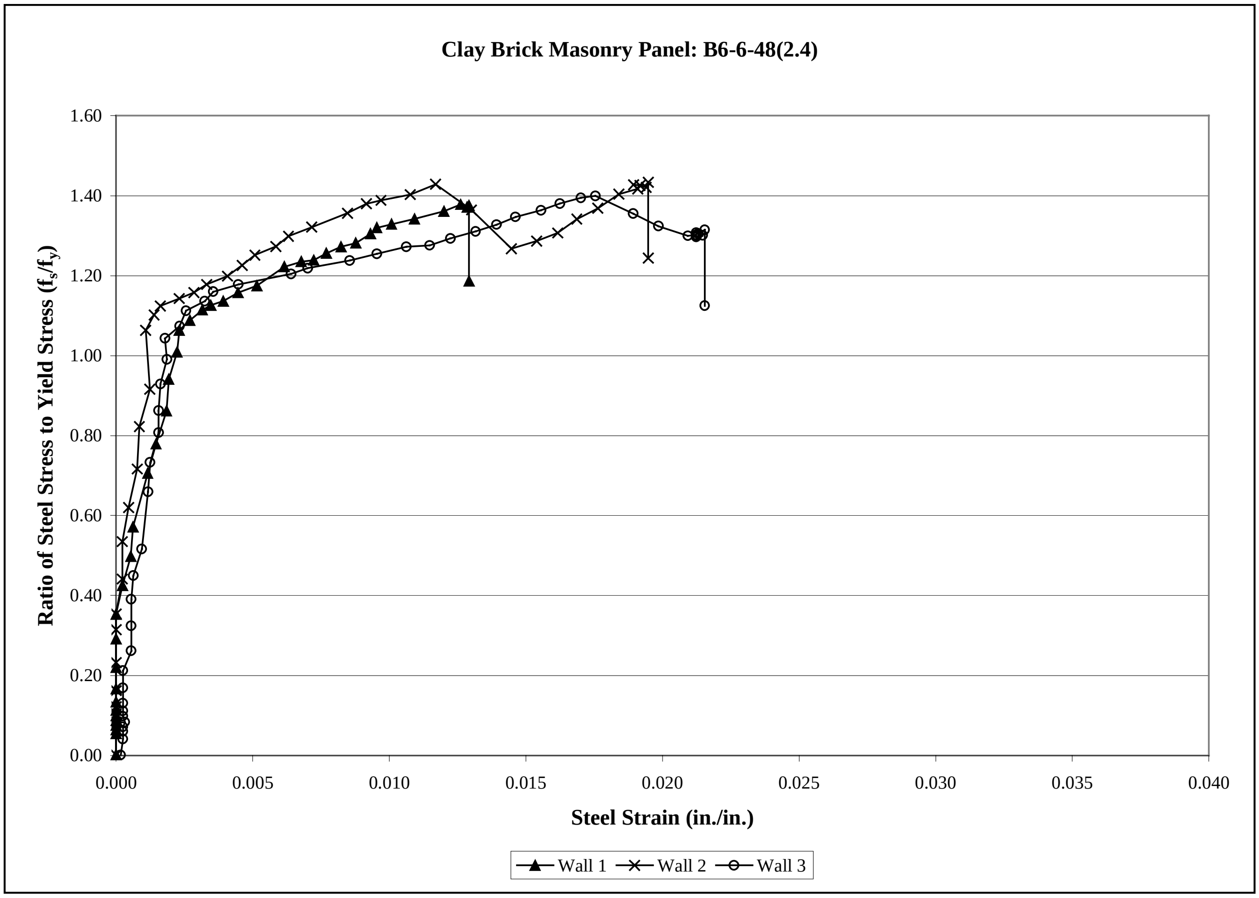
Specimen B6-5-48(1.5)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	24,746	23,164	24,924	24,278	lb.
Average Maximum Stress in Reinforcement (psi)	79,827	74,722	80,402	78,317	psi
Average Ratio of Maximum Steel Stress to f_y	1.330	1.245	1.340	1.305	
Average Maximum Steel Strain (in./in.)	0.0118	0.0065	0.0107	0.0097	in./in.
Average Maximum Splice Displacement (in.)	0.3791	0.2081	0.3421	0.3097	in.

A.1.2.5 – 6 inch Brick Panels B6-6-48 (2.0)-1, 2, and 3



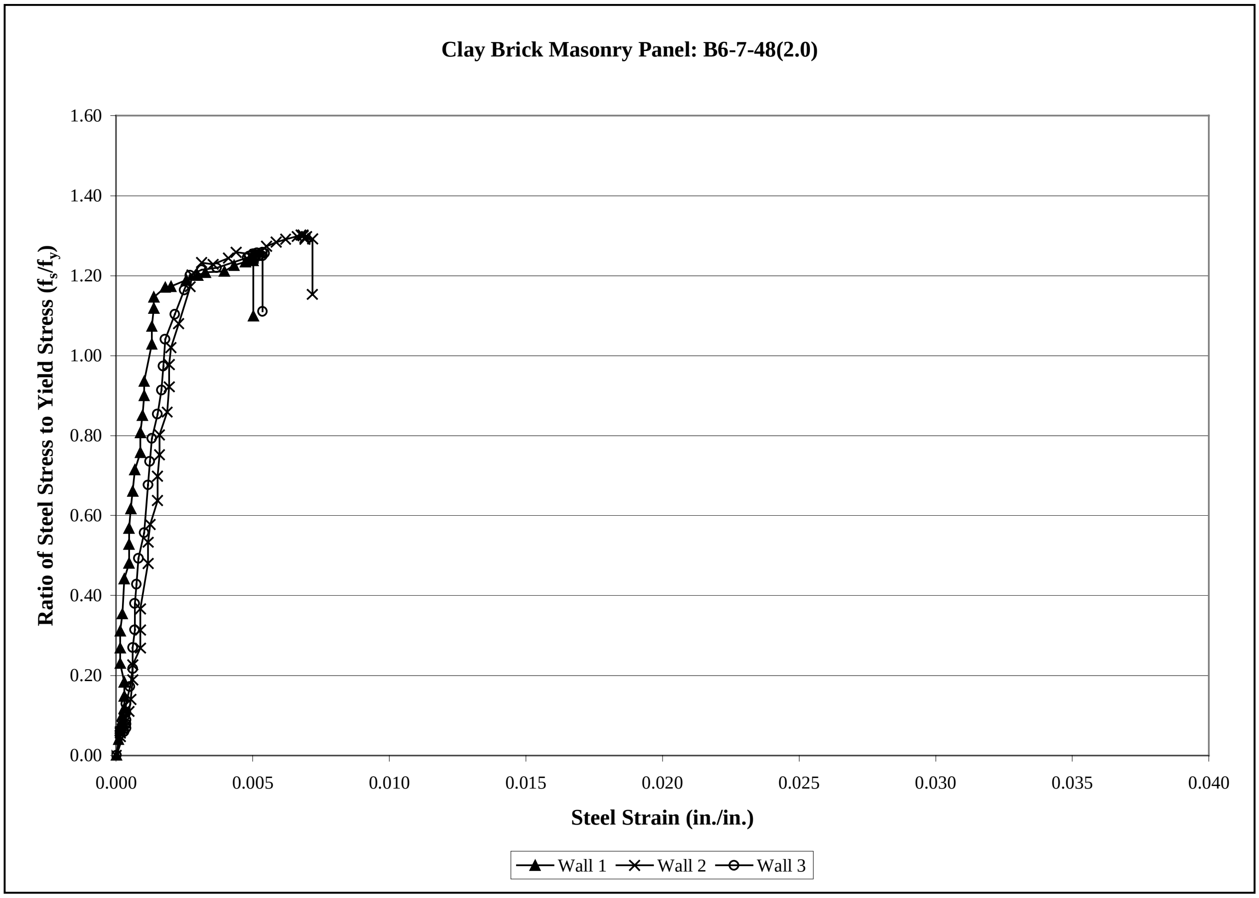
Specimen B6-6-48(2.0)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	39,237	35,582	39,312	38,044	lb.
Average Maximum Stress in Reinforcement (psi)	89,175	80,869	89,346	86,463	psi
Average Ratio of Maximum Steel Stress to f_y	1.486	1.348	1.489	1.441	
Average Maximum Steel Strain (in./in.)	0.0179	0.0156	0.0167	0.0168	in./in.
Average Maximum Splice Displacement (in.)	0.7170	0.6250	0.6681	0.6700	in.

A.1.2.6 – 6 inch Brick Panels B6-6-48 (2.4)-1, 2, and 3



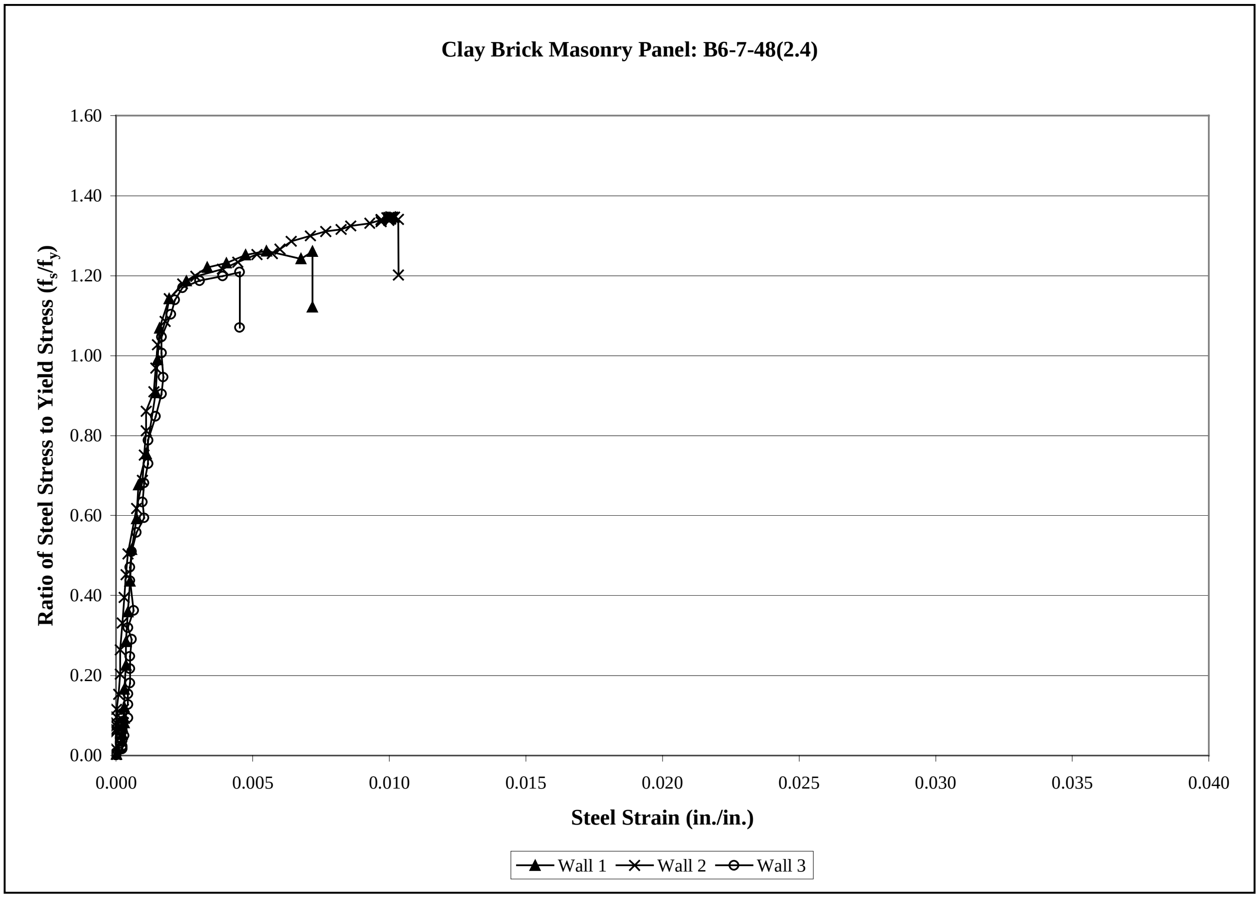
Specimen B6-6-48(2.4)	**Wall 1**	**Wall 2**	**Wall 3**	**Average**	
Average Maximum Load (lb.)	36,386	37,839	36,945	37,056	lb.
Average Maximum Stress in Reinforcement (psi)	82,694	85,997	83,965	84,219	psi
Average Ratio of Maximum Steel Stress to f_y	1.378	1.433	1.399	1.404	
Average Maximum Steel Strain (in./in.)	0.0129	0.0195	0.0216	0.0180	in./in.
Average Maximum Splice Displacement (in.)	0.5170	0.7792	0.8620	0.7194	in.

A.1.2.7 – 6 inch Brick Panels B6-7-48 (2.0)-1, 2, and 3



Specimen B6-7-48(2.0)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	44,921	46,852	45,297	45,690	lb.
Average Maximum Stress in Reinforcement (psi)	74,868	78,086	75,496	76,150	psi
Average Ratio of Maximum Steel Stress to f_y	1.248	1.301	1.258	1.269	
Average Maximum Steel Strain (in./in.)	0.0050	0.0072	0.0054	0.0059	in./in.
Average Maximum Splice Displacement (in.)	0.2210	0.3162	0.2391	0.2588	in.

A.1.2.8 – 6 inch Brick Panels B6-7-48 (2.4)-1, 2, and 3



Specimen B6-7-48(2.4)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	45,431	48,511	43,514	45,819	lb.
Average Maximum Stress in Reinforcement (psi)	75,718	80,852	72,524	76,365	psi
Average Ratio of Maximum Steel Stress to f_y	1.262	1.348	1.209	1.273	
Average Maximum Steel Strain (in./in.)	0.0072	0.0103	0.0045	0.0074	in./in.
Average Maximum Splice Displacement (in.)	0.3160	0.4550	0.1992	0.3234	in.

A.1.3 – 8 inch Concrete Masonry Panels—Phase I

Panel Number	Average[1] Maximum Tension Force in Splice (lb.)	Average[1] Maximum Stress in Reinforcement (psi)	Ratio of Average Maximum Stress in Reinforcement to f_y	Average[1] Maximum Steel Strain (in./in.)	Average[1] Maximum Displacement of Splice (in.)
1C8-4-36 (2)-1	20,283	101,415	1.690	N.M.	N.M.
1C8-4-36 (2)-2	19,925	99,625	1.660	N.M.	N.M.
1C8-4-36 (2)-3	20,164	100,820	1.680	N.M.	N.M.
Average	20,124	100,620	1.677	N.M.	N.M.
1C8-5-48 (2)-1	29,953	96,623	1.610	N.M.	N.M.
1C8-5-48 (2)-2	28,397	91,603	1.527	N.M.	N.M.
1C8-5-48 (2)-3	28,178	90,897	1.515	N.M.	N.M.
Average	28,843	93,041	1.551	N.M.	N.M.
1C8-6-48 (2)-1	39,536	89,855	1.498	N.M.	N.M.
1C8-6-48 (2)-2	37,845	86,011	1.434	N.M.	N.M.
1C8-6-48 (2)-3	37,964	86,282	1.438	N.M.	N.M.
Average	38,448	87,383	1.457	N.M.	N.M.
1C8-7-48 (2)-1	47,216	78,693	1.312	N.M.	N.M.
1C8-7-48 (2)-2	48,846	81,410	1.357	N.M.	N.M.
1C8-7-48 (2)-3	48,846	81,410	1.357	N.M.	N.M.
Average	48,303	80,504	1.342	N.M.	N.M.
1C8-8-48 (2)-1	57,707	73,047	1.217	N.M.	N.M.
1C8-8-48 (2)-2	56,911	72,039	1.201	N.M.	N.M.
1C8-8-48 (2)-3	59,693	75,561	1.259	N.M.	N.M.
Average	58,104	73,549	1.226	N.M.	N.M.
1C8-9-48 (2)-1	64,454	64,454	1.074	N.M.	N.M.
1C8-9-48 (2)-2	64,733	64,733	1.079	N.M.	N.M.
1C8-9-48 (2)-3	67,523	67,523	1.125	N.M.	N.M.
Average	65,570	65,570	1.093	N.M.	N.M.

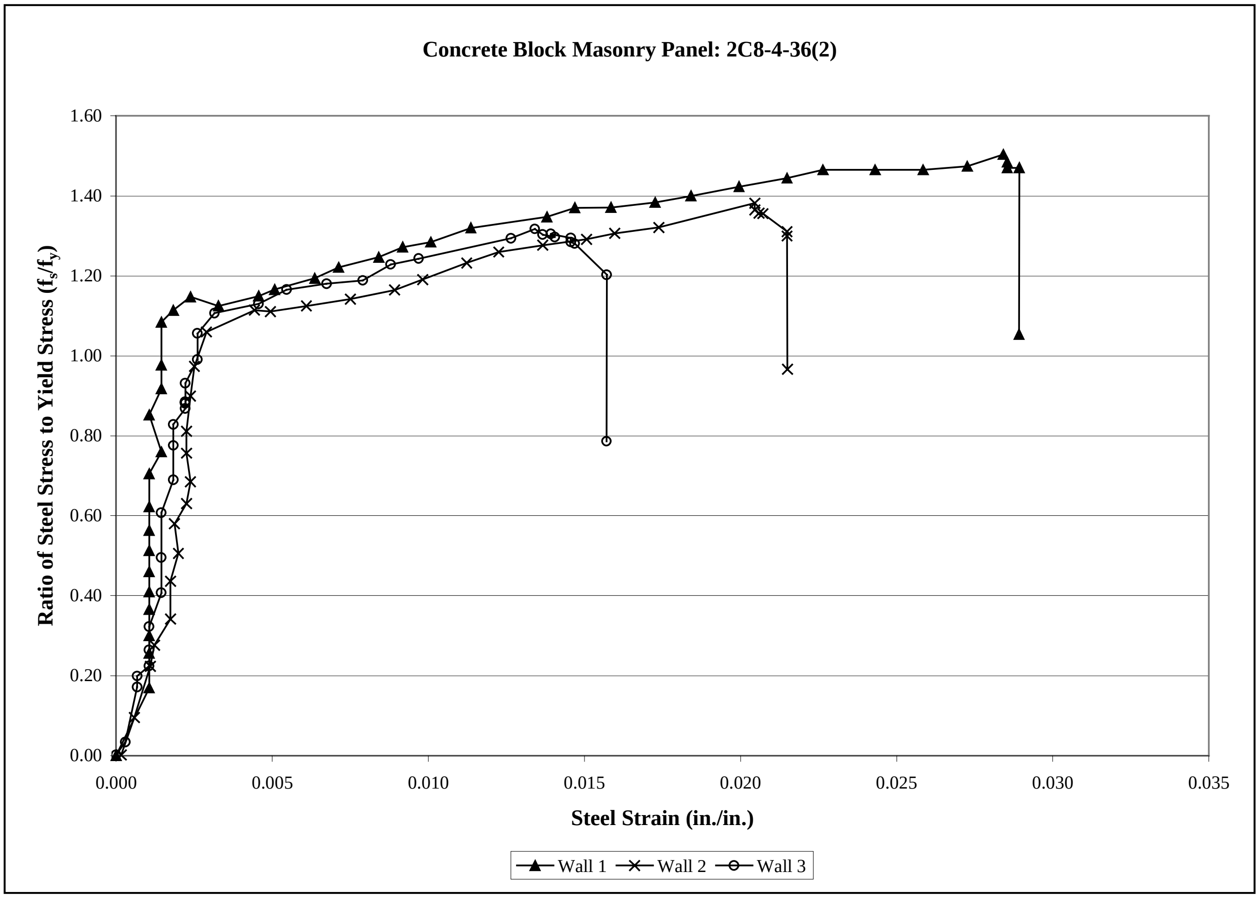
N.M. - displacements not measured for Phase I concrete masonry panels.
[1]Average value of two lap splices per panel.

A.1.4 Concrete Masonry Panels—Phase II

Panel Number	Average[1] Maximum Tension Force in Splice (lb.)	Average[1] Maximum Stress in Reinforcement (psi)	Ratio of Average Maximum Stress in Reinforcement to f_y	Average[1] Maximum Steel Strain (in./in.)	Average[1,2] Maximum Displacement of Splice (in.)
2C8-4-36 (2)-1	18,040	90,199	1.503	0.0289	0.6942
2C8-4-36 (2)-2	16,582	82,912	1.382	0.0215	0.5160
2C8-4-36 (2)-3	15,805	79,026	1.317	0.0157	0.3772
Average	16,809	84,046	1.401	0.0220	0.5291
2C8-5-48 (2)-1	21,565	69,566	1.159	0.0072	0.2294
2C8-5-48 (2)-2	23,852	76,943	1.282	0.0100	0.3190
2C8-5-48 (2)-3	-----	-----	-----	-----	-----
Average	22,709	73,255	1.221	0.0086	0.2742
2C8-6-48 (2)-1	31,608	71,836	1.197	0.0048	0.1930
2C8-6-48 (2)-2	31,310	71,159	1.186	0.0044	0.1750
2C8-6-48 (2)-3	27,979	63,588	1.060	0.0023	0.0920
Average	30,299	68,861	1.148	0.0038	0.1533
2C8-6-48 (3)-1	32,944	74,873	1.248	0.0054	0.2170
2C8-6-48 (3)-2	31,725	72,103	1.202	0.0073	0.2914
2C8-6-48 (3)-3	34,864	79,237	1.321	0.0130	0.5191
Average	33,178	75,404	1.257	0.0086	0.3425
2C8-7-48 (2)-1	39,543	65,906	1.098	0.0025	0.1222
2C8-7-48 (2)-2	34,215	57,024	0.950	0.0013	0.0640
2C8-7-48 (2)-3	37,410	62,350	1.039	0.0014	0.0670
Average	37,056	61,760	1.029	0.0018	0.0844
2C8-7-48 (3)-1	45,271	75,452	1.258	0.0055	0.2640
2C8-7-48 (3)-2	43,395	72,325	1.205	0.0031	0.1500
2C8-7-48 (3)-3	44,045	73,408	1.223	0.0050	0.2423
Average	44,237	73,728	1.229	0.0046	0.2188
2C8-8-48 (3)-1	34,261	43,369	0.723	0.0009	0.0520
2C8-8-48 (3)-2	43,146	54,615	0.910	0.0010	0.0577
2C8-8-48 (3)-3	37,787	47,832	0.797	0.0014	0.0761
Average	38,398	48,605	0.810	0.0011	0.0619
2C8-8-64 (2)-1	-----	-----	-----	-----	-----
2C8-8-64 (2)-2	50,699	64,177	1.070	0.0020	0.1407
2C8-8-64 (2)-3	52,343	66,257	1.104	0.0026	0.1900
Average	51,521	65,217	1.087	0.0023	0.1653
2C8-9-48 (3)-1	53,996	53,996	0.900	0.0014	0.0761
2C8-9-48 (3)-2	50,155	50,155	0.836	0.0017	0.0950
2C8-9-48 (3)-3	38,329	38,329	0.639	0.0006	0.0330
Average	47,493	47,493	0.792	0.0012	0.0680
2C8-9-64 (2)-1	49,200	49,200	0.820	0.0011	0.0884
2C8-9-64 (2)-2	61,120	61,120	1.019	0.0018	0.1470
2C8-9-64 (2)-3	58,643	58,643	0.977	0.0021	0.1654
Average	56,321	56,321	0.939	0.0017	0.1336

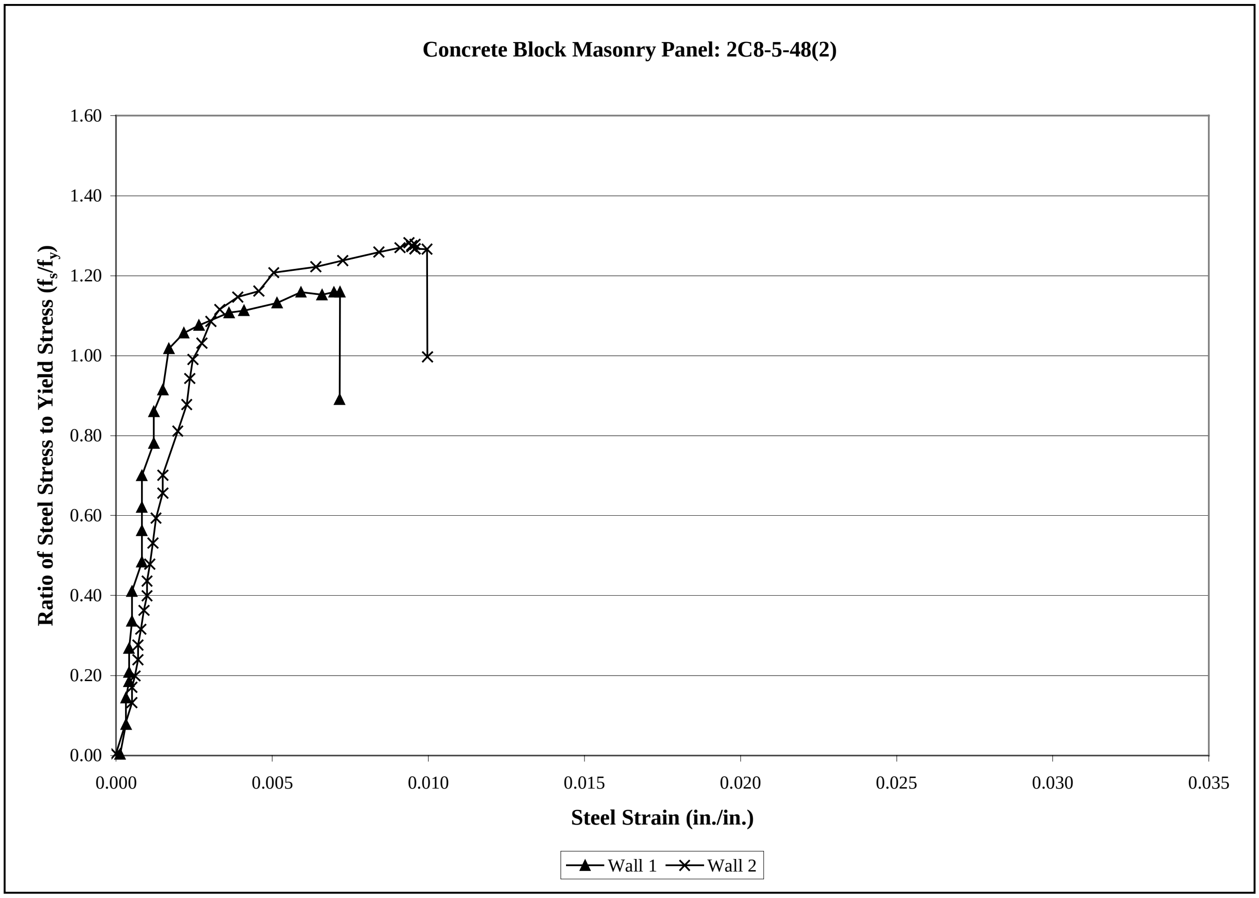
[1]Average value of two lap splices per panel.
[2]The displacements are the movement between the established gage points. The measured displacements include deformations of the bars and any slippage between the bars and the surrounding grout.
Note: Some entries are blank due to equipment failure that prevented reporting the results of these specimens.

A.1.4.1 – 8 inch Concrete Masonry Panels 2C8-4-36(2)-1, 2, and 3



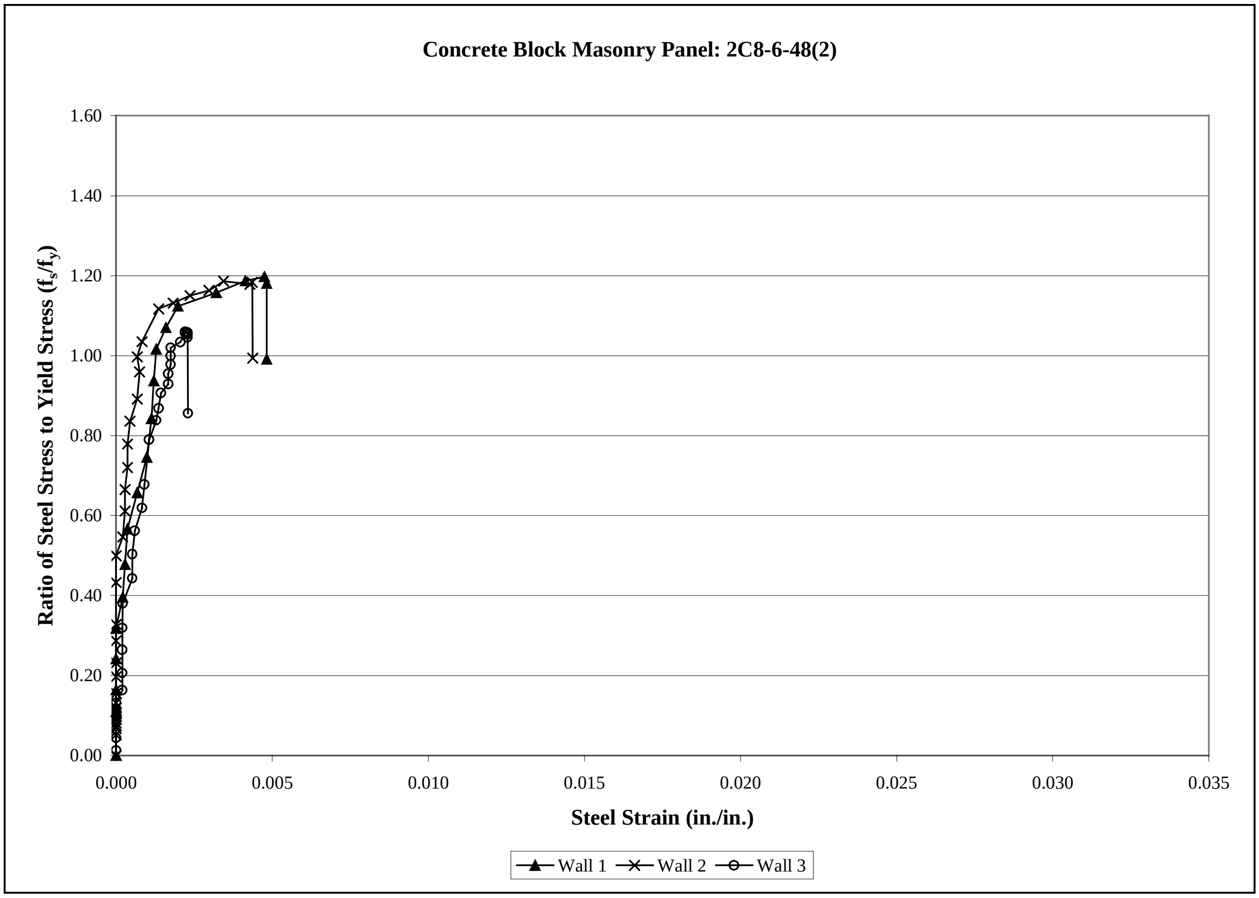
Specimen 2C8-4-36(2)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	18,040	16,582	15,805	16,809	lb.
Average Maximum Stress in Reinforcement (psi)	90,199	82,912	79,026	84,046	psi
Average Ratio of Maximum Steel Stress to f_y	1.503	1.382	1.317	1.401	
Average Maximum Steel Strain (in./in.)	0.0289	0.0215	0.0157	0.0220	in./in.
Average Maximum Splice Displacement (in.)	0.6942	0.5160	0.3772	0.5291	in.

A.1.4.2 – 8 inch Concrete Masonry Panels 2C8-5-48(2) -1, 2, and 3



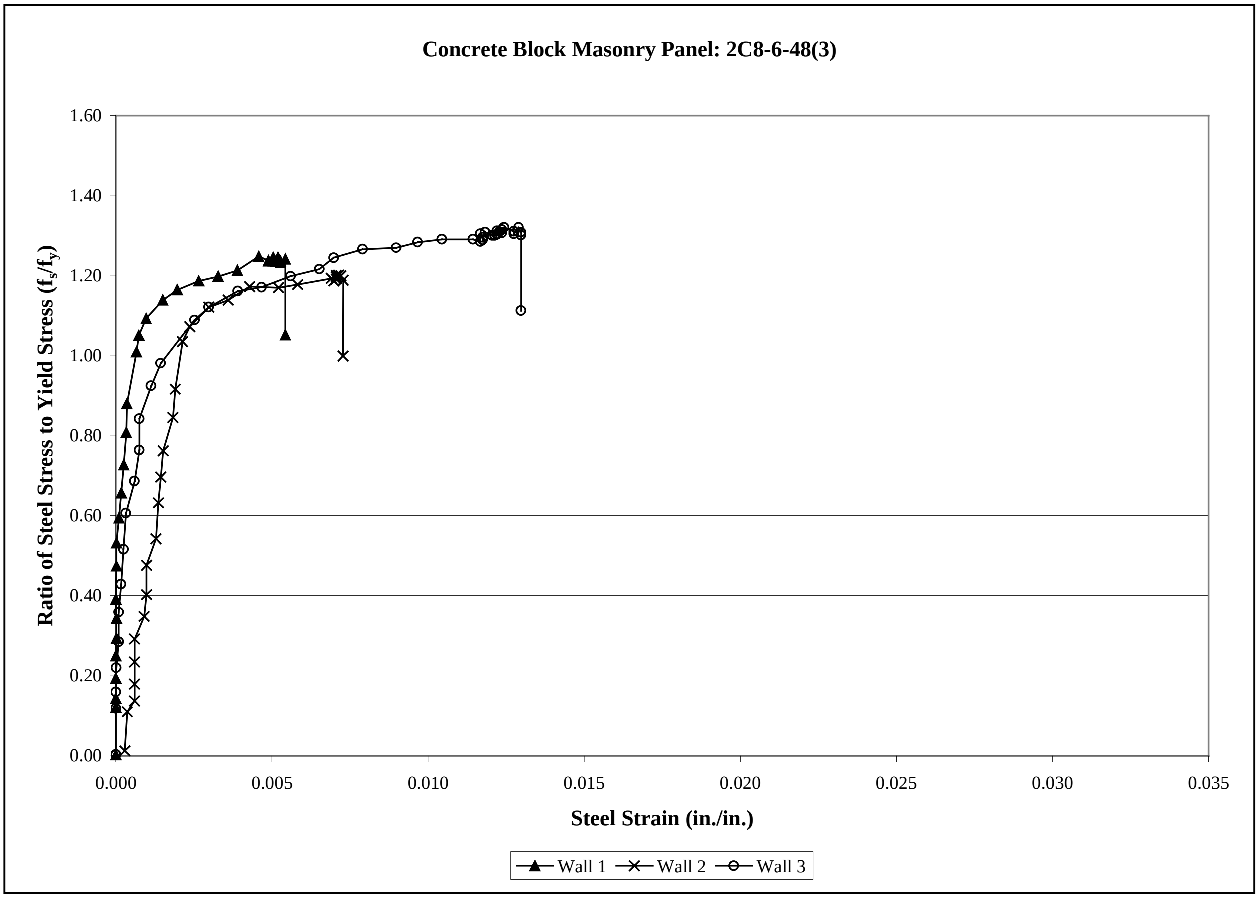
Specimen 2C8-5-48(2)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	21,565	23,852	N.A.	22,709	lb.
Average Maximum Stress in Reinforcement (psi)	69,566	76,943	N.A.	73,255	psi
Average Ratio of Maximum Steel Stress to f_y	1.159	1.282	N.A.	1.221	
Average Maximum Steel Strain (in./in.)	0.0072	0.0100	N.A.	0.0086	in./in.
Average Maximum Splice Displacement (in.)	0.2294	0.3190	N.A.	0.2742	in.

A.1.4.3 – 8 inch Concrete Masonry Panels 2C8-6-48(2) -1, 2, and 3



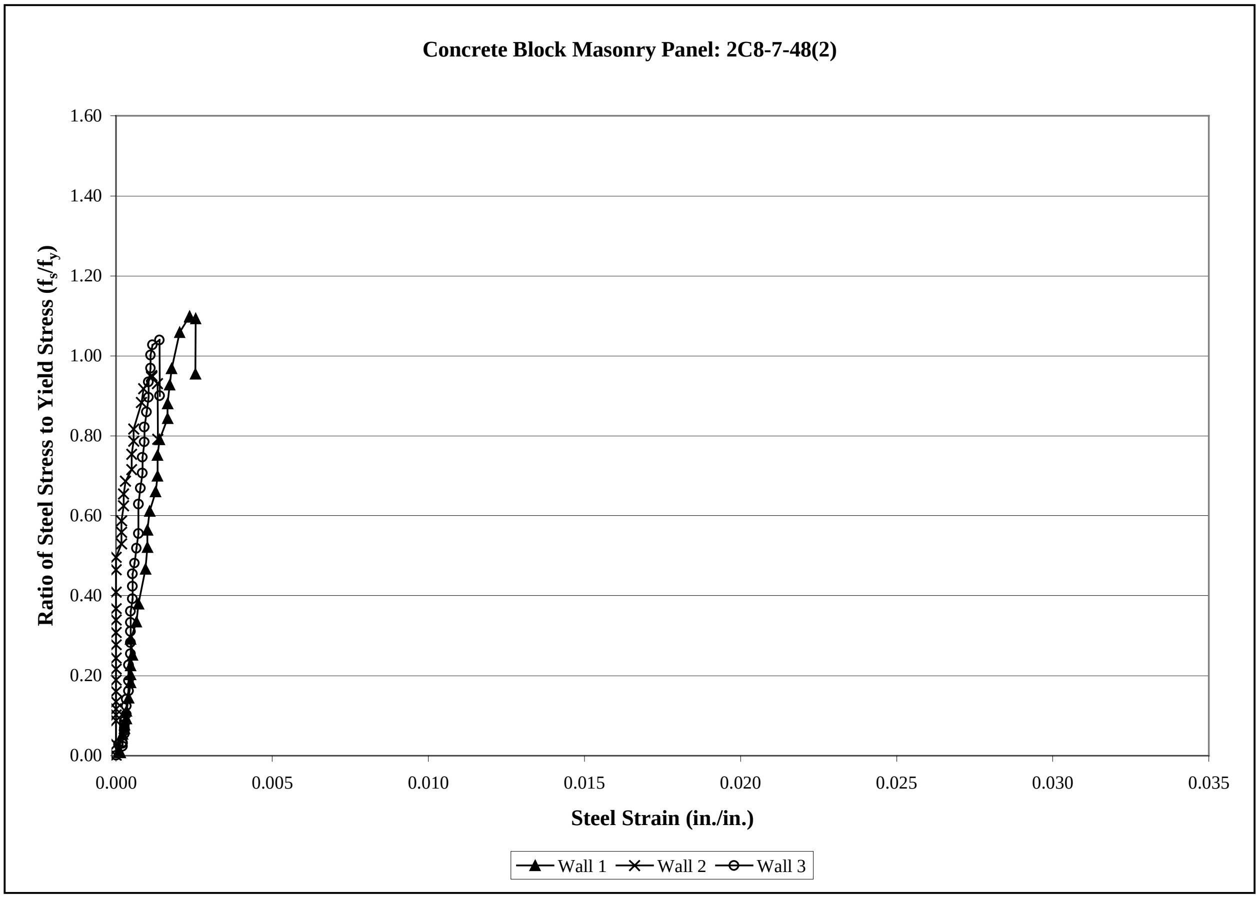
Specimen 2C8-6-48(2)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	31,608	31,310	27,979	30,299	lb.
Average Maximum Stress in Reinforcement (psi)	71,836	71,159	63,588	68,861	psi
Average Ratio of Maximum Steel Stress to f_y	1.197	1.186	1.060	1.148	
Average Maximum Steel Strain (in./in.)	0.0048	0.0044	0.0023	0.0038	in./in.
Average Maximum Splice Displacement (in.)	0.1930	0.1750	0.0920	0.1533	in.

A.1.4.4 – 8 inch Concrete Masonry Panels 2C8-6-48(3) -1, 2, and 3



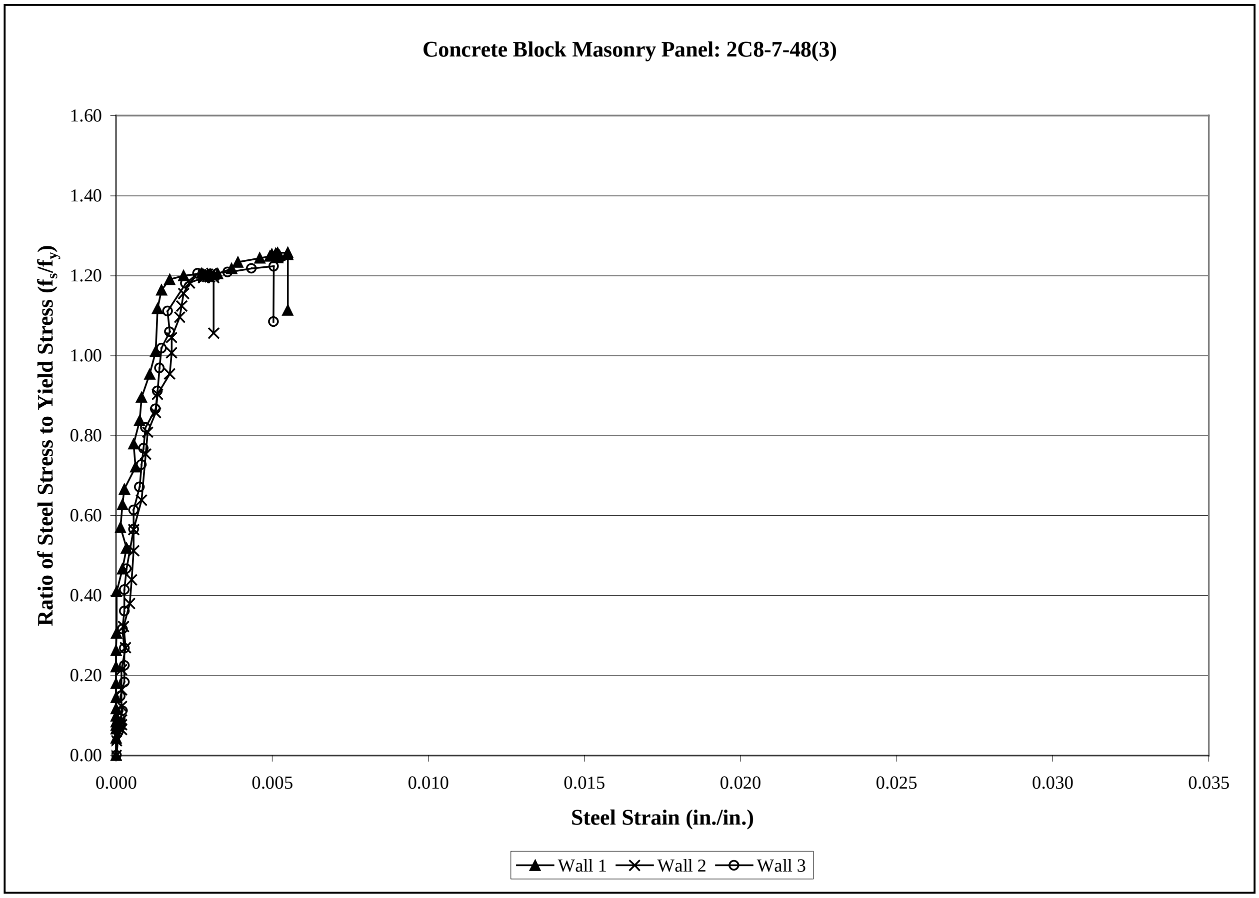
Specimen 2C8-6-48(3)	**Wall 1**	**Wall 2**	**Wall 3**	**Average**	
Average Maximum Load (lb.)	32,944	31,725	34,864	33,178	lb.
Average Maximum Stress in Reinforcement (psi)	74,873	72,103	79,237	75,404	psi
Average Ratio of Maximum Steel Stress to f_y	1.248	1.202	1.321	1.257	
Average Maximum Steel Strain (in./in.)	0.0054	0.0073	0.0130	0.0086	in./in.
Average Maximum Splice Displacement (in.)	0.2170	0.2914	0.5191	0.3425	in.

A.1.4.5 – 8 inch Concrete Masonry Panels 2C8-7-48(2) -1, 2, and 3



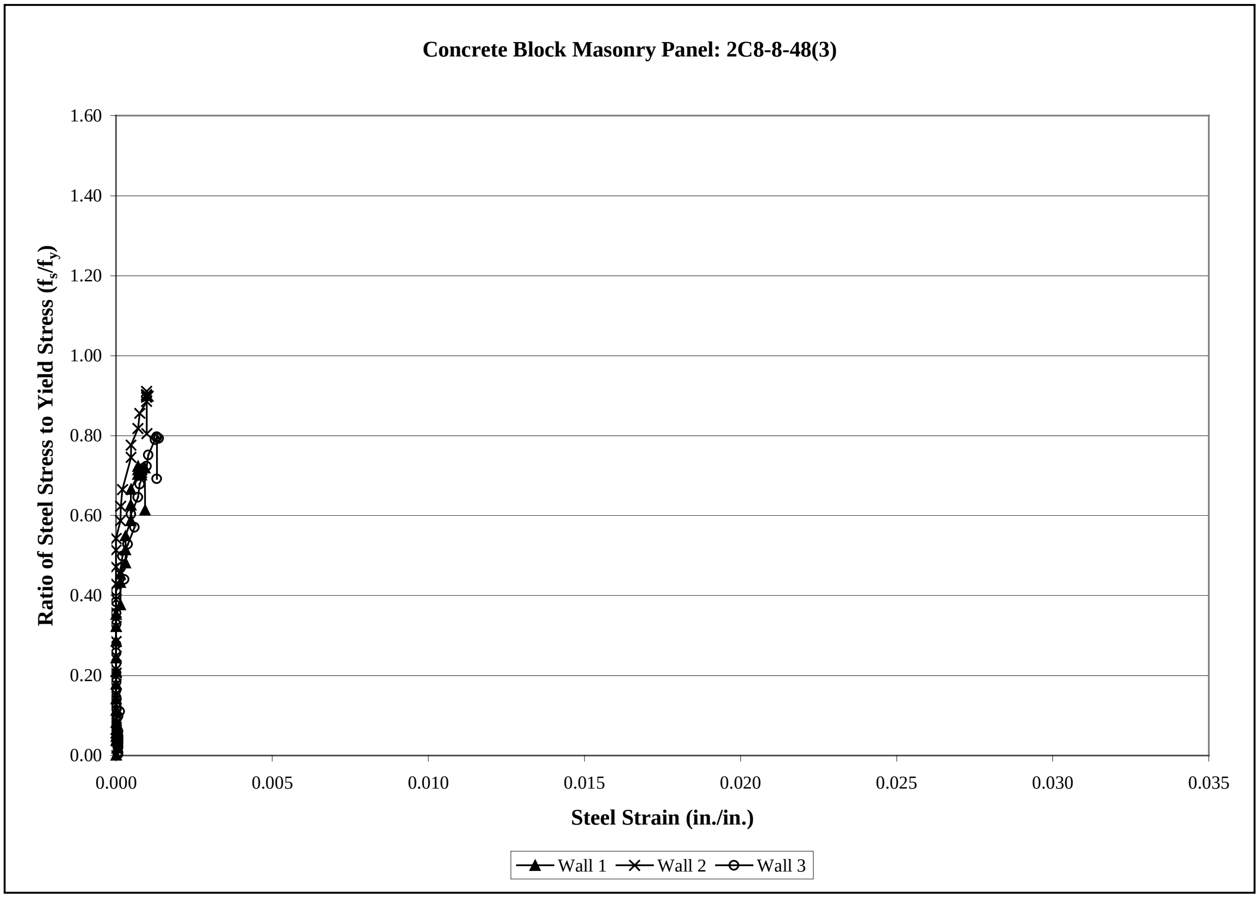
Specimen 2C8-7-48(2)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	39,543	34,215	37,410	37,056	lb.
Average Maximum Stress in Reinforcement (psi)	65,906	57,024	62,350	61,760	psi
Average Ratio of Maximum Steel Stress to f_y	1.098	0.950	1.039	1.029	
Average Maximum Steel Strain (in./in.)	0.0025	0.0013	0.0014	0.0018	in./in.
Average Maximum Splice Displacement (in.)	0.1222	0.0640	0.0670	0.0844	in.

A.1.4.6 – 8 inch Concrete Masonry Panels 2C8-7-48(3) -1, 2, and 3



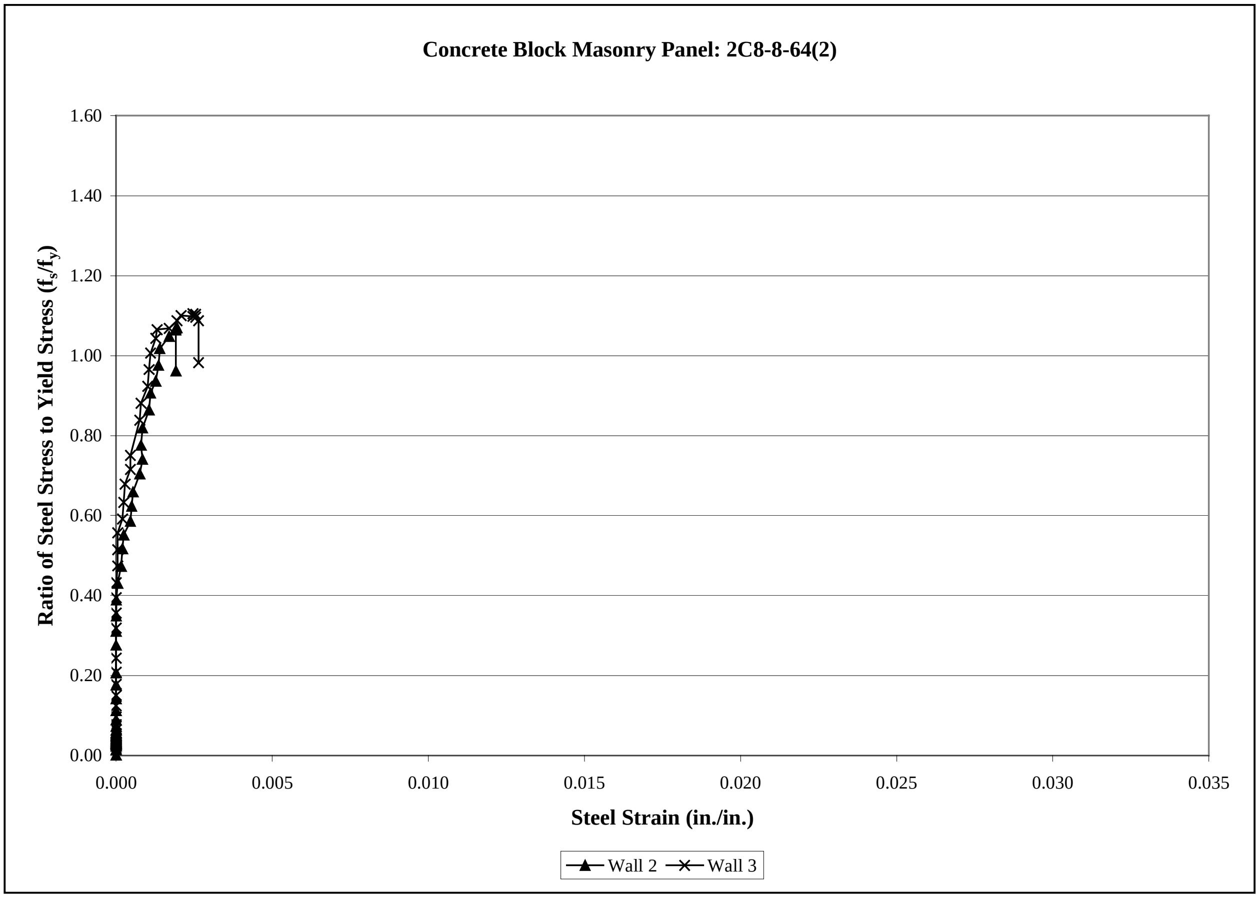
Specimen 2C8-7-48(3)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	45,271	43,395	44,045	44,237	lb.
Average Maximum Stress in Reinforcement (psi)	75,452	72,325	73,408	73,728	psi
Average Ratio of Maximum Steel Stress to f_y	1.258	1.205	1.223	1.229	
Average Maximum Steel Strain (in./in.)	0.0055	0.0031	0.0050	0.0046	in./in.
Average Maximum Splice Displacement (in.)	0.2640	0.1500	0.2423	0.2188	in.

A.1.4.7 – 8 inch Concrete Masonry Panels 2C8-8-48(3) -1, 2, and 3



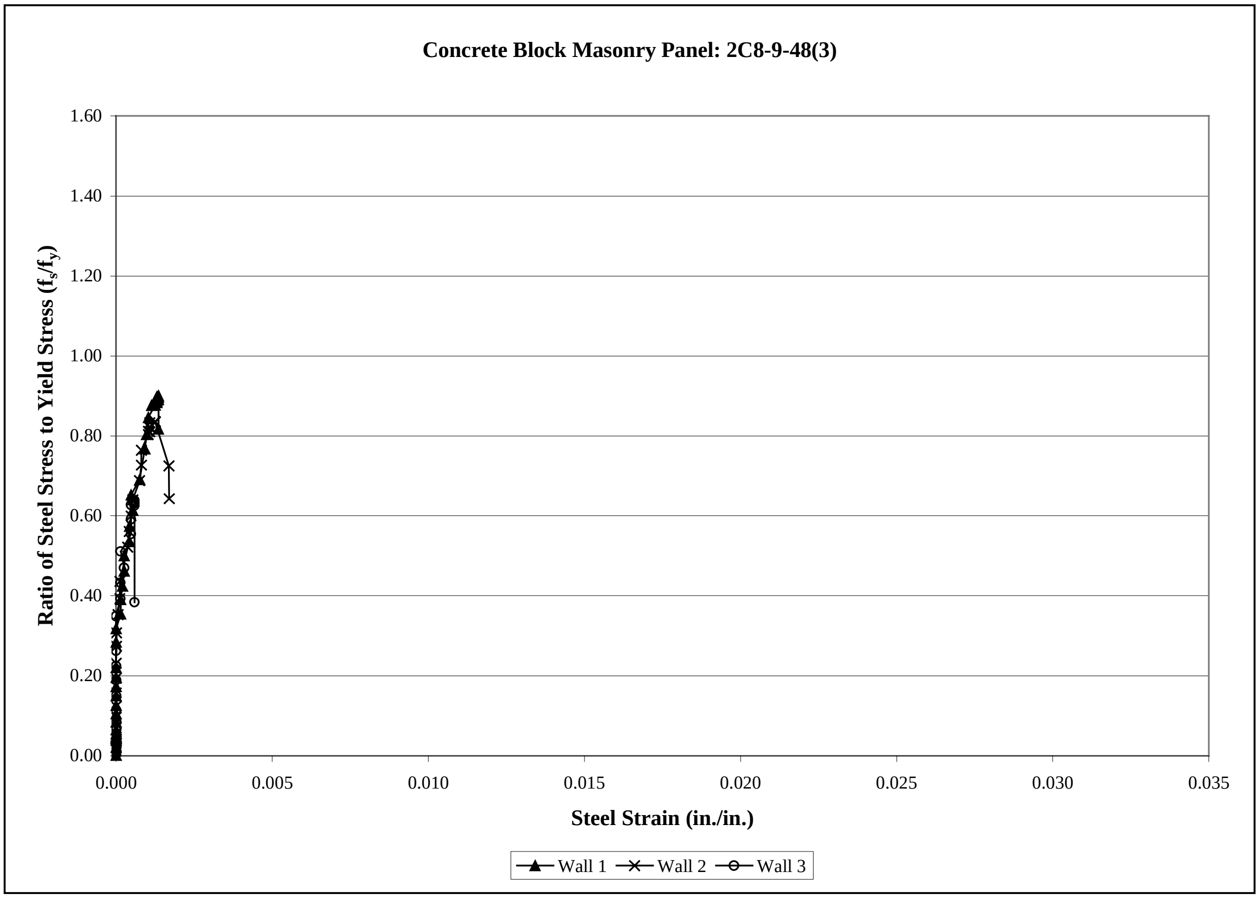
Specimen 2C8-8-48(3)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	34,261	43,146	37,787	38,398	lb.
Average Maximum Stress in Reinforcement (psi)	43,369	54,615	47,832	48,605	psi
Average Ratio of Maximum Steel Stress to f_y	0.723	0.910	0.797	0.810	
Average Maximum Steel Strain (in./in.)	0.0009	0.0010	0.0014	0.0011	in./in.
Average Maximum Splice Displacement (in.)	0.0520	0.0577	0.0761	0.0619	in.

A.1.4.8 – 8 inch Concrete Masonry Panels 2C8-8-64(2) -1, 2, and 3



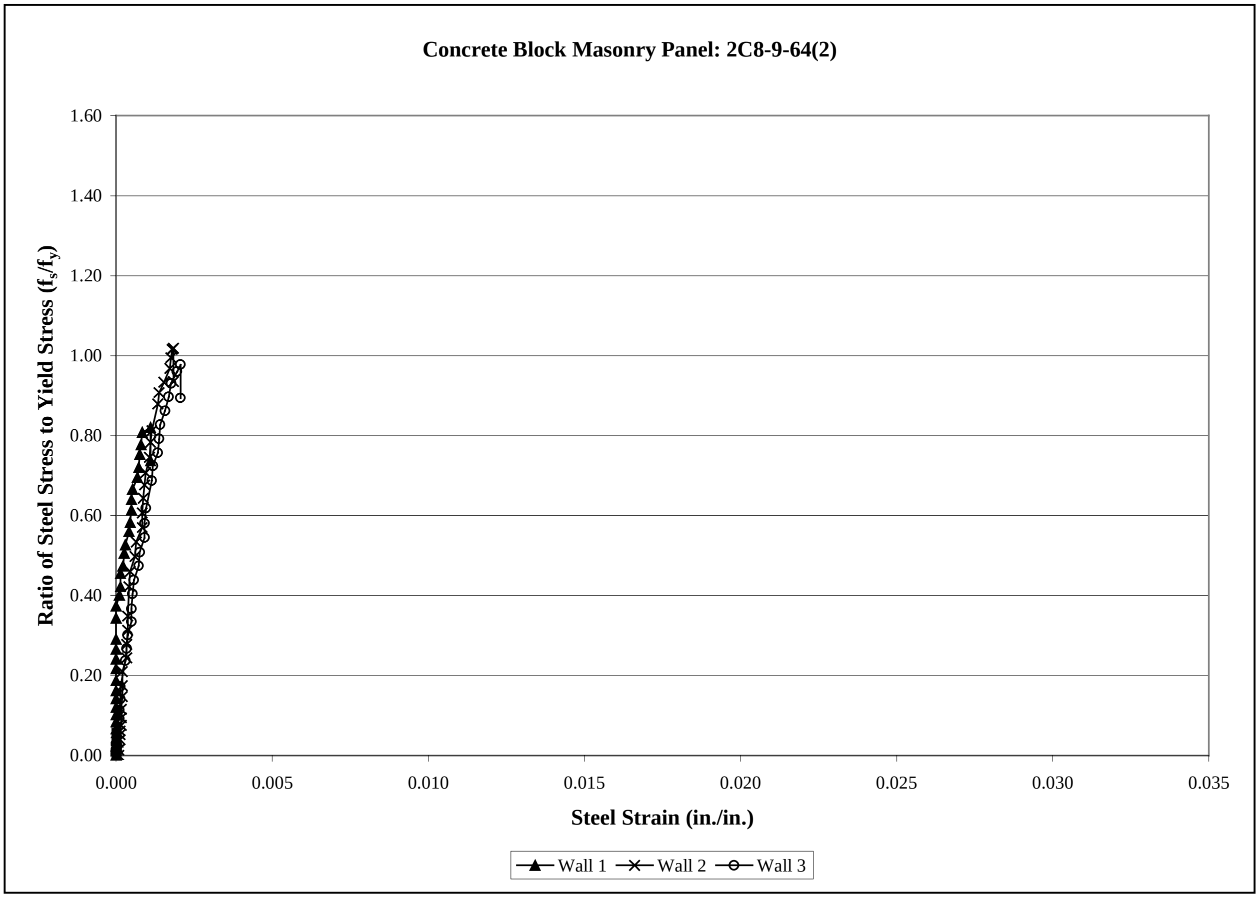
Specimen 2C8-8-64(2)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	N.A.	50,699	52,343	51,521	lb.
Average Maximum Stress in Reinforcement (psi)	N.A.	64,177	66,257	65,217	psi
Average Ratio of Maximum Steel Stress to f_y	N.A.	1.070	1.104	1.087	
Average Maximum Steel Strain (in./in.)	N.A.	0.0020	0.0026	0.0023	in./in.
Average Maximum Splice Displacement (in.)	N.A.	0.1407	0.1900	0.1653	in.

A.1.4.9 – 8 inch Concrete Masonry Panels 2C8-9-48(3) -1, 2, and 3



Specimen 2C8-9-48(3)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	53,996	50,155	38,329	47,493	lb.
Average Maximum Stress in Reinforcement (psi)	53,996	50,155	38,329	47,493	psi
Average Ratio of Maximum Steel Stress to f_y	0.900	0.836	0.639	0.792	
Average Maximum Steel Strain (in./in.)	0.0014	0.0017	0.0006	0.0012	in./in.
Average Maximum Splice Displacement (in.)	0.0761	0.0950	0.0330	0.0680	in.

A.1.4.10 – 8 inch Concrete Masonry Panels 2C8-9-64(2) -1, 2, and 3



Specimen 2C8-9-64(2)	Wall 1	Wall 2	Wall 3	Average	
Average Maximum Load (lb.)	49,200	61,120	58,643	56,321	lb.
Average Maximum Stress in Reinforcement (psi)	49,200	61,120	58,643	56,321	psi
Average Ratio of Maximum Steel Stress to f_y	0.820	1.019	0.977	0.939	
Average Maximum Steel Strain (in./in.)	0.0011	0.0018	0.0021	0.0017	in./in.
Average Maximum Splice Displacement (in.)	0.0884	0.1470	0.1654	0.1336	in.

A.1.5 – Concrete Masonry Panels—Phase III

Panel Number	Average[1] Maximum Tension Force in Splice (lb.)	Average[1] Maximum Stress in Reinforcement (psi)	Ratio of Average Maximum Stress in Reinforcement to f_y	Average[1] Maximum Steel Strain (in./in.)	Average[1] Maximum Displacement of Splice (in.)
3C8-7-48A (C)-1	49,942	83,237	1.387	N.M.	N.M.
3C8-7-48A (C)-2	46,612	77,687	1.295	N.M.	N.M.
3C8-7-48A (C)-3	38,289	63,815	1.064	N.M.	N.M.
Average	44,948	74,913	1.249	N.M.	N.M.
3C8-7-48B (C)-1	50,892	84,820	1.414	N.M.	N.M.
3C8-7-48B (C)-2	49,734	82,890	1.382	N.M.	N.M.
3C8-7-48B (C)-3	50,150	83,583	1.393	N.M.	N.M.
Average	50,259	83,764	1.396	N.M.	N.M.
3C8-7-76 (C)-1	53,271	88,785	1.480	N.M.	N.M.
3C8-7-76 (C)-2	53,063	88,438	1.474	N.M.	N.M.
3C8-7-76 (C)-3	51,814	86,257	1.439	N.M.	N.M.
Average	52,716	87,860	1.464	N.M.	N.M.
3C8-7-85 (C)-1	51,919	86,532	1.442	N.M.	N.M.
3C8-7-85 (C)-2	52,231	87,052	1.451	N.M.	N.M.
3C8-7-85 (C)-3	49,942	83,237	1.387	N.M.	N.M.
Average	51,364	85,607	1.427	N.M.	N.M.
3C8-8-48 (C)-1	49,942	63,218	1.054	N.M.	N.M.
3C8-8-48 (C)-2	59,722	75,597	1.260	N.M.	N.M.
3C8-8-48 (C)-3	57,225	72,437	1.207	N.M.	N.M.
Average	55,630	70,417	1.174	N.M.	N.M.
3C8-8-88 (C)-1	63,051	79,811	1.330	N.M.	N.M.
3C8-8-88 (C)-2	62,635	79,285	1.321	N.M.	N.M.
3C8-8-88 (C)-3	62,531	79,153	1.319	N.M.	N.M.
Average	62,739	79,416	1.324	N.M.	N.M.
3C8-8-98 (C)-1	62,843	79,548	1.326	N.M.	N.M.
3C8-8-98 (C)-2	64,404	81,524	1.359	N.M.	N.M.
3C8-8-98 (C)-3	63,572	80,471	1.341	N.M.	N.M.
Average	63,606	80,514	1.342	N.M.	N.M.
3C8-9-48 (C)-1	67,629	67,629	1.127	N.M.	N.M.
3C8-9-48 (C)-2	65,756	65,756	1.096	N.M.	N.M.
3C8-9-48 (C)-3	69,814	69,814	1.164	N.M.	N.M.
Average	67,733	67,733	1.129	N.M.	N.M.
3C8-9-101 (C)-1	77,305	77,305	1.288	N.M.	N.M.
3C8-9-101 (C)-2	74,912	74,912	1.249	N.M.	N.M.
3C8-9-101 (C)-3	77,201	77,201	1.287	N.M.	N.M.
Average	76,473	76,473	1.275	N.M.	N.M.
3C8-9-113 (C)-1	75,329	75,329	1.255	N.M.	N.M.
3C8-9-113 (C)-2	75,849	75,849	1.264	N.M.	N.M.
3C8-9-113 (C)-3	75,849	75,849	1.264	N.M.	N.M.
Average	75,676	75,676	1.261	N.M.	N.M.

N.M. - displacements not measured for Phase III concrete masonry panels.
[1]Average value of two lap splices per panel.

A.1.6 – Concrete Masonry Panels—Phase IV

Panel Number	Average[1] Maximum Tension Force in Splice (lb.)	Average[1] Maximum Stress in Reinforcement (psi)	Ratio of Average Maximum Stress in Reinforcement to f_y	Average[1] Maximum Steel Strain (in./in.)	Average[1] Maximum Displacement of Splice (in.)
4C12-5-32 (C2)-1	27,297	88,055	1.468	N.M.	N.M.
4C12-5-32 (C2)-2	24,751	79,841	1.331	N.M.	N.M.
4C12-5-32 (C2)-3	24,343	78,527	1.309	N.M.	N.M.
Average	25,464	82,141	1.369	N.M.	N.M.
4C12-5-32 (C)-1	29,945	96,598	1.610	N.M.	N.M.
4C12-5-32 (C)-2	28,519	91,998	1.533	N.M.	N.M.
4C12-5-32 (C)-3	29,844	96,269	1.604	N.M.	N.M.
Average	29,436	94,955	1.583	N.M.	N.M.
4C8-5-32 (C)-1	30,964	99,884	1.665	N.M.	N.M.
4C8-5-32 (C)-2	27,297	88,055	1.468	N.M.	N.M.
4C8-5-32 (C)-3	31,168	100,541	1.676	N.M.	N.M.
Average	29,810	96,160	1.603	N.M.	N.M.
4C12-6-31 (C)-1	40,335	91,670	1.528	N.M.	N.M.
4C12-6-31 (C)-2	41,557	94,447	1.574	N.M.	N.M.
4C12-6-31 (C)-3	41,557	94,447	1.574	N.M.	N.M.
Average	41,149	93,521	1.559	N.M.	N.M.
4C12-6-48 (C)-1	41,149	93,521	1.559	N.M.	N.M.
4C12-6-48 (C)-2	41,659	94,679	1.578	N.M.	N.M.
4C12-6-48 (C)-3	41,557	94,447	1.574	N.M.	N.M.
Average	41,455	94,216	1.570	N.M.	N.M.
4C12-7-37 (C)-1	50,113	83,521	1.392	N.M.	N.M.
4C12-7-37 (C)-2	52,659	87,765	1.463	N.M.	N.M.
4C12-7-37 (C)-3	48,585	80,975	1.350	N.M.	N.M.
Average	50,452	84,087	1.401	N.M.	N.M.
4C12-7-57 (C)-1	57,446	95,744	1.596	N.M.	N.M.
4C12-7-57 (C)-2	51,539	85,898	1.432	N.M.	N.M.
4C12-7-57 (C)-3	51,946	86,577	1.443	N.M.	N.M.
Average	53,644	89,406	1.490	N.M.	N.M.
4C12-7-76 (C)-1	56,428	94,046	1.567	N.M.	N.M.
4C12-7-76 (C)-2	56,631	94,386	1.573	N.M.	N.M.
4C12-7-76 (C)-3	56,835	94,725	1.579	N.M.	N.M.
Average	56,631	94,386	1.573	N.M.	N.M.
4C12-8-45 (C)-1	56,631	71,685	1.195	N.M.	N.M.
4C12-8-45 (C)-2	57,446	72,717	1.212	N.M.	N.M.
4C12-8-45 (C)-3	55,715	70,525	1.175	N.M.	N.M.
Average	56,597	71,642	1.194	N.M.	N.M.
4C12-8-71 (C)-1	59,687	75,553	1.259	N.M.	N.M.
4C12-8-71 (C)-2	61,113	77,358	1.289	N.M.	N.M.
4C12-8-71 (C)-3	63,965	80,968	1.349	N.M.	N.M.
Average	61,588	77,960	1.299	N.M.	N.M.

N.M. - displacements not measured for Phase III concrete masonry panels.
[1]Average value of two lap splices per panel.

A.2 Compressive Strength Test Results of Masonry Prisms

A.2.1 – 4 inch Clay Masonry Prisms (Ungrouted)

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	3.492	15.545	11.523	35.12	120,120	3,420	3.30	1.094	3,740
2	3.491	15.568	11.503	35.16	99,520	2,830	3.30	1.094	3,100
3	3.491	15.560	11.425	35.14	196,380	5,589	3.27	1.092	6,100
Avg.	3.491	15.558	11.484	35.14	138,673	3,946	3.29	1.093	4,310

1 Net area calculated based on an average void area of 35.3%: A_n = (100-35.3)(W)(L)/100
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained from ASTM C 1314.

A.2.2 – 6 inch Clay Masonry Prisms (Ungrouted)

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	5.456	15.413	11.405	53.74	351,660	6,544	2.09	1.007	6,590
2	5.475	15.465	11.625	54.10	344,960	6,376	2.12	1.010	6,440
3	5.480	15.460	11.510	54.14	332,760	6,146	2.10	1.008	6,200
Avg.	5.470	15.446	11.513	53.99	343,127	6,356	2.10	1.008	6,410

1 Net area calculated based on an average void area of 36.1%: A_n = (100-36.1)(W)(L)/100
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained from ASTM C 1314.

A.2.3 – 8 inch Concrete Masonry Prisms - Grouted, Phase I

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	7.68	7.77	15.6	59.67	244,240	4,093	2.03	1.003	4,110
2	7.67	7.74	15.67	59.37	238,820	4,023	2.04	1.003	4,040
3	7.66	7.75	15.67	59.37	241,360	4,066	2.05	1.004	4,080
Avg.	7.67	7.75	15.65	59.47	241,473	4,060	2.04	1.003	4,070

1 Net area calculated as average width multiplied by the average length.
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained form ASTM C 1314.

A.2.4 – 8 inch Concrete Masonry Prisms - Grouted, Phase II

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	7.650	7.735	15.734	59.17	98,220	1,660	2.06	1.005	1,670
2	7.644	7.746	15.778	59.21	103,860	1,754	2.06	1.005	1,760
3	7.638	7.759	15.748	59.26	98,160	1,656	2.06	1.005	1,660
Avg.	7.644	7.747	15.753	59.22	100,080	1,690	2.06	1.005	1,700

1 Net area calculated as average width multiplied by the average length.
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained from ASTM C 1314.

A.2.5 – 8 inch Concrete Masonry Prisms - Grouted, Phase III

Using 3,000 psi Target Strength Grout

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	7.65	15.65	8.40	64.26	208,880	3,251	2.05	1.004	3,260
2	7.64	15.68	8.43	64.41	200,420	3,112	2.05	1.004	3,120
3	7.61	15.66	8.40	63.92	200,760	3,141	2.05	1.004	3,150
Avg.	7.63	15.66	8.41	64.20	208,353	3,168	2.05	1.004	3,180

1 Net area calculated as average width multiplied by the average length.
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained from ASTM C 1314.

Using 5,000 psi Target Strength Grout

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	7.63	15.74	8.37	63.86	209,060	3,274	2.06	1.005	3,190
2	7.64	15.68	8.42	64.33	219,900	3,418	2.05	1.004	3,440
3	7.63	15.66	8.42	64.24	200,400	3,120	2.05	1.004	3,130
Avg.	7.63	15.69	8.40	64.14	209,787	3,271	2.05	1.004	3,290

1 Net area calculated as average width multiplied by the average length.
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained from ASTM C 1314.

A.2.6 – 8 inch and 12 inch Concrete Masonry Prisms - Grouted, Phase IV

8 inch Concrete Masonry Units

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	7.64	8.36	15.61	63.87	184,000	2,881	2.04	1.003	2,890
2	7.64	8.35	15.61	63.79	177,340	2,780	2.04	1.003	2,790
3	7.63	8.34	15.61	63.63	182,400	2,867	2.05	1.004	2,880
Avg.	7.64	8.35	15.61	63.76	181,247	2,843	2.04	1.003	2,850

1 Net area calculated as average width multiplied by the average length.
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained from ASTM C 1314.

12 inch Concrete Masonry Units

Prism No.	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Net Area[1] (in^2)	Maximum Compressive Load (lb.)	Tested Net Area Compressive Strength (psi)	Aspect Ratio[2]	Aspect Ratio Correction Factor[3]	Corrected Net Area Compressive Strength (psi)
1	11.63	8.36	15.73	97.23	219,360	2,256	1.88	0.966	2,180
2	11.62	8.37	15.66	97.34	216,160	2,221	1.87	0.964	2,140
3	11.63	8.37	15.74	97.34	223,160	2,293	1.88	0.966	2,210
Avg.	11.63	8.37	15.71	97.30	219,693	2,256	1.88	0.965	2,180

1 Net area calculated as average width multiplied by the average length.
2 Aspect ratio = Height/Width.
3 Aspect ratio correction factor obtained from ASTM C 1314.

A.3 Mortar Test Results

Mortar Used in Construction of	Construction Date	Initial Cone Penetration (mm)	28-Day Cube Compressive Strength (psi)
Phase I - 8 in. CMU Panels	03/13/96	42	2,320
Phase I - 8 in. CMU Panels	03/14/96	54	2,440
Average		48	2,380
4 in. Clay Panels	11/19/96	51	3,170
6 in. Clay Panels	11/14/96	55	3,050
6 in. Clay Panels	11/14/96	53	3,170
Average		53	3,130
Phase II - 8 in. CMU Panels	12/04/96	50	3,290
Phase II - 8 in. CMU Panels	12/05/96	54	3,190
Phase II - 8 in. CMU Panels	12/06/96	45	2,980
Phase II - 8 in. CMU Panels	12/09/96	54	3,320
Average		51	3,200
Phase III - 8 in. CMU Panels	04/06/98	53	3,090
Phase III - 8 in. CMU Panels	04/07/98	62	2,720
Phase III - 8 in. CMU Panels	04/08/98	54	3,110
Phase III - 8 in. CMU Panels	04/09/98	60	3,220
Phase III - 8 in. CMU Panels	04/10/98	57	3,400
Average		57	3,110
Phase IV - 8 in./12 in. CMU Panels	09/24/98	53	3,910
Phase IV - 12 in. CMU Panels	09/25/98	53	4,070
Average		53	3,990

A.4 Grout Test Results

A.4.1 – Grout Used for 4 inch Clay Masonry Panels

Grout Specimen No.	Measured Dimensions			Maximum Compressive Load (lb.)	Net Area (in^2)	Compressive Strength (psi)
	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)			
1	3.287	6.022	3.182	65,020	10.459	6,220
2	3.024	6.018	3.129	60,720	9.462	6,420
3	3.030	6.017	2.984	56,800	9.042	6,280
Avg.	3.114	6.019	3.098	60,847	9.654	6,310

A.4.2 – Grout Used for 6 inch Clay Masonry Panels

Grout Specimen No.	Measured Dimensions			Maximum Compressive Load (lb.)	Net Area (in^2)	Compressive Strength (psi)
	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)			
1	3.052	6.012	3.134	56,660	9.565	5,920
2	3.035	6.022	3.218	61,460	9.767	6,290
3	3.022	6.032	3.120	59,460	9.429	6,310
Avg.	3.036	6.022	3.157	59,193	9.587	6,170

A.4.3 – Grout Used for Phase I 8 inch Concrete Masonry Panels

Grout Specimen No.	Measured Dimensions			Maximum Compressive Load (lb.)	Net Area (in^2)	Compressive Strength (psi)
	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)			
1	3.533	7.243	3.670	67,880	12.966	5,240
2	3.600	7.246	4.013	73,100	14.447	5,060
3	3.731	7.265	3.894	73,860	14.529	5,080
Avg.	3.621	7.251	3.859	71,613	13.981	5,130

A.4.4 – Grout Used for Phase II 8 inch Concrete Masonry Panels

Grout Specimen No.	Measured Dimensions			Maximum Compressive Load (lb.)	Net Area (in^2)	Compressive Strength (psi)
	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)			
1	3.503	6.959	3.803	27,160	13.322	2,040
2	3.620	7.045	3.629	28,100	13.137	2,140
3	3.732	7.077	3.580	26,240	13.361	1,960
Avg.	3.618	7.027	3.671	27,167	13.273	2,050

A.4.5 – Grout Used for Phase III 8 inch Concrete Masonry Panels

Using 3,000 psi Target Strength Grout

Grout Specimen No.	Measured Dimensions			Maximum Compressive Load (lb.)	Net Area (in^2)	Compressive Strength (psi)
	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)			
1	3.552	7.173	3.523	59,280	12.512	4,740
2	3.674	7.221	3.632	61,200	13.344	4,590
3	3.543	7.179	3.558	59,500	12.284	4,720
Avg.	3.590	7.191	3.571	59,993	12.821	4,680

Using 5,000 psi Target Strength Grout

Grout Specimen No.	Measured Dimensions			Maximum Compressive Load (lb.)	Net Area (in^2)	Compressive Strength (psi)
	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)			
1	3.601	7.029	3.657	70,460	13.163	5,350
2	3.745	7.143	3.653	77,600	13.680	5,670
3	3.758	7.158	3.728	77,760	14.009	5,550
Avg.	3.701	7.110	3.679	75,273	13.617	5,530

A.4.6 – Grout Used for Phase III 8 inch and 12 inch Concrete Masonry Panels

Grout Specimen No.	Measured Dimensions			Maximum Compressive Load (lb.)	Net Area (in^2)	Compressive Strength (psi)
	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)			
1	3.560	6.858	3.595	46,460	12.798	3,630
2	3.560	6.758	3.555	44,620	12.656	3,520
3	3.590	6.863	3.575	45,160	12.834	3,520
Avg.	3.570	6.826	3.575	45,413	12.763	3,560

A.5 Masonry Unit Test Results

A.5.1 – 4 inch Clay Brick Test Results

ASTM C 67 Test Report
By the NCMA Research and Development Laboratory

Client:	Interstate Brick	Lab Proj. No.:	96-183
Address:	9780 South 5200 West	Date Received:	11/07/96
	West Jordan, UT 84088-5689	Report Date:	04/13/97

Unit Designation/Description: 4x4x16 Solid Shell Hollow Brick Units (ASTM 652, Grade SW, Type HBX, Class H40V)

Summary of Results

Average Test Properties	Tested Value		ASTM C 652 Required Value
Compressive Strength (gross area)	7752	psi	3000 minimum
Compressive Strength (net area)	11990	psi	****
Cold Water Absorption	6.1	%	****
5-Hour Boil Water Absorption	9.1	%	17.0 maximum
Saturation Coefficient	0.66		0.78 maximum
Oven-Dried Initial Rate of Absorption	3.9	g/30 in^2/min	****
Minimum Face Shell Thickness	0.77	in.	0.75 minimum
Minimum End Shell Thickness	0.92	in.	0.75 minimum
Minimum Web Thickness	0.64	in.	0.50 minimum
Maximum Deviation from Specified Width	0.05	in.	0.125 maximum
Maximum Deviation from Specified Height	0.04	in.	0.093 maximum
Maximum Deviation from Specified Length	0.19	in.	0.281 maximum

Measurement of Size and Void Area

Unit No.	Received Weight (lb.)	Average Width (in.)	Average Height (in.)	Average Length (in.)	Core Volume (ml)	Void Area (%)	Minimum Face Shell Thickness (in.)	Minimum End Shell Thickness (in.)	Minimum Web Thickness (in.)
1	9.16	3.47	3.51	15.57	1099	35.3	****	****	****
2	9.15	3.48	3.54	15.56	1105	35.2	****	****	****
3	9.14	3.46	3.53	15.55	1110	35.7	****	****	****
4	9.10	3.46	3.50	15.54	1100	35.6	****	****	****
5	9.14	3.45	3.51	15.55	1075	34.8	****	****	****
6	9.14	3.48	3.52	15.57	1112	35.6	0.81	0.93	0.64
7	9.10	3.47	3.54	15.55	1110	35.4	0.83	0.92	0.65
8	9.22	3.50	3.54	15.55	1108	35.2	0.82	0.95	0.65
9	9.16	3.48	3.53	15.56	1110	35.4	0.83	0.97	0.64
10	9.16	3.47	3.53	15.69	1110	35.2	0.77	0.95	0.66
Average	9.15	3.47	3.52	15.57	1104	35.3	0.81	0.95	0.65
Specified Dimensions		3.50	3.50	15.50	in.				

***** Report Continued on Following Page***

ASTM C 67 Test Report
By the NCMA Research and Development Laboratory

Client:	Interstate Brick	Lab Proj. No.:	96-183
Address:	9780 South 5200 West	Date Received:	11/07/96
	West Jordan, UT 84088-5689	Report Date:	04/13/97

Unit Designation/Description: 4x4x16 Solid Shell Hollow Brick Units (ASTM 652, Grade SW, Type HBX, Class H40V)

Compressive Strength Tests

Unit No.	Gross Area (in^2)	Net Area (in^2)	Maximum Load (lb.)	Gross Area Compressive Strength (psi)	Net Area Compressive Strength (psi)
1	54.05	34.95	434460	8038	12432
2	54.10	34.98	443680	8201	12684
3	53.76	34.76	386480	7188	11118
4	53.76	34.76	441300	8209	12696
5	53.69	34.71	382460	7124	11018
Average	53.87	34.83	417676	7752	11990

Note: Compression tests were performed on full-size units.

Absorption Tests and Oven-Dry Initial Rate of Absorption

Unit No.	Dry Weight (lb.)	Cold-Water Saturated Weight (lb.)	Cold-Water Absorption (%)	5-Hour Boil Weight (lb.)	5-Hour Boil Absorption (%)	Saturation Coefficient	Net Area* (in^2)	Weight Gain due to IRA Test (g)	Oven-Dried Initial Rate of Absorption (g/30 in^2/min)
6	9.13	9.69	6.13	9.96	9.09	0.67	34.90	4.5	3.9
7	9.10	9.67	6.26	9.95	9.34	0.67	34.87	4.5	3.9
8	9.22	9.75	5.75	10.04	8.89	0.65	35.24	4.5	3.9
9	9.15	9.70	6.01	9.98	9.07	0.66	34.91	4.5	3.9
10	9.15	9.71	6.12	9.99	9.18	0.67	35.32	4.5	3.9
Average	9.15	9.70	6.06	9.98	9.12	0.66	35.05	4.5	3.9

* Net area determined as: (W x L)((100-Void Area) / 100)

Robert D. Thomas
Director of Research

A.5.2 – 6 inch Clay Brick Test Results

ASTM C 67 Test Report
By the NCMA Research and Development Laboratory

Client:	Interstate Brick	Lab Proj. No.:	96-183
Address:	9780 South 5200 West	Date Received:	11/07/96
	West Jordan, UT 84088-5689	Report Date:	04/13/97

Unit Designation/Description: 6x4x16 Solid Shell Hollow Brick Units (ASTM 652, Grade SW, Type HBX, Class H40V)

Summary of Results

Average Test Properties	Tested Value		ASTM C 652 Required Value
Compressive Strength (gross area)	11807	psi	3000 minimum
Estimated Compressive Strength (net area)	17950	psi	****
Cold Water Absorption	5.3	%	****
5-Hour Boil Water Absorption	7.4	%	17.0 maximum
Saturation Coefficient	0.72		0.78 maximum
Oven-Dried Initial Rate of Absorption	2.5	g/30 in^2/min	****
Minimum Face Shell Thickness	1.25	in.	1.00 minimum
Minimum End Shell Thickness	1.33	in.	1.00 minimum
Minimum Web Thickness	1.13	in.	0.50 minimum
Maximum Deviation from Specified Width	0.06	in.	0.125 maximum
Maximum Deviation from Specified Height	0.05	in.	0.093 maximum
Maximum Deviation from Specified Length	0.12	in.	0.281 maximum

Measurement of Size and Void Area

Unit No.	Received Weight (lb.)	Average Width (in.)	Average Height (in.)	Average Length (in.)	Core Volume (ml)	Void Area (%)	Minimum Face Shell Thickness (in.)	Minimum End Shell Thickness (in.)	Minimum Web Thickness (in.)
1	14.88	5.44	3.53	15.41	1765	36.4	****	****	****
2	14.80	5.45	3.53	15.43	1735	35.7	****	****	****
3	14.72	5.46	3.55	15.44	1755	35.8	****	****	****
4	14.44	5.45	3.52	15.42	1856	38.2	****	****	****
5	14.32	5.46	3.53	15.39	1717	35.3	****	****	****
6	14.96	5.46	3.55	15.45	1760	35.9	1.25	1.33	1.15
7	14.82	5.45	3.55	15.39	1750	35.9	1.29	1.37	1.15
8	14.74	5.45	3.55	15.39	1750	35.9	1.28	1.33	1.13
9	14.66	5.45	3.55	15.43	1750	35.8	1.28	1.36	1.14
10	14.32	5.46	3.55	15.43	1755	35.8	1.28	1.34	1.13
Average	14.67	5.45	3.54	15.42	1759	36.1	1.28	1.35	1.14

Specified Dimensions	5.50	3.50	15.50	in.

***** Report Continued on Following Page***

ASTM C 67 Test Report
By the NCMA Research and Development Laboratory

Client:	Interstate Brick	Lab Proj. No.:	96-183
Address:	9780 South 5200 West	Date Received:	11/07/96
	West Jordan, UT 84088-5689	Report Date:	04/13/97

Unit Designation/Description: 6x4x16 Solid Shell Hollow Brick Units (ASTM 652, Grade SW, Type HBX, Class H40V)

Compressive Strength Tests

Unit No.	Saw-Cut Length (in.)	Gross Area (in^2)	Maximum Load (lb.)	Gross Area Compressive Strength (psi)
1	7.00	38.08	511240	13425
2	7.00	38.12	418880	10990
3	7.00	38.24	475860	12445
4	7.00	38.17	455800	11942
5	7.00	38.22	391170	10235
Average	7.00	38.16	450590	11807

Note: Units saw-cut to half-length prior to capping and testing in compression. (Estimated net area of reduced size specimen was 25.1 in^2 , resulting in an average net strength of 17950 psi)

Absorption Tests and Oven-Dry Initial Rate of Absorption

Unit No.	Dry Weight (lb.)	Cold-Water Saturated Weight (lb.)	Cold-Water Absorption (%)	5-Hour Boil Weight (lb.)	5-Hour Boil Absorption (%)	Saturation Coefficient	Net Area* (in^2)	Weight Gain due to IRA Test (g)	Oven-Dried Initial Rate of Absorption (g/30 in^2/min)
6	14.29	15.07	5.46	15.34	7.35	0.74	54.09	4.5	2.5
7	14.31	15.03	5.03	15.33	7.13	0.71	53.72	4.5	2.5
8	14.26	15.02	5.33	15.32	7.43	0.72	53.80	4.5	2.5
9	14.23	14.99	5.34	15.32	7.66	0.70	53.94	4.5	2.5
10	14.30	15.08	5.45	15.38	7.55	0.72	54.11	4.5	2.5
Average	14.28	15.04	5.32	15.34	7.42	0.72	53.93	4.5	2.5

* Net area determined as: (W x L)((100-Void Area) / 100)

Robert D. Thomas
Director of Research

A.5.3 – Phase I 8 inch Concrete Masonry Unit Test Results

ASTM C 140-94a Test Report

Job No.: 94-179b
Report Date: 9/14/94

Client:	Blocklite	Testing Agency:	National Concrete Masonry Association Research and Development Laboratory
Address:	1900 Ross Street P.O. Box AE Selma, CA 93662-0540	Address:	2302 Horse Pen Road Herndon, Virginia 22071
Unit Specification:	ASTM C 90-94a	Sampling Party:	Blocklite
Unit Designation/Desription:	8x8x16 Hollow, Square-Cornered CMU	Job No./Description:	Units for use with NCMA Structural Cover Research, 94-179 and 1996 Splice Research

Summary of Test Results

Physical Property	Required Values	Tested Values	
Net Compressive Strength	1900 min	3070	psi
Gross Compressive Strength	****	1600	psi
Density	****	97.8	pcf
Absorption	18 max	11.9	pcf
Percent Solid	****	52.3	%
Moisture Content	****	12.7	%

Physical Property	Required Values	Tested Values	
Min. Faceshell Thickness (FST)	1.25 min	1.30	in.
Min. Web Thickness (WT)	1.00 min	1.22	in.
Equivalent Web Thickness	2.25 min	2.83	in.
Equivalent Thickness	****	3.99	in.
Max. Var. from Spec. Dimensions	.125 max	0.06	in.
Net Cross-Sectional Area	****	62.1	in^2
Gross Cross-Sectional Area	****	119.3	in^2

Individual Unit Test Results

		Avg Width in.	Avg Height in.	Avg Length in.	Avg. Min. FST in.	Avg. Min. WT in.	Received Wt, W_R lb	Max. Load lb
Measured Values	Unit #1	7.65	7.67	15.60	1.31	1.23	27.30	179960
	Unit #2	7.67	7.64	15.57	1.31	1.23	27.56	198840
	Unit #3	7.66	7.61	15.58	1.28	1.22	27.29	192720
	Average	7.66	7.64	15.58	1.30	1.22	27.38	190510

		Cross-Sectional Area		Compressive Strength	
		Gross in^2	Net in^2	Gross psi	Net psi
Calculated Values	Unit #1	119.26	61.87	1510	2910
	Unit #2	119.34	62.16	1670	3200
	Unit #3	119.26	62.36	1620	3090
	Average	119.29	62.13	1600	3070

		Avg Width in.	Avg Height in.	Avg Length in.	Received Wt, W_R lb	Immersed Wt, W_I lb	Saturated Wt, W_S lb	Oven-Dry Wt, W_D lb
Measured Values	Unit #4	7.62	7.64	15.61	27.08	12.90	29.88	26.64
	Unit #5	7.63	7.64	15.60	27.40	12.98	30.21	26.93
	Unit #6	7.63	7.63	15.60	27.31	13.06	30.26	26.98
	Average	7.62	7.63	15.60	27.26	12.98	30.12	26.85

		Absorp pcf	Density pcf	Gross Volume ft^3	Net Volume ft^3	Percent Solid %
Calculated Values	Unit #4	11.9	97.9	0.5250	0.2721	51.8
	Unit #5	11.9	97.5	0.5259	0.2761	52.5
	Unit #6	11.9	97.9	0.5251	0.2756	52.5
	Average	11.9	97.8	0.5253	0.2746	52.3

Robert D. Thomas
Director of Research

Comments: These units meet or exceed the requirements of ASTM C 90-94a.

A.5.4 – Phase II 8 inch Concrete Masonry Unit Test Results

ASTM C 140-96 Test Report

Job No.: 96-183-07
Report Date: 12/30/96

Client:	Tarmac America, Inc.	Testing Agency:	National Concrete Masonry Assoc. Research and Development Laboratory
Address:	P.O. Box 2016 Norfolk, VA 23501	Address:	2302 Horse Pen Road Herndon, Virginia 20171
Unit Specification:	ASTM C90-96	Sampling Party:	Tarmac America, Inc.
Unit Designation/Description:	8 x 8 x 16 Hollow CMU Double Square End Units	Job No./Description:	Splice Length Research, Phase II

Summary of Test Results

Physical Property	Required Values	Tested Values	
Net Compressive Strength	1900 min	2170	psi
Gross Compressive Strength	****	1130	psi
Density	****	95.7	pcf
Absorption	18 max	13.4	pcf
Percent Solid	****	52.2	%

Physical Property	Required Values	Tested Values	
Min. Faceshell Thickness (FST)	1.25 min	1.29	in.
Min. Web Thickness (WT)	1.00 min	1.21	in.
Equivalent Web Thickness	2.25 min	2.79	in.
Equivalent Thickness	****	3.98	in.
Max. Var. from Spec. Dimensions	.125 max	0.04	in.
Net Cross-Sectional Area	****	62.1	in^2
Gross Cross-Sectional Area	****	119.0	in^2

Individual Unit Test Results

		Received	Cross-Sectional Area		Max.	Compressive Strength	
		Wt, W_R	Gross	Net	Load	Gross	Net
		lb	in^2	in^2	lb	psi	psi
Compression Units	Unit #1	27.42	119.10	62.03	133180	1120	2150
	Unit #2	28.00	119.03	61.95	134840	1130	2180
	Unit #3	27.35	118.87	62.23	135620	1140	2180
	Average	27.59	119.00	62.07	134550	1130	2170

		Avg Width	Avg Height	Avg Length	Avg. Min. FST	Avg. Min. WT
		in.	in.	in.	in.	in.
Absorption Units	Unit #4	7.63	7.66	15.61	1.29	1.20
	Unit #5	7.63	7.67	15.61	1.29	1.22
	Unit #6	7.61	7.63	15.62	1.28	1.21
	Average	7.62	7.65	15.61	1.29	1.21

	Received Wt, W_R	Immersed Wt, W_I	Saturated Wt, W_S	Oven-Dry Wt, W_D	Absorp	Density	Net Volume	Percent Solid
	lb	lb	lb	lb	pcf	pcf	ft^3	%
Unit #4	27.47	12.72	29.83	26.20	13.2	95.6	0.2742	52.0
Unit #5	27.31	12.64	29.83	26.25	13.0	95.3	0.2755	52.2
Unit #6	28.10	13.12	30.26	26.40	14.1	96.1	0.2747	52.3
Average	27.63	12.83	29.97	26.28	13.4	95.7	0.2748	52.2

Comments: These tested properties meet or exceed the applicable requirements of ASTM C 90-96.

Robert D. Thomas
Director of Research

A.5.5 – Phase III and Phase IV 8 inch Concrete Masonry Unit Test Results

ASTM C 140 Test Report

Job No.: Splice
Report Date: 01/25/99

Client: NCMA - Splice Length Research
Address: 2302 Horse Pen Road
Herndon, VA 20171

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

Unit Specification: ASTM C90

Sampling Party: NCMA - Splice Length Research

Unit Designation/Description:
8-inch Hollow CMU
Double Square End, Square Cores
Units Supplied by Tarmac

Job No./Description: Phase III - Splice Research

Summary of Test Results

Physical Property	Required Values	Tested Values	
Net Compressive Strength	1900 min	2670	psi
Gross Compressive Strength	****	1400	psi
Density	****	102.0	pcf
Absorption	18 max	13.7	pcf
Percent Solid	****	52.0	%

Physical Property	Required Values	Tested Values	
Min. Faceshell Thickness (FST)	1.25 min	1.28	in.
Min. Web Thickness (WT)	1.00 min	1.20	in.
Equivalent Web Thickness	2.25 min	2.77	in.
Equivalent Thickness	****	3.96	in.
Max. Var. from Spec. Dimensions	.125 max	0.08	in.
Net Cross-Sectional Area	****	61.9	in^2
Gross Cross-Sectional Area	****	118.5	in^2

Individual Unit Test Results

		Received Wt, W_R	Cross-Sectional Area * Gross	Cross-Sectional Area * Net	Max. Load	Compressive Strength Gross	Compressive Strength Net
		lb	in^2	in^2	lb	psi	psi
Compression Units	Unit #1	29.96	118.51	61.90	166860	1410	2700
	Unit #2	29.76	118.51	61.90	170180	1440	2750
	Unit #3	29.58	118.51	61.90	159180	1340	2570
	Average	29.77	118.51	61.90	165410	1400	2670

* Unit areas determined as the average of the three absorption units and are assumed to be the same as those units tested in compression.

		Avg Width	Avg Height	Avg Length	Avg./Min. FST**	Min. WT	Average Gross Area
		in.	in.	in.	in.	in.	in^2
Absorption Units	Unit #4	7.62	7.62	15.57	1.27	1.20	118.57
	Unit #5	7.61	7.62	15.57	1.27	1.20	118.45
	Unit #6	7.62	7.55	15.57	1.28	1.19	118.53
	Average	7.61	7.59	15.57	1.28	1.20	118.51

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

	Received Wt, W_R	Immersed Wt, W_I	Saturated Wt, W_S	Oven-Dry Wt, W_D	Absorp	Density	Net Volume	Percent Solid	Avg. Net Area
	lb	lb	lb	lb	pcf	pcf	ft^3	%	in^2
Unit #4	29.42	14.17	31.14	27.48	13.5	101.0	0.2720	52.0	61.7
Unit #5	29.94	14.67	31.64	27.70	14.5	101.9	0.2720	52.1	61.7
Unit #6	29.86	14.46	31.22	27.66	13.3	103.0	0.2686	51.9	62.3
Average	29.74	14.43	31.33	27.61	13.7	102.0	0.2708	52.0	61.9

Comments: These tested properties meet or exceed the applicable requirements

Robert D. Thomas

A.5.6 – Phase IV 12 inch Concrete Masonry Unit Test Results

ASTM C 140 Test Report

Job No.: Splice
Report Date: 01/25/99

Client: NCMA - Splice Length Research
Address: 2302 Horse Pen Road
Herndon, VA 20171

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

Unit Specification: ASTM C90

Sampling Party: NCMA - Splice Length Research

Unit Designation/Description:
12-inch Hollow CMU
Double Square End, Square Core
Units Supplied by Tarmac

Job No./Description: Phase IV - Splice Research

Summary of Test Results

Physical Property	Required Values	Tested Values	
Net Compressive Strength	1900 min	2640	psi
Gross Compressive Strength	****	1310	psi
Density	****	101.6	pcf
Absorption	18 max	11.0	pcf
Percent Solid	****	49.1	%

Physical Property	Required Values	Tested Values	
Min. Faceshell Thickness (FST)	1.5 min	1.51	in.
Min. Web Thickness (WT)	1.13 min	1.37	in.
Equivalent Web Thickness	2.5 min	3.16	in.
Equivalent Thickness	****	5.70	in.
Max. Var. from Spec. Dimensions	.125 max	0.05	in.
Net Cross-Sectional Area	****	89.7	in^2
Gross Cross-Sectional Area	****	181.0	in^2

Individual Unit Test Results

		Received	Cross-Sectional Area *		Max.	Compressive Strength	
		Wt, W_R	Gross	Net	Load	Gross	Net
		lb	in^2	in^2	lb	psi	psi
Compression Units	Unit #1	41.07	181.05	89.68	236660	1310	2640
	Unit #2	40.87	181.05	89.68	243560	1350	2720
	Unit #3	41.05	181.05	89.68	230700	1270	2570
	Average	41.00	181.05	89.68	236970	1310	2640

* Unit areas determined as the average of the three absorption units and are assumed to be the same as those units tested in compression.

		Avg Width	Avg Height	Avg Length	Avg./Min. FST**	Min. WT	Average Gross Area
		in.	in.	in.	in.	in.	in^2
Absorption Units	Unit #4	11.62	7.63	15.58	1.51	1.38	180.98
	Unit #5	11.63	7.63	15.59	1.51	1.34	181.18
	Unit #6	11.62	7.58	15.58	1.51	1.38	180.98
	Average	11.62	7.61	15.58	1.51	1.37	181.05

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

	Received Wt, W_R	Immersed Wt, W_I	Saturated Wt, W_S	Oven-Dry Wt, W_D	Absorp	Density	Net Volume	Percent Solid	Avg. Net Area
	lb	lb	lb	lb	pcf	pcf	ft^3	%	in^2
Unit #4	40.52	19.61	44.26	39.73	11.5	100.6	0.3950	49.5	89.5
Unit #5	40.98	19.74	44.12	39.93	10.7	102.2	0.3907	48.8	89.5
Unit #6	40.44	19.54	43.74	39.51	10.9	101.9	0.3878	48.9	90.1
Average	40.65	19.63	44.04	39.72	11.0	101.6	0.3912	49.1	89.7

Comments: These tested properties meet or exceed the applicable requirements

Robert D. Thomas