Effects of Confinement Reinforcement on Bar Splice Performance

Testing Conducted by:

NATIONAL CONCRETE MASONRY ASSOCIATION
RESEARCH & DEVELOPMENT LABORATORY

Project No. 03-389
Date: July 2004
Report No. MR 26

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The following groups within NCMA provided technical assistance in the development of the scope of work and testing procedures outlined in this report. These groups also evaluated test results and conclusions drawn based on that evaluation and assessment of the test specimens. Their input and guidance is highly appreciated.

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Allied Concrete Products, LLC of Chesapeake, Virginia is gratefully acknowledged for the donation of materials used in the construction of the test specimens of this project.

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Abstract

Research was conducted at the NCMA Research and Development Laboratory to investigate the effects of confinement provided by lateral reinforcement on lap splice performance. This first phase of research was conducted to establish if improvements in splice performance exists and to provide guidance for future research. Recognition of beneficial effects from lateral reinforcement by building codes may result in smaller required splice lengths.

Fifteen concrete masonry panels were constructed using 8-in. (203 mm) units. Two sets of No. 8 (M#25) reinforcing bars were placed in the center of the cells and incorporated a splice length of 48 in. (1219 mm). To evaluate the effects of confinement reinforcement on splice behavior, five different arrangements of lateral reinforcement in the panels were considered: no transverse reinforcement, one No. 4 (M#13) bar at each end of the splice, two No. 4 (M#13) bars at each end of the splice, one No. 4 (M#13) bar in each course, and two No. 4 (M#13) bars in each course. The spliced bars were loaded in direct tension to determine the capacity of the splice.

Test results show that bar reinforcement placed transversely to a splice is effective at providing some degree of confinement and results in significantly improved performance and greater capacity of the splice. When compared to a similarly configured splice without lateral confining reinforcement, the addition of one No. 4 (M#13) bar placed transversely at each end of the splice resulted in an increase in splice capacity of 27%. Panels with two No. 4 (M#13) bars at each end of the splice exhibited an increase in splice capacity of 36%. Panels with one No. 4 (M#13) in each course had an increase in capacity of 34%, slightly lower than that obtained for panels with transverse reinforcement only at the ends of the splice but with significantly reduced cracking at failure. Two No. 4 (M#13) bars placed transversely in each course over the length of the splice resulted in an increase in splice capacity of 50%, with one specimen failing by bar fracture.

The effects of transverse reinforcement on splice performance should be evaluated further with the aim of understanding and quantifying the confining behavior, potentially leading to new lap splice design equations for possible inclusion in the building codes. It is recommended that other forms of confinement reinforcement also be investigated.

EFFECTS OF CONFINEMENT REINFORCEMENT ON BAR SPLICE PERFORMANCE

1.0 INTRODUCTION

1.1 Purpose

The purpose of this research study is to investigate the effects of confinement provided by lateral reinforcement on lap splice performance. This first phase of research is intended to establish if potential improvements in splice performance from confinement reinforcement exist and to provide guidance for any future research. Recognition of beneficial effects from lateral reinforcement by building codes may result in smaller required splice lengths.

1.2 Background

Through the early to mid 1990s, NCMA (in cooperation with Washington State University, the Council for Masonry Research, the Brick Industry Association, and the Western States Clay Products Association) completed a comprehensive series of tests focused on quantifying necessary lap splice requirements for reinforced masonry construction[1]. Results from these research studies pointed to the need for longer lap lengths, primarily for large diameter bars.

Although somewhat intuitive, testing confirmed that the following parameters have a significant effect on the capacity of a splice:

- compressive strength of the masonry;
- diameter of the reinforcing bar;
- length of lap; and
- clear cover.

Three basic types of failure modes were observed in the test panels of past research projects: rupture of the reinforcing steel, pullout of the reinforcement, and longitudinal splitting of the masonry. Of the 141 specimens tested by NCMA, 124 failed by splitting (see Figure 1), 16 failed due to rupture of the reinforcement, and 1 specimen failed due to pullout.

Figure 1 – Longitudinal Splitting

Past building code design equations for lap splices did not account for longitudinal splitting of the masonry. Based on the results of the previous research, a revised lap splice design equation was developed. The resulting equation submitted to building codes and reference documents and subsequently adopted is given in Equation 1. Note that the equation is for use with strength design.

The minimum length of lap splices for reinforcing bars in tension or compression, l_d, shall be calculated by Equation 1, but shall not be less than 12 in. (305 mm).

$$l_d = \frac{0.13 d_b^{2} f_y \gamma}{K\sqrt{f'_m}} \qquad \text{(Equation 1)}$$

where:

d_b = diameter of reinforcement, in.;
f_y = specified yield stress of the reinforcement or the anchor bolt, psi;
f'_m = specified compressive strength of masonry, psi;
l_d = required splice length of reinforcement, in.;
K = lesser of the masonry cover, clear spacing between adjacent reinforcement, or 5 times d_b, in.;
γ = 1.0 for No. 3 (M#10) through No. 5 (M# 16) reinforcing bars;
1.4 for No. 6 (M#19) through No. 7 (M#22) reinforcing bars; and
1.5 for No. 8 (M#25) through No. 11 (M#36) reinforcing bars.

The metric form of Equation 1 is as follows:

$$l_d = \frac{1.5 d_b^{2} f_y \gamma}{K\sqrt{f'_m}} \qquad \text{(Equation 2)}$$

where d_b, K, and l_d are in mm and f'_m and f_y are in MPa.

The result of this new equation in the design codes was a substantial increase in the required lap splice length, especially for larger diameter bars. Figure 2 illustrates the difference between the lap splice length given by Equation 1 and other design equations used in various design documents. This chart is based on 8-in. (203 mm) masonry having a compressive strength of 1,500 psi (10.3 MPa) and Grade 60 (414 MPa) reinforcement centered within the cells of the masonry.

Unlike the lap provisions in current masonry building codes, the provisions in concrete building codes recognize the effects of confinement reinforcement on splice performance, thereby permitting reduced splice lengths when confinement reinforcement is used. Confinement reinforcement for concrete applications typically consists of hoops or spirals, which is often not practical for masonry applications. However, reinforcement transverse to spliced bars is typically present in reinforcement masonry structures, and some confinement to the spliced bars may be produced by this reinforcement. It is the focus of this study to evaluate these effects and any impact on splice performance.

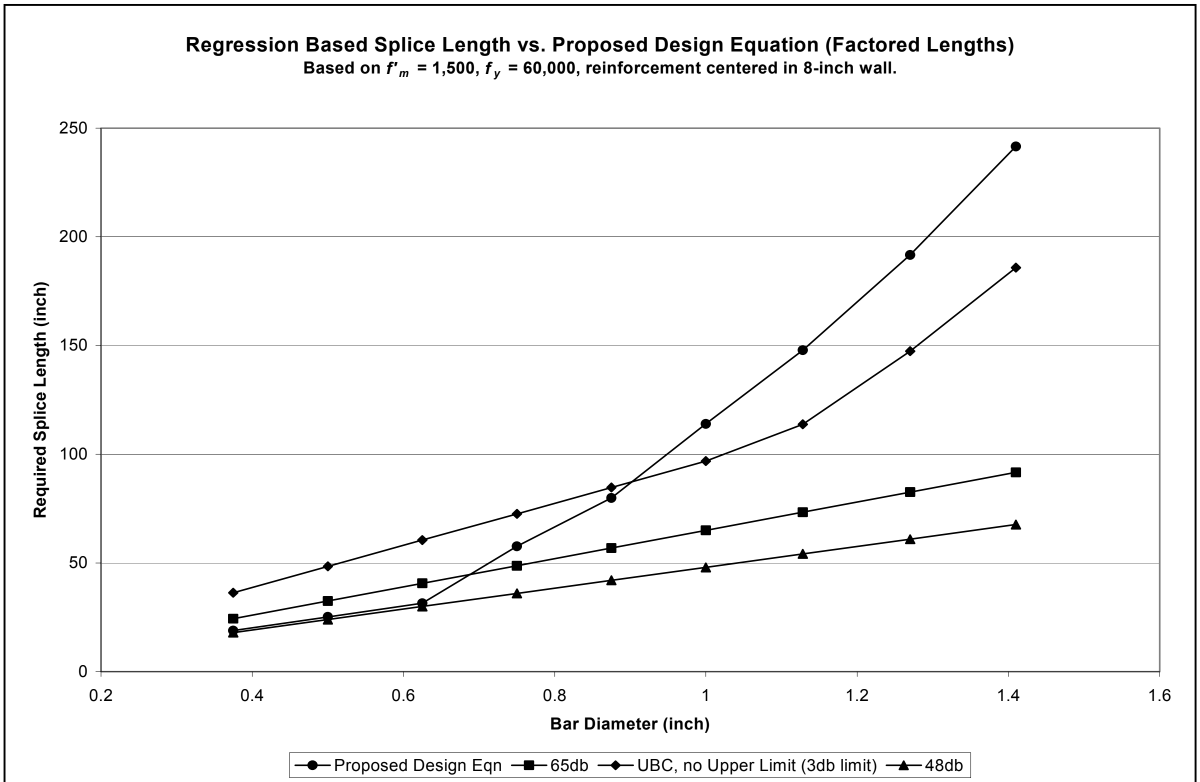

Figure 2 – Required Splice Lengths

1.3 Scope of Research

Fifteen concrete masonry panels were constructed using 8-in. (203 mm) units; consisting of five sets of specimens, each with three identical panels. All specimens were fully grouted. Two sets of No. 8 (M#25) reinforcing bars were placed in the center of the cells and incorporated a lap splice length of 48 in. (1219 mm). This combination of bar size and lap length was selected to be less than that required by Equation 1 (and inherently current code provisions through their adoption of Equation 1) and to be likely to produce splitting in the masonry in those walls without confinement reinforcement. The 48 in. (1219 mm) lap length was also selected due to the historical use of a $48d_b$ lap length, which for a No. 8 (M#25) bar yields 48 in. (1219 mm). The $48d_b$ (for Grade 60 (413.7 MPa) reinforcement) lap length was derived from an assumed limiting bond strength that could be developed between masonry grout and reinforcing steel. Based on such historical design assumptions, lap lengths less than $48d_b$ could conceivably fail due to pullout (bond failure) of the reinforcement.

To evaluate the effects of confinement reinforcement on splice behavior, five different arrangements of transverse reinforcement in the panels were considered: no transverse reinforcement, one No. 4 (M#13) bar at each end of the splice, two No. 4 (M#13) bars at each end of the splice, one No. 4 (M#13) bar in each course, and two No. 4 (M#13) bars in each course. Hooks were provided on the ends of the transverse bars to provide anchorage.

Each panel included two sets of spliced bars to reduce eccentric moments induced when loading the spliced bars in tension. For each splice, one bar protruded from the top and the other from the bottom of the panel. Each bar was loaded in direct tension to determine the capacity of the splice. The testing setup is shown in Figure 3. The width, height, and length of each panel measured nominally 8 x 48 x 40 in. (203 x 1,219 x 1,016 mm).

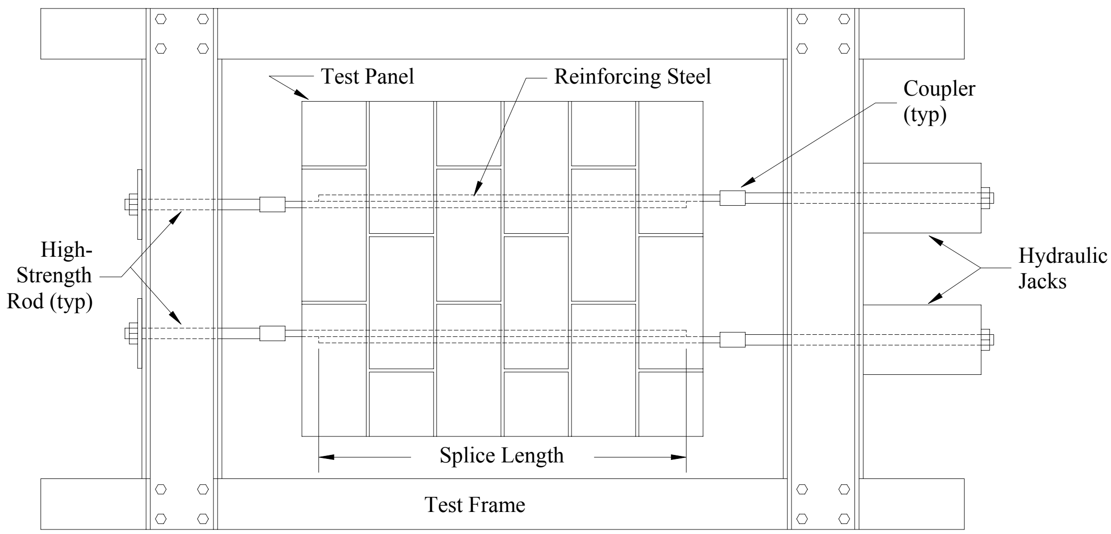

Figure 3 – Test Setup

Ancillary tests were performed to document the properties of the materials used in the research as follows:

- concrete masonry unit compressive strength;
- mortar compressive strength;
- grout compressive strength;
- masonry prism compressive strength; and
- reinforcing bar tension yield and ultimate strengths and elongations.

Results from each of these tests are given in Appendices A.2 through A.6, respectively.

2.0 MATERIALS

2.1 Masonry Units

All test specimens were constructed using concrete masonry units from the same lot. The specified dimensions of the units were 7.625 x 7.625 x 15.625 in. (194 x 194 x 397 mm) (8 x 8 x 16 in. (203 x 203 x 406 mm) nominal dimensions). All of the units had square corners and square cores.

The masonry units were sampled and tested in accordance with ASTM C 140, *Standard Test Methods of Sampling and Testing Concrete Masonry Units* [2a]. Unit test results are summarized in Table 1. Detailed results from the unit tests are given in Appendix A.2.

Table 1 – Concrete Masonry Unit Properties	
Unit Property	**Measured Value**
Net Area Compressive Strength	3,420 psi (23.6 MPa)
Oven-dry Density	116.8 pcf (1,871 kg/m^3)
Absorption	9.8 pcf (157 kg/m^3)
Dimensions • Width (W) • Height (H) • Length (L) • Face shell thickness (t_{fs}) • Web thickness (t_w)	 7.63 in. (194 mm) 7.53 in. (191 mm) 15.62 in. (397 mm) 1.24 in. (31.5 mm) 1.18 in. (30.0 mm)
Percent Solid	51.7 %

With the exception of the face shell thickness, these units complied with the applicable requirements of ASTM C 90, *Standard Specification for Loadbearing Concrete Masonry Units*[2c]. Although the measured face shell thickness of the concrete masonry units used in this research project were 0.01 in. (0.25 mm) smaller than the minimum face shell thickness required by ASTM C 90, the impact of this deviation is not felt to have a significant bearing on the observations, results, and conclusions of this research.

2.2 Mortar

Type S masonry cement 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* [2b]. Mix proportions, as parts by volume, were 1 part masonry cement and 3 parts masonry sand.

Type S masonry cement conforming to ASTM C 91, *Standard Specification for Masonry Cement* [2d] was purchased in bags from a local supplier. Masonry sand conforming to ASTM C 144, *Standard Specification for Aggregates in Masonry Mortar* [2e], 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.

The average compressive strength of 2 in. (51 mm) mortar cubes was determined in accordance with ASTM C 780, *Standard Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry* [2f], with a measured value of 1,660 psi (11.4 MPa).

Detailed results from the mortar tests are given in Appendix A.3.

2.3 Grout

A local ready-mix concrete and grout supplier furnished the grout used in the specimens of this study. All grout came from one truck. The grout was a coarse grout with mix proportions designed to produce a compressive strength of approximately 3000 psi (20.7 MPa).

Slump was determined in accordance with ASTM C 143, *Standard Test Method for Slump of Hydraulic Cement Concrete* [2g], with a measured value of 10 in. (254 mm).

The compressive strength for the grout was determined in accordance with ASTM C 1019, *Standard Test Method for Sampling and Testing Grout* [2h], with an average measured value of 3,810 psi (26.7 MPa).

Detailed results from the grout tests are given in Appendix A.4.

2.4 Masonry Prisms

At the same time as the panels were constructed, three half-length 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* [2i]. Immediately following construction, 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 bags and allowed to equalize with the temperature and moisture conditions of the laboratory environment. The average measured prism compressive strength was 3,250 psi (22.4 MPa).

Detailed results from the prism tests are given in Appendix A.5.

2.5 Reinforcing Steel

Grade 60 (413.7 MPa) deformed reinforcing bars were used for the specimens of this study. Stock bar lengths were delivered to the laboratory and were cut to specified lengths. The No. 8 (M#25) bars used for the splices contained special upset threads milled onto the ends to accommodate a threaded coupler to connect the spliced bas to the loading system. The use of the upset threads is to eliminate a weakened bar cross-section and reduce the possibility of bar failure in the threaded area. Conventional No. 4 (M#13) bars were used for the confinement reinforcement.

Tension tests were performed on the No. 8 (M#25) splice bars in accordance with ASTM A 370, *Test Methods and Definitions for Mechanical Testing of Steel Products* [2j], and the results verified to meet the requirements of ASTM A 615, *Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement* [2k]. An independent laboratory performed these tests. The average measured yield and ultimate strengths were 78,900 psi (544 MPa) and 119,400 psi (823 MPa), respectively.

Detailed results from the bar tests are given in Appendix A.6.

3.0 TEST SPECIMENS

3.1 Workmanship

Two journeyman masons with over 30 years combined experience in masonry construction constructed the test specimens using construction techniques in accordance with ACI 530.1/ASCE 6/TMS 602 [3].

3.2 Specimen Variables

Fifteen panel specimens were constructed and tested for this study, comprised of five sets of three identical specimens. Table 2 provides a summary of the test specimens.

Table 2 – Test Matrix

Specimen Set (3 Panels Per Set)	Splice Length (in.) for No. 8 (M#25) Bar	Transverse Reinforcement Providing Confinement
A	48 (1,219 mm)	None
B	48 (1,219 mm)	one No. 4 (M#13) top and bottom course
C	48 (1,219 mm)	two No. 4's (M#13) top and bottom course
D	48 (1,219 mm)	one No. 4 (M#13) each course
E	48 (1,219 mm)	two No. 4's (M#13) each course

Details of the five sets of specimens are given in Figures 4 through 8.

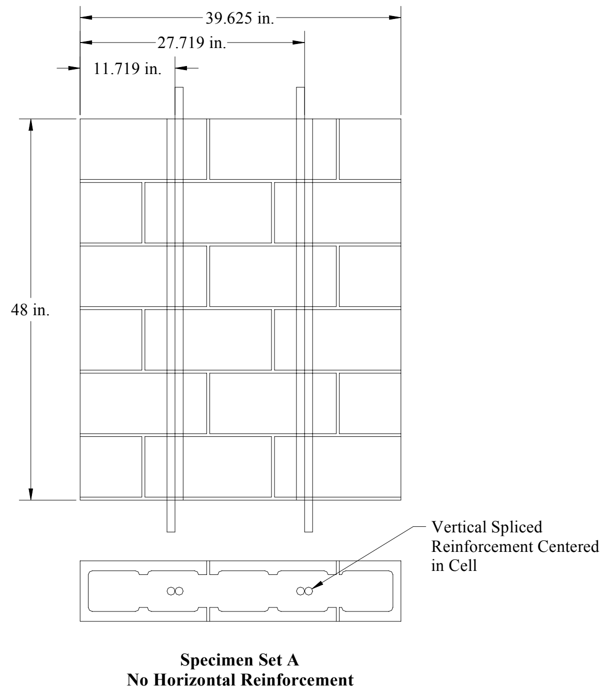

Figure 4 – Specimen Set A

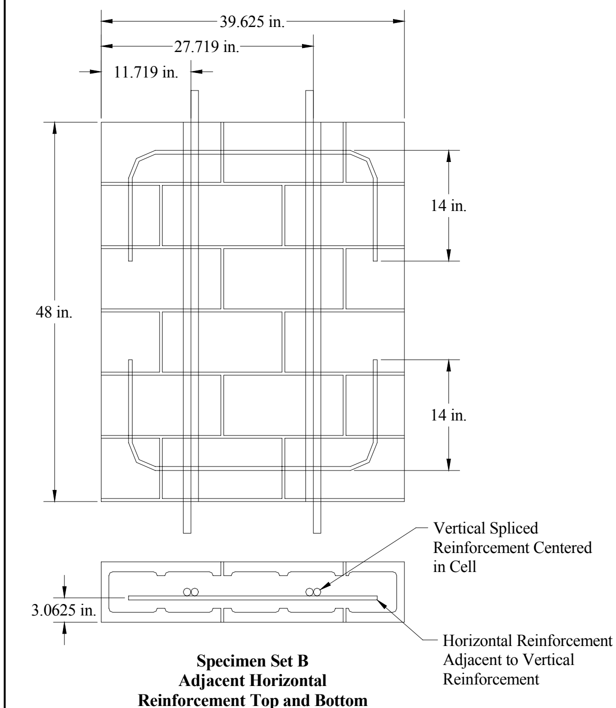

Figure 5 – Specimen Set B

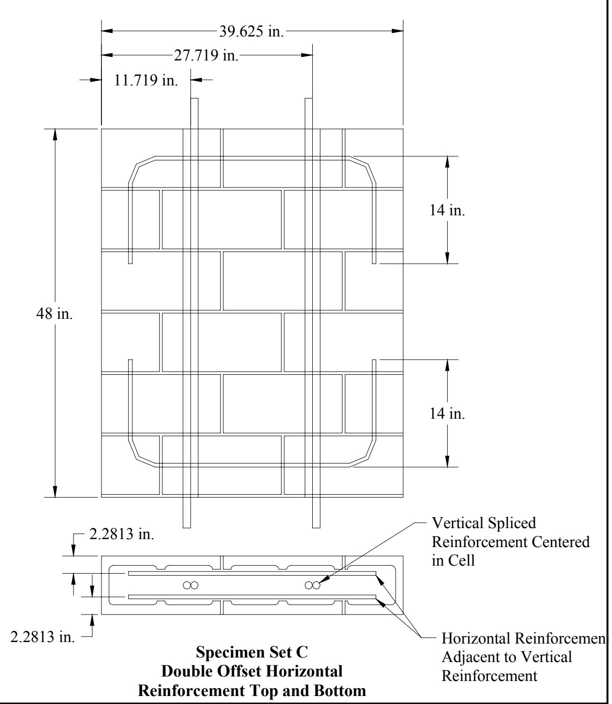

Figure 6 – Specimen Set C

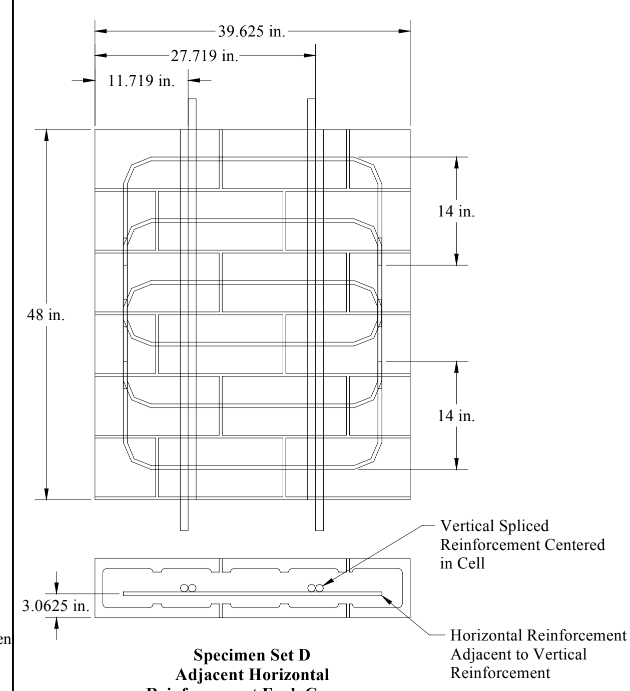

Figure 7 – Specimen Set D

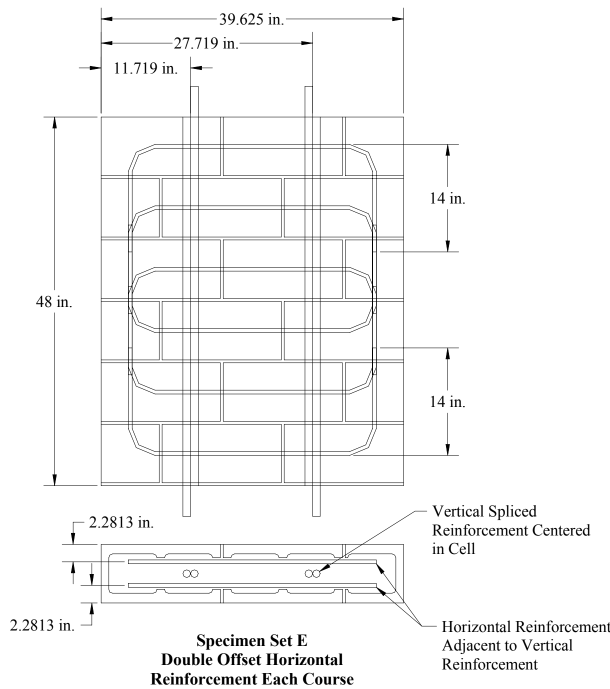

Figure 8 – Specimen Set E

3.3 Specimen Construction

The masonry panels were constructed on a dry-stacked course of concrete masonry units for easy removal of any mortar that dropped into the cores as the construction of the panels progressed. No initial leveling bed of mortar was used. The panels were laid in a running bond configuration using face shell bedding except at the ends of the panels where the end webs were mortared. Mortar joint thickness was 3/8 ± 1/8 in. (9.5 ± 3.2 mm). Mortar joints at 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 in. (13 mm) was removed with a hand trowel as the construction of the panels progressed. Hardened mortar that protruded more than ½ in. (13 mm) into the grout space was removed by rodding prior to placing the grout. Each panel contained six courses, resulting in a height of 48 in. (1219 mm).

Approximately one week after construction, each panel was carefully raised using an overhead crane. The bottom course was cleared of any debris or obstructions resulting from the construction process, and the panel was then placed on a plywood platform that served to seal the panels at the bottom for grouting. Two circular holes, slightly larger than the spliced reinforcing bars, were drilled in the plywood platform.

Reinforcing steel was secured in position using either small rigid plastic templates or small-gage steel wire. The No. 8 (M#25) spliced bars were lapped using contact splices and tied together using the small-gage wire. The No. 4 (M#13) confining reinforcement was placed in the appropriate courses and tied in place using the small-gage wire.

The grout was placed in the panels in a single lift. Grout in each core was consolidated using a ¾-in. (19 mm) diameter vibrator. The grout was reconsolidated using the same vibrator approximately 10 minutes after initial consolidation, with additional grout being added as necessary to compensate for the reduction in volume occurring due to water from the grout being absorbed by the masonry. The panels were cured under ambient conditions in the laboratory until the time of testing.

Photos of the specimens during construction are given in Figures 9 through 11.

Figure 9 – Panel Construction

Figure 10 – Placement of Confinement Reinforcement

Figure 11 – Grouting of Specimens

4.0 TEST PROCEDURES

4.1 Test Setup

A testing frame used for previous splice slice tests was adapted for use in this study. The frame was constructed of four structural steel members bolted together to form a rectangular perimeter around the test panel (see Figures 3 and 12). To alleviate the need for bracing or shoring of the testing equipment or test specimens, the structural frame was placed horizontally on the laboratory floor.

Each panel was lowered in the horizontal position into the testing frame onto two steel pipes. The pipes supported the panels in position while allowing panel movement through rolling action. A photo of the test setup is shown in Figure 12.

Once a panel was positioned in the frame, high-strength steel couplers were attached to each of the four reinforcing bars protruding from the panel. On the other end of the coupler, another reinforcing bar, threaded on both ends and having a diameter greater than the spliced bars, was attached. These connector bars extended through the holes in the steel frame and were anchored with steel washers and threaded nuts. At one end, the connector bars passed through two center-hole hydraulic rams before being anchored.

A hydraulic pump was used to supply pressure to the rams. The hydraulic hoses to the rams were connected in parallel using a "T" connector, resulting in equal pressure to each of the rams.

Figure 12 – Test Setup

4.2 Instrumentation

Force applied by each of the hydraulic rams to the splice bars was measured using 100-kip (445 kN) capacity load cells. A pressure gage was also monitored visually to confirm the load readings from the load cells. Displacement potentiometers were attached to one of the splice bars and the measuring string connected to the mating splice bar at the other end of the panel, thereby providing a rough measure of bar extension and/or slip during testing (see Figure 13). Slip of the anchorages for the potentiometers occurred in a number of walls as yielding developed in the splice bars, resulting in variability in the measured displacements.

Figure 13 – Displacement Potentiometers

4.3 Testing Procedures

In general, load was applied to the specimens at a constant rate until failure occurred, defined by rupture of the reinforcing steel or longitudinal splitting of the masonry, at which time testing was stopped. An exception to this procedure occurred for one specimen in Set D and two specimens in Set E, in which the splice bar extension due to yielding exceeded the 3 in. (76 mm) stroke capacity of the hydraulic rams. For those specimens, once maximum extension of the ram occurred, the hydraulic pressure was released, steel spacer plates were added, and the specimen reloaded until failure.

Photographs were taken during testing. Panel distress in the form of cracking in the bed joints and masonry units was monitored and recorded. An electronic data acquisition system recorded readings from the load cells and displacement potentiometers during testing.

5.0 TEST RESULTS

5.1 Test Observations

Specimen Set A: The panels of Specimen Set A contained no transverse reinforcement as confinement to the spliced bars. Very little cracking was observed in the panels prior to the development of a sudden splitting failure in the masonry directly over one of the spliced bars and running the full length of the panel, as shown in Figure 14. While major splitting did not occur over the other spliced bars, Figure 15 shows tension cracking in the end of the panel about the spliced bar (taken after removal from the testing frame). The average splice capacity for Specimen Set A was 55,400 lbs (246 kN).

Figure 14 – Typical Cracking in Specimen Set A

Figure 15 – Onset of Splitting Cracks in Specimen Set A

Additional results from the tests of the panels in Specimen Set A are given in Appendix A.1.1.

Specimen Set B: The panels of Specimen Set B contained one No. 4 (M#13) bar in the top and bottom courses of the panel (one bar at each end of the splice). Small cracks were observed in the masonry units, parallel to and directly above the splice, as well as in the mortar joints perpendicular to the splice, prior to the peak load being reached. Failure occurred suddenly when large splitting cracks occurred in the masonry over one of the spliced bars, as shown in Figure 16. Note that the splitting was primarily in the center of the panels, away from the panel ends where the No. 4 (M#13) confining bars were located. The average splice capacity for Specimen Set B was 70,500 lbs (314 kN).

Figure 16 – Typical Cracking in Specimen Set B

Additional results from the tests of the panels in Specimen Set B are given in Appendix A.1.2.

Specimen Set C: The panels of Specimen Set C contained two No. 4 (M#13) bars in the top and bottom courses of the panel (two bars at each end of the splice). Small cracks were observed in the masonry units, parallel to and directly above the splice, as well as in the mortar joints perpendicular to the splice, prior to the peak load being reached. Failure occurred suddenly when large splitting cracks occurred in the masonry over both of the spliced bars, as shown in Figure 17. As with the panels of Specimen Set B, the splitting was primarily in the center of the panels, away from the panel ends where the No. 4 (M#13) bars were located. At the ends of the panel, adjacent to the location of the No. 4 (M#13) transverse reinforcing bars, cracking occurred in the masonry units perpendicular to the splice. The average splice capacity for Specimen Set C was 75,500 lbs (336 kN).

Figure 17 – Typical Cracking in Specimen Set C

Additional results from the tests of the panels in Specimen Set C are given in Appendix A.1.3.

Specimen Set D: The panels of Specimen Set D contained one No. 4 (M#13) bar in each course of the panel. Small cracks were observed in the masonry units, parallel to and directly above the splice, as well as in the mortar joints perpendicular to the splice, prior to the peak load being reached. Significant yielding of the spliced bars occurred in all three panels, resulting in the hydraulic rams reaching their maximum stroke for one of the specimens. When maximum ram extension occurred, pressure was released, spacers installed in the loading frame, and the spliced bars reloaded until panel failure occurred. Failure occurred suddenly when splitting cracks occurred in the masonry over both of the spliced bars, as shown in Figure 18. The splitting cracks occurred over the full length of the splice. The extent of cracking and the width of the cracks were substantially less than the cracks observed in the previous panel tests. The average splice capacity for Specimen Set D was 74,500 lbs (331 kN).

Figure 18 – Typical Cracking in Specimen Set D

Additional results from the tests of the panels in Specimen Set D are given in Appendix A.1.4.

Specimen Set E: The panels of Specimen Set E contained two No. 4 (M#13) bars in each course of the panel. This arrangement of transverse reinforcement was considered to represent an upper bound on what might be practical for providing confinement reinforcement in a masonry wall using straight bars. Small cracks were observed in the masonry units, primarily perpendicular to the splice and near the ends of the panel, as well as in the mortar joints perpendicular to the splice, prior to the peak load being reached. Extensive yielding of the spliced bars occurred in all three panels, resulting in the hydraulic rams reaching their maximum stroke for two of the specimens. When maximum ram extension occurred, pressure was released, spacers installed in the loading frame, and the spliced bars reloaded until panel failure occurred. Typical cracking present in the panels of Specimen Set E is shown in Figure 19. For two of the panels, the splitting cracks in the masonry increased in size and the spliced bars slipped. In one panel, fracture of the spliced bars occurred. The average splice capacity for Specimen Set E was 82,900 lbs (369 kN).

Figure 19 – Typical Cracking in Specimen Set E

Additional results from the tests of the panels in Specimen Set E are given in Appendix A.1.5.

5.2 Discussion of Test Results

Table 3 provides a summary of the failure loads for the panels of this study.

Table 3 – Summary of Results					
Specimen Set	**Horizontal Reinforcement**	**Failure Load, lbs (kN)**	**Failure Stress in Splice Bars, ksi (MPa)**	**Ratio of Failure Stress to Measured Yield Stress in Spliced Reinforcement**	**Ratio of Failure Stress to Nominal Yield Stress in Spliced Reinforcement**
A	None	55,400 (246)	70.2 (484)	0.89	1.17
B	one No. 4 (M#13) top and bottom course	70,500 (314)	89.2 (615)	1.13	1.49
C	two No. 4's (M#13's) top and bottom course	75,500 (336)	95.6 (659)	1.21	1.59
D	one No. 4 (M#13) each course	74,500 (331)	94.3 (650)	1.20	1.57
E	two No. 4's (M#13's) each course	82,900 (369)	104.9 (724)	1.33	1.75

Based upon previous splice tests[1], and using the actual material properties for the specimens of this study, a splice capacity of approximately 51,600 lbs (230 kN) can be predicted for the splice bars in Specimen Set A. This splice capacity corresponds to a bar stress of 65.3 ksi (450 MPa). The measured capacity for the splices in Specimen Set A was 55,400 lbs (246 kN), approximately 7% larger than the predicted capacity. The effects of the various arrangements of confinement reinforcement can be evaluated by comparing the performance of the specimens with transverse reinforcement to the performance observed in Specimen Set A.

In all cases, the presence of transverse reinforcement significantly improved splice capacity. The addition of one No. 4 (M#13) bar placed transversely at each end of the splice resulted in an increase in splice capacity of 27%. Panels with two No. 4 (M#13) bars at each end of the splice exhibited an increase in splice capacity of 36%. Panels with one No. 4 (M#13) in each course had an increase in capacity of 34%, slightly lower than that obtained for panels with transverse reinforcement only at the ends of the splice but with significantly reduced cracking at failure. Two No. 4 (M#13) bars placed transversely in each course over the length of the splice resulted in an increase in splice capacity of 50%, with one specimen failing by bar fracture.

6.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of this research, the following conclusions are reached:

1. Straight-bar reinforcement placed transversely to a bar splice is effective at providing some degree of confinement and results in significantly improved performance and greater capacity of the splice.

2. Two No. 4 (M#13) bars, spaced 40 in. apart and with one bar at each end of the splice, resulted in an increase in splice capacity of 27% when compared to the capacity for similar splices without any transverse reinforcement.

3. Four No. 4 (M#13) bars, spaced 40 in. apart and with two bars at each end of the splice, resulted in an increase in splice capacity of 36% when compared to the capacity for similar splices without any transverse reinforcement. A similar increase in capacity was observed for splices with six No. 4 bars spaced evenly over the length of the splice, though reduced cracking was evident with the distributed transverse reinforcement.

4. Twelve No. 4 (M#13) bars spaced evenly over the length of the splice provided the most improvement in performance, resulting in extensive yielding of the bars and bar fracture for one specimen. The increase in capacity for this case was 50% when compared to the capacity for similar splices without any transverse reinforcement.

5. The effects of transverse reinforcement on splice performance should be evaluated further with the aim of understanding and quantifying the confining behavior, potentially leading to new lap splice design equations for possible inclusion in the building codes. Specific recommendations for topics to consider with future research include:

    a. Evaluate the effects of transverse reinforcement on lap splices using different bar sizes and lap lengths.

    b. Consider using smaller numbers of larger bars to provide the transverse reinforcement (e.g., one No. 5 (M#16) bar at each end).

    c. Develop a relationship between confinement and splice lengths as a function of the transverse reinforcement provided and the required lap length.

    d. Evaluate the effects of other forms of confinement reinforcement (e.g., spirals within cell cores, confinement plates within the mortar joints) on lap splices.

REFERENCES

[1] *Evaluation of Minimum Reinforcing Bar Splice Criteria for Hollow Clay Brick and Hollow Concrete Block Masonry*, Research Report MR 12, National Concrete Masonry Association, Herndon, VA, 1999.

[2] *2003 Annual Book of ASTM Standards,* Volumes 1.01, 1.02, 1.03, 1.04, 1.05, 4.01, 4.02, 4.05, and 4.07.

- [2a] C 140, *Standard Test Methods of Sampling and Testing Concrete Masonry Units*
- [2b] C 270, *Standard Specification for Mortar for Unit Masonry*
- [2c] C 90, *Standard Specification for Loadbearing Concrete Masonry Units*
- [2d] C 91, *Standard Specification for Masonry Cement*
- [2e] C 144, *Standard Specification for Aggregates in Masonry Mortar*
- [2f] C 780, *Standard Test Method for Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry*
- [2g] C 143, *Standard Test Method for Slump of Hydraulic Cement Concrete*
- [2h] C 1019, *Standard Test Method for Sampling and Testing Grout*
- [2i] C 1314, *Constructing and Testing Masonry Prisms Used to Determine Compliance with Specified Compressive Strength of Masonry*
- [2j] A 370, *Test Methods and Definitions for Mechanical Testing of Steel Products*
- [2k] A 615, *Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement*

[3] ACI 530.1-02/ASCE 6-02/TMS 602-02, *Specification for Masonry Structures and Commentary*, Masonry Standards Joint Committee, 2002.

APPENDIX

A.1 PANEL TEST RESULTS

A.1.1 Specimen Set A

Specimen A1:

Average load on spliced bars at failure = 54,990 lbs (245 kN)

Figure A.1 – Photo of Specimen A1 After Testing

Specimen A2:

Average load on spliced bars at failure = 55,140 lbs (246 kN)

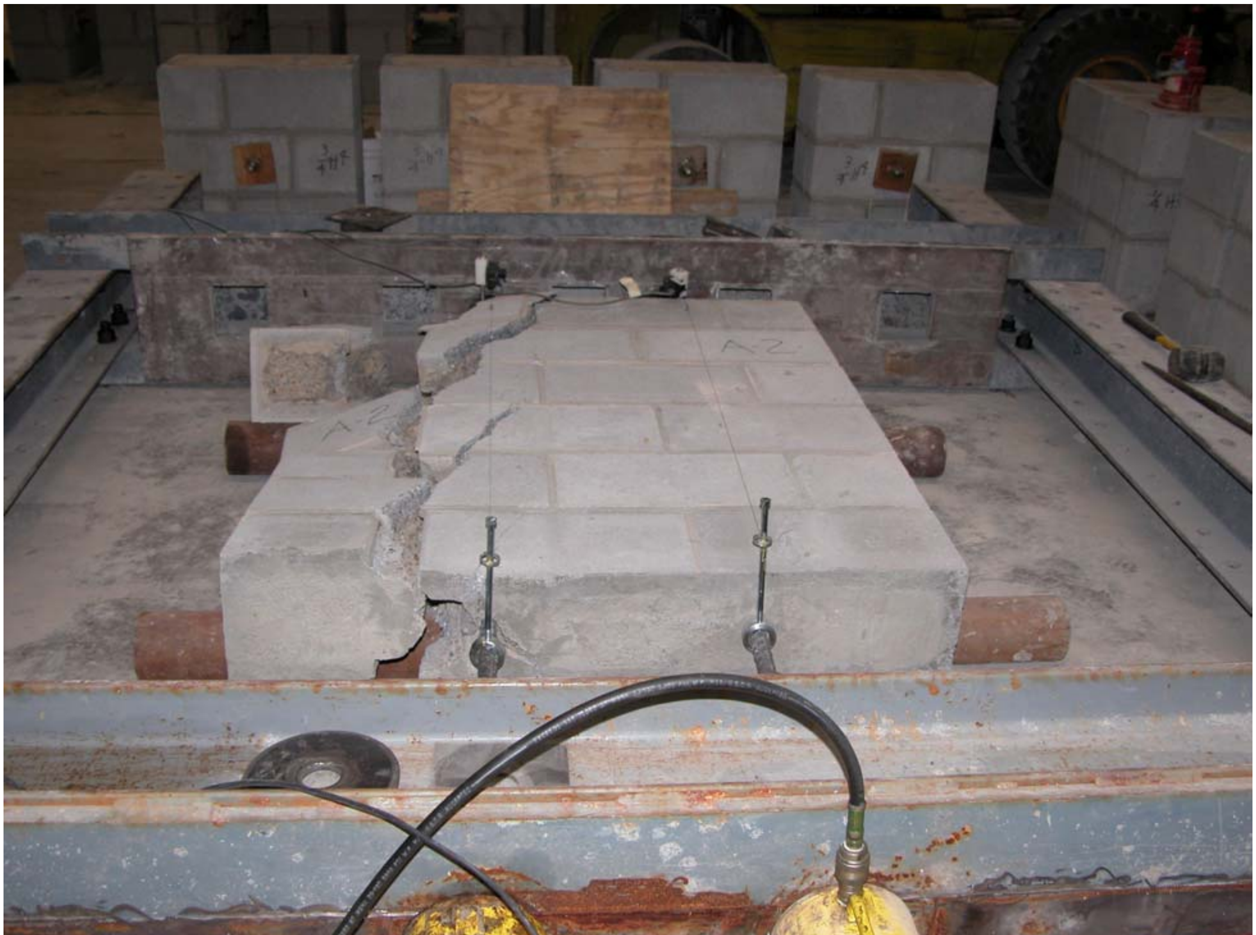

Figure A.2 – Photo of Specimen A2 After Testing

Specimen A3:

Average load on spliced bars at failure = 56,188 lbs (250 kN)

Figure A.3 – Photo of Specimen A3 After Testing

A.1.2 Specimen Set B

Specimen B1:

Average load on spliced bars at failure = 71,729 lbs (320 kN)

Figure A.4 – Photo of Specimen B1 After Testing

Specimen B2:

Average load on spliced bars at failure = 68,757 lbs (306 kN)

Figure A.5 – Photo of Specimen B2 After Testing

Specimen B3:

Average load on spliced bars at failure = 71,043 lbs (317 kN)

Figure A.6 – Photo of Specimen B3 After Testing

A.1.3 Specimen Set C

Specimen C1:

Average load on spliced bars at failure = 77,425 lbs (345 kN)

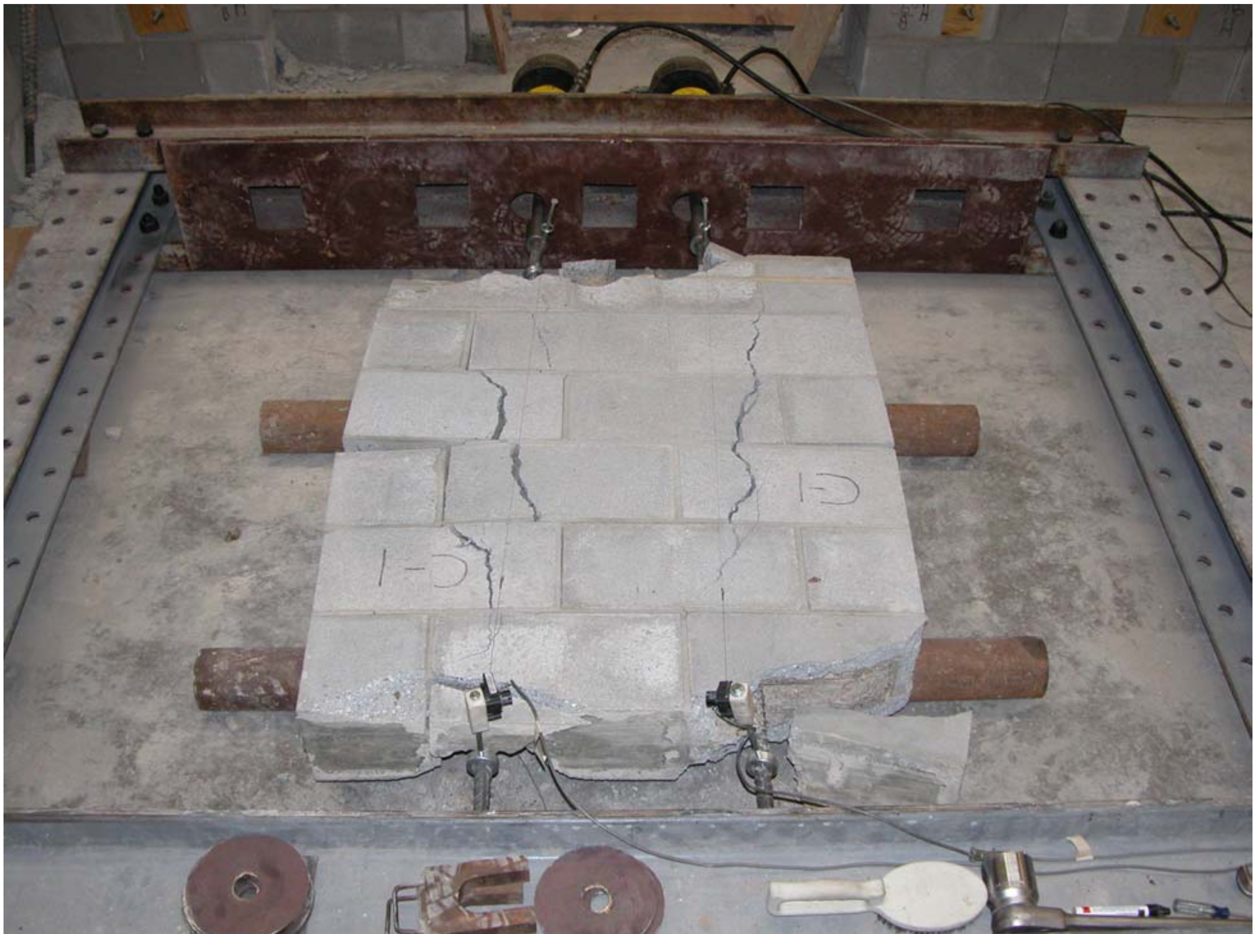

Figure A.7 – Photo of Specimen C1 After Testing

Specimen C2:

Average load on spliced bars at failure = 76,898 lbs (343 kN)

Figure A.8 – Photo of Specimen C2 After Testing

Specimen C3:

Average load on spliced bars at failure = 72,152 lbs (321 kN)

Figure A.9 – Photo of Specimen C3 After Testing

A.1.4 Specimen Set D

Specimen D1:

Average load on spliced bars at failure = 73,553 lbs (328 kN)

Figure A.10 – Photo of Specimen D1 After Testing

Specimen D2:

Average load on spliced bars at failure = 74,667 lbs (333 kN)

Figure A.11 – Photo of Specimen D2 After Testing

Specimen D3:

Average load on spliced bars at failure = 75,368 lbs (336 kN)

Figure A.12 – Photo of Specimen D3 After Testing

A.1.5 Specimen Set E

Specimen E1:

Average load on spliced bars at failure = 80,960 lbs (361 kN)

Figure A.13 – Photo of Specimen E1 After Testing

Specimen E2:

Average load on spliced bars at failure = 83,081 lbs (370 kN)

Figure A.14 – Photo of Specimen E2 After Testing

Specimen E3:

Average load on spliced bars at failure = 84,681 lbs (377 kN)

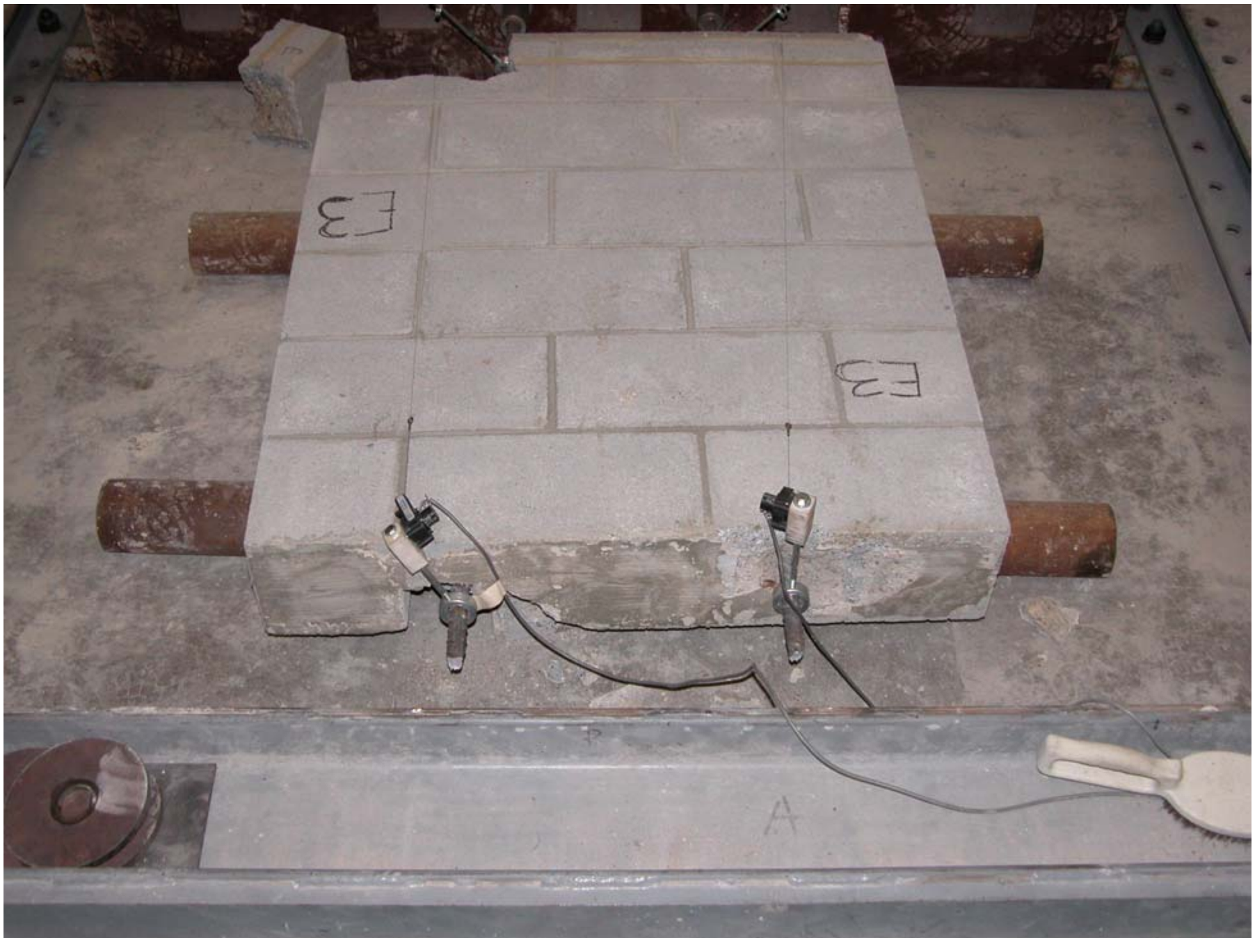

Figure A.15 – Photo of Specimen E3 After Testing

A.2 MASONRY UNIT TEST RESULTS

ASTM C 140 Test Report

Job No.: 03-389-01
Report Date: 08/25/03

Client: NCMA / Engineering Department
Address: 13750 Sunrise Valley Drive
Herndon, VA 20171-4662

Testing Agency: National Concrete Masonry Association Research and Development Laboratory
Address: 13750 Sunrise Valley Drive
Herndon, Virginia 20171-4662

Unit Specification: ASTM C90

Sampling Party: NCMA Research Lab.
Unit Producer: Allied Concrete Products
Job No./Description: Confined Splice Research

Unit Designation/Description:
8x8x16"
Hollow, Medium Weight CMU
Double square end/square core

Summary of Test Results

Physical Property	Required Values	Tested Values	
Net Compressive Strength	1900 min	3420	psi
Gross Compressive Strength	****	1770	psi
Density	****	116.8	pcf
Absorption	15 max	9.8	pcf
Percent Solid	****	51.7	%

Physical Property	Required Values	Tested Values	
Min. Faceshell Thickness (t_{fs})	1.25 min	1.24	in.
Min. Web Thickness (t_w)	1.00 min	1.2	in.
Equivalent Web Thickness	2.25 min	2.7	in.
Equivalent Thickness	****	3.9	in.
Max. Var. from Spec. Dimensions	.125 max	0.11	in.
Net Cross-Sectional Area	****	61.6	in^2
Gross Cross-Sectional Area	****	119.1	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	32.73	119.12	61.60	215120	1810	3490
	Unit #2	32.42	119.12	61.60	209640	1760	3400
	Unit #3	32.38	119.12	61.60	208460	1750	3380
	Average	32.51	119.12	61.60	211070	1770	3420

* 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. t_{fs}**	Min. t_w
		in.	in.	in.	in.	in.
Absorption Units	Unit #4	7.62	7.55	15.63	1.23	1.19
	Unit #5	7.64	7.53	15.62	1.24	1.18
	Unit #6	7.63	7.52	15.60	1.24	1.18
	Average	7.63	7.53	15.62	1.24	1.18

**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
	lb	lb	lb	lb	pcf	pcf	ft^3	%
Unit #4	32.62	17.31	34.13	31.49	9.8	116.8	0.2696	51.9
Unit #5	32.53	17.19	33.94	31.30	9.8	116.6	0.2684	51.7
Unit #6	32.38	17.16	33.84	31.23	9.8	116.8	0.2673	51.6
Average	32.51	17.22	33.97	31.34	9.8	116.8	0.2684	51.7

A.3 MORTAR TEST RESULTS

NCMA Research and Development Laboratory
ASTM C 780 Worksheet - Preconstruction and Construction Evaluation of Mortars for Plain and Reinforced Unit Masonry

Job No.: 03-389
Report Date: 07/12/04

Client: NCMA Engineering Department
Address: 13750 Sunrise Valley Drive
Herndon VA, 20171-4662

Mortar Sample No.: Mix 1
Corresponding Wall/Specimen: (Wall Panels 1 through 15)
Mortar Type: Type S masonry cement mortar

Project /Description: Confined Splice Research
Comments: Masonry cement provided by NCMA Lab.

NCMA Lab Aggregate Unit Weight = 80 pcf
Weight of Cement Bag = 75 lb.

Batch Information (C270)

Batch Factor = Agg Wt /(Agg Unit Wt x Agg. Vol. Proportion) = ____ / (___ x ___) = ____
Cement Weight = Cmt Prop x Bag Weight x Batch Factor = __ x ___ x ___ = ____ lb.
Lime Weight = Lime Prop x 40 x Batch Factor = _____ x 40 x _____ = ____ lb.
Aggregate Weight = Agg Prop x Agg Unit Weight x Batch Factor = ___ x ___ x ___ = ____ lb.
Agg:Cmt Proportion = Agg Prop / (Cement Prop + Lime Prop) = 3 / 1 = 3

Material	Type/Brand/Source	Volume Proportions	Weight (lb.)		
Portland Cement	__________	________	*****		
Lime	__________	________	*****		
Masonry Cement	Essroc	1	15.0	Mixed By:	DR
Masonry Sand	C144 Sand / NCMA Lab	3	50.0	Date:	09/03/03
Water Added to Mix	Tap Water	________	Unknown		
Admixture	__________	________	*****		
		Total Wt. =	65.0		

2-inch Cube Compressive Strength (C 780)

Cube Age: 28 days

Cube #	Cube Wt (g)	Load (lbs)	Cube Strength (psi)
1	269.60	6740	1685
2	266.80	6580	1645
3	268.50	6560	1640
Average	**268.30**	**6627**	**1660**

Testing by: LB   Date: 12/17/03

Cone Penetration (C 780)

Penetration (spaded) = 63 mm

Cube Curing History: 28 days+ moist cure

Jeffrey H. Greenwald, P.E.
Vice President of Research and Development

A.4 GROUT TEST RESULTS

NCMA Research and Development Laboratory
ASTM C 1019: Sampling and Testing Grout

Project No.: 03-389
Report Date: 12/23/03

Client: NCMA Engineering Department
Address: 13750 Sunrise Valley Drive
Herndon VA, 20171-4662

Testing Agency: National Concrete Masonry Association
Research and Development Laboratory
Address: 13750 Sunrise Valley Drive
Herndon VA, 20171-4662

Project /Description: Confined Splice Research

Sampling Party: Research and Development Laboratory

Mix Design: Course Grout (appx. 3000 psi strength)
Specimen Set No.: (Wall Panels 1 through 15)
ASTM C 143 Slump: 10 inches

Date Made: 10/01/03
Date Tested: 12/17/03
Tested By: L Breeding

		28 Days+ Break			
		Specimen 1	Specimen 2	Specimen 3	Average
Height (in.)	1	7.385	7.463	7.463	
(H = 2W)	2	7.350	7.477	7.440	
	3	7.333	7.49	7.414	
	4	7.379	7.48	7.444	
Average		**7.36**	**7.48**	**7.44**	**7.43**
Width (in.)	1	3.782	3.633	3.601	
(> 3 inches)	2	3.642	3.608	3.735	
	3	3.813	3.630	3.579	
	4	3.631	3.579	3.674	
Average		**3.72**	**3.61**	**3.65**	**3.66**
Weight (lb.)		8.14	7.99	8.11	**8.08**
Plumb (in.)		0.13	0	0.06	**0.06**
Plumb (%)		1.7	0.0	0.8	**0.4**
Compressive Load (lb.)		51720	49220	51940	**50960**
Compressive Strength (psi)		**3743**	**3772**	**3905**	**3810**

Curing Conditions: 1 day in mold & 28 days+ in moist cabinet

Failure mode: Conical - Shear

Jeffrey H. Greenwald, P.E.
Vice President of Research and Development

A.5 MASONRY PRISM TEST RESULTS

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

Project No.: 03-389-01
Report Date: 12/19/03

Client: NCMA / Engineering Department
Address: 13750 Sunrise Valley Drive
Herndon, VA 20171-4662

Testing Lab: National Concrete Masonry Association
Research and Development Laboratory
13750 Sunrise Valley Drive
Herndon, VA 20171-4662

Project Identification: Confined Splice Research
Prism Identification: 8x16x8", Grouted, Stack Bond, Concrete Masonry Prism

Gout Mix No.: Mix #1

Prism Details:
Number of Mortar Bed Joints: 1
Number of Masonry Units Used: 2
Date Constructed: 09/03/03
Date Grouted: 10/01/03
Date Retrieved from Site: N/A
Date Delivered to Lab: N/A
Date Tested: 12/17/03

Masonry Unit Information:
Unit Supplier: Allied Concrete Products
Unit Dimensions: 8x8x16"
Unit Net Area (hollow units): 61.6 in^2

Grout Information
Grout Supplier / Preparer: Tarmac Concrete
Grout Type / Description: Coarse Grout (Mix #1)
Grout Slump (ASTM C 143): 10 in.
Method of Consolidation: Vibrator

Mortar Information
Mortar Supplier / Preparer: NCMA Lab
Mortar Type / Description: Type S

Compression Test Machine Information
Diameter of Spherical Seat: 10 in.
Required Upper Bearing Plate Thickness: 0.7 in.
Required Lower Bearing Plate Thickness: 1.0 in.
Provided Upper Bearing Plate Thickness: 5.1 in.
Provided Lower Bearing Plate Thickness: 2.5 in.

Tested Prism Properties:

Prism No.	Age at Test (days)	Avg. Width (in.)	Avg. Height (in.)	Avg. Length (in.)	Gross Area (in^2)	Max Load (lb.)	Gross Compr. Strength (psi)	h/t Ratio	h/t CF*	Corrected Gross Strength (psi)
1	____	7.70	15.65	8.43	64.87	217980	3360	2.03	1.00	3370
2	____	7.70	15.65	8.41	64.78	204060	3150	2.03	1.00	3160
3	____	7.70	15.60	8.44	64.97	209400	3223	2.03	1.00	3230
									Average	3250

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

Compressive strength of masonry (average for the three 28 days+ prisms): **3250** **psi**

Comments:
1. Building codes require prism strengths to be reported on a net area basis. These prisms are grouted solid, therefore gross area is equivalent to net area.

Jeffrey H. Greenwald, P.E.
Vice President of Research and Development

A.6 REINFORCING STEEL TEST RESULTS

City Testing & Research Laboratories, Inc.

TESTING RESEARCH INSPECTION CTRL

967 E. HAZELWOOD AVE.
RAHWAY, N.J. 07065

TEL: (732) 388-7711
FAX: (732) 388-5683

REPORT OF TESTS

Reference No. S-106975

MATERIAL Grade 60 Deformed Reinforcing Bars, No. 8

Your Order No. NCMA LAB-IRP-03-389

FROM National Concrete Masonry Assoc.
Herndon, VA 20171-4662

Specification No. ASTM A 615-01b
Grade 60

Specimen Number	1	2	3			
Marked						
Dimensions, in.	0.503	0.510	0.505			
Area, sq. in.	0.199	0.204	0.200			
Dimensions after Fracture, in.						
Fractured Area, sq. in.						
Yield Strength, lbs. actual @ 0.2% or Extension under load, in./2 in.	15,800	16,200	15,600			
Maximum Load, lbs. actual	23,700	24,300	24,000			
Elongation in 2 inches	0.38	0.38	0.38			
				Required		
Yield Strength, lbs. per sq. in.	79,400	79,400	78,000	60,000	Minimum	
Tensile Strength, lbs. per sq. in.	119,100	119,100	120,000	90,000	Minimum	
Percent Elongation in 2 inches	19.0	19.0	19.0	9.0	Minimum	
Percent Reduction of Area						

REMARKS

The submitted rebar conforms to the requirements noted.

We certify that this report is a true report of results obtained from our tests of this material

Subscribed and sworn to before m
this .... day of .... 20.....

Tests by

N. Copeland,
Mechanical Lab Manager

Date Dec. 29, 2003

Witnessed by