EFFECTS OF CONFINEMENT REINFORCEMENT ON THE PERFORMANCE OF LAP SPLICES IN CONCRETE MASONRY

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SUMMARY

This paper provides an update to ongoing research investigating the performance of lap splices in masonry when confined by transverse reinforcement.

INTRODUCTION

Research conducted in the 1990s to quantify the performance of lap splices in reinforced masonry construction identified failure modes not previously accounted for in masonry design standards (CMR 1999). While conclusive in its investigation, the results of this research program pointed to the need for longer lap lengths, primarily for large diameter bars or small cover distances, both of which encourages longitudinal splitting of the masonry along the length of the lap splice as shown in Figures 1 and 2.

Figure 1. Example of Longitudinal Splitting

Figure 2. Catastrophic Longitudinal Splitting

The culmination of this research, as well as subsequent discussion and analysis, resulted in equation (1) for calculating the minimum required length of lap for splices in masonry

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

$$l_d = \frac{1.5 d_b^{\,2} f_y \gamma}{K\sqrt{f'_m}} \qquad \left[l_d = \frac{0.13 d_b^{\,2} f_y \gamma}{K\sqrt{f'_m}} \quad \text{inch}-\text{pounds} \right] \tag{1}$$

Where:
d_b = diameter of reinforcement, mm (in.)
f_y = specified yield stress of the reinforcement, MPa (psi)
f'_m = specified compressive strength of masonry, MPa (psi)
l_s = required lap splice length of reinforcement, mm (in.)
K = the lesser of the masonry cover, clear spacing between adjacent reinforcement, or 5 times d_b, mm (in.)
γ = 1.0 for 10 mm (No. 3) through 16 mm (No. 5) reinforcing bars;
1.3 for 19 mm (No. 6) thought 22 mm (No. 7) reinforcing bars;
1.5 for 25 mm (No. 8) through 36 mm (No. 11) reinforcing bars.

Equation (1), as adopted into the 2005 MSJC standard (ACI 530/ASCE 5/TMS 402 2005), directly accounts for longitudinal splitting of the masonry along the length of the lap splice and indirectly accounts for pullout (bond failure) of the reinforcement through lower limits on permitted lap splice lengths. The result of this change to the U.S. design codes was a substantial increase in the required lap splice length, particularly for larger diameter bars or small cover distances. For some combinations of bar diameter and cover distance, the required length of lap using equation (1) increased two to four times that historically used in masonry construction. Because unexplainable lap splice failures have not been documented in the U.S. following natural disasters or excessive loading events using the historical lap splice lengths, conjecture pointed to some other previously unaccounted for mechanism increasing the strength and performance of lap splices in masonry. It was this premise that formed the basis of this research investigation.

ANALYTICAL CONCEPT

As documented in previous research (CMR 1999), when loaded in direct tension the dominant mode of failure of lap splices in masonry construction was longitudinal splitting of the masonry along the length of the lap. Figure 3 shows the bottom of a test specimen with the initiation of a longitudinal crack forming transverse to the lap spliced reinforcement indicating impending failure of the splice. Once developed, this crack propagates almost instantaneously along the length of the lap releasing the spliced reinforcing bars, sometimes with catastrophic results as shown in Figure 2.

In concept, spanning the longitudinal crack with reinforcement running transverse to the lap splice as shown in Figure 4 would mitigate the propagation of the crack and increase the strength of the lap splice. The purpose of this research was to quantify the effect the transverse reinforcement has on confining the lap splice as well as document how detailing variables, such as location and quantity of the confining reinforcement, influences this behavior.

Figure 3. Initiation of Longitudinal Splitting

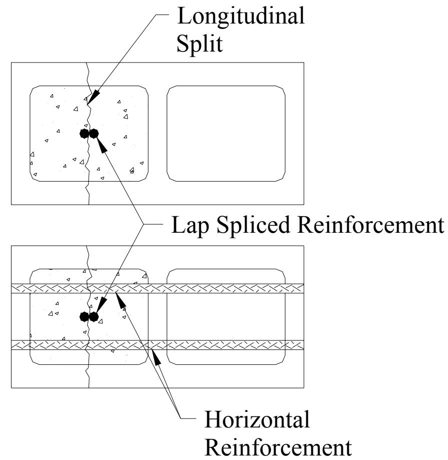

Figure 4. Confinement of Longitudinal Splitting

TESTING PROGRAM

Seventy two concrete masonry panels were constructed using 203 mm and 305 mm (8 in. and 12 in.) units. Each specimen set was replicated three times for a total of 24 unique sets of test specimens. Two sets of lap spliced 25 mm (No. 8) reinforcing bars were placed in the center of two of the cells of each panel. Four different lap lengths were considered: 1,220 mm (48 in.), 1,016 mm (40 in.), 813 mm (32 in.), and 610 mm (24 in.). This combination of bar size and lap length was selected to be less than that required by equation (1) and to be likely to produce longitudinal splitting in the masonry in those panels without confinement reinforcement based on the observed performance of similarly configured specimens tested in past research investigations (CMR 1999). All specimens were fully grouted and the height of the specimens was varied to correspond to the length of the tested lap splice.

To evaluate the effects of confinement reinforcement on splice behavior, ten different arrangements of transverse reinforcement in the panels were considered: no transverse reinforcement, one No. 4 (M#13) bar near each end of the splice, two No. 4 (M#13) bars near each end of the splice, one No. 4 (M#13) bar in each course of the panel, and two No. 4 (M#13) bars in each course of the panel. Standard 90-degree hooks were provided on the ends of the transverse bars to anchor the horizontal reinforcement. An elevation and cross-section of a typical test specimen is illustrated in Figure 5. Table 1 summarizes all specimens evaluated in this program.

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 frame was constructed of four structural steel members bolted together to form a rectangular perimeter around the test panel as illustrated in Figure 6. 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.

Table 1. Summary of Test Specimens

Specimen	Splice Length	Specimen Thickness	Confining Reinforcement
3A-1	1,220 mm (48 in.)	203 mm (8 in.)	None
3A-2	1,016 mm (40 in.)	203 mm (8 in.)	
3A-3	813 mm (32 in.)	203 mm (8 in.)	
3A-4	610 mm (24 in.)	203 mm (8 in.)	
3B-1	1,220 mm (48 in.)	203 mm (8 in.)	One 13 mm (No. 4) transverse bar top and bottom course only
3B-2	1,016 mm (40 in.)	203 mm (8 in.)	
3B-3	813 mm (32 in.)	203 mm (8 in.)	
3B-4	610 mm (24 in.)	203 mm (8 in.)	
3C-1	1,220 mm (48 in.)	203 mm (8 in.)	Two 13 mm (No. 4) transverse bars top and bottom course only
3C-2	1,016 mm (40 in.)	203 mm (8 in.)	
3C-3	813 mm (32 in.)	203 mm (8 in.)	
3C-4	610 mm (24 in.)	203 mm (8 in.)	
3D-1	1,220 mm (48 in.)	203 mm (8 in.)	One 19 mm (No. 6) transverse bar top and bottom course only
3D-2	610 mm (24 in.)	203 mm (8 in.)	
3E-1	1,220 mm (48 in.)	203 mm (8 in.)	Two 19 mm (No. 6) transverse bars top and bottom course only
3E-2	610 mm (24 in.)	203 mm (8 in.)	
3F-1	1,220 mm (48 in.)	203 mm (8 in.)	One 25 mm (No. 8) transverse bar top and bottom course only
3F-2	610 mm (24 in.)	203 mm (8 in.)	
3G-1	1,220 mm (48 in.)	203 mm (8 in.)	Two 25 mm (No. 8) transverse bars top and bottom course only
3G-2	610 mm (24 in.)	203 mm (8 in.)	
3H-1	1,016 mm (40 in.)	305 mm (12 in.)	None
3H-2	1,016 mm (40 in.)	305 mm (12 in.)	Offset splice with one 13 mm (No. 4) transverse bar near cover adjacent
3H-3	1,016 mm (40 in.)	305 mm (12 in.)	Offset splice with one 13 mm (No. 4) transverse bar far cover adjacent
3H-4	1,016 mm (40 in.)	305 mm (12 in.)	Offset splice with one 13 mm (No. 4) transverse bar offset

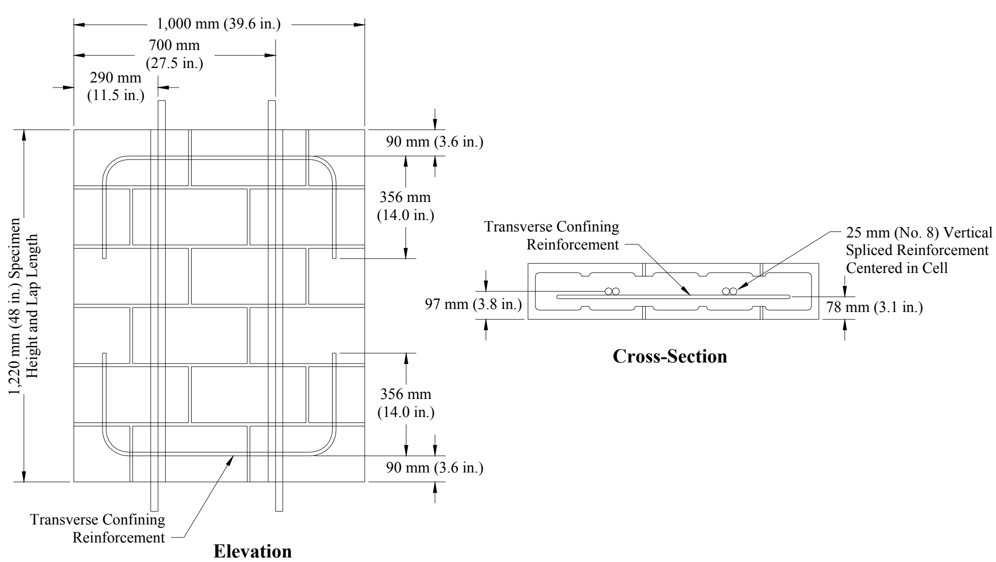

Figure 5. Typical Test Specimen

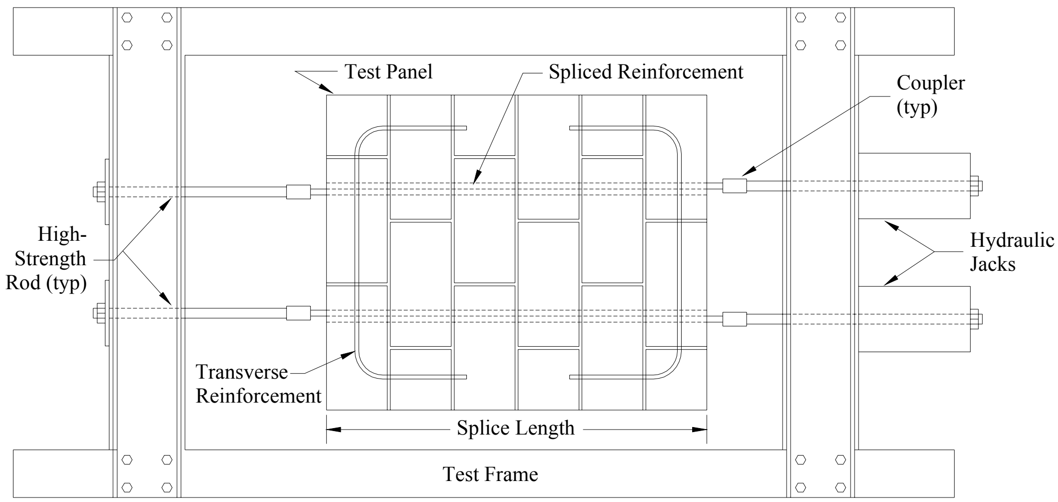

Figure 6. Test Setup and Loading Configuration

Materials

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.

The concrete masonry units were sampled and tested in accordance with ASTM C 140, *Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units* (ASTM C 140 2007). Unit test results are summarized in Table 2. All concrete masonry units used in this study complied with the applicable requirements of ASTM C 90, *Standard Specification for Loadbearing Concrete Masonry Units* (ASTM C 90 2006).

Table 2. Concrete Masonry Unit Properties

Unit Property	Measured Value	
	203 mm (8 in.)	305 mm (12 in.)
Net Area Compressive Strength	26.5 MPa (3,850 psi)	15.3 MPa (2,220 psi)
Oven-dry Density	1988 kg/m^3 (124.1 lb/ft^3)	1,873 kg/m^3 (116.9 lb/ft^3)
Absorption	157 kg/m^3 (9.8 lb/ft^3)	202 kg/m^3 (12.6 lb/ft^3)
Dimensions		
• Width (W)	194.1 mm (7.64 in.)	295.9 mm (11.65 in.)
• Height (H)	192.5 mm (7.58 in.)	192.8 mm (7.59 in.)
• Length (L)	396.5 mm (15.61 in.)	396.7 mm (15.62 in.)
• Face shell thickness (t_{fs})	32.5 mm (1.28 in.)	40.6 mm (1.60 in.)
Web thickness (t_w)	35.1 mm (1.38 in.)	38.4 mm (1.51 in.)
Percent Solid	54.4 %	49.6 %

Type S masonry cement mortar was used to construct all panels and prisms. Constituent mortar materials and mixing procedures were in accordance with applicable ASTM standards. Mix proportions, as parts by volume, were one part masonry cement and three parts masonry sand. The average compressive strength of 51 mm (2 in.) 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* (ASTM C 780 2006). Mortar was evaluated in two batches. The first batch was used to construct the 8 inch masonry specimens and had an average measured compressive strength of 20.7 MPa (3,000 psi). The second batch was used to construct the 12 inch masonry specimens and had an average measured compressive strength of 12.1 MPa (1,750 psi). The differences in the measured compressive strengths of the mortar is attributed to the different times in which the mortar batches were mixed, which necessitated varying the volume of mixing water to achieve quality construction, rather than a tangible variation in the mortar's constituent materials. The differences in the mortar compressive strength are not expected to influence the results or conclusions of this investigation.

A local ready-mix concrete and grout supplier furnished the coarse grout used in the specimens of this study. Due to the total volume of grout necessary for this investigation the grout was provided in two truckloads. The compressive strength for the grout was determined in accordance with ASTM C 1019, *Standard Test Method for Sampling and Testing Grout* (ASTM C 1019 2007). The average measured compressive strength of the grout from the first truck was 18.3 MPa (2,650 psi), whereas the average compressive strength of the grout from the second truck was 21.9 MPa (3,170 psi).

At the same time as the panels were constructed, half-length grouted masonry prisms were constructed in accordance with ASTM C 1314, *Standard Test Method for Compressive Strength of Masonry Prisms* (ASTM C 1314 2003), using the 203 mm (8 in.) concrete masonry units. One set of prisms was constructed for each truckload of grout. The average measured prism compressive strength using grout from the first truck was 22.0 MPa (3,190 psi), whereas the average prism compressive strength using grout from the second truck was 23.0 MPa (3,340 psi).

Conventional 414 MPa (60,000 psi) 25 mm (No. 8) deformed bars were used for the spliced bars. The bars used for the splices contained upset threads milled onto the ends to accommodate a threaded coupler to connect the spliced bars to the loading system. The use of the upset threads attempted to eliminated a weakened bar cross-section and reduce the possibility of bar failure in the threaded area. For lateral confinement, 13 mm (No. 4), 19 mm (No. 6), and 25 mm (No. 8) conventional 414 MPa (60,000 psi) deformed bars were used.

Tension tests were performed on the 25 mm (No. 8) splice bars in accordance with ASTM A 370, *Standard Test Methods and Definitions for Mechanical Testing of Steel Products* (ASTM A 370 2007). The physical properties of the 25 mm (No. 8) mild reinforcement met the requirements of ASTM A 615, *Standard Specification for Deformed and Plain Carbon-Steel Bars for Concrete Reinforcement* (ASTM A 615 2007). The average measured yield and ultimate strengths were 497.0 MPa (72,070 psi) and 744.6 MPa (108,000 psi), respectively.

Specimen Construction

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. The height of each specimen was varied to fully encompass the splice length. The 25 mm (No. 8) spliced bars were lapped using contact splices and tied together using small-gage wire. The transverse confining reinforcement was placed in the appropriate courses and tied in place using small-gage wire. Figure 7 shows placement of the confinement reinforcement within the walls.

The grout was placed in the panels in a single lift and mechanically consolidated following placement as shown in Figure 8. The panels were cured under ambient conditions in the laboratory until the time of testing.

Figure 7. Placement of Reinforcement

Figure 8. Consolidation of Grout

Testing

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.

Force applied by each of the hydraulic rams to the splice bars was measured using 445 kN (100,000 lb) 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. 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.

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. Early during the testing, fracture of the steel rebar was seen in the upset thread portion of the bar on some specimens. This occurred below the ultimate strength of the bar as tested. It was determined that the upset threaded portion of the spliced reinforcing were not able to carry the applied tension loads. A new high strength steel coupler was designed, and these couplers were fabricated by a local machine shop. After these couplers were put into use, fracture in the threads was not seen again during testing.

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.

TEST RESULTS AND OBSERVATIONS

The data obtained by this research is summarized in Table 3. Due to equipment malfunction or premature failure of the threaded connections or specimens, several data points were lost and as such are not included in Table 3. These include: Specimens 3A-1C, 3B-2B, 3D-1A, 3E-1C, and 3F-1B. A more comprehensive review of the test results and observations is provided in Effects of Confinement Reinforcement on Bar Splice Performance – Phase III (NCMA, 2007).

Effect of Confinement Reinforcement Placement

The purpose of Specimen Sets 3H-1 through 3H-4 was to quantify whether the relative placement of the confining reinforcement influenced the strength or performance of the lap splice. As reported in an earlier phase of this research project (Greenwald, et al. 2005), changing the placement of the horizontal reinforcement longitudinally along the length of the lap affected both the lap strength and performance.

In concept, placing the transverse reinforcement between the spliced reinforcement and the nearest outside cover consistent with Specimen Set 3H-2 would prove to be more effective at mitigating longitudinal splitting than placing the confining reinforcement on the deeper cover side of the spliced reinforcement, as done with Specimen Set 3H-3. In addition, moving the confining reinforcement further away from the spliced reinforcement, as shown in Figure 9 for Specimen Set 3H-4, would conceptually decrease the strength and performance of the splice further.

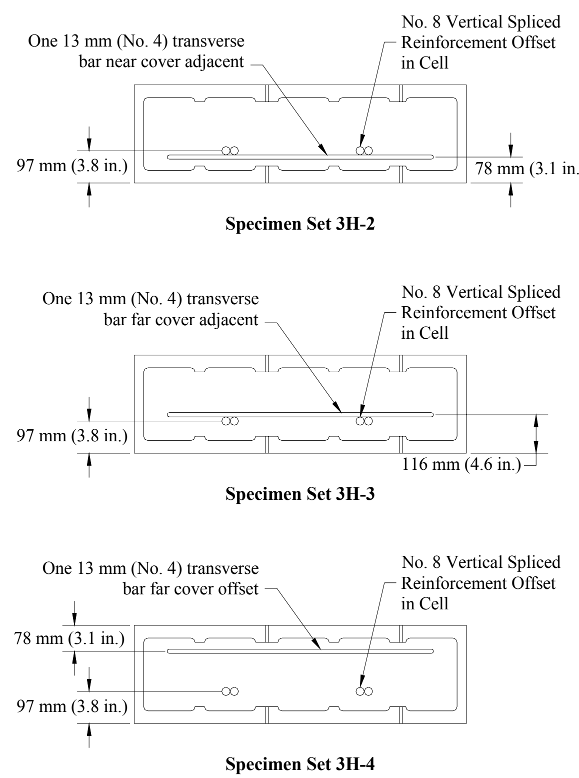

Figure 9. Offset Confinement Locations

Table 3. Summary of Test Results

Specimen Set	Average Maximum Failure Load, kN (lb)	Average Maximum Failure Stress, MPa (psi)	Standard Deviation, kN, (lb)	Ratio of Failure Stress to: Nominal Yield Stress	Ratio of Failure Stress to: Measured Yield Stress	Ratio of Failure Stress to: Measured Ultimate Stress
3A-1	273.8 (61,550)	537.2 (77,910)	8.1 (1,810)	1.30	1.08	0.72
3A-2	262.8 (59,080)	515.6 (74,780)	18.2 (4,100)	1.25	1.04	0.69
3A-3	223.2 (50,170)	437.9 (63,510)	9.4 (2,120)	1.06	0.88	0.59
3A-4	145.0 (32,590)	284.4 (41,250)	43.7 (9,810)	0.69	0.57	0.38
3B-1	296.4 (66,630)	581.5 (84,340)	3.2 (730)	1.41	1.17	0.78
3B-2	296.1 (66,560)	580.9 (84,250)	0.7 (160)	1.40	1.17	0.78
3B-3	243.6 (54,770)	478.0 (69,330)	12.6 (2,840)	1.16	0.96	0.64
3B-4	196.7 (44,220)	385.9 (55,970)	13.9 (3,120)	0.93	0.78	0.52
3C-1	319.2 (71,760)	626.3 (90,840)	9.7 (2,190)	1.51	1.26	0.84
3C-2	312.7 (70,290)	613.4 (88,970)	15.6 (3,520)	1.48	1.23	0.82
3C-3	253.4 (56,970)	497.1 (72,100)	16.0 (3,600)	1.20	1.00	0.67
3C-4	206.8 (46,500)	405.8 (58,860)	18.4 (4,130)	0.98	0.82	0.55
3D-1	318.4 (71,570)	624.6 (90,590)	1.6 (370)	1.51	1.26	0.84
3D-2	192.2 (43,210)	377.2 (54,700)	8.0 (1,790)	0.91	0.76	0.51
3E-1	328.5 (73,860)	644.6 (93,490)	3.1 (700)	1.56	1.30	0.87
3E-2	209.2 (47,040)	410.5 (59,540)	10.2 (2,290)	0.99	0.83	0.55
3F-1	331.5 (74,520)	650.4 (94,330)	4.9 (1,100)	1.57	1.31	0.87
3F-2	201.4 (45,270)	395.1 (57,300)	12.9 (2,900)	0.96	0.80	0.53
3G-1	344.7 (77,500)	676.4 (98,100)	11.5 (2,600)	1.64	1.36	0.91
3G-2	245.5 (55,180)	481.6 (69,850)	18.3 (4,120)	1.16	0.97	0.65
3H-1	288.9 (64,940)	566.8 (82,200)	23.8 (5,350)	1.37	1.14	0.76
3H-2	319.6 (71,850)	627.1 (90,950)	3.3 (730)	1.52	1.26	0.84
3H-3	303.2 (68,170)	595.0 (86,290)	11.1 (2,500)	1.44	1.20	0.80
3H-4	295.6 (66,450)	579.9 (84,110)	5.7 (1,270)	1.40	1.17	0.78

As seen from the data in Table 3, changing the location of the confining reinforcement transversely relative to the lap splice does have a moderate affect on the measured tensile strength of the lap splice. Relative to Specimen Set 3H-2, the average maximum failure load decreased 5.1% for Specimen Set 3H-3 and 7.5% for Specimen Set 3H-4. The resulting strength of Specimen Set 3H-4 was only slightly higher than the unconfined specimens of Set 3H-1. Therefore, the relative location of the confining reinforcement transverse to the lap splice, as well as its location along the length of the splice as previously documented (Greenwald, et al. 2005), will be an important detailing consideration in the event lap splice confinement mechanisms for formally incorporated into masonry design standards.

ANALYSIS

In developing the design equation for masonry lap splices (CMR 1999), a linear regression analysis was performed to generate an expression for the lap splice strength as a function of the variables known to influence the splice strength and performance; including lap length, diameter of spliced reinforcement, strength of masonry, and structural cover distance. Equation 1, in turn, was then fitted to this regression equation for design application. This same analysis was repeated introducing the test specimens of this investigation without transverse confining reinforcement. The resulting product is shown in Equation 2.

$$T_p = -80.7 + 0.06 l_s + 0.17 d_b^{\,2} + 17.7\sqrt{f_{mt}} + 0.59K \tag{2}$$

$$\left[T_p = -18134 + 312 l_s + 24969 d_b^{\,2} + 330\sqrt{f_{mt}} + 3391K \quad \text{inch} - \text{pounds} \right]$$

Where:
T_p = predicted tensile strength of the lap splice, kN (lb)
l_s = lap splice length, mm (in.)
d_b = diameter of spliced reinforcement, mm (in.)
f_{mt} = tested compressive strength of masonry, MPa (psi)
K = masonry cover, mm (in.)

Equation 2, which does not account for the effects of transverse confinement reinforcement, predicts lap splice strength of unconfined specimens with reasonable accuracy (r^2 = 0.91), but under predicts lap splice strength when confinement is present. To account for the effects of the confining reinforcement, Equation 2 is rewritten as:

$$T_p = -80.7 + 0.06 l_s + 0.17 d_b^{\,2} + 17.7\sqrt{f_{mt}} + 0.59K + A_{st} f_{yt} \tag{3}$$

$$\left[T_p = -18134 + 312 l_s + 24969 d_b^{\,2} + 330\sqrt{f_{mt}} + 3391K + A_{st} f_{yt} \quad \text{inch} - \text{pounds} \right]$$

Where:
A_{st} = area of transverse confining reinforcement, mm^2 (in.^{2})
f_{yt} = yield strength of transverse confining reinforcement, MPa (psi)

Equation 3 adds the product of the area of confining steel (A_{st}) and the nominal yield strength of the confining reinforcement (f_{yt}) to the regression equation. For an unconfined lap splice, A_{st} is

zero, and the additional argument drops out. As can be seen from the testing results in Table 3 and Figure 10, however, the contribution of the confining reinforcement is not directly proportional to the area of transverse confining reinforcement and as a result significantly over-predicts the strength of the splice, particularly for larger areas of confining reinforcement. As such, there is a limit on the contribution the confining reinforcement has on the measured strength of a lap splice.

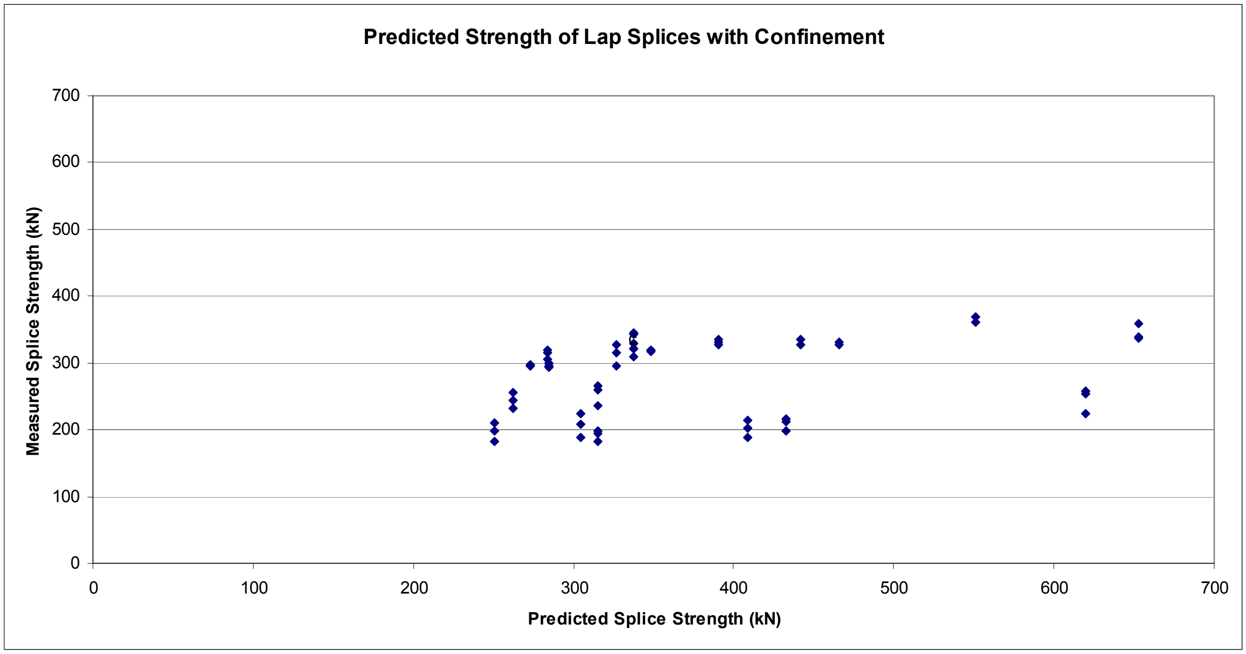

Figure 10. Predicted Strength of Confined Lap Splices Using Equation 3

To quantify the contribution the confining reinforcement has on the strength of the lap splice, the ratio of the predicted lap splice strength given by Equation 3 versus the measured lap splice strength was plotted against the area of the confining reinforcement provided as shown in Figure 11. As can be seen graphically, as well as derived mathematically from the test results, when the area of the transverse confining reinforcement exceeds 225 mm^2 (0.35 in.^{2}), Equation 3 over predicts the lap splice capacity. As such, based upon the results available, a design limit on the confining reinforcement of 225 mm^2 (0.35 in.^{2}) would seem prudent. A physically larger area of confining reinforcement could be provided, but in terms of application to Equation 3, the contribution of the transverse reinforcement to increasing lap splice capacity should be appropriately limited.

A similar analysis was conducted to determine a lower bound on the minimum lap length permitted for use with Equation 3. Comparing the predicted versus measured lap splice strength for each specimen set, as shown briefly in Table 4, Equation 3 is conservative when the lap length is 914 mm (36 in.) or greater, but over predicts the lap splice strength at lower lap lengths. Given that current U.S. codes stipulate a minimum required lap splice length to preclude possible pullout failures, a similar lower limit on the lap length when incorporating transverse confining reinforcement also seems appropriate.

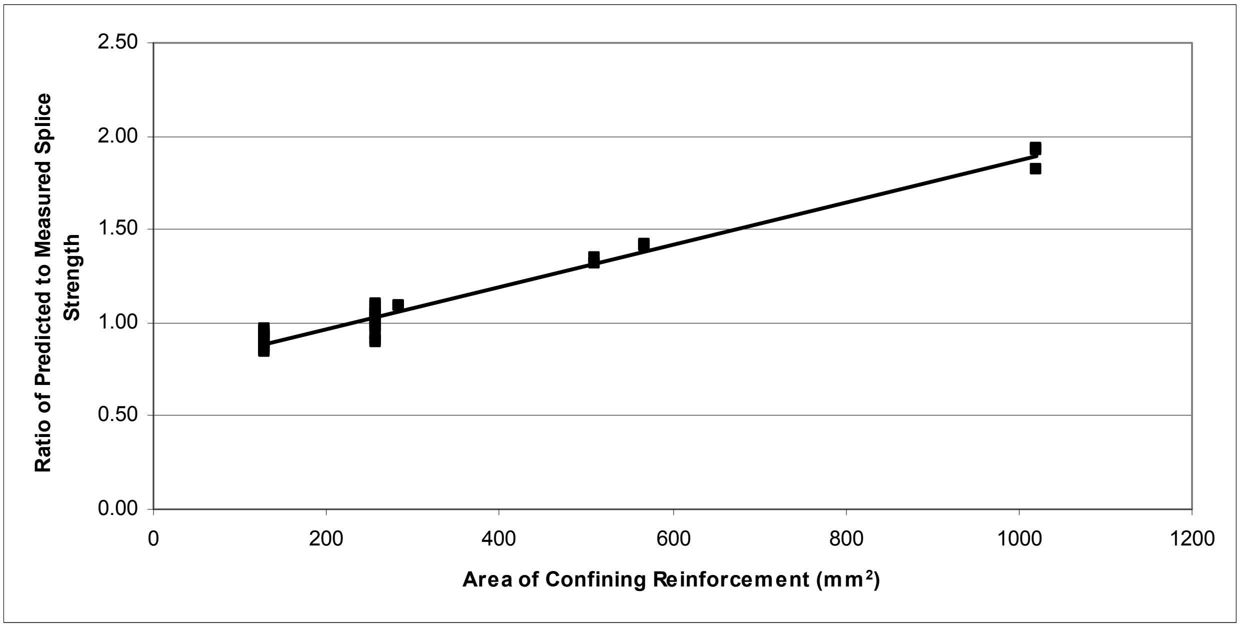

Figure 11. Limiting Contribution of Confining Reinforcement

Table 4. Specimen Sets 3B-2 and 3B-3 with ratio of capacity

Specimen	Lap Length	Ratio of Predicted Capacity to Actual Capacity
3B-2A	40	.92
3B-2B	40	.94
3B-2C	40	.92
3B-3A	32	1.13
3B-3B	32	1.07
3B-3C	32	1.02

CONCLUSIONS

While research is still ongoing on quantifying the influence transverse confining reinforcement has on the strength and performance of lap splices in masonry construction, these research does indicate that Equation 3 provides a reasonably good predictor of lap splice strength as shown in Figure 12, provided that the area of transverse reinforcement used in Equation 3 does not exceed 225 mm^2 (0.35 $in.^2$) and the length of lap is not taken less than 914 mm (36 in.). As previously reported (CMR 1999), Equation 3 can be solved for the lap splice length and the resulting expression used in design application, or a simpler equation fitted to the resulting regression equation.

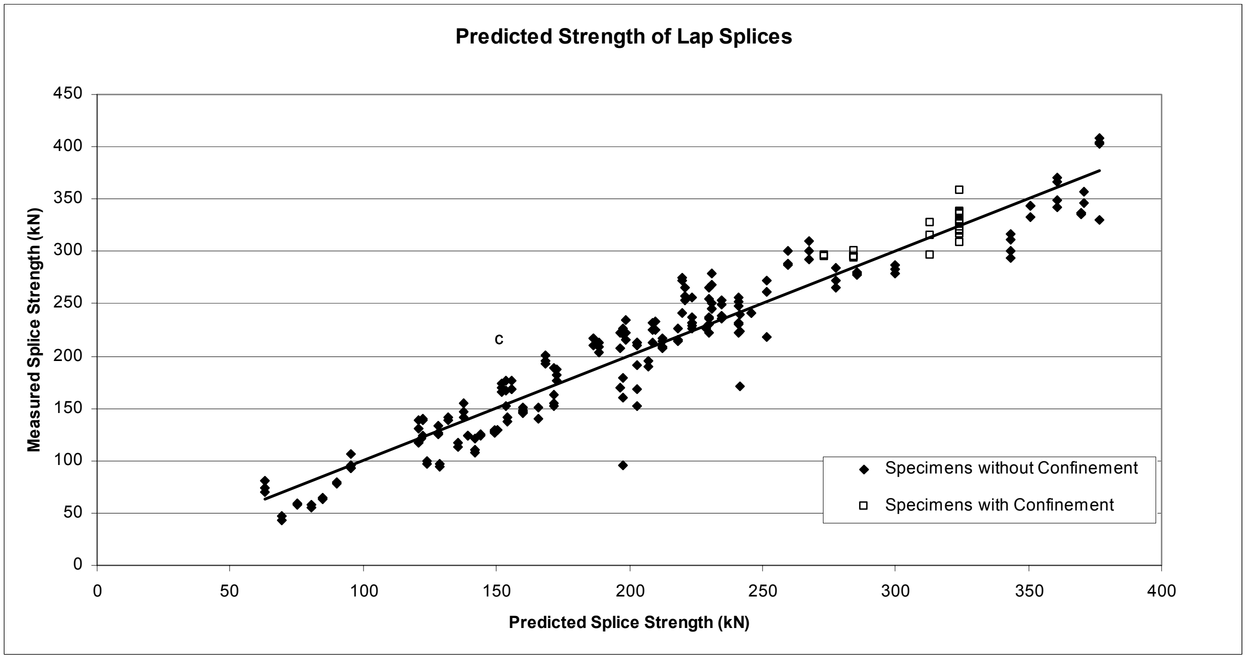

Figure 12. Regression of Confined and Unconfined Lap Splice Specimens

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