EFFECTS OF CONFINEMENT REINFORCEMENT ON BAR SPLICE PERFORMANCE

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ABSTRACT

This paper reports on research to investigate the effects of confinement provided by lateral reinforcement on lap splice performance. Fifteen concrete masonry panels were constructed using 200 mm (8-in.) units. Two sets of 25M (No. 8) lap spliced reinforcing bars were placed in each specimen. A lap length of 1200 mm (48 in.) was used for each splice tested in this study. 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 10M (No. 4) bar near each end of the splice, two 10M (No. 4) bars near each end of the splice, one 10M (No. 4) bar in each course of the panel, and two 10M (No. 4) bars in each course of the panel. 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 increased splice strength.

KEYWORDS: concrete masonry, reinforcement, splices, laps, confinement

INTRODUCTION

The purpose of this research study is to investigate the effects of confinement provided by lateral reinforcement on lap splice performance. This research investigation was intended to establish if potential improvements in splice performance from lateral 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.

Fifteen concrete masonry panels were constructed using 200 mm (8-in.) units; consisting of five sets of specimens, each with three identical panels. All specimens were fully grouted. Two sets of lap spliced 25M (No. 8) reinforcing bars were placed in the center of two of the cells of each panel. Each lap measured 1200 mm (48 in.) in length, equal to the total height of the test specimens. This combination of bar size and lap length was selected to be less than that required by current code provisions [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 [2]. The 1200 mm (48 in.) lap length was also selected due to the historical use of a $48d_b$ lap length, which for an 25M (No. 8) bar yields 1200 mm (48 in.). The $48d_b$ (for 413.7 MPa (Grade 60) 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 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 10M (No. 4) bar near each end of the splice, two 10M (No. 4) bars near each end of the splice, one 10M (No. 4) bar in each course of the panel, and two 10M (No. 4) bars in each course of the panel. Standard 90-degree 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 1. The width, height, and length of each panel measured nominally 200 x 1200 x 1000 mm (8 x 48 x 40 in.).

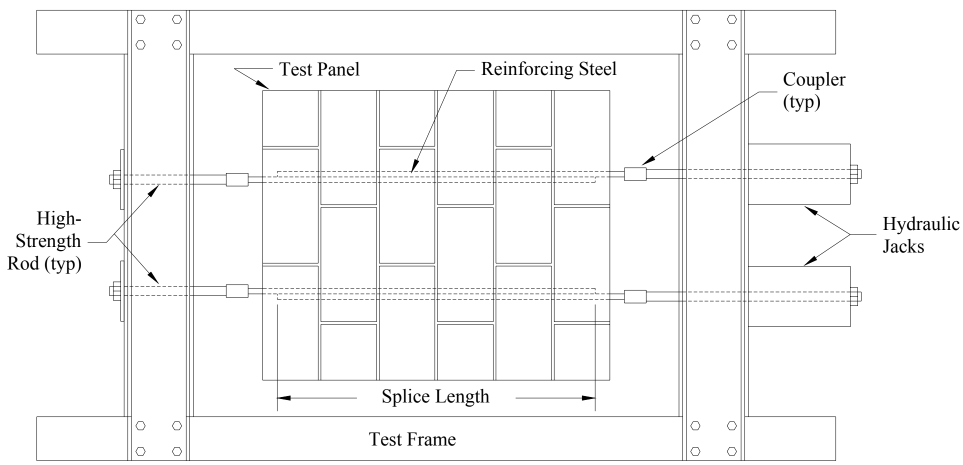

Figure 1 – Test Setup

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.

MATERIALS

All test specimens were constructed using concrete masonry units from the same lot. The specified dimensions of the units were 194 x 194 x 397 mm (7.625 x 7.625 x 15.625 in.), (8 x 8 x 16 in. (200 x 200 x 400 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*. Unit test results are summarized in Table 1.

Table 1 – Concrete Masonry Unit Properties

Unit Property	Measured Value
Net Area Compressive Strength	23.6 MPa (3420 psi)
Oven-dry Density	1871 kg/m^3 (116.8 pcf)
Absorption	157 kg/m^3 (9.8 pcf)
Dimensions • Width (W) • Height (H) • Length (L) • Face shell thickness (t_{fs}) • Web thickness (t_w)	 194 mm (7.63 in.) 191 mm (7.53 in.) 397 mm (15.62 in.) 31.5 mm (1.24 in.) 30.0 mm (1.18 in.)
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*. Although the measured face shell thickness of the concrete masonry units used in this research project were 0.25 mm (0.01 in.) 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.

Type S masonry cement mortar meeting the requirements of ASTM C 270, *Standard Specification for Mortar for Unit Masonry* was used to construct all panels and prisms. 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* with a measured value of 11.4 MPa (1660 psi).

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 20.7 MPa (3000 psi). Slump was determined in accordance with ASTM C 143, *Standard Test Method for Slump of Hydraulic Cement Concrete* with a measured value of 254 mm (10 in.). The compressive strength for the grout was determined in accordance with ASTM C 1019, *Standard Test Method for Sampling and Testing Grout* with an average measured value of 26.7 MPa (3810 psi).

At the same time as the panels were constructed, 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*. The average measured prism compressive strength was 22.4 MPa (3250 psi).

413.7 MPa (Grade 60) deformed reinforcing bars were used for the specimens of this study. The 25M (No. 8) 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 10M (No. 4) bars were used for the confinement reinforcement.

Tension tests were performed by an independent laboratory on the 25M (No. 8) splice bars in accordance with ASTM A 370, Test Methods and Definitions for Mechanical Testing of Steel Products and the results verified to meet the requirements of ASTM A 615, Specification for Deformed and Plain Billet-Steel Bars for Concrete Reinforcement. The average measured yield and ultimate strengths were 544 MPa (78900 psi) and 823 MPa (119400 psi), respectively.

TEST SPECIMENS

Fifteen panel specimens, comprised of five sets of three identical specimens, were constructed in accordance with the requirements of ACI 530.1/ASCE 6/TMS 602 and tested for this study. Table 2 provides a summary of the test specimens.

Table 2 – Test Matrix

Specimen Set (3 Panels Per Set)	Splice Length, mm (in.) For 25M (No. 8) Bar	Transverse Reinforcement Providing Confinement
A	1200 (48)	None
B	1200 (48)	One 10M (No. 4) top and bottom course
C	1200 (48)	Two 10M (No. 4) top and bottom course
D	1200 (48)	One 10M (No. 4) each course
E	1200 (48)	Two 10M (No. 4) each course

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. Each panel contained six courses, resulting in a height of 1200 mm (48 in.) to fully encompass the splice length. The 25M (No. 8) spliced bars were lapped using contact splices and tied together using small-gage wire. The 10M (No. 4) confining reinforcement was placed in the appropriate courses and tied in place using small-gage wire Figure 2 shows placement of the confinement reinforcement within the walls.

The grout was placed in the panels in a single lift. Grout in each core was consolidated using a 19 mm (¾-in.) 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.

The testing frame was constructed of four structural steel members bolted together to form a rectangular perimeter around the test panel (see Figures 1 and 3). 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.

Figure 2 – Placement of Confinement Reinforcement

Figure 3 – Test Setup

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-kip) 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. 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 76 mm (3 in.) 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.

TEST RESULTS

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 4. While major splitting did not occur over the adjacent spliced bars, Figure 5 shows tension cracking in the end of the panel indicating the onset of longitudinal splitting (taken after removal from the testing frame). The average splice capacity for Specimen Set A was 246 kN (55400 lbs).

Figure 4 – Typical Cracking in Specimen Set A

Figure 5 – Onset of Splitting Cracks in Specimen Set A

Figure 6 – Typical Cracking in Specimen Set B

Specimen Set B: The panels of Specimen Set B contained one 10M (No. 4) 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 6, these cracks were predominately concentrated in the middle of the panel. The average splice capacity for Specimen Set B was 314 kN (70500 lbs).

Specimen Set C: The panels of Specimen Set C contained two 10M (No. 4) bars in the top and bottom courses of the panel (two bars near 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 7. 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 10M (No. 4) bars were located. At the ends of the panel, adjacent to the location of the 10M (No. 4) transverse reinforcing bars, cracking occurred in the masonry units perpendicular to the splice. The average splice capacity for Specimen Set C was 336 kN (75500 lbs).

Figure 7 – Typical Cracking in Specimen Set C

Figure 8 – Typical Cracking in Specimen Set D

Specimen Set D: The panels of Specimen Set D contained one 10M (No. 4) 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 8. 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 Specimen Sets A through C. The average splice capacity for Specimen Set D was 331 kN (74500 lbs).

Specimen Set E: The panels of Specimen Set E contained two 10M (No. 4) 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 parallel 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 9. 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 369 kN (82900 lbs).

Figure 9 – Typical Cracking in Specimen Set E

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, kN (lbs)	Failure Stress in Splice Bars, MPa (ksi)	Ratio of Failure Stress to Measured Yield Stress in Spliced Reinforcement	Ratio of Failure Stress to Nominal Yield Stress in Spliced Reinforcement
A	None	246 (55400)	484 (70.2)	0.89	1.17
B	One 10M (No. 4) top and bottom course	314 (70500)	615 (89.2)	1.13	1.49
C	Two 10Ms (No. 4s) top and bottom course	336 (75500)	659 (95.6)	1.21	1.59
D	One 10M (No. 4) each course	331 (74500)	650 (94.3)	1.20	1.57
E	Two 10Ms (No. 4s) each course	369 (82900)	724 (104.9)	1.33	1.75

OBSERVATIONS AND RECOMMENDATIONS

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. 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 10M (No. 4) bars, spaced 1000 mm (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 of similarly configured splices without any transverse reinforcement.
3. Four 10M (No. 4) bars, spaced 1000 mm (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 of similarly configured splices without any transverse reinforcement. A similar increase in capacity was observed for splices with six 10M bars spaced evenly over the length of the splice, though reduced cracking was evident with the distributed transverse reinforcement.
4. Twelve 10M (No. 4) 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 of similarly configured splices without any transverse reinforcement.
5. Based on the relative increased benefit of concentrating the transverse reinforcement near the ends of the splices, the stress transfer between lap splices appears to be nonlinear, with the highest stresses occurring near the ends of the splices. Conversely, uniformly distributing the transverse reinforcement over the length of the splice resulted in the least amount of masonry cracking and distress.
6. 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 a smaller number of larger bars to provide the transverse reinforcement (e.g., one 10M (No. 5) 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 or confinement plates within the mortar joints) on lap splices.

REFERENCES

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