8TH CANADIAN MASONRY SYMPOSIUM

Jasper, Alberta
May 31 - June 3, 1998

TENSION LAP SPLICES IN REINFORCED CONCRETE MASONRY

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ABSTRACT

Experimental tests were conducted on 25 specimens consisting of 203-mm (8-inch) concrete block walls incorporating lap splices of No. 5 and No. 7 reinforcing bars. The lapped bars in each of the specimens were loaded in tension until failure occurred. Test results showed that current code specifications for lap splices overestimated the required lap for the smaller size bars and underestimated the required lap for the larger bars. The experimental results of this study were combined with the results from other research efforts investigating the performance of lap splices in concrete masonry. It was found that the reinforcement diameter, clear cover, and masonry compressive strength had a significant effect on the required length of lap for splices. A new design equation is proposed which more accurately represents the observed performance of tension lap splices in reinforced concrete masonry.

INTRODUCTION

In recent years, a number of studies have investigated the required length of lap for reinforcement splices in masonry. However, many conclusions drawn from this research are inconsistent with current design specifications and, in some cases, with each other. The main objectives of this study were to obtain a better understanding of splice performance and to make recommendations for the safe design of lap splices in reinforced concrete masonry.

In this project, tests were performed on wall panels, constructed of nominal 203-mm (8-inch) concrete masonry blocks, which were reinforced with lap spliced No. 5 and No. 7 Grade 60 reinforcing bars. Various lap lengths were tested, with each reinforcing splice centered within the masonry cell and positioned at mid-height of the test panels. Testing consisted of subjecting the lapped reinforcement in each specimen to monotonic direct tension. Loading was continued until some form of failure within the specimen occurred.

Based on the observed performance of the test specimens, parameters affecting the performance of lap splices were identified and conclusions stemming from these observations were drawn. The experimental results of this study were then combined with results from other research efforts investigating the performance of lap splices in concrete masonry and analyzed. Based on these analyses, recommendations for the required length of lap for spliced reinforcement in concrete masonry are made.

CURRENT CODE PROVISIONS

1997 Uniform Building Code

Section 2108.2.2.7 of the 1997 Uniform Building Code (UBC, 1997) requires a minimum lap splice length, l_d, based on strength design as follows:

$$l_d = \frac{1.8 d_b^2 f_y}{\phi K \sqrt{f'_m}} \leq 52 d_b \geq 305\ mm \qquad (1)$$

where: d_b = bar diameter, mm;
f_y = reinforcement yield strength, MPa;
ϕ = 0.80;
K = smaller of reinforcement clear cover or clear spacing in mm $\leq 3d_b$; and
f'_m = compressive strength of masonry assemblage, MPa.

Except for large values of f'_m or low grades of reinforcement, the $52d_b$ limitation controls for most applications of Equation 1.

American Concrete Institute 318-95

Section 12.2.3 of the 1995 American Concrete Institute Building Code presents the following equation for the basic development length for No. 11 bars and smaller (ACI, 1995):

$$l_d = \frac{15 f_y d_b \alpha \beta \gamma \lambda}{16 \sqrt{f'_c} \dfrac{c + K_{tr}}{d_b}} \geq 305\ mm \qquad (2)$$

where: α = reinforcement location factor;
β = reinforcement coating factor;
γ = reinforcement size factor;
λ = lightweight concrete factor;
f'_c = ultimate compressive strength of concrete, MPa;
c = cover or spacing between reinforcement, mm; and
K_{tr} = transverse reinforcement factor defined in ACI 12.2.4.

From Section 12.15.1 of the ACI, the minimum lap length is 1.3 l_d for splices where more than one-half of the total reinforcement is spliced at a splicing location. The ratio $(c + K_{tr})/d_b$ is specified to be less than or equal to 2.5 "...to safeguard against pullout type failures." This provision reflects a limiting effectiveness of the combination of cover and transverse steel.

Masonry Limit States Design (Proposed)

Recent research (Hammons, et al, 1994) has led to a proposed masonry limit states design equation for tension splices:

$$\phi l_d = \frac{0.0045 d_b^2 f_{ye}}{(t-d_b)} \geq 12 \textit{ inches} \tag{3}$$

where: f_{ye} = expected yield strength of reinforcement, psi;
d_b = bar diameter, inches;
t = thickness of masonry, inches; and
ϕ = 0.80.

Equation 3 was adapted from a splice length relationship proposed by Soric and Tulin (1987) by assuming a limiting value of 400 psi (2.76 MPa) for the tensile strength of the grout and an average value of the empirical constant C of 1.75, which was proposed to account for nonuniform bond distribution.

In contrast to other design equations, Equation 3 is inversely proportional to the thickness of the masonry. As $(t - d_b)$ equals twice the clear cover for reinforcement centered in the cell, this term may partially account for the adverse effect of diminishing clear cover with smaller wythe size. An additional difference in the equation is the implied correlation with the grout strength as opposed to the compressive strength of the masonry assemblage.

Masonry Standards Joint Committee

The length of lap given by the Masonry Standards Joint Committee (MSJC, 1995), based on working stress design, is as follows:

$$l_d = 0.002 d_b F_s \geq 12 \textit{ inches} \tag{4}$$

where: F_s = allowable steel stress.

Equation 4 is largely inconsistent with current thinking that cover (or spacing of reinforcement) and masonry strength play a significant role in the performance of lap splices.

However, a benefit of Equation 4 is its inherent simplicity. By using a constant allowable steel stress, the resulting length of lap for a given bar diameter remains constant and thus translates into simplified design and field inspection requirements.

Canadian Masonry Code, CSA S304

The Canadian Standards Association Standard 304 (CSA, 1984) design provision for lap spliced reinforced masonry is:

$$l_d = \frac{d_b F_s}{4\mu} \tag{5}$$

The CSA S304 equation is based on a working stress design format. The allowable reinforcement stress is limited to 166 MPa (24,000 psi) for Grade 60 reinforcement and the average bond stress, μ, is limited to 1.10 MPa (160 psi). For these limits, the resulting splice length is approximately 37.5 d_b.

EXPERIMENTAL PROGRAM

The following is a summary of the experimental program of this study. Additional details are given by Thompson (1997).

Specimen Description

All test panels consisted of nominal 203-mm (8-inch) concrete masonry units stacked in a running bond pattern. Both full and half concrete blocks were used during the construction of the panels, all of which were obtained from a local block producer and selected from the same lot to facilitate uniformity among the blocks. The mortar used during construction consisted of commercially-available Type S sacked mortar. Face shell bedding was used throughout the construction of the panels, except on the ends of the panels where the units where placed in a full mortar bed. The grout used to fill the cells of the specimens was a high-slump grout and was obtained in a single load from a local ready-mix supplier.

Each of the twenty-five panels of this study had nominal dimensions of 1.0 m (40 inches) in length and 20 cm (8 inches) in width. The height of each panel depended on the length of the splice being tested, with each panel having a height sufficient to enclose the splice completely. The panels were constructed by experienced masons under continuous supervision.

For each specimen, the reinforcement was placed within the center of the inside two cells with the middle of the lap splice centered at the mid-height of the panel. The spliced reinforcement was oriented parallel to the face shell of each specimen such that each bar was centered with respect to the width of the panel. Each panel contained two sets of spliced reinforcement. To create a symmetric test specimen, the bottom-spliced reinforcement was placed on the outside of the center of the cell, and the top-spliced reinforcement was placed on the inside of the center of the splicing cell.

Test Matrix

Nine different specimen sets were constructed with varying splice lengths and reinforcement sizes. To ensure repeatability of the tests, three identical panels were constructed using the same configuration, except panel sets 8 and 9 which had only two identical panels. Of the twenty-five specimens, fifteen contained only vertical spliced reinforcement. The remaining ten specimens contained similar vertical reinforcement and also introduced additional steel in the form of bed joint reinforcement or spirals. The bed joint reinforcement used was conventional 9-gage galvanized ladder-type reinforcement placed within each course during construction. This horizontal reinforcement satisfies Sections 2106.1.5.4 and 2106.1.12.4 of the 1997 Uniform Building Code. Spiral reinforcement consisted of undeformed 9-gage wire wrapped into the form of a spring with an approximate diameter of 127 mm (5 inches) and a pitch of 19 mm (0.75 inches). Table 1 summarizes details of the test specimens.

Table 1 Summary of Test Specimens

Panel Designation	Bar Size (mm)	Slice Length (cm)	Transverse Reinforcement
1-A, 1-B, 1-C	No. 7 (22)	$60d_b$ (133)	None
2-A, 2-B, 2-C	No. 7 (22)	$48d_b$ (107)	None
3-A, 3-B, 3-C	No. 7 (22)	$48d_b$ (107)	Horizontal Joint Steel
4-A, 4-B, 4-C	No. 7 (22)	$35d_b$ (78)	None
5-A, 5-B, 5-C	No. 5 (16)	$48d_b$ (76)	None
6-A, 6-B, 6-C	No. 5 (16)	$48d_b$ (76)	Horizontal Joint Steel
7-A, 7-B, 7-C	No. 5 (16)	$35d_b$ (56)	None
8-A, 8-B	No. 7 (22)	$20d_b$ (45)	Spiral Steel
9-A, 9-B	No. 5 (16)	$20d_b$ (32)	Spiral Steel

Material Properties

The reinforcing steel used in the construction of the test specimens was conventional Grade 60 rolled reinforcement with an upset thread milled onto one end. This upset thread allowed the reinforcement to be connected to the loading frame using high-strength couplers without compromising the diameter of the bar. The average measured yield and ultimate strengths for the No. 5 bars were 510 MPa (74 ksi) and 711 MPa (103 ksi), respectively, and the similar strengths for the No. 7 bars were 489 MPa (71 ksi) and 688 MPa (100 ksi), respectively.

Following established ASTM and UBC standards, samples of the grout, mortar, and block used to construct the test specimens were collected and tested. The resulting net average ultimate compressive strengths of the grout, mortar, and block were 18.4 MPa (2680 psi), 12.0 MPa (1730 psi), and 27.3 MPa (3960 psi), respectively. In addition, three grouted prisms were fabricated at the time of construction. The average compressive strength of the prisms was 18.0 MPa (2610 psi).

Test Setup

The lap splices of this study were tested in a pull-pull testing configuration, as shown in Figure 1. Connecting each test specimen to the loading frame was accomplished as follows. The bottom reinforcement of each panel was anchored to the lower cross member of the loading frame via high-strength rods connected to threaded couplers. The top reinforcement was connected in the same manner to hydraulically-driven jacks that supplied the load to the specimen. The jacks were connected in parallel such that each splice was subjected to an approximately equal load. Once securely positioned in the loading frame, loading of the specimen was initiated. As load to each specimen increased, incremental cracking stages and their corresponding loads were recorded along with the final failure load. Loads were obtained from a pressure gauge connected to the hydraulic pump and calibrated to the load supplied by the jacks.

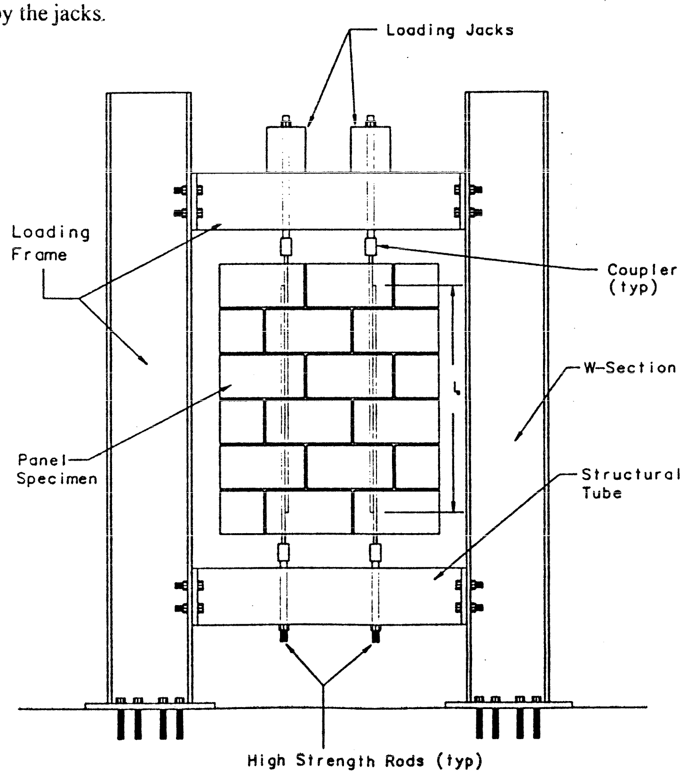

Figure 1 Test Setup

EXPERIMENTAL RESULTS

Table 2 summarizes the results of the testing program. With the exception of Panels 5-A and 6-C, which failed by reinforcement fracture, and Panels 6-A, 9-A, and 9-B, which failed by reinforcement pullout, the remaining panels all failed due to longitudinal splitting of the masonry.

Table 2 Summary of Test Results

Panel No.	Splice Length	Mode of Failure	Load in Bar at Failure (Stress)	% of Yield Strength	% of Ultimate Strength
1A	#7 @ 60d_b (133 cm)	longitudinal split	213.5 kN	113	80
1B			210.8 kN	111	79
1C			213.5 kN	113	80
Average			212.6 kN (550 MPa)	112	80
2A	#7 @ 48d_b (107 cm)	longitudinal split	208.2 kN	110	78
2B			213.5 kN	113	80
2C			202.8 kN	107	76
Average			208.2 kN (538 MPa)	110	78
3A	#7 @ 48d_b (107 cm)	longitudinal split	213.5 kN	113	80
3B			216.2 kN	114	81
3C			213.5 kN	113	80
Average			214.4 kN (554 MPa)	113	80
4A	#7 @ 35d_b (78 cm)	longitudinal split	186.8 kN	99	70
4B			181.5 kN	96	68
4C			176.1 kN	93	66
Average			181.5 kN (469 MPa)	96	68
5A	#5 @ 48d_b (76 cm)	longitudinal split/ bar fracture	144.1 kN	143	102
5B			141.5 kN	140	100
5C			138.8 kN	137	99
Average			141.5 kN (707 MPa)	140	101
6A	#5 @ 48d_b (76 cm)	longitudinal split/ bar pullout/ bar fracture	138.8 kN	137	99
6B			144.4 kN	143	103
6C			144.4 kN	143	103
Average			142.3 kN (712 MPa)	141	101
7A	#5 @ 35d_b (56 cm)	longitudinal split	138.8 kN	137	99
7B			130.8 kN	13	93
7C			117.4 kN	116	83
Average			129.0 kN (645 MPa)	128	92
8A	#7 @ 20d_b (45 cm)	longitudinal split	106.8 kN	56	40
8B			117.4 kN	62	44
Average			112.1 kN (290 MPa)	59	42
9A	#5 @ 20d_b (32 cm)	bar pullout	80.1 kN	79	57
9B			66.7 kN	66	47
Average			73.4 kN (367 MPa)	73	52

Effect of Lap Length

As anticipated, increasing the length of lap improved the performance of the splice. However, there appears to be a point of diminishing benefit as lap lengths are increased, particularly with the larger size bars. Take for example panel sets 1, 2, and 4 of this study. Increasing the length of lap by approximately 29 cm (panel set 4 and 2: No. 7 bars with lap lengths of 78 and 107 cm, respectively), resulted in an average increase of 26.7 kN in the capacity of the splice. By increasing the length of lap by a further 27 cm (panel set 2 and 1: No. 7 bars with lap length of 107 and 133 cm, respectively), the average increase in the capacity of the splice was only 4.4 kN. Thus, with larger-sized bars, it becomes increasingly difficult to achieve a ductile response, possibly even making the use of lap splicing impractical in some applications.

Effect of Size of Reinforcement

The test results show that it is more difficult to develop a splice with the larger bar size. In comparison to the results for the No. 5 splices, lower percentages of the yield and ultimate strength are achieved for the No. 7 splices with the same splice length expressed as a multiple of the bar diameter. Further, although an increase in the diameter of the reinforcement does increase the load capacity of a splice, this does not necessarily imply an increase in the stress of the reinforcement. Therefore, it may be difficult to achieve the required ductility for a structure containing large diameter bars. Effectively, the ductility of a structure may be decreased by increasing the diameter of the reinforcement.

An additional observation is an apparent nonlinear influence of the reinforcement diameter. This may be due in part to the diameter of the reinforcement changing as the wythe remains constant. That is, for a given masonry thickness, as the diameter of the reinforcement is increased, the clear cover is simultaneously decreased. The result is that as the diameter of the reinforcement is increased, longer lap lengths are required. Simultaneously, as the bar diameter is increased, the clear cover is decreased, requiring a further increase in the length of lap.

Effect of Spirals on Capacity of Splices

Although it was anticipated that the presence of spiral reinforcement would substantially improve the performance of splices in reinforced concrete masonry, the limited results contained in this study do not bear this out. It may be that the splice lengths selected for study were too small. Further research is recommended to investigate the use of spiral reinforcement before abandoning consideration of this form of reinforcement to improve splice performance.

Effect of Bed Joint Reinforcement on Capacity of Splices

Comparing panels sets 2 to 3 and 5 to 6, it can be seen that the addition of bed joint reinforcement had only a small effect on splice capacity. However, the bed joint reinforcement did have a pronounced effect on the post-failure condition of the specimens. Although most of these specimens failed by longitudinal splitting of the masonry, the specimens containing bed joint reinforcement remained essentially intact after failure. This form of post-failure performance can be a critical aspect of the behavior of a structure, especially where the possibility of overloading exists (e.g., in seismically-active regions).

A possible issue warranting further investigation is the shifting of the mode of failure from longitudinal splitting/bar fracture to pullout of the reinforcement in one of the specimens of panel set 6. This transition in the mode of failure may have been caused by an increase in confinement due to the presence of the transverse steel.

Splice Performance Compared to Codes

For panel set 7 (No. 5's with a 56-cm splice length) the average failure load was 128% of the measured yield strength of the steel. Based on a criterion of developing 1.25 times the measured yield strength as an indicator of a ductile failure, this splice length was adequate for the given material properties. When compared to the code equations presented earlier, this combination of material properties, diameter of reinforcement, and length of lap were less than that required by most of the design equations.

For panel set 1 (No. 7's with a 133-cm lap length) the average failure load was only 112% of the measured yield strength of the steel. Therefore, this length of lap was not sufficient to achieve a ductile failure even though it was a conservative splice length with respect to that required by current code provisions.

Based on the test results, current code requirements for lap splices appear to overestimate the required length of lap for smaller diameter bars and underestimate the required length of lap for larger diameter bars.

ANALYSIS OF TEST RESULTS

The experimental results of this study were combined with test results obtained from similar investigations conducted by the National Concrete Masonry Association (NCMA 1996 and 1997), the Construction Productivity Advancement Research program (Hammons, et al, 1994), and Soric and Tulin (1987 and 1989). The resulting data set consisted of 135 individual concrete masonry specimens which were reinforced with Grade 60 bars ranging in size from No. 4 to No. 11 and with lap lengths varying from 20 to 64 times the splice bar diameters. The masonry units varied from nominally 10 cm (4 inches) to 30 cm (12 inches) in thickness.

Multiple-linear regression analyses were performed to determine the contributions to splice strength of a number of parameters that have historically been shown to affect performance. Parameters investigated included compressive strength of masonry assemblage, grout and/or block, diameter of the spliced bar, length of lap and cover over the splice. The general form of the multiple linear regression equation used in this analysis was:

$$T_r = C_1 + C_2 l_s + C_3 d_b^n + C_4 \sqrt{f'_i} + C_5 c_i \tag{6}$$

where: T_r = predicted load capacity of splice, kN;
C_i = regression coefficient;
l_s = splice lap length, mm;
d_b = bar diameter, mm;
n = exponential integer, 1 or 2;
f'_i = compressive strength of masonry assemblage, grout or block, MPa; and
c_i = minimum clear cover or cover to center of reinforcement, mm.

After several trials, the following multiple-linear regression equation was found to give the best prediction of the observed splice capacities in the data set:

$$T_r = -102.77 + 0.0972 l_s + 0.127 d_b^2 + 17.13 \sqrt{f'_m} + 0.641 c_{cl} \tag{7}$$

where: f'_m = compressive strength of masonry assemblage; and
c_{cl} = minimum clear cover.

Equation 7 results in an r^2 correlation value of 0.94. A plot of the measured splice capacities versus the predicted splice capacities is shown in Figure 2.

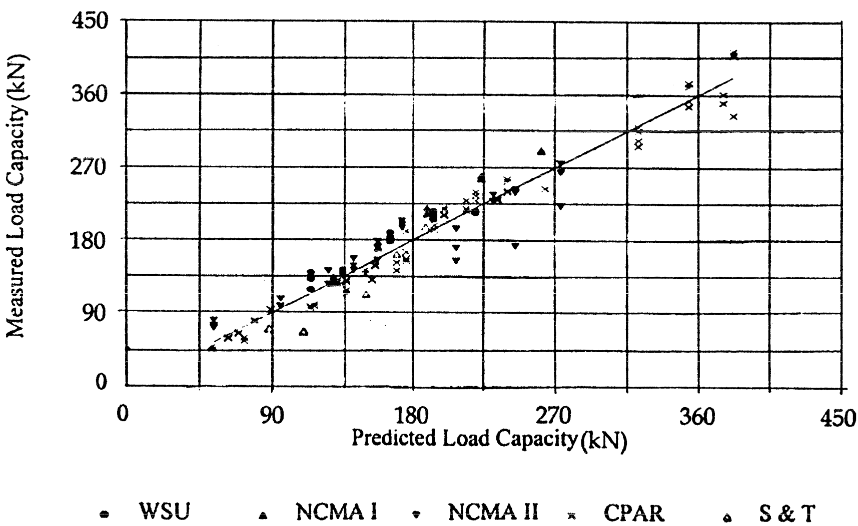

Figure 2 Comparison of measured and predicted splice capacities.

The splice capacity in Equation 7 can be replaced with a desired splice capacity, e.g., 1.25 times the bar yield capacity, and the equation solved for the required lap length as:

$$l_s = \frac{1.25 A_b f_y + 102.77 - 0.127 d_b^2 - 17.13\sqrt{f'_m} - 0.641 c_{cl}}{0.0972} \tag{8}$$

Since Equation 8 is impractical for design purposes, simplified equations were developed and compared to the lap lengths required from Equation 8. In developing these simplified equations, an attempt was also made to maintain a format similar to existing code equations. Based on this approach, the following design equation for lap splices in concrete masonry structures is proposed:

$$l_d = \frac{1.8 d_b^2 f_y \gamma}{\phi K \sqrt{f'_m}} \tag{9}$$

where: l_d = development length of reinforcement ≥ 305 mm;
d_b = bar diameter, mm;
f_y = reinforcement yield strength, MPa;
γ = reinforcement size factor;
= 1.0 for No. 3 through No. 6 reinforcing bars;
= 1.4 for No. 7 through No. 11 reinforcing bars;
ϕ = 0.80;
$K = c_{cl}/d_b$ with c_{cl} = minimum clear cover, mm; and
f'_m = compressive strength of masonry assemblage, MPa.

Since tested c_{cl}/d_b ratios were as high as 5.8, and no pullout failures were observed in specimens without transverse reinforcement, it is suggested that c_{cl}/d_b be taken as not greater than 5.0.

CONCLUSIONS

The following general conclusions are drawn from this lap splice study:

1. Increasing the reinforcing bar diameter increases the length of lap required to fully develop the reinforcement. Also, increasing the compressive strength of the masonry or clear cover decreases the length of lap required to develop the reinforcement.
2. Current code provisions do not adequately account for the required length of lap for spliced reinforcement in concrete masonry. In general, code provisions overestimate the required length of lap for smaller diameter bars and underestimate the required length of lap for larger diameter bars.
3. The presence of bed joint steel did not significantly increase the capacity of a splice loaded in tension, but it did have a pronounced effect on the post-failure condition of the test specimens.

ACKNOWLEDGMENTS

The first author would like to acknowledge the support received through the *Bob Fraser Graduate Fellowship* at Washington State University. Bill Cooper and Central Premix of Spokane, Washington are thanked for their donation of the masonry blocks, Harry Lancelot and Richmond Screw Anchor are thanked for their donation of the reinforcing steel, and Kevin Krebs and the Eastern Washington Masonry Promotion Group are thanked for their financial and technical support of this research project.

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