# A RATIONALIZED APPROACH FOR DETERMINING SPLICE LENGTH REQUIREMENTS IN REINFORCED MASONRY

**R.D. Kelln[1] and L.R. Feldman[2]**

[1] M.Sc. Student, University of Saskatchewan, Dept. of Civil and Geological Engineering, 57 Campus Drive, Saskatoon, SK, S7N 5A9; rdk277@mail.usask.ca

[2] Associate Professor, University of Saskatchewan, Dept. of Civil and Geological Engineering, 57 Campus Drive, Saskatoon, SK, S7N 5A9; lisa.feldman@usask.ca

## ABSTRACT
Splice and development length requirements significantly impact the safety, constructability, and economy of masonry walls. Due to a lack of research in this area, provisions for bond in CSA S304.1-04 are taken directly from CSA A23.3-04: Design of Concrete Structures. The provisions for reinforced concrete design do not account for all parameters influencing bond in reinforced masonry. In contrast, provisions in American code TMS 402-11/ACI 530-11/ASCE 5-11 are based on test results of double splice pullout specimens. While various configurations of pullout specimens have been used to evaluate splice length requirements in masonry, researchers studying bond in reinforced concrete construction have identified shortcomings in using this type of specimen. Furthermore, results of a recent masonry study established that wall splice specimens developed higher tensile resistances and higher strains in the spliced reinforcement as compared to the reinforcement in double splice pullout specimens. This study critically examines available literature related to bond research in reinforced masonry. Differences between current Canadian and American codes, research philosophies used in masonry and reinforced concrete research and respective code calibrations, and specimen types are discussed. The discussion highlights that further work is required to refine and calibrate Canadian masonry bond provisions. Based on these findings, it would seem most appropriate to use wall splice specimens designed to fail in bond prior to the yielding of reinforcement to achieve this goal.

**KEYWORDS**: splices, reinforcement, bond, code provisions, concrete block

## INTRODUCTION
Common features in masonry such as connections, openings, and construction details, as well as the overall length or height of members, frequently prevent the use of continuous reinforcement. In such situations, two reinforcing bars are overlapped or spliced; however, the length of these splices must be sufficiently long for tensile stresses to be transferred between lapped bars to prevent a bond failure [1,2,3].

Despite the significant impact of splice length requirements on the safety, economy, and constructability of masonry, relatively few research efforts have focused on such requirements for reinforced masonry. In contrast, bond has been studied extensively by reinforced concrete researchers, with tests for development and lap splice lengths dating as far back as 1876 [4]. As a result, the Canadian masonry provisions for development and lap splice lengths provided in

CSA S304.1-04 [5] are taken directly from the Canadian concrete design standard, CSA A23.3-04 [6]. However, factors exclusive to masonry construction are not accounted for in these provisions. Furthermore, provisions for splice lengths in the American masonry code TMS 402-11/ACI 530-11/ASCE 5-11 (herein referred to as TMS 402-11) [7] were developed from the results of an experimental program consisting exclusively of masonry pullout specimens. Based on their experimental results, the National Concrete Masonry Association (NCMA) developed provisions for splice lengths that were first included in the 2008 edition of the American masonry code [8]. However, overly conservative provisions, particularly for larger bar sizes, may have resulted, since the stress state in the cementitious materials surrounding the reinforcing bars is not adequately captured in pullout specimens. Significant differences in splice length provisions presented in CSA S304.1-04 [5] and TMS 402-11 [7] and in the philosophical approach to bond studies between reinforced masonry and reinforced concrete researchers motivate a further examination of bond in masonry construction.

## OBJECTIVES & SCOPE

The objective of this paper is to highlight the requirement for a further investigation of bond specific to masonry construction. Existing relevant literature is presented and the philosophies upon which the provisions for development length in both reinforced concrete and masonry are evaluated. Although mentioned anecdotally, discussion of non-contact lap splices and the effects of transverse reinforcement are limited within the scope of this paper.

## EVOLUTION OF BOND TEST SPECIMENS IN MASONRY RESEARCH

Most past work examining bond in reinforced masonry have included pullout-type specimens, where direct tension is applied to reinforcing bars extending beyond the specimen ends. Figure 1(a) shows the single bar masonry pullout specimens used in early studies, which are similar to pullout specimens previously used in concrete research. The cementitious materials surrounding the reinforcement in these specimens were subject to compression due to the support reaction created when tension was applied to the reinforcement. As such, these specimens fail to capture the true stress state as would be experienced by masonry walls subject to out-of-plane loads. Single splice pullout specimens shown in Figure 1(b) were tested by Hammons et al [9] and allowed for a neutral, rather than compressive, stress state in the cementitious materials. The eccentricity between the loads applied at either end of the specimen caused an in-plane bending moment simultaneously with the intended axial load. Figure 1(c) shows a double splice pullout specimen, developed by NCMA [1,8] and also examined by Ahmed & Feldman [10]. These specimens minimize the effects of eccentric loads, and hence in-plane moments, by including two reinforcement splices in the panel.

Pullout-type specimens offer the advantages of being relatively easy and inexpensive to construct and relatively simple to analyze. Additionally, double splice pullout specimens have shown good repeatability for contact splice specimens. Ahmed & Feldman [10] tested eight replicates of double pullout specimens with the same material and geometric properties to establish the statistical significance of the results. A coefficient of variation of 2.37 percent was observed for contact lap splices, which typically failed by bar pullout, while the non-contact lap splice specimens with the lapped bars located in adjacent cells typically failed by masonry splitting due to the relatively large in-plane moments that are created. Despite the aforementioned advantages, double pullout specimens still do not offer an accurate replication of the stress state experienced by masonry walls in flexure and cannot be used to assess the performance of non-contact lap splices. A study by Mjelde [11] at Washington State University compared double pullout splice specimens tested in direct tension and wall splice specimens tested by in-plane bending and observed no significant differences between the two specimen types. However, the capacity of lap splices in tension would be best ascertained using flexural specimens loaded by out-of-plane bending, replicating the lateral loads to which exterior buildings walls are often subjected.

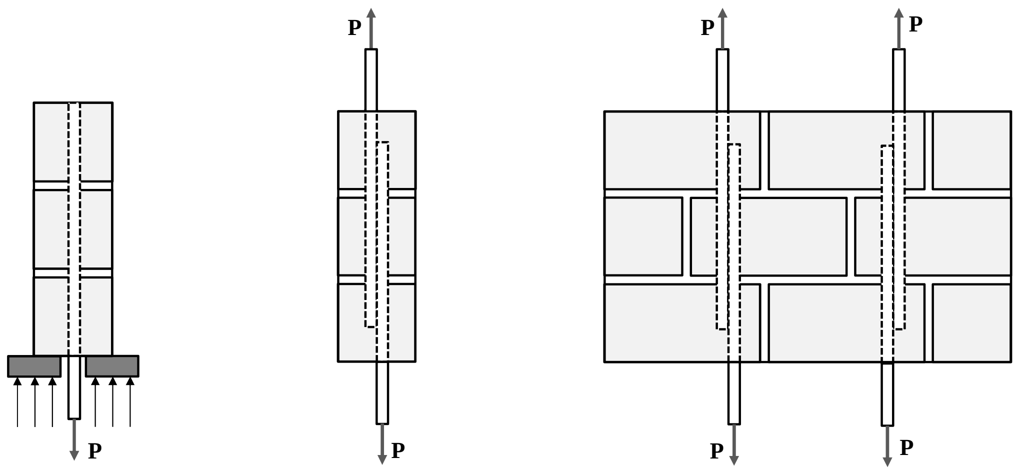


**Figure 1: Pullout-type specimens in reinforced masonry research: a) single bar (after [12,13]), b) single splice (after [9]), and c) double splice (after [1,10])**

Figure 2 shows the test setup for wall splice specimens designed by Ahmed and Feldman [10] and currently being used in further experimental investigations at the University of Saskatchewan. Ahmed & Feldman [10] reported a statistically significant increase in the capacity of the wall splice specimens as compared to the corresponding double pullout specimens, with increased capacities of 8.47 percent and 41.2 percent for contact and non-contact splices, respectively. The wall splice specimens reinforced with lap spliced bars in contact exhibited strain hardening of the bars while the corresponding reinforcement in the double pullout specimens did not. The higher splice capacity and higher reinforcement strains result from the improved ductility of the wall splice specimens that cannot be evaluated using double pullout specimens. Contact lap splices shorter than the 300 mm splices used in this study may lead to results that reflect yielding of the reinforcement in wall splice specimens, whereas a bond failure prior to the yielding of the reinforcement may occur in double splice pullout specimens with the same lap splice length. The wall splice specimens are also capable of assessing the capacity of non-contact splices.

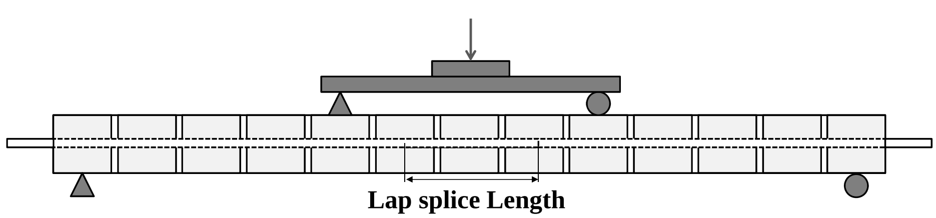


**Figure 2: Wall splice specimens (after [10]).**

## DEVELOPMENT OF TMS 402 MASONRY SPLICE PROVISIONS

NCMA conducted an extensive experimental program to examine splice length capacities in reinforced masonry as a function of several parameters in 1999 [1]. NCMA tested various double pullout splice specimens constructed in running bond with standard concrete masonry blocks (nominal dimensions 8x8x16 in, or approximately 200x200x400 mm). The double pullout specimens were 2.5 blocks wide, with specimen heights chosen for each selected lap splice length such that the specimen was sufficiently long to accommodate the lap splice length. Lap splice lengths were chosen as multiples of the bar diameter, $d_b$ and ranged from 36 $d_b$ to 113 $d_b$. This selection of splice lengths paralleled American code provisions at that time, which specified lap splice length requirements in inches as shown in Equation {1}:

$$l_d = 0.002 d_b F_s \tag{1}$$

where $d_b$ is the bar diameter in inches and $F_s$ is the allowable stress in the reinforcement in psi, which is specified as 24,000 psi (165 MPa) for Grade 60 (413 MPa) reinforcement [14]. Equation {1} results in a minimum splice length of $48d_b$ for Grade 60 (413 MPa) reinforcement.

Based on their experimental results from 177 individual double pullout splice test specimens, NCMA performed a regression analysis to predict the capacity of the lap splice, $T_r$ in lb, based on the lap splice length $l_s$ in inches, bar diameter $d_b$ in inches, masonry compressive strength $f'_m$ in psi, and clear cover $c_{cl}$ in inches, as presented in equation {2}:

$$T_r = -17624 + 305 l_s + 25204 d_b^2 + 322\sqrt{f'_m} + 3332 c_{cl} \tag{2}$$

The predicted capacity of the splice was taken as 1.25 times the force in the reinforcing bar at yield ($A_b f_y$) such that splices would develop 1.25 times the yield strength of the bar, and the equation was solved for the required splice length. The equation was then simplified such that it took on the same form as that used in the Uniform Building Code equation for splice lengths [1]. The expression recommended by the NCMA for splice length requirements, with splice lengths expressed in inches, therefore became:

$$l_d = \frac{0.13 d_b^2 f_y \gamma}{K\sqrt{f'_m}} \tag{3}$$

where $\gamma$ is a bar size factor (unitless); $f_y$ is the nominal yield strength of the reinforcing steel in psi; $K$ is the smallest of the minimum cover, clear spacing between adjacent reinforcement, and $9d_b$ (inches); and all other parameters are in Imperial units.

The recommended equation for splice length requirements, derived from the 1999 NCMA study [1], was first adopted in the 2008 edition of the Masonry Standards Joint Committee (MSJC) *Building Code Requirements for Masonry Structures* (TMS 402/ACI 530/ASCE 5) [15]. The new provisions required significantly longer lap splices, especially for larger bar sizes. Further NCMA research in 2009 [8] led to additional refinement of TMS 402-11 [7], which now incorporates a modification factor to reduce splice length requirements if sufficient transverse reinforcement is provided. These requirements are presented in Clauses 2.1.7.3 and 2.1.7.7

(*Allowable Stress Design of Masonry*) and Clause 3.3.3.3 and 3.3.3.4 (*Strength Design of Masonry*) of TMS 402-11 [7]. TMS 402-11 provisions require splice lengths for black bars of 1.0 times the development length ($l_d$) calculated.

**CSA S304.1-04 SPLICE PROVISIONS**

The development and splice length provisions in CSA S304.1-04 [5] are taken directly from CSA A23.3 [6] with slight modifications. Despite these modifications, many parameters unique to masonry construction including weak bed joints, the limiting of flexural cracks to these bed joints, and reduced lever arms in members subject to flexure, are not accounted for in the provisions used to design splice lengths in masonry. The aforementioned parameters likely have a negative effect on bond strength, thus requiring longer splices in masonry than in reinforced concrete.

The minimum development length requirement in Clause 12.4.2.3 of CSA S304.1-04 [5] is given in equation {4a}, where $k_1$, $k_2$, and $k_3$ are factors for bar location, epoxy coating, and bar size, respectively (dimensionless); $d_{cs}$ is the lesser of the distance between the reinforcing bar and the closest masonry surface and two-thirds the distance between bars being developed in mm; $K_{tr}$ is the transverse reinforcement index; $f_y$ is the nominal yield strength of the reinforcement in MPa; $f'_{gr}$ is in situ compressive strength of the grout or mortar in MPa; and $A_b$ is the cross-sectional area of the reinforcement in mm$^2$. Equation {4b}, which reproduces Clause 12.4.2.4 of CSA S304.1-04 [5], presents a simplified equation used for walls when the clear spacing between the lap spliced reinforcing bars exceeds two times the bar diameter, where $d_b$ is the bar diameter in mm and all other parameters as defined above.

$$l_d = 1.15\frac{k_1 k_2 k_3}{(d_{cs} + K_{tr})}\frac{f_y}{\sqrt{f'_{gr}}}A_b \qquad \{4a\}$$

$$l_d = 0.45 k_1 k_2 k_3 \frac{f_y}{\sqrt{f'_{gr}}}d_b \qquad \{4b\}$$

These equations are identical to those provided in CSA A23.3 [6] apart from substituting the compressive strength of the grout, $f'_{gr}$, for that of the concrete, $f'_c$, and the exclusion of an additional $k$ factor used in CSA A23.3 [6] to account for the concrete density.

As stated in Clause 12.5.4.2 of CSA S304.1-04 [5], splice length requirements depend on the calculated development length and the class of the splice. The required splice length of a Class A lap splice in CSA S304.1-04 [5] is equal to $l_d$, where a Class A lap splice is defined as one in which at least twice the area of reinforcement is provided and no more than 50 percent of the reinforcement is spliced at a given location. A lap splice is otherwise considered a Class B splice, where the splice length must be at least 1.3 $l_d$ as calculated from equation {4a} or {4b}.

**COMPARISON OF PROVISIONS PRESENTED IN CSA S304.1-04 AND TMS 402-11**

The form and parameters used in CSA S304.1-04 [5] and TMS 402-11 [7] as presented in the previous two sections are similar, yet the resulting numerical values for splice length requirements from each standard differ. Figure 3 highlights these differences in a quantifiable manner and shows the required splice lengths as calculated using provisions in the two codes, presented in equations {3}and {4b} in this paper, versus nominal bar diameter. Numerical values represented in Figure 3 were calculated without transverse reinforcement provided, and were made assuming equivalent material and geometric properties for both the reinforcement and the masonry assemblage.

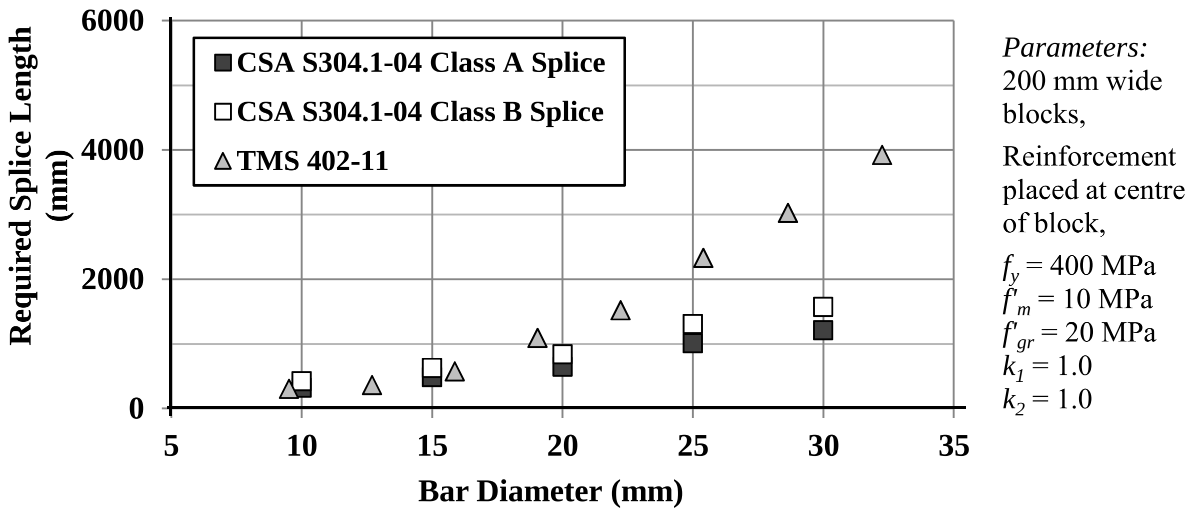


**Figure 3: Splice requirements calculated from CSA S304.1-04 and TMS 402-11 provisions**

The first noteworthy difference between the Canadian and American masonry code provisions is the property used to reflect the compressive strength of the masonry assembly. The compressive strength of the grout, $f_{gr}$, is used in CSA S304.1-04 [5]. Suter and Fenton [16] observed that minimum splice lengths calculated from splice provisions at the time using $f_{gr}$ agreed more closely with their experimental results than splice lengths calculated using $f'_m$. TMS 402-11 [7] instead uses the compressive strength of the masonry assembly, $f'_m$. NCMA [1] chose the masonry assembly compressive strength for inclusion in their prediction and design equations because they believed it better reflects the composite action between the masonry block and grout.

Recent experimental results underscore the effect of poor bond between the masonry blocks and grout on splice performance and the relevance of using $f'_m$ to represent the strength of the masonry assemblage, especially for the case when non-contact lap splices have been provided with the lap spliced bars located in adjacent cells. The poor bond between the masonry block and the grout prevented the development of diagonal compressive struts. These compressive struts are required to develop an internal moment resistance sufficient to carry the applied external in-plane moment created by the lapped reinforcing bars. Failure of these specimens by masonry splitting, prior to the yielding of the reinforcement, therefore resulted [10].

The second noteworthy difference between the Canadian and American masonry code provisions is their respective dimensional coefficients for the equations used to calculate lap splice length requirements. The TMS 402-11 [7] equation for development length, as presented in equation {3} and converted to metric units as stated in TMS 402-11 [7], simplifies as follows for walls if the minimum cover is at least 2 $d_b$, as required in CSA S304.1-04 [5].

$$l_d = \frac{0.75 d_b f_y \gamma}{\sqrt{f'_m}} \ (metric) \ [mm] \qquad \{5\}$$

When the dimensional coefficient in equation {5} of 0.75 is multiplied by the difference between the compressive strength of the grout and the masonry assemblage based on experimental data [3,10], the dimensional coefficient then ranges from 0.88 to 1.23 for grout compressive strengths ranging from 10 MPa to 40 MPa, respectively. The resulting coefficient is noticeably greater than the coefficient used in the CSA S304.1-04 [5] equation, as presented in equation {4b}, which is equal to 0.59 for Class B splices as are more typically required in walls.

The third significant difference between CSA S304.1-04 [5] and TMS 402-11 [7] provisions is their respective bar size factors accounting for the diameter of the reinforcement. The bar size factor $k_3$ in CSA S304.1-04 [5], set equal to 0.8 for No. 20 bars and smaller and 1.0 for bars larger than No. 20, is taken directly from those used in CSA A23.3-04 [6]. The long form of the CSA A23.3-04 [6] provisions for bond are nearly identical to those presented in ACI 318-08 Clause 12.2.3 (equation 12-1) [17], as both are based on the work of Orangun et al [18]. In their review of ACI 318 bond provisions, ACI Committee 408 notes the bar size reduction of 80 percent for No. 20 and smaller bars is potentially unconservative [19]. ACI Committee 318 justified the reduction for smaller bar sizes based on past code provisions and experimental results [17]. However, at the time when this factor was added to the provisions, only specimens with development or splice lengths less than 300 mm, shorter than permitted by Canadian and American concrete and masonry codes, were available. ACI Committee 408 [19] reports that the inclusion of the 0.8 bar size factor results in a greater probability of failure in bond than in flexure. Scollard and Bartlett conducted a Monte Carlo simulation of reinforced concrete beams to establish the resulting target reliability indices in flexure and bond based on provisions in the 1999 and 2002 editions of ACI 318 [20]. It was determined that the target reliability indices calculated in bond had more scatter than those obtained for flexure, and showed that bond failures were more likely to occur than flexural failures for beams reinforced with the smaller diameter bars that are subject to the 0.8 bar size factor by code. The simulation was repeated using a bar size factor of 0.85, and showed that the change in the resulting development lengths would increase the resulting target reliability sufficiently to ensure that a flexural failure would then govern.

In contrast, the bar size factor $\gamma$ in TMS 402-11 [7] is set equal to 1.0 for No. 3 (M#10) to No. 5 (M#16) bars, 1.3 for No. 6 (M#19) to No. 7 (M#22) bars, and 1.5 for No. 8 (M#25) to No. 11 (M#36) bars. Instead of reducing required splice lengths for smaller bars, the required splice lengths for larger bars are increased to prevent the longitudinal splitting failure modes observed by NCMA in their 1999 study for specimens with larger bar sizes [1]. The bar size factor, combined with the larger coefficient as presented in equation {5} above, result in the much larger splice requirements in TMS 402-11 [7].

The class factor for lap splices used in CSA S304.1-04 [5] provisions increases splice lengths by 30 percent for Class B splices. ACI Committee 408 [19] stated this factor increases the target reliability index in bond for the resulting designs such that a flexural failure would then govern. However, the purpose of including the Class B splice factor was not primarily based on probabilities of bond failure. Rather, Darwin [21] stated that the factor was incorporated into design provisions to encourage designers to stagger lap splices. A specific basis for the magnitude of the increase was not identified in the literature reviewed.

## COMPARISON OF PHILOSOPHIES USED BY MASONRY AND CONCRETE RESEARCHERS

A resolution of the differences in the specimen type used and the desired failure point of the reinforcement given the extensive bond research in reinforced concrete and limited bond research in reinforced masonry is needed. Resolving these differences will lead to a rationalization of the differences between the Canadian masonry code provisions, taken directly from the Canadian concrete code [6], and American masonry code provisions [7], derived from experimental pullout specimen results.

The development of flexural and shear cracks that occur in concrete or masonry elements subject to flexure cannot be replicated in pullout-type specimens since the cementitious material surrounding the longitudinal reinforcement is subject to either compressive or neutral stresses, depending on specific specimen type [19]. More recent reinforced concrete research has been based on the testing of beam end and splice specimens that better capture the stress state in the concrete surrounding the reinforcing bar. The ACI 10-2001 database therefore contains these test results exclusively, and has been used for the calibration of ACI 318 code provisions for development and lap splices since 1997 [19]. A standard specimen type for masonry bond research has not yet been established. Only a select number of studies [10,22,23] have included flexural masonry elements, so there is currently insufficient reliable data from which splice length provisions can be established.

In addition, the research used to establish and calibrate TMS 402-11 [7] is based on pullout specimens with capacities well in excess of yielding of the reinforcement. NCMA's 1999 and 2009 studies [1,8] tested double pullout splice specimens with lap splice lengths that were 48 $d_b$ at minimum, with specimens with splice lengths less than 36 $d_b$ excluded from their regression analysis. When tested, these specimens generally failed by yielding of the reinforcement rather than in bond.

The approach taken by concrete researchers contrasts the approach taken by NCMA in that concrete researchers have typically tested specimens designed to fail in bond prior to yielding of the reinforcement [18]. Since the purpose of their study was to evaluate bond strength rather than ductility, Orangun et al. [18] excluded specimens from their analysis in which the reinforcement reached its yield strength prior to failure. The capacity of a lap splice in a masonry or reinforced concrete element subject to flexure is limited by the yield strength of the reinforcement. Increases in splice capacity in flexural specimens will generally not be observed with increasing splice lengths that are beyond the length sufficient to prevent bond failure.

Furthermore, current bond provisions in TMS 402-11 [7] are based on a steel stress equal to 1.25 times the yield strength of the reinforcement. Orangun et al [18] noted this increase in stress proportionately increased required splice lengths by 25 percent. Such an increase does not directly consider the stress-strain behaviour of the steel reinforcement, as the strain hardening when loaded beyond the yield point results in a non-proportional increase of stress with strain.

Instead, Orangun et al [18] recommended a capacity reduction factor be applied to the calculated development length, as this factor would account for variances in material and geometric properties of concrete members. Further, a change in splice length provisions of ACI 318-08 [17] has been recommended by ACI Committee 408 [19], which would incorporate a strength reduction factor to ensure a bond failure is one fifth as probable as a flexural failure.

A strength reduction factor applied to a splice length calculated using the nominal yield strength of the reinforcement would be a more logical approach for bond provisions in reinforced masonry as it would be consistent with the limit states design methodology already used in CSA S304.1-04 [5]. A probability-based approach could be used to reliably establish this strength reduction factor once a database of reliable test results with a standardized and representative specimen type is established.

## RESOLUTION OF SPECIMEN TYPE AND FAILURE MODE FOR BOND RESEARCH

Hammons et al [9] identified four primary failure modes have been observed for investigations of lap splices in reinforced masonry:

1. Reinforcement pullout,
2. Reinforcement yielding,
3. Reinforcement rupture, and
4. Masonry longitudinal splitting.

NCMA [1,8] observed the latter three failure modes in their experimental work. Hammons et al. [9] identified reinforcement yielding as the preferred failure mode for design since it provides both the most efficient use of the reinforcement and sufficient ductility. Hammons et al. [9] note that splitting failures are most probable when limited cover, large bar sizes, or inadequate splice lengths are used. However, a flexural failure will occur in masonry elements provided that the lap splice is sufficiently long.

NCMA observed a handful of specimens that failed by rupture of the reinforcement with a stress ranging from 1.56 to 1.69 times its nominal yield strength in their 1999 study [1]. Achieving stresses well in excess of the yield strength of the reinforcement can only be expected to occur in double splice pullout tests. In elements such as concrete beams or masonry walls, a flexural failure is dependent on both the compressive strength and lack of tensile strength of the concrete or masonry. Specimen failure would therefore occur prior to rupture of the reinforcement.

NCMA stated in their subsequent 2009 [8] study that the new bond provisions first incorporated in TMS 402-08 [15] specifically consider the longitudinal splitting failure mode for lap splices and the potential for reinforcement pullout failure. Data from specimens that failed by longitudinal splitting was used exclusively for the calibration of these design provisions. However, the majority of these specimens achieved capacities exceeding the yield strength of the reinforcement, with the target splice capacity set equal to 1.25 times the yield strength of the reinforcement. Regardless of the failure mode, the splice capacity observed in the testing of these double splice pullout specimens is likely conservative if compared to the splice capacity that would be observed in comparable wall splice specimens, given the results of Ahmed & Feldman's work [10]. The choice of test specimen is the most probable reason for the increased splice length requirements presented first in TMS 402-08 [15] and in TMS 402-11 [7].

Failure in bond occurs without warning, making bond failure an undesirable limit state. The probability of a bond failure should therefore be less than the probability of a flexural failure [19]. However, for the purpose of designing an effective experimental investigation with the aim of rationalizing design provisions for the development and lap splice length of reinforcement, specimens must be designed to fail in bond as would be indicated by either bar pullout or longitudinal masonry splitting prior to the yielding of the reinforcement.

## CONCLUSIONS

This paper examined current masonry code provisions for bond in CSA S304.1-04 and TMS 402-11 as well as available literature focusing on bond research in both reinforced masonry and reinforced concrete construction. Types of test specimens used by masonry researchers to evaluate bond were examined. Differences in splice length requirements between the Canadian and American masonry codes were quantified. Research philosophies of reinforced concrete and masonry researchers were also compared.

The following conclusions are offered:

- Splice length provisions in CSA S304.1-04 do not account for parameters unique to masonry construction since these provisions are taken directly from CSA A23.3.
- The compressive strength of the masonry assembly, $f'_m$, should be used in bond provisions since it reflects the composite action of the masonry blocks and grout.
- The bar size factors in both CSA S304.1-04 and TMS 402-11 should be reconsidered. The CSA bar size factor may be unconservative. Results of reinforced concrete research show that the target reliability index in bond is lower than the target reliability index in flexure for small bar sizes in particular.
- A better understanding of lap splices in tension is needed. Flexural wall splice specimens are recommended as the specimen type for all future research since these specimens are able to capture the stress state in the cementitious materials surrounding the reinforcement.
- Bond tests to establish and calibrate code provisions for masonry should use specimens designed to fail in bond, indicated by bar pullout or longitudinal masonry splitting, prior to reinforcement yielding.

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