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Evaluation of Splice Length for Reinforcement in Masonry Walls

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ABSTRACT

Slender masonry walls are widely used in single-story commercial structures due to their efficiency and cost-effectiveness. However, increasing architectural demands for taller and thinner walls have amplified the vulnerability of these structures to buckling and second-order effects. Current design practices often rely on conservative approaches, such as increasing wall thickness, which raises material costs and compromises productivity. An alternative strategy—placing vertical reinforcement closer to the edge of the grouted cell—offers the potential to enhance out-of-plane performance but raises concerns about bond strength and durability. This paper presents the first phase of an experimental program aimed at evaluating the bond behaviour of vertical reinforcement placed at varying positions within masonry block cells. A total of ten pullout tests were conducted using a novel setup designed to replicate realistic conditions while eliminating compressive stresses on the surrounding masonry. Results demonstrated that current splice length provisions in CSA S304:24 are overly conservative, particularly for reinforcement located near the cell edge. Most tests resulted in steel yielding before bond failure, which indicates the need for reduced splice lengths in future specimens to ensure bond-dominated failure mechanisms. These findings show that more efficient design expressions that consider innovative reinforcement configurations without compromising the safety or structural integrity of walls are needed. By addressing key gaps in current standards and leveraging a scalable test setup, this work sets up for the next phase of research, which will expand the experimental database. These insights contribute to the development of more competitive and sustainable masonry construction practices.

KEYWORDS

bond, experimental test, lap splice, pullout test, reinforced masonry

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INTRODUCTION

Single-storey load-bearing slender masonry walls (SMW) are widely used in warehouses, gymnasiums, and retail stores. A masonry wall in this market typically ranges between 8 to 10 metres tall and is partially grouted (PG) – grout is only present in some cells. Masonry walls use vertical reinforcement to provide post-cracking tensile capacity for resisting out-of-plane (OOP) loads. With increasing architectural and structural demands, taller and thinner walls are becoming more common. As a wall's height-to-thickness ratio (*kh*/*t*) increases, it becomes more vulnerable to second-order (P-delta) effects, which can lead to buckling [1–3].

Large design moments imposed on SMW, amplified by second-order effects, result in a high demand on reinforcement in the wall's cross-section. However, North American design standards S304:24 [4] and TMS 402/602-22 [5] require walls to have a steel reinforcement ratio equal to or less than the balanced ratio to guarantee a ductile failure. Due to this requirement and concerns about constructibility, designers often opt to increase the wall's thickness to increase the moment arm between tensile and compressive forces instead of adding more reinforcement area. This is not an ideal solution since thicker and heavier blocks increase material costs and reduce productivity [6], reducing masonry competitiveness.

The current practice of placing vertical bars at the centre of the wall cross-section is also not ideal and leaves room for improvement. Several researchers [7]–[10] showed that placing vertical reinforcement away from the cross-section's centre increases OOP strength and stiffness. However, the reduction of clear cover for vertical reinforcing bars presents the risk of a potential bond-loss mechanism. None of the mentioned studies investigated the effects of the bond between reinforcement and grout or bar exposure to the elements.

Development length (l_d) is defined as the minimum embedment length of a bar within masonry to develop its yield capacity. The current Canadian masonry design standard CSA S304:24 [4] defines l_d as:

$$(1)\quad l_d = 1.15\frac{k_1k_2k_3}{(d_{cs}+K_{tr})}\frac{f_y}{\sqrt{f'_{gr}}}A_b$$

Where k_1, k_2, and k_3 are dimensionless factors for bar location, epoxy coating, and bar size, respectively, d_{cs} is the lesser of the distance between the reinforcing bar and the closest masonry surface and two-thirds of the distance between bars being developed (mm), K_{tr} is the transverse reinforcement index (mm); f_y is the nominal yield strength of the reinforcement (MPa), f'_{gr} is the in-situ compressive strength of the grout (MPa), and A_b is the cross-sectional area of the reinforcement (mm^2). The term $(d_{cs}+K_{tr})$ is limited to a maximum value of $2.5d_b$. Splice lengths required in S304 depend on l_d and the class of lap splice. A Class A splice has at least twice the required area of reinforcement is provided, and no more than 50% is spliced at a given location; otherwise, the splice is Class B, and its length must be at least $1.3l_d$.

Equation (1) was taken directly from the Canadian concrete design standard A23.3 [11] and adapted to masonry. A few changes were made, including the substitution of f'_{gr} instead of f'_c and the exclusion of the k factor that accounts for concrete density. However, due to the unique nature of masonry (e.g., non-homogeneous construction, cracks localized at joints), the adequacy of the current expression applied to masonry requires further investigation [12], [13]. Conversely, the American TMS 402/602 [5] provisions were developed from two NCMA studies [14], [15]. TMS 402/602 sets splice length requirements equal to l_d, calculated as:

$$(2)\quad l_d = \frac{1.57d_b^2f_y\gamma}{K\sqrt{f'_m}}$$

Where γ is a dimensionless bar size factor (1.0 for No. 5 (16 mm diameter) bars or smaller, 1.3 for No. 6 (19 mm diameter) through No. 7 (22 mm diameter) bars, and 1.5 for larger sizes of bars), and K is the minimum value of the masonry cover, the clear spacing between adjacent bars, and $9d_b$. Neither Equation (1) nor (2) were derived considering vertical reinforcement at the edge of the grouted cell and cannot safely provide adequate splice lengths to prevent a pullout failure before a flexural failure occurs in such arrangements.

This paper presents the first stage of an experimental program to assess the bond strength of steel reinforcement in masonry walls when bars are placed at different locations inside the block cell. A modified pullout test, partly based on pullout tests on reinforced concrete (RC) members, is proposed to assess the effect of having vertical reinforcing bars closer to the face shell of blocks as opposed to the typical use of centrally located bars. Ten pullout tests were performed with bars in different positions inside the block cell. These tests serve as the basis for the next stage of the program, where fifty more tests will be performed. Empirical equations derived from experimental data will be the basis for predictive and design expressions on development and splice lengths that account for the novel reinforcement configuration. Results and their implications, as well as next steps, are provided in the following sections.

EXPERIMENTAL PROGRAM

In masonry, there is no standard bond or splice length test. The majority of previous research investigating bond in reinforced masonry has been conducted using pullout-type specimens, which are relatively simple and cost-effective [16]. However, typical pullout tests misrepresent realistic conditions by inducing compressive stresses in the surrounding cementitious material, artificially enhancing the bond strength [12]. The NCMA [14, 15] tested eighty-one double pullout specimens (Figure 1) with splice lengths ranging from $36d_b$ to $113d_b$ to examine splice length capacities in reinforced masonry. These studies formed the basis of the development and lap splice length provisions in the American standard TMS 402/602-13 [17].

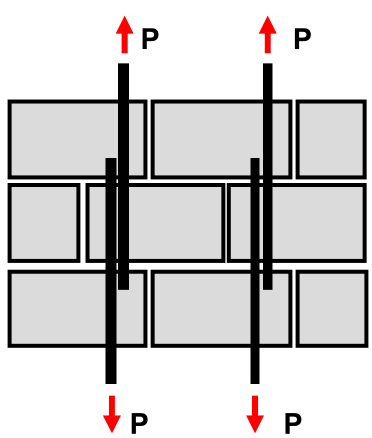

Figure 1: Double pullout specimen, adapted from [14,15].

Relatively recent RC research on bond has been based on the testing of beam end splice specimens that arguably better capture the stress state in the concrete surrounding the reinforcement [18–20]. Researchers typically test specimens with splice lengths shorter than the development length requirement to induce bond failure before reinforcement yield [21–23]. Test specimens that fail in bond allow for a direct determination of the relationship between splice length and the tensile capacity of the splice. Those two parameters are assumed to vary in a linear but non-proportional manner when the forces in the reinforcement are below yielding [16]. Yet, the fabrication of masonry beam end splice specimens introduces multiple complications, making them impractical.

A few studies [24–27] have used flexural masonry elements, arguing a more realistic stress state between the reinforcement and the surrounding cementitious materials. Despite this advantage, wall splice specimen construction and test setup are characterized by a higher degree of complexity and an indirect measurement of the tension in the lap splice. This study presents a new setup to evaluate splice length requirements in masonry walls that combines the principles of the NCMA double pullout specimens with a simpler setup and the efficiency of constructing multiple pullout bars simultaneously.

Setup and Instrumentation

A modified pullout test was employed by Pei et al. [28] to evaluate the bond strength and development length of steel bars coated with cementitious waterproofing materials in RC elements. Their setup, shown in Figure 2, combined the advantages of both beam-end and conventional pullout tests. The test was designed to simulate the bond failure mode of reinforcing bars in flexural members and to eliminate compressive stresses in the surrounding concrete.

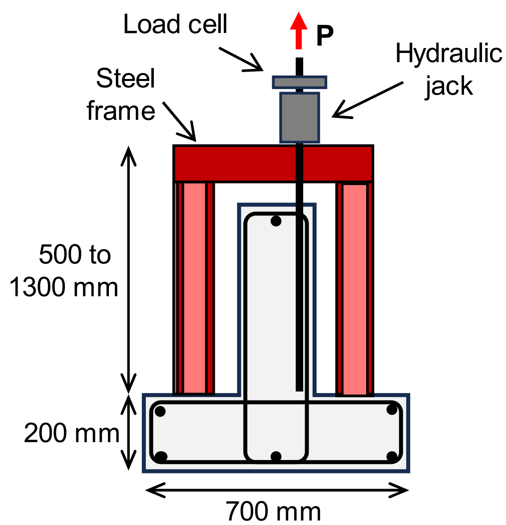

Figure 2: Modified pullout test to assess the development length of steel reinforcement with cementitious coatings in RC, adapted from [28]**.**

Partly inspired by [28], the test setup used in this study consisted of a reacting steel frame that bore the load exerted by bars being pulled and passed the compressive force to the concrete slab at the base. This configuration prevents compressive forces from being applied to the wall. A 30-ton (~294 kN) jack was mounted on top of the steel frame and couplers were used to allow the jack to grab each test bar to pull it out from the wall. A load cell was installed on top of the jack to measure pullout force. Two linear potentiometers (LP), each one mounted at each side of the wall, measured the slip and elongation to the loaded end of the reinforcing bar.

Before testing, the reacting steel frame was positioned above each bar using a 10-ton overhead crane. The frame rested on rubber pads located on top of the foundation concrete slab. These pads were used to distribute the load from the frame base to the concrete. The hydraulic jack and load cell were then mounted on top of the reacting steel frame. A shear screw cap lock was installed close to the top end of the test bar to serve as a mechanism for the jack to react against and pull the bar. Then, the sensors were installed to record load and displacements during the test. The test setup is shown in Figure 3.

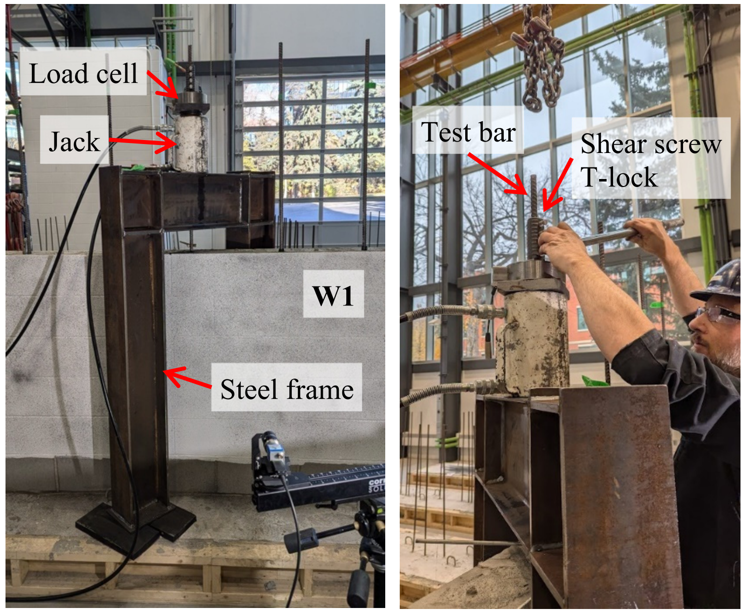

Figure 3: New pullout test setup for masonry walls.

Specimen Design

Researchers investigating development and lap splice lengths in RC use relatively short embedment lengths to induce bond failure prior to yielding of the reinforcement. Bond failure often occurs without sufficient warning, making it an undesirable limit state. The probability of bond failure should be less than that of flexural failure [16]. Therefore, most bond strength specimens must be designed to fail in bond prior to yielding of the reinforcement. This is the approach intended in this study to investigate the development length inside masonry walls.

A total of sixty tests will be conducted using the setup previously described. The parameters to be assessed include the position of vertical reinforcing bars, grout type, and the number of bars inside the grouted cell. For the first stage of the experimental program, a six-course masonry wall (W1) with 20 cm standard concrete masonry units (CMU) was built to accommodate ten 15M pullout bars with different splice lengths and positions inside the block cells. The bar's position was either centred inside the grouted cell, 13 mm away from the block as required by CSA A371 [29], or at the edge of the cell touching the inner face of the block. The specimen test matrix is presented in Table 1. Bars had splice lengths (l_s) between 0.1 and 0.6 of the theoretical development length (l_d) calculated with Equation (1) to induce pullout failure prior to yielding of the reinforcement. Each bar had a 75 mm ($5d_b$) plastic tube at the top end of the wall to debond it from the surrounding grout, preventing an early pullout failure in that region. A schematic representation of the different vertical bar positions and the arrangement of vertical bars in W1 are shown in Figure 4a and Figure 4b, respectively.

Table 1: Pullout test matrix for steel bars in specimen W1.

Pullout bar	Bar position	l_s	l_d	l_s/d_b	l_s/l_d	Bar ID
1-15M	Centred	150	531	10	0.3	7, 10
		300	531	20	0.6	1
	13 mm cc	300	1012	20	0.3	4
		450	1012	30	0.4	2, 9
		600	1012	40	0.6	3
	At edge	300	2656	20	0.1	6
		600	2656	40	0.2	5
		800	2656	53	0.3	8

cc: clear cover measured from the steel bar surface to the inner face of the block.

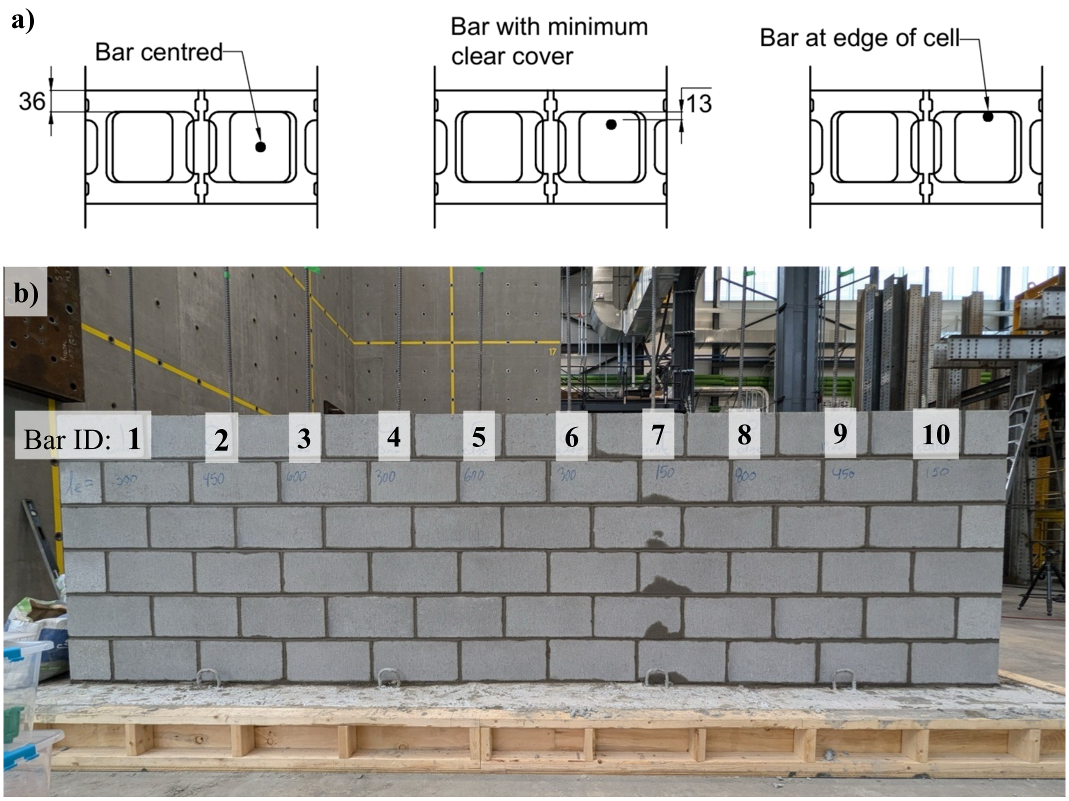

Figure 4: Arrangement of vertical bars in specimen W1, a) Schematics of the different bar positions, b) Lateral view of the masonry wall (dimensions in mm).

Material Properties

The wall was built by certified masons using Type S mortar and masonry coarse grout from premixed bags with specified compressive strengths of 12.5 and 20.6 MPa, respectively. The compressive strength of the mortar was determined by crushing six 50×50×50 mm mortar cubes under concentric axial loads per CSA A179 [30]. The compressive strength of grout was found by crushing four 100 × 200 mm cylindrical samples under concentric axial load per A179. S304:24 requires a minimum of five masonry prisms to determine the compressive strength of the masonry assemblage. Five grouted and five ungrouted prisms, three courses tall and one unit wide were built in a running bond pattern and tested under concentric compressive force. A summary of masonry material properties is presented in Table 2.

Vertical steel reinforcement used in the masonry wall comprised Grade 400 15M bars. Three bar coupons were tested in tension following A615-20 [31]. Bars had a yield strength of 464 MPa with a COV of 0.8% and an elastic modulus of 194 GPa with a COV of 2.6% based on a nominal bar area of 200 mm^2.

Figure 5: Compressive strength test of grouted and ungrouted three-course masonry prisms.

Table 2: Masonry materials properties.

Material	Compressive strength [MPa]	COV [%]
Mortar, type S	21.1	11.7
Coarse grout	24.9	4.7
Masonry, ungrouted	15.9	10.7
Masonry, grouted	20.4	9.0

Procedure

A hydraulic jack pulled each test bar to failure. The jack was loaded at a rate that made for a five to ten-minute test by adjusting the valves connected to the hydraulic pump. Bars that did not fail were unloaded shortly after the 15M steel bar yielded (~93 kN based on coupon tests). A few tests were taken beyond this value to evaluate the response of the test setup and to better visualize failure modes.

Results and Discussion

Pullout maximum force (P_{max}) and peak bar stresses are presented in Table 3. Almost all test bars, except for Bars 7 and 10, yielded before a pullout failure. Bars 7 and 10 had the smallest splice length (150 mm) and were centrally located in the block cell. The failure mechanisms observed varied partly depending on splice length. Bar 7 ($ls = 150$ mm) exhibited pullout failure, with minor observable cracks. For Bars 1, 4, and 10 ($ls = 300$ mm or shorter), major cracks at the top course suggest a combination of pullout and masonry breakout, likely influenced by shorter splice lengths and stress concentrations near the bar ends (Figure 6). In contrast, bars with longer splice lengths ($ls = 450$ mm or longer) showed no significant cracking or slippage, indicating a better anchorage performance.

Table 3: Pullout test results for steel bars embedded in a masonry wall.

Bar ID	l_s [mm]	P_{max} [kN]	Bar stress [MPa]
1	300	99.1	496
2	450	117	582
3	600	117	584
4	300	114	568
5	600	111	556
6	300	116	581
7	150	85.9	429
8	800	110	551
9	450	118	590
10	150	67.3	337

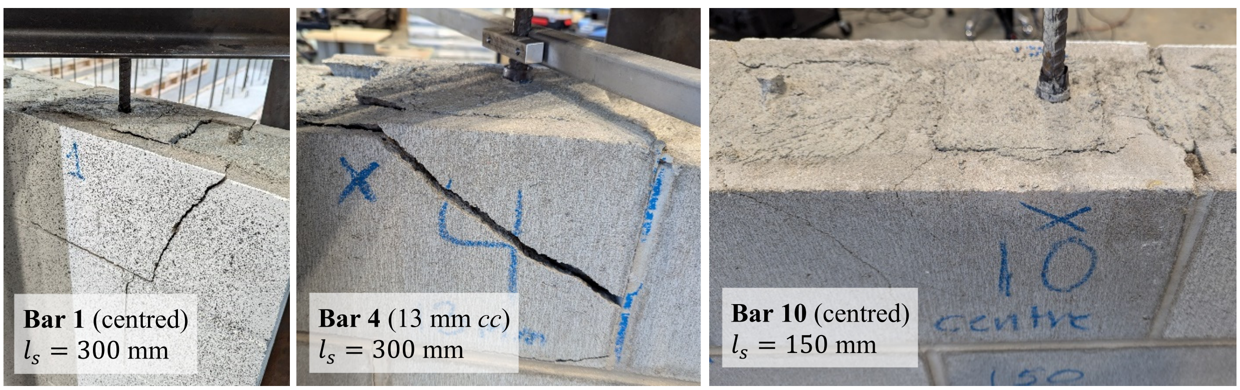

Figure 6: Observable cracks at the top course for bars with short splice lengths ($l_s = 150$ to 300 mm) (cc = clear cover).

For the first three tests (Bars 6, 7, and 10), the bar displacement was measured from the bar, close to the top end of the wall, to an LP installed on one face of the wall. It was noticed that readings were negative during most of the test. These negative readings were due to some bending of the rod attached to the pullout bar. The setup was modified with a stiffer fixture attached to the pullout bar and with displacements being measured from both faces of the wall and averaged, as shown in Figure 7. The new setup successfully corrected the problem and resulted in better bond-slip curves. Bond-slip curves from tests before and after this change are shown in Figure 8.

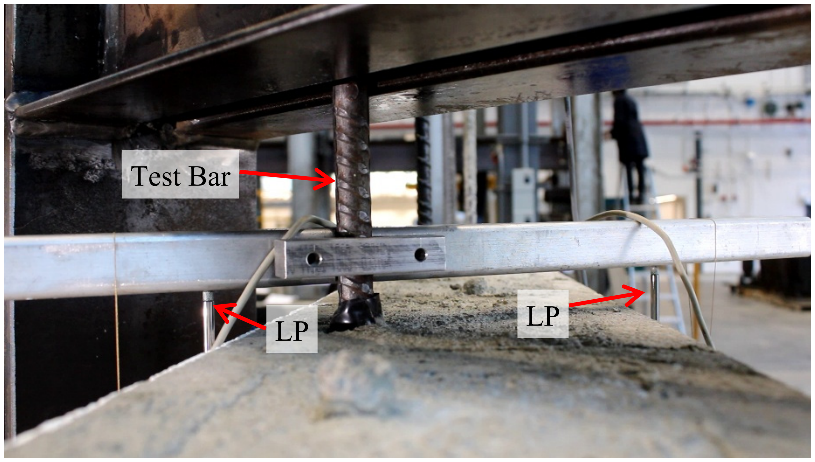

Figure 7: Improved configuration to read average displacement from both sides of the masonry wall (LP: linear potentiometer).

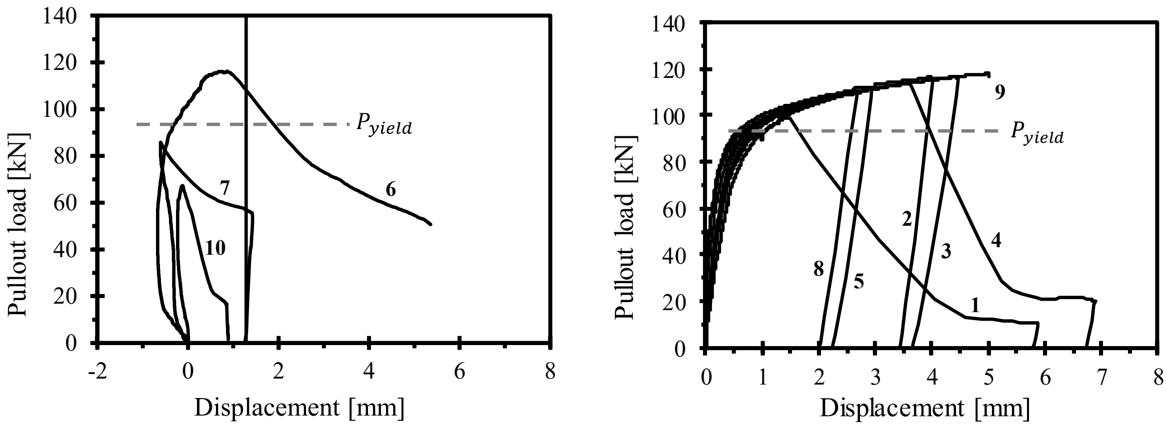

Figure 8: Bond slip curves from pullout test on masonry, (a) Before measurement correction, (b) After measurement correction.

Figure 9 shows pullout force versus splice length for all tests. Almost all bars exceeded their yield strength, with only two bars (Bars 7 and 10) failing in pullout before yielding. The pullout tests indicate that the current splice length expressions in the Canadian standard S304 are overly conservative, particularly for bars placed at the edge of the grouted cell (Bars 5, 6, and 8). These bars had a l_s between 10% and 30% of the theoretical l_d, yet they exceeded yielding. However, this outcome presents a limitation to deriving a new expression for development length using regression analysis. Development length equations are intended to predict the bond behaviour between reinforcement and the surrounding material up to the point of yielding, ensuring that bond failure governs the design rather than steel yielding. In cases where the bars yielded, the test results predominantly reflected the tensile properties of the steel rather than the bond characteristics. This diminished the bond behaviour and made it challenging to establish a direct relationship between splice length and bond strength. Without bond failure as the primary mechanism, the data was unsuitable for regression analysis.

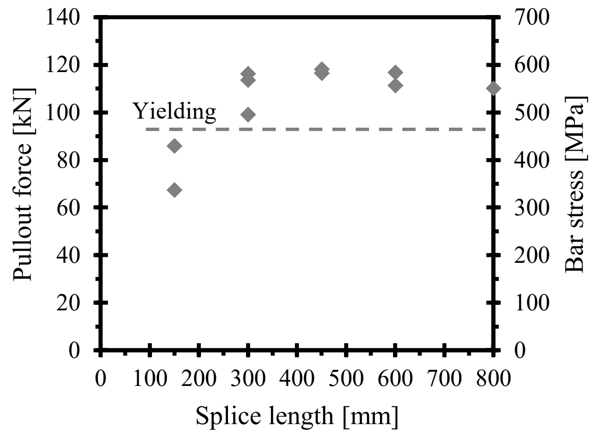

Figure 9: Pullout strength versus splice length of steel bars in specimen W1.

Future tests must ensure that pullout failure occurs prior to yielding to provide meaningful regression models to derive expressions for development length. This could be achieved by adjusting parameters such as reducing the splice length or modifying the grout properties to strategically weaken bond strength relative to steel capacity. Additionally, the effect of having different vertical bar sizes could be explored. However, any such modifications should be carefully calibrated to ensure the tests remain representative of practical conditions. Ensuring bond failure as the governing limit state would provide the necessary data to establish accurate relationships and predictive models for development length in masonry walls. The modified pullout test setup presented in this study allowed for efficient testing in series by accommodating multiple pullout bars simultaneously, streamlining the experimental process while providing reliable data. Changes in the specimen design of future tests should serve to obtain adequate data to develop new development length expressions.

CONCLUSIONS

This paper presents the first stage of an experimental program to investigate the bond strength of vertical bars in different positions inside the wall's cross-section. The use of vertical bars at the edge of the grouted cell intends to increase the out-of-plane (OOP) performance of slender masonry walls (SMW). The test setup consisted of a reacting steel frame that bore the load exerted by bars being pulled and passed the compressive force to the concrete slab at the base. A six-course masonry wall (W1) with 20 cm standard concrete masonry units (CMU) was built to accommodate ten 15M pullout bars with different splice lengths and positions. Bars had splice lengths (l_s) between 0.1 and 0.6 of the theoretical development length (l_d) per S304 to induce pullout failure prior to yielding of the reinforcement. Based on the results, the following conclusions are presented:

1. Most bars yielded before pullout failure, indicating that the bond strength exceeded the steel's yielding capacity in those cases. Shorter splice lengths, particularly for test bars at the edge of the grouted cell, will ensure that bond failure occurs before yielding, enabling the development of predictive equations.
2. The pullout tests showed that the current splice length expressions in the Canadian standard S304 are overly conservative, particularly for bars placed at the edge of the grouted cell.
3. The modified pullout test setup allowed for efficient testing in series by accommodating multiple pullout bars simultaneously, streamlining the experimental process while providing reliable data.

Empirical equations derived from experimental data are the basis for predictive and design expressions on development and splice lengths. Future tests should include reduced splice lengths, particularly for bars with a reduced clear cover, to ensure a pullout failure before the bar yields. Future tests should also include the use of other types of grouts.

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