RESILIENT INFRASTRUCTURE
June 1–4, 2016

FEASIBILITY OF USING UNBONDED REINFORCEMENT IN CONCRETE BLOCK WALLS

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ABSTRACT

An experimental program is underway at the University of Saskatchewan to investigate the performance of concrete masonry block walls reinforced with non-prestressed, unbonded reinforcement that are subjected to out-of-plane lateral loads. By eliminating the need for grouting, this construction method may lead to substantially lower wall weights, as well reduced construction times, cost, and workplace injuries. At the same time, the presence of reinforcement will increase the flexural strength of the walls, significantly increasing the allowable vertical spans of such walls as compared to those of comparable unreinforced walls. Unreinforced walls, as well as conventionally reinforced and partially grouted walls, were included in the experimental program to serve as control specimens against which the performance of the walls with unbonded, ungrouted reinforcement could be compared. Although the experimental program is ongoing, preliminary results described herein comparing the unbonded reinforced walls to unreinforced companion specimens suggest that this construction method is a promising alternative to more conventional systems.

Keywords: Concrete block walls, unbonded reinforcement, out-of-plane loading

1. INTRODUCTION

Unreinforced masonry walls cannot efficiently resist out-of-plane lateral loads such as those due to wind and earthquake since the resulting failure mode is generally governed by tensile cracking at mortar bed joints (Udey 2014). The inherently low tensile strength of the mortar, exacerbated by imperfect bond between the mortar and the concrete block units, severely limits the flexural capacity of such walls. As a result, the compressive strength of the concrete blocks is never fully realized.

The efficiency and flexural resistance of masonry walls can be significantly improved by providing longitudinal reinforcement in select block cells, with the reinforcement grouted in place to ensure strain compatibility with the surrounding cementitious materials (Drysdale & Essawy 1988). While the vertical span of unreinforced masonry walls is typically limited to a single storey, reinforced and partially grouted masonry walls can span much greater distances, making them more useful for open interior spaces such as building atriums or school gymnasiums. However, such construction methods are time consuming, increase project costs, and increase the risk of workplace injuries since they require workers to thread blocks up and over reinforcement that has already been grouted in place. The grout also substantially increases the dead load, with a fully grouted wall weighing approximately twice as much as a comparable ungrouted wall.

A novel solution is therefore sought to increase the out-of-plane flexural capacity of masonry walls while avoiding the disadvantages associated with conventionally grouted reinforcement. As such, an experimental investigation is being undertaken to study the use of non-prestressed, ungrouted and unbonded internal reinforcement in masonry walls to determine the resulting load-carrying capacity and serviceability characteristics. By anchoring the unbonded reinforcement at the top and bottom ends of the wall specimens, arching action can be engaged to resist the applied lateral loads and better utilize the compressive capacity of the masonry assemblage.

This paper presents the details of specimen construction, instrumentation, and testing for an experimental program comprising 21 full-scale wall specimens. Two sets of control specimens, one consisting of unreinforced walls, and the other of conventionally reinforced and partially grouted walls, were included to allow for comparison with the ungrouted walls that included unbonded reinforcement; since the test program is currently ongoing, only results from the unreinforced and unbonded reinforced specimens will be discussed herein. Preliminary test results, including visual observations of crack patterns and failure modes, load-deflection response, and ultimate capacity are also presented.

At this stage, the experimental program was intended to provide a proof-of-concept validation for the use of unbonded reinforcement in masonry walls with well-defined loading and boundary conditions. Since it is recognized that the construction method employed herein would not be feasible in practice, a subsequent phase of the study will be undertaken to develop and evaluate a practical implementation strategy.

2. EXPERIMENTAL PROGRAM

A total of twenty-one concrete masonry block wall specimens were included in this experimental program. All specimens were constructed with standard 200 mm concrete blocks laid in running bond by an experienced mason using standard Canadian construction practices. Three general categories of specimens were included: unreinforced walls, conventionally reinforced and partially grouted walls, and ungrouted walls that included unbonded reinforcement. The walls were tested to failure under monotonically increasing quasi-static lateral loading using a four-point loading system. The unreinforced and conventionally reinforced specimens served as control specimens that were used as benchmarks against which the performance of the wall specimens with unbonded reinforcement could be compared. The following sections describe the construction, testing, instrumentation, and material properties used.

2.1. Specimen Details

Figure 1 shows the elevation and representative cross-sections for the wall specimens included in this investigation. All specimens were 14 courses high and two-and-one-half blocks wide (approximately 2.8 m tall by 1 m wide), and featured standard 10 mm concave tooled mortar joints. Six of the wall specimens were unreinforced, six were conventionally reinforced and partially grouted, while the remaining nine specimens featured unbonded reinforcement. Longitudinal reinforcement for both the conventionally reinforced and unbonded reinforced specimens consisted of 6.4 mm diameter deformed steel bars conforming to ASTM A1064/A1064M-15 (ASTM 2015).

All specimens were laid atop reinforced concrete grade beams to reproduce a support condition commonly encountered in practice, with a 10 mm mortar joint included between the grade beams and the first masonry course. The grade beams were 1,700 mm long, extending beyond the block wall on each side to allow the grade beams to be securely clamped to the laboratory strong floor during testing. Grade beams used for the unreinforced (Fig. 1(a)) and conventionally reinforced (Fig. 1(b)) wall specimens were 300 mm wide and 400 mm tall. These grade beams were longitudinally reinforced with four No. 15 bars, one located at each corner, and No. 10 stirrups at 300 mm on-centre.

The grade beams used for the walls featuring unbonded reinforcement (Fig. 1(c)) had the same overall dimensions as those used for the other specimens, but required modifications to accommodate the dead-end anchors used to secure the unbonded reinforcement. For this purpose, two 200 mm tall x 200 mm long full-width blockouts were constructed at the bottom of the grade beams, centred on the wall reinforcing locations. The anchors bore against 240 mm x 153 mm x 9.7 mm thick steel plates embedded at the top of the blockouts; the anchor plates included a central 20 mm hole through which the wall reinforcing bars could pass. In addition, 20.9 mm PVC ducts were cast into the grade beams directly above both anchor plates to allow the wall reinforcing bars to pass freely through the grade beam. Modifications to the grade beam reinforcement required to accommodate the blockouts are illustrated in Figure 1(c).

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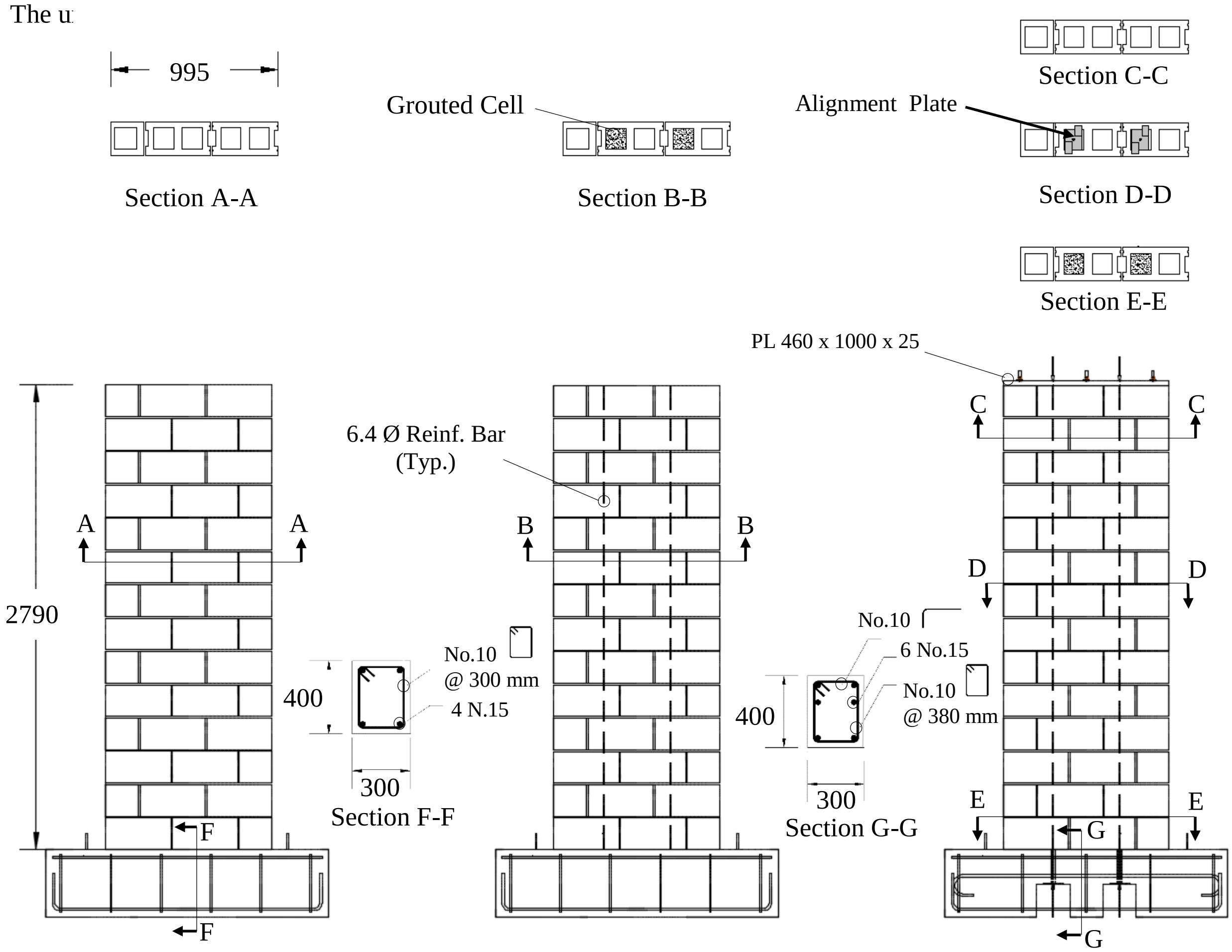

Figure 1: Cross-section and elevation of the wall splice specimens: (a) unreinforced walls, (b) conventionally reinforced and grouted walls, and (c) walls with unbonded reinforcement.

The conventionally reinforced walls shown in Figure 1(b) were constructed in two lifts, with the first lift consisting of eight courses, and the second of six courses. After the blocks for the first lift were placed, the reinforcing bars were placed in the first interior cell on either end of the wall specimens. The reinforced cells in the first lift were then grouted and allowed to cure for a minimum of 24 hours prior to erecting the second lift of masonry. Blocks in the second lift needed to be threaded over the reinforcing bars that were already grouted in place. Reinforced cells within the second lift were then grouted. The steel reinforcing bars were centered in the cells and held in place using welded wire mesh templates cast in the bed joints above the second, fourth, sixth, ninth, eleventh, and thirteenth masonry courses. Plywood strips with holes drilled through to accommodate the reinforcing bars were laced on top of the fully erected walls to further maintain the position of the reinforcing bars during placement of the second lift of grout and curing.

As shown in Figure 1(c), the configuration of the reinforcing bars for the nine wall specimens constructed with unbonded reinforcement was similar to that of the conventionally reinforced specimens, except that the reinforced cells were not grouted; instead, the reinforcing bars were anchored at the top and bottom ends of the wall using anchor chucks. In the absence of grout, one of the challenges for the unbonded specimens was to maintain the position of the reinforcing bars, not just during construction, but also during testing, since the relative distance between the compression face of the wall and the reinforcing bars was a critical factor in determining the flexural resistance of the wall. As a result, more robust reinforcing spacers were required for these walls than the welded wire templates used in the conventionally reinforced specimens. Figure 2 therefore shows the 100 mm x 110 mm x 5 mm thick steel alignment plates that were placed horizontally in the bed joints above the second, sixth, eighth, and twelfth block courses. These plates had 12 mm diameter holes drilled at their centre to allow the reinforcing bars to pass through. To maintain the position of the alignment plates within the block cell, vertical 60 mm x 40 mm x 5 mm thick steel plates were welded to the underside of the horizontal plates 25 mm in from both ends such that the vertical plates would fit fairly snugly adjacent to the face shell on either side of the cell when the alignment plate was seated within the block; the vertical plates also helped transfer to the block face shells any lateral loads generated by kinking of the reinforcing bars at the alignment plate locations during testing. Guide ropes were threaded up through the holes in the alignment plates as construction progressed so that the reinforcing bars could ultimately be pulled through the reinforced cells from the top end once the walls were constructed to their full height.

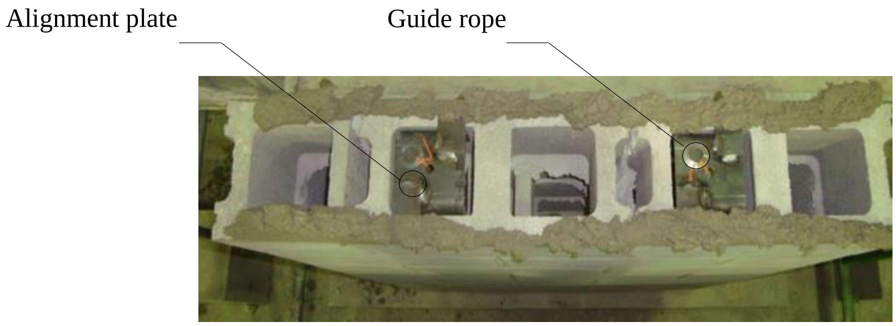

Figure 2: Reinforcement alignment plates used in specimens with unbonded reinforcement: restraints fabricated from steel plates embedded within the wall section

Prior to installation, the reinforcing bars for the unbonded specimens were instrumented with eight UFLA-1-11 120 Ω full bridge circuit metallic strain gauges (Tokyo Sokki Kenkyujo Co., Ltd.); pairs of gauges were located at 2500, 2600, 2700, and 2900 mm from the bottom of the wall or, in other words, at the level of the bed joints above the eleventh, twelfth, and thirteenth masonry courses. Wires connected to the strain gauges were threaded up through the reinforced cells and out the top of the specimen to be connected to the data acquisition unit.

To generate the required arching action in the walls with unbonded reinforcement once significant cracking and displacements had occurred, the compressive thrust force had to be transferred through the block webs from the loaded face of the wall at the crack located near mid-height to the unloaded face at the base of the wall (see Figure 3). The resulting potential for premature web failure near the base of the wall was mitigated in six of the nine unbonded specimens by fully grouting the bottom course of blocks; for purposes of comparison, the bottom course in the remaining three unbonded walls was left ungrouted. In the case of the six specimens with a grouted bottom course, the PVC tubing used in the grade beams was extended into the wall by a minimum of 200 mm so that the reinforcement could pass freely through the grouted first course. The grouted first course in those six walls was allowed to cure for a minimum of 24 hours prior to proceeding with the remainder of the wall construction. Since the top course of the wall bore more uniformly on the top support plate, web failure due to the action of the thrust force near the top support was deemed to be less of an issue, so that grouting of the top course was not done in any of the unbonded specimens.

Transverse reinforcement was not required in any of the specimens since the shear resistance of the specimens was determined to be greater than the flexural resistance and so would not govern. Once constructed, all specimens were cured in the laboratory for a minimum of 28 days prior to testing.

In preparation for testing, the walls were moved from their as-constructed position to the test bed, taking care to avoid cracking the specimens. Steel cross-beams were then placed over the grade beams on either side of the masonry wall and anchored to the strong floor in the Structures laboratory. To simulate a roller support at the top of the wall capable of providing lateral restraint without generating a bending moment or axial force, a 100 mm x 460 mm x 25 mm steel top plate was placed on top of the wall and connected to a rigid reaction frame by three horizontal steel rods that were pin-connected to lugs on the top plate at one end, and to the reaction frame at the other (see Figure 4); a layer of plaster was first applied to the top of the wall to ensure uniform contact. The horizontal reaction was provided by a 1.0 m long L75x75x5 structural angle bolted to the underside of the top plate and positioned with its vertical leg in secure contact with the unloaded face of the wall.

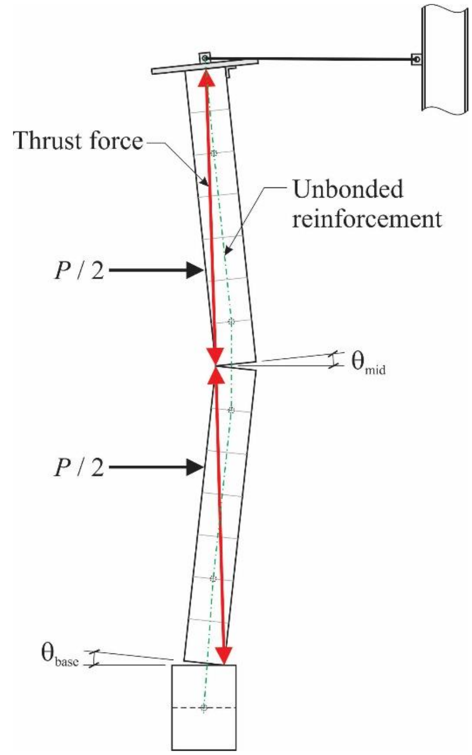

Figure 3: Schematic of wall with unbonded reinforcement in displaced position showing thrust forces.

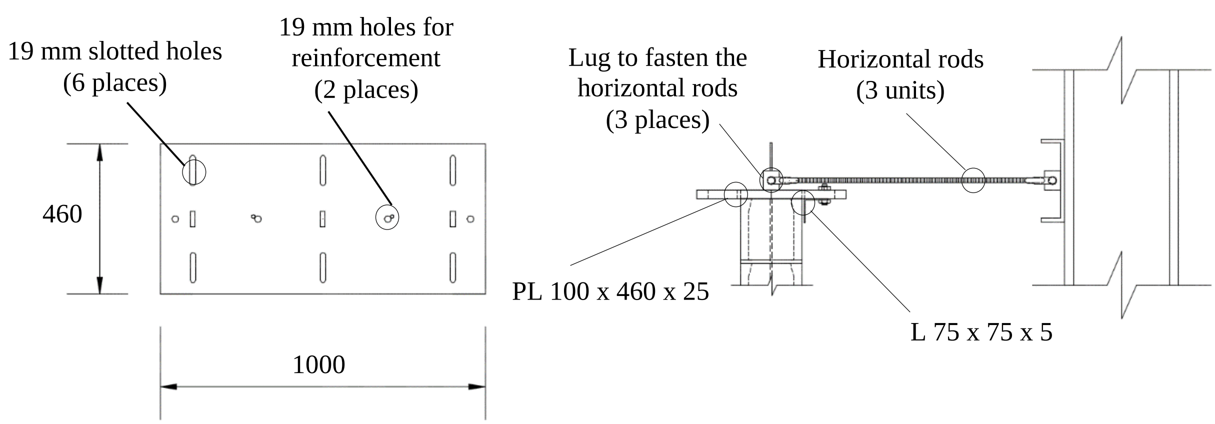

Figure 4: Top wall support, showing plan view of top plate and an elevation of the entire support assembly.

The roller support at the top of the wall was deemed to be a conservative approximation of the guide-angle supports typically used in practice for non-loadbearing walls. Binding of the wall within the guide-angle supports as the wall displaces laterally can generate compressive axial forces in the wall (Udey 2014) that would effectively supplement the arching action associated with the unbonded reinforcement. In a similar manner, moderate levels of superimposed axial loads applied at the top of a loadbearing wall would increase the resultant compressive force, thus complimenting the arching behaviour. The boundary conditions at the base of the wall specimens (i.e. a mortar joint laid directly on a concrete grade beam), on the other hand, was representative of a detail typically employed in practice

For specimens featuring unbonded reinforcement, the reinforcing bars were passed through 19 mm diameter holes drilled in the top plate and secured with the live-end anchor chucks that bore on the top plate, as well as with the dead-end anchor chucks positioned within the grade beam blockouts. Just prior to testing, the unbonded reinforcing bars were minimally stressed to an initial load of approximately 630 N (i.e. 3.6% of their actual yield force) to eliminate any slack in the bars and so ensure that they would resist load from the start of testing. Reinforcement in the conventionally reinforced walls terminated at the top of the wall.

A typical test setup is shown in Figure 5. A single MTS™ hydraulic actuator and a spreader beam assembly, centered at mid-height of the wall, was used to produce the four-point loading arrangement; the load points were vertically separated by 930 mm (i.e. the load points were located 465 mm on either side of the specimen mid-height), creating a constant bending moment zone in that region. The statically determinate spreader beam system was designed to ensure a symmetrical load distribution. Furthermore, the spreader beams extended the full width of the wall to produce uniform loading in the transverse direction. The load was applied under displacement control at a constant rate of 3 mm/min, except during the first test (specimen UG-1) for which the loading rate was set at 1 mm/min.

2.2. Material Properties

Hollow concrete masonry units with frogged ends were obtained from a single batch of material via a local supplier. The concrete units were delivered to the Structures Laboratory well in advance of construction to allow for the block temperature to equilibrate with that of the laboratory. The 15 MPa standard concrete blocks measured 390 mm long x 190 mm wide x 190 mm high. Half blocks were produced by cutting whole blocks in two, thus ensuring all masonry units had the same material properties. The compressive strength of the blocks was determined by using the testing protocol in ASTM Standard C140/C140M-15 (ASTM 2015). An average compressive strength value of 22.2 MPa (COV = 6.8%) was calculated using the net cross-sectional block area based on the results of 6 tests.

Mortar was prepared in the laboratory in accordance with CSA Standard A179-04 (CSA R2014) using Type S mortar cement, and a 3:1 masonry cement-to-sand ratio. Nineteen mortar cubes were cast and tested in accordance with CSA Standard A179-04 (CSA R2014). The average overall compressive strength for the mortar batches included in wall specimens tested thus far was 18 MPa with a coefficient of variation of 20%.

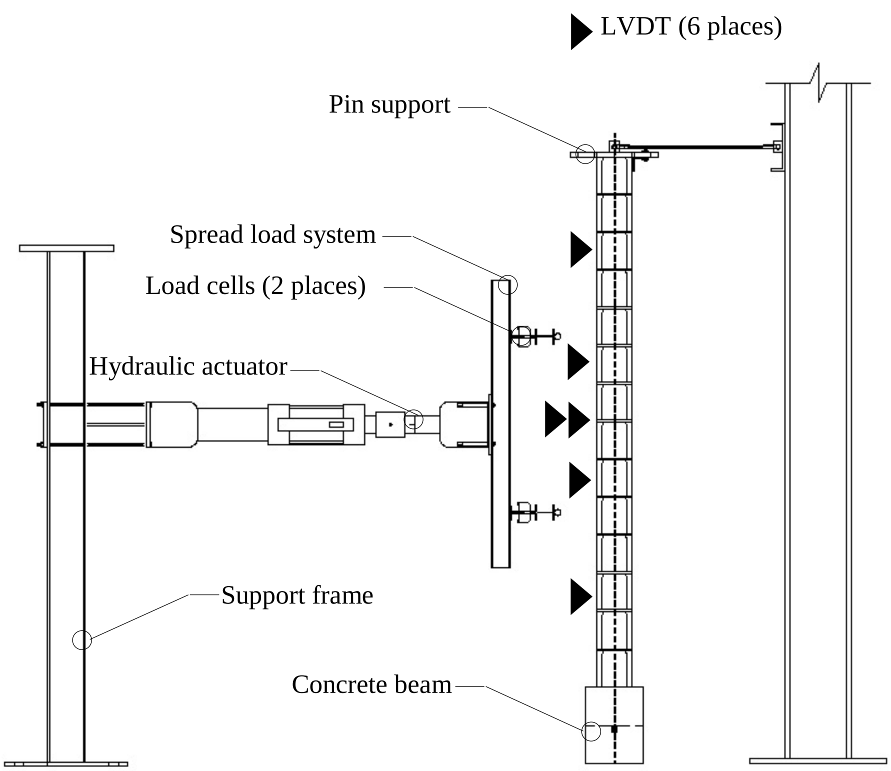

Figure 5: Schematic of loading arrangement and LVDT placement.

The grout was also prepared in the laboratory in accordance with CSA Standard A179-04 (CSA R2014) and consisted of Type GU cement, aggregate with a maximum particle size of 10 mm, and a 5:1 aggregate to cement ratio by weight. A water-to-cement ratio between 0.95 and 1.00 was used in the batching process and the target slump value immediately following batching was 250 mm. The average result from the slump tests was approximately 232 mm. Thirty non-absorbent 75 mm diameter by 150 mm long grout cylinders (three from every batch of grout) were cast and tested in accordance with ASTM Standard C1019-16 (ASTM 2016). In addition, thirty absorbent 190 mm high x 100 mm wide grout prisms (three per grout batch) were cast and tested in conjunction with the wall specimens tested thus far. The average compressive strength of the non-absorbent grout cylinders and absorbent grout prisms were 21.9 MPa (COV = 5.1%) and 13.4 MPa (COV = 4%), respectively.

One standard ungrouted one-block-wide by three-course-tall prism was constructed alongside each unreinforced wall and specimens containing unbonded reinforcement. One fully grouted prism of the same dimensions was constructed in conjunction with each conventionally reinforced and partially grouted wall specimen. Prisms were constructed and tested in accordance with CSA Standard S304-14 Annex D (CSA 2014a) with compressive strengths as reported in Table 1.

No. 10 and 15 Grade 400 hot-rolled reinforcing bars conforming to CSA Standard G30.18-9 (CSA R2014) were used to reinforce the grade beams. Bars of each size were obtained from a single heat batch of material. Six samples of each bar size acquired from excess lengths of material were tested in accordance with ASTM Standard A370-15 (ASTM 2015) to establish their material properties. The average yield strengths were 533 MPa (COV = 0.7%) and 464 MPa (COV = 1.8%) for the No. 10 and 15 bars, respectively.

Deformed steel bars with a diameter of 6.4 mm conforming to ASTM A1064/A1064M-15 (ASTM 2015) were used to longitudinally reinforce the wall specimens, where applicable. These bars had a nominal yield stress of 515 MPa. Six samples of these bars were tested in accordance with ASTM Standard A370-15 (ASTM 2015) to establish their average actual yield strength of 537 MPa (COV = 2.7%).

3. DISCUSSION OF TEST RESULTS

At the time of writing, 12 of the 21 wall specimens had been tested: six unreinforced (UR) specimens, three specimens with unbonded reinforcement and no grouting in the first course of blocks (UB-U), and three specimens with unbonded reinforcement and grouting in the first course of blocks (UB-G). A summary of preliminary test results is presented in Table 1, including strength of the masonry prism associated with each wall specimen, the applied load at cracking, the ultimate load, and the midspan deflection at the ultimate load. In addition, plots of the total applied load vs. midspan deflection for the wall specimens tested to date are provided in Figure 6(a); a separate plot featuring only the unreinforced specimens is also included in Figure 6(b) for clarity due to the large difference in vertical scale between the unbonded reinforced (UB) and unreinforced (UR) wall plots. More detailed discussions of the test results are presented in the sections below.

3.1 Cracking, Deflections and Failure Modes

As suggested in Table 1, the initial cracking in the unreinforced walls occurred at relatively low applied load levels ranging from 0.28 – 0.49 kN. It should be noted that this first cracking load was detected based on observed discontinuities in the load-deflection plots, rather than by the identification of visible cracks. In all but one specimen (UR-2), the first visible crack appeared in the mortar joints within the constant moment region near mid-height (at the bottom of courses 7 – 10); specimen UR-2, on the other hand, exhibited a visible crack above of the first course of blocks prior to a second crack forming near mid-height. Somewhat surprisingly, only unreinforced walls UR-1 and UR-2 developed visible cracks at the base of the wall.

Table 1: Preliminary Test Results

Specimen ID	Prism Strength (MPa)	Measured Cracking Load (kN)	Ultimate Load (kN)	Midspan Deflection at Ultimate Load (mm)
Unreinforced:				
UR-1	22.3	0.28	1.49	1.6
UR-2	21.9	0.37	2.32	0.4
UR-3	21.2	0.49	1.60	16.7
UR-4	19.4	0.37	1.42	0.6
UR-5	18.4	0.35	1.75	18.1
UR-6	20.2	0.35	1.12	3.5
Average:	20.5	0.37	1.62	6.8
COV (%):	6.7	18.5	22.8	110.9
Unbonded:				
UB-U1	23.2	1.77	4.35	85.5
UB-U2	20.6	0.75	6.89	62.0
UB-U3	21.4	1.54	6.80	58.0
UB-G1	19.7	1.69	8.51	39.2
UB-G2	20.2	1.69	7.49	52.9
UB-G3	20.6	2.27	7.27	56.6
Average:*	20.5	1.59	7.39	53.7
COV (%):*	2.7	30.7	8.3	14.6

* Excluding specimens UB-U1

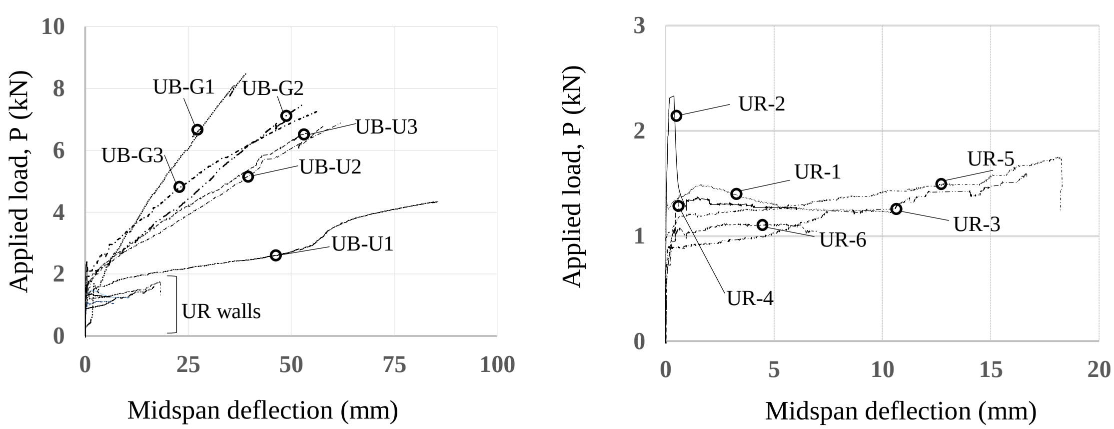

Figure 6: Applied lateral load vs. midspan lateral displacement plots: (a) All wall specimens; and (b) Unreinforced wall specimens only.

Figure 6(b) shows that five of the six unreinforced specimens exhibited a definite load-deflection plateau, maintaining load levels similar to the peak applied load over a displacement range that varied between approximately 7 – 18 mm prior to failure. Since an ideal pinned support at the base of the wall would have resulted in the creation of a collapse mechanism immediately after the formation of a mid-height crack (Udey 2014), this suggests that the grade beam support used in this study retained some minimal moment capacity after the initial formation of cracks, possibly as a result of the accumulation of mortar within the cells at the base of the wall and/or the resisting couple created by the wall self-weight as the vertical support reaction shifted from the middle of the wall to the unloaded face as the base of the wall rotated under load. For all of the unreinforced wall tests, the failure condition was defined as the point at which the MTS™ load control system detected a sudden drop in load-carrying capacity, rather than by total collapse of the wall.

As indicated in Figure 6(a), the first wall tested with unbonded reinforcement (UB-U1) exhibited much higher displacements and a lower apparent load-carrying capacity than the remaining five unbonded specimens. This was attributed to the fact that the system used to anchor the grade beam to the strong floor was not sufficiently rigid, allowing the grade beam to rotate appreciably; this rotational flexibility may have also contributed to the lack of visible cracking at the base of the unreinforced specimens, as noted above. For subsequent tests of walls with unbonded reinforcement, a more rigid anchorage system was implemented that appeared to adequately restrain the grade beam. This proved to be an important factor in improving the performance of the unbonded reinforced walls since it forced a sizable crack to form at the base of the wall, as well as near mid-height, thereby increasing the rate at which strain was induced into the unbonded reinforcement with increasing wall displacements.

For the remaining five wall tests featuring unbonded reinforcement, the initial visible cracks appeared in a mortar joint on the unloaded face of the wall within the constant moment region, followed by a crack at the base of the wall on the loaded face of the wall. As shown in Figure 6(a), the load carrying capacity of those specimens then continued to increase at any approximately constant rate with increasing wall displacements until large lateral displacements in the range of 30 – 60 mm had been attained. At this point, the mortar joint cracks near mid-height and at the base of the wall were both excessively large (12-17 mm); in addition, the wall segments above and below the mid-height crack were noticeably inclined (approximately 2 to 3 °). The tests were terminated prior to collapse of the wall when the MTS^{TM} loading system detected a sudden drop in load-carrying capacity. Although the cause of the drop in capacity could not be determined precisely, it is speculated that there may have been shear-related slippage at the mid-height cracked mortar joint brought about by large axial loads in the increasingly inclined wall segments on either side of the crack. It should be noted, though, that crushing was not observed in the mortar joints or blocks at that point, and that the average strain in the unbonded reinforcement was below (or, in the case of specimen UB-G3, just above) the nominal yield strain. Furthermore, upon unloading, the walls returned very nearly to their initial undeformed position, with the cracks closing to the point where they were hardly visible. When an attempt was made to reload specimen UB-U1 after the initial failure and unloading, it was found that the wall specimen exhibited a stiffness comparable to its post-cracking response in the initial load cycle.

3.2 Lateral Load-Carrying Capacity

It is evident from Table 1 and Figure 6(a) that the addition of unbonded reinforcement substantially increased the lateral load-carrying capacity of the masonry walls considered in this experimental program as compared to similar unreinforced walls. Specifically, the average ultimate applied load resisted by the walls featuring unbonded reinforcement was 356 % higher than that for the unreinforced specimens; in making this comparison, specimen UB-U1 was excluded due to the issues related to support anchorage flexibility, as discussed above. It is also interesting to note that the ultimate condition in the walls with unbonded reinforcement appeared to be limited as much by geometric considerations (large crack widths and the slope of the cracked wall segments) as by material strength.

In the wall specimens featuring unbonded reinforcement without grouting in the bottom course, no evidence of premature failure or distress was observed in the webs of the lower block courses due to transfer of the thrust force from the loaded to the unloaded face under arching action. The slight increase in capacity and stiffness seen in the unbonded specimens with a grouted first course (particularly UB-G2 and UB-G3), as compared to those with an ungrouted first course (UB-U2 and UB-U3) may be attributed more to an improvement in the alignment of the unbonded reinforcing bars over the first course due to the presence of the grout than to any strengthening effect that the grout had upon the webs. It can therefore be concluded that, for the wall configurations considered in this program, that grouting of the first course was not necessary to avoid web-related failures under arching action.

4. SUMMARY AND CONCLUSIONS

An experimental program has been undertaken at the University of Saskatchewan to investigate the use of unbonded, non-prestressed reinforcement in concrete masonry block walls in order to enhance resistance to out-of-plane lateral loads. When completed, a total of 21 full-scale wall specimens will have been tested to failure: six unreinforced specimens, six conventionally reinforced and partially grouted specimens, and nine specimens with unbonded reinforcement. This paper considers the preliminary results from tests of the six unreinforced walls, along with six of the nine walls featuring unbonded reinforcement.

The inclusion of unbonded, non-prestressed reinforcement was found to produce a dramatic improvement in both the lateral load-carrying capacity and ductility of the wall specimens. For the wall configurations considered in the study to date, the average lateral capacity of the walls was seen to increase by 356 %, while the average lateral displacement at failure increased by 690 %, as compared to those of unreinforced companion specimens. Since the strain, and hence the resisting force, in the reinforcing steel was largely governed by the displaced geometry of the wall specimens (i.e. rigid-body rotation of the wall segments after cracking near mid-height and at the base), it can be expected that walls with the same thickness but higher slenderness ratios will experience similar increases in bending moment capacity with the addition of unbonded reinforcement, albeit at higher lateral displacements; however, that would have to be verified by further testing. It was evident, though, that the tensile action of the unbonded reinforcement greatly enhanced the lateral stability of the walls considered in this study, even at very large lateral displacements, since the walls invariably returned to a straight configuration immediately after the load was removed.

Due to the potential benefits associated with the elimination of the need for grouting, including lower wall self-weight and improved constructability, the use of unbonded, non-prestressed reinforcement appears to hold significant promise based on the outcomes of this study to date. While the current phase of this study was intended primarily as a proof-of-concept investigation, subsequent phases will focus on the development and evaluation of methods by which unbonded reinforcement can be effectively incorporated into masonry walls in practice.

ACKNOWLEDGEMENTS

This research project is being funded by the Saskatchewan Centre for Masonry Design and the Saskatchewan Masonry Institute. The authors also gratefully acknowledge the financial support provided by the Ecuadorian Army, and by the government of Ecuador through the SENESCYT scholarship program.

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