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Towards Examining the Influence of Web Geometry on the Out-of-Plane Shear Resistance of Concrete Masonry Walls

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ABSTRACT

The current standards governing concrete masonry unit (CMU) geometry in Canada and the United States are CSA A165-14 - Concrete Masonry Units and ASTM C90-24 - Standard Specification for Loadbearing Concrete Masonry Units, respectively. Starting with its 2011 edition, ASTM C90 allows for a minimum web thickness of 19 mm irrespective of CMU size. In contrast, CSA A165 has maintained historically used minimum web thicknesses that increase with CMU size, starting at 26 mm for 100 mm nominally sized CMUs. The reduced CMU weight resulting from minimizing web geometry reduces the likelihood and severity of workplace injuries to masons. Additionally, the use of thinner webs increases the energy efficiency of masonry walls.

Webs are responsible for the transfer of shear forces within masonry assemblages. However, limited information was identified during a literature review relating to out-of-plane shear behavior and test methods for shear transfer in masonry assemblages. A recent experimental investigation at the University of Saskatchewan showed that grout columns in partially grouted and reinforced walls as typically constructed obscured the behavior of the CMU webs. This paper therefore provides the background and experimental design of a novel investigation underway at the University of Saskatchewan. Walls with unbonded reinforcement anchored at their top and bottom are being constructed and subject to out-of-plane loading to evaluate the shear capacity of CMUs with varying web geometries. The use of unbonded reinforcement isolates the effect of varying web geometry on shear capacity. Four CMU geometries are included for evaluation: regular stretcher CMUs meeting the minimum web thickness requirements specified in either CSA A165 or ASTM C90, and two types of knock-out CMUs. The experimental investigation as described herein will begin in Spring 2025.

KEYWORDS

Concrete masonry units, out-of-plane loading, shear, web geometry, unbonded reinforcement

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Background

Concrete masonry is a commonly used building material. The standards used to dictate the geometric properties of CMUs in Canada and the United States are CSA A165-14 [1] and ASTM C90-24 [2], respectively. Minimum web thickness as prescribed in CSA A165 [1] varies with CMU size, starting at 26 mm for 100 mm CMUs. In contrast, the 2011 [3] and more recent editions of ASTM C90 permit a minimum web thickness of 19 mm independent of CMU size and additionally define the normalized web area, A_{nw}, expressed in mm^2/m^2, for a regular two-cell stretcher CMU, as:

$$(1)\quad A_{nw} = \frac{t_w h_w N_w}{L_n H_n} \geq 45140 \text{ mm}^2/\text{m}^2$$

where t_w, expressed in mm, is the web thickness; h_w, expressed in mm, is the web height; N_w is the number of webs in the CMU; L_n, expressed in m, is the nominal length of the CMU; and H_n, expressed in m, is the nominal height of the CMU. Note that web height h_w differs from CMU height for the case of knock-out CMUs.

The primary motivation for reducing web geometry in the United States was based on improving the energy efficiency of masonry construction. Thermal resistance is inversely proportional to web thickness given that the webs of a CMU serve as thermal bridges [4].

Reduced web geometry also leads to a reduction in the weight of the CMU. Repetitive lifting of CMUs by masons can result in lower back compressive forces of up to 4.3 kN and so exceeds the US National Institute for Occupational Health and Safety recommended maximum of 3.4 kN [5]. Injuries resulting from lifting CMUs can lower the retention rate of masons in the construction industry and increase the rate of insurance claims. A reduction in CMU weight would therefore reduce the risk of workplace injuries and improve productivity on the jobsite. Finally, reduced CMU weight decreases transportation costs and so enables the use of masonry construction in remote areas and Indigenous communities.

There is a lack of experimental research on the structural effects of reduced web geometry ([6], [7], [8]) as masonry walls typically experience flexural failure and so are governed by face shell geometry. However, the webs of CMUs transfer shear stresses between the face shells [4] and so premature web cracking and failure could occur in walls constructed with CMUs with reduced web geometry. Thus, CSA A165 [1] has not adopted the web geometry requirements specified in ASTM C90 [2] despite the aforementioned benefits of using CMUs with reduced web geometry. The CSA S304 [9] code committee has requested confirmation of the structural adequacy of CMUs with reduced web geometry prior to implementing any changes to allowable CMU geometry in Canada.

Previous investigations ([6], [7]) evaluated the effect of web geometry on the axial compression capacity of masonry prisms. Most recently, Heide and Feldman [8] examined the influence of web geometry on the out-of-plane loading behaviour of reinforced concrete masonry walls and showed that web geometry did not influence out-of-plane flexural capacity. However, using grouted reinforcement tended to mask the influence of web geometry as the grout filled in the voids of the reinforced CMU cells. Thus, it was recommended to further the experimental program by testing walls that instead included unbonded and ungrouted reinforcement to isolate the effect of varying web geometry.

The current investigation as described herein aims to identify the effect of web geometry on the out-of-plane shear capacity of concrete masonry walls. Walls will be built using CMUs with different web geometries and reinforced with unbonded reinforcement. The desired failure mechanism is web shear in which the webs of the CMUs in walls are hypothesized to split under out-of-plane loading as they transfer shear stresses between face shells [4]. It is hypothesized that reduced web geometry will decrease the out-of-plane shear capacity of the walls.

Predicting the Capacity of Unbonded Reinforced Masonry Walls

It is necessary to identify analytical methods to predict the flexural and shear capacity of walls with unbonded reinforcement to ensure that the desired shear failure mechanism occurs during testing. This review therefore included an examination of multiple methods of predicting flexural and shear capacity.

Flexural Capacity of Unbonded Reinforced Walls

Most of the existing research regarding unbonded reinforcement involved either its use in reinforced concrete beams ([10], [11], [12]), or as prestressing or post-tensioning for masonry walls ([13], [14], [15], [16]). One investigation by Miranda et al. [17] proposed a novel technique to construct masonry walls with unbonded reinforcement as an alternative to conventionally grouted and reinforced walls to reduce construction time. The walls were 14 courses tall by 2.5 CMUs wide with one 6.4 mm diameter steel reinforcing bar placed in the first interior cell from each side and anchored at the top and bottom of the wall. A pretension force of around 600 N was applied to the reinforcement to remove any slack in the bars. Additionally, spacers were placed in the mortar joints throughout the wall height to maintain a constant moment arm between the tensile reinforcement and the extreme compression fibre. All walls were cast on top of a grade beam and pinned at the top to simulate a realistic simply supported condition. The walls were loaded until a substantial drop in lateral load capacity was observed. However, no crushing of masonry or snap-through failure was observed. Additionally, removal of the applied load restored the walls to their original shape. The flexural capacity of walls with unbonded reinforcement was found to be 82% that of conventionally reinforced and partially grouted walls. Miranda et al. [17] concluded that unbonded reinforcement resulted in a similar flexural capacity to that obtained when conventional reinforcement techniques are implemented.

The walls in Miranda et al.'s investigation [17] did not behave like conventionally reinforced walls due to the lack of strain compatibility resulting in a uniform tensile force in the reinforcement and so produced a three-hinge deflected shape. Thus, analysis methods for conventionally reinforced walls may not adequately capture the behaviour of unbonded reinforced walls. Miranda et al. [17] therefore developed an analytical load-deflection procedure to evaluate the lateral load capacity of the walls. This method involved establishing the geometry of the wall knowing the lateral deflection at mid-height at the peak load level, determining the strain in the reinforcing steel, and solving for the applied lateral load through equilibrium of forces. This analytical model was typically accurate to within 15% when predicting the peak applied load. However, the analytical model did not allow for the calculation of the maximum capacity of the wall prior to testing as the magnitude of the mid-height deflection served as an input parameter. This model can therefore only be used following testing and data collection rather than as a predictive tool and so is not applicable to the investigation as planned.

Several methods of predicting flexural capacity were compared to the test data as reported by Miranda et al. [17]. Equations to predict the stress in prestressed tendons as provided in Clause 14.3.2.3 and Clause 4.4.3.7.2 of CSA S304 [9] and TMS 402 [18], respectively, were used to determine the stress in the unbonded reinforcement at maximum load and so predict flexural capacity. As-tested material strengths from Miranda et al.'s investigation [17] were included in this prediction with experimental observations showing the presence of a plastic hinge at mid-height in the walls and an additional hinge present at each support. However, the code equations were developed assuming large prestress forces in the reinforcing steel prior to lateral load application which was not the case for walls as reported by Miranda et al. [17]. Thus, this method under-predicted the flexural response of the walls by over 100% as compared to experimental results. Equations for the design of prestressed masonry are therefore not able to predict the flexural capacity of unbonded reinforced walls.

Bartlett [10] investigated concrete beams with exposed sections of reinforcement to evaluate the strength of members in which concrete cover has been removed to repair corroded steel reinforcement. Analysis methods were presented to predict the flexural capacity of such beams. Bartlett [10] tested a 4000 mm long reinforced concrete T-beam with flange and web widths of 800 mm and 200 mm, respectively, a flange thickness of 90 mm, and an overall height of 400 mm. Flexural reinforcement consisted of two 25M longitudinal bars with an effective depth of 350 mm. A 2000 mm long region of exposed and unbonded reinforcement was included at midspan. Shear reinforcement consisted of 10M stirrups spaced at 200 mm on-centre outside of the region of exposed reinforcement. This beam was tested to failure under four-point loading to assess its flexural capacity.

Bartlett [10] proposed three analysis techniques to predict the flexural capacity of this beam: (1) a flexural analysis as per Clause 10.1.1 of CSA A23.3 [19] assuming the beam had no exposed sections of reinforcement, (2) a strut-and-tie model that assumed a neutral axis depth that was contained within the top flange of the test beam, and (3) an analytical model that relied on deformation compatibility at the ends of the exposed length of reinforcement. The elongation along the exposed length of reinforcing steel predicted by the third model was equated to the theoretical elongation of the concrete at the level of the reinforcing steel assuming strain compatibility. This model satisfied horizontal force equilibrium, moment equilibrium, and deformation compatibility requirements at the ends of the exposed length of reinforcement. The three models were able to predict the flexural capacity of the beam to within 5%, 1%, and 5% of experimentally observed values, respectively.

Analysis techniques developed by Bartlett [10] were compared to the as-tested results of walls reported by Miranda et al. [17]. None of Bartlett's [10] three models were accurate in describing the physical behaviour of these walls. Method 2 relied on the presence of shear reinforcement. Method 3 relied on a large variation in the neutral axis depth along the beam length which is not possible for ungrouted masonry walls subject to lateral loading: the neutral axis must be contained to the front face shell of the CMUs as there is no contact between the webs of successive masonry courses. However, Method 1 implemented by Bartlett [10] predicted flexural capacity to within 8% of the as-tested value for Miranda et al.'s [17] walls and so appears to be a suitable technique to predict the flexural capacity of unbonded reinforced walls despite the differences in their mechanics as compared to conventional partially grouted and reinforced masonry walls.

Out-of-Plane Shear Capacity of Unbonded Reinforced Walls

Very little research was found related to the out-of-plane shear resistance of masonry as shear rarely governs its structural design. Unbonded reinforcement will not contribute any dowel action to resistance and so will have little effect on shear strength. Additionally, the compressive stresses resulting from flexure will be carried solely by the front face shell of the wall. The face shell bedding used for walls constructed in running bond eliminates contact between the CMU webs in subsequent courses and so they do not aid in the transfer of compressive stresses. The mechanisms of shear resistance of the walls are therefore expected to be more similar to unreinforced walls rather than conventionally grouted and reinforced walls.

Both Clauses 8.10.3 and 2.2.5.2 as are included in CSA S304 [9] and TMS 402 [18], respectively, can be used to predict the shear strength of unreinforced masonry, f_v. The equations as they appear in the codes are a function of the square root of masonry assemblage strength f'_m. However, use of the compressive strength of the CMU, f'_{CMU}, in place of masonry assemblage strength f'_m is hypothesized to result in a more accurate prediction of the shear strength of masonry given that the shear failure of walls reinforced with unbonded reinforcement is expected to occur in the webs of the CMU [20]. The two code equations were therefore modified to include f'_{CMU} in place of f'_m. Eq. (2) can be used to predict the shear strength of masonry, $f_{v,CSA}$, based on modifying Clause 8.10.3 in CSA S304 [9] as previously discussed while Eq. (3) can be used to predict the shear strength of masonry, $f_{v,TMS}$, converted so as to be in MPa rather than psi, as per modifying Clause 2.2.5.2 in TMS 402 [18]:

(2) $f_{v,CSA} = 0.16\sqrt{f'_{CMU}}$ [MPa]

(3) $f_{v,TMS} = 0.12\sqrt{f'_{CMU}}$ [MPa]

While otherwise identical, the coefficients included in Eqs. (2) and (3) differ as the prism geometry used to determine masonry assemblage strength as included in CSA S304 Annex D [9] and ASTM C1314 [21] are different and so result in dissimilar as-tested values of f'_m [22]. However, both CSA S304 Annex D [9] and ASTM C140 [23] specify the same CMU compressive strength testing procedures and so the as-tested value of f'_{CMU} is identical. Thus, Eqs. (2) and (3) result in different predictions of the shear strength of masonry simply as a result of the coefficients included.

The shear strength of masonry calculated in accordance with Clause 8.10.3 of CSA S304 [9] is multiplied by the effective area of the wall cross-section to determine shear resistance of the wall with the underlying assumption that shear failure will occur either by mortar joint sliding or by diagonal cracking through the CMU and mortar [24]. However, Clause 2.2.5.2 of TMS 402 [18] is intended for use with the shear flow equation included in Clause 2.2.5.1 and so predicts web splitting shear failure at the neutral axis [20]. Shear capacity calculated using the shear flow equation was substantially lower than shear capacity predicted using Clause 8.10.3 of CSA S304 [9] for the walls as included in this investigation and so it is anticipated that web splitting shear failure will occur. Thus, Eq. (3) and Clause 2.2.5.1 of TMS 402 [18] will be used to predict the shear strength of masonry.

Eq. (4) modifies Clause 2.2.5.1 of TMS 402 [18] by replacing the section thickness, b, with the combined thickness of all the webs along the wall cross-section, $t_w N_w N_{CMU}$, and can be used to predict the shear stress τ, expressed in MPa, at any location along the wall cross-section.

(4) $\tau = \frac{1000VQ}{It_w N_w N_{CMU}}$ [MPa]

where I, expressed in mm^4, is the second moment of area of the wall cross-section; N_{CMU} is the number of CMUs along the wall width; V, expressed in kN, is the shear force; and Q, expressed in mm^3, is the first moment of area of the wall cross-section at the location of where the shear stress is being evaluated.

Eq. (4) provides the maximum value of shear stress in the wall if calculated at the neutral axis location. Thus, Eq. (4) can be rearranged to solve for the maximum shear force, $V = V_{max}$, expressed in kN, that the wall can sustain when the shear stress at the neutral axis reaches the shear strength of masonry:

(5) $V_{max} = \frac{f_{v,TMS} I t_w N_w N_{CMU}}{1000Q}$ [kN]

Note however that Eq. (5) does not provide an indication of the effect of web height on shear resistance. Knock-out CMUs have a reduced normalized web area when compared to full height CMUs and so have less web area to resist shear stresses. Additionally, the knock-out webs result in discontinuities in the distribution of shear stress across the wall. It is therefore assumed for the purposes of the experimental design that the shear capacity of walls made from knock-out web CMUs is proportional to web height. Eq. (5) was therefore modified for use in predicting the shear resistance of walls constructed using knock-out CMUs by multiplying by the ratio of knock-out web height to the full web height of 190 mm.

The purpose of this literature review was to evaluate existing research related to unbonded reinforcement and out-of-plane shear capacity of masonry walls. Very little research was found on either of these topics. However, web geometry is hypothesized to influence the out-of-plane shear capacity of concrete masonry walls as webs are responsible for the transfer of shear stresses [4]. Therefore, walls with different web geometries will be constructed and tested under out-of-plane loading to experimentally verify how web geometry influences shear capacity.

EXPERIMENTAL DESIGN

Twenty-eight walls will be constructed with CMUs with four different web geometries and tested under four-point out-of-plane loading to evaluate the effects of web geometry on shear capacity. The following sub-sections describe the walls, including the CMU types used and the wall test setup. The anticipated construction and data analysis is also presented. All experimental work will begin in Spring 2025.

Unbonded Reinforced Walls

Walls will be constructed using standard two-cell 200 mm stretcher CMUs with four different web geometries. Fig. 1 shows the overall geometry and reinforcing arrangement of the walls. All walls will be constructed in running bond and will be 9 courses tall by 2.5 CMUs wide resulting in total wall dimensions of 1790 mm by 990 mm. Type S mortar will be applied in face shell bedding by an experienced mason.

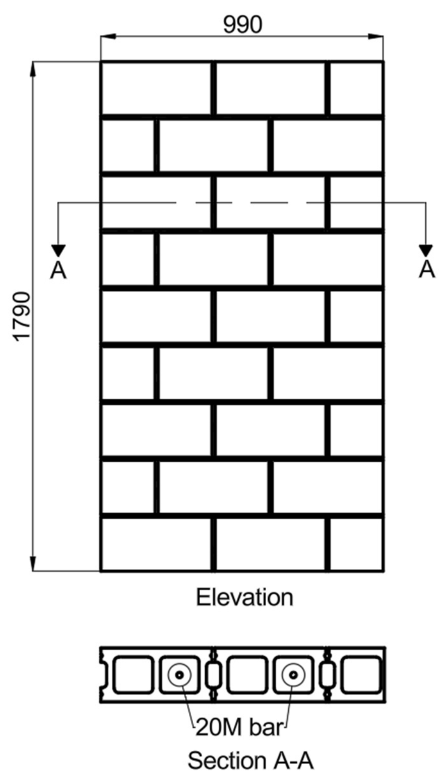

Figure 1: Overall wall geometry (all dimensions in mm)

The walls will be reinforced with one unbonded 20M bar in the first interior cell adjacent to each side face to achieve a symmetric reinforcement arrangement. The cells in which the bars are located are inconsequential as shear resistance is provided by the masonry alone. Reinforcement installation will be similar to the method used by Miranda et al. [17] and so bars will be anchored at the top and bottom once the walls have been constructed and cured for the required 28-day period. The bars will be machined to have a threaded length of 50 mm at each end allowing nuts to be installed at the ends of the bar and bear against a steel plate on the top and bottom of the wall to serve as an anchoring mechanism. Finally, 5 mm thick and 110 mm wide steel plate alignment spacers will be placed in the bed joints above the second, fourth, fifth, and seventh CMU courses from the bottom of the wall. The bars will be placed through a pre-drilled 25 mm diameter hole in these spacers during their installation to ensure that the moment arm between the reinforcement and the loaded face of the wall is kept constant.

Types of CMUs

Fig. 2 shows the four CMU types that will be used to construct each of the four series of walls. Fig. 2(a) shows the regular stretcher CMUs with full height webs that correspond to CSA A165 [1] web geometry requirements and so have a web thickness of 26 mm. Fig. 2(b) shows the regular stretcher CMUs with full height webs that correspond to ASTM C90 [2] requirements and so have a web thickness of 19 mm. Fig. 2(c) shows the knock-out CMUs meeting the geometric requirements included in CSA A165 [1] with a web height of 140 mm and a thickness of 26 mm. These CMUs were chosen as they have a normalized web area of 136500 mm^2/m^2 and so differ by less than 1% from the normalized web area of 135375 mm^2/m^2 for full web height CMUs meeting the geometric requirements included in ASTM C90 [2] with a web thickness of 19 mm. Finally, Fig. 2(d) shows the knock-out CMUs meeting the geometric requirements included in ASTM C90 [2] and so have a web height of 65 mm and a thickness of 19 mm. This web geometry results in a normalized web area of 46312 mm^2/m^2 and so exceeds the minimum requirements of ASTM C90 [2] by 2.5%. Comparison of the peak load sustained by the four different test series of walls will determine the influence of web height, web thickness, and normalized web area on the shear capacity of walls. The use of seven replicates for each test series was established using an independent two-tailed equal variance t-test as the number necessary to identify statistically significant differences between the mean values of shear capacity of different test series at a 95% confidence level.

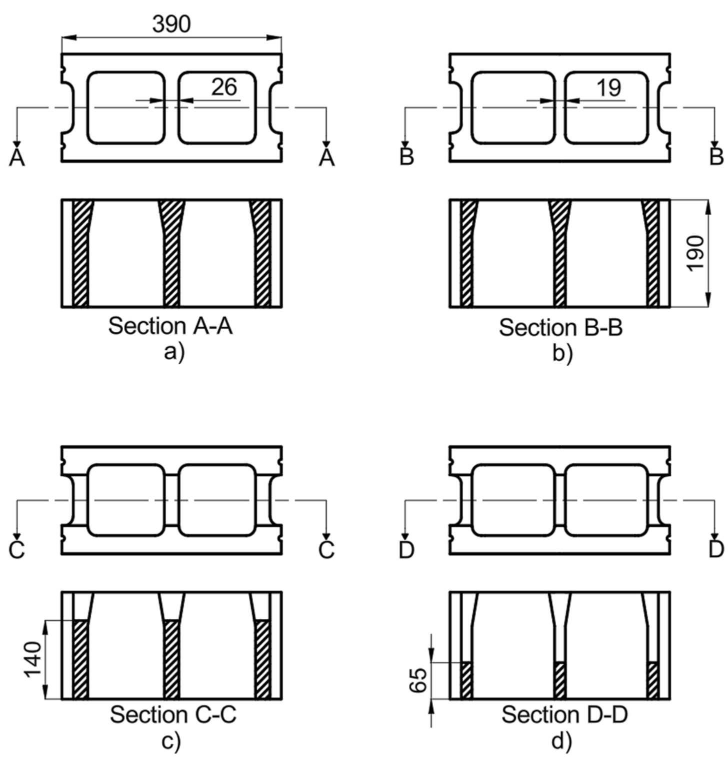

Figure 2: CMU types: a) CSA A165 regular stretcher unit, b) ASTM C90 regular stretcher unit, c) CSA A165 knock-out unit, and d) ASTM C90 knock-out unit (all dimensions in mm)

Table 1 shows the predicted shear capacity and maximum load capacity of walls built using each type of CMU. A nominal CMU strength of 20 MPa was assumed for these calculations. The flexural capacity for all test series is 20.2 kN-m as calculated in accordance with Clause 11.1.1 of CSA S304 [9]. Shear capacity was calculated using Eq. (5) with masonry shear strength calculated using Eq. (3). Static equilibrium was then used to determine the maximum load capacity of the walls. The factor of safety against flexural failure was then calculated by dividing the flexural capacity of the walls by the midspan bending moment at the maximum load. The factor of safety against flexural failure is greater than 1.0 for all test series indicating that a shear failure is likely to occur.

Table 1: Wall Testing Matrix

CMU Used for Test Series	Shear Capacity (kN)	Predicted Maximum Load (kN)	Factor of Safety Against Flexural Failure at Maximum Load
CSA A165 regular stretcher unit	16.3	32.6	2.47
ASTM C90 regular stretcher unit	12.1	24.2	3.33
CSA 165 knock-out unit	12.0	24.0	3.36
ASTM C90 knock-out unit	4.14	8.28	9.74

Test Setup

Fig. 3 shows the test setup that will be used in this investigation. All walls will be constructed on a steel base plate similar to that used by Heide and Feldman [8] to create an idealized pin support condition at the bottom of the wall. Two steel angles will be bolted onto the plate flush with the front and back of the wall to prevent any horizontal displacement of the bottom of the wall. A steel plate with a machined groove along its length will connect to the bottom of the base plate and rest on top of a thin vertical plate. Additionally, a roller support condition will be created at the top of the wall by using another steel plate with two angles supported against the testing frame using three solid circular steel braces connected via ball joints. An MTS actuator with a capacity of 250 kN connected to a spreader beam with two round HSS sections spaced 800 mm centre-to-centre apart from one another will be used to apply quasi-static four-point loading to the wall. This will create an 800 mm region of pure bending in the middle of the wall and two shear spans of roughly 500 mm each at the top and bottom of the wall. The circular sections on the spreader beam will be equipped with load cells to record the force applied to the walls as testing progresses and allow for measurement of the maximum out-of-plane capacity of the wall. Three laser gauges will be set up on the testing frame at the mid-height location on the wall and at the locations of applied load to record out-of-plane displacements of the wall throughout testing.

Construction and Data Analysis Forecast

Construction of the 28 walls will be split into three phases, beginning in Spring 2025. An initial construction and testing phase will consist of four walls with one wall built using each CMU type. The smaller initial phase will be used to verify that the desired shear failure mode occurs for the walls constructed with each of the CMU types. The remaining walls will be constructed and tested in two phases with each phase consisting of 12 walls. The walls tested in the first phase will be constructed with the two CMU types meeting the geometric requirements as specified by CSA A165 [1]. The walls tested in the second phase will be constructed with the two CMU types meeting the geometric requirements as specified by ASTM C90 [2].

Data analysis will focus on determining the influence of web geometry on the shear capacity of the walls by comparing the maximum out-of-plane loads sustained by walls constructed using the four CMU types.

The effect of web thickness will be evaluated by comparing full web height CMUs corresponding to CSA A165 [1] and ASTM C90 [2] web geometry requirements, respectively. The effect of web height will be evaluated by comparing the two types of CMUs with full height webs to their respective knock-out counterparts. Finally, the effect of normalized web area will be evaluated by comparing full web height CMUs corresponding to ASTM C90 [2] web geometry requirements to 140 mm knock-out web CMUs corresponding to CSA A165 [1] web geometry requirements as they have a similar normalized web area.

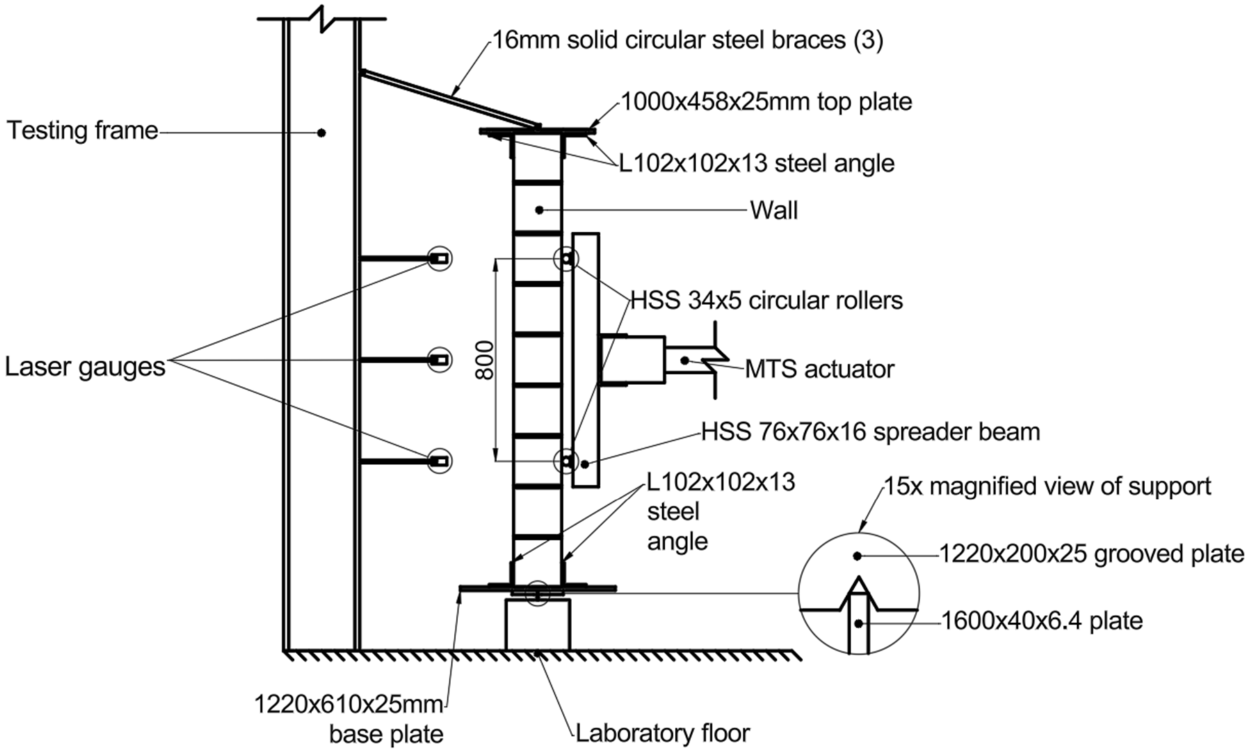

Figure 3: Test setup (all dimensions in mm)

SUMMARY AND CONCLUSIONS

This paper presents the background and experimental design of an investigation to examine the influence of concrete masonry unit (CMU) web geometry on the out-of-plane shear capacity of masonry walls. The objective of this investigation is to evaluate whether CMUs with reduced web geometry requirements as specified in ASTM C90 can be incorporated into CSA A165 for use in Canada. Advantages of using CMUs with reduced web geometry include energy efficiency, workplace safety, and reduced transportation costs. However, results of previous investigations indicated that the presence of grout in reinforced cells of masonry walls tended to mask the influence of varying web geometry. Thus, the walls constructed in this investigation will be reinforced with unbonded steel bars to isolate the effects of varying web geometry. Walls will be designed to fail in shear to evaluate the influence of web geometry on shear capacity. Multiple analytical models were examined to identify a method to predict the flexural and shear capacity of the walls and revealed the following:

- Equations used to predict the flexural capacity of prestressed masonry walls do not adequately capture the flexural capacity of unbonded reinforced walls when only a small amount of pretension is applied.

- A flexural analysis as per Clause 11.1.1 of CSA S304 predicts the capacity of unbonded reinforced walls to within 8% of experimentally observed values.
- The shear flow equation combined with the equation for the shear strength of masonry presented in Clause 2.2.5.2 of TMS 402 was used to predict the out-of-plane shear capacity of unbonded reinforced walls. The resulting predictions will be compared to the test data.
- The proposed analytical methods were used to design 1790 mm tall by 990 mm wide walls reinforced with two unbonded 20M steel bars. These walls will be simply supported and tested to failure under four-point loading.
- Four test series of walls constructed with different types of CMUs will allow for an assessment of the influence of web geometry on shear capacity: CMUs meeting CSA A165 requirements for minimum web thickness, CMUs meeting ASTM C90 requirements for minimum web thickness, knock-out CMUs meeting CSA A165 requirements of minimum web thickness and having similar normalized web area to full-height ASTM C90 CMUs, and knock-out CMUs meeting ASTM C90 requirements for minimum normalized web area.

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