SURFACE REINFORCED CONCRETE MASONRY UNITS: AN INTRODUCTION TO THE FUTURE OF CONCRETE MASONRY

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Abstract: Conventional concrete masonry assemblies using hollow Concrete Masonry Units (CMU) are reinforced by bonding reinforcing bars within the cores using masonry grout. This locates the reinforcing bars at or near the masonry system's out-of-plane neutral axis. The innovative Surface Reinforced Concrete Masonry Unit (SRCMU) uses vertical channels in its external faces to allow near-surface-mounting (NSM) of reinforcement. This hollow concrete masonry construction method allows greater flexural strength to be achieved while reducing the over-all weight of the concrete masonry assembly by up to 50% due to the elimination of grout filling of the cores. This fosters significant construction cost savings and reduced environmental impact. A comparison of masonry systems utilizing the SRCMU with NSM reinforcement and conventional hollow concrete masonry systems was performed using CSA S-304.1 for out-of-plane loading conditions. Under conditions of combined axial and out-of-plane flexural loading, the SRCMU masonry systems had an increased flexural capacity of up to 30% over the conventional concrete masonry systems with the same reinforcement ratio and effective cross sectional area. Physical testing of unreinforced conventional masonry prisms and SRCMU prisms showed that they have similar behaviour and modes of failure under axial loading conditions. SRCMU flexural specimens reinforced using epoxy-grouted reinforcing bars were tested under 4-point bending. Steel reinforced specimens achieved an average resistance that matched the prediction from the CSA S-304.1 analysis; this demonstrates SRCMU systems can achieve greater load carrying capacities with less material while maintaining modes of failure and design characteristics similar to conventional CMU construction.

1 BACKGROUND.

Conventional reinforced hollow concrete masonry walls are typically constructed as shown in Figure 1. The Concrete Masonry Units (CMU) are placed in running bond, and bonded together with 10mm mortar joints. The hollow cores of the masonry blocks form a continuous vertical channel into which reinforcing steel bars can be placed. This channel is then filled with masonry grout in order to bond the reinforcing bars to the masonry system. Joint reinforcement, in the form of a steel wire ladder placed within the mortar joints, may also be used and acts as horizontal reinforcement.

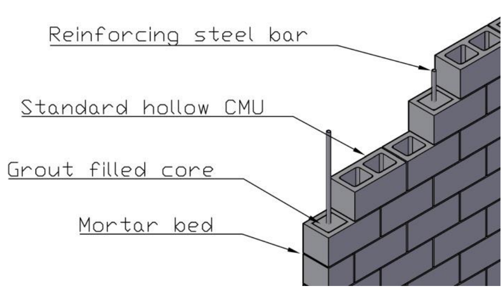

Figure 1: Conventional reinforced CMU construction

A new trend in rehabilitation of existing concrete and masonry structures is to use near surface mounted (NSM) reinforcement as opposed to surface mounted Fibre Reinforced Polymer (FRP) reinforcement (De Lorenzis and Teng 2006). The reasons for rehabilitation include deterioration of the existing structure and upgrading of reinforcement to increase resistance to service loads or environmental loading such as wind or earthquake loading. The benefits of NSM over surface mounted FRP include better esthetics because the reinforcement can be made inconspicuous, as well as reduced vulnerability to fire and surface abrasion. With this technique it is possible to achieve strength equivalent to that of members with cast-in reinforcement (Almusallam et al 2013). This technique has been successfully applied to structures in Italy (Tumialan et al 2001) and Australia (Dizhur et al. 2013).

The NSM technique for reinforcement appears as of yet unexplored for new construction. Application of this technique for new concrete structures would be impractical because of the additional expense of cutting or casting in channels for the NSM reinforcement, however by using a modified hollow concrete masonry unit, NSM reinforcement could be applied to new masonry constructions without additional surface preparation. Using NSM reinforcement for new hollow masonry construction would have many benefits over conventional reinforced masonry construction; e.g. less mass, more efficient reinforcement location, better control of reinforcement placement, faster construction.

NSM reinforcement has been explored for application to stack bonded hollow concrete masonry (Carney and Myers 2003); NSM reinforced stack-bonded hollow concrete masonry test walls were constructed by chipping out the mortar from the vertical mortar joints to a depth of twice the diameter of the reinforcing bar used and fixing the bars in place using an epoxy paste. However this technique is impracticable with running bonded masonry assemblages without modifying the blocks. Researchers at the University of Manitoba have developed a new masonry unit called the Surface Reinforced Concrete Masonry Unit (SRCMU). In this new unit the shape of the conventional hollow CMU is modified to accommodate a vertical channel along the outer surface. Figure 2 shows how a masonry wall built using SRCMUs in running bond can be reinforced by placing reinforcing bars within the surface channels using an epoxy dowelling adhesive. This innovative system allows reinforcing bars to be placed further away from the neutral axis than with conventional masonry, resulting in a potential for greatly increased flexural resistance. This system also allows masonry walls to be reinforced without filling the hollow cores of CMUs with masonry grout, allowing for a decrease in self-weight of up to 50%.

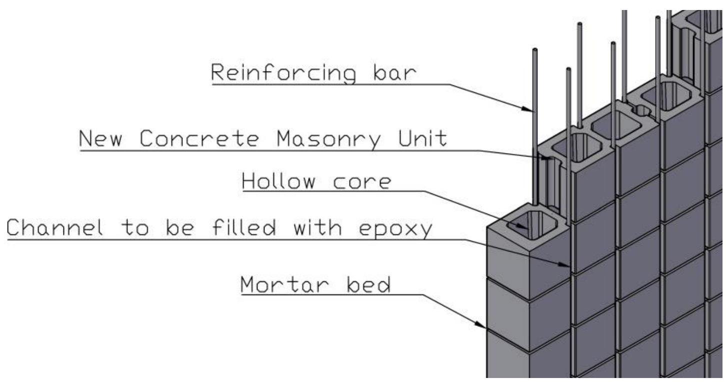

Figure 2: Reinforced SRCMU construction

The timely implementation of this system will be contingent on engineers being able to use current codes to design structures with this new masonry system. To this end, testing of small SRCMU specimens was performed under two of the main loading conditions for which conventional hollow masonry systems are commonly used; that is to say, axial compression and out of plane flexure.

2 PHYSICAL TESTING

To justify the use of the Canadian masonry design code, CSA S304.1, for the design of masonry systems utilizing the SRCMU block with NSM reinforcement, the behaviour of the three elements of the reinforced SRCMU system were studies individually, and as a system. That is to say, the compression behaviour of the unreinforced masonry assembly, the bonding characteristics of the adhesive used to bond the reinforcing bars to the assembly, and the tensile characteristics of the reinforcing bars were determined in addition to the flexural behaviour of a reinforced SRCMU assembly. Since the SRCMU is not yet available commercially, the blocks used for this testing series had to be produced in the laboratory.

Commercially produced hollow CMUs are manufactured using a heavy vibrating compactor in order to consolidate an extremely lean concrete mix (with low cement content and a very low water to cement ratio) into heavy steel moulds. This allows for very rapid block production since the consolidated lean concrete mix can be immediately unmoulded after casting without significant deformation. For block production in laboratory, the use of a similar concrete mix is desirable, however the specialized casting machinery was not available. A concrete mix design obtained from a local concrete block manufacturer was therefore modified in order to allow casting and consolidation without vibratory compaction. The nominal size of all blocks in this section was 200mm (190mmX190mmX390mm). The nominal strength for the commercial blocks as well as the laboratory produced concrete was 15MPa.

2.1 Block validation

To ensure that the blocks cast in laboratory maintained similar compressive properties to commercially produced hollow CMUs, the properties of unreinforced masonry prisms constructed using commercially available CMUs (CCMU) as well as laboratory-cast Conventional CMUs (LCCMU) and laboratory-cast SRCMUs were compared. Three CCMU prisms, three LCCMU prisms, and five SRCMU prisms were cast on the same day by a professional mason and allowed to cure for 28 days at room temperature and 100% relative humidity. Each specimen was constructed to be 4-blocks high and have three mortar joints.

After curing, the specimens were tested to failure under axial compression following the procedures outlined in CSA S304.1 Annex D. The mortar strength and modulus of elasticity of the prisms, in addition to the ultimate strength of each prism was recorded.

2.1.1 Prism test results

The maximum stress sustained by each specimen is shown in Figure 3. The average strength of the prisms was 22.5MPa, no clear outliers exist in this set, and the coefficient of variation for the entire data set is 9.8%, which is well within the 15% mark suggested in S304.1 Annex D clause 3.2.3 for small numbers of replicates.

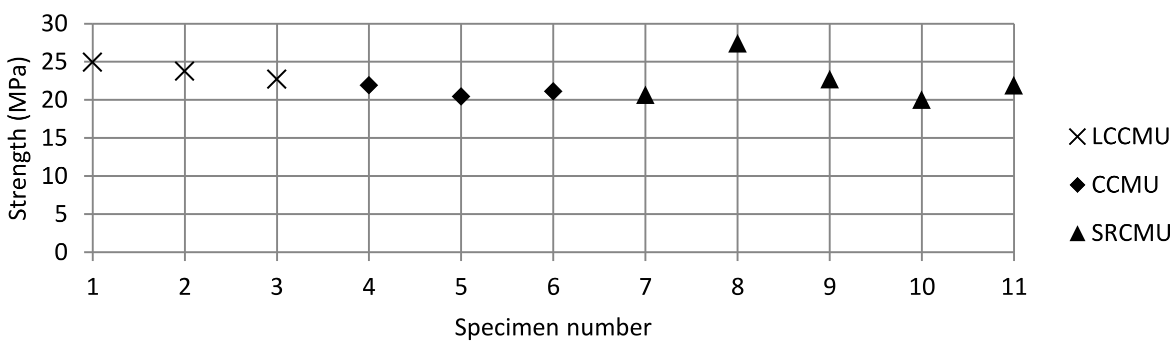

Figure 3: Prism strength distribution

The average strength of the SRCMU prisms was also 22.5MPa and the standard deviation for those five prisms was 2.92MPa. It should be noted that all the values of strength of the remaining prisms fall within one standard deviation from the mean of the SRCMU prisms. This does not give cause to reject the null hypothesis that there is no difference between the average strength of masonry prisms built with commercially available CMUs and laboratory-cast SRCMUs or between LCCMU and the SRCMUs.

The modulus of elasticity (E) of each prism was calculated by averaging the secant moduli of the stress-strain curves collected by each sensor used for that prism between the values of 5% and 33% of the measured prism strength as suggested by S304.1 Annex D clause 4.6. Table 1 compares the average E for each prism group calculated from S304.1 Annex D to the E value estimated using S304.1 clause 6.5.2 based on the average strength of the prisms in that group. Again the sample size is small, however results do not suggest that the null hypothesis, that the average E for all three sets of prisms are the same, should be abandoned.

Table 1: Elastic Modulus of Masonry prisms

Prism code	Block cross section	Block casting method	Measured average E from CSA S304.1 D.4.6 [GPa]	Estimated E from CSA S304.1 6.5.2 [GPa]
LCCMU	Conventional	Laboratory	19.3	20.2
SRCMU	SRCMU	Laboratory	18.1	19.1
CCMU	Conventional	Commercial	18.3	18.0

2.1.2 Prism mode of failure analysis

The modes of failure of all three groups of prisms were consistent throughout this experimental procedure. These modes of failure also conform to failure modes described by other research groups (e.g. Mohamad et al. 2007, Barbosa et al. 2010). In all cases, the prisms collapsed following splitting of the blocks vertically within the web, and some level of spalling of the face shells.

2.2 Bonding characteristics of the bar-adhesive-masonry interfaces

In order to determine the bonding characteristics of the reinforcing bars to the masonry assembly, pull-out specimens were constructed and tested to failure.

2.2.1 Pull-out test setup

The configuration of the pull-out specimens is shown in Figure 4; these specimens consisted of a three-high SRCMU prism into which a reinforcing bar was anchored. During the test, the specimen was restrained against the lower crosshead and the free end of the reinforcing bar was locked into the upper cross-head of the loading frame as shown in Figure 5. The specimen was then loaded under displacement control by exerting tension on the free end of the reinforcing bar until the bar either ruptured or mechanically separated from the rest of the specimen (pull-out). The loading rate was set in order to allow the test duration of approximately 15 minutes.

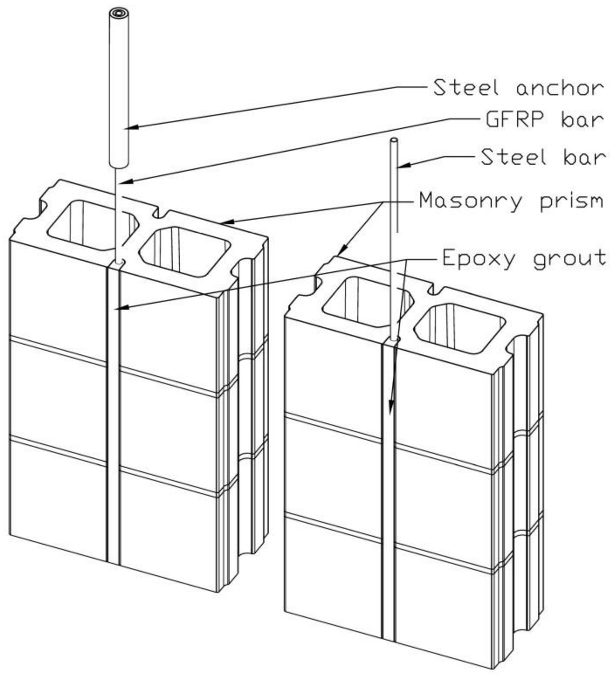

Figure 4: Pull-out specimens

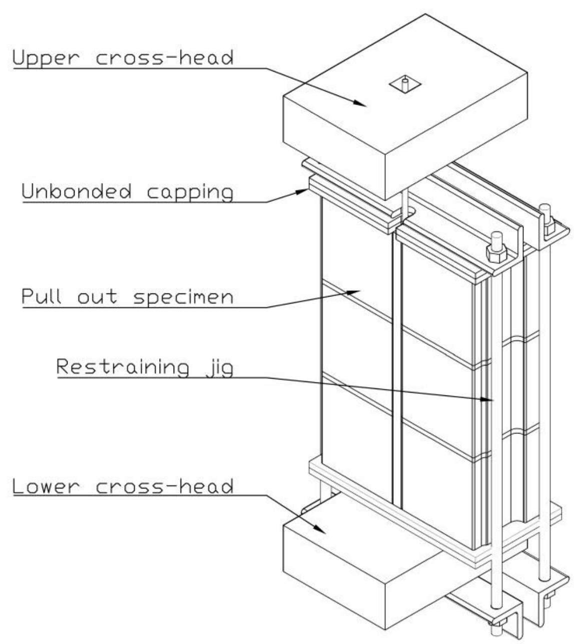

Figure 5: Pull-out test set-up

2.2.2 Pull-out specimen preparation

Six (6) three-high masonry prisms were constructed using SRCMUs. The mortar preparation and prism construction was performed by a professional mason. The prisms were cured for 14 days under a polymer sheet at room temperature, then allowed to dry for an additional 14 days. Seven days prior to testing, a reinforcing bar was anchored into one of the central face grooves in each of the masonry prisms. Anchoring was achieved by filling the SRCMU groove with commercial dowelling epoxy as per the manufacturer's instructions, then inserting the reinforcing bar laterally from the face before troweling the surface of the epoxy. A plywood jig ensured that the centroid of the bars were located 20mm from the face of each prism. Three of the reinforcing bars were grade 400 10M steel rebar, and three of the reinforcing bars were 9.5mm sand-coated Glass Fibre Reinforced Polymer (GFRP). The GFRP bars were previously anchored on one end into a steel tube as per ASTM D7205 in order to allow the bar to be properly gripped within the crosshead.

Strain gauges were installed on each bar at 100mm intervals within the embedded portion. The stress-strain relationship as well as the ultimate tensile strength of the steel and GFRP bars had been previously determined using ASTM A615 and D7205, respectively. The average ultimate strength was 61.2kN for the steel bars and 56.4kN for the GFRP bars.

2.2.3 Pull-out test results

The mode of failure and ultimate load of each of the pull-out specimens is shown in Table 2. The average ultimate load for the steel reinforced specimens was 59.0kN, which is 22% higher than the average of the GFRP reinforced specimens, despite the ultimate strength of the reinforcing steel being only 8.5% higher than the ultimate strength of the GFRP.

Table 2: Pull-out test results

Specimen ID	Reinforcing bar material	Ultimate load (kN)	Fraction of average ultimate bar strength	Failure Mode
P1	grade 400 10M steel	59.8	.98	Bar Rupture
P2	grade 400 10M steel	57.7	.94	Pull-out
P3	grade 400 10M steel	59.5	.97	Bar Rupture
P4	9.5mm GFRP	52.6	.93	Bar Rupture
P5	9.5mm GFRP	47.3	.84	Pull-out
P6	9.5mm GFRP	46.5	.82	Pull-out

Note that the required development length for 10M rebar within a conventional masonry system under normal conditions (grout strength of 10MPa), at 570mm (from CSA S304.1 clause 12.4.2.4), is only 3.5% shorter than the anchored length for this test set-up.

Figure 6 shows the average distribution of tensile forces along the length of the steel and GFRP reinforcing bars at 40kN (74.2% of the average ultimate load). At this level of loading, the behaviour of the steel and GFRP reinforcing bars can be clearly distinguished; the stiffer steel bars distribute the load more evenly over the length of the bar with lower stresses near the loaded end of the bar, and higher stresses near the base of the masonry specimen when compared to the GFRP specimens.

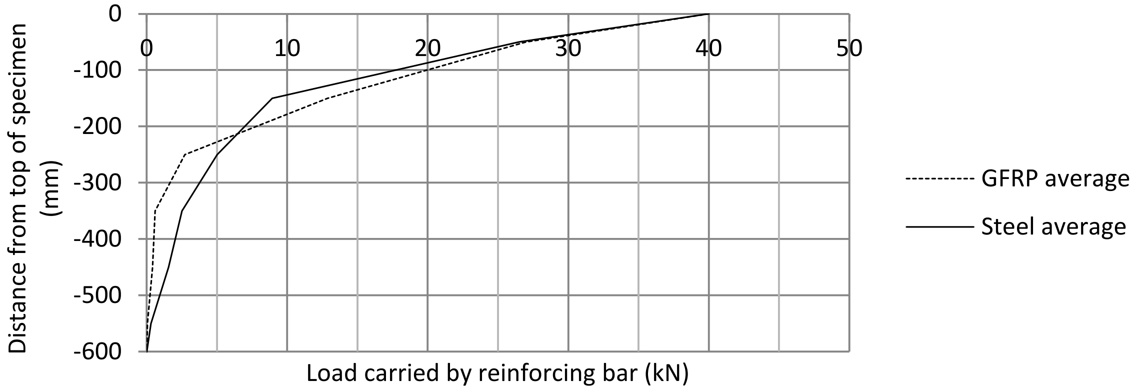

Figure 6: Pull-out stress distribution

2.3 Flexural Testing

In order to estimate the performance of the reinforced SRCMU system under flexural conditions, doubly reinforced beams were constructed and tested in flexure.

2.3.1 Flexural test set-up

The configuration of the flexural specimens is shown in Figure 7; each specimen was composed of a 6-high SRCMU prism, both central face grooves being fitted with an epoxy-grouted reinforcing bar. The specimens were loaded at third points along their height starting from the middle of the bottom block up to the middle of the top block. Testing occurred in the upright position to minimize the potential for damage from handling the specimens. Loading was performed under displacement control at a steady rate allowing for a test duration of approximately 15 minutes.

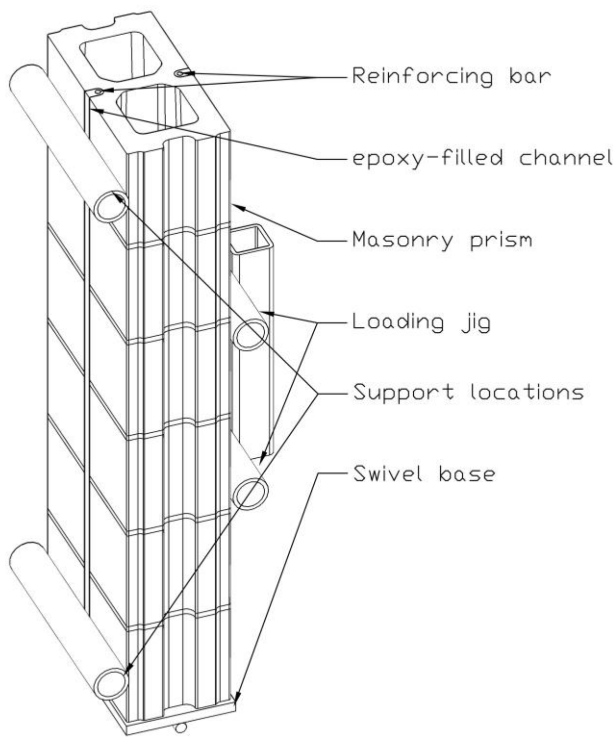

Figure 7: Flexural loading test set-up

2.3.2 Flexural specimen preparation

Six (6) six-high masonry prisms were constructed from SRCMUs. The mortar preparation and construction was performed by a professional mason. The prisms were cured for 14 days under a polymer sheet at room temperature, then allowed to dry for an additional 14 days. Seven days prior to testing, reinforcing bars were anchored into the central groove of both external faces of each specimen. Anchoring was achieved by filling the SRCMU groove with commercial dowelling epoxy as per the manufacturer's instructions, then inserting the reinforcing bar laterally from the face before troweling the surface of the epoxy smooth. Three of the specimens were reinforced using grade 400 10M steel rebar, the remaining specimens were reinforced using 9.5mm sand-coated GFRP bars.

2.3.3 Flexural test results

The maximum moment developed by each specimen, and the associated deflection at maximum load are shown in Table 3. The average moment resistance of the steel reinforced and GFRP reinforced specimens was 7.15kNm and 5.89kNm, respectively. The average deflection at the maximum load for the steel reinforced and GFRP reinforced specimens was 3.79mm and 7.0mm, respectively.

Table 3: Flexural test results

Specimen ID	Reinforcing bar material	Maximum developed moment (kNm)	Average deflection at maximum load at mid-span (mm)
F1	grade 400 10M steel	6.81	2.33
F2	grade 400 10M steel	6.96	2.57
F3	grade 400 10M steel	8.30	6.45
F4	9.5mm GFRP	6.41	9.10
F5	9.5mm GFRP	5.48	4.70
F6	9.5mm GFRP	5.77	7.30

The mode of failure in all cases was a combination of sliding shear and diagonal tension shear in the second block from the top and/or bottom from each specimen, as shown in Figure 8, indicating that the full flexural strength of the section was not developed in any case. It is interesting to note, however, that the theoretical maximum flexural resistance (reinforcing bar capacity multiplied by the distance from the centroid of the tension reinforcement to the extreme compression fibre) for the steel-reinforced specimens and GFRP reinforced specimens is 10.4 kNm and 9.5kNm, respectively.

Figure 8: Typical combined sliding shear and diagonal tension shear failure (Specimen F1)

Deflection at mid-span of the reinforced specimens was recorded at either end of the specimen near the tension face of the specimens. The average of the mid-span deflection was compared to the anticipated deflection based on S304.1 clause 10.12.2 (Transformed cross-section). Figure 9 shows the relationship between load and displacement at mid-span for a sample steel-reinforced specimen and a sample GFRP reinforced specimen. It also shows the anticipated load-displacement curves based on S304.1. For the GFRP-reinforced specimen, two analytical curves are shown; the CSA S304.1 model with bonded epoxy takes into account the tensile stiffness of the dowelling epoxy within the transformed section, the other model does not.

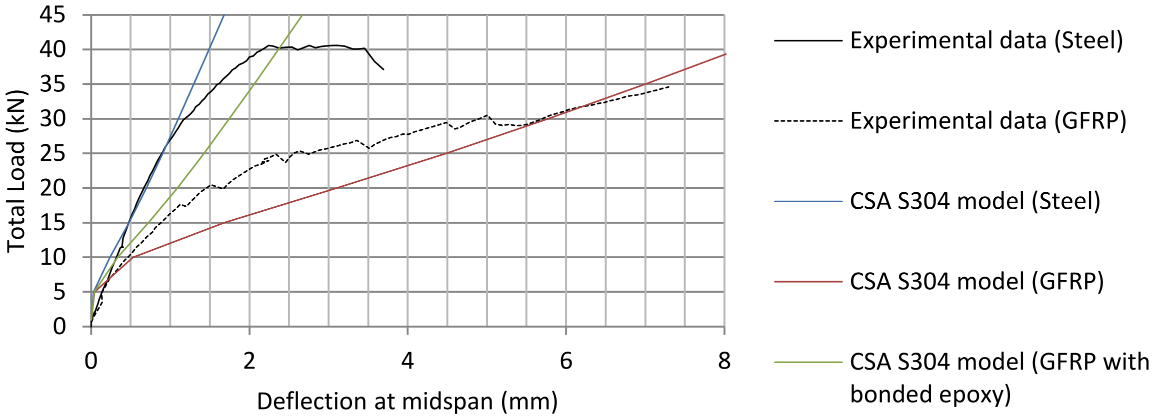

Figure 9: Load-deflection diagram

3 Expected system performance

CSA S304.1 describes standards for the design and construction of hollow concrete masonry structures in Canada. The authors offer the following justification for the application of this code to design scenarios utilising the SRCMU. :

- No change in the composition of the masonry block or mortar is proposed
- The addition of a narrow channel on the outer vertical surface of hollow concrete blocks does not constitute a significant change in cross sectional properties of the blocks

To illustrate the expected performance of masonry wall systems using SRCMUS, four theoretical configurations were analyzed. The configurations, listed in Table 4, are all made up of 200mm nominal (190mmX190mmX390mm) hollow concrete blocks. 200mm blocks were selected because they are the most commonly used type of block. The various configurations differ in the spacing of reinforcement and number of cores grouted. Figure 9 shows side-by-side comparisons of the interaction diagram (compressive strength (P_r) vs. flexural strength (M_r)) for all four configurations.

To develop the curves in Figure 10 for configurations employing the conventional masonry blocks, the reinforcing steel was assumed to be located along the centre line of the cross-section of the wall (d=95mm). For the analysis of the SRCMU configurations, the reinforcing steel was placed at a depth of 20mm from either surface of the wall (d=170mm).

Table 4: Wall-type designation meaning

Designation	Block type	Reinforcement ratio	Grouting (grouted cores per metre)
SR/PG	SRCMU	0.26%	2
C/PG	Conventional	0.26%	2
SR/UG	SRCMU	0.47%	None
C/UG	Conventional	0.0%	None

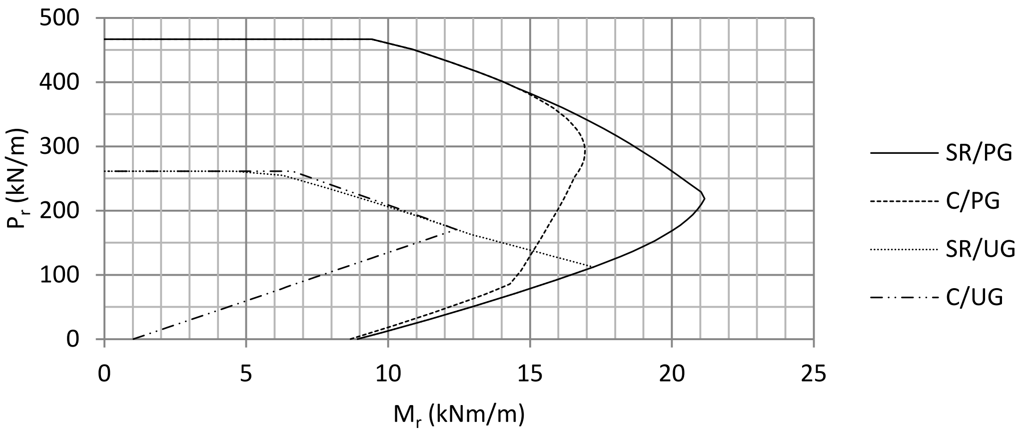

Figure 10: Conventional masonry vs. SRCMU interaction diagram

4 Analysis

For the experimental procedures described herein, the laboratory cast concrete blocks were shown to have similar compressive properties to that of commercially produced CMUs. The surface texture of these blocks, however, did differ significantly. Because the type of concrete used to cast the blocks in laboratory was flowing (in contrast to the much leaner mixes used commercially), the outer surfaces of the laboratory cast blocks were much smoother than those of commercially produced units, resulting in a marked decrease in its capacity to bond with masonry mortar. However, this property appeared to have had little impact in this study since the compressive and shear properties of the blocks themselves governed the behaviour of the specimens in most cases. In fact, the tensile capacity of masonry elements is neglected in most design situations. It is therefore reasonable to assume commercially-produced SRCMU assemblies would behave similarly to those described herein.

The results of the pull-out tests suggest that the yield strength of 10M steel rebar can be easily developed by anchoring it within and SRCMU system with dowelling epoxy utilizing the same development length as used for conventional masonry systems. In addition, despite the low stiffness of the GFRP bars, which caused much more cracking in the epoxy than for the steel bars, over 80% of the bar strength was developed in all cases. However some caution must be used in interpreting these results as the compression at the top of the pull-out specimens induced by the restraining jig may have enhanced the bond between the masonry and the epoxy. It is however telling that none of the flexural specimens failed by pull-out.

A closer look at the results from the flexural tests shows some important discrepancies in behaviour from that predicted by the CSA S304.1 model. At low flexural stress levels, the specimens are expected to show higher stiffness levels until cracking of the mortar joint occurs. The specimens tested did not exhibit this behaviour due to the poor bond between the blocks and mortar as discussed above. Furthermore, the CSA model accurately predicts the deflection for the steel reinforced specimens for moderate flexural stress levels but underestimates the deflection at stress levels approaching failure. This discrepancy is likely due to sliding shear deformation observed in all specimens at higher stress levels. The GFRP reinforced specimens exhibited much lower stiffness than the steel reinforced specimens. This is expected because of the lower stiffness of the GFRP reinforcing bars. At lower flexural stress levels, the doweling epoxy appears to have a large effect on the over-all stiffness of the specimens, however this effect disappears as more and larger cracks appear in the epoxy at higher stress levels.

It should be noted that in most construction applications the height to thickness ratio of masonry systems is much higher than that of the specimens tested herein. Larger height to thickness ratios often lead to much higher flexural stresses for similar out-of-plane shear conditions (because of the longer moment arm), this makes it unlikely for out-of-plane shear to govern the behaviour.

All these results indicate the current Canadian design codes may be sufficient for the implementation of the SRCMU system. The major advantage of this system is the reduction in over-all weight of reinforced masonry structures by allowing them to be completely hollow. Under conditions of low axial stress, where out-of-plane flexure governs the design, SRCMU masonry systems have a clear advantage over conventional systems.

5 Conclusions

Results obtained from the testing of physical specimens suggest that the assumptions necessary in order to design SRCMU systems using the current design code are correct since:

- SRCMU systems can be made up to 50% lighter than comparably reinforced conventional systems
- SRCMU systems have an increased flexural resistance of up to 30% over conventional systems with the same reinforcement ratio
- No statistically significant difference in behaviour was observed between the conventional masonry and SRCMU prisms under axial compression
- Bond strength above 80% of the rupture strength of steel and GFRP reinforcing bars were developed within a 590mm embedment length
- Flexural strength near that predicted using the CSA S304 design code were achieved with the steel-reinforced specimens despite shear failure of these specimens
- Load-deflection behaviour under flexural conditions closely matched that predicted using the CSA S304 design code

6 Acknowledgements

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