THE STRAIN EFFECT IN REINFORCED MASONRY STRUCTURES

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ABSTRACT

Both the CSA S304.1-04 and TMS 402-2011 masonry design codes provide shear design methods for masonry beams that do not account for some of the key parameters that are known to affect shear strength, such as the reinforcement ratio, horizontal steel distributed along the height of the web, and stiffness of reinforcement (E). These parameters have been studied through a series of tests on large, shear-critical reinforced masonry beams. The research described in this paper reports the results of these tests. It is shown that TMS 402 code overestimates the shear strength of the RCM beams by a very wide range. The CSA S304.1-04, while it does not account for the studied parameters, exhibited less overestimation. This is due to that fact that the CSA S304.1-04 accounts for the size effect (the effective depth), while the TMS 402-2011 does not. Surprisingly, the results showed the General Method of shear design from the CSA A23.3-04 code for reinforced concrete can give safe, accurate predictions for the RCM beams tested in this research. This code accounts for both strain effect (the three parameters studied in this paper) and the size effect.

KEYWORDS: reinforced masonry beams, reinforcement ratio, size effect, shear, strain effect

INTRODUCTION

Reinforced concrete masonry (RCM) beams are common structural members in masonry buildings. RCM beams are used to span various openings, such as doors, windows and passages. Also, RCM beams are often used as part of masonry walls. Such beams are called coupling beams because they "couple" the shear walls or piers [1].

The shear strength very often determines the ultimate load-carrying capacity of a masonry beam, and thus shear strength is a critical property of masonry. RCM beams without web reinforcement can experience brittle failure with very little deformation to warn of impending failure [2]. Furthermore, as it is difficult to provide web reinforcement in masonry, it is preferable to design RCM beams with as little stirrups as possible [3]. As such, the design provisions used to determine shear strength of RCM beams without web reinforcement must be accurate, safe, and rational. In addition, although the similarity of the reinforced masonry and reinforced masonry have been verified experimentally by many studies [4-6], the well recognized "aggregate interlocking mechanism" is still not considered in masonry design codes.

This paper presents the experimental results of six full scale shear critical masonry beams which is a part of a more extensive ongoing experimental program on shear behaviour of reinforced masonry beams at Carleton University. The study of the shear behaviour of this six beams is intended to explore the effects of the main reinforcement ratio , the existence of horizontal steel distributed along the height of the web and longitudinal bar stiffness (E) on shear strength reinforced masonry beams. Based on the results of these beams the predictive capability of the CSA S304.1-04[7] and TMS 402- 2011[8] masonry design code will be evaluated and compared with the general method of shear design from the CSA A23.3-04[9] code for reinforced concrete.

SHEAR DESIGN METHODS FOR RCM BEAMS

TMS 402- 2011

Nominal masonry shear strength provided by the masonry is computed using Equation [1].

$$V_m = \left[4.0 - 1.75(M_u/V_u d_v)\right] A_n \sqrt{f'_m} \qquad (1)$$

Where:
$M_u/V_u d_v$ = a/d for point-loaded beams, and need not be greater than 1.0,
A_n = net cross-sectional area of beam (taken as b_w h),
b_w = the width of the beam,
h = hight of the beam, and
f'_m = masonry compressive strength.

For slender beams with a/d > 1.0 the formula can be simplified to the following (in MPa units):

$$v_m = V_m/b_w h = 0.187\sqrt{f'_m} \qquad (2)$$

CSA S304.1-04

The nominal shear resistance of grouted, normal density RCM beams is calculated as follows:

$$v_m = 0.16\sqrt{f'_m}\left(1.0 - \frac{d - 400}{2000}\right) \qquad (3)$$

Where v_m should neither be taken greater than $0.16(f'_m)^{0.5}$ nor less than $0.07\,(f'_m)^{0.5}$.

CSA A23.3-04 General Method of Shear Design for Reinforced Concrete Beams

While most shear design equations for reinforced concrete beams are empirically derived from curve fitting to experimental test data, the CSA A23.3-2004 general shear design method is derived from the simplified modified compression field theory (SMCFT) [10] which describes the behaviour of reinforced concrete subjected to shear. In the general method, the shear strength is calculated using Equation [4]:

$$V_c = \beta\sqrt{f'_c}\, b_w d_v \qquad (4)$$

The term β in Equation [4] is calculated by an expression consisting of a strain effect term and a size effect term:

$$\beta = \frac{0.4}{(1+1500\varepsilon_X)} \cdot \frac{1300}{(1000+s_{xe})} = \text{(strain effect term)} . \text{(size effect term)} \qquad (5)$$

β is a function of 1) the longitudinal strain at the mid-depth of the web, ε_x, 2) the crack spacing at the mid-depth of the web and 3) the maximum coarse aggregate size, a_g.

The general method assumes that the longitudinal strain at the mid-depth of a beam web, ε_x, is conservatively equal to one-half the strain in the longitudinal tensile reinforcing steel. For a section which is neither prestressed nor subjected to axial loads, ε_x is calculated simply as:

$$\varepsilon_X = \frac{M_f/d_v + V_f}{2E_s A_s} \qquad (6)$$

The effect of the crack spacing at the beam mid-depth is accounted for by the crack spacing parameter, s_x, which is equal to the flexural lever arm d_v (d_v=0.9d or 0.72h, whichever is greater). It can be also taken as the maximum distance between layers of distributed longitudinal reinforcement of an area of 0.003($b_w s_x$) per layer. The term s_{xe} is an "equivalent crack spacing factor" that was developed to model the effects of different maximum aggregate size on shear strength by modifying the crack spacing parameter:

$$s_{xe} = \frac{35\, s_x}{15+a_g} \geq 0.85\, s_x \qquad (7)$$

For members with very low reinforcement ratios or with low modulus reinforcement, Hoult et al. [11] developed a more accurate version of Eq. (8)

$$\beta = \frac{0.3}{0.5+(0.15+1000\varepsilon_X)^{0.7}} \cdot \frac{1300}{(1000+s_{xe})} \qquad (8)$$

In the case of RCM beams, it is suggested that V_m and f'_m can replace V_c and f'_c, respectively. More focused research is required to rationally account for the effect of aggregate size on shear strength of RCM, but for the current study, it was found that using a_g=5mm for members with fine grout produced appropriate results. To calculate the shear strength using the general method, Equations [4] through [7] must be solved by a quadratic equation or using a simple iterative approach. For simply-supported point-loaded beams, the critical section can be taken at a distance d from the location of maximum moment (at the applied load). This conservative assumption permits the effect of the moment on crack widths to be accounted for in a rational way, and results in a more accurate shear strength prediction.

Based on the fact that masonry compressive strength has only a minor effect on the shear strength of beams with a/d ≥ 2.5, CSA S304.1-04 and TMS 402 codes relate masonry shear strength to the square root of the compressive strength [2]. Both codes disregard the effects of longitudinal reinforcement. Unlike TMS 402, CSA S304.1 accounts for the size effect (when the depth of the section exceeds 400 mm). The general method adopted in the CSA A23.3-04 code is the only code that accounts for all the key parameters affecting shear strength.

EXPERIMENTAL PROGRAM

Six reinforced masonry beams without stirrups were constructed and loaded to failure in the John Adjeleian Laboratory at Carleton University. The purpose of theses tests was to investigate the ability of SMCFT-based shear design methods to predict the hear strength of RCM beams. All beams were of nominal 190mm width, height 990mm height, and 6400mm length. The beams consist of 5 courses: bond beam units in the first course and depressed web units in the top courses. The experiments are described in Fig. 1. The average block unit strength was 40 MPa (units were tested as face shell bedding). Type S mortar was used through the program. The mix proportion by weight was 1:0.21:3.53 for Portland cement: lime: sand respectively. The average mortar strength was 26 MPa. A fine grout mix was used to grout the beams. The average grout strength was 19 MPa. In order to obtain the compressive strength of the masonry beams, 4-unit high prisms as control specimens were constructed from the same material at the time of construction. All the beams were built in the laboratory experienced masons. The following construction procedure was used: first, the lower course was laid, next the reinforcing cage was placed then the top courses were laid with face shell bedding. To facilitate moving, steel lifting hooks were cast in the end portions that lie outside the test region. After the units were laid, the ends of the beams and the sides f the prisms were sealed with plywood, and the specimens were then fully grouted. The grout was a fine grout with a slump of 220mm and was supplied by a local ready-mix company. The grout was carefully vibrated with no segregation observed. The grout was well consolidated and fully surrounded the reinforcement (this was verified after testing). The top surfaces of the specimens were smoothly levelled and covered with plastic sheeting for seven days at room temperature.

Three of the specimens (SM1, SM1D (duplicate of SM1) and SM6) were reinforced in flexure with two 30M conventional steel bars, with Sm6 additionally reinforced with one M15 longitudinal steel bar distributed in each of the three middle courses. Specimen SM2 was reinforced with four 30M conventional steel bars while SM4 was reinforced with a 26 mm Dywidag bar (yield strength of 860 MPa). SM5, on the other hand, was reinforced with a one #8 GFRP bar (with a cross-sectional area of 507mm^2, a Young's modulus of 51,900MPa and an ultimate strength of 675MPa). All the beams were reinforced with two 20M bars in the upper course as compression reinforcement.

The main reinforcing rebars were instrumented with multiple strain gauges at locations indicated in Fig. 1 and were supported on individual plastic chairs. The locations of these chairs were staggered within the forms to avoid forming vertical planes of weakness where cracking could preferentially initiate. The rebars were firmly tied together to maintain the correct bar spacing, and in turn firmly tied at the ends to prevent shifting during casting. Prior to testing, the specimens were painted with diluted white latex paint to make it easy to see cracks when they formed during testing. The specimens were tested in three-point bending under monotonic load until shear failure occurred (see Fig. 2). Plaster-of-Paris was placed between the steel loading/support plates and the beam to ensure uniform bearing stresses. Deflection was measured at different locations, and longitudinal, transverse and shear strains were measured on the side faces of each specimen at the six locations (three sets in each shear span) using side-mounted LVDTs (see Fig. 1). Loading was paused regularly during the test at "load stages" to permit detailed investigation of the cracking patterns, including marking cracks with a felt-tip pen, measuring crack widths to a precision of 0.02mm and photographing the specimens. At load stages, the load was reduced by 10% for safety. When shear failure occurred on one side of the beam (first side) that side was clamped using external steel straps, and the beam was reloaded to failure on the opposite side (repeat side).

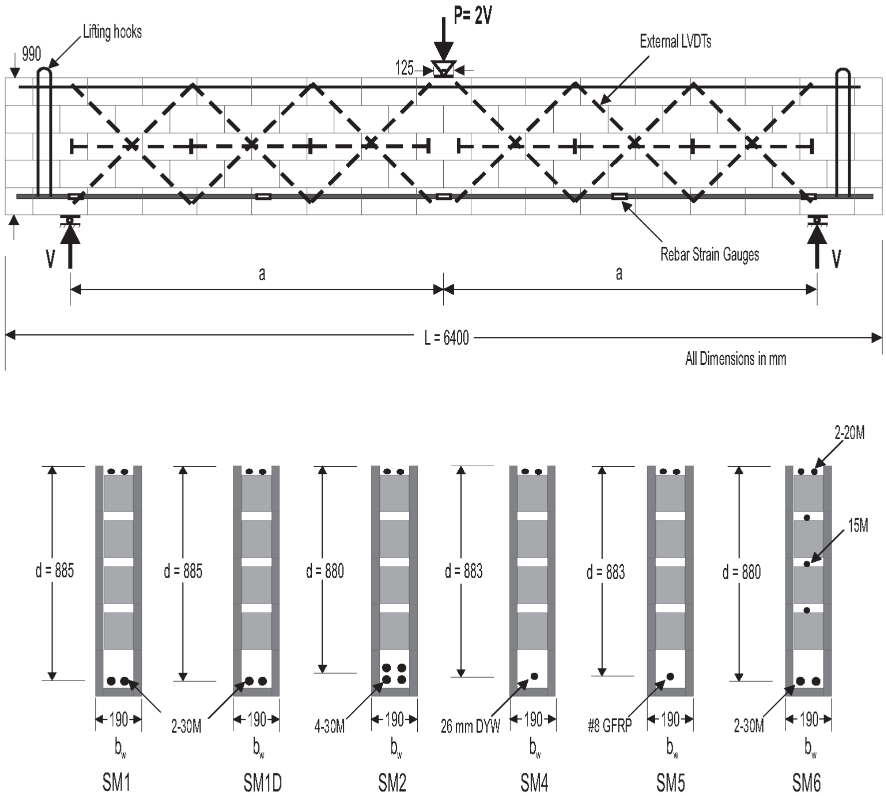


Figure 1: Design of Test Specimens

Figure 2: Test Set Up

RESULTS AND DISCUSSION

Failure Mechanisms and Load-Deflection Behaviour

All specimens exhibited an obvious diagonal shear failure (breakdown of beam action) before reaching their flexural capacity for the first failed sides. Interestingly, repeated sides exhibited formation of a shallow strut extending from the point load to the support. Thus a combination of beam and strut-and-tie actions led to shear failure. Only the results and behaviour of the first failed sides will be reported in this paper. Figure 3 shows the failures patterns of the specimens.

The failures for SM1D, SM2, and SM4 were brittle due to the breakdown of beam action mechanism, while SM5 (reinforced with GFRP) exhibited large deflections prior to failure. No post-peak response was noticed for these beams. SM1 and SM6, on the hand, while failed suddenly in shear, they also exhibited some post-peak response. The load vs. mid-span deflection curves are presented in Fig. 4. It can be seen that the maximum loads reached in the tested beams increased with increasing ρ, increasing reinforcement stiffness and the existence of longitudinal distributed steel.

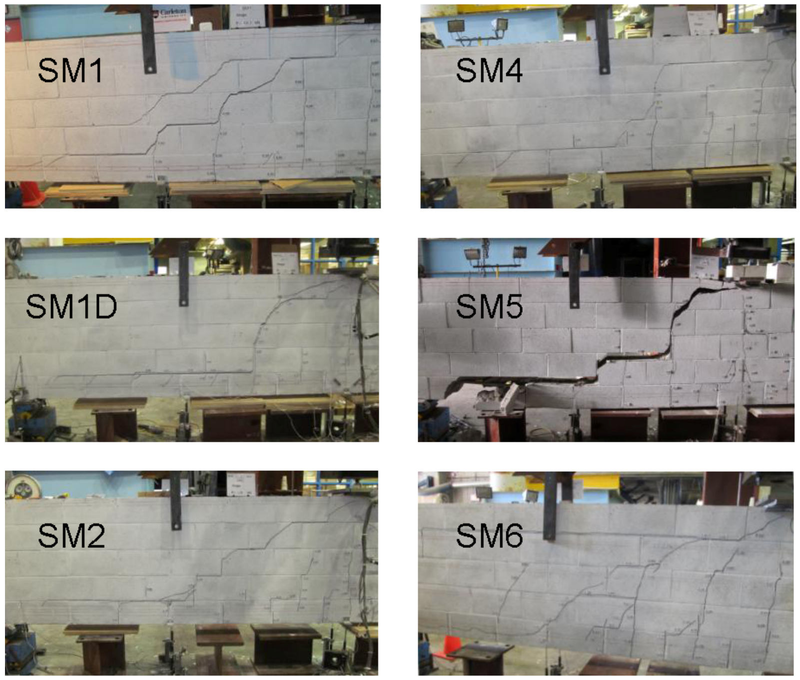


Figure 3: Failure Patterns of the Specimens (for the First Failure Sides)

The progression of shear failure in SM1D (with ρ =0.83 %) will be explained as an example. At early stages of loading, flexural cracks began to appear in the mortar head joints in the bottom course at the mid span of the beam under the load. As the load increased, flexural cracks started to appear in the shear zones. Two of these flexural cracks started to go up crossing the block in the second course reaching the head joints in the third course and forming masonry tooth between the vertical cracks. In one of the shear zones (first failed side), one of the tooth vertical
The widening of the diagonal cracks caused a noticeable increase in the shear strains measured by the side-mounted LVDTs (see Fig. 5). Shortly after the peak load was reached, the crack propagated towards the loading point and backwards towards the support and due to dowel action a horizontal crack formed along the length of the reinforcement. The horizontal cracking at the level of the reinforcement was a post-peak phenomenon.

SM1 exhibited similar cracking behaviour with two teeth forming and more post peak cracks observed. SM2 (with ρ =1.67 %) exhibited similar behaviour with the tooth formed at the second course (lower height) and further from the point load with less post peak cracks. SM4 (ρ =0.32 %) had the tooth vertical crack propagates towards the point load at high level (forth course) and exhibited fewer post peak cracks. SM6 (reinforced with distributed steel) witnessed formation of four masonry teeth and less crack widths. Finally, SM5 (reinforced with GFRP) SM5 (reinforced with GFRP) exhibited a brittle behaviour up to 75% of the failure load, after which three actions were noticed and led to large deflections. These action are: the aggregate interlock action around the vertical part of the diagonal crack, the compression zone resistance due to the existence of the compression reinforcement in top course (despite the very shallow compression zone depth at this stage of loading), and the dowel action for main reinforcement. SM5 failed right after the break down of the aggregate interlocking action.

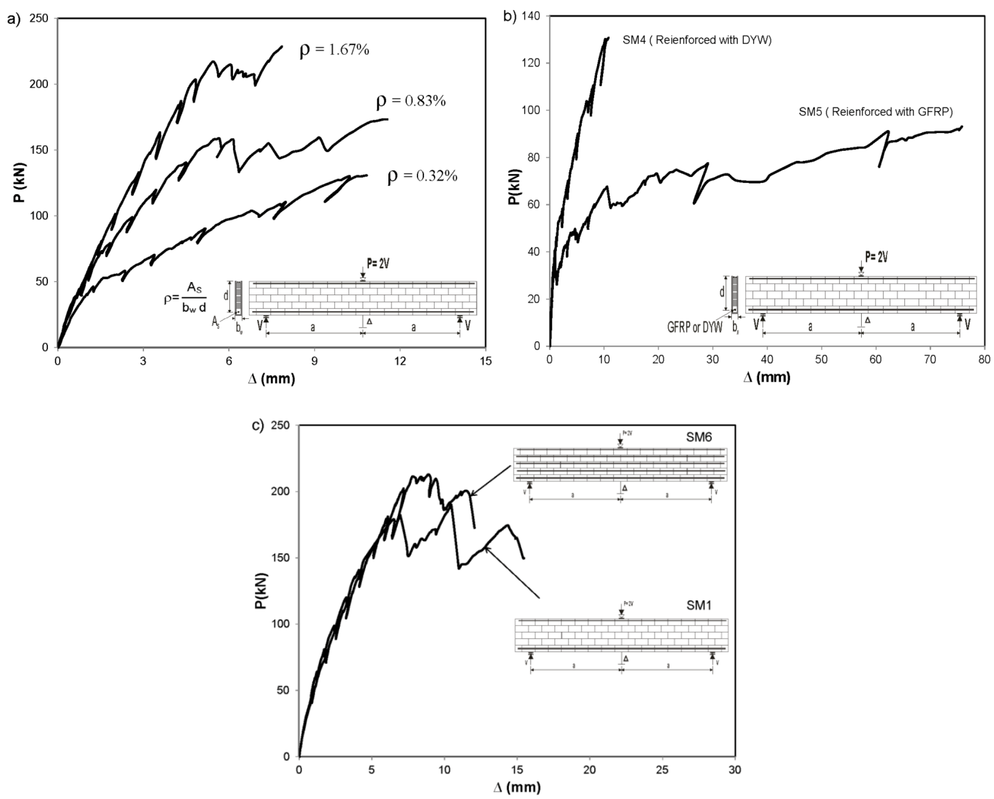
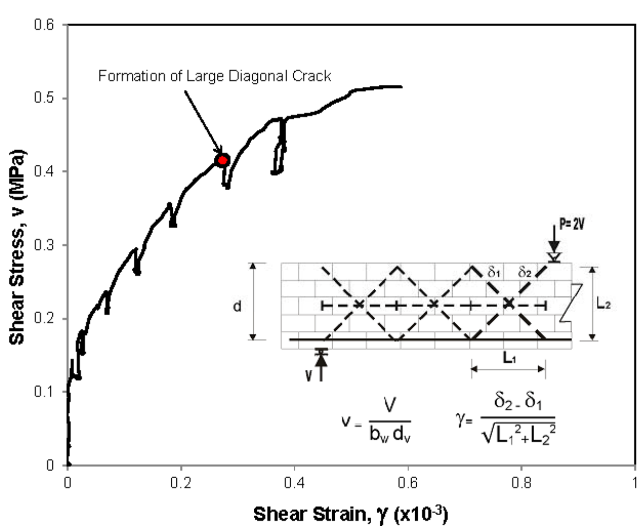


Figure 4: Load vs. Mid-Span Deflection Curves



$$v = \frac{V}{b_w d_v} \qquad \gamma = \frac{\delta_2 - \delta_1}{\sqrt{L_1^2 + L_2^2}}$$

Figure 5: Shear Strains vs. Shear Stresses Curve for SM1D

Predictions of Failure Shear Stress

Failure shear are summarized in Table 1. It is shown that the TMS 402 code extremely overestimates the shear strength of the RCM beams with average of V_{exp}/V_{pred} of 0.64 and COV of 29% (when SM5 is not included). The TMS 402 overestimation can be attributed to the three reasons: first the TMS 402 code dose not account for strain effect parameters, secondly it does not account for the size effect and lastly it uses height of the beam instead of the effective depth to find the shear strength. The CSA S304.1 code, while not accounting for strain effect parameters, exhibited less overestimation predictions. This is due to that fact that the tested beams are large and the CSA S304.1 code accounts for the size effect (the effective depth). However, the CSA S304.1 code exhibited the highest variation of 29% (when SM5 is not included). Surprisingly, both the general method and hoult et al. (intended to be used for reinforced concrete) can predict the variation in failure shear stresses of the RCM beams very well, with the Hoult et al. method being more accurate (a mean of V_{exp}/V_{pred} of 1.09) and giving the least variation (COV% of 12).

Table 1: Summary of Experimental Results

Beam Name	Geometry			Main Reinforcement			Crack Control Steel	Experimental Observation			CSA S304.1		TMS402		CSA A23.3 General Method		Hoult et. Al.	
	a (mm)	d (mm)	a/d	Type	As (mm^2)	ρ (%)		f'_m (MPa)	Pexp (kN)	V_{Exp} (kN) *	V_{Pred} (kN)	V_{Exp} / V_{Pred}	V_{Pred} (kN)	V_{Exp} / V_{Pred}	V_{Pred} (kN)	V_{Exp} / V_{Pred}	V_{Pred} (kN)	V_{Exp} / V_{Pred}
SM1	2700	885	3.05	S	1400	0.83		17.90	94.94	98.40	86.20	1.14	148.81	0.66	80.46	1.22	83.85	1.17
SM1D	2780	885	3.14	S	1400	0.83		20.00	86.66	90.20	91.14	0.99	157.31	0.57	82.87	1.09	86.77	1.04
SM2	2780	875	3.17	S	2800	1.67		19.10	114.32	117.78	88.64	1.33	153.74	0.77	98.22	1.20	99.48	1.18
SM4	2780	880	3.16	DYW	540	0.32		18.50	65.38	68.89	87.40	0.79	151.29	0.46	58.14	1.18	64.63	1.07
SM5	2780	880	3.16	FRP	507	0.30		19.50	46.56	50.09	89.78	0.56	155.33	0.32	32.80	1.53	41.17	1.22
SM6	2780	880	3.16	S	1400	0.83	15M@ 200	18.00	106.47	109.98	86.26	1.27	149.23	0.74	117.46	0.94	126.81	0.87
Mean**												1.01 (1.10)		0.59 (0.64)		1.19 (1.13)		1.09 (1.07)
STDV**												0.30 (0.22)		0.17 (0.13)		0.19 (1.12)		0.13 (0.13)
COV (%)*												29 (20)		29 (20)		16 (10)		12 (12)

* $V_{exp}=\frac{P_{exp}}{2}+V_{@d\ from\ point\ load}$ (self weight)

** values in brackets do not include results of SM5

Failure shear stresses for specimens SM1, SM1D, SM2 and SM4 are plotted in Fig. 6 versus ρ. It can be seen that the general method and the Hoult et al. method can accurately and safely capture the variation in failure shear stresses of the RCM beams. The accuracy of the General Method in predicting the shear strength of RCM beams can be attributed to its rationality. The term β in Eq. (4) or Eq. (8) is a factor that describes the ability of cracked concrete to transfer shear stress by aggregate interlock. Thus, wider cracks precipitate shear failure at a lower shear stress due to reduced aggregate interlock capacity. Higher β values refer to higher aggregate interlock capacity. Given that the aggregate interlock capacity of a crack is inversely related to the width of that crack [12], it can be concluded that any action that increases the longitudinal strain in a member (such as increase in the main reinforcement ratio) will reduce the shear strength. Figure 7 shows that at the same shear stress, the crack widths at the reinforcement level or mid depth of beam are less in SM2 and SM6 than those in SM1D.

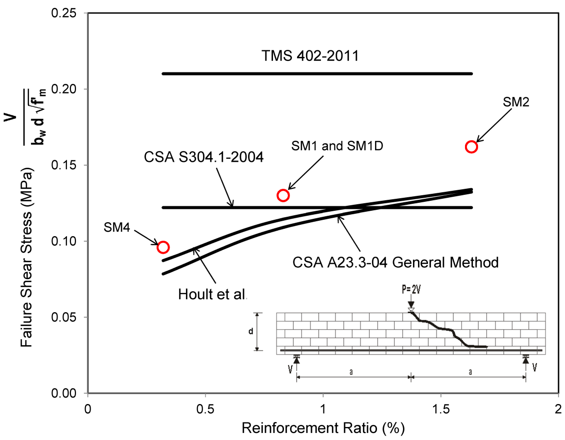


Figure 6: Effect of Reinforcement Ration on Failure Shear Stress

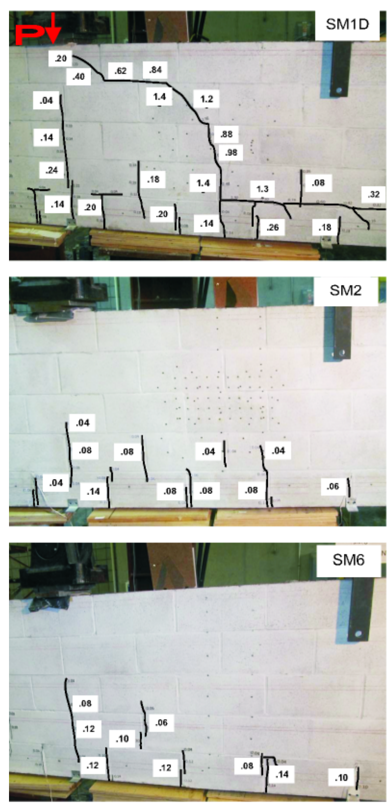


Figure 7: Crack patterns and widths at v= 0.48 MPa for SM1, SM2 and SM6
(Crack digitally enhanced for clarity)

SM4 and SM5 had the same reinforcement ratio but different longitudinal bar stiffness (E). Figure 7 shows that the crack widths at the reinforcement level or mid depth of beams are greater in SM5 than those in SM4 at the same shear stress. This can be attributed to the increase in the longitudinal strain in SM5 due to the use of lower stiffness reinforcement (GFRP rebar). This observation is proven by the values of longitudinal strain measured by in the first set of external LVDTs in the first failed side for SM5 and SM4.

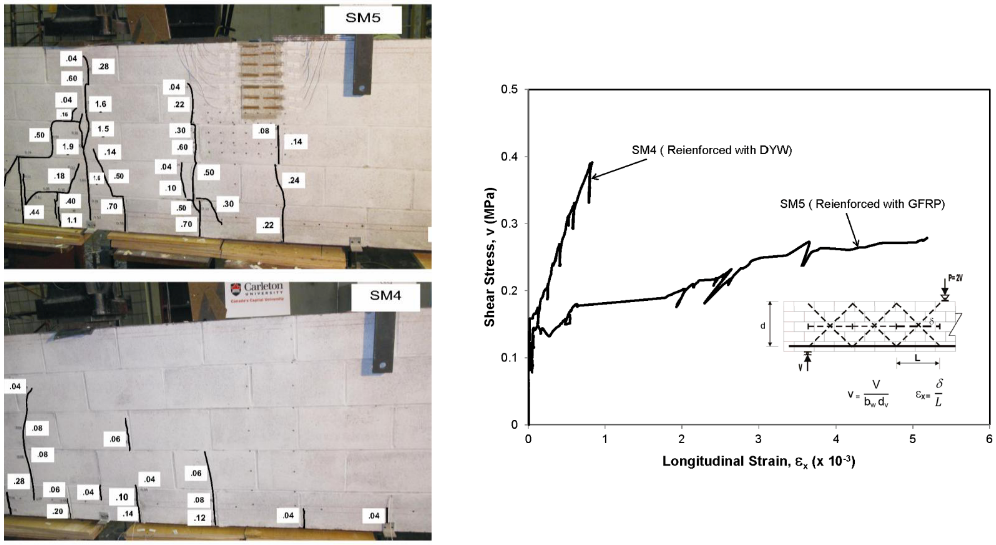


Figure 8: Crack patterns and widths at v= 0.21 MPa and Longitudinal Strain (ε_x) curves for SM4 and SM5 (crack digitally enhanced for clarity)

CONCLUDING REMARKS

A masonry design code should provide guidance to engineers about the effect of different design choices. The use of different reinforcement ratios, the use of crack control steel and building large sections are examples for such choices. TMS 402 is not sensitive to these options and can lead to serious overestimation of shear strength. The CSA S304.1 accounts for size effect but disregards other key factors affecting shear strength of RCM. Both codes can be improved to accurately predict the effect of different design choices. The general method adopted by CSA A23.3-04 (which account for all the key factors affecting shear strength) can serve as the base for this improvement.

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Appendix (Sample of Calculations for CSA A23.3-04 General Method)

Specimen SM1D, a=2780mm, d=885mm, b_w=190mm, f'm=20 MPa

Crack spacing parameter(s_x) $= 0.9 * 885 = 796.5 \geq 0.72 * 990 = 712.8$

Equivalent crack spacing factor $= s_{xe} = \frac{35*796.5}{15+5} = 1393.875 \geq 0.85 * 796.5 = 677$

Size effect term $= \frac{1300}{(1000+1393.875)} = 0.543$

<u>Self weight calculation</u>
Self weight = 2100 kN/m^3 x0.99x0.19 = 3.95 kN/m
Shear force at critical section = 3.95 x 0.885 = 3.5 kN
Moment at critical section = 12.85 kN.m

Guess ε_x= 0.5 x10^{-3} (**first iteration**)
Srain effect term $= \frac{0.4}{(1+1.5x0.5)} = 0.229$
$\beta = 0.229 * 0.543 = 0.124$
$V_m = 0.124\sqrt{20}\,(190)\,\frac{796.5}{1000} = 84.0$ kN
V due to load = 84.0 – 3.5 =80.5 kN

Check $\varepsilon_X = \frac{(\frac{80.5*(a-d)+12.85}{0.796})+84.0}{2(200000)\,(14000)} = 0.523$

New $\varepsilon_X = \frac{(0.5+0.523)}{2} = 0.5115$ (**2nd iteration**), Srain effect term = 0.226,
$\beta = 0.226x0.543 = 0.123$, $V_m = 83.2$ kN, V due to load =79.7 kN, Check $\varepsilon_X = 0.518$

New $\varepsilon_X = 0.515$ (**3rd iteration**), Srain effect term $= 0.2257$, $\beta = 0.2257 * 0.543 = 0.1224$, $V_m = 82.87$ kN, V due to load =79.37 kN, Check $\varepsilon_X = 0.516$ Good convergence, $V_{predicted}$ =82.87 kN