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THE SOURCE OF CANADIAN DESIGN STANDARD REQUIREMENTS FOR SHEAR DESIGN IN BEAMS

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ABSTRACT
Shear resistance is an important part of the design process of masonry beams. Section 12.3.5 of CSA S304.1-94 gives the parameters for masonry shear design. All clauses in this section are based on previous research, some of them developed many years ago. The objective of this paper is to study the background of these clauses for a better understanding of the standards for masonry design. Masonry beam shear design is based on the Simplified Method for Design for Shear in Flexural Regions in the Concrete Design Standard (CSA A23.3) but also on the research work of Suter, Hendry and Keller at Carleton University. They studied the parameters that influence shear resistance of masonry (concrete and brick) beams so that improvements to the Canadian design standard could be made. The CSA S304.1 has recently been revised and published in 2004. The clauses pertaining to masonry beam shear design have been revised and so a comparison between the 1994 and 2004 editions will be presented.

KEYWORDS: beams; brick; block; shear; codes.

INTRODUCTION
One part of the design of flexural members is design against shear failure. Masonry, like concrete, is stronger in compression than in shear or tension. Therefore, shear design of masonry becomes quite important.

Masonry beam design in the Canadian Standard (CSA S304.1) [1] is very similar to that in concrete design. Specifically, masonry beam shear design is based on the Simplified Method for Design for Shear in Flexural Regions (Clause 11.3 from the Design of Concrete Structures Standard: CSA A23.3-94 [2]). However, since masonry is not an isotropic material, masonry shear resistance is more complex and difficult to understand than for concrete.

EARLY RESEARCH
Many people have conducted research in this field and, hence, there is much information to be found in the literature. Focussing on Canada, a very extensive research program on shear strength of masonry (brick and block) beams has been carried out by G.T. Suter et al. at Carleton University, Ottawa.

Some of the work published by Suter was carried out in Edinburgh when he was on leave there. In 1974, Suter and A.W. Hendry [3] indicated the need for a change to limit states design of masonry. At the time, reinforced concrete design had already changed to limit states design from working stress design, but masonry had not. Based on experimental results carried out by others, they acknowledged the possible influence of shear span to effective depth ratio, the amount of flexural reinforcement, and brickwork compressive strength on the ultimate shear resistance of reinforced brickwork beams.

In Table 1, some of the experimental results reported by Suter and Hendry are presented. As a conclusion of this research, they noted the significant increase in shear resistance when the ratio of shear span to effective depth decreased and when the brickwork compressive strength increased. They also noticed that the influence of tensile reinforcement ratio was of not much relevance.

Table 1. Summary of Shear Strength Test Results [3]

Ref.	Beam No.	v_c (N/mm²)	a / d	ρ (%)	Unit strength (N/mm²)	Mortar composition by vol.
[4]	6	1.512	1.00	1.46	9.42	1:0.25:3
	7	0.788	1.50			
	8	0.596	2.00			
	9	0.437	2.50			
	10	0.491	3.00			
	11	0.410	4.00			
	12	0.330	5.00			
[10]	B6	0.641	2.50	0.52	14.76	1:0.15:3
	2C3	0.752		0.92		
[11]	11	1.069	2.50	1.89	10	1:0.25:3
	13	0.993		2.30		
[12]	A-1	0.717	1.21	1.43	5.10	1:0.16:3
	B-3	1.062			7.65	
	AA-1	0.869	1.21	1.39	4.62	1:0.3:4.5
	BB-2	1.234	1.21	1.43	6.84	

A year later, they reported on a systematic investigation [4] of some of the parameters mentioned before (a/d and ρ). Two series of beams were investigated with values of ρ of 0.24% and 1.46%, representing the lower and upper amounts of steel used in practice. The values of a/d ranged from 1 to 3 in increments of 0.5 for the series with ρ = 0.24% and values of 1, 1.5, 2, 2.5, 3, 4 and 5 were used for the series with ρ = 1.46%.

The test results are summarized in Table 2. The beams were subject to 2 point loads on the span so that essentially 2 failures occurred, hence the 2 values, v_{c1} and v_{c2}. The conclusions drawn from this study were similar to those in the previous work, noting that the data show "a significant increase in v_c with decreasing a/d value"[4]. They also added: "the authors feel that the increase in v_c with increasing ρ is so small that, contrary to the case of reinforced concrete, this increase should be neglected and design be based on a conservative value of shear stress which would represent all ρ values encountered in practice" [4].

Table 2. Summary of Test Results [4]

Beam Series	Beam No.	ρ (%)	a/d	Total ultimate **ν** **νc1** (N/mm²)	**νc2** (N/mm²)	Mortar strength (N/mm²)	Brick pier strength (N/mm²)
1	1	0.24	1.00	1.056	>1.110	12.10	7.94
	2		1.50	0.556	0.857	11.65	
	3		2.00	0.402	0.648	12.07	7.72
	4		2.50	0.479	-	10.68	
	5		3.00	0.364	0.364	10.43	
2	6	1.46	1.00	1.512	>1.512	11.33	11.03
	7		1.50	0.788	0.942	9.27	
	8		2.00	0.596	0.684	12.10	
	9		2.50	0.437	0.555	11.10	10.98
	10		3.00	0.491	0.494	13.80	
	11		4.00	0.410	>0.426	12.94	
	12		5.00	0.330	0.415	13.06	

Later work by Suter and H. Keller [5] noted that the allowable shear stress at the time, $0.7\sqrt{f'_m} \leq 0.345$MPa (50 psi), where f'_m was in psi units, occasionally gave unsafe values and did not take into account the effect of a/d in the shear resistance of masonry beams. They proposed an ultimate shear stress criterion, as follows:

for $a/d > 2$ $\quad \nu_c = 0.345$ MPa Equation 1

for $a/d \leq 2$ $\quad \nu_c = 0.345 * \left(\frac{2d}{a}\right)$ MPa Equation 2

According to their research, for values of a/d > 2, there was almost straight line behaviour of ν_c, independent of changes in a/d, ρ, and f'_m. Therefore they suggested specifying a constant stress value. On the other hand, for values of a/d ≤ 2, a significant increase in ν_c was observed as a/d decreased, so the proposed equation depended only on the most influential factor.

Suter and Keller continued their research [6] with a study where the main objective was to compare three types of beams; Reinforced Concrete (RC), Grouted Reinforced Masonry (GRM) and Reinforced Masonry (RM), where RM are beams made of solid brick units and mortar and the reinforcement is placed between the mortar joints; and GRM are beams made of solid brick units, mortar and grout, the reinforcement is placed in a cavity inside the beam and then grouted.

They tested two series of 8 beams of GRM and RM and compared the results with available published data for RC beams. For this research, the only variable was the factor a/d with values ranging between 1 and 7, and a large value of ρ = 1.4% was used. Again they found that as a/d decreased, the shear strength increased. They also observed that GRM ν_c values were between RC and RM ν_c values, noting that as the GRM beam width decreased, ν_c values approached those of RM, and as the GRM beam width increased, values approached those of RC.

In 1982 Suter and Keller reported on an extensive research program [8], testing over 70 concrete masonry beams, studying the effect of key parameters affecting shear capacity (a/d, joint spacing, type and slump of fill, effective depth, coursing of beams, and ρ).

The results showed the significant influence of a/d and how shear resistance of concrete masonry was between that of reinforced concrete and reinforced brick masonry. One new finding of this research program was that as the beam depth increased, the shear resistance decreased. This had already been established by Kani in 1967 [9] for reinforced concrete beams, but in reinforced concrete masonry beams this effect was more pronounced.

SHEAR RESISTANCE OF MASONRY BEAMS IN THE CODE

The 1994 edition of CSA S304.1 [1] provides the requirements for shear design of beams in Clause 12.3 Beams - Shear.

Clause 12.3.5.4 defines the equation to calculate the shear resistance for continuously grouted hollow block masonry (Equation 3). The shear resistance depends on the compressive strength of the masonry (based on prism tests or tabulated values based on unit strength and mortar type), on the effective depth and width of the beam, and on the density of the units, as these parameters are known for their influence on shear strength according to the research carried out by Suter, Hendry and Keller.

$$V_m = \phi_m\, 0.2\, \lambda \sqrt{f'_m} \left(1.0 - \frac{(d - 400)}{1500}\right) b_w d \qquad \text{Equation 3}$$

There are both similarities and differences between this equation and that for concrete beams [2]:

$$V_c = \left(\frac{260}{1000+d}\right) \lambda\, \phi_c \sqrt{f'_c}\, b_w d \qquad \text{Equation 4}$$

The upper limit on the shear resistance in Equation 3 is $\phi_m\, 0.2\, \lambda \sqrt{f'_m}\, b_w\, d$, which is the same as the upper limit for concrete beams, $0.2\lambda\phi_c\sqrt{f'_c}\, b_w\, d$. The coefficient of 0.2 comes from the old ACI expression for V_c using psi units where $V_c = 2\sqrt{f'_c}\, b_w d$. 2 psi are 0.167 MPa but in 1984 it was raised to 0.2 MPa. ϕ_m is the resistance factor for masonry, 0.55 (for concrete is 0.6), and takes into account the effects of variability in strength and dimensions, and mode of failure (brittle or ductile) of masonry. λ is a factor to account for using low density masonry units, since a decrease in unit density results in a decrease in masonry shear resistance. The expression $\phi_m \sqrt{(f'_m)}$ is the strength of masonry related to tensile stresses that are produced inside the beam due to shear forces.

The lower limit on the shear resistance, V_m, is $\phi_m\, 0.12\lambda\sqrt{f'_m}\, b_w\, d$. The coefficient of 0.12 is slightly different than the coefficient for concrete beams (0.10). These upper and lower limits for shear strength apply for all cases when the effective depth of the beam is less than 400 mm and more than 1000 mm, respectively [7].

In masonry not constructed with lintel or U blocks the continuity of the grout is interrupted by webs and the grout would also not be continuous across the head joints. In addition, the grout inside the masonry unit shrinks and the bond between the grout and concrete masonry units is not well defined. Therefore, V_m must be multiplied by a factor of 0.6 when the beam is not built using lintel or U shaped blocks.

In the 2004 edition of the CSA S304.1 [2], the provisions for shear design of masonry beams are contained in Clause 11.3. The equation set out for grouted hollow and semi-solid concrete block masonry and grouted hollow clay masonry has remained largely the same (Equation 5). The coefficient has been reduced and other changes (ϕ_m changed from 0.55 in 1994, to 0.6 in 2004) in the equation mean that the minimum shear resistance governs now for deeper sections, 1525 mm effective depth vs. 1000 mm effective depth previously.

$$V_m = 0.16\phi_m\lambda\sqrt{f'_m}\left(1.0 - \frac{(d - 400)}{2000}\right) b_w d \qquad \text{Equation 5}$$

The upper limit on the shear resistance in Equation 5 is $0.16\phi_m\lambda\sqrt{f'_m}\, b_w\, d$ and this limit is applied to beams with effective depth less than 400mm. The lower limit is now $0.07\phi_m\lambda\sqrt{f'_m}\, b_w d$. Note also that the 2004 edition does not have a 0.6 multiplication factor for non-continuous vs. continuous grout.

The provisions for shear resistance for solid masonry units are contained in Clause 12.3.5.5 of the 1994 edition (Equation 6) and in Clause 11.3.4.4 in the 2004 edition (Equation 7). The 1994 edition did not differentiate between “reinforced grouted brick masonry of solid units” and “reinforced brick masonry of solid units”, whereas 2004 does, and uses different equations for each.

$$V_m = \phi_m\, \chi\, 0.08\sqrt{f'_m}\, b_w d \qquad \text{Equation 6}$$

$$V_m = 0.056\phi_m\lambda\sqrt{f'_m}\left(1.0 - \frac{(d - 400)}{2000}\right) b_w d \qquad \text{Equation 7}$$

Equation 6 had no lower limit and the upper limit of $\phi_m\chi 32\sqrt{f'_m}\, b_w$ governs for beams with effective depth greater than 400 mm (where χ is a factor that accounts for direction of the compressive forces inside the masonry element, $\chi = 0.5$ for compressive forces normal to the head joints and $\chi = 1.0$ for compressive forces normal to the bed joints). The lower limit on Equation 7 is 0 and the upper limit is $0.056\phi_m\lambda\sqrt{f'_m}\, b_w d$ governing for effective depth less than 400 mm. There is a significant difference in the form of Equation 7 compared to the previous edition, and the coefficient used in Equation 7 is almost 3 times less than that used in Equation 5. This is consistent with findings by Suter and Keller [6] that showed that reinforced brick masonry beams had lower shear resistance than reinforced concrete masonry beams.

A comparison of shear resistance of reinforced brickwork beams for the 1994 and 2004 editions of CSA S304.1 is shown on Figure 1, where f’m = 14.76 MPa and tensile reinforcement is 0.92%. 1994 CSA S304.1 edition establish a fixed value for beam depths greater than 400mm.

Even so, 2004 CSA S304.1 edition allows upper values for shear resistance, both editions work with very low values compared with test results of ultimate shear resistance [10].

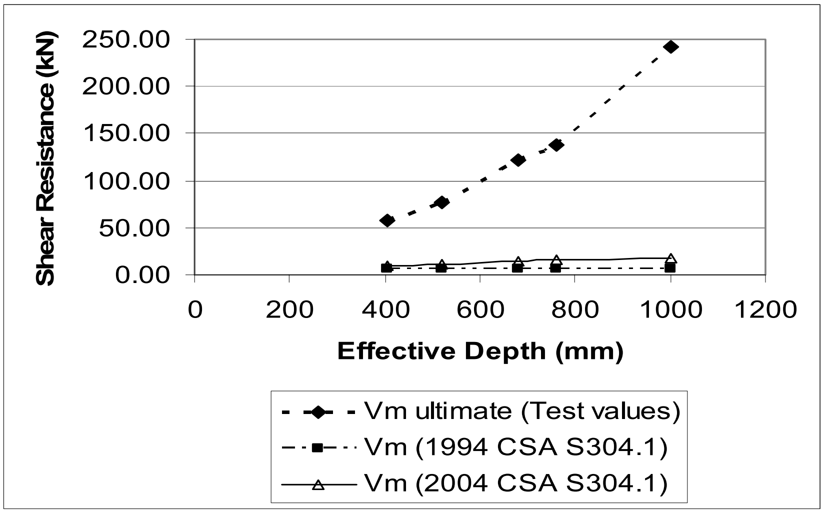


Figure 1. Shear Resistance of Reinforced Brickwork Beams

The allowable contribution to shear resistance from shear reinforcement has not changed from the 1994 edition to the 2004 edition:

$$V_s = \phi_s A_v f_y d/s \leq 0.36 \phi_m \sqrt{f'_m}\, b_w d \qquad \text{Equation 8}$$

The requirements for spacing limits and minimum shear reinforcement have also not changed in that the spacing is limited to the lesser of 600 mm or d/2 and, if V_f is less than V_m but greater than ½ V_m, minimum shear reinforcement must be provided such that:

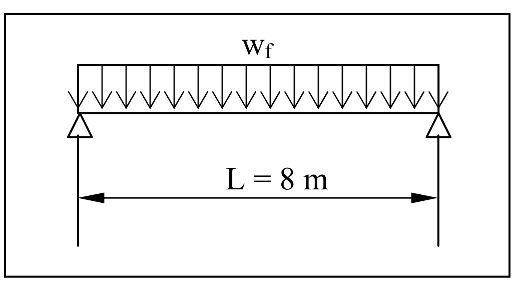
$$s_{max} = \frac{A_v f_y}{0.35 b_w} \qquad \text{Equation 9}$$

With the changes that have occurred in the recent edition, it is interesting to see how these changes actually affect the design of masonry beams. This will be illustrated using design examples in the following section.

COMPARISON OF BEAM SHEAR DESIGN PROVISIONS IN 1994 AND 2004 EDITIONS OF CSA S304.1

Example 1:
For a normal density grouted 200 mm beam:

$V = w_f * L / 2 = 96$ kN



d = 850 mm
bw = 190mm
f'm = 14 MPa
fy = 400 MPa
DL = 4.8 kN/m
LL = 12 kN/m
$w_f = 1.25*DL + 1.50*LL$
$w_f = 24$ kN/m

At a distance d = 850mm from the support $V_f = 75.6$ kN.

Tables 3 and 4 show the shear design for the beam using the 1994 and 2004 editions of CSA S304.1. Table 3 is the design if the beam is constructed of 5 courses of concrete block. For this beam the 2004 equation permits higher shear strength of masonry by about 60% compared with the 1994 equation. Table 4 shows the design if the beam is constructed of clay brickwork. In this case for brickwork beams the shear strength from the 2004 equation is approximately 20% higher than the 1994 equation. Table 5 shows the design for Example 2 which is a 3 course concrete block beam with the same loading as Example 1 but with a shorter span of 4 m. In the case of the shorter span, the shear strength calculated by the 2004 equation is approximately 50% higher compared with the 1994 equation.

Table 3. Comparison between Shear Design Provisions in 1994 and 2004 Editions of CSA S304.1 (5 course Hollow Concrete Block Beam, fully grouted)

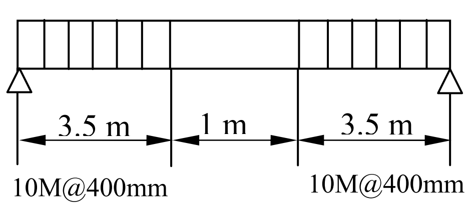
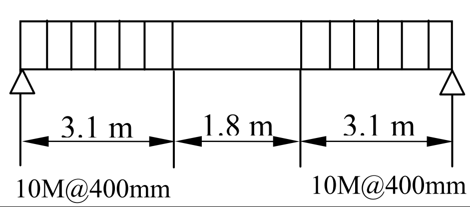
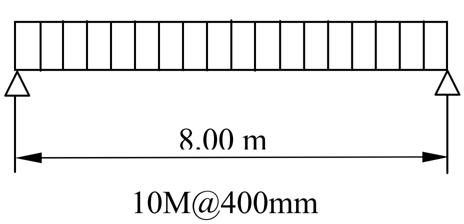
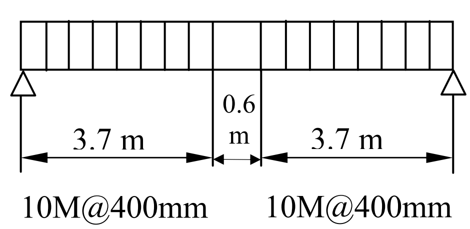
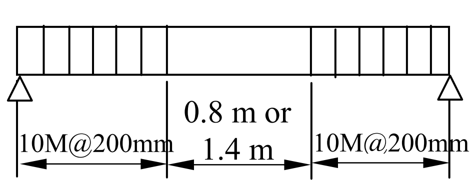
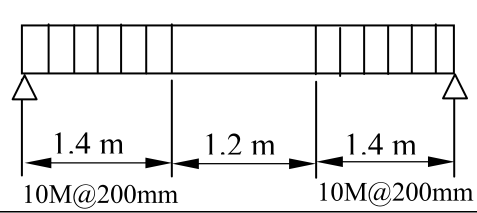
1994 CSA S304.1	**2004 CSA S304.1**
Masonry Shear Resistance (grout is not continuous): $V_m = 0.60 \left\{ \phi_m 0.2\, \lambda \sqrt{f'_m} \left(1.00 - \frac{d - 400}{1500}\right) b_w d \right\}$ $V_m = 27.92$ kN	Masonry Shear Resistance: $V_m = 0.16 \phi_m \lambda \sqrt{f'_m} \left(1.00 - \frac{d - 400}{2000}\right) b_w d$ $V_m = 44.96$ kN
Reinforcement Shear Resistance: $V_s = V_f - V_m = 47.68$ kN	Reinforcement Shear Resistance: $V_s = V_f - V_m = 30.64$ kN
Spacing between stirrups: Use single leg stirrups 10M (A_v= 100 mm²) $s \leq \phi_s A_v f_y d / V_s = 606$ mm but d/2 = 425 mm s = 400 mm	Spacing between stirrups: Use single leg stirrups 10M (A_v= 100 mm²) $s \leq \phi_s A_v f_y d / V_s = 943$ mm but d/2 = 425 mm s = 400 mm
When $V_f \leq V_m = 27.92$ kN $s = A_v f_y / 0.35 b_w$ s = 600mm spacing limit d/2 still governs No stirrups when $V_f \leq 0.5 V_m = 13.96$ kN Applies for middle 1.16 m of beam.	When $V_f \leq V_m = 44.96$ kN $s = A_v f_y / 0.35 b_w$ s = 600mm spacing limit d/2 still governs No stirrups when $V_f \leq 0.5 V_m = 22.48$ kN Applies for middle 1.87 m of beam.
Stirrup distribution: 	Stirrup distribution: 

Table 4. Comparison between Shear Design Provisions in 1994 and 2004 Editions of CSA S304.1 (Solid Brick Unit grouted Beam)

1994 CSA S304.1	**2004 CSA S304.1**
Masonry Shear Resistance (χ = 0.5 because compressive forces are normal to the head joints): $V_m = \phi_m \chi\, 0.08 \sqrt{f'_m}\, b_w d$ V_m = 13.29 kN But the upper limit is $\phi_m \chi 32 \sqrt{f'_m}\, b_w$ V_m = 6.26 kN	Masonry Shear Resistance: $V_m = 0.056 \phi_m \lambda \sqrt{f'_m} \left(1.0 - \frac{(d - 400)}{2000}\right) b_w d$ V_m = 15.74 kN
Reinforcement Shear Resistance: $V_s = V_f - V_m$ = 69.34 kN	Reinforcement Shear Resistance: $V_s = V_f - V_m$ = 59.86 kN
Spacing between stirrups: Use single leg stirrups 10M (A_v= 100 mm²) $s \le \phi_s A_v f_y d / V_s$ = 417 mm s = 400 mm	Spacing between stirrups: Use single leg stirrups 10M (A_v= 100 mm²) $s \le \phi_s A_v f_y d / V_s$ = 482 mm but d/2 = 425mm s = 400 mm
When $V_f \le V_m$ = 6.26 kN $s = A_v f_y / 0.35 b_w$ s = 600mm spacing limit d/2 still governs No stirrups when $V_f \le 0.5 V_m$ = 3.13 kN Applies for middle 0.13 m of beam.	When $V_f \le V_m$ = 15.74 kN $s = A_v f_y / 0.35 b_w$ s = 600mm spacing limit d/2 still governs No stirrups when $V_f \le 0.5 V_m$ = 7.87 kN Applies for middle 0.66 m of beam.
Stirrup distribution: 	Stirrup distribution: 

Example 2: For Example 1 with a reduced span of 4 m, the required beam height is only 590 mm (3 course beam). In this case d = 450 mm and at a distance d from the support, V_f = 37.2 kN.

Table 5. Comparison between Shear Design Provisions in 1994 and 2004 Editions of CSA S304.1 (3 course Hollow Concrete Block Beam, fully grouted, shorter span than Table 3)

1994 CSA S304.1	**2004 CSA S304.1**
Masonry Shear Resistance(grout is continuous): $V_m = \phi_m 0.2\, \lambda \sqrt{f'_m} \left(1.00 - \frac{d - 400}{1500}\right) b_w d$ $V_m = 34.02$ kN Masonry Shear Resistance(grout is notcontinuous): $V_m = 0.6 * 34.02$ kN $= 20.41$ kN	Masonry Shear Resistance: $V_m = 0.16 \phi_m \lambda \sqrt{f'_m} \left(1.00 - \frac{d - 400}{2000}\right) b_w d$ $V_m = 29.94$ kN
Reinforcement Shear Resistance: $V_s = V_f - V_m = 3.18$kN (grout continuous) $V_s = 16.79$ kN (grout not continuous)	Reinforcement Shear Resistance: $V_s = V_f - V_m = 7.26$ kN
Spacing between stirrups: Use single leg stirrups 10M (A_v= 100 mm²) $s \leq \phi_s A_v f_y d / V_s$ $s \leq 4811$ mm (grout continuous) $s \leq 911$ mm (grout not continuous) but d/2 = 225mm s = 200 mm	Spacing between stirrups: Use single leg stirrups 10M (A_v= 100 mm²) $s \leq \phi_s A_v f_y d / V_s = 2107$ mm but d/2 = 225mm s = 200 mm
When $V_f \leq V_m$ $s = A_v f_y / 0.35 b_w$ s = 600mm spacing limit d/2 still governs No stirrups when $V_f \leq 0.5 V_m$ Applies for middle 1.42 m of beam (grout continuous) Applies for middle 0.85 m of beam (grout not continuous)	When $V_f \leq V_m = 29.94$ kN $s = A_v f_y / 0.35 b_w$ s = 600mm spacing limit d/2 still governs No stirrups when $V_f \leq 0.5 V_m = 14.97$ kN Applies for middle 1.25 m of beam.
Stirrup distribution: 	Stirrup distribution: 

SUMMARY

Shear strength design of masonry beams stems from the method used in the shear design of concrete beams however, past research by Suter et al. permitted the equations to be adjusted to include factors affecting the shear strength specifically in masonry. The beam shear design provisions of the 2004 edition of the Canadian Standard, CSA S304.1, permit increased masonry shear strength for both blockwork and brickwork beams, which leads to more economical designs. It is interesting to note that for a shorter span beam, although a shallower section is sufficient for flexural capacity and the shear to be resisted by reinforcement is only 1/3 or less than that for the longer span, due to the limitation of d/2 the requirements for shear reinforcement are substantially increased. If the depth of the beam had not been reduced (d = 850 m), then according to 1994 S304.1 shear reinforcement at 400 mm spacing would still be required for the entire length of the beam and according to the 2004 edition, no stirrups would be required at all.

There are still questions remaining: Are there other factors in masonry shear design that should be taken into account in the calculation of shear strength? Since masonry is formed by the interaction of mortar, grout and masonry units, should the bond between these elements be included in the shear strength equation? What is the effect of the bond, that is, stack bond vs. running bond. Further research could lead to the development of more realistic, accurate and economical designs.

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