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Effect of Vertical Reinforcement on the In-Plane Shear Strength of Concrete Masonry Walls

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ABSTRACT

In-plane shear strength is one of the key parameters in the design of concrete masonry shear walls. According to the North American masonry standards (i.e. CSA S304-14 and TMS 402/602-22), the shear forces are primarily resisted by masonry and horizontal shear reinforcement, whereas the New Zealand standards (NZS 4230) consider the contribution of vertical reinforcement in the calculation of shear strength. This study experimentally investigates the in-plane shear (diagonal tensile) strength of full-scale masonry assemblages (i.e., wallets). Nine full-scale wallets with dimensions of 1.2 ×1.2 m in accordance with ASTM E519 were constructed to yield three groups of specimens, namely, fully grouted wallets with a reinforcement ratio of 0.42%, partially grouted wallets with a reinforcement ratio of 0.17%, and ungrouted wallets. The results showed that the vertical reinforcement reached its yield capacity, significantly contributing to the shear strength of the tested wallets. Using these results, along with data from 44 previously studied masonry walls with different vertical and horizontal reinforcement ratios, the shear strength expressions proposed by the North American and New Zealand masonry standards were evaluated. The results demonstrated that CSA S304-24 produced the most conservative predictions compared to other standards, while NZS 4230 had the closest shear strength values to those observed by the experimental data. This can be attributed to the contribution of vertical reinforcement in resisting the shear stress in the NZS 4230. An adjustment to the shear strength equation of CSA S304-14 is proposed to incorporate vertical reinforcement, which has resulted in more precise and reliable predictions of masonry shear strength.

KEYWORDS

Vertical reinforcement, Shear strength, Concrete masonry walls, Diagonal tension, Horizontal reinforcement.

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INTRODUCTION

Masonry construction is favorable due to its durability, fire resistance, design versatility, and soundproofing characteristics. Unreinforced masonry construction is often characterized by its lower shear strength and brittle failure compared to other construction materials; however, grouting and placing reinforcement in some or all the cells improve the shear strength, change the type of failure, and enhance overall stability [1–8] .Accurately evaluating the shear strength of masonry structures is crucial to ensure their stability and reliable performance when subjected to lateral forces, such as those caused by earthquakes or wind.

Several studies [1–8] have been conducted to evaluate the shear strength of concrete masonry walls. Shing et al. [1] investigated the effect of axial stress and vertical and horizontal reinforcement ratios on the flexure and shear strength of masonry walls. Sixteen full-scale fully grouted masonry shear walls were tested under cyclic loading. Shing et al. [1] concluded that the axial load has little significance on the shear strength. However, the specimens with higher vertical reinforcement exhibited a ductile failure. El-Dakhakhni et al. [2] tested eight full-scale concrete masonry walls under quasi-static cyclic loading with different aspect ratios and vertical and horizontal reinforcement ratios to evaluate the shear strength expressions of CSA S304-04 [9] and other international standards. El-Dakhakhni et al. [2] concluded that CSA S304-04 [9] had the most conservative predictions compared with other standards.

Seif ElDin et al. [3-4] investigated the in-plane shear behavior of fully grouted shear walls by testing nine shear walls constructed with different horizontal and vertical reinforcement ratios, levels of axial load, and anchorage end details. The results showed that increasing the vertical reinforcement ratio significantly enhanced the walls' shear strength. Similarly, Amgad et al. [6] tested 41 concrete masonry assemblages to evaluate the diagonal tensile (shear) strength of masonry assemblages. The research focused on the effects of spacing and reinforcement ratios of both vertical and horizontal reinforcements. Amgad et al. [6] concluded that adding vertical reinforcement improved shear strength by 21%, while combining horizontal and vertical reinforcement resulted in a 15% increase in shear capacity. Researchers (Shing et al. [1], Voon and Ingham [8], and Seif ElDin et al. [5]) developed expressions for computing the shear strength of reinforced masonry shear walls that incorporate the effect of vertical reinforcement in resisting shear stress.

In this study, the diagonal tensile (shear) strength was evaluated experimentally for full-scale masonry assemblages. Three groups of specimens, namely fully grouted (FG), partially grouted (PG), and ungrouted (UG), were constructed and tested. All the grouted assemblages were reinforced vertically only. The shear strength equation (Eq. (3)) in CSA S304-14 [10] is modified by adding a term to reflect the contribution of vertical reinforcement. The proposed equation was validated using data from 44 full-scale reinforced masonry shear walls obtained from the literature and was compared against existing masonry building codes.

CODE EQUATIONS FOR SHEAR STRENGTH.

Several equations [10–13] have been proposed to calculate the nominal shear strength of reinforced masonry shear walls (V_n), derived from experimental investigations on different wall configurations. These expressions are commonly presented in the form of a summation of the contributions of three main components: shear strength provided by masonry (V_m), shear strength provided by axial load (V_p), and shear strength provided by horizontal shear reinforcement (V_s), as illustrated in Eq. (1).

(1) $V_n = V_m + V_p + V_s$

According to the North American masonry standards (i.e., CSA S304-14 [10] and TMS 402/602-22 [11]), the shear forces are primarily resisted by masonry and horizontal shear reinforcement. In contrast, the New Zealand standards NZS 4230:2004[12] consider the contribution of vertical reinforcement in the calculation of shear strength based on the study conducted by Voon and Ingham [8]. To account for the effect of vertical reinforcement on the shear strength, Eq. (1) is extended to include an additional term representing the contribution of vertical reinforcement (V_v), as illustrated in Eq. (2). Table 1 summarizes the predictive equations (Eqs. (3-7)) for computing the shear strength of fully grouted reinforced masonry shear walls for different masonry standards in SI units.

(2) $V_n = V_m + V_v + V_p + V_s$

The shear strength equations are expressed in terms of the compressive strength of the masonry (f'_m), the wall dimensions (b_w, l_w, d_v), the shear span-to-depth ratio ($M_f/V_f.d_v$), axial load (P_d), and the horizontal and vertical reinforcement ratios (ρ_h, ρ_v). CSA S304-24 introduces a refined shear strength calculation method that explicitly incorporates the effect of axial compressive force on the masonry contribution (V_m), in addition to the shear resistance provided by horizontal reinforcement (V_s).

Table 1: Code equations for shear strength

Eq.	V_m	V_v	V_p	V_s	Reference
(3)	$0.16\left(2.0 - \frac{M_f}{V_f d_v}\right) b_w d_v \sqrt{f'_m}$	-	$0.25\ P_d$	$0.6\left(\frac{A_h}{S_h}\right) f_{yh}\ d_v$	CSA S304-14 [10]
(4)	$0.083\left(4.0 - 1.75\frac{M_f}{V_f d_v}\right) b_w l_w \sqrt{f'_m}$	-	$0.25\ P_d$	$0.5\left(\frac{A_h}{S_h}\right) f_{yh}\ l_w$	TMS 402/602-22 [11]
(5)	$(c_2)\ b_w d_v\ v_{bm}$	$(c_1)\ b_w d_v\ v_{bm}$ where $c_1 = 33\frac{\rho_v f_{yv}}{300}$	$0.9\ P_d \tan\alpha$	$0.8\left(\frac{A_h}{S_h}\right) f_{yh}\ d_v$	NZS 4230 [12]
(6)	$\beta\sqrt{f'_m}\ b_w d_v$ where $\beta = 0.18\ , \frac{230}{1000+1.4d_v}$	-	-	$\left(\frac{A_h}{S_h}\right) f_{yh}\ d_v\ \cot\theta_1$ where $\theta_1 = 42°$	CSA S304-24 [13] (Simplified Method)
(7)	$\beta\sqrt{f'_m}\ b_w d_v$ where $\beta = \frac{0.40}{1+1500\epsilon_x} \times \frac{1300}{1000+z_e}$	-	Incorporated in ϵ_x	$\left(\frac{A_h}{S_h}\right) f_{yh}\ d_v\ \cot\theta_1$ where $\theta_1 = 29 + 7000\epsilon_x$	CSA S304-24 [13] (General Method)

Experimental Program

Nine full-scale masonry assemblages (wallets) were constructed and tested in accordance with ASTM E519[14] to evaluate the effect of different vertical reinforcement ratios on the diagonal tensile (shear) strength. In addition, six full-scale concrete masonry prisms were constructed in a running bond pattern and tested in accordance with CSA S304-14 [10] to determine the masonry compressive strength, f'_m. The masonry assemblages had dimensions of 1.2 m × 1.2 m, while the prisms had a cross-sectional area of 390 mm × 190 mm and a height of three courses, as illustrated in Table 2. Professional masons constructed the masonry assemblages and prisms at the Structures Lab of Concordia University to ensure the consistency of the construction. The masonry assemblages were categorized into three groups of specimens, namely, fully grouted wallets with a vertical reinforcement ratio of 0.42%, partially grouted wallets with a vertical reinforcement ratio of 0.17%, and ungrouted wallets, as shown in Figure 1-b. Each grouted cell in the fully and partially grouted wallets was reinforced with 15M vertical deformed steel bars (nominal cross-sectional area of 200 mm^2), except the two outermost cells in the fully grouted assemblages were reinforced with 10M rebars. Rebar positioners were used to ensure the bar was vertically aligned and centered in the cell.

The six masonry prisms were divided into three ungrouted prisms and three grouted ones to determine the masonry compressive strength, f_m'.

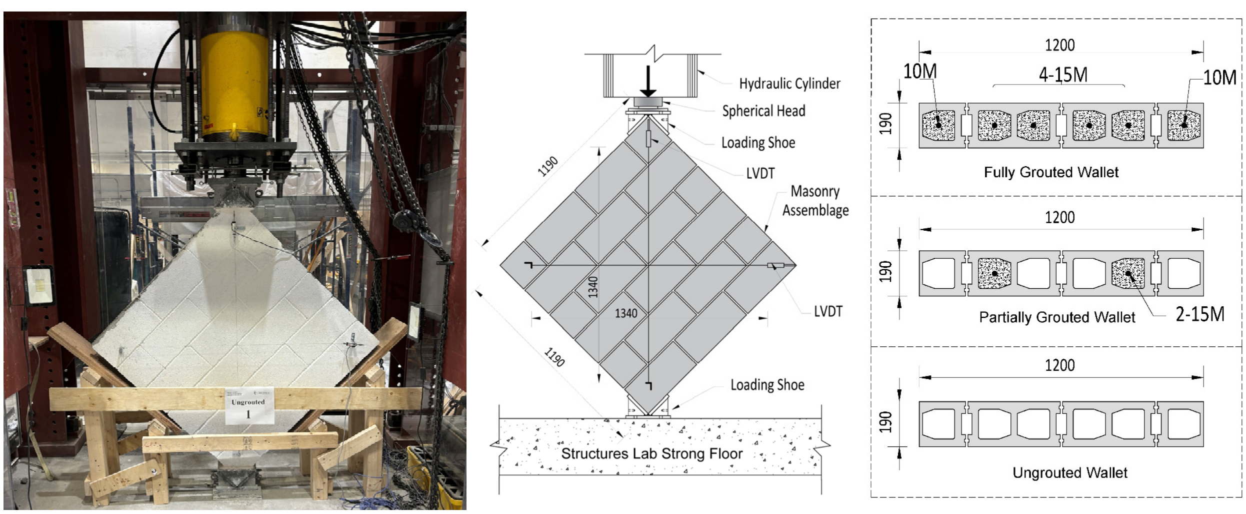
The average yield and ultimate strengths of three specimens from 15M steel bars were 460 and 663 MPa, respectively. The full-scale stretcher and half blocks used in masonry assemblages were tested per ASTM C140 [15], showing average compressive strengths of 20.0 MPa and 20.6 MPa, respectively. Prebagged Type S mortar was used to construct the prims and assemblages. The average compressive strength of three 50 mm mortar cubes was 23 MPa. Premixed grout bags were used in filling the masonry cells. The average compressive strength of six grout cylinders was 29 MPa.

Table 2. Test matrix.

Group	Assemblages				Prisms	
	Dimensions	Reinforcement	Number of grouted cells	Count	Dimensions	Count
Fully grouted	1.2 m ×1.2 m	4- 15M + 2-10M (0.42%)	6	3	390 mm x 190 mm	3
Partially grouted		2- 15M (0.17%)	2	3		-
Ungrouted		-	-	3		3

TEST SETUP AND INSTRUMENTATIONS

The masonry assemblages and prisms were tested under vertical loads using a servo-controlled 5000 kN hydraulic cylinder with a displacement-controlled loading system to capture the post-peak behavior of the specimens. Figure 1-a illustrates the test setup, specimen dimensions, and the instrumentation used for testing the masonry assemblages and prisms. To ensure uniform stress distribution across the cross-sectional area and maintain the vertical alignment of the specimens, the loading and reaction sides of the masonry assemblages and prisms were capped with 50 MPa dry-stone.



a- Test setup of masonry assemblages b- Assemblages' configurations

Figure 1: Test Setup, dimensions, instrumentation, and configurations for Masonry assemblages

For the testing of masonry assemblages, two loading steel shoes were fixed to the loading and reaction corners of the specimen, as shown in Figure 1-a. These steel shoes were designed to meet the specifications and dimensions recommended by ASTM E519 [14]. Using the lab's overhead crane, the specimen was lifted with a steel clamping setup that gripped all four sides. It was then carefully positioned into the steel shoe which was pre-filled with dry stone, ensuring proper vertical and horizontal alignment before the dry stone hardened. After fixing the first steel shoe, the specimen was rotated, and the same process was repeated to fix the second shoe. Four linear variable displacement transducers (LVDTs) were installed (two on each side) to measure the vertical and horizontal deformations. The gauge length for vertical and horizontal LVDTs was consistent across all specimens, measuring 1340 mm.

EXPERIMENTAL RESULTS AND DISCUSSION

This section summarizes the shear stress and strain behaviour of the full-scale masonry assemblages and failure modes. The axial compressive stress, failure modes, and crack propagation of the full-scale masonry prisms are also discussed. The aim of the experimental work is to contribute new data to the existing literature on shear strength calculation and to evaluate the influence of varying vertical reinforcement ratios on shear strength.

Masonry assemblages

The failure modes of the fully grouted, partially grouted, and ungrouted masonry assemblages are shown in Figure 2. The fully grouted assemblages exhibited two vertical cracks running through the blocks and grout cores, along with minor stepped cracks at the center of the assemblages. The reinforcement in the grout cores effectively prevented the assemblages from splitting and significantly reduced the width of the cracks. The fully grouted assemblages demonstrated a ductile failure behavior with only minor vertical cracking. The partially grouted assemblages showed stepped cracks along the mortar joints in the ungrouted cells, while diagonal cracks developed in the grouted cells. This behavior is attributed to cracks following the weakest path through the mortar joints. Similar to the fully grouted assemblages, the reinforcement effectively prevented the separation of the partially grouted assemblages and mitigated brittle shear failure. In contrast, the failure of the ungrouted masonry assemblages started with a stepped crack propagating through the mortar joints. After reaching the peak stress, the assemblages split vertically. This failure mechanism can be attributed to lateral tensile strains induced by vertical compressive loading, leading to splitting failure.

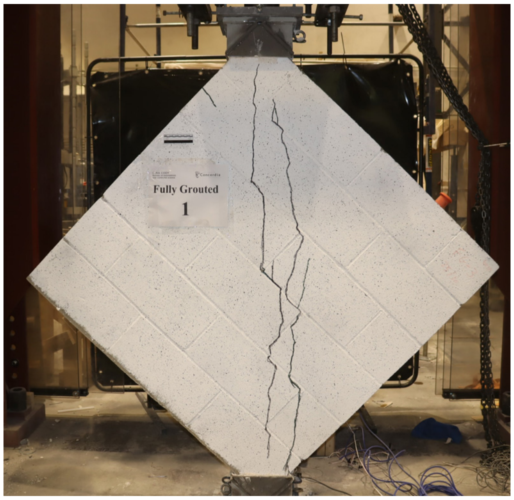
Table 3 presents a summary of the results of the tested masonry assemblages. Shear strength, shear strain, and modulus of rigidity were calculated by ASTM E519 [14], using Eqs. (8-10). For all the tested assemblages, the coefficient of variation (CoV) for the peak load remained below 9.8%, indicating the constituency of the results. The shear strength of the fully and partially grouted assemblages was 3.71 times and 2.56 times higher, respectively, compared to ungrouted assemblages. Additionally, the reinforcement within the grouted cells significantly enhanced strain at peak load, increasing it by 667% for the fully grouted assemblages and 561% for the partially grouted assemblages relative to the ungrouted ones. Figure 3 illustrates the average shear stress-strain relationship of the masonry assemblages.

$$(8)\quad S_s = \frac{0.707\,P}{A_n}$$

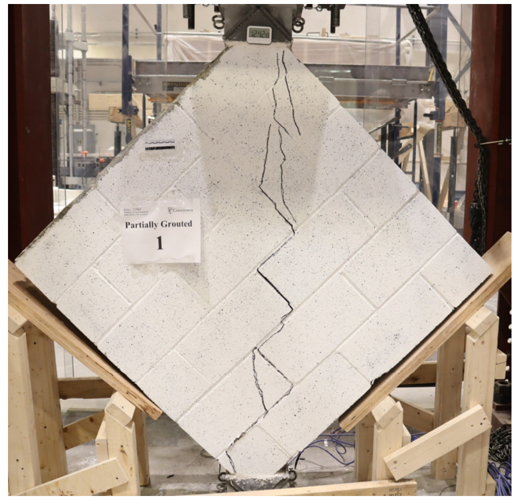
$$(9)\quad \gamma = \frac{\Delta x + \Delta y}{g_l}$$

$$(10)\quad G = \frac{S_s}{\gamma}$$

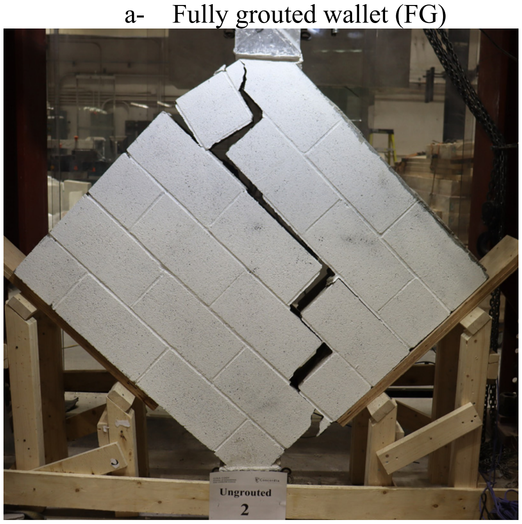
where S_s is the shear stress based on the net area, MPa, A_n is the net area, mm^2, P is the applied axial load, kN, Δx and Δy are the shortening in the perpendicular and parallel directions to the loading, mm, respectively, γ is the shear strain, mm/mm, g_l is the gauge length, mm and equals to 1340 mm, and G is the modulus of rigidity, MPa.



a- Fully grouted wallet (FG)



b- Partially grouted wallet (PG)



c- Ungrouted wallet (UG)

d- Grouted prisms

e- Ungrouted prisms

Figure 2: Typical failure of masonry assemblages and prisms

Table 3 Masonry assemblages' results

Group	ID	Peak load, kN	Average (CoV)	$S_{s,max}$ MPa	Average (CoV)	$\gamma_{max,}$ mm/mm	Average (CoV)	Modulus of rigidity, G, MPa	Average (CoV)
Fully grouted	FG 1	873.2	789.0 (9.8%)	2.71	2.45 (9.8%)	0.00245	0.00240 (19.0%)	1105	1040 (16.3%)
	FG 2	721.9		2.24		0.00192		1168	
	FG 3	771.8		2.39		0.00283		847	
Partially grouted	PG 1	291.0	309.6 (7.2%)	1.59	1.69 (7.0%)	0.00167	0.00202 (16.8%)	953	849 (15.3%)
	PG 2	334.2		1.82		0.00205		890	
	PG 3	303.6		1.66		0.00235		704	
Ungrouted	UG 1	105.3	101.0 (6.0%)	0.69	0.66 (6.4%)	0.00040	0.00036 (18.1%)	1715	1892 (13.2%)
	UG 2	96.7		0.63		0.00031		2069	
	UG 3*	-		-		-		-	

* Specimen broken during handling

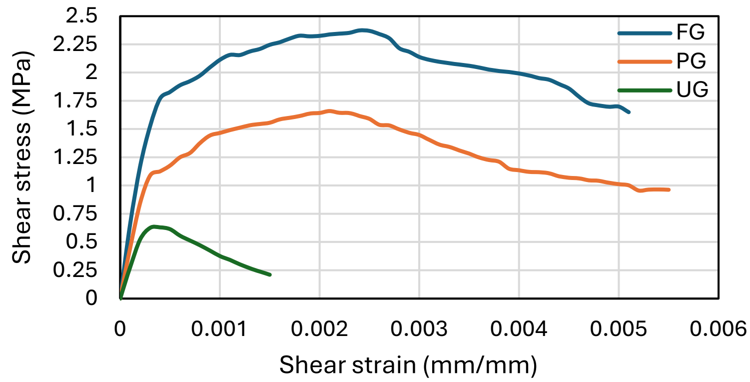


Figure 3: Average shear stress-strain curves of masonry assemblages

Masonry prisms

The axial compressive stress for the ungrouted and grouted prisms was calculated using their respective cross-sectional net areas of 35,432 mm² and 70,232 mm². Figure 2 illustrates the typical failure modes for both grouted and ungrouted prisms. The average compressive strengths were 17.0 MPa for grouted prisms and 16.7 MPa for ungrouted prisms.

PROPOSED EQUATION FOR THE CONTRIBUTION OF THE VERTICAL REINFORCEMENT

The proposed equation for calculating the contribution of vertical reinforcement to the in-plane shear resistance of fully grouted reinforced masonry shear walls is given in Eq. (11). This equation is based on the general form used for determining the shear resistance of horizontal shear reinforcement, $V_{s,h}$ as given in Eq. (12). The proposed equation incorporates a factor (k_1), representing the percentage of vertical reinforcement assumed to contribute to the ultimate shear resistance of the wall effectively. This approach aligns with the factors 0.6 and 0.5 specified in CSA S304-14 [10] and TMS 402-602 [11] respectively, which account for the fact that not all horizontal reinforcement may achieve its yield strength when the nominal shear capacity of the wall is reached. This limitation arises because reinforcement near the top or bottom of a shear crack may lack sufficient development length to fully develop its yield strength ([1], [16])

$$(11)\quad V_{s,v} = k_1 \left(A_v \frac{d_v}{s_h}\right) f_{yv} \quad \text{where } k_1 = 0.13$$

$$(12)\quad V_{s,h} = 0.6 \left(A_h \frac{d_v}{s_v}\right) f_{yh}$$

where $A_v \frac{d_v}{s_h}, A_h \frac{d_v}{s_h}$ are the total cross-sectional area of horizontal shear reinforcement and vertical reinforcement crossing the potential diagonal crack, respectively, d_v is the effective depth, f_{yv}, f_{yh} are the yield strength of the vertical and horizontal reinforcement, respectively, and s_h, s_v are the spacing of the vertical and horizontal reinforcement, respectively.

The factor k_1 was determined based on experimental data from seven pairs of shear walls. Each pair of walls shared identical dimensions, masonry compressive strength (f'_m), applied axial compression load, and horizontal shear reinforcement (A_h). The difference in the experimental shear strength between the walls in each pair was attributed solely to the variation in the percentage of vertical reinforcement. This methodology effectively isolates the contribution of vertical reinforcement by eliminating the influence of other factors, such as masonry shear strength (V_m), axial compression (V_p), and horizontal shear reinforcement (V_s). Table 4 provides a summary of the calculations used to determine the percentage contribution of vertical reinforcement to the overall shear resistance.

Table 4: Calculation for the factor k_1

Reference	Wall ID	σ_n	f'_m	ρ_h	Vertical reinforcement	$V_{exp.}$	k_1
Seif ElDin, et al. (2016)	W-S$_v$800	1.0	13.1	0.13	9-15M	381	0.16
	W-S$_h$800				3-30M	352	
Shing et al. (1990)	S-9	1.86	20.7	0.14	5-#5	427	0.14
	S-14				5-#6	467	
	S-13	1.86	22.8	0.25	5-#6	500	0.10
	S-16				5-#7	536	
Ibrahim and Suter (1999)	W-2	0.69	21.3	0.2	6-15M	389	0.11
	W-4				6-20M	412	
Mahrous et al. (2025)	C5	-	22.8	0.3	-	139	0.11
	C10				4-10M	159	
	C5	-	22.8	0.3	-	139	0.19
	C8				4-#3	164	
	C8	-	22.8	0.3	4-#3	164	0.09
	C10				4-10M	159	
Average							**0.13**
COV							24%

EVALUATION OF ACCURACY OF THE PROPOSED EQUATION

To validate the accuracy of the proposed equation for predicting the in-plane shear strength of reinforced masonry (RM) shear walls, data from 44 fully grouted RM shear walls from previous experimental studies and the tested fully grouted masonry assemblages were analyzed, as illustrated in

Table 5. These tests were selected to cover a variety of configurations, including different horizontal and vertical reinforcement ratios, as well as varying axial stress levels, with shear failure being the dominant failure mode. The dataset was sourced from five key studies: Matsumura [17], Shing et al. [1], Voon and Ingham [7], El-Dakhakhni et al. [2], and Seif El-Din et al.[3]. The experimental shear strength and the results from tested masonry assemblages were compared against the shear strength equations provided by various masonry design standards, including CSA S304-14[10], TMS 402/602-22[11], NZS 4230[12], and CSA S304-24 [13] (using both the General and Simplified Methods), as shown in Figure 4. Furthermore, the proposed modification to the CSA S304-14[10] equation was also evaluated against the experimental results.

The results showed that NZS 4230[12] provides accurate predictions for shear strength, primarily due to its incorporation of the effect of vertical reinforcement in its calculations with an average experimental-to-predicted ratio of 1.11. TMS 402/602-22[11] exhibited good predictions as it increases the contribution of the masonry in resisting the shear strength, V_m compared to CSA S304-14[10]. In contrast, the simplified method of CSA S304-24 [13] exhibited poor alignment with the experimental results, particularly for walls with low horizontal reinforcement. It underestimated the shear strength by 72%, highlighting its limitations in adequately accounting for variations in reinforcement ratios and applied axial compressive stress. Although the general method of CSA S304-24 [13] incorporates more detailed calculations and demonstrated better accuracy than the simplified method, it remains conservative compared to other codes. Notably, the proposed modification to CSA S304-14[10], which accounts for the contribution of vertical reinforcement as defined by Eqs. (2) and (11) achieved predictions closely aligned with experimental results, with an average experimental-to-predicted ratio of 1.17.

Table 5: $V_{Experimental}$ / $V_{Predicted}$ for Shear Design Expressions

Reference	ID	TMS 402/602-22	NZS 4230	CSA S304-24 Simplified Method	CSA S304-24 General Method	CSA S304-14	Proposed Modification
		V_{exp}/V_{pred}	V_{exp}/V_{pred}	V_{exp}/V_{pred}	V_{exp}/V_{pred}	V_{exp}/V_{pred}	V_{exp}/V_{pred}
Matsumra (1986)	1	1.35	1.36	1.56	1.64	1.81	1.68
	2	1.17	1.20	1.36	1.50	1.58	1.47
	3	1.30	1.33	1.50	1.65	1.74	1.62
	4	1.17	1.18	1.36	1.56	1.58	1.46
	5	1.04	1.29	3.85	2.68	1.36	1.25
	6	1.16	1.29	1.55	1.78	1.44	1.35
	7	1.30	1.44	1.74	2.00	1.61	1.51
	8	1.31	1.37	1.75	1.77	1.62	1.52
	9	1.38	1.37	1.51	1.68	1.66	1.58
	10	1.12	1.13	1.24	1.50	1.36	1.35
	11	1.02	1.03	1.12	1.34	1.23	1.22
	12	1.16	1.15	1.21	1.51	1.39	1.38
	13	0.94	0.90	0.90	1.03	1.11	1.10
	14	1.18	1.19	1.30	1.59	1.43	1.42
Shing et al. (1990)	15	1.12	1.08	1.56	1.55	1.37	1.10
	16	1.31	1.33	1.26	1.43	1.71	1.22
	17	1.22	1.21	1.37	1.47	1.54	1.16
	18	1.31	1.30	1.47	1.61	1.65	1.25
	19	1.05	1.10	1.46	1.74	1.28	1.15
	20	1.02	0.99	1.12	1.40	1.20	1.06
	21	1.12	1.14	1.55	1.70	1.37	1.18
	22	1.16	1.08	1.27	1.40	1.35	1.12
Voon and Ingham (2006)	23	0.96	1.16	1.99	1.37	1.33	1.05
	24	0.92	1.16	2.56	1.46	1.33	1.02
	25	0.97	1.15	1.96	1.36	1.35	1.06
	26	1.02	1.18	2.44	1.58	1.37	1.12
	27	1.03	1.21	2.30	1.54	1.40	1.12
	28	0.71	0.79	1.83	1.25	1.06	0.79
	29	1.03	1.29	3.76	1.90	1.49	1.25
El-Dakhakhni (2013)	30	0.96	0.97	2.06	1.20	1.21	0.88
	31	1.23	1.24	1.15	1.24	1.61	1.11
	32	1.13	1.03	1.11	1.12	1.49	0.87
	33	0.96	0.96	2.17	1.15	1.21	0.73
	34	1.05	1.03	1.29	1.30	1.30	0.98
	35	1.19	1.26	2.31	1.26	1.63	1.04
Seif ElDin (2016)	36	1.00	0.93	1.18	1.18	1.25	1.01
	37	1.16	1.15	4.85	1.70	1.53	1.16
	38	1.04	1.03	0.97	1.05	1.35	1.04
	39	1.00	0.91	1.30	1.25	1.22	1.02
	40	0.73	0.68	0.87	0.86	0.91	0.74
	41	0.99	0.92	1.17	1.17	1.23	1.00
	42	0.94	0.88	1.11	1.11	1.17	0.95
	43	0.92	0.86	1.09	1.09	1.15	0.93
	44	0.85	0.81	1.01	1.04	1.06	0.88
FG Wallets	45	1.30	0.94	4.00	1.78	1.93	1.61
Average (COV)		**1.09 (14%)**	**1.11 (16%)**	**1.72 (50%)**	**1.46 (22%)**	**1.40 (15%)**	**1.17 (20%)**

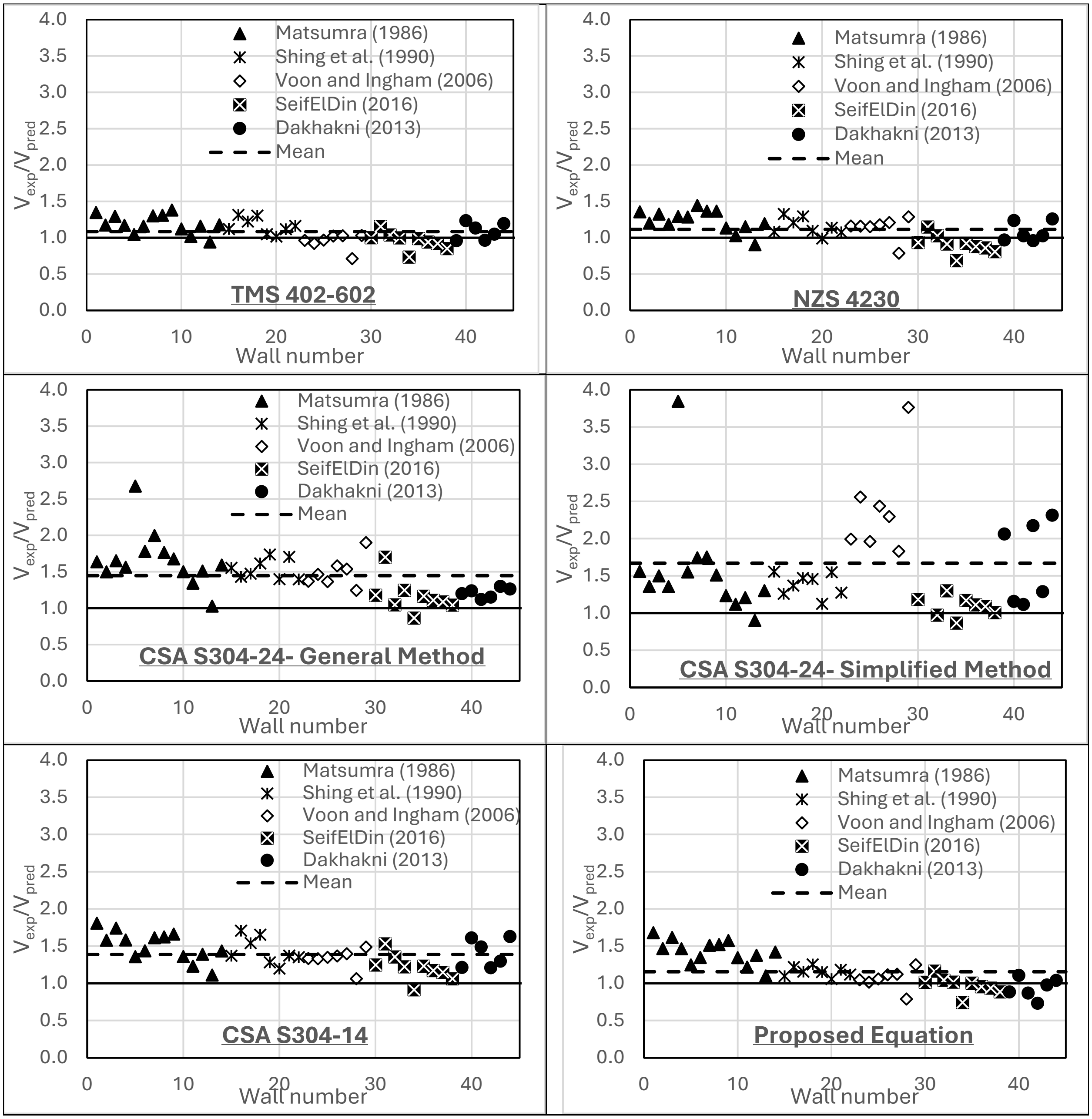


Figure 4: VExperimental /VPredicted for Shear Design Equations

Conclusions

This study presents an experimental investigation of the shear strength of full-scale masonry assemblages. Nine full-scale masonry assemblages with dimensions of 1.2 × 1.2 m were constructed to yield three groups of specimens, namely, fully grouted wallets with a vertical reinforcement ratio of 0.42%, partially grouted wallets with a vertical reinforcement ratio of 0.17%, and ungrouted wallets. The test results show that vertical reinforcement for the FG and PG wallets increased the shear strength of the assemblages by 371% and 256% compared to the hollow assemblages (UG), respectively. A modification was introduced to the shear strength equation in CSA S304-14[10] by incorporating an additional term to account for the contribution of vertical reinforcement. This contribution was determined by analyzing the differences in experimental results from seven pairs of specimens with identical masonry compressive strength, applied axial load, and horizontal reinforcement, thereby isolating the effect of vertical reinforcement on shear strength. The proposed modification was further validated using data from 43 reinforced masonry shear walls obtained from five sources in the literature, as well as results from tested fully grouted wallets. The findings demonstrated that the proposed modification yielded more accurate predictions than CSA S304-24 [13] and CSA S304-14[10]. Further testing with varying axial stress and reinforcement ratios is recommended to better quantify the effect of vertical reinforcement on shear strength.

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