Shear Strength Expression for Partially Grouted Masonry Walls

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Abstract: The majority of reinforced masonry constructions in the mid-western and eastern parts of the United States are partially grouted. Previous studies on partially grouted walls have shown that design provisions of the masonry standard joint committee overestimate the shear capacity of such walls. As a result of these studies, masonry standard joint committee has introduced a reduction factor in its newest version in order to overcome this problem. At 2016, the masonry standard joint committee was changed to building code requirements and specifications for masonry structures. The first part of the paper is devoted to this issue by comparing the shear strength of 42 full-scale partially grouted wall tests available in the literature with the current building code requirements and specifications for masonry structures' shear strength expression. Results indicated that the current shear strength expression can accurately predict the shear strength of partially grouted walls with grouted cells spaced to a maximum of 1.2 m because it was developed based on test results of fully grouted walls. Unfortunately, implementing this factor has not solved the fundamental problem of the code shear strength expression for those walls with grouted cells spaced higher than 1.2 m (ordinary reinforced masonry). The over-estimation can be traced to the existence of insufficient length of horizontal reinforcement (steel

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effectiveness), and the assumption of linearly varying flexural strains, which the ungrouted portions of the partially grouted wall have proven unable to carry. The goal of the current study is to investigate a new, more realistic approach to the shear mechanism of ordinary reinforced masonry partially grouted walls through adopting modified infilled wall model as opposed to the monolithic wall model that already underpins the problem with the current shear strength expression. A new expression, therefore, was developed for ordinary reinforced masonry partially grouted walls based on modified infilled-frame mechanism. The shear strength predicted by the proposed model was in a good agreement with available test data including walls tested by authors.

Keywords: Partially grouted; Masonry wall; Masonry standard joint committee; Infilled-frame.

1. Introduction

Most of the shear expressions available for reinforced masonry (RM) walls are statistical-based and no analytical equation has been adopted for these types of walls. This is mainly due to the complexity of the shear mechanism in RM walls. Practically, the shear strength of RM walls has been calculated by the summation of three components: masonry contribution, friction due to axial compression stress, and shear reinforcement contribution. The first expression was developed almost three decades ago by [1]. Various other shear expressions have also been reported in literatures including; [2-13], some of them will be discussed here. While many parameters were investigated in these studies, accuracy of the reported expressions, especially in the case of partially grouted (PG) walls, is questioned.

1 Much of the RM wall database used for developing shear expressions was derived
2 from testing completed as part of [14]. These assemblages tested were both traditional
3 flexural walls, subjected to double curvature, known as piers. The shear strength formula in
4 [13] is largely based on experimental data obtained from fully-grouted (FG) reinforced
5 masonry walls and piers. A review and comparison of different shear strength formulas
6 reported in the literature were provided in [10]. They explained the derivation of the shear
7 formula adopted in [8] by evaluating the shear expression with data only obtained from FG
8 walls.

9
10 In this study, we proceed to evaluate the shear expression of [13] based on the
11 compiled database of full-scale PG masonry walls. Expressions and discussions existing in
12 literature along with results of the current survey show that introducing new parameters
13 and constants have not solved the inaccuracy of the shear expression for PG walls with
14 grouted cells spaced higher than 1.2 m classified as ordinary reinforced masonry walls
15 (ORM). Therefore, this issue needs to be viewed from a new perspective separate from
16 introducing arbitrary factors into the FG shear expression, due to the different behavior of
17 ORM PG walls.

18
19 The mechanical behavior of PG walls is more similar to confined masonry and
20 masonry-infilled frames than FG masonry. However, concrete elements of these two
21 systems have more reinforcement than grouted masonry elements in PG walls, where a
22 single bar is placed in the middle of a grouted cell. Vertical grouted cells in PG walls are
23 highly vulnerable to shear failure [15 and 16]. This vulnerability is primarily attributed to

the initiation and propagation of horizontal shear cracks along the bed joints from the hollow masonry panels (weak elements) into the vertically reinforced grouted cells. Regions around the intersection of bond beams and vertically grouted cells are especially susceptible to damage. This is due to interaction between the grouted masonry elements and ungrouted masonry panels near the intersections. This behavior observed in the PG walls is very similar to the strut mechanism developed in masonry infill in infilled-frame structures. A new expression, therefore, was developed for PG walls based on infilled-frame mechanism. The development of the new formula was additionally developed with refined finite element models developed by authors [17 and 18]. A parametric study was conducted considering different spacing of horizontal and vertical reinforced cells, axial compressive loads, masonry compressive strength, and area of steel. Finally, the proposed expression was verified with experimental results of tested walls by authors [18-20] in addition to 42 full-scale PG wall tests available in the literature [10, 15, 21-28].

1.1 Available reinforced masonry shear strength expressions

There are almost 10 different shear equations available in the literature for reinforced masonry walls. It should be mentioned that these design provisions and statistical equations are different from each other. The number of contributions to the final shear strength is different and this confirms that the approach used to build the analytical equations/design is different. Even if the coefficients of equations are calibrated through experimental fitting or statistical adjustment. However, the common fact about the most of them is that they are developed solely based on the results of fully grouted specimens. The first shear expression for RM shear walls was developed by [1] to predict the in-plane

shear strength based on testing nearly 60 concrete and 30 brick masonry shear walls. This expression was proposed based on regression analysis of test results in the conventional form of three terms: the contribution of the shear strength of masonry, axial stress, and shear reinforcement, as can be seen in Eq. 1. Factors k_u (factor concerning the type of grouting) and γ (factor concerning the action to confine grout) were considered 0.64 and 0.6 for PG walls, respectively.

$$V_n = \left[k_u k_p \left(\frac{0.76}{(h/d)^{0.7}} + 0.012 \right) \sqrt{f_m} + 0.2 \sigma_n + 0.18 \gamma \delta \sqrt{\rho_h f_{yh} f_m} \right] \times (0.875 t d) \quad (1)$$

Another shear expression was proposed by [2] for concrete RM walls, employing regression analysis of the shear strength of 22 FG shear walls tests (Eq. 2). All vertical reinforcement was considered effective due to the dowel action. Although, this expression was developed based solely on FG wall test results, it was directly applied for predicting the shear strength of PG masonry as well. In this expression, the contribution of the bottom and top transverse reinforcement were neglected because of insufficient development length caused by diagonal cracks. Additionally, a distinct term for the effect of axial stress was not considered and instead is included with the masonry term.

$$V_n = (0.166 + 0.0217 \rho_v f_{yv}) \sqrt{f_m} A_n + (0.0217 \sigma_n A_n) \sqrt{f_m} + \left(\frac{L - 2d}{S_h} - 1 \right) A_h f_{yh} \quad (2)$$

In order to calculate the in-plane shear strength of FG and PG reinforced masonry shear walls, New Zealand masonry design standard [3] developed an expression (Eq. 3)

based only on test results conducted in 1980 or earlier. As can be seen from the expression, the effect of axial compressive stress was included in the V_m term ($V_m = 0.1f'_m + \sigma_n$). The effect of vertical reinforcement was not considered here and full contribution of the horizontal reinforcement was included without any limitation on their locations in the wall.

$$V_n = V_m b_w d + A_h f_{yh} d / s_h \quad (3)$$

[4] developed an expression (Eq. 4) for reinforced masonry shear strength utilizing the test results in [1, 2 and 29]. The effect of dowel action of vertical reinforcement was neglected in this expression. Although the shear strength of masonry is affected by the type of masonry and effect of inelastic deformation of masonry shear strength, previous studies overlooked these two parameters in deriving their expressions. However, in the first part of Eq. 3 a reduction factor (C_{ap}) was introduced based on wall ductility for considering the degradation of masonry shear strength in the inelastic response. Additionally, in this expression another coefficient factor (K) was introduced for different types of masonry employed in the wall construction. Again, no specific parameter was defined for PG masonry shear walls and the same shear mechanism was considered for both FG and PG walls.

$$V_n = C_{ap} K A_n \sqrt{f'_m} + 0.25 \sigma_n A_n + 0.5 A_h f_{yh} d / s_h \quad (4)$$

[5] adopted an expression similar to Eq. 4. The ductility factor was expressed in terms of the wall aspect ratio ($\frac{M}{VL}$) as shown in Eq. 5. This shear expression took one step back in considering the inelastic behavior of masonry and just relied on the wall aspect

ratio. The effect of dowel action and vertical reinforcement were neglected in the expression and no clear distinction was defined for PG walls.

$$V_n = 0.083 \left[4.0 - 1.75 \frac{M}{V L} \right] A_n \sqrt{f_m'} + 0.25 \sigma_n A_n + 0.5 A_h f_{yh} d_v / s_h \quad (5)$$

The shear strength expression (Eq. 6) for RM walls available in [6] was exclusively based on the test results of NSF-sponsored US coordinate program for masonry building research [14], that dealt solely with the seismic performance of FG walls. However, unlike the Eq. 1, no modification factor was introduced for PG walls.

$$V_n = 0.083 \left[2.8 - 1.6 \left(\frac{h_e}{d_v} \right) \right] A_n \sqrt{f_m'} + A_{sh} f_{yh} \quad (6)$$

In 2007, [10] introduced modification factors to the [3] equation, creating the new shear expression [8]. This expression (Eq. 7) also did not provide any special consideration for PG walls.

$$V_n = \left[0.022 \rho_v f_{yv} + 0.084 \left(4.0 - 1.75 \frac{h_e}{l_w} \right) \right] \left(1 - \frac{\mu - 1.25}{2.75} \right) A_n \sqrt{f_m'} + (0.9 \tan \alpha) P^* + A_h f_{yh} d_e / s_h \quad (7)$$

A reduction factor ($\gamma_g = 0.75$) was introduced by [12] to the shear expression developed by [5] for PG walls, see Eq. 8. The reduction factor is associated with the results of PG and FG walls reported by [16 and 30], respectively. In each category, the ratio of the actual shear strength over the shear strength of [13] estimation (called TMS (2016) expression herein) were calculated. Ultimately, the reduction factor was extracted by dividing the mean results of PG walls to FG walls.

$$V_n = \gamma_g \left(0.083 \left[4.0 - 1.75 \frac{M}{V_L} \right] A_n \sqrt{f_m^{\square}} + 0.25 \sigma_n A_n + 0.5 A_h f_{yh} d_v / s_h \right) \quad (8)$$

α and β modification factors, as shown in Eq. 9, were introduced by [31] in order to improve the TMS (2016) expression. Factor α was introduced to include the effect of reinforcement spacing and the wall aspect ratio. It was concluded in [31] that strain in the horizontal reinforcing bars reduced as horizontal reinforcement ratio increased. Therefore, factor β was introduced to account for this reduction in the steel efficiency.

$$V_n = \alpha \left[4.0 - 1.75 \frac{M}{V_L} \right] A_n \sqrt{f_m^{\square}} + 0.25 \sigma_n A_n + \beta (0.5 A_h f_{yh} d_v / s_h) \quad (9)$$

2. Database

Not many studies have been carried out on the seismic performance of PG walls. Among those available, research focusing on full scale tests of PG walls are selected here and demonstrated in Table 1. These studies are explained to some extent in this section.

Table 1 Partially grouted wall database

Name	Height	Length	Thickness	Boundary	Reinforcement			Grout Spacing m		Prism Strength (f'm)	Axial Load kN
	m	m	mm		Vertical	Shear	Joint-Reinforcement	Horizontal	Vertical	MPa	
Chen et al. (1978) [20]	1.4	1.2	194	Fixed	-	-		1.4			188
					2D16	1D16		0.7			134
					2D25	-	-	1.4	0.9	10.8	130
					2D25	2D16		0.5			139
Yancey & Scribner (1989) [23]	1.4	1.2	194	Fixed		-	9-ga. ladder	1.4			321
						-	9-ga. ladder	1.4			268
						2D13	-	0.7			368
						3D16	-	0.7			275
						1D16	-	0.5	1	21.1	294
						1D10	9-ga. ladder	0.7			346
						2D13, 1D16	9-ga. ladder 1D16	0.7			395
						1D16	-	0.5			310
Brammer (1995) [24]	2.4	2.6	90	Cantilever	4D10	1D10					
		1.8	140		3D13	1D16					
		2.6	140		4D13	1D13					
		4.2	140		6D13	1D16	-	2	0.8	16.3	0
		1.8	190		3D13	1D16					
		2.6	190		4D13	1D16					
Schultz (1996) [14]	1.4	4.2	190	Fixed	6D13	1D16					
		2.8				2D10			2.7		267
		2.1				2D10			1.9		191
		1.4	194		2D19	2D10	-	0.7	1.3	14.5	133
		2.8				1D13, 1D16			2.7		266
		2.1				1D13, 1D16			1.9		177
Voon & Ingham (2006) [10]	1.8	1.8	140	Cantilever	5D22	-	-	0.4			
					3D22			0.8	1.6	18.5	0
Minaie et al. (2010) [15]	2.6	3.9	194	Cantilever							222
				Cantilever							222
				Fixed	4D19	1D19	-	1.2	1.2	17.2	0
				Fixed							0
Elmapruk (2010) [26]	1.6	2.6	194	Cantilever	4D19	1D16			1.2		
					4D19	1D16			1.2		
					4D19	1D19	-		1.2		
					4D19	2D16		0.7	1.2	17.4	49.4
					4D19	1D16			1.2		
					3D13, 2D19	1D16			0.8		
Nolph & ElGawady (2010) [27]	2.3	2.6	194	Cantilever					0.6		
					4D22	1D16			1.2		
					4D22	1D19			1.2		
					4D22	2D16	-	1.1	1.2	11.3	49.3
					4D19	1D16			0.8		
					5D16	1D16			0.6		

Four PG concrete masonry piers under axial stress of 0.38 MPa with an aspect ratio of 1.0 were tested by [21]. Although the spacing between vertically grouted cells was held constant, different spacings for bond beams were chosen. Cyclic load pattern and fixed-fixed boundary conditions were used. It was demonstrated that the shear strength of the PG walls were lower than that of FG walls. However, the net ultimate shear stress of the PG piers was 22% higher than that of FG specimens. All the specimens failed in a shear mode. Additionally, walls with more vertical reinforcement did not show better performance than those containing less. Therefore, the extra amount of flexural reinforcement was redundant, showing no influence on the shear strength of the specimens. This can be attributed to the pier frame action which carried additional axial load. As a result, the existing vertical compression stress on the piers counteracted some parts of the tensile stress that vertical reinforcement was designed to resist. Results of [21] showed that the energy dissipation of all the specimens increased as the drift ratio increased. The amount of reinforcement in both the horizontal and vertical directions appeared to be negligible as well.

Eight PG walls were tested by [24] to determine the effect of varying the amount and distribution of horizontal reinforcement on the shear resistance of PG masonry walls. All of the walls were constructed using only horizontal bed joint reinforcement with no vertical reinforcement as shown in Table 1. These walls were subjected to reversed cyclic load pattern combined with constant axial compressive stress and all the specimens had fixed-fixed boundary conditions. All the walls exhibited shear mode of failure by presenting cracks propagating along the major diagonals of the panels. It was concluded that small

1 amount of shear reinforcement were effective in postponing crack initiation by increasing
2 the cracking strength. Observation of [24] showed that the ultimate shear strength of the
3 walls increased as shear reinforcement increased, however not proportionally.

4
5 [25] performed quasi static reversed cyclic tests on seven cantilevered PG walls. All
6 the specimens were built with the same height, but different width and length and tested
7 under in-plane lateral load with no axial load present. As anticipated, results showed that
8 all walls but one failed in shear through diagonal tension cracks due to the lack of
9 horizontal reinforcement. The main aim of this research was to evaluate the accuracy of
10 shear strength expression proposed in [4]. It was concluded that these walls have shear
11 strength significantly greater than the code's prediction and the level of conservatism
12 increased as the length of walls increased.

13
14 [15] tested twelve PG concrete masonry shear walls under in-plane loads. Six of
15 them contained deformed bars as horizontal reinforcement in grouted bond beams and the
16 six other contained bed-joint reinforcement. The specimens were subjected to quasi-static
17 cyclic lateral displacements with fixed-fixed boundary conditions and nearly constant
18 vertical compressive stress. It was concluded that the lateral resisting mechanism for PG
19 reinforced masonry is vastly different from that of FG reinforced masonry. His results
20 showed that as the aspect ratio increased, the ultimate shear stress increased but the
21 stiffness decreased. Cracks that formed between the grouted and ungrouted cells tended to
22 grow and disturb the anchorage region of the horizontal reinforcement where they
23 intersect the vertical cells, thus, increasing the level of horizontal reinforcement slightly

1 increased the ultimate shear strength but had a negligible effect on stiffness. Because of the
2 limited number of tests, and the scatter of the data that is typical in shear strength tests, no
3 definitive conclusions could be drawn from this study. However, upon investigating the
4 performance of different shear strength formulas, he concluded that a formula developed
5 by [1] best represents the trends observed.

6
7 Thirteen PG concrete masonry walls with and without opening configurations were
8 tested by [32] with varying aspect ratios and reinforcement quantities. The height to length
9 ratios ranged from 0.6 to 3.0. These walls were nominally reinforced and had shear
10 reinforcement only at the top two courses of the walls. All walls exhibited diagonal shear
11 failure due to lack of shear reinforcement. [10] further tested ten walls, only two of which
12 were PG, to investigate the shear strength of concrete masonry and to measure the
13 accuracy of the [8] and [5] expressions. These walls constructed with a range of flexural
14 reinforcement, however, no shear reinforcement or axial load were present. Their
15 observations and test results revealed that the [5] expression can predict the experimental
16 shear strengths of the walls better than the [8] expression. However, both expressions
17 underestimate the shear strength of PG walls significantly because of neglecting the area of
18 grouted cells.

19
20 [16] tested four PG shear walls to examine the appropriateness of the [11] shear
21 strength expression and evaluate the seismic vulnerabilities of PG reinforced masonry
22 shear walls. The walls were tested under reversed displacement control cyclic quasi-static
23 patterns. All the walls were constructed with the same aspect and steel ratios, varying in

boundary conditions and axial load. It was shown that the [11] code's provisions overestimate the shear capacity of PG masonry shear walls by an order of two. Their observation showed that the [11] expression for PG walls became increasingly more unconservative as the spacing of both horizontal and vertical steel increased. Results of this study indicated that the behavior of PG walls is similar to that of infilled frames. They also concluded that frame mechanism caused part of the low shear strength of PG walls due to the loss of coupling between vertical reinforcing steel.

[27] tested six full scale PG walls with identical aspect ratios, vertical reinforcement ratio, and level of pre-compressive load. This study was aimed to investigate the effect of spacing of vertical reinforcement on the seismic performance of PG walls. An inverse relationship between the shear strength and spacing of vertical reinforcement was observed. The same ultimate lateral shear stress was reported for the specimens having vertical reinforcement spacing of 610 mm and 813 mm. Ductile failure mode was observed for the specimen having a vertical reinforcement spacing of 1.2 m due to the yielding of flexural reinforcement. He also reported that the shear strength decreased and ductility increased as the grout vertical spacing increasing. Increasing the grouted vertical spacing also changed the failure mode of walls from masonry toe crushing to yielding of vertical steel. [27] concluded that the [12] expression is highly unconservative by over-predicting the shear strength of PG walls.

Recently, five full-scale PG masonry shear walls were tested by [29]. It was found that [12] shear strength expressions are inadequate in estimating the shear strength of PG

walls with 1.2 m horizontal spacing between vertical reinforcement because of overestimating the contribution of the horizontal reinforcement. However, it was adequate for walls with spacing between vertical steel less than 0.8 m. The amount and distribution of horizontal and vertical reinforcement was effective for both development of shear strength and crack control. When the spacing of vertical reinforcement was relatively close, less than 1.2 m, they reported that the behavior of PG wall became more similar to FG walls, due to the percentage of grouted cells. As mentioned before, PG walls are designed based on an expression developed for FG walls. Therefore, by changing the hollow space between the horizontal and vertical reinforcement cells from 1.2 m to 0.8 m or less, the shear expression became more conservative.

3. Experimental program

3.1 PG Walls without openings

Portland cement lime (PCL) mortar type S according ASTM C270 ingredient proportion, as shown in Table 2 for each mix, with an average 28 days compressive strength of 3 MPa meeting ASTM C109 was used. Coarse grout according to standard specification for grout for masonry ASTM C 476 with an average net compressive strength of 23 MPa meeting ASTM 1019 was used (see Table 2). Grout slump of 250 mm was used.

Table 2 Mortar and grout proportions

Material	Code	Mortar, Kg	Grout, Kg
Fine Sand	ASTM C 144-11	42	-
Coarse Sand	ASTM C 404-11	-	29.3
Gravel	ASTM C 404-11	-	22.7
Lime	-	2.4	-
Portland Cement	ASTM C 150-15	11	11.5

Although large amounts of materials for constructing the walls were needed, mortar and grout were mixed in-house (see Figure 1). Because of wall height, a forklift was used to carry and pass the material to the masons.

Fig. 1 Grout and mortar preparation, unit cutting and material delivery

Because of the specific shape of the setup, there were some extra works to prepare units at the top bond beam level. Units with side holes were needed for attaching the load beam to the wall. Figure 1 shows the cutting and details of these units. Some of the top course units were perforated and paper tubes placed in their holes in order to prepare them for attaching the loading beams using rods. Vertical reinforcing steel rods were embedded into the foundation using Epoxy Hilty 500 MC with 254 mm depth as shown in Fig. 2.

Fig. 2 Epoxy vertical reinforcement into the foundation

Figure 3 shows the pouring and vibrating grout into the cells, also it is representing the wet surface after pouring and vibrating the grout into the vertical cells. This wet surface is a proof that grout had enough amount of slump.

Fig. 3 Pouring grout and vibration, wet surface of grouted column

A horizontal actuator at the top and two vertical actuators at the two sides transferred the horizontal and vertical load, through two stiff channel beams (called loading beam herein) which were post-tensioned to the wall, see Fig. 4. The boundary conditions of the walls were considered as a cantilever with the inflection point at the top of the walls with an aspect ratio of 1.0. The two vertical actuators were programmed to

1 impose axial compressive stress without any resistance against wall rotation at top (i.e.
2 when the horizontal actuator is pushing the wall, right side actuator arm becomes shorter
3 and left side actuator becomes longer without altering the applied force). To prevent the
4 wall from moving out-of-plane, the test specimen was restrained with hinged out-of-plane
5 supports at the top. Also, two additional courses were built above the target height of the
6 wall to provide enough development length of the vertical bars, and also properly
7 transferring the horizontal load to the wall. This configuration along with the pre-stress
8 loading beam was used to simulate the connection of floor diaphragm to the bearing
9 masonry wall.

10
11 The test setup for constructing and testing the masonry walls was designed and
12 fabricated in-house. Based on pre-analysis of the walls, the setup components were
13 designed to carry and transfer 1,335 kN shear force and 445 kN axial loads. The setup
14 included two 2.4x1.2x0.6 m foundation blocks (called footing herein) next to each other
15 and 4.3x0.9x0.3 m foundation beam (called concrete beam herein) that was tightened down
16 and post tensioned to the strong floor (see [Fig. 4](#)). The setup also included two H shape
17 steel columns under the vertical actuators sitting on foundation blocks, two steel channel
18 as load beam connected to the horizontal and vertical actuators, out-of-plane support and
19 another external long column standing out of setup just to hold the LVDT's (Linear Variable
20 Differential Transformers) sensors to record the horizontal displacement of specimens.

Fig. 4 Test setup

The first wall was built with single horizontal and vertically grouted cells under the conventional design details following [13] provisions. However, in order to investigate the role of the frame on the behavior of PG walls, the second specimen was built with two side-by-side horizontal and vertical grouted cells. The first and second specimens were designated by (single reinforcement) SR and (double reinforcement) DR, respectively. These walls are ordinary reinforced masonry (ORM) based on the [13] provisions only differing between the number of bars while the steel ratio remains the same. As a result, there are two grouted cells instead of one, in both the beams and cells of the double reinforced specimen. Vertical reinforcement in the SR and DR specimens consisted of 1D19 and 2D13, respectively. Specimens had vertical reinforcement ratio of approximately 0.1% of the net cross section area. The reinforcement of the first specimen (1D 19) bars spaced at 1.8 m on center in the vertical and horizontal directions. However, the reinforcement configuration for the second specimen was 2D13 bars in each direction. Fig. 5 shows the detail of these two specimens. Walls were tested with 0.14 MPa of axial stress on the net

area of walls. The summarized experimental shear strength (V_u), and predicted strength using TMS (2016) expression (V_n) are shown in Table 3. The expression showed 70% and 64% error in predicting the capacity of single and double grouted cells walls, respectively. More details of test set-up and wall configurations can also be found in [18 and 19].

Fig. 5 Reinforcement configurations

3.2 PG Wall with opening

One extra SR PG wall with opening (SRO) was built and tested for evaluating the effect of opening on the shear strength expression. Lateral loads were imposed by two actuators attached to loading beams that were anchored into the concrete topping shown in Fig. 6. An external vertical load was applied to the wall by means of four Dywidag post-tensioned rods that extended from the foundation to the roof structure. Because load would increase as strain was induced in the rods from drift, a spring was placed between the monitoring load cells and the nut to introduce compliance and keep this load approximately constant. In addition to the externally applied load, the weight of the roof and the loading beams added additional axial stress. The total axial stress on each pier (the two sections of the wall on either side of the opening) was approximately 0.3 MPa to

simulate gravity loads. Load cells were used to monitor the load in the Dywidag post tensioned rods, as well as the actuators.

Fig. 6 Test-setup for the wall with opening

The wall experienced the maximum shear force, V_u , of 273 kN corresponding to the wall drift of 0.29%. The low drift proved that the wall failed in a low ductile shear mechanism (Table 3). The TMS (2016) expression showed a high accuracy in predicting the capacity of this wall. More detail on test results can be found in [33].

Table 3 Experimental and predicted shear strength of walls

Wall ID.	Experimental V_u kN	Shear Capacity V_n kN	Failure Mode
SR	168	285	Low ductility shear
DR	221	363	Ductile shear
SRO	273	272	Low ductility shear

4. Evaluation of MSJC's (2013) shear strength expression for PG masonry

Reported shear strength of walls tested in the literatures (V_{exp}) is compared with the shear strength predicted by TMS (2016) expression (V_n) and the final results are plotted in Fig. 7a. Surprisingly, TMS (2016) fails to predict the shear strength of walls in 65% of cases. Although the TMS (2016) expression cannot exactly predict the shear strength of walls, only 36% of specimens fell out of the standard deviation of samples ($SD=0.25$ kN). This overestimation is shown in Figs. 7a and 7b using the lower limit and histogram plot, respectively. It should be mentioned that the upper bound covers the specimens very well and few data appeared out of the standard deviation limit. Hence, it can be concluded that the TMS (2016) expression, even after introducing reduction factor, is still unconservative for many of tested walls and, therefore, need to be revised.

(a) Scattered data

(b) Histogram graph

Fig. 7 MSJC shear strength prediction

[31] found that grout spacing and horizontal steel effectiveness are two key factors in predicting the shear strength of PG walls. He concluded that increasing the grout spacing and amount of shear reinforcement significantly effect on the over-prediction of the TMS (2016) expression. Consequently, he introduced two new factors to replace the one general reduction factor based on [27, 28 and 34] test results. Although these two mentioned parameters improved the accuracy of the TMS (2016) expression, other parameters, such as wall aspect ratio, masonry compressive strength, axial load and spacing between bond beams still need to be taken into account for an accurate prediction.

To that end, the effect of three terms in the TMS (2016) expression (masonry contribution V_n -A, axial load V_n -B, and shear reinforcement V_n -C) are investigated using available test data. Results showed that TMS (2016) expression is more un-conservative

1 when masonry contribution term (V_n-A) increases (Fig. 8a). Increasing the axial load has a
2 negligible effect on the over-prediction of shear strength, as shown in Fig. 8b. However,
3 surprisingly, as the shear reinforcement increases the over-prediction of the TMS (2016)
4 expression also increases, which is in agreement with [31] findings (Fig. 8c). By factoring
5 out the net area term from the TMS (2016) expression, overestimation can be related to
6 the following design details: aspect ratio and f'_m , axial stress (σ_n), spacing of vertical cells
7 and shear reinforcement amount for masonry contribution, and axial load and shear
8 reinforcement terms. Fig. 9 shows the effect of different parameters on the accuracy of TMS
9 (2016) expression. All of these parameters were plotted as compared with the shear
10 strength ratio by showing the standard deviation.

(a) Masonry

(b) Axial stress

(c) Shear reinforcement

Fig. 8 TMS (2016) expression terms

(a) Aspect ratio, H_{eff}/L

(b) Compressive strength, f_m

(c) Level of axial stress, σ

(d) Spacing of horizontal reinforcement

(e) Spacing of vertical reinforcement

(f) Area of horizontal reinforcement

Fig. 9 Effect of different parameters on TMS (2016) expression accuracy

4.1 Masonry contribution

As can be noted from Fig. 9, low aspect ratio of the wall and high f'_m value led to lower ratios of experimental to predicted strengths using TMS (2016) expression. As the aspect ratio decreases, the term $\left(4.0 - 1.75 \frac{M}{VL}\right)$ increases. Therefore, the masonry contribution becomes higher and consequently, the shear expression becomes more unconservative. This increase in masonry contribution can be also attributed to f'_m , since by increasing this parameter, the shear expression prediction becomes higher than values predicted experimentally. Although, a new reduction factor has been introduced to compensate the code over-prediction, these results are still in parallel with [16] findings that were based on [11], showing that [11] expression is unconservative.

4.2 Axial stress

The level of axial stress (based on net area) has a little effect on the accuracy of the TMS (2016) expression (Fig. 9c). It should be noted that the results were match with the test results before applying the reduction factor in TMS (2016) expression. However, applying the new reduction factor has a significant effect on overestimating the shear strength. By considering the effect of axial stress [16] showed that the average of trend line has about 20% difference with the experimental results in its maximum state.

4.3 Shear reinforcement

In accordance with Fig. 9d, spacing between bond beams has an insignificant effect on the accuracy of TMS (2016) expression. However, as the amount of horizontal steel increases the TMS (2016) expression becomes more unconservative. This is also in

1 agreement with [31] findings on the effect of shear reinforcement. This issue was
2 addressed for the first time by [4]. They argued that the insignificant effect of the
3 horizontal shear reinforcement is due to the insufficient bar development length especially
4 at the lower and upper level of bond beams where diagonal shear cracks cross at the ends
5 of the reinforcement.

6
7 Reduced contributions of horizontal shear reinforcement are a combination of the
8 fact that shear cracks are localized near the wall edges at the top and bottom of walls, thus
9 limiting the development length and possible contribution from horizontal reinforcement
10 spanning the crack. In addition, historically it was not possible to test true full-scale
11 masonry (e.g. 5m floor heights, 6m wall lengths) and as such the smaller 'piers' that could
12 be accommodated in structures labs would unlikely have sufficient wall length to develop
13 horizontal reinforcement spanning cracks even where the crack appears in the middle of
14 the wall. Furthermore, since design standards rely on full yield strength of horizontal
15 reinforcement as their basic assumption then shouldn't it be the onus of the designer of
16 record to first theoretically determine necessary the development length of the horizontal
17 reinforcement then the location of the principal shear crack and then what force could be
18 carried by each reinforcing bar at each discrete crack location. This would be similar to
19 how in-plane flexure is designed for, whereby not all vertical bars are necessarily yielding.

20
21 In addition to all mentioned parameters, the spacing between vertical grouted cells
22 is also investigated and plotted in Fig. 9f. As previously described, TMS (2016) expression
23 is developed for FG reinforced masonry. Therefore, this parameter is not included in the

shear expression. From the results given in the plot, it can be observed that increasing the vertical spacing between cells increases the over-prediction of the TMS (2016) expression. TMS (2016) expression reasonably predicts the shear strength of PG walls with horizontal and vertical grouted cells spaced up to 1.2 m, becoming less accurate for greater values. Hence, this research focuses on developing a shear expression for ordinary PG masonry walls with grout spacing larger than 1.2 m.

5. Investigating the infilled-frame mechanism of PG walls

Applying a diagonal load to an ungrouted masonry panel led to an abrupt tensile failure initiated by a stepped crack at the block-mortar interfaces. In the grouted specimens, however, the failure plane followed a straight line through a combination of head joints and masonry units. This behavior was investigated experimentally by [34] and numerically by [36 and 17] using ANSYS 5.3 [37] and Abaqus 6.10 [38] finite element programs, respectively. Fig. 10 shows the test specimen and test setup representing the standard diagonal tension test [39] in addition to the finite element model employed to simulate the specimen. The minimum principle stresses of diagonal tension specimen are plotted in Fig. 11. Plots show that as the diagonal load increases compression struts start to form, reaching three struts before failure takes place (Fig. 11). These compression zones are forming between the head joints.

Fig. 10 Diagonal tension specimen [15]

Fig. 11 Minimum principal stress contours obtained at different load stages using Abaqus

As opposed to the ungrouted masonry panel, when a diagonal crack initiates within an infilled-frame, it can still carry load due to the partial confinement that the frame offers to the infill panel. This partial confinement prevents sudden failure of the panel and provides more strength and deformation capacity to the masonry panel [40-42]. Separation between the frame and the infill panel reduces the contact length. However, even this confinement would be enough for new diagonal cracks to form within the panel until reaching its ultimate strength (Fig. 12a). [40] reported that the tensile stress is equal to zero at both ends of the tensile diagonal path of infilled-frames and reaches maximum at the middle of the diagonal. This scenario for compressive stress is reverse (Fig. 12b).

[41] altered the length of contact between the frame and the infill by changing their relative stiffness. Results illustrated that the central strain of the infill is not sensitive to the contact length and frame stiffness. This mechanism is very similar to steel plate shear wall with infill and the same results were reported in [43 and 44]. However, as can be seen from Fig. 13, the corners strain is highly affected by contact size and frame stiffness.

Fig. 12 Strut mechanism and stress distribution of infilled-frame [39]

Fig. 13 Compressive strain distribution along compressive diagonal length for different contact size [41]

Three types of crack can be recognized in infilled-frames: crack at the boundaries, diagonal, and toe crushing cracks [40]. Discontinuities between the frame and the infill during the pushover test which occurs close to the tensile zones are called boundary cracks. These cracks form approximately at one tenth of wall strength [40] by referencing the load displacement curve of an infill-frame. These cracks do not have any bearing on the strength of frames and only reduce the initial stiffness of the frame. The force-displacement relationship remains linear after the presence of boundary cracks. Nonlinear-plastic behavior of the infilled-frame begins with forming diagonal cracks at the center of the infill. These cracks initiate at the center of the infill diagonal due to the high tensile stress. Infill cracks are divided into two categories, tensile and shear diagonal cracks. Finally by increasing the load, infill will be crushed at the corners showing dramatically reduced stiffness or forming plastic hinges in the frame. All these mechanisms were also observed in PG walls [2, 15, 16 and 18]. Fig. 14 is depicting the crack propagation and different load-displacement stages for an infilled-frame structure.

Fig. 14 Failure mechanism [40]

PG masonry walls consist of hollow and grouted portions that include reinforcing bar. The behavior of vertical grouted cells and horizontal bond beams (grouted parts) are similar to the reinforced concrete frame, differing in that RC frames have more reinforcement in the concrete elements than grouted masonry elements. Although the strength of reinforced cells of PG walls is not the same as common reinforced concrete frame, the overall behavior of these two systems is alike. Although the behavior of grouted parts of PG masonry and RC frame is similar, there is a distinct difference in the interaction mechanism between a masonry frame and infill with that of infilled-frame. This discrepancy is corresponded to the length of units exceeded from vertical grouted cells. This can be noted from Fig. 15, a stepped-wise surface due to the unit interaction with the infill instead of concrete frame straight plane. Therefore, the interaction mechanism of infilled concrete masonry frame within the PG wall cannot be accurately described by [41] model because of unit interlocking of the frame with the hollow infill. Since grouted frame within the wall confines the hollow masonry infill, no separation can be observed between the two elements. However, the infilled reinforced concrete frame behaves the same as predicted by [41] model because of weak interaction between the RC frame and the masonry infill due to the low cohesive bond and friction.

a) Grouted concrete masonry
frame as a part of the wall

b) Reinforced concrete frame

Fig. 15 Interaction of infill with frame

To investigate the infilled-frame mechanism in PG walls, a simplified micro model of PG walls was tested experimentally by [17 and 18] and built in *Abaqus*. A simplified 3D micro model was employed to build the model by considering the units, the grout and their interfaces. Masonry units and grout are modeled as solid blocks and solid concrete, respectively. In order for simplicity, only the thickness of units' face-shells was considered when modeling the hollow units in the panels. Details of the model are presented in Table 4 and Fig. 16. Additionally, mortar joints and blocks are combined into a homogeneous unit material using *Abaqus* concrete damage plasticity (CDP) model. Interface elements were also employed to model the discontinuity between the hollow units using cohesive surface-based behavior. This model was verified by the experimental results of grout, unit, mortar and hollow and grouted masonry assemblages [17]. It is worth mentioning that the model was able to successfully capture the stress and strain curve, crack pattern and strain

- 1 distribution of hollow and grouted assemblages. As a result, the same model was employed
- 2 in the current study for modeling the wall specimens.

Fig. 16 Model detail

Table 4 Parameter of the model

Hollow				Grouted			
E GPa	Dilation Angle ψ	f_{b0}/f_{c0}	Viscosity Parameter	E GPa	Dilation Angle ψ	f_{b0}/f_{c0}	Viscosity Parameter
26.2	32	1.16	0.001	33.7	34	1.16	0.001
Compressive Behavior		Tensile Behavior		Compressive Behavior		Tensile Behavior	
Yield Stress MPa	Inelastic Strain	Yield Stress MPa	Cracking Strain	Yield Stress MPa	Inelastic Strain	Yield Stress MPa	Cracking Strain
13.8	0	0.64	0	17.4	0	1.5	0
17.2	0.00012	0.62	0.00006	20.7	0.0003	1.15	0.00029
17.9	0.00032	0.34	0.00028	21.6	0.0005	0.83	0.00044
13.8	0.00057	0.21	0.00045	20.9	0.0010	0.3	0.00092
7.6	0.00131	0.12	0.00079	19.6	0.0015	0.1	0.00300
4.4	0.00191	0.05	0.00139	15.1	0.0029	0.06	0.00450
2.2	0.00245	0.03	0.00299	10.3	0.0045	0.05	0.00530
1.0	0.00295	0.02	0.00349	2.0	0.0099	-	-

Fig. 17 shows a comparison between the experimental and numerical results regarding crack pattern and load displacement relationship for the singly grouted wall specimen (please refer to [18] for more details). Before forming cracks at the edges of the

1 wall, stepped-wise cracks initialized at the center of the hollow panels due to the shear
2 action. The size of the cracks at the middle of the hollow panels increased as the lateral
3 imposed displacement increased. Higher tensile stress concentration with increased load
4 caused these cracks to widen diagonally to ultimate failure of the infill masonry. With
5 increasing the lateral load cracks developed at the edges of the hollow panel, close to the
6 intersection between the middle bond beam and the vertical grouted cells. Finally, these
7 cracks propagated in the grouted cells because of weak connection (using a single bar in
8 the wall) between the lower bond beam and the vertical grouted cells. As a result, no
9 coupling took place and a large portion of the load was transmitted directly through the
10 frame to the wall toes causing toe crushing.

11
12 The same mechanism was observed in the numerical model. The minimum principal
13 stress of the wall at different stages of the test is plotted in Fig. 17. As shown in Fig. 17i,
14 wall behaves elastically up to point d and reaches to the maximum load at point f, test was
15 stopped by criterion that the degradation of load fell below 80% of the maximum load
16 (point g). It can be seen that in the elastic zone both the frame and the infill contributed in
17 load carrying by showing stress distribution at the edges of the frame and forming uniform
18 struts in the infill (Figs. 17a to 17d show how compression struts are forming). At the
19 maximum load, five struts in each panel and the frame were carrying the load, as shown in
20 Fig. 17f. Additionally, plastic hinges began to form in the frame. At the failure point, the
21 number of struts become less by forming diagonal cracks in the panels and these strut
22 mechanisms cannot transfer the load anymore.

a-g) Minimum principal stress at different stages of applying load, h) Experimental final crack pattern

Fig. 17 Numerical and experimental results for singly grouted wall

i) Experimental and numerical load-displacement curves

Fig. 17 Numerical and experimental results for singly grouted wall (contd.)

They completely disappeared at the center of bottom panels with only two struts remaining. At this load level, the frame assumes the role of vanished struts, and as can be seen in Fig. 17g, numbers of plastic hinges form in the frame due to the high load present. Although, the top part of the wall can still carry additional load, the wall fails due to the failure at the bottom of the wall which is attributed to both panel and frame. The numerical results are generally in good agreement with the test results at different stages. Not only are the final results of the test predicted, but the model is able to show the mechanism of stress pattern in the wall. It can be concluded that mechanical properties of the PG wall can be simulated using infilled-frame concept. However, these struts are formed based on the number of restrained half units in the grouted cells, see Fig. 18.

Fig. 18 shows the number of struts schematically that are attributed to the wall load-carrying mechanism at peak load (Fig. 17f). The width of all struts are the same and can be approximately equal to: $w = \frac{L_u}{2} \sin \theta$, where l_u is the length of the unit, as shown in Fig. 18.

Fig. 18 Strut configuration

6. Proposed shear strength expression

Ultimate strength of infilled-frame is a function of two parameters; infill and frame shear strengths. Therefore, the below expression is proposed for calculating the shear strength of PG wall based on the infilled-frame theory.

$$V_n = V_{infill} + V_{frame} \quad (10)$$

$$\text{where, } V_{infill} = V_{masonry} + V_{load} \quad (11)$$

6.1 Infill contribution

As per Fig. 19, this idealization is simple and intuitive to use in conventional computer codes for analysis of frames. Since the masonry infill is idealized as an equivalent strut, the forces in the frame elements bounding the infill must be slightly adjusted. To design the infill, the force in the equivalent diagonal strut should be calculated followed by comparing it with the strength of the infill.

Fig. 19 Idealization of equivalent strut

As discussed earlier, three struts are formed while the wall reaches at its peak. These struts as presented in the numerical modeling are formed based on the compression stress flow in the infills. Compression struts in the wall are idealized with three uniform

struts as depicted in Fig. 17b. The middle strut is the only one which reaches its ultimate strength and as seen in Fig. 17g, this strut disappears after the peak stage. Since the other struts are functional even after the peak, instead of considering three struts, one equivalent strut has been assigned to each panel, shown in Fig. 20c, for the analytical purpose and simplification of the shear expression. The horizontal components of these struts would be equal to the shear strength of each panel.

Fig. 20 Idealization of equivalent strut

Infills have been shown by experiment to have three possible failure modes: local crushing of the infill at the corner, mixed-mode failure caused by racking deformation of

the infilled bay, and a shearing failure of the infill along a bed joint. Since the effect of confinement was considered in forming different struts, even after they cracked, the shear failure strength of diagonal tension specimens is considered here as the ultimate strength of struts (Eqs. 13 and 14). Shear cracks in masonry without shear reinforcement cannot be tolerated, propagating into compression faces and reducing effective compression areas which are meant to resist shear. Therefore, the nominal shear stress at which diagonal tension cracks form and propagate can be written as $V_c = 2A\sqrt{f'_c}$ [45].

$$\tau_{DT} = 2\sqrt{f'_m} \quad (12)$$

$$f = 2A_{strut}\sqrt{f'_m} \quad (13)$$

where, A_{strut} is wt_{eff} . Where, t_{eff} is the effective wall thickness, for example in a fully grouted wall this is equal to the total width of unit, however in the hollow masonry it would be equal to thickness of faceshells. Therefore, the horizontal component of shear strength in each panel can be calculated as below:

$$V_{masonry} = 2wt_{eff}\sqrt{f'_m}\cos\theta \quad (14)$$

After crack initiation the panel can still carry load due to friction. The Coulomb's law expression for shear strength due to friction has been implemented here, however, the cohesive term will be ignored due to the presence of diagonal crack in the panel. Therefore, the extra shear strength due to the presence of axial load can be written as;

$$V_{load} = \mu P_{infill} \quad (15)$$

where, P_{infill} is defined based on the relative stiffness between frame grouted cells and infilled components and the vertical component of equivalent struts (Eq. 15). This is

associated to the ratio of panel area to that of grouted cells. It should be noted that only half of each grouted cell located in the center is involved for a single panel.

$$P_{infill} = \frac{P * A_{infill}}{(n-1) [A_{infill} + 1.5 A_{vertical\ grouted\ cells}]} + \frac{wt_{eff} \sqrt{f'_m} \sin \theta}{(n-1)} \quad (16)$$

$$A_{vertical\ grouted\ cells} = A_{grouted\ cells} + A_s \left(\frac{E_{steel}}{E_{grout}} - 1 \right) \quad (17)$$

Finally, the shear strength of each panel is driven by summation of $V_{masonry}$ and V_{load} as following;

$$V_{infill} = 2wt_{eff} \sqrt{f'_m} \cos \theta + \mu P_{infill} \quad (18)$$

6.2 Frame contribution

No base moments are considered for the frame due to the use of the single bar in the grouted cells as a common detail configuration of PG walls. The shear strength that frame can obtain is considered only based on the plastic hinges forming in the vertical grouted cells. There are two scenarios concerning to infilled-frame strength; weak frame-stiff infill and strong frame-weak infill. Weak frame-stiff infill mechanism due to the small amount of reinforcement in grouted cells and bond beams was observed in the numerical and experimental study as discussed earlier (Fig. 21 indicates the number and location of these mechanisms). The shear resistance of the frame in each panel, can be expressed as following expression by taking moment about point A in the column segment AB :

$$V_{panel} = \frac{4M_p}{h} \quad (19)$$

So the total shear strength of the frame would be;

$$V_{frame} = \frac{4(n-1)M_p}{h} \quad (20)$$

Fig. 21 Failure mode shapes, location of plastic hinges and force diagram for the frame

where, h is the space between the first two horizontal bond beams at the bottom of walls as shown in Fig. 22 and also $n (\geq 2)$ is the number of vertical grouted cells (piers) and will be discussed in the next part for generalizing the expression for different PG masonry configurations.

Fig. 22 Definition of h and n for different PG masonry wall configurations

PG walls can be constructed using different grout/reinforcement spacing (s) as seen in Fig. 22. Therefore, n can be calculated as:

$$n = \frac{L}{s} + 1 \quad (21)$$

where, L is the wall length and s is grout spacing. Therefore, the general shear expression can be presented as:

$$V_n = (n - 1) [wt_{eff} \sqrt{f'_m} \cos \theta + \mu P_{infill} + \frac{4M_p}{h}] \quad (22)$$

It is to be noted that the proposed expression is best applicable to walls with vertical and horizontal grout spacing more than 1.2 m. While walls with grout spacing less than 1.2 m acting more like a monolithic wall, TMS (2016) expression is good enough to be used.

7. Parametric study and model verification

The model was used to predict the shear strength of three walls that were tested. Data needed for calculating the shear strength of these walls are presented in Table 5. The shear strength of walls is calculated based on the Eq. 21. P_{infill} is calculated followed by procedure in Eqs. 15 and 16, it can be seen that wall DR infill carries 17% load less than wall SR. In another word, frame taking more axial load by grouting the double cells instead of one. Model predicted the shear strength of SR, DR and SRO walls of 160 kN, 207 kN and 232 kN, respectively. The model showed 5%, 6% and 15% errors in predicting the shear strength of these walls compared to the experimental values. This excellent agreement clearly demonstrates adequacy and efficiency of the analytical model used in this study.

In order to investigate the effect of different parameters presented in the Eq. 22 on the shear strength formula, a simple parametric study is reported here. To do so, the effects of parameters; n , f'_m , P and M_p are studied. In each try, all other numbers were considered constant using 194 mm, 65 mm, 0.7, 0.25 and 3.0 m values for w , t_{eff} , $\cos\theta$, μ and H , respectively and results are plotted in Fig. 23. It is notable that n and μ as expected, have a considerable effect on the shear strength formula and doubling the number of vertical grouted cells almost doubled the shear strength of the wall. Frame (reinforced grouted vertical cells) plastic moment, having a direct effect on the shear strength is the next important parameter. Results showed that increasing the size or number of vertical grouted cells increases the shear strength that frame can carry. Consequently, the shear strength of the wall increases. However, masonry compressive strength and axial load parameters show insignificant effects.

1 **Table 5** Summary of data used in the proposed model for SR and DR walls

ID	P	Grouted cells	Area vertical Grouted cells	infill	μ	n	f'_m	h	t_{eff}	α	As	f_y	w	M_{pc}	P_{infill}
	kN		mm ²		-	-	MPa	mm	mm		mm ²	MPa	mm	MN.mm	kN
SR	58	37,636	113,000	35,436	0.25	3	15.3	1,627	66	45	258	414	194	8.1	28.1
DR	50	75,272	226,000	31,015				1,424			284			8.7	23.2
SRO	213	37,636	113,000	35,436		4		813			258			8.1	52.1

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(a) Number of vertical grouted cells

(b) Masonry compressive strength

(c) Axial load

(d) Plastic moment

(e) Coefficient of friction

(e) Effect of θ

Fig. 23 Parametric study

The shear strength of database was evaluated using the proposed model and plotted in Fig. 24, in addition to shear strength predicted by TMS (2016) expression. In the most cases, the model predicts the exact value of wall shear strength while in other cases model tend to be more conservative.

Fig. 24 Measured vs. nominal shear strength of database, TMS (2016) expression and proposed model

To see the different predicted shear strength given by the different equations, results of some of these equations are presented in Fig. 25. All equations have been applied on the data base presented in this study. Results showed that the proposed equation can accurately predict the shear strength of walls. However, the rest of the equations are unconservative. Results presented here are also in agreement with results reported in [16] that performed a similar analysis.

c. 25 Measured vs. nominal shear strength of database, TMS (2016) expression and proposed model

8. Conclusions

Results of this research revealed that even implementing γ factor in the TMS (2016) expression has failed to solve the fundamental problem of the code shear expression. The current study introduced a new and more realistic approach to the shear strength mechanism of PG walls through adopting an infilled wall model as opposed to the monolithic wall model that already underpins the problem with the TMS (2016) expression. A new expression, therefore, was developed for PG walls based on infilled-frame mechanism. It was concluded that shear mechanism of PG wall can be simulated using the infilled-frame concepts. This implies that the shear strength of PG walls is equal to the summation of the masonry infill and the frame shear strength. The infill masonry can be modeled by an equivalent strut in the wall. The shear strength the frame can obtain was

considered based on the plastic hinges forming in the vertical grouted cells. Proposed model was verified by calculating the shear strength of database including walls tested by authors and a good match was observed.

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Abbreviation

ACI	American Concrete Institute
AS	Australian Standard;
ATC	Applied Technology Council;
CSA	Canadian Standards Association;
FEMA	Federal Emergency Management Agency;
MSJC	Masonry Standards Joint Committee;
NEHRP	National Earthquake Hazards Reduction Program;
NZS	New Zealand Standard;
TCCMAR	Technical Coordinating Committee for Masonry Research; and
UBC	Uniform Building Code.

1 Nomenclature

2 The following symbols are used in this paper:

- 3 A_{infill} =area of a single infill panel, (mm²);
- 4 A_n =net cross-sectional area, (mm²);
- 5 A_g =gross cross-sectional area, (mm²);
- 6 A_h =area of single horizontal reinforcing steel bar, (mm²);
- 7 A_n =net cross-sectional area, (mm²);
- 8 A_{strut} =net cross-sectional area of equivalent strut, (mm²);
- 9 A_v =area of longitudinal reinforcement, (mm²);
- 10 $A_{vertical\ grouted\ cells}$ =area of vertical grouted cells including vertical steel, (mm²);
- 11 $A_{grouted\ cells}$ =area of grouted cells without area of steel, (mm²);
- 12 a =depth of compression stress block, (mm);
- 13 b_w =wall width, (mm);
- 14 C_{ap} =coefficient to account the type of masonry used in construction;
- 15 d =distance from extreme compression fiber to centroid of longitudinal tension
- 16 reinforcement, or 0.8 L for walls, (mm);
- 17 d' =distance between wall edge and outermost wall vertical reinforcing steel, (mm);
- 18 d_{eff} =effective depth of section, (mm);
- 19 d_s =actual depth of a member in direction of shear considered, (mm);
- 20 d_y =drift index of yielding point of displacement ductility curve;
- 21 d_u =ultimate drift index for displacement ductility curve;
- 22 f =diagonal force in equivalent strut, (kN);
- 23 E_{steel} =module of elasticity of steel, (MPa);

- 1 E_{grout} =module of elasticity of concrete, (MPa);
- 2 f'_m = masonry compressive strength, (MPa);
- 3 f_{yh} =yield strength of horizontal reinforcing steel, (MPa);
- 4 f_{yv} =yield strength of vertical reinforcing steel, (MPa);
- 5 h =wall height, (mm);
- 6 h_u =masonry unit height, (mm);
- 7 k =ductility reduction factor;
- 8 k_p =coefficient of the effect of flexural reinforcement;
- 9 k_u =reduction factor;
- 10 L =wall length, (mm);
- 11 L_{dh} =development length of shear reinforcement, (mm);
- 12 M_p =plastic moment, (N.mm)
- 13 M/VL = aspect ratio;
- 14 n =number of vertical grouted cells
- 15 N^* = factored axial compression load, (kN);
- 16 P =axial load, (kN);
- 17 P_{infill} =axial load in the infill panel, (kN);
- 18 s_h =spacing of horizontal shear reinforcement, (mm);
- 19 t_{eff} =effective wall thickness, (mm);
- 20 V_n-A =shear strength provided by the first term of MSJC (2013), (kN);
- 21 V_n-B =shear strength provided by the second term of MSJC (2013), (kN);
- 22 V_n-C =shear strength provided by the third term of MSJC (2013), (kN);
- 23 V_{exp} =experimentally measured shear strength, (kN);

- 1 V_{frame} =shear strength provided by grouted frame, (kN);
- 2 V_i =initial shear strength, (kN);
- 3 V_{infill} =shear strength provided by infill, (kN);
- 4 V_m =shear strength provided by masonry, (kN);
- 5 V_n =nominal shear strength, (kN);
- 6 V_p =shear strength provided by axial load, (kN);
- 7 V_r =residual shear strength, (kN);
- 8 V_s =shear strength provided by shear reinforcement, (kN);
- 9 V_u =ultimate shear force, (kN);
- 10 V_{bm} =basic shear stress provided by masonry, (MPa);
- 11 V_m =shear stress provided by masonry, (MPa);
- 12 w = unit width, (mm);
- 13 w_s =strut width, (mm);
- 14 τ_{DT} = shear stress of diagonal tension specimen, (MPa);
- 15 v_n =total shear stress corresponding to V_n , (MPa);
- 16 $v_{n(\text{max})}$ =maximum permitted total shear stress, (MPa);
- 17 v_p =shear stress provided by axial compression stress, (MPa);
- 18 v_s =shear stress provided by shear reinforcement, (MPa);
- 19 α =angle formed between centers of load application and reaction;
- 20 γ =factor concerning the type of grouting;
- 21 γ_g =factor for partially grouted walls;
- 22 δ =factor concerning loading method;
- 23 μ =coefficient of friction;

- 1 μ =ductility level;
2 μ_{Δ} =displacement ductility;
3 ρ_h =ratio of shear reinforcing steel;
4 ρ_v =ratio of vertical reinforcing steel;
5 ρ_{ve} =ratio of outermost wall vertical reinforcing steel;
6 $\rho_w = \sum A_v / (b_w d)$; and
7 σ_n =axial stress, (MPa).
8

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