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ORIGINAL ARTICLE

Partially grouted concrete masonry shear walls subject to in-plane shear load: a critical review

Paredes de alvenaria de concreto parcialmente grauteadas submetidas a cargas de cisalhamento no plano: uma revisão crítica

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Abstract: Although structures have been built from masonry for many years, little is still known about the behaviour of a wall subject to in-plane shear. This is particularly true of partially grouted concrete masonry. The factors that are known to affect the strength of this type of masonry are discussed with the varying results and interpretations highlighted. There is consensus that increasing axial stress increases the shear strength and reduces the ductility of the masonry. However, whether reinforcement (both horizontal and vertical) contributes to strength remains an issue of debate, as is the effect of aspect ratio. Most codes and standards do not differentiate fully grouted from partially grouted masonry, often over predicting the shear strength of the latter. Wall versus panel failure is not considered. Much work needs to be done to improve our understanding of this material subject to in-plane shear.

Keywords: concrete masonry, partially grouted, in-plane shear, strength, failure mode.

Resumo: Embora as estruturas sejam construídas em alvenaria há muitos anos, pouco ainda se sabe sobre o comportamento de uma parede sujeita a cisalhamento no plano. Isto é particularmente verdadeiro para alvenaria de concreto parcialmente grauteado. Os fatores que sabidamente afetam a resistência deste tipo de alvenaria são discutidos destacando as variações de resultados e interpretações. É consenso que o aumento da tensão axial aumenta a resistência ao cisalhamento e reduz a ductilidade da alvenaria. No entanto, se a armação (horizontal e vertical) contribui para a resistência permanece uma questão de debate, assim como o efeito da relação de aspecto. A maioria dos códigos e normas não diferencia alvenaria totalmente grauteada de alvenaria parcialmente grauteada, muitas vezes superestimando a resistência ao cisalhamento desta última. A ruptura de parede versus painel não é considerada. Muito trabalho precisa ser feito para melhorar a compreensão deste material sujeito a cisalhamento no plano.

Palavras-chave: alvenaria de concreto, parcialmente rebocada, cisalhamento no plano, resistência, modo de falha.

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1 INTRODUCTION

Although masonry has been used for centuries, the structural behaviour of masonry is still not understood thoroughly by researchers and engineers. Within a building, shear walls are structural components that resist in-plane lateral load. Concrete masonry shear walls can be categorized according to different grouting and reinforcement conditions: unreinforced masonry (URM), partially grouted masonry (PGM), and fully grouted masonry (FGM). PGM shear walls are the focus of this review, including the modes of failure, parameters that can impact the masonry shear capacity, and

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equations for estimating that shear capacity. In addition, brief discussions on the scaling effect and ductility of masonry are provided.

2 MODES OF FAILURE

There are three commonly recognized independent failure modes for masonry shear walls, namely sliding failure, diagonal shear failure, and flexural / rocking failure [1], as in Figure 1. These failure modes can happen independently or in combinations of any of the two modes, or even all three modes together [2]. The transition points from one failure mode to another are not yet understood, but some parameters have been found to influence the transition or combination of failure modes, such as reinforcement, axial load, and wall aspect ratio [3], [4], [5], [6].

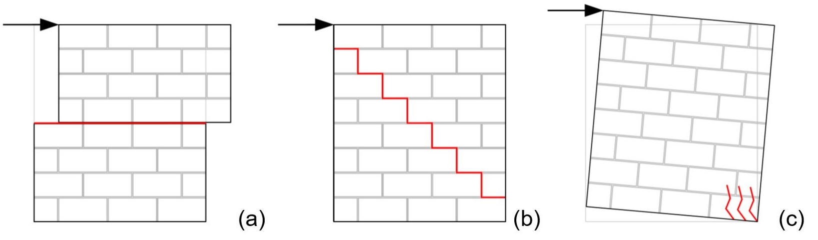

Figure 1. Modes of failure: (a) sliding; (b) diagonal shear; (c) flexural / rocking.

Sliding failure normally occurs along horizontal bed joints, and thus the part of wall above the horizontal crack slides relative to the part below [1]. Gao and Zhai [7] reported that the compressive strength of the mortar and the applied axial load could impact the shear strength of sliding failure, and that the higher these two components were, the higher the shear strength, which could be higher than the strength of the wall in the diagonal failure mode. Diagonal shear failure refers to the step-like cracking pattern that runs diagonally across a wall, with the cracking typically in the head and bed mortar joints but sometimes through the masonry units [1]. Voon and Ingham [3] observed that diagonal cracking started with tension splitting cracks in the compression strut in the wall, so major cracks would develop along the diagonal. The flexural / rocking failure mode is characterized by crushing of the toe and lifting of the heel of the shear wall [1], and this mode can also be accompanied by yielding of steel reinforcement in the wall. Therefore, the flexural failure mode has higher deformation capacity, and can be considered as more ductile than when a wall fails in the diagonal shear mode [4].

When there is an outer frame or confinement, such as reinforced concrete confinement or grouted cores and bond beams, the wall could be seen as a larger wall system consisting of multiple panels in the wall frame [8]. In realistic engineering designs, there could be multiple bays and stories of walls in a structure. Such a shear wall can fail in either wall action, in which the cracks propagate through panel to panel continuously, or in panel action, where the cracks only exist in individual panels separately, as in Figure 2.

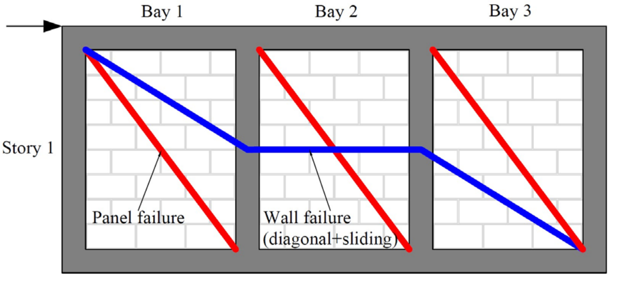

Figure 2. Panel failure and wall failure.

A unique failure mode was reported by Gao and Zhai [7]. They subjected a 2000 x 1300 x 240 mm brick wall with two reinforced bond beams to cyclic loading and observed failure in the lowest panel only, which had very low panel aspect ratio. A combined sliding and diagonal shear failure was observed in their experiment, as in Figure 3. Their other wall specimens failed at the bond beam and panel interfaces or had more vertically angled cracks from the upper panel to the lower panel.

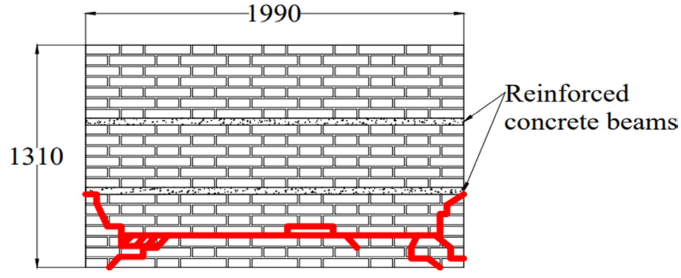

Figure 3. Failure of plain wall with reinforced concrete beams [7].

3 PARAMETERS THAT INFLUENCE SHEAR STRENGTH OF MASONRY WALLS

There are many parameters that have been observed to influence the shear strength of masonry walls. These parameters can be grouped into two categories: material variables such as grouting, mortar, reinforcement; and structural variables such as aspect ratio, axial load, detailing [5], [9], [10], [11]. The variability in masonry causes difficulty in determining whether there is a correlation between the parameter and shear strength or whether the apparent effect was just a random coincidence. In addition, many of the early studies did not have enough experimental group repetitions to validate conclusions statistically, and/or variables were not isolated – for example, changing the horizontal and vertical reinforcement ratios simultaneously, or changing one of these and not accounting for the additional grout so one doesn't know if it was the additional grout that made the difference or the additional steel. The lack of rigour in many studies thus means that the prediction of masonry shear wall performance often has many limitations.

Much of the prediction of masonry shear wall behaviour started from masonry beams [12], which were assumed to have some similarities with reinforced concrete beams. Some early studies had the shear load applied vertically on beams, and walls were thought of as beams rotated through 90 degrees, so the shear load was equivalently applied horizontally. If masonry were homogeneous and isotropic, this assumption might be correct, if shear walls were deep beams. However, masonry is heterogenous and anisotropic, so the analogy of simply rotating from a beam to a wall may not be accurate.

3.1 Mortar Joints

Mortar plays a critical role in masonry as it binds units together. Mortar normally consists of cement, lime, sand, and water, so different proportions of each component create mortar pastes with different properties which in turn influence the performance of masonry. For example, the more coarse the sand, the higher the compressive strength and tensile bond strength [13]. Curing mortar by covering the masonry with sheets could increase its strength, but this is not very applicable in actual construction.

Whether or not the mortar joints contribute to the ultimate shear strength of masonry, either for small specimen tests or grouted walls, remains controversial. Some researchers found that there was no correlation between the compressive strength of the mortar and the shear strength of the masonry [6], [14], [15], while others found that while higher mortar strength increased the masonry shear strength, the increase was not very significant [7], [16], [17].

3.2 Grouting

Grouting is used in conjunction with reinforcement to connect the reinforcement to the masonry. The impacts of the two materials on shear capacity was not examined separately in some cases, so the researcher might change the reinforcement ratio whilst also changing the grouted area [3]. The strength of the grout, size of aggregate and area of grouting can affect grout performance. Numerical modelling shows the spacing of vertical and horizontal grout has a non-linear relationship with the shear capacity of a wall [18]. As for the net area of PGM, the TMS 402/602-22 standard [19] specifies that the web of a concrete masonry unit and the void space between two units should be accounted for [10]. However, there is not always grout on the web, so a slightly larger net area could result in a lower shear stress.

Since grout strength can be lower, equal to, or higher than the masonry unit strength, results from each case can be quite different. If the grout strength is higher than the strength of the masonry unit, the resulting compressive strength of the grouted masonry can be much higher than that of the ungrouted masonry [20]. If the grout strength is lower or equal to the unit strength, the maximum shear load of PGM can be higher than the maximum load for the equivalent URM, but the shear stress is lower [3], [21], [22]. Cracks normally start where the effective width is reduced, such as in the ungrouted cores or in the mortar joints between the ungrouted and grouted cores.

3.3 Masonry Compressive Strength

Research on concrete beams showed the ultimate shear strength of a reinforced concrete beam is positively related to the square root of the compressive strength of the concrete [23]. The same was deemed to apply to masonry. Drysdale et al. [14] were one of the earliest to normalize the shear strength of brick masonry by its compressive strength. A non-linear increasing trend between the compressive strength and shear strength of masonry was observed by Mastumura [24], [25], and a linear trend was developed between the square root of compressive strength and shear strength by Janaraj and Dhanasekar [8]. Others found that the increasing trend has certain limits: for example, shear load capacity increased by 15% with a 40% increase in compressive strength of PGM walls (from 14.75 MPa), but a further increase in the masonry compressive strength did not affect the load capacity [11].

3.4 Reinforcement

Horizontal and/or vertical steel reinforcement is often used in masonry shear walls to increase their ductility. It is still controversial as to whether these reinforcements contribute to the shear strength of a masonry wall. Some researchers claim that the reinforcement did not yield [25] or did not contribute to wall stiffness until cracking [6], while others reported that the reinforcement yielded before cracking [4]. One of the common findings from the literature is that researchers tended only to compare walls with various type of vertical reinforcement embedded in grout, or horizontal reinforcement together with vertical reinforcement, but neglected to compare their results to walls with grout only or without any reinforcement. Therefore, the impact of each component was unclear, i.e., the variables were not controlled effectively. Discussion is included separately for horizontal and vertical reinforcement in the following sections.

3.4.1 Horizontal reinforcement

Horizontal reinforcement could be in various forms. Continuous steel bars can be placed in the masonry courses with grout to form a bond beam, and the ends of the horizontal steel could be bent to 90-degree or 180-degree hooks. Joint reinforcement, another type of horizontal reinforcement, is normally thinner ladder steel or straight bars placed in the mortar or on top of the mortar between courses. Joint reinforcement can be mounted easily and has better performance in crack control, ductility, and energy dissipation than continuous steel bars [26], [27], [28]. Oan [9] tested 66 squat walls with repetitive specimens in each group. Statistical analysis of the results showed that changing the type of horizontal reinforcement did not have significant impact on shear strength or energy dissipation.

The ratio of horizontal reinforcement to the area is one of the key factors examined. Some studies showed that increasing the reinforcement ratio could increase the ultimate shear capacity [3], [24], [25], [29], while others claimed that there was a threshold for the impact [20]. Researchers mostly examined the performance of horizontal reinforcement on squat or square walls, seldom considering the effect on walls with aspect ratio greater than 1. On the other hand, several researchers have reported that horizontal reinforcement can only be activated after cracking, and thus such reinforcement improved the post cracking performance and ductility of walls, which could cause change from the brittle failure mode to a more ductile failure [4], [5], [6], [16], [20], [30], [31]. It is only the steel in the middle third or so of a square wall that can contribute to ductility as it is only that steel which will have sufficient embedment in the wall to develop tension: bars at the top and bottom of the wall where the cracking is towards the corner will have insufficient development length on the short side of the masonry post-cracking. A statistical analysis showed that neither strength nor area of horizontal reinforcement had impact on masonry shear wall capacity [17]. Some researchers claimed that masonry and horizontal reinforcement had a changing proportion and rate of contribution at different phases in the load-displacement relationship [2], [32]: cracking could decrease the capacity of masonry to carry load, so tension could build up in the steel. The concept of energy dissipation, which is the area under load-displacement curve during cyclic-loading, was used to compare the effect of different levels of horizontal reinforcement: changing the reinforcement ratio did not have significant impact on energy dissipation in shear walls [30].

Since reinforcement can be of different sizes, for the same reinforcement ratio, the choice can be a few bars of large diameter or more bars with smaller diameter. The latter was found to result in smoother strength degradation and thinner but a larger number of cracks than the former [3], [31]. The distribution or spacing of horizontal reinforcement was also an important factor. As the grouted bond beams with horizontal reinforcement could be considered to separate the wall into multiple panels, the angle of a diagonal crack varies with the spacing [22].

3.4.2 Vertical reinforcement

The impact of using vertical reinforcement on the shear strength of a masonry wall is also controversial. Early studies showed that increasing the ratio of vertical reinforcement could increase the ultimate shear capacity [6], [33], [34], but some researchers did not differentiate the effects of the additional grout vis-a-vis the reinforcement. Others therefore claimed that there was no correlation between vertical reinforcement and the shear strength of masonry walls [2], [5]. Oan and Shrive [5] also explained that the material and structural contribution of reinforcement should be examined separately. When the vertical reinforcement was embedded into the foundation of wall, dowel action would occur during a sliding or diagonal failure, so the steel would bend near the base. Some studies reported that, similar to horizontal reinforcement, vertical reinforcement was also only activated after cracking, and therefore, the ductility of the wall increased [5], [34], [35]. The use of vertical reinforcement could also shift the failure mode from wall failure to panel failure, as the grout and vertical reinforcement could be considered as a frame to the panels [20].

Changing the horizontal spacing of vertical reinforcement did not have a significant effect on the peak load or initial stiffness [20], [35], [34]. A smaller spacing could result in a larger number of narrower cracks [20], [26]. Widely spaced reinforced masonry shear walls are defined differently for various standards. For example, the Canadian standard allows up to 2.4 m spacing [36] and the American standard up to 2.44 m spacing [19], whereas the Australian code specifies that “wide” spacing should be within the range of 0.8 m to 2.4 m [37]. Current standards used literature sources primarily of FGM to determine the masonry shear strength, and the equations were simply factored to obtain an estimation for PGM, but the shear strength and distribution of FGM and PGM was different [10]. Therefore, those clauses should be updated.

3.5 Axial stress

It is commonly reported and agreed that increasing the level of applied axial stress in a masonry shear wall increases its ultimate shear strength, aggregate interlocking forces or friction force, and delays crack initiation [3], [5], [6], [9], [38]. A linear relationship, or the Mohr-Coulomb criterion, was observed by many researchers until a certain limit [4], [6], [7], [9], [14], [24], [25], [39], [40], and then the rate of increase changed or the relationship became more curved [14]. Numerical modelling can present the distribution of stress over the wall, and it was found that increasing the axial load changed the principal stresses of the diagonal compression strut, in which the principal compressive stress increased and principal tensile stress decreased [41], [42]. Increasing the axial stress can also result in changing the failure mode from a more ductile failure to a more brittle failure [3], [5], [6], [33], [34], and cause a smoother degradation in stiffness. With low axial load and high shear load, sliding was the dominant mode of failure; whereas high axial load and low shear load on the other hand could create more vertically angled diagonal cracks [9], [15]. For those specimens with intermediate compressive and shear stress, a mix of the two failure modes occurred. When a wall was reinforced with some vertical steel bars, low or zero axial load could result in partial pull-out of vertical reinforcement, so the tensile straining and strain hardening of steel was lowered, and the contribution of the vertical reinforcement to the ultimate shear capacity was also lowered [6].

3.6 Aspect Ratio

The aspect ratio of a wall is the ratio between the wall height and length and was generally found to correlate negatively with the shear strength of masonry walls [3], [7], [8], [24], [25], [40], [42]. The well-understood range of aspect ratio is from 0.5 to 2.0: shear walls with higher aspect ratios have not been studied thoroughly. Matsumura [24], [25] reported a decreasing hyperbolic relationship, while Hamedzadeh [40] concluded a logarithmic equation of shear stress normalized with the compressive strength of masonry represented the effect more accurately. Schultz et al. [30] were the only exception, finding a positive relationship between aspect ratio and nominal shear stress. They tested 6 fixed brick walls under cyclic loading, with aspect ratios of 0.5, 0.7, and 1.0 with the walls having the same height but changing length. A significant increase in shear strength was found when increasing the aspect ratio from 0.5 to 0.7, but no change was found from 0.7 to 1.0. However, they used the same size and number of vertical reinforcing bars in the outermost cells of the walls, so it was unclear that the impact on peak shear load was caused by the aspect ratio or the different ratios and moments of resistance of the vertical reinforcement.

Increasing the aspect ratio creates a larger moment arm for the horizontal load at the top of a wall and thus a larger moment reaction at the bottom of the wall. Increasing the aspect ratio also causes a narrower compression strut at more a vertically oriented slope [41], [42], [43]. Pan [42] modelled the effect of a circular void lying in the compression strut of a shear wall numerically and explained that the wall could experience higher surface stress, so a crack would be very likely to be initiated at the high tensile stress zone at the void.

The aspect ratios of the wall and the panels within it can be examined separately if there are multiple bays and stories [8]. Hamedzadeh [40] examined the performance of half-scale concrete block walls with various aspect ratios. The walls had vertical reinforcement welded to the foundation to create multiple panels. He reported that with high axial load on square and squat walls, major diagonal cracks occurred across the middle vertical reinforcement. If there were three panels, the first and second panels (closest to the point of application of the lateral shear load (top left in Figure 4) tended to fail together while the third panel cracked individually.

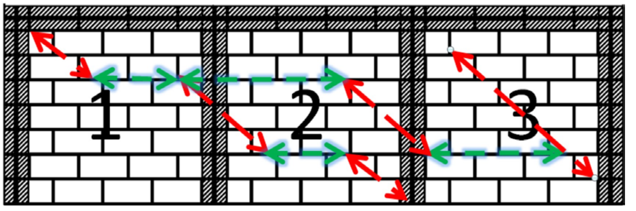

Figure 4. Cracking pattern of multi-panel walls [40].

3.7 Shear Span Ratio

Shear span ratio is a concept that was imported from reinforced concrete beams and applied to masonry beams, and eventually to masonry shear walls. In the beam context, the shear span ratio refers to the ratio of the shear span to the effective depth, shown as a/d [12], as in Figure 5a. In the wall context the concept was rotated 90 degrees from the beam, so the ratio becomes the ratio of the vertical height from the base to the inflection point of the wall to the length of the wall in the shear direction [10]. In some standards, the ratio is often represented as M/Vd, which is the external moment versus shear load times the effective depth, as in Figure 5b. Dillon and Fonseca [10] claimed that this ratio varied with the boundary conditions of tested specimens and that some researchers had mistakenly used the incorrect ratio. However, using the external loads estimated for design to predict strength appears to violate the fundamental concepts of limit states design [44]. Many researchers have tried to prove that there is a relationship between shear span ratio and masonry shear strength. For example, Okamoto et al. [4] reported that increasing shear span ratio could decrease the maximum shear stress but increase the ultimate drift angle of the masonry wall.

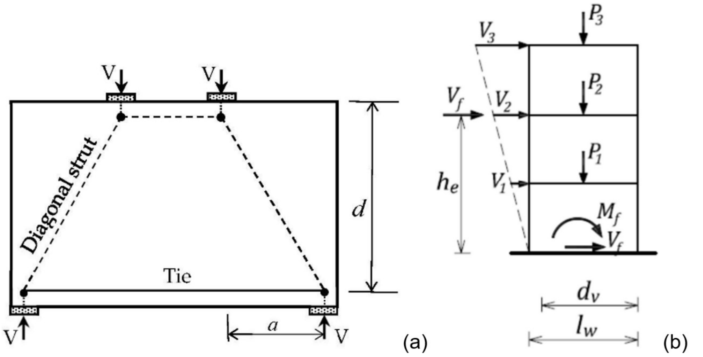

Figure 5. Shear span to depth ratio of: (a) a beam [45]; (b) a wall [46].

3.8 Loading Pattern

Commonly used loading patterns include monotonic, cyclic, and seismic loading [32], as in Figure 6. Monotonic loading is simply a constant rate of increasing load until the specimen fails. Cyclic loading refers to repeated loading at different (typically increasing) amplitudes. Seismic loading is the simulation of past or synthetic seismic activity and could be considered as a type of cyclic loading. The loading pattern and rate will have some influence on the shear capacity, ductility, and strength degradation of a masonry wall [6], [10], [32]. Walls subjected to monotonic loading could have a higher peak shear load than those subjected to cyclic load, since the latter could experience closing and opening of cracks, and the strength of the wall can drop quite significantly after the first loading cycle [10], [33]. Different types of cyclic loading did not result in significant differences in peak load or energy absorption. Therefore, using values obtained from experimental tests should be factored down based on their loading type in the equations for prediction.

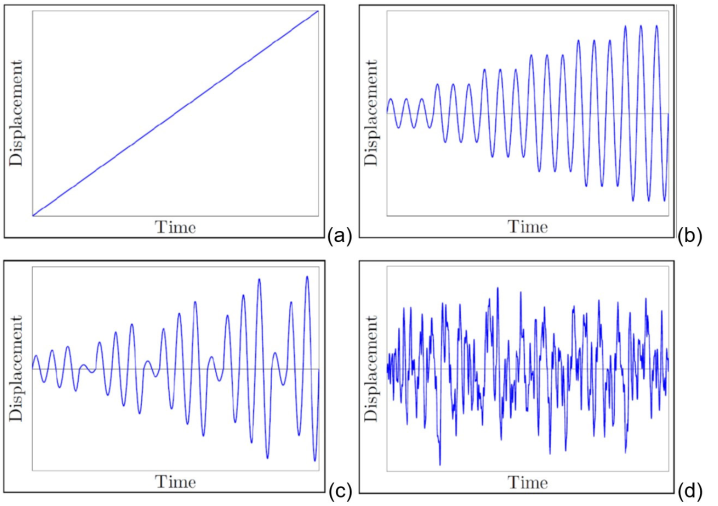

Figure 6. Loading patterns: (a) monotonic; (b) amplified cyclic; (c) sequential-phase cyclic; (d) simulated seismic [10].

Most of the existing studies used one or two actuators on the top of the wall to apply axial load and one actuator on one side of the wall for lateral shear load. Application of a point or distributed load can also create different cracking patterns in the wall [40]. As shown in Figure 7, applying a distributed load can cause the major crack to appear along the diagonal of the shear wall with some minor parallel cracks, but applying a concentrated load can shift the major crack down a couple of courses and cause a slight increase in peak shear load. However, numerical models of the different load applications did not present a distinct change in the compression strut, but the area of extreme compression was slightly wider for those subjected to distributed load [42].

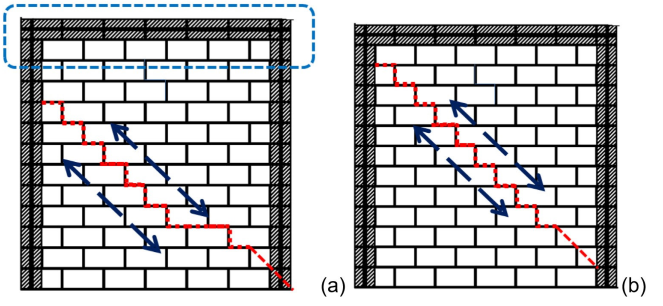

Figure 7. Cracking pattern of (a) concentrated load and (b) distributed load [40].

3.9 Openings

Perforated walls, or walls with window or door openings, have gained more and more attention from researchers [47]. Several studies have considered the effect of the size and location of openings on the behaviour of a shear wall. The presence of an opening can lower the peak load and increase wall deflection [18], [39], [48], [49], [50]. Increasing the size of the opening did not have significant impact on peak load, but could increase the height of the piers, which impacts the wall stiffness and cracking pattern [11], [18], [48], [49], [50], [51]. Increasing the number of stories can also limit the effect caused by openings [18]. Other parameters such as axial load and horizontal reinforcement below the opening can not only increase shear strength, but also alter the cracking pattern of perforated walls [39], [47], [48].

3.10 Confinement

Confined masonry refers to masonry walls with both an infill panel and an outer "frame", such as grouted outermost cores or a reinforced concrete frame. The former is more convenient and economical to construct but the latter had better performance [52]. The confinement can maintain the integrity of a masonry wall, and thus increase the shear capacity, ductility, and energy dissipation [7], [28], [50]. The level of influence of confinement on the peak shear capacity is still controversial, with some researchers reporting that confinement can only contribute after cracking. Thus, this would not have a significant impact in terms of shear strength if net area is taken into consideration [52]. The stiffness ratio between the confinement and the infill panel could alter the failure mode of the wall. For example, similar moduli of elasticity of the frame and infill or high cohesion between them can result in wall action and higher mechanical interlocking forces: in contrast, with frames much stiffer than the panels, panel action or local failure could be observed [31], [34], [42], [53], [54].

4 SCALING

Laboratory experiments are often constrained by space, budget, and testing equipment. Therefore, many researchers have explored the possibility to scale down the size of the specimens tested. For instance, 1/2 scale specimens have been used commonly, and sometimes 1/3, 1/4, or 1/6 scale specimens have been considered. The scaling effect of the material properties is of importance as masonry is an anisotropic and heterogenous material. Researchers have found that with similar density, a similar percentage of the cross-section being solid, and similar absorption rate of the bricks and blocks, reducing the size and volume could result in equal or higher compressive strength of the masonry unit and prism [55], [56], [57], [58], [59]. Some explained that smaller sizes of the masonry units could have a lower possibility and distribution of flaws during the manufacturing processes, and smaller-scale prisms had thinner mortar joints. Other properties such as tensile strength, shear strength and initial stiffness were not different based on size effect. However, one major concern was that the experiments have normally been done with only one type of masonry unit, mortar, or grouting, so the effect of different masonry combinations should be examined. Another issue raised was to relate the properties of prisms to those of walls of various sizes. Changing the overall geometry of the wall or adding vertical constraints to the top of the wall could change the peak shear capacity or stiffness [58], [60]. Reports in the literature generally conclude that using reduced size masonry was mostly in agreement with full scale masonry but results should be given careful consideration.

5 DUCTILITY

The ductility of a masonry wall does not have a strict definition, but ductility is normally used to describe the deformation of wall at different loading stages. Tomaževič and Žarnić [16] extracted a bi-linear load-displacement relationship from the backbone curves using the theorem of energy conservation, and this relationship has been used in many studies and standards. The commonly used ductility index is the ratio of ultimate displacement to the effective yielding displacement [61], but the definition for the displacements is varied. For example, the effective yielding displacement has been defined as 40% of the displacement at peak load [62], or the displacement at the intercept of the initial stiffness line with a horizontal line through the peak load [61]. The ultimate displacement could be taken as the displacement of the wall corresponding to the load dropping to 80% of the peak load [61], or 67% [27] or 75% [30]. Except for the bi-linear models, the tri-linear approach is also considered by some researchers [34], which accounts for yielding, peak, and post-peak performance, as shown in Figure 8. These methods are all idealized load-displacement responses of masonry shear walls, so researchers could choose the approach that best represents their experimental results for further analysis or numerical modelling.

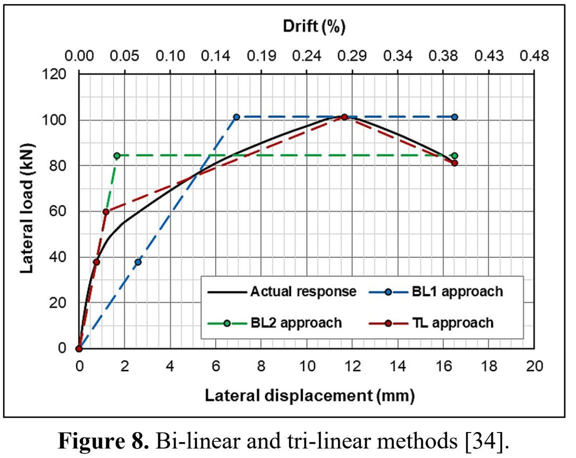

Figure 8. Bi-linear and tri-linear methods [34].

6 STANDARDS AND CODES OF PRACTICE

The estimation of in-plane shear capacity of masonry walls varies for different countries or regions. In this section, some published standards are included and compared with each other.

6.1 Canadian CSA S304-14

The Canadian Standard CSA S304-14 [36] defines the in-plane shear capacity separately for diagonal shear, as Equation 2, and sliding shear failure, as Equation 3. For the diagonal shear equation, three components are considered: masonry shear strength, axial load, and the contribution from the reinforcement. In addition, the term γ_g is a factor to account for the grouting condition. The shear span term was included to determine the shear strength of the masonry, as Equation 1, but including factored loads in the resistance side of a limit state is contradictory to the principle of ultimate limit states design [44]. Many researchers who have examined their own experimental results and other results reported in the literature found that the value predicted by the standard had large variation [8], [9], [18], [40], [43], [63], and that the factor 0.25 for applied axial load could be unconservative at very high or very low axial loads [8]. Some researchers have argued that the standard was developed from FGM, so cannot predict the capacity of PGM accurately, especially with all the different possibilities for PGM (grout and bar spacing, panel failure, etc.) [10], [18].

$$v_m = 0.16\left(2 - \frac{M_f}{V_f d_v}\right)\sqrt{f'_m} \text{ and } 0.25 < \frac{M_f}{V_f d_v} < 1 \tag{1}$$

$$V_r = \phi_m(v_m b_w d_v + 0.25P_d)\gamma_g + \left(0.60\phi_s A_v f_y \frac{d_v}{s}\right) \leq 0.4\phi_m\sqrt{f'_m} b_w d_w \gamma_g \tag{2}$$

$$V_r = \phi_m \mu C \tag{3}$$

6.2 American TMS 402/602-22

There are similar components in the American TMS 402/602-22 [19] as in the Canadian standard, but there is only one equation for all failure modes, as Equation 4. Although in TMS 402/602-22 there is no stand-alone clause for masonry shear strength, a similar shear span term is included in the shear capacity equation. This standard is developed from the MSJC standards [64], for which the only difference is that the coefficient for masonry contribution changed from 0.083 to 0.042. Studies have found that the MSJC clause significantly overestimates the contribution of horizontal reinforcement and the nominal shear load, especially for PGM and walls with openings [8], [18], [20], [43], [63].

$$F_v = \left\{0.042\left[\left(4.0 - 1.75\left(\frac{M}{V d_v}\right)\sqrt{f'_m}\right)\right] + 0.25\frac{P}{A_n} + 0.5\left(\frac{A_v F_s d_v}{A_{nv} s}\right)\right\}\gamma_g \tag{4}$$

6.3 Australian AS 3700:2018

Unlike the American and Canadian standards, the Australian standard AS 3700:2018 [37] has separate clauses for plain and reinforced walls. For unreinforced shear walls, Equation 5 includes the shear bond strength and friction strength of the masonry. The shear strength of unreinforced masonry is defined as a factor times the tensile bond strength of the masonry, which are both associated with cohesion: the correlation was verified and considered generally conservative [65]. The default value 0.2 MPa for tensile bond strength could overestimate some masonry combinations. For reinforced masonry, walls with aspect ratios above and below 2.3 are defined differently, and the terms include masonry shear strength and vertical reinforcement, see Equation 6. The clauses were reported to overestimate PGM or multi-panel wall strength significantly [8], [43].

$$V_d \leq V_o + V_l = \phi f'_{ms} A_d + k_v f_d A_d \tag{5}$$

$$V_d \leq \phi\left(f_{vr} A_d + 0.8 f_{sy} A_s\right) for\ H/L \leq 2.3 \tag{6}$$

6.4 Brazilian ABNT NBR 16868-1

In the equation to determine shear wall capacity in the Brazilian standard [66], contributions from the masonry and the horizontal reinforcement are included, as in Equation 7.

$$V_{design} \leq V_a + V_s = f_{vd}bd + 0.75 f_{yd}bd\frac{A_{sw}}{s} \tag{7}$$

The design shear strength, f_{vd}, should be half of f_{vk}, which for unreinforced masonry includes axial load. For reinforced masonry f_{vk} includes the reinforcement ratio.

- $f_{vk} = 0.10 + 0.5\sigma \leq 1.0$, for mortar strength from 1.5 to 3.4 MPa
- $f_{vk} = 0.15 + 0.5\sigma \leq 1.4$, for mortar strength from 3.5 to 7.0 MPa
- $f_{vk} = 0.35 + 0.5\sigma \leq 1.7$, for mortar strength above 7.0 MPa
- $f_{vk} = 0.35 + 17.5\rho \leq 0.7$

6.5 Chinese GB50003-2011

The Chinese standard GB5003-2011 [67] not only specifies equations for unreinforced and reinforced walls, as Equation 8, but also uses two equations for reinforced shear walls subjected to eccentric compression or tension, Equations 9 and 10, respectively. For the unreinforced walls, terms for the masonry contribution and the applied axial load are included. For reinforced walls, the masonry contribution, the axial load, and the horizontal reinforcement are considered. There are different ways to determine the masonry shear strength in the Chinese standard. For unreinforced masonry, values can be found in a table according to the masonry type and mortar grade. For reinforced masonry with concrete units, values depend on the masonry compressive strength.

$$V \leq (f_v + \alpha\mu\sigma_0)A \tag{8}$$

$$V \leq \frac{1}{\lambda - 0.5}\left(0.6 f_{vg}bh_0 + 0.12N\ \frac{A_w}{A}\right) + 0.9 f_{yh}\frac{A_{sh}}{s}h_0 \tag{9}$$

$$V \leq \frac{1}{\lambda - 0.5}\left(0.6 f_{vg}bh_0 - 0.22N\ \frac{A_w}{A}\right) + 0.9 f_{yh}\frac{A_{sh}}{s}h_0 \tag{10}$$

6.6 European Eurocode 2005

The Eurocode [68] also uses different equations for the shear strength of unreinforced and reinforced masonry walls, Equations 11 and 12, respectively. For the unreinforced masonry, the shear capacity of a masonry wall is deemed solely dependent on the masonry contribution, while for the reinforced masonry, the capacity includes contributions from both the masonry and the horizontal reinforcement, and a factor is applied to account for different grouting conditions. The masonry shear strength can be either determined from sample testing, such as the triplet test [69], or using the Mohr-Coulomb criterion with the initial shear strength to determine the characteristic strength. The Eurocode is quite conservative, with an experimental to predicted shear capacity ratio of 2.62 and a 56% COV, but the code can also overestimate some walls in some circumstances [9].

$$V_{Rd} = f_{vd}tl_c \tag{11}$$

$$V_{Rd1} + V_{Rd2} = f_{vd}tl + 0.9A_{sw}f_{yd} \tag{12}$$

7 SHEAR MODELS

As there have been many studies of masonry shear wall capacity completed since the 1970s. Researchers have developed various equations to estimate the in-plane shear capacity of masonry walls with different conditions. Some of the models are included in this section. Some more recently proposed models have not yet been studied by other researchers, so the accuracy and precision of these latter models could be questionable.

7.1 Matsumura (1987, 1988)

Matsumura [24], [25] tested 60 reinforced concrete masonry walls and 30 brick walls and concluded with the following Equation 13 for ultimate shear capacity, including terms for the masonry contribution, aspect ratio, horizontal reinforcement, and axial load. Factors also include the grouting condition and vertical reinforcement. Matsumura was one of the earliest to include the square root of the masonry compressive strength and the horizontal reinforcement in the prediction, and thus influenced many later standards and other models. The ratio of predicted value to experimental value showed both overestimation and underestimation with different grouting conditions, and the horizontal reinforcement term could greatly impact the accuracy [2], [18], [40], [63].

$$V_u = \left(K_u K_p \left(\frac{0.76}{h/d+0.7} + 0.012\right)\sqrt{f'_m} + 0.18\gamma\delta\sqrt{\rho_h \sigma_{hy} f'_m} + 0.2\sigma_o\right) tj \cdot 10^3 \tag{13}$$

7.2 Shing et al. (1990)

Shing et al. [6] tested 22 reinforced masonry walls under both monotonic and cyclic loading and proposed Equation 14 to estimate shear capacity in imperial units. There are terms representing contributions from the masonry, vertical reinforcement, horizontal reinforcement and axial load, as well as the square root of the masonry compressive strength. However, Shing et al. [6] did not test replicate walls, and the accuracy of prediction was of great concern [40], [63].

$$V_n = \left[0.0018\left(\rho_v f_y + \sigma_c\right) + 2\right] A\sqrt{f'_m} + \left(\frac{l-2d\prime}{s} - 1\right) A_h f_y \tag{14}$$

7.3 Anderson and Priestley (1992)

Anderson and Priestley [2] summarized the results of the testing of 69 walls from three sources. All the walls had aspect ratio greater or equal to one, while some had fixed-fixed boundary conditions and others were cantilever wall type tests. The Anderson and Priestly equation (Equation 15) again included terms of the masonry contribution, axial load, and horizontal reinforcement. It was reported that this equation significantly overestimates the capacity of PGM walls [63].

$$V_u = 0.24\sqrt{f'_m} wt + 0.25P + 0.50 A_h f_{yh} \frac{d}{s} \tag{15}$$

7.4 Voon and Ingham (2007)

Voon and Ingham [70] summarized their experimental results from 10 cantilever walls [3] and the same literature resources as Anderson and Priestley [2]. They suggested Equation 16 which includes terms for the masonry contribution, axial load, and horizontal reinforcement. Similar factors as the CSA [36] and TMS [19] standards to account for the shear span ratio were included as $C_b = 0.083[4 - 1.75(M/VL)]$. In addition, there were also factors of vertical reinforcement and ductility included in the equation, in which $C_a = 0.022\rho_v f_{yv}$ and the ductility factor, k, decreased with increasing ductility, as in Figure 9a. Other than the previously mentioned issues of various boundary conditions from the literature, the determination of angle α in the equation was primarily from experimental results for reinforced concrete columns [71], which have much higher aspect ratios than regular masonry shear walls. The definition also varied with single or double curvature, which might be confusing in some cases, as in Figure 9b.

$$V_n = 0.8k(C_a + C_b)A_n\sqrt{f'_m} + 0.9N^* \tan\alpha + A_h f_{yh} \frac{D_{eff}}{s_h} \leq 0.33 A_n \sqrt{f'_m} \tag{16}$$

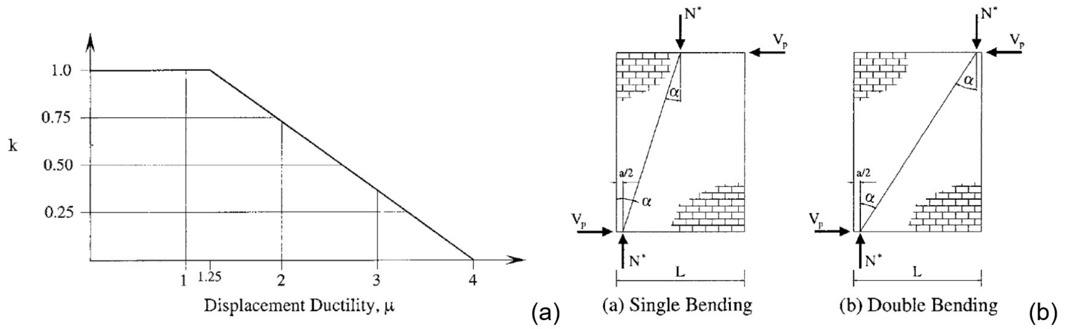

Figure 9. (a) Relationship between ductility and reduction factor; (b) Contribution of axial load to masonry shear strength [70].

7.5 Oan and Shrive (2014)

Oan and Shrive [72] tested 66 masonry walls and used the results of 130 walls from various literature sources to propose a revision to the CSA S304-14 [36] equation for masonry shear capacity. They stated that their proposed equation showed relatively good accuracy and precision to their experimental and the literature results [18]. Their equation including terms accounting for masonry strength, axial load, and vertical reinforcement, as Equation 17. However, they did not change the definition for masonry shear strength, and it was still dependent on masonry compressive strength and the shear span ratio.

$$V_r = \phi_m\left(v_m d_v b_w \gamma_g\right) + 0.27P + \phi_s(0.05A_{sv}f_{yv}) \tag{17}$$

7.6 Dillon and Fonseca (2015)

Dillon and Fonseca [10] completed a meta-analysis of 340 walls from 47 research projects from 1968 to 2010 and concluded that the MSJC equation had acceptable accuracy for FGM but overestimated PGM. The walls tested in the different sources were also subject to various boundary conditions, aspect ratio, axial load, etc. However, the weighting assigned to each study was still based on the terms included in the equation, with the inclusion of some being controversial. Dillon and Fonseca [10] further examined the grouting factor for PGM and confirmed that the current adjustment factor of 0.75 was acceptable, but that a factor of 0.73 would be better [73]. They also confirmed that the strengths of FGM and PGM had different statistical distributions, so it might not be accurate simply to apply a single factor to account for the wall capacity.

7.7 Janaraj and Dhanasekar (2016)

One major difference in the Janaraj and Dhanasekar [8] equation and others was that they ignored the contribution of steel reinforcement and introduced separate terms for panel aspect ratio and wall confinement, as Equation 18. The number of bays was included as n, and number of stories was included as m. The confinement efficiency η was dependent on panel aspect ratio and the type of material in the masonry wall. They used their experimental results of confined masonry walls and literature values from various sources.

$$V_n = \left[0.17\left(2 - 0.9\lambda_p\right)\sqrt{f'_m}A_n + 0.25P\right] \times (0.9)^{m-1} \times \eta \tag{18}$$

7.8 Bolhassani et al. (2016)

Bolhassani et al. [63] proposed a shear capacity equation for PGM with or without a frame, as Equation 19. They used their experimental results and some existing studies. They claimed that this equation predicted the shear capacity more accurately if the spacing of the reinforcement was larger than 1.2 m.

$$V_n = (n-1)\left[wt_{eff}\sqrt{f'_m}\cos\theta + \mu P_{infill} + \frac{4M_p}{h}\right] \tag{19}$$

7.9 Izquierdo et al. (2021)

Izquierdo et al. [17] randomly selected 25% of 292 PGM walls from 27 existing studies as their dataset. They used stepwise regression and examined several equations to predict masonry shear capacity from the statistical analysis. Some of their equations showed that the compressive strength of the mortar had a positive impact on masonry shear capacity, but they omitted those equations because they claimed that it was "unconventional". Izquierdo et al. [17] confirmed that neither the strength nor the area of horizontal reinforcement contributed to the shear strength of a wall. Out of several equations, they selected Equation 20 as their optimal model, which includes wall length, the compressive strength of the grouted masonry, the area and spacing of vertical reinforcement, and the axial load.

$$V_n = 0.0538L + 4.83f'_{mg} + 0.067A_{vf} - 0.0553s_{v,ave} + 0.245P \tag{20}$$

7.10 Medeiros et al. (2022)

Medeiros et al. [18] summarized their numerical modelling results of multi-panel walls with openings and experimental results from various sources and proposed Equation 21, which includes terms of masonry contribution with grouting factors, axial load, and horizontal and vertical reinforcements. For the dataset they utilized, their equation gave the best predictive results, being slightly more consistent than the equations of Oan and Shrive [72], Dillon and Fonseca [10], and Izquierdo et al. [17].

$$V_n = k_{gv}k_{gh}\beta A_{eh}\sqrt{f'_m} + 0.4P_d\tan\theta + 0.02A_v f_{yv}\sqrt{f'_m} + 0.02\rho_h A_{ev} f_{yv}\sqrt{f'_m} \tag{21}$$

8 METHOD OF TESTING

Experimental testing and numerical modeling are both commonly used methods found in existing studies of masonry shear walls. These approaches have both advantages and disadvantages. Testing a wall is normally used to determine the performance, ultimate load and displacement, but construction and handling is very time and budget consuming. Small specimens can be used to reflect some properties, but the properties measured might not represent the actual ones of the wall (because of different boundary conditions, material proportions, construction etc.). On the other hand, numerical modeling can be applied to assess various aspects, but without the input of actual experimental results, the output may not be trustworthy. In this section, some testing methods are discussed.

8.1 Tests for Shear Strength

The triplet test is widely used in Europe to determine the shear strength of masonry [69]. As in Figure 10a, the assembly of a triplet specimen is relatively easy as only three masonry units are required. By supporting the outer units and loading only the middle unit, the mortar joints can experience close to a pure shear load. It is also possible to apply pre-compression to the specimen. The results of many studies involving clay brick specimens generally demonstrate that precompression and shear strength followed a linear Mohr-coulomb relationship [14], [33], [74], [75], [76], [77], but only a few studies have been performed on concrete block specimens (hollow, partially and fully grouted) [78], [79].

The diagonal tension test, as defined by ASTM C1006 [80], is another test to determine the diagonal shear strength of masonry. The specimen is a square wall with dimensions of 1.2 m x 1.2 m: steel loading shoes are placed at each end of the diagonal compression strut, as in Figure 10b and displaced towards each other. The stress distribution induced by the test is different from a typical shear wall in practice, because the specimens can only fail in the diagonal failure mode and suffer from compressive stress concentration around the loading shoes [44], [52], [56], [81], [82]. Moreover, the diagonal tension test is similar to the Brazilian splitting tensile strength test for determining tensile strength of rock samples [83], so the results from this diagonal test may need adjustment before being used as the shear strength of a masonry wall.

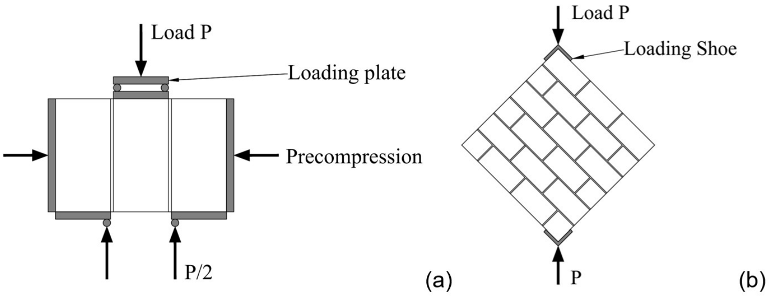

Figure 10. (a) Triplet test; (b) diagonal tension test.

8.2 Numerical Modelling

Numerical modelling is an important and powerful tool to assess structural performance, as it can save time and budget compared to experimental testing. There are three common methods of modelling for masonry structures, namely macro-modelling, simplified micro-modelling, and detailed micro-modelling [40], [41], [42], [84]. Macro-modelling refers to using homogenous material properties and element mesh for the structure, while micro-modelling includes the different properties for each masonry component, i.e., separate sets of properties and elements for unit, mortar, grout, steel, and their interfaces. Simplified micro-modeling treats the mortar and mortar-unit interface as discontinuous elements with units as continuous elements - detailed micro-modelling has only the interface properties as discontinuous elements. There are pros and cons for each method, so researchers should consider which would be more suitable for their project. For example, micro-modelling is helpful to examine local failure of a structure, but macro-modelling could be more applicable for the holistic view of a complex structure. To simulate crack propagation, researchers often use either the discrete or the smeared crack approach [11], [18], [34], [85]. The discrete crack approach allows crack or joint opening from using interface elements, whereas the smeared crack approach displays damaged areas with the distortion or deformation of continuum elements [85]. Researchers should also carefully study mesh sensitivity to make sure that the output is stable and reliable.

9 CONCLUSIONS

Partially grouted masonry (PGM) walls have gained more and more interest worldwide. In this review, parameters influencing the shear strength and capacity of PGM walls that were examined by many researchers were discussed. A few of them have a commonly agreed impact on the strength and stiffness of PGM walls, such as axial load, aspect ratio and loading pattern, but the effects of many parameters remain controversial, such as grouting, reinforcement, and confinement. Standards in different regions have different clauses to estimate the peak shear capacity, and many researchers summarized their findings into their equations. This all shows that a commonly acceptable understanding of the shear behaviour of masonry is deficient. A lot more needs to be learned about the fundamentals of this subject. In terms of experimental testing, reduced sized units are often used due to many constraints, and results are mostly in agreement with full-scale units. A ductility index is a good measurement for the wall performance after peak load. However, the question remains as to how to predict masonry shear strength accurately and precisely, and to convert strength into the masonry contribution component of the shear capacity estimation of PGM. The controversy of the impact of different parameters should be resolved, or at least updated, in the estimation. Furthermore, the transition point of different failure modes or equations to estimate the peak shear load for different failure types should be identified.

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