Design approach of shear strengthened masonry: welded wire meshes, Reticulatus and cementitious plastering methods

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Abstract

Masonry often requires strengthening to withstand against extreme actions such as earthquakes, cyclones and flooding. Recently, new solutions have been developed to strengthen masonry, such as Fabric Reinforced Cementitious Matrixes (FRCM) and Fibre Reinforced Polymers (FRP). However, other strengthening systems such as Welded Wire Meshing (WWM), Reticulatus and Plastering with Cementitious matrixes/mortar (CP) have been also practiced, conversely no systematic design guidelines are available for these methods. In this study, an attempt has been made to establish rational design approaches to predict the shear resistance of WWM, Reticulatus and CP strengthening systems. Three sets of experimental database have been developed for design verification. The effectiveness of these strengthening methods was appraised by comparing their structural performances. The available formulations to predict the shear resistance of unreinforced masonry (URM) and CP strengthened masonry were assessed against the established database, and suitable modifications were proposed to effectively account the contribution of cementitious matrix. A unified approach to estimate the shear strength was proposed based on the contribution of URM, CP and reinforcements. The design approach is shown to conservatively predict the shear strength of reinforced masonry.

1 Introduction

Masonry constitutes a major portion of the world building stock and heritage structures are often built with masonry assemblages. Despite masonry being preferred in construction, it exhibits low tensile strength and deformation capacity. This makes masonry vulnerable to high intensity actions such as earthquakes, foundation settlements and flooding. Structural analysis and vulnerability assessment are important methods to study masonry structures and to define appropriate retrofitting interventions (Valluzzi et al 2005; Ramos et al. 2010; Roca et al. 2010; Thamboo and Dhanasekar 2020).

Depending on the masonry quality and building configuration, the masonry walls in the upper floors are often susceptible to fail under out-of-plane mechanism and the walls at lower levels are prone to fail in in-plane shear. Various strengthening solutions have been developed and assessed in the past to strengthen masonry against in-plane and out-of-plane effects. These included the application of Fibre reinforced polymeric (FRP) sheets and bars (Martinelli et al. 2016; Monaco et al. 2017), Fabric reinforced cementitious matrixes (FRCM) (Dong et al. 2021; Castori et al. 2021), Composite Reinforced Mortar (CRM) (Faella et al. 2010; D'Antino et al. 2019), and plastering with cementitious matrixes etc…(Mustafaraj and Yardim 2019; Sandoval et al. 2021). Nonetheless, different masonry typologies can be found across the world, such as pebble, irregular stone, barely-cut, multi-leaf, earth based adobe, perfectly-cut and regular masonry (Corradi and Borri 2018; Parisi et al. 2018). Thus, the choice of a retrofitting method that has to be used is highly depends on the type of masonry (Corradi et al. 2017; Thamboo and Dhanasekar 2020; Maione et al. 2021).

The masonry subjected to in-plane shear behavior is the focus of this study. Although, different test set-ups can be used to assess the in-plane shear resistance of masonry, it is well accepted that the diagonal

compression test is more frequently used (Brignola et al. 2008; Calderini et al. 2010). The diagonal compression test is carried out to determine the tensile strength that produces diagonal cracking during the in-plane shear loading of a wall. This testing method is considered to be versatile, as it can also be applied in-situ (Corradi et al. 2010; Dizhur and Ingham 2013; Gattesco and Boem 2019). Subsequently, this testing method is standardised in ASTM E519 / E519M – 21 (2021) and RILEM (1994).

Many studies used the diagonal compression testing method to assess the effectiveness of shear strengthening techniques (Prota et al. 2006; Borri et al. 2011; Parisi et al. 2013; Gattesco and Boem 2015) and different rational analytical models were specified in the various guidelines and studies in the past (ACI 440.2R-17; ACI 549.4R-13 CNR-DT-200; CNR-DT-215). However, most of these studies have evaluated and reported the results independently, without a holistic view on the effectiveness of the different strengthening methods available. Recently, Del Zoppo et al. (2019) have analysed the effectiveness of FRCM and CRM methods based on the experimental data gathered and the applicability of the available analytical models to predict the in-plane shear resistance. A similar study was recently conducted by Meriggi et al. (2021), where experimental data for FRCM/CRM strengthened masonry were used to calibrate new design relationships. However, these studies have analysed the effectiveness of only one type of strengthening system (i.e. FRCM/CRM) and did not comprehend a critical comparison between different strengthening solutions and their applicability. Similarly to FRCM/CRM techniques, welded wire mesh (WWM) is also a technique used in many developing countries (Tripathy and Singhal 2021; Banerjee et al. 2020;). The Reticulatus method was typically applied for irregular stone masonry (Borri et al. 2011; Corradi et al. 2016), when the fair-face aspect needs to be preserved. In addition, there are applications of using only high strength cementitious mortars/plasters/matrixes (CP) on masonry surface to strengthen against shear effects (Corradi et al 2018; Najafgholipour et al 2018).

Subsequently, in this paper, an attempt was made to analyse the effectiveness of different strengthening solutions. An experimental database has been established using available literature studies. The experimental database consists of three strengthening solutions, namely (1) WWM (2) Reticulatus and (3) CP. Subsequently, the established database was classified into three different categories. Thereafter, the applicability of the available provisions to design such strengthening solutions were assessed using the experimental database and a unified design approach was calibrated and proposed in section 3. Finally, key conclusions are outlined in section 4.

2 Experimental Database

In order to study the effectiveness of above mentioned (WWM, Reticulatus and CP) strengthening methods, a database has been developed from the experimental campaigns reported in the scientific literature. These strengthening systems are shown in Fig. 1. The experimental results are reported in journals articles, conference papers, master theses and scientific reports.

In total, 75, 18, and 48 shear tests were gathered for WWM, Reticulatus and CP strengthened tests respectively. The database established for these three retrofitting methods are further classified according

to the types of masonry constituents, strengthening types/methods and their load resisting mechanisms. It has to be mentioned, although a quite large amount of data was found, in-depth quantitative comparisons among the influencing parameters were not possible, due to the intrinsic limitations for the large number of variables. The following appraisal was mainly carried out to compare and qualitatively evaluate the influence of masonry typologies, strengthening methods on the overall shear performance of strengthened masonry subject to diagonal compression test.

2.1 WWM strengthening

The dataset obtained of WWM strengthened masonry panels included 75 test results. General purpose cement mortars, ferrocement, fine graded granular concrete and lime based mortars were used as matrixes for the WWM strengthening. It is worth noting that there is a clear difference between the FRCM and WWM strengthening methods. FRCM consists in an initial application of a mortar coating and then the composite mesh is added over the mortar, and finally plastered again with the same mortar to cover up the mesh. The WWM application is normally started by connecting or anchoring the WWM to the masonry surface and then plastered to required thickness to develop adequate bonding between the WWM and masonry substrate. Another difference is that the use of reinforcement, where the FRCM is made of flexible bonded open fabrics (composite mesh) and the WWM system is made of conventional welded open steel bars or wires. The load resisting and transferring mechanisms of these two systems are clearly different, thus have to be treated separately.

Banerjee et al. (2020a) and Banerjee et al. (2020b) investigated the influence of steel WWM strengthening for brickwork (f_u=2.0 MPa) and concrete block (f_u=15.0 MPa) masonry. Brickwork masonry has shown relatively smaller gain in shear strength (1.4 to 3.2 times) than the concrete masonry (3.2 to 4.6 times). Shermi and Dubey (2018) examined the influence of WWM dimensions (using grids with spacing of 25 mm×25 mm, 38 mm×38 mm, and 50 mm×50 mm) and WWM tensile strengths (873 MPa, 936 MPa, and 1005 MPa) to the shear behavior of strengthened masonry. No specific studies were found on the influence of the matrix types (e.g. depending on the matrix mechanical properties) on the shear characteristics of strengthened masonry systems. However, it can be presumed, as observed in FRCM matrixes, that a higher the matrix strength would lead to better WWM behaviour. Moreover, different types of anchorages are typically used in WWM applications, such as bolting and sealed with epoxy and cement bond anchorages of the wire ends (Cheng et al. 2020; Bustos-García et al 2019). However no systematic studies have been dedicated so far to investigate the influences of anchoring methods to the overall shear strength characteristics of WWM strengthened masonry. In addition, despite of several experimental studies and applications in the past, no systematic design approach was found in the scientific literature to assess the in-plane shear resistance of WWM strengthened masonry.

2.2 Reticulatus method

The Reticulatus method consists of inserting flexible metallic or composite fiber wires into the masonry mortar joints. The wires are normally connected across the thickness of the masonry via transverse anchors at an interval that depending on wall type. Mortar joints have to be recessed about 50–60 mm

(depending on the thickness of bed joints) prior to inserting the transverse anchors and wires, and then repointed with compatible mortar after inserting the wires. This type of strengthening is preferred to preserve fair-faced masonry, especially for stone masonry. In total, 18 data were gathered from the past studies reported from testing of 29 wall panels. Borri et al. (2014) have investigated the characteristics of Reticulatus strengthened stone and clay brick masonry. Increments in the shear resistance of strengthened masonries in the range of 1.15 to 1.5 times were observed from their testing. Similar test results were recorded in other experimental campaigns conducted by the same research group (Borri et al. 2011a; Borri et al. 2011b; Corradi et al. 2016). Despite this system is quite well established, no design guidelines are available to evaluate the shear resistance of Reticulatus strengthened masonry.

2.3 CP strengthening

The 48 datasets of CP strengthened masonry panels tested under diagonal compression were gathered from 78 test results. Mainly, clay brick (42 data), stone (4 data) and concrete block (2 data) masonry assemblages were strengthened with CP. The mechanical characteristics of the matrix vary across the studies considered: matrices were mostly mortars with high tensile/adhesive properties such as engineered cementitious composites (ECC) and fibre reinforced mortars (e.g. Polypropylene and glass fibre). However, conventional Portland cement and lime-based mortars were also recorded (De Felice et al. 2014). Nonetheless, the CP strengthening is a rather simple method compared to other strengthening solutions, where a high strength/adhesive cementitious matrix is directly applied to the masonry surface to improve the tensile strength of masonry.

Cheng et al. (2020) evaluated the performance of clay brick masonry strengthened with conventional cement mortar (f_{mat}=11.8 MPa) and ECC (32 MPa) plastering. The results revealed an increment of shear resistance in the range of 90–125%, where the conventional cement plastering increased the shear resistance only up to 50–100%. Quite similar findings were reported from the study conducted by Bustos-Garcıa et al. (2019) using conventional Portland cement mortar and glass fibre reinforced mortar. No specific studies were conducted to compare the performances between the different masonry types. The CP plastering was generally applied to the entire face of the panels, either on one or both sides. However, the single sided CP reinforcement is less effective compared to the double sided method (Shabdin et al. 2018; Del Zoppo et al 2020). Similar to WWM and Reticulatus methods, no design formulations exist to estimate the shear resistance of CP strengthened masonry.

3 Analytical Prediction Of Shear Resistance

Although the experimental assessment of the characteristics of strengthened masonry is very useful, the analytical approaches are very important to routinely design such retrofit interventions. However, it can be said there is no systematic design approach specified for the strengthening techniques considered in this study. Nonetheless, there are well developed design guidelines for FRCM and FRP strengthened masonry (ACI 440.2R-17; ACI 549.4R-13 CNR-DT-200; CNR-DT-215. Thus, it is worth exploring the suitable design approaches for WWM, Reticulatus and CP methods. The generalised analytical approach to predict the

shear resistance (V_s) of strengthened masonry is given in Eq. (1), where, V_{URM} and V_{RM} are the contributions of masonry and strengthening system respectively.

$$V_s = V_{URM} + V_{RM}$$

1

This implies that the prediction of shear resistance of strengthened masonry relies on the appropriate assessment of shear resistance of URM and strengthening system. Several studies have been dedicated to characterise the shear resistance of unreinforced masonry (URM), and this depends on the possible failure modes under in-plane action, such as sliding, shear friction, diagonal tension and toe crushing (Siano et al. 2019; Zahra et al. 2018). The formulations to predict the URM resistance against these possible failure modes are given in Table 1. The parameters associated with these formulations are explained in the remarks column of Table 1. In order verify the applicability of the formulations outlined, their accuracy has to be assessed. In the next sub-section, the applicability of the given formulations to predict the URM shear resistance will be discussed and verified.

Table 1
Shear resistance formulations related to their failure modes of URM.

Failure mode	Formulation	Remarks
Shear sliding	$V_{ss} = \frac{\tau_0}{1-\mu_0 \tan\theta} A_n$	$A_n = \left(\frac{h+w}{2}\right) t$, where h, w and t are height, length and thickness of the masonry. τ_0 and μ_0 are the shear bond strength and coefficient of shear friction of masonry. θ is the inclination angle between horizontal and diagonal of the masonry element.
Shear friction	$V_{sf} = \frac{\tau_{0,m}}{1-\mu_m \tan\theta} A_n$	$\tau_{0,m} = \frac{\tau_0}{1+1.5\mu_0 \frac{h}{w}}$, $\mu_m = \frac{\mu_0}{1+1.5\mu_0 \frac{h}{w}}$
Diagonal tension	$V_{dt} = \frac{\tan\theta + \sqrt{21.2 + tan^2\theta}}{10.6} f_t A_n$	f_t is the masonry tensile strength
Toe crushing	$V_{tc} = \frac{2wf_m}{3h+2w\tan\theta} A_m$	A_m and f_m are the interface bonding area of loading shore and compressive strength of masonry.

3.1 Prediction of shear resistance of URM masonry

In order to validate the literature formulations, the established database was used to verify the applicability of these formulations to predict the shear resistance of the URM masonry. The masonry constitutive and geometric properties acquired in the database from the relevant studies were used to compute the V_{URM} values. The minimum of resistance value out of four possible failure modes of URM masonry (i.e. minimum of V_{ss}, V_{sf}, V_{dt}, and V_{tc} (Table.1)) is recommended to be considered. The

computed shear resistance corresponds to those four failure modes for each data are provided in Appendix A to C for all three strengthening systems deliberated. It has to be mentioned that not all the database studies have provided the complete masonry constitutive properties required to compute the shear resistance of URM, thus in the absences of required data certain assumptions had to be made to appropriately calculate corresponding resistance. Table 2 provides the details of unreinforced masonry constitutive characteristics taken from the existing literature.

Table 2
Material constitutive parameters used to compute URM shear resistance.

Parameter	Masonry type	Values selected	References
Initial shear bond strength (τ_0) (MPa)	Clay brick	0.13–0.4	EN 1996-1-1 (2005)
	Concrete block	0.13–0.26	EN 1996-1-1 (2005)
	Tuff block	0.08–0.15	Augenti and Parisi (2011)
	Stone	0.13	EN 1996-1-1 (2005)
Coefficient of shear friction (μ)	Clay brick	0.3–0.55	Thamboo (2020)
	Concrete block	0.25–0.45	Thamboo et al. (2013)
	Tuff block	0.3	Augenti and Parisi (2011)
	Stone	0.2–0.3	Kržan et al. (2015)
Compressive strength of masonry (f_m)	All masonry types	$Kf_b^{0.7} f_j^{0.3}$	EN 1996-1-1 (2005)
Tensile strength of masonry (f_t)	All masonry types	$0.67\sqrt{f_m}$	Silva et al. (2008)

The initial shear bond strength (τ_0) was taken from the values recommended in EN 1996-1-1 (2005). However, the characteristic shear bond strength values were converted to mean values, by considering COVs of 15% and normal distribution of the data. It has to be noted, in some cases, range of values was used, and that depended on the mortar strength used in the masonry assemblages (i.e. in EN 1996-1-1, τ_0 is varied based on the mortar class used). Then, for tuff block masonries, τ_0 was taken from Augenti and Parisi (2011). Moreover, the coefficient of shear friction (μ) was taken from several studies for different masonry typologies as given in Table 2. Most of these studies have reported the masonry compressive strength (f_m). However, in the absence of such data, the f_m value was computed using the equation given in EN 1996-1-1 (2005) based on the normalised unit and mortar strengths gathered in the database. Furthermore, the tensile strength (f_t) of the masonry was computed from the equation given in Silva et al. (2008). Consequently, the material constitutive parameters provided in Table 2 and the formulations outlined in Table 1 were used to predict the shear strength of the masonry gathered in the database. Fig, 2

shows the experimental and predicted shear strengths of the URM data gathered. Since the database was purposely divided into three strengthening systems, the corresponding predictions of URM shear strengths are also separately predicted. However, in Fig. 8(d), all the URM data were grouped to show the overall comparison of predicted URM shear strengths against the experimental cases in the database. It can be noted that the provision (i.e. minimum resistance out of four possible failure modes) to compute the URM shear resistance is conservative, as it predicts the shear resistance fairly less than corresponding experimental values.

The derived statistical parameters are given in Table 3 for comparison purpose. The model error (*ME*) (i.e. experimental value divided by predicted strength value) was computed for each case to evaluate the accuracy of predictions. Although in some cases, the minimum *ME* values are closed to 1, the number of these cases is low (4 out of 127 data gathered) compared to total data. Given the diverse masonry typologies in the database, existing formulations to evaluate the shear resistance of URM have shown to be quite conservative predictions.

Table 3
Basic statistical parameters of ME computed for URM database.

Strengthening system	Mean of ME	Minimum ME	Maximum ME	COV (%)
WWM	2.93	0.86	4.73	44.4
Reticulatus	1.84	0.92	3.13	44.2
CP	4.07	0.91	12.7	46.6
Overall	3.27	0.86	12.7	0.72

3.2 Prediction of shear resistance of strengthened masonry

Although the strengthening systems considered are very different, all of them have shown to improve the masonry shear resistance. In the Reticulatus method, the additional shear resistance (V_R) is obtained by the reinforcement embedded in the mortar joints. It can be postulated that the contribution of the repointing mortar in the shear resistance is negligible. In the CP system, the matrix contributes to the gain in shear resistance (V_M) and no additional reinforcement (V_R) is incorporated. Finally, in WWM, the gain in shear resistance ($V_R + V_M$) was attributed by reinforcement and matrix applied on the surface of the walls. In order to determine the enhancement of shear resistance in different strengthening systems considered, the term V_{RM} defined in Eq. (1), can be generalised as given in Eq. (2), where, the shear strength contribution is either from the matrix (i.e. CP/ V_M) or reinforcement (i.e. Reticulatus/ V_R), or from both components (i.e. in the case of WWM/ $V_R + V_M$).

$$V_{RM} = V_M + V_R$$

2

Subsequently, it can noted that the shear resistance of the strengthening system depends on the shear resistance of individual contributions from matrix (V_M) and reinforcement (V_R). In the following sections, the methods to evaluate the contribution of matrix and reinforcement in the shear resistance are verified.

3.2.1 Contribution of matrix strength

Cementitious matrix is commonly used in FRCM, WWM and CP strengthening systems. Specifically, the CP strengthening systems only use cementitious matrix to plaster on the masonry surface. Primarily, it is accepted that the matrix cracks and de-bonds, once its tensile resistance or interface shear bond resistance is attained under diagonal compression loading. Few different formulations have been used in the past to account the contribution of matrix in the diagonal shear resistance of masonry assemblages. Table 4 summaries these literature formulations to account the contribution of surface matrix/mortar plastering into the shear resistance of masonry. Especially, the study from Lin et al. (2014) used the formulation specified in JSCE (2008) to compute the shear resistance of ECC plastered masonry panels, where the resistance is determined from the minimum between the tensile strength of ECC matrix and bond strength between matrix and substrate. Almeida et al. (2015) adopted the same formulation given in Table 1 (i.e. V_{dt}) to determine the tensile resistance of URM and to compute the resistance of matrix ($f_{t,max}$) with relevant cross section area (A_{mx}). It can be noted that Ferretti and Mazzotti (2021) used the matrix flexural strength ($f_{fl,mx}$), where other studies used matrix tensile strength.

Determining the direct tensile strength of matrix is a quite difficult task, however, most of the experimental studies invariably reported the compressive strength of matrix along with the diagonal compression testing data. Thus the matrix tensile strength has to be derived from the compressive strength values. Consequently, the formulation given in EN 1992-1-1 (2004) to determine the direct tensile strength of concrete from the compressive strength is used in this study. The formulation is given in Eq. (3). This formulation is recommended for concrete grades less than 50 MPa, thus it could be also applicable to most of the cementitious matrixes used for masonry strengthening.

$$f_{t,mx} = 0.3 f_{c,mx}^{0.67}$$

3

In order to assess the contribution of matrix to the strengthened masonry, the established database was used in this study. Especially, the CP database comprised only the assemblages strengthened with mortar matrixes, and it was used to verify the applicability of available design provisions in the literature. It has to be mentioned that, if the CP strengthening is applied only to one side of the masonry, the predicted resistance is reduced by 0.7 factor as recommended in CNR-DT-200 (2013) and CNR-DT-215 (2018)for FRP and FRCM strengthened assemblies. Consequently, the predictive formulations suggested by Almeida et al. (2015) and Donnini et al. (2021) were taken to predict the contribution of matrix strength to the strengthened systems. The other two formulations were not considered for comparison, because (1) the Lin et al. (2014) formulation involves bond characteristics between masonry and matrix (but not

experimental data were found) and (2) the Ferretti and Mazzotti (2021) uses the matrix flexural strength (most of the literature studies do not provide the matric flexural strength, however only the corresponding compressive strength).

Figure 3 shows the experimental and predicted shear resistances of the strengthened masonry assemblages (i.e. CP). It has to be mentioned that the experimental shear resistances (on the horizontal axis) are the overall strengths of the strengthened assemblages. The predicted reistance is given by the contribution of URM (from previous section) and matrix incorporated. This exercise was implemented to verify the contribution from the matrix to the overall strength of strengthened masonry. In general, it can be noted that the formulations considered conservatively predicted the contribution of matrix to the shear resistance of strengthened masonry. The basic statistical parameters, derived in the analyses, are given in Table 5 for comparison purpose. However, the formulation trilled by Almeida et al. (2015) (i.e. also used to predict the tensile resistance of URM assemblages) conservatively predict the resistance than the formulation used by Donnini et al. (2021).

Table 4
Formulations given to predict the contribution of matrix strength

References	Formulation	Remarks
Lin et al. (2014)	$Min\left(t_m 0.72 L f_{t,mx}, 0.18\sqrt{f_{c,mx}} L t_m\right)$	Taken from JSCE [], for ECC strengthened elements, where the minimum contribution of ECC matrix and their bond between the substrate is accounted. $f_{c,mx}$ is the compressive strength of matrix.
Almeida et al. (2015)	$\frac{\tan\theta + \sqrt{21.2 + tan^2\theta}}{10.6} f_{t,mx} A_{mx}$	Similar to the tensile resistance of URM, however instead of tensile strength of URM, the tensile strength of matrix is used.
Ferretti and Mazzotti (2021)	$\frac{f_{fl,mx} A_{mx}}{0.5}$	$f_{fl,mx}$ and A_{mx} are the flexural strength the net cross section of matrix.
Donnini et al. (2021)	$\frac{L}{0.707} f_{t,mx} n t_m$	L is the width of the panel, $f_{t,mx}$ and t_m are the tensile strength and thickness of matrix. Then n is the number of layers of strengthening FRCM.

Table 5
Basic statistical parameters of ME computed for matrix strengthened assemblages.

Strengthening system	Formulation considered	Mean of ME	Minimum ME	Maximum ME	COV (%)
CP	Donnini et al (2021)	1.97	0.68	8.33	85.1
	Almeida et al (2015)	3.84	0.75	14.66	81.9

3.2.2 Contribution of the reinforcement

The contribution of the reinforcement to the shear resistance of strengthened masonry (WWM and Reticulatus) involves complex mechanisms. However, different rational approaches have been developed to account the contribution of fabric/fibre in the FRCM and FRP strengthened masonry assemblages. A similar method can be used to establish the contribution of reinforcement in WWM and Reticulatus techniques. However, unlike FRCM and FRP, the WWM and Reticulatus methods involve the use of steel meshes and wires, which are anchored to the walls. For these strengthening methods, slippage mechanisms between the matrix and masonry substrate are relatively rare to occur compared to FRCM systems. Especially, due to the debonding mechanism in FRP, the term “effective/design tensile strain” and “effective/design tensile stress” limits are considered in FRP systems to account for the fibre contribution to the shear resistance (Vaculik et al. 2018; Porta et al. 2008). Due to the slippage and debonding mechanisms in FRCM systems, commonly referred as “telescopic effect”, similar analogy is considered to deduce the “conventional/design tensile strain” and “conventional/design tensile stress” limits (Thermou et al. 2021; Araya-Letelier et al. 2019). However, it can be hypothesized that such debonding and slippage would not be the major failure modes in WWM and Reticulatus methods as reinforcement bars/meshes are stiffer, and matrix cracking (due to failure of matrix/mortar under tensile stress) was more frequently recorded in the experimental studies. Further matrix impregnation in fabrics, and their debonding observed in FRCM, are not the mechanism observed in steel bars/wire in Reticulatus and WWM systems. Subsequently, the cracking of matrix depicts as debonding in WWM and Reticulatus reinforced, however it is not primarily due to the relative slippage between reinforcement and matrix.

Since the WWM and Reticulatus systems involve the use of steel meshes, bars and wires, the design approach of Reinforced Masonry (RM), which is widely used in North America and Australasia for concrete block walls will be considered (Araya-Letelier et al. 2019; Zahra et al 2021), where the reinforcing steel bars are inserted in the vertical cores of the hollow blocks and additional bars are embedded in the horizontal bed joints. In this RM, the reinforcement bars (horizontal and vertical) are placed mainly at the center of the walls (hollow cores), however in WWM and Reticulatus, the reinforcement is provided on the surface or near the surface (i.e. for Reticulatus). Figure 4 illustrates the reinforcement arrangement in different systems considered. Thus, similar approach can be drawn from the design concepts provided for Near Surface Mounted (NSM) FRP bars in masonry (ACI 440.2R-17). The available design formulations are given in Table 6 with their references.

It can be noted that the formulations have similar forms, where the contribution of reinforcement is accounted using the effective horizontal cross-sectional area of reinforcement and its yield strength. It is widely understood that the horizontal reinforcement provides resistance to the in-plane shear action, and the vertical reinforcement is primarily effective in resisting the in-plane flexural action. Therefore, the generalized formulation to predict the contribution of reinforcement in resisting the shear action can be written:

$$V_R = CA_r f_r \frac{d}{s}$$

4

Where, A_r and f_r are the area and yield strength of reinforcement, respectively. S and d are the spacing of reinforcement and effective depth of shear resistance (it is equal to the width of the panels tested). C is a coefficient taken to account the contribution/efficiency of horizontal reinforcement in resisting shear. It has been well established that the horizontal reinforcement does not fully contribute to resisting the shear effects. It was highlighted, that the shear effect is initially carried by masonry, and the reinforcement is fairly unstressed. The horizontal reinforcement only starts contributing to resist shear once the cracks appeared and opned in masonry. Therefore, the contribution of shear reinforcement is reduced to conservatively predict the shear resistance (Voon et al 2007; Augenti et al. 2010) in RM assemblages. A similar analogy was used to verify the established methodology to predict the shear resistance of WWM and Reticulatus strengthened masonry.

Subsequently, using the formulations established to calculate the URM (section 3.1), matrix (section 3.2.1) and reinforcement contributions (section 3.2.2), the shear resistance of strengthened masonry (WWM and Reticulatus) was predicted. The generalised formula developed to evaluate the shear resistance of strengthened masonry is given in Eq. (5). Since the contribution of the URM and matrix have been already discussed in the previous sections, the accuracy of predicting the contribution of reinforcement in the shear resistance was verified in this section.

Table 6
Formulations to account the contribution of reinforcement.

Reference	Formulation	Remarks
ACI 440.7R-10 (2017)	$\rho_f \frac{d}{s}$	*s is the center-to-center spacing between the bars, and d is the effective masonry depth (i.e. minimum of length and width of wall/panel)*
CSA S304.1-04 (2013)	$0.6A_r f_r \frac{d}{s}$	*A_r is the effective area of reinforcement, and f_r is the yield strength of reinforcement,*
AS 3700 (2018)	$0.8A_{sr} f_r$	*A_{sr} is equal to $A_r \frac{H}{L}$, if $\frac{H}{L}$, is less than 1, otherwise, A_{sr} should be considered minimum of horizontal and vertical reinforcement area.*
MSJC (2013)	$0.5A_r f_r \frac{d}{s}$	*Similar terms are defined as per CSA S304.1-04* [70].

$$V_s = V_{URM} + \frac{\tan\theta + \sqrt{21.2 + tan^2\theta}}{10.6} f_{t,mx} A_{mx} + CA_r f_r \frac{d}{s}$$

5

Where, V_{URM} has to be determined from the set of formulations outlined in section 3.1. θ, $f_{t,mx}$ and A_{mx} are the inclination angle between horizontal and diagonal of the masonry element, tensile strength of matrix and cross sectional area of matrix, respectively. Moreover, A_r and f_r are the area and yield strength of reinforcement, respectively. S and d are the spacing of reinforcement and effective depth of shear resistance. As mentioned, C is a constant incorporated to reduce the contribution of horizontal reinforcement to the shear resistance.

Consequently, the experimental database established for WWM and Reticulatus systems was utilised to verify the accuracy of the design approach. If the CP strengthening is applied only to one side of the masonry (WWM or Reticulatus), the predicted resistance is reduced by 0.7 factor as recommended, similar to other strengthened assemblies. Figure 5 shows the predicted and experimental shear capacity of WWM and Reticulatus strengthened masonry lateral capacities considered in the database. In order to conservatively predict the shear capacity of strengthened masonry, and to conservatively account for the contribution of reinforcement, C in the Eq. (5) was calibrated against the experimental database established. The value of C constant corresponding to the 95th percentile of ME was computed to achieve relatively conservative prediction. Thus, C was initially assumed as unity, then calibrated to achieve the 95th percentile of the data. Successively, the calibrated C values for WWM and Reticulatus systems were 0.51 and 0.37, respectively. It can be concluded that the proposed unified approach can be used to conservatively predict the shear resistance of the strengthened masonry, where the minimum ME is closed to 1 and the 95th percentile ME is fairly higher (> 4) for the cases considered in the analyses. Thus, it can be stated that the established formulation to predict the shear resistance of WWM, Reticulatus and CP strengthened masonry can conservatively be used.

Table 6
Statistical parameters obtained for the WWM and Reticulatus shear resistances.

Strengthening system	Mean of ME	Minimum ME	Maximum ME	COV (%)	95th Percentile ME	Calibrated C value
WWM	2.75	1.04	5.35	38.0	4.48	0.51
Reticulatus	2.47	0.94	6.01	51.7	4.85	0.37

Although the unified formulation developed has shown to predict the shear resistance of WWM, Reticulatus and CP strengthened masonry, the formulation could be further calibrated with more data from future research studies. Because masonry typologies varies between regions and counties, the applicability of the strengthening methods examined and their contribution to the shear resistance need further systematic experimental verification incorporating the variability in the constitutive properties, masonry geometry and strengthening types.

4 Summary And Conclusions

Masonry is weak in tension, thus requires strengthening to withstand lateral actions such as earthquake and cyclonic actions. Various strengthening systems have been recently developed to enhance the lateral resistance of different masonry typologies, especially FRCM sand FRP are widely used and design guidelines are available for such systems. However, there are other strengthening systems such as WWM, Reticulatus and CP when fair faced aspect needs to be preserved or when there are economic considerations, but no guidelines or instructions are available in design. In this study, a unified design approach was developed to predict the shear capacity of WWM, Reticulatus and CP strengthening systems. An experimental database of the shear strength characteristics has been developed. The applicability of the design approach was verified against the experimental resistances obtained for different masonry typologies. The following conclusions can be drawn from the analyses of this study.

- The established experimental database revealed that WWM, Reticulatus and CP retrofitting methods invariably enhance the masonry shear resistance, where CP strengthening application is a fairly simple technique and Reticulatus strengthening system is preferred when fair-faced aspect masonry needs to be preserved. WWM systems are quite similar to FRCM strengthening systems, nonetheless use conventional steel/wire mesh instead of fabrics.
- The prediction of URM shear resistance considering four possible failure modes (shear sliding, shear friction, diagonal tension, toe crushing) has shown to be conservative against the database established.
- The formulations proposed to predict the matrix contribution to the shear resistance of CP and WWM strengthened masonry have demonstrated to be appropriate and quite conservative against the experimental results. A simplified equation has been proposed to predict the matrix tensile resistance from the compressive strength
- An approach similar to the one used to determine shear resistance of reinforced concrete block masonry (RM) was adopted to account for the contribution of reinforcement (WWM and Reticulatus). Consequently, the formulation proposed was calibrated to compute the reduced contribution of reinforcement in the shear resistance in the systems.
- A unified approach was also established to account the shear resistance of strengthened masonry from the contribution of URM, matrix and reinforcement used. WWM systems comprised of all three components, while Reticulatus only included URM and reinforcement contributions, and CP strengthening URM and matrix. The design approach has shown to conservatively predict the shear resistance, and it can cautiously be employed to predict the shear resistance of strengthened masonry, despite of the diverse masonry types considered.

However, the formulation developed should further be verified with full scale shear testing results. It should be highlighted that the Reticulatus methods was only verified with a limited number (18) of test results.

Declarations

CRediT authorship contribution statement

Julian Thamboo: Conceptualization, Formal analysis, Data Curation, Writing - Original Draft. **Marco Corradi**: Data Curation, Writing - review & editing. **Keerthan Poologanathan**: Writing - review & editing.

Conflict of interest

The authors declare no conflicts of interest in this research.

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Figures

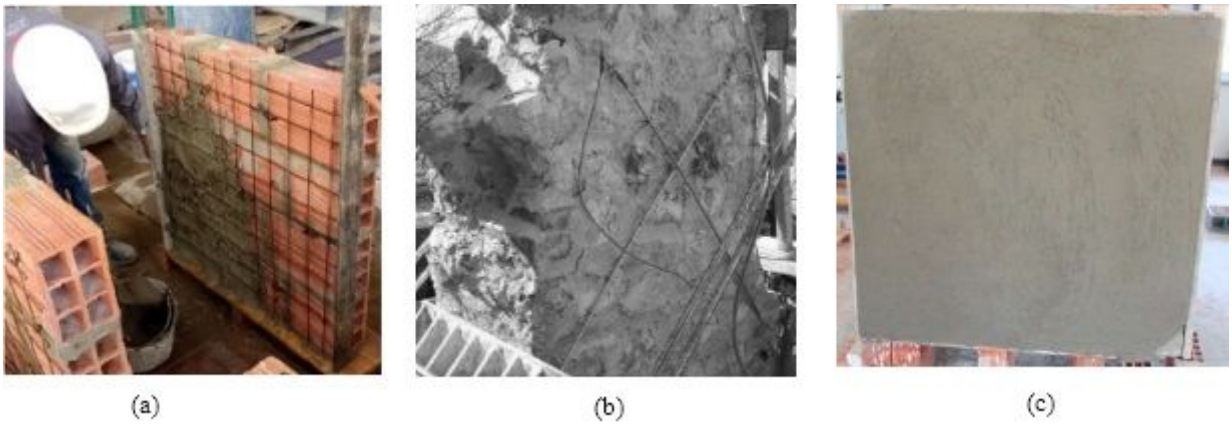

Figure 1

Diagonal shear testing of WWM (Sandoval et al. 2021), Reticulatus (Corradi et al. 2017) and CP (Corradi and Borri 2018) strengthened masonry panels.

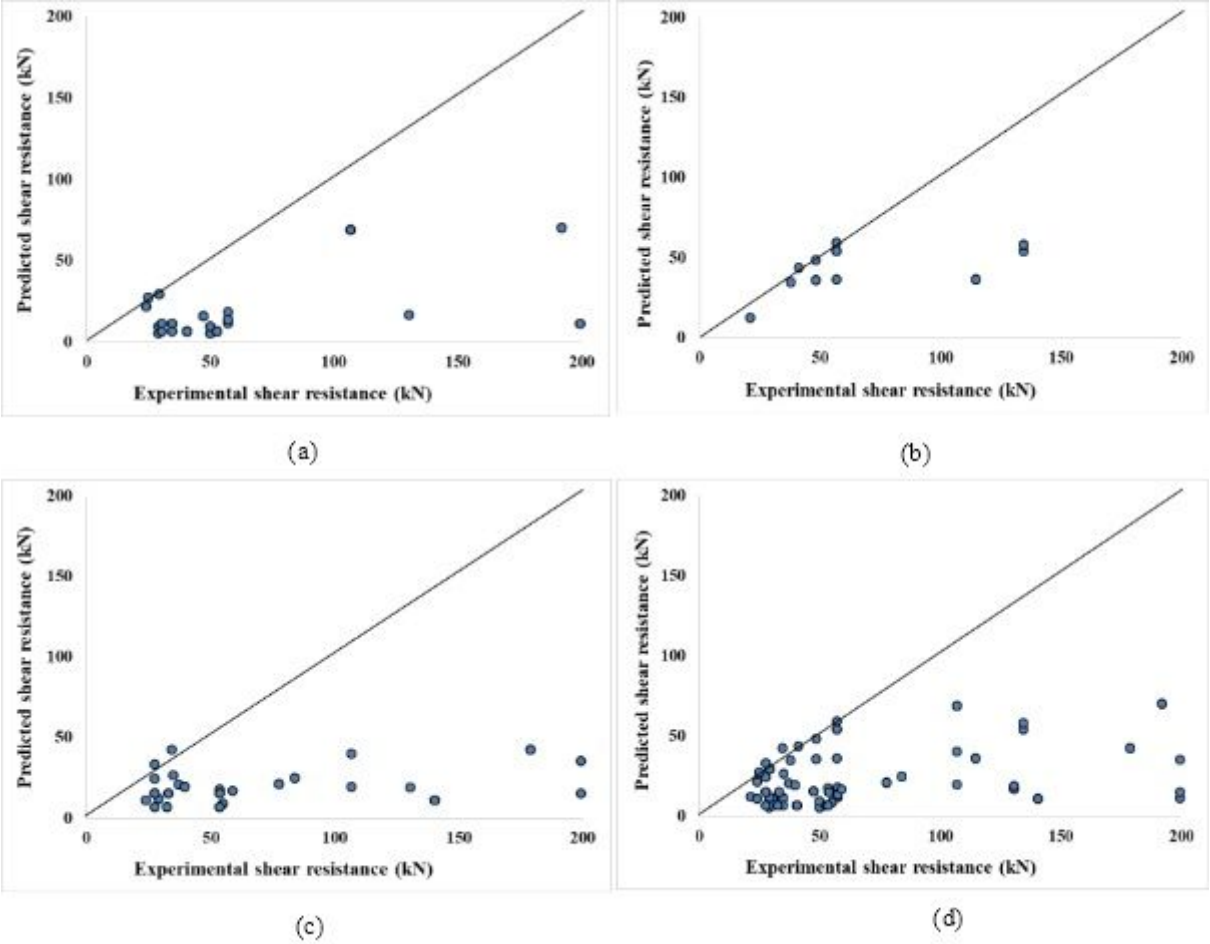

Figure 2

Predicted resistance of URM panels from the database (a) WWM (b) Reticulatus (c) CP and (d) and overall.

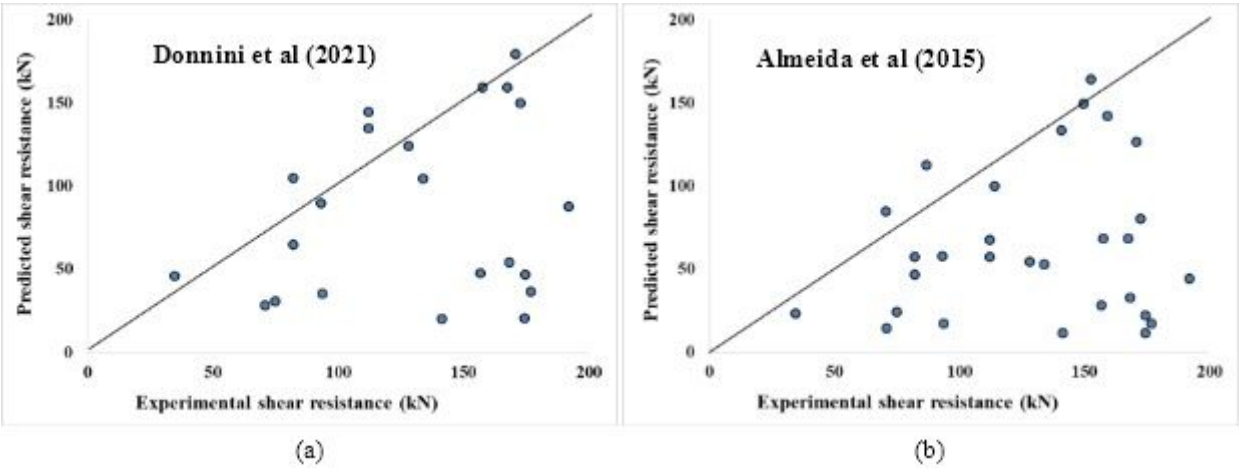

Figure 3

Predicted resistance of matrix strengthened panels from the database.

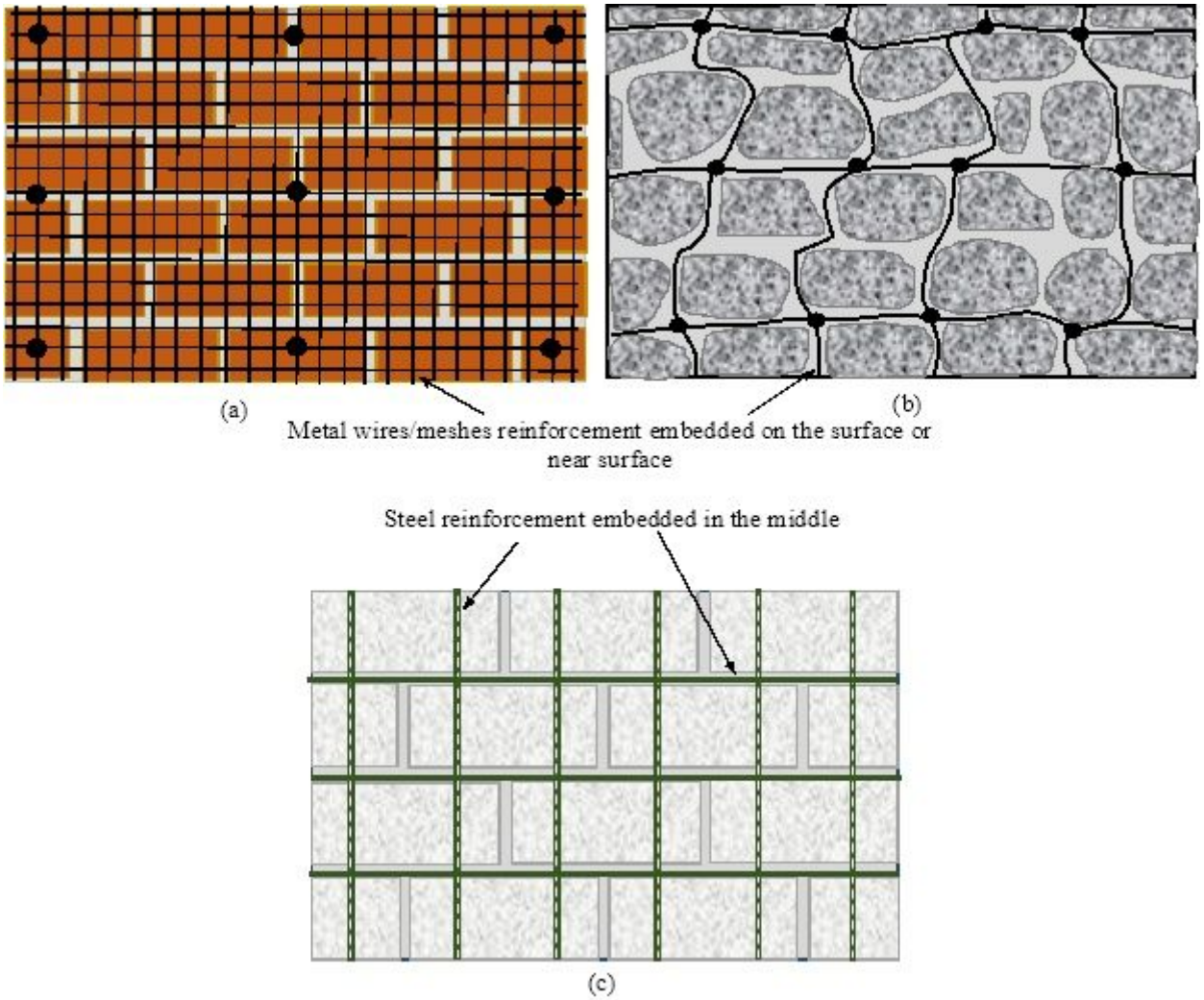

Figure 4

Graphical illustration of the similarities between (a) WWM, (b) Reticulatus and (c) RM masonries.

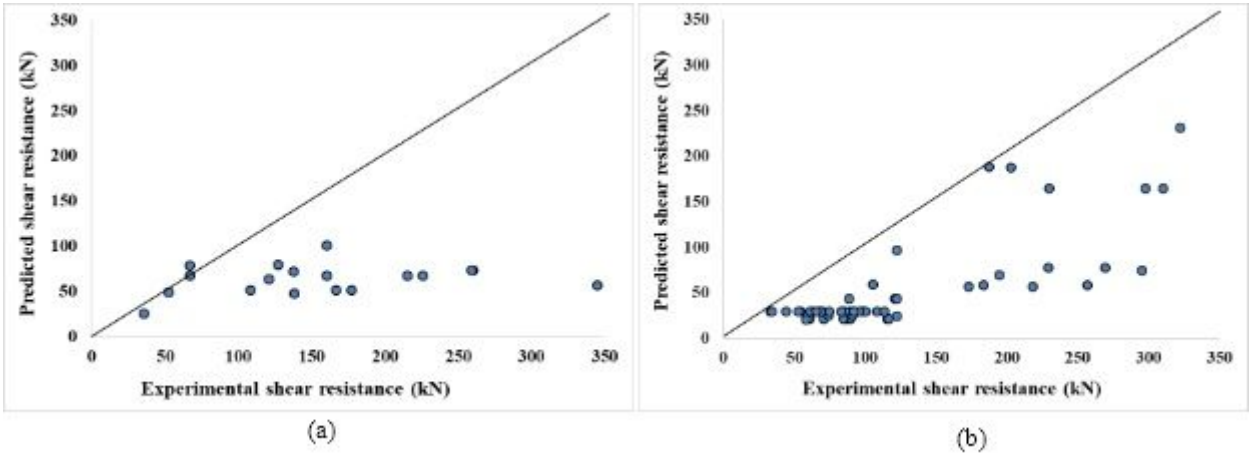

Figure 5

Prediction of overall shear resistances (a) Reticulatus and (b) WWM.

Supplementary Files

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