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REINFORCED CLAY MASONRY WALLS: EFFECTIVENESS OF REINFORCEMENT AND SHEAR EQUATIONS

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ABSTRACT
In the framework of the DISWall project, funded by the European Commission, innovative construction systems for reinforced masonry walls were developed for the application in seismic areas. In particular, a new reinforced masonry system made with horizontally perforated clay units was developed on purpose for typical low-rise residential buildings to withstand in-plane actions. Thirty specimens of this type of masonry were characterized by means of uniaxial tests and by means of in-plane cyclic shear compression tests. In the present contribution, the effectiveness of reinforcement in the tested specimens is discussed. The tests results are compared with code proposed formulations and with newly calibrated formulations, in order to check their reliability in predicting the ultimate load capacity of reinforced masonry walls.

KEYWORDS: reinforced masonry; reinforcement effectiveness; shear equations.

INTRODUCTION
Modern structural concepts for masonry buildings require that they resist earthquake actions with box-type of behaviour. With this assumption, the horizontal seismic actions are transferred to the walls parallel to the direction of load application [1; 2]. Therefore, when developing reinforced and confined masonry systems, particular attention is paid to the study of the in-plane behaviour. This is carried out, experimentally, by means of cyclic shear-compression tests. Once experimentally characterized, design of reinforced masonry walls for in-plane actions remains an issue. The shear capacity of reinforced masonry walls is governed by several mechanisms induced by the presence of the reinforcement. The tensioning of the horizontal reinforcement becomes effective when the first shear crack appears, by preventing the separation of the cracked portions of the wall. The vertical reinforcement is mainly effective in case of flexural behaviour of the wall. However, it also gives a contribution to the shear capacity of the wall, by means of the dowel-action mechanism. The combination of vertical and horizontal reinforcement leads to the development of a global mechanism, which lies in between the arch-beam and truss mechanism [1; 3]. According to the different interpretations of the mechanical behaviour, the theoretical shear capacity can be evaluated by means of an amount of design formulations, given by structural codes or proposed by researchers.

In the framework of the European funded DISWall project, a new reinforced masonry system, based on the use of horizontally perforated units and concentrated vertical reinforcement placed in confining columns (made with vertically perforated clay units) was developed (Figure 1). The horizontally perforated units have recesses for placing the horizontal reinforcement. The main advantages of the system are that all the problems related to cover of bars and mortar shrinkage are overcome. Furthermore, this system preserves the use of a unit type (with horizontal holes) which is very traditional for the countries facing the Mediterranean basin, as it allows reaching good thermal and acoustic insulation. The system and the tests carried out to characterize it are described in [4; 5]. A detailed discussion on results obtained by means of in-plane cyclic shear-compression tests is also presented in this conference [6]. In the present contribution, the effectiveness of horizontal and vertical reinforcement and the validity of different formulations, proposed by codes or researchers to predict the ultimate load capacity of reinforced masonry walls, are discussed.

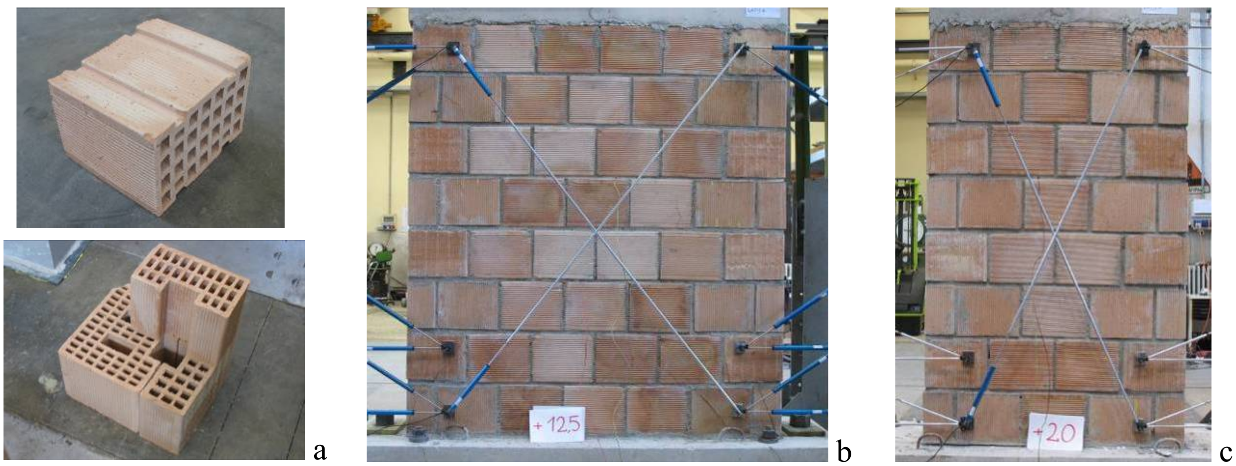


Figure 1: Horizontally perforated clay units and vertically perforated units for the confining columns (a); squat (b) and slender (c) specimens for in-plane cyclic testing.

EXPERIMENTAL PROGRAM

The main objective of the testing program was to asses the behaviour under in-plane cyclic actions of this kind of reinforced masonry wall system. The tests were repeated on two series of specimens, with different horizontal reinforcement. One series was built with usual steel rebars (specimens named SR, horizontal reinforcement percentage of 0.045%), the other with prefabricated truss reinforcement (specimens named TR, horizontal reinforcement percentage of 0.040%). In all of the specimens, the horizontal reinforcement was distributed on the specimens each other course. Specimens built with the entire reinforced masonry system and masonry panels without the confining columns (HS) were tested under in-plane cyclic shear compression tests. The shear compression tests on the entire reinforced masonry system were carried out on specimens characterized by two slenderness ratio, in order to force the shear behaviour (slenderness ratio “a” equal to 1.09) and the flexural behaviour (slenderness ratio “b” equal to 1.64). For these specimens, the vertical reinforcement was constituted by two rebars with diameter of 16 mm at each masonry edge for the squat specimens “a” (reinforcement percentage of 0.17%) and by one rebar with diameter of 16 mm at each masonry edge for slender specimens “b” (reinforcement percentage of 0.13%). The specimens were tested with cantilever type boundary condition, with fixed base and top end free to rotate, by applying a centred and constant vertical load of 0.4 and 0.6 N/mm^2, and horizontal cyclic displacements, with increasing amplitude and with peaks repeated three times for each amplitude. The effectiveness of reinforcement was checked by means of strain-gauges mounted before the construction of the specimens. Further details on the test series and the results can be found in [4-6].

EFFECTIVENESS OF HORIZONTAL REINFORCEMENT

The contribution of shear reinforcement to lateral resistance of walls was calculated as the ratio between the horizontal load carried by the horizontal reinforcement (H_{rh}, evaluated starting from the strain measured on the horizontal reinforcement and assuming that the same average value of strain develops in all horizontal reinforcement) and the maximum load carried by the wall (H_{max}). The effectiveness of the reinforcement (C_{eff}) was evaluated as the ratio between H_{rh} and the yielding load of horizontal reinforcement ($H_{rh,y}$), which is 116kN and 129kN, respectively for specimens with rebars and truss reinforcement. Table 1 lists these parameters. In case of squat specimens, the contribution of horizontal reinforcement to lateral resistance is almost 40%, and the effectiveness of reinforcement C_{eff} is more than 60%. The same parameters are very low in the case of slender specimens, due to the developed flexural failure mechanism.

Table 1: Contribution and effectiveness of horizontal reinforcement

Specimens	H_{max} (kN)	H_{rh} (kN)	H_{rh} / H_{max}	$C_{eff} = H_{rh}/H_{rh,y}$	Eq. 2 DM 14/01/08		Eq. 2 calibrated		Failure mode
					$(H_{max}-H_{u,s})/H_{max}$	C_{rh}	$(H_{max}-H_{u,s})/H_{max}$	C_{rh}	
SRSa0.6*	218	-	-	-	0.10	0.14	0.35	0.60	shear
TRSa0.6*	211	-	-	-	0.07		0.33		shear
SRSa0.4	201	78.6	0.39	0.67	0.20	0.34	0.39	0.64	shear/fl
TRSa0.4	201	78.3	0.39	0.61	0.20		0.39		shear/fl
SRSb0.6	90	6.0	0.07	0.05	-0.43	-0.30	-0.02	0.00	flex
TRSb0.6	93	21.5	0.23	0.17	-0.38		0.02		flex
SRSb0.4	79	1.4	0.02	0.01	-0.31	-0.19	0.00	0.01	flex
TRSb0.4	81	4.8	0.06	0.04	-0.28		0.02		flex

*Data non reliable due to errors in strain-gauges

In [7], the same type of data is given for a different type of reinforced masonry system, tested under heavier axial load levels. In that case, the contribution of horizontal reinforcement on the lateral resistance is around 40%, and the effectiveness is about 37%, depending on the quality of units. Therefore, the contribution of reinforcement seems to be almost the same, even under different conditions. Regarding the effectiveness of shear reinforcement, it decreases with the increase of reinforcement ratio [1]. The horizontal reinforcement ratio in [7] is twice than in our research, thus it seems reasonable to assume that, if shear reinforcement was halved, the effectiveness would be 74% (0.37x2=0.74), much closer to the value obtained in our research. According to [1], it is thus possible to interpret the lateral strength of reinforced masonry as a sum of contributions: the strength of unreinforced masonry ($H_{u,s}$) and the amount due to shear reinforcement (H_{rh}). The latter can be seen as the yielding load of horizontal reinforcement ($H_{rh,y}$), adequately reduced by C_{rh} (eq. 1), which is the horizontal reinforcement reduction factor:

$$C_{rh} = \frac{H_{\max} - H_{u,s}}{H_{rh,y}} \tag{1}$$

where H_{max} is the experimental maximum load, and the strength of unreinforced masonry can be evaluated experimentally or, e.g., by means of the typical Mohr-Coulomb relation (eq. 2):

$$H_{u,s} = \left(c + \mu \cdot \sigma_0\right) \cdot td \tag{2}$$

In equation 2, σ_0 is the constant axial stress calculated on the gross horizontal section, t and d are thickness and effective length (or depth) of the wall section. Cohesion (c) and friction (μ) have been fixed equal to 0.2 N/mm^2 and 0.4, respectively, according to [8]. Subsequently, they have been calibrated, because of the unit and the type of bed joint (units with horizontal holes and smooth bed faces without any hole), to the values of 0.2 N/mm^2 and 0.2. The values of C_{rh} factor given by [7] are 0.24 and 0.40, according to the strength of masonry units. In [1], it is said that the C_{rh} factor can vary from about 0 to 0.5. The values of horizontal reinforcement reduction factor C_{rh} obtained from our tests, applying the general eq. 2 given by the Italian code, is given in Table 1. They seem to vary according to the axial load levels, as they are 0.14 and 0.34, respectively for pre-compression of 0.6 N/mm^2 and 0.4 N/mm^2, according also to the experience of [3]. However, researchers and codes generally hypothesize that C_{rh} factor remains constant, independently by the axial load level, which is the result obtained by adopting the calibrated version of eq. 2. In this case, C_{rh} is 0.60÷0.64, for the two different axial load levels. Furthermore, this value is also consistent with the value obtained for the effectiveness of reinforcement C_{eff}. In any case, the possible variation of the horizontal reduction factor C_{rh}, according to the axial load level, requires further investigations.

EFFECTIVENESS OF VERTICAL REINFORCEMENT

The contribution and the effectiveness of vertical reinforcement have been also investigated by measuring strain on vertical reinforcement, close to the base of the wall. The bending moment (M_{rv}) obtained by the strains measured at maximum lateral load (H_{max}) was compared to the actual bending moment developed at the base of the walls ($M_{max}= h \cdot H_{max}$), and to the flexural capacity of vertical reinforcement ($M_{rv,y}$) calculated by the second part of eq. 3:

$$M_u = M_{u,s} + M_{rv,y} = \frac{\sigma_0 t\ l^2}{2}\left(1 - \frac{\sigma_0}{f}\right) + z \cdot A_{rv} f_y \tag{3}$$

where M_u is the theoretical flexural capacity of the reinforced masonry wall, $M_{u,s}$ represents the flexural capacity of unreinforced wall section, l is the length of the wall, z is the distance between vertical reinforcing bars, f is the compressive strength of masonry, and A_{rv} is the area of vertical reinforcement, symmetrically placed at both wall ends. Equation 3 is based on the usual hypothesis for flexural calculation such as plain sections, etc., and it assumes that yielding occurs in both compression and tension, which requires implicitly a compressed strain of masonry of about 3.5‰. This equation was proposed by [1] to evaluate the flexural capacity of reinforced masonry wall.

Table 2 gives the values of the moments and their ratios and shows that the contribution of the vertical reinforcement (M_{rv} /M_{max}) to the flexural capacity of the reinforced masonry walls is on average 0.71 for squat and 0.52 for slender specimens, with a trend to increase with decrease of the axial load level. This makes sense, since with low axial loads, the flexural mechanism depends more on the tensile strength introduced by vertical reinforcement. The effectiveness of the vertical reinforcement, given in Table 2 as ratio $M_{rv}/M_{rv,y}$, is 100% for both slender and squat specimens. In the case of slender specimens, this is consistent with the failure mode, whereas squat specimens generally failed in shear, although vertical reinforcement yielded, indicating that flexural failure was not so far and vertical reinforcement was completely exploited. The combination of 100% effectiveness factor and high contribution factor means that plasticity is starting but compression capacity of masonry (related to its flexural capacity) is not completely exploited. In squat specimens, the shear capacity is thus reached before completely exploiting the flexural capacity. For slender specimens, the contrary occurs; therefore the vertical reinforcement contribution to the total bending moment is lower than for squat specimens and masonry in compression is fully exploited.

Table 2: Contribution and effectiveness of vertical reinforcement

Specimens	H_{max} (kN)	M_{max} (kNm)	M_{rv} (kNm)	M_{rv}/M_{max}	$M_{rv,y}$ (kNm)	$M_{rv}/M_{rv,y}$	Equation 3		Failure mode
							$H_{u,f}$ (kN)	$H_{u,f}/H_{max}$	
SRSa0.6	218	384	262	0.68	262	1.00	253	1.16	shear
TRSa0.6	211	370	-	-		-	253	1.20	shear
SRSa0.4	201	353	-	-		-	223	1.11	shear/fl
TRSa0.4	201	353	257	0.73		0.98	223	1.11	shear/fl
SRSb0.6	90	158	-	-	79	-	91	1.01	flex
TRSb0.6	93	163	79	0.48		1.00	91	0.98	flex
SRSb0.4	79	139	-	-		-	77	0.98	flex
TRSb0.4	81	142	79	0.55		1.00	77	0.95	flex

Table 2 also compares experimental and theoretical lateral capacity calculated for flexural failure mechanisms, according to eq. 3. It can be seen that, for the slender walls failed in flexure, the results of eq. 3 are in agreement with the experimental values, whereas they overestimate the lateral capacity of squat specimens failed in shear or in mixed mode. Available formulations for evaluating the shear capacity of reinforced masonry walls are discussed in the following section.

AVAILABLE SHEAR EQUATIONS

As already mentioned in the introduction, the shear mechanisms of reinforced masonry are complex, and the evaluation of how much each mechanism affects the shear capacity is hard, since these quantities are not directly measurable during an experimental test. Therefore, shear strength of reinforced masonry walls is generally calculated as a sum of contributions, better than on the basis of theoretical models. The usual procedure to write a shear strength formulation is to introduce terms related to the various mechanisms and, subsequently, to calibrate each term.

Four main contributions are usually considered by formulations proposed to predict the nominal shear strength V_R of reinforced masonry walls: V_m is the shear strength of unreinforced masonry, V_P is the contribution of axial load, V_s is the contribution due to horizontal reinforcement and V_{dw} is the contribution due to dowel-action of vertical reinforcement, as expressed by equation 4. Formulations of this type are proposed by many standards [8-13], with some variations. Table 3 gives, for various shear formulations, the single terms of eq. 4. For the meaning of each symbol, refer to [1-2; 8-16], and to [4] for a general overview.

$$V_R = V_m + V_P + V_s + V_{dw} \tag{4}$$

Table 3: Terms in equation 4:

Ref.	Unreinforced Masonry V_m	Axial Load V_P	Horizontal Reinforcement V_s	Dowel-Action V_{dw}
[9]	$\left(f_{v0}+0.4\sigma_0\right)\cdot tl$	implicit	$0.9\cdot A_{sw} f_y$	-
[8]	$\left(f_{v0}+0.4\sigma_0\right)\cdot td$	implicit	$0.6\cdot\frac{d\cdot A_{rh} f_y}{s}$	-
[14]	$\left(\frac{f_t}{b}\cdot\sqrt{\frac{\sigma_0}{f_t}+1}\right)\cdot tl$	implicit	$0.3\cdot A_{sw} f_y$	-
[1]	$\left(f_{v0}+0.4\sigma_0\right)\cdot tl$	implicit	$0.3\left(\frac{0.9dA_{rh}}{s}\cdot f_y\right)$	$0.806nd_{rv}^2\sqrt{f_m f_{yv}}$
[13]	$\left(0.35+0.6\sigma_0\right)\cdot tl$	implicit	$A_{sw}\cdot f_y$	-
[12]	$\left(1.5-0.5\frac{h}{l}\right)\cdot tl$	-	$0.8\cdot\frac{l}{h}\cdot A_{sw} f_y$	-
[11]	$0.083\left(4-1.75\frac{M}{Vl}\right)\cdot A_n\sqrt{f_m^{'}}$	$0.25\cdot\sigma_0 A_n$	$0.5\cdot\left(\frac{A_{rh}}{s}\right)\cdot f_y l$	-
[10]	$\left(\left(C_1+C_2\right)\cdot v_{bm}\right)\cdot td$	$0.9N^*\tan\alpha$	$C_3\cdot A_{rh} f_y\frac{d}{s}$	implicit
[2]	$\left(0.166+0.0217\rho_v f_{yv}\right)A_n\sqrt{f_m^{'}}$	$0.0217\sigma_0 A_n\sqrt{f_m^{'}}$	$\left(\frac{l-2d'}{s}-1\right)A_{rh} f_y$	implicit
[16]	$0.12\cdot k_1 k_2\cdot A_n\sqrt{f_m^{'}}$	$0.25\cdot\sigma_0 A_n$	$0.5\cdot\frac{dA_{rh} f_y}{s}$	evaluated to provide null contribution

The term to evaluate the shear strength of unreinforced masonry, V_m, is based on a Mohr-Coulomb friction criterion (such as that expressed by eq. 2) in the European [9], Italian [8] and British [13] codes and in Tomaževič [1]. Tomaževič and Lutman [14] proposes to use, in alternative, the tensile strength of masonry according to the Turnšek and Čačovic criterion [17]. V_m is based on the shear stress acting on the masonry cross-section, evaluated on the basis of the aspect ratio of the wall, for the Australian code [12]. The US standard [11] evaluates V_m as a function of square root of the masonry compressive strength f'_m, taking into account also the aspect ratio, which implicitly recalls the tensile strength of masonry. The New Zealand code [10] takes into account, in this term, the dowel-action of vertical bars, the aspect ratio and the degradation of shear strength, see also [15]. These terms are derived by [2] and [16]. The American and New Zealand formulations [2; 10-11; 16] consider separately the effect of axial load (V_P), which is implicitly taken into account by the V_m contribution when this is calculated according to Mohr-Coulomb or Turnšek and Čačovic criteria. The Australian code [12] is the only one to neglect, implicitly or explicitly, the effect of axial load. The contribution of horizontal reinforcement, V_s, is calculated as for stirrups in reinforced concrete members, taking into account the number of stirrups, each of area A_{rh}, across the diagonal crack (with 45° slope). The maximum tensile capacity of shear reinforcement is multiplied by a reduction factor C_{rh} that varies from 0.5 to 0.8 [1; 8; 10; 12; 16]. C_{rh} is 0.3 and 0.9 respectively in [14] and [9], but in these cases it multiplies the total area of horizontal reinforcement, A_{sw}. The British Standard does not apply any reduction factor [13]. Shing et al. [2] propose to neglect the contribution of the top and bottom reinforcing bars, and to consider fully effective all the others. Tomaževič [1] applies such a large reduction to horizontal reinforcement strength on the basis of experimental results, but he is the only one to take into account separately the contribution of vertical reinforcement, by analytical evaluation of dowel-action.

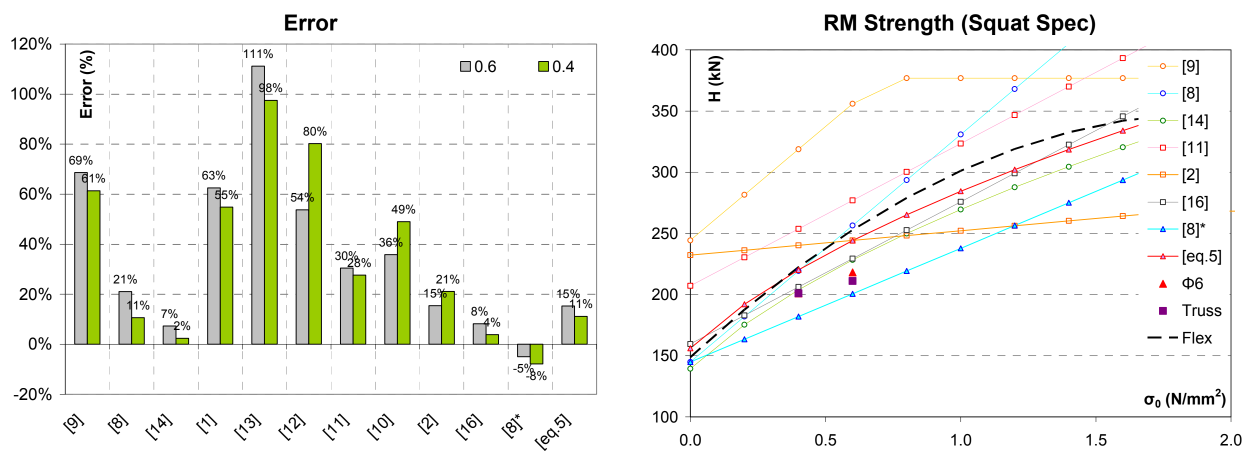
COMPARISON OF PREDICTED AND EXPERIMENTAL VALUES

The comparison between the experimental values of shear capacity (H_{max}) and the theoretical values ($H_{u,rs}$) has been carried out by taking into account all of the formulations in Table 3. Results (Table 4) are reported only for those specimens that actually failed in shear, and for which, in fact, the flexural model given in the last column of Table 4 does not provide good estimation of the ultimate capacity. Figure 2a gives the errors of the theoretical formulations given in Table 3. Figure 2b gives the shear capacity of reinforced masonry walls at different levels of applied axial stress, evaluated according to some of the proposed models.

The Australian Standard [12] overestimates the shear capacity despite it has been applied with a 45% reduction to take into account the percentage of holes in the unit, which reflects the code proposal for hollow unit (but not, in general, for perforated unit as in our research). The British Standard [13] overestimates the contribution of both unreinforced masonry and horizontal reinforcement. The European [9] and New Zealand [10] codes are characterized by similar trends. They are both limited at a certain axial load level, but [10] is closer to experimental values as it better evaluates the shear reinforcement contribution. Tomaževič [1] and the Italian code [8] still have a trend similar to the latter, but they provide better evaluation of reinforcement contribution. Shing et al. [2] is characterized by low increase of shear strength with the increase of applied axial load which, conversely, is too high for all the previous relations. The contribution of shear reinforcement is quite similar to [8], although it is obtained by two different procedures. Anderson and Priestley relation [16] provides the best prevision of the experimental data together with Tomaževič and Lutman's [14], since they are able to catch the influence of axial load and the contribution of shear reinforcement (which is still similar to that of [8] and [2]). The US code [11] substantially traces Anderson and Priestley's formulation [16], with a difference, on unreinforced masonry contribution, which leads to a small overestimation. Tomaževič and Lutman formulation [14] estimates the unreinforced masonry shear strength similarly to Anderson and Priestley's [16], but adopting the phenomenological model based on the referential tensile strength of masonry. The formulation proposed by the Italian Code [8] provides the best fit with the experimental data among the various formulations proposed by standards (Figure 2a). This is probably because the reinforcement reduction factor, C_{rh}=0.6, proposed by the code, is very close to the experimental one (see C_{eff} in Table 1). Obviously, the calibrated version of the Italian code formulation (see [8]* in Table 4), where the reinforcement reduction factor is kept to 0.6, and the value of friction for the unreinforced masonry shear strength (according to eq. 2) is calibrated on the basis of the masonry type, gives even better results. The errors (similar to those obtained with [14; 16]) are in the lowest range, and they show that the experimental shear capacity is only slightly underestimated.

Tomaževič and Lutman [14] and Anderson and Priestley [16] formulations give the smallest errors, and they seem to interpret adequately the influence of the axial load and the importance of the horizontal reinforcement contribution. In addition, in both equations, the unreinforced masonry shear strength is not based on a frictional, Mohr-Coulomb type of criterion, which is very often not consistent with experimental observed failure modes, but it is based on the unreinforced masonry tensile strength. Further modification of the available shear equations, considers keeping the contribution for the horizontal reinforcement V_s as in [8] and [8]*, but to use tensile strength to calculate the V_m (unreinforced masonry) contribution (equation 5). The results of eq. 5 are compared with experimental data in Figure 2 and Table 4. It is possible to observe that the experimental data are well approximated and that the trend of the new formulation is similar to [14] and [16], being much simpler than the latter.

$$V_R = V_m + V_s = \left(\frac{f_t}{b} \cdot \sqrt{\frac{\sigma_0}{f_t} + 1} \right) \cdot td + 0.6 \cdot \frac{d \cdot A_{rh} f_y}{s} \tag{5}$$



(a) (b)

Figure 2: Errors of theoretical formulations for shear strength of RM walls (a), shear equations vs experimental data for squat specimens (b).

Table 4: Comparison between experimental and calculated shear capacity of walls.

Specimen		H_{max}	$H_{u,rs}$												$H_{u,f}$
			[9]	[8]	[8]*	[14]	[1]	[13]	[12]	[11]	[10]	[2]	[16]	[eq.5]	[eq.3]
		kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN	kN
SRSa 0.6	V_s	-	105	59	59	35	27	117	86	54	79	61	49	59	-
	others	-	251	197	141	193	320	330	244	223	212	183	180	183	-
	V_R	218	356	256	201	228	347	447	330	277	291	244	229	242	253
TRSa 0.6	V_s	-	116	66	66	39	30	129	118	59	87	68	55	66	-
	others	-	251	197	141	193	320	330	244	223	212	183	180	183	-
	V_R	211	367	263	207	232	350	459	362	282	299	251	235	249	253
SRSa 0.4	V_s	-	105	59	59	35	27	117	86	54	79	61	49	59	-
	others	-	214	160	123	169	283	274	244	200	212	179	157	160	-
	V_R	201	319	219	182	204	310	391	330	254	291	240	206	219	223
TRSa 0.4	V_s	-	116	66	66	39	30	129	118	59	87	68	55	66	-
	others	-	214	160	123	169	283	274	244	200	212	179	157	160	-
	V_R	201	330	225	189	208	313	403	362	259	299	247	211	226	223

CONCLUSIONS

From the analysis of the experimental results, it is possible to say that the contribution of the shear reinforcement, H_{rh}/H_{max}, remains constant and around 40%, independently by the shear reinforcement ratio and by the axial load level. The effectiveness of shear reinforcement, C_{eff}, decreases with increasing reinforcement ratios. This is consistent with the previous conclusion. The effect of the axial load level on the C_{eff} factor is not proved, since there are not reliable data. The horizontal reinforcement reduction factor, C_{rh}, can be of 0.6 for this reinforced masonry system, which is consistent with the experimental C_{eff} factor, and it is in agreement with the values obtained by the previous research works. The horizontal reinforcement reduction factor is usually consider not to be affected by the axial load level, but as for the C_{eff} factor, further investigation are needed to define this dependence. Furthermore, C_{rh} can be used to derive the number of bed joint reinforcement involved by diagonal shear crack, imposing the equivalence with C_{eff}. The vertical reinforcement is completely exploited for both type of specimens, squat and slender, i.e. in both flexural and shear failure modes. The contribution of vertical reinforcement to the flexural capacity is at least 50%; and the flexural capacity returned by eq. 3 is adequate.

By applying various shear equations, it can be seen that, in general, the contribution of unreinforced masonry and axial load level is greatly variable. This generally leads to overestimation of shear capacity of reinforced masonry walls. A large number of shear equations keeps quite far from flexural failure curve also for high axial load levels (higher than those adopted during our tests), where presumably shear failure would occur. These equations are thus, very likely, incorrect. The contribution of horizontal reinforcement seems to be adequate for those formulations that consider a reduction factor between 0.30 and 0.60, as also confirmed by strain measurements. The shear capacity is adequately foreseen by the empirical formulation given by [16], and by those based on unreinforced masonry tensile strength [14 and 16]. Among the code prescribed formulations, that given by the Italian code [8] brings, for this masonry type, to acceptable results. The new formulations proposed, on the basis of these considerations, are consistent with the experimental data, although further validation is necessary.

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