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ON THE DETERMINATION OF STRENGTH OF ANCIENT MASONRY WALLS VIA EXPERIMENTAL TESTS

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SUMMARY

This paper presents a discussion on the two most commonly used in situ shear tests for masonry walls: the direct shear test and the diagonal compression test. Difficulties of interpretation of the data obtained with the two different tests are briefly discussed. Two experimental research projects were performed by the Department of Civil Engineering of Florence to characterize the mechanical properties of stone masonry walls in old buildings of Tuscany. The results of these researches are used to furnish a first estimate of shear strength, deformation properties and ductility capacity of the walls. Design shear strengths here obtained are higher up to 50 % with respect to the values established by the current Italian recommendations.

INTRODUCTION

In the last decades, vulnerability of masonry structures to earthquakes have focused the attention of politicians, researchers and structural engineers to prevent losses of human lives and damages to the buildings. These problems are of most relevance in old urban and rural nuclei in which masonry buildings are particularly prone to seismic actions. Past earthquakes demonstrated that especially old stone masonry buildings suffered severe damages, due to the poor seismic resistance of the shear walls [Chiostrini, Galano and Vignoli, 1998].

For structural engineers a main problem is the mechanical characterization of old masonry walls, i.e. shear strength and deformation parameters should be predicted. Knowledge of the textures and the properties of blocks and mortar are often insufficient to these previsions; so, experimental tests should be performed to achieve a reliable estimation of the above masonry’s characteristics. Several past researches have been performed on this topic but the literature concerning experimental studies on stone masonry walls with chaotic texture is rather sparse.

A contribution was given by in situ shear tests on stone wall-panels of buildings in Florence and Pontremoli (Tuscany), that were performed by the Department of Civil Engineering of Florence [Chiostrini and Vignoli, 1994]. More recently another experimental project concerning in situ tests on masonry walls of old buildings in Garfagnana and Lunigiana was performed.

These zones, in the North-West part of Tuscany, are seismic regions with prevalence of ancient stone masonry buildings, where a main fraction of inhabitants lives in old urban and rural nuclei. A main earthquake occurred on 7 September 1920 and interested particularly the center of Fivizzano (Lunigiana). In October 1995, a new earthquake of minor intensity (Magnitude 4.8 Ricther) caused further damaging to several masonry buildings in the area around Fivizzano, Aulla and other communes of Lunigiana; most of these areas are today considered of medium to high seismic risk.

Tests were performed on nine panels selected in six different buildings, according with compression, shear-compression and diagonal test setup; values of shear strength and deformation parameters were obtained from the tests and presented in [Chiostrini, Galano and Vignoli, 1998].

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The main part of this paper presents a discussion about the determination of the *referential shear strength* (τ_k or f_{vk0}), of shear modulus (G) and ductility capacity (μ) of these types of stone masonry walls; shear and diagonal

test results obtained in the above researches are used. A discussion about the two different test methods employed for in situ tests is performed. The Coulomb friction type (linear) and the Turnsek – Cacovic (POR method) equations are alternatively used to give an interpretation of experimental data, furnishing a first estimate of shear strength of the walls under examination. Average values of G modulus and ductility capacity under shear loads are also given.

Despite the insufficient set of data considered in this study, the discussion allows critical remarks on the strength values suggested by current recommendations [Eurocode 6, 1994; Italian Standards, 1981 and 1987] and the definition of appropriate safety coefficients to be applied to experimental strength values.

A BRIEF REVIEW OF TWO MOST COMMON IN SITU SHEAR TESTS

Direct Shear Test and Diagonal Compression Test

Two different tests are used for the determination of masonry walls shear strength: the direct shear test and the diagonal compression test [ASTM Standards, 1981]. Indicative setup of the two tests are depicted in Fig. 1. In the first scheme, the masonry specimen is considered as a short beam subjected to an average shear stress evaluated as $\tau = P/2A$ (P = horizontal load, A = area of the section of the panel); however, maximum shear stress in the center of the panel is evaluated as $\tau_{max} = 1.5\ \tau$ or more generally as $\tau_{max} = b\tau$, in which b is a shape factor that lies in the range between 1 and 1.5. Values of b are affected mainly by the height-to-depth ratio of the specimen. Despite its variability the b factor has little influence on the evaluation of τ_k.

Various enhancements of the original in situ test by Sheppard [Turnsek and Sheppard, 1980; Sheppard, 1985] permit the imposition of a vertical compression σ_0, so to obtain the shear strength for a well-established value of the confining action [Chiostrini and Vignoli, 1994]. The interpretation of the second test poses some interrogatives: this test was introduced to simulate a pure shear stress state, according to the scheme in Fig. 2a. In these conditions the Mohr circle of the stress state reduces to the one of Fig. 2b, leading to the corresponding value of average shear stress:

$$\tau = \frac{P_d}{\sqrt{2}A}, \tag{1}$$

being A the area of the side of the square panel; hence, the principal tensile stress σ_I is equal to the shear stress. If P_{du} denotes the maximum value of P_d, the shear strength is given by:

$$\tau_u = \frac{P_{du}}{\sqrt{2}A}. \tag{2}$$

On the other hand, a linear elastic analysis of the panel considered as an homogeneous solid, gives the localized value of the principal tensile stress in the center of the specimen as $\sigma_I \cong P_d/2A$ ($0.519\ P_d/A$), whereas the maximum value of the shear stress is $\tau_{max} = 1.1\ P_d/A$ (Fig. 3). According to this interpretation some Authors, as in [Tubi, 1993] and [Calderoni, 1996], equalize the shear without normal stress (namely f_{vk0} in the Italian Standards, 1987) to the tensile strength, assuming the equation:

$$\tau_u = f_{vk0} = \frac{P_{du}}{2A}. \tag{3}$$

Therefore, two different interpretations of diagonal test results are possible: according to the first one, the referential shear strength is evaluated as:

$$f_{vk0} = \frac{P_{du}}{\sqrt{2}A}, \tag{4}$$

whereas the second assumption gives the value of Eqn. (3). The first interpretation is the most commonly used for comparison purposes although some Authors proposed modifications to be used for interpreting and evaluating tensile strength of masonry by diagonal tests [Ghanem, Sheirf and Honsy, 1994].

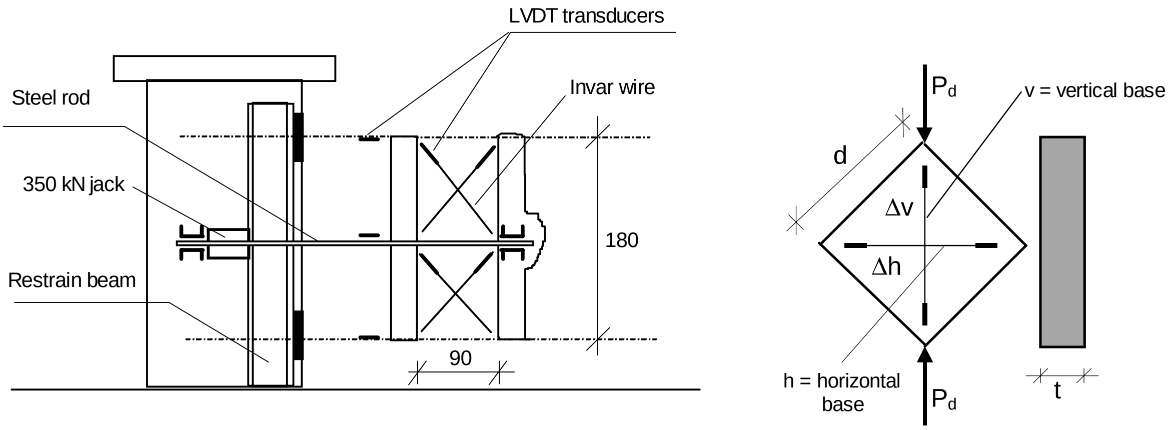


Figure 1: Direct shear and diagonal compression test setup

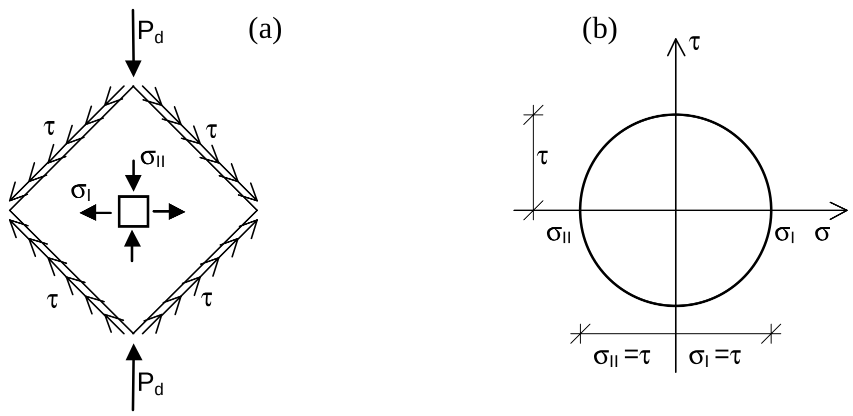


Figure 2: Pure shear stress state (a) and Mohr circle (b) (diagonal test)

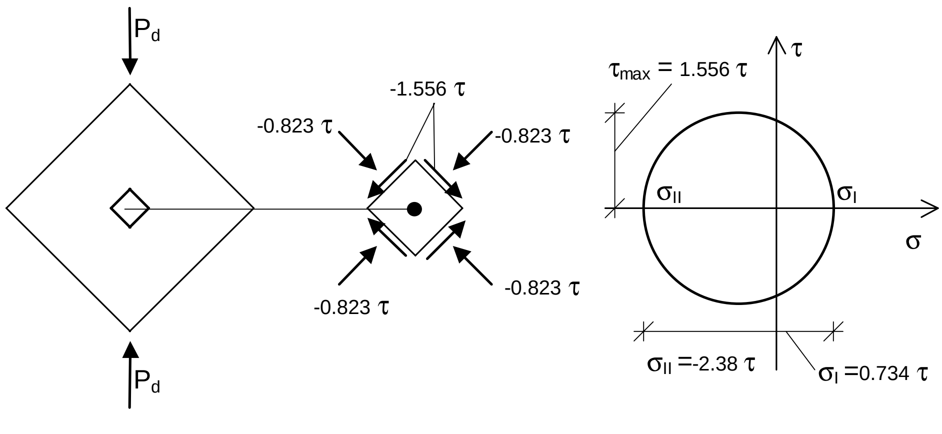


Figure 3: State of stress and Mohr circle in the center of the panel (diagonal test)

Code Recommendations

Actual codes evaluate the shear strength f_{vk} of masonry walls using two different groups of equations. In a first group the shear strength is evaluated with reference to a Coulomb type friction failure. In this hypothesis, a constant coefficient, said cohesion, is added to a linear contribution due the vertical stress σ_0 to give:

$$f_{vk} = f_{vk0} + tan(\phi)\sigma_0 , \qquad (5)$$

in which tan(ϕ) is the friction coefficient. Different formulations for tan(ϕ) have been proposed, based on different experimental results; with reference to new masonry walls, values of tan(ϕ) in the range between 0.3 and 0.8 are generally accepted. Despite this great variability, both actual Italian Standards (1987) and the recommendations of Eurocode 6 adopt the previous equation with tan(ϕ) = 0.4. The shear strength in absence of vertical stress, i.e. the cohesion f_{vk0}, is assumed as a function of the quality of the mortar and the characteristic compressive strength of the units and varies from 0.1 to 0.2 N/mm^2.

In a second group of equations the shear strength is evaluated as the average shear stress in a panel subjected to a vertical compression and to an horizontal load in its plane; the failure condition is achieved when the principal tensile stress σ_I in the center of the panel is equal to the tensile strength of the masonry, generally indicated with f_{wt}. This formulation is the one assumed in the well-known POR method and it has been widely verified through experimental tests on walls under shear-compression loads. The shear strength τ_u is defined as:

$$\tau_u = \tau_k \sqrt{1 + \frac{\sigma_0}{b\tau_k}} , \qquad (6)$$

in which b is a shape factor that takes into account the variability of the shear stresses on the horizontal section of the wall ($\tau_k = f_{wt}/b$). The determination of the parameters τ_k and f_{vk0} for ancient masonry walls should be based on extensive experimental tests for the typology of masonry texture under examination.

DISCUSSION

In the light of the previous considerations, two different questions appear: the first one regards the more appropriate way for the evaluation of the masonry shear strength from diagonal test whereas the second one concerns the reliability of results from equations (5) and (6) in calculating the masonry shear strength.

The evaluation of the "referential shear stress in absence of compression (τ_k or f_{vk0})" from diagonal tests and based on Eqn. (2) seems more appropriate when it is used to predict the shear strength via Eqn. (5). This conclusion is mainly due to the following considerations:

a) Eqn. (5) furnishes the shear strength of a wall in the hypothesis of constant shear stresses distribution equal to the average value according to Eqn. (2);

b) Eqn. (3) is derived considering a local failure criterion and a distribution of stresses evaluated by linear elastic analysis.

Anyway, in this case the interpretation scheme appears too crude, especially for ancient stone masonry walls, in which the chaotic texture causes a distribution of stresses inside the panel surely different from the one calculated considering an homogeneous elastic body.

Evaluation of τ_k from shear-compression test is generally based on Eqn. (6) in which σ_0 and τ_u are directly obtained from the test. The b shape factor varies from 1 (hypothesis of constant shear stress distribution) to 1.5 (according with Jourawski). Given the span-to-depth ratio of the panel (short beam) is generally accepted that the b factor must be calibrated for each particular type of masonry and boundary conditions. Italian Standards use b = 1.5. In the next section the in situ tests performed by the Department of Civil Engineering of Florence are used to furnish a contribute on these topics.

EVALUATION OF EXPERIMENTAL DATA

In the recent past the Department of Civil Engineering of Florence performed two extensive experimental research projects to assess the strength of in situ masonry walls of old buildings of Tuscany. In the first research [Chiostrini and Vignoli, 1994] in situ shear tests were performed on nine masonry panels selected from four different buildings: the S. Orsola monastery in the historical center of Florence (four panels, T1, T2, T3 and T4), an existing building in Florence (three panels, COR1, COR2 and COR3) and two buildings in Pontremoli (Lunigiana), "Istituto Belmesseri" and "Palazzo Comunale" (one panel for each case, BEL and COM). Characteristics of these buildings were: bearing walls made with stone or mixed stones and bricks masonry with chaotic textures and wood floor slabs with insufficient linkage between slabs and walls. Thickness of the panels varied from about 300 mm to 600 mm.

In the second research [Chiostrini, Galano and Vignoli, 1998], in situ tests were performed on seven masonry panels selected in five different buildings, according with shear-compression and diagonal test setup. Common characteristics of these buildings were: two or three stories height, bearing walls made by stone masonry with typical chaotic textures and wood or steel floor slabs.
Fig. 4 shows indicative setup used for shear-compression and diagonal tests, whereas in Fig. 5 a panel before the test and a typical masonry texture are depicted. For a more detailed description of the two experimental projects and results, see the above cited References.

Discussion

Tables 1 and 2 show results by shear and diagonal tests performed on wall panels and reported in [Chiostrini and Vignoli, 1994], with exception of T2 test and [Chiostrini, Galano and Vignoli, 1998] respectively. For diagonal tests the G shear moduli were evaluated as secant values at the load level $P_d = P_{du}/3$ ($G = G_{1/3}$) being P_{du} the ultimate load; for shear-compression tests G were calculated in the elastic range using equivalent shear beam models that taken into account the different boundary conditions applied to the specimens. Measure of ductility was defined as $\mu = \delta_{0.9}/\delta_E$, being $\delta_{0.9}$ the displacement in the middle of the panel at a load level equal to 0.9 P_u.
Panels can be tentatively classified in three main groups, according to different ranges for τ_k and τ_u: i) panels A, B, T4, COR3 and COM (High Strength, HS); ii) panels F, G, T1, T3 and BEL (Medium Strength, MS); iii) panels E, H, I, COR1 and COR2 (Low Strength, LS). Results of T2 test were not considered here.

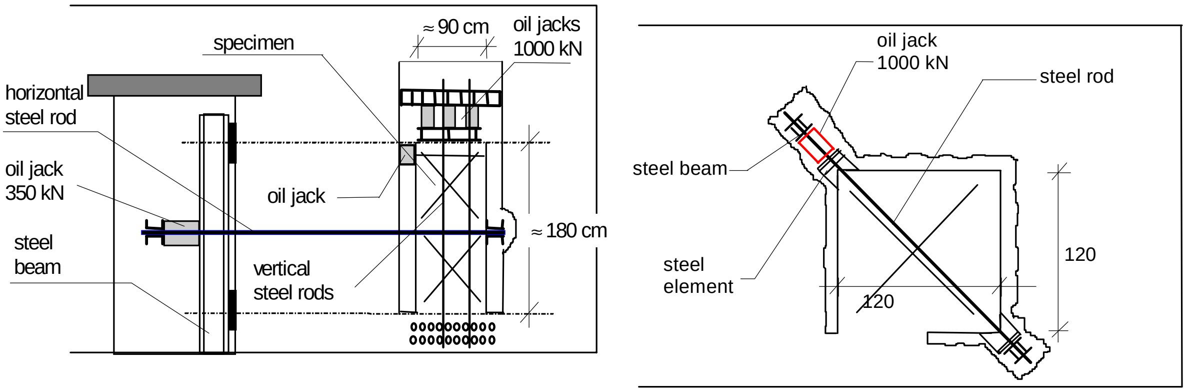


Figure 4: Shear-compression and diagonal test setup

Figure 5: Panel prepared for the shear test and typical masonry texture

Table 1: Shear test results

Test	Section A (cm^2)	σ_0 (N/mm^2)	τ_u (N/mm^2)	σ_I (N/mm^2)	b	τ_k (N/mm^2)	G (N/mm^2)	μ
A	5764.5	0.378	0.379	0.234 (0.410)	1.0 (1.5)	0.234 (0.273)	213	2.95
B	4797.5	0.433	0.491	0.320 (0.551)	1.0 (1.5)	0.320 (0.367)	781	2.87
E	5400.0	0.165	0.114	0.079 (0.107)	1.21 (1.5)	0.065 (0.072)	96	2.06
T1	4503.0	0.800	0.282	0.157	1.37	0.114	200	4.32
T3	4648.0	0.400	0.213	0.140	1.29	0.109	274	4.48
T4	3669.0	0.400	0.294	0.203	1.19	0.170	241	4.11
COR1	2640.0	0.230	0.150	0.100	1.23	0.081	173	3.03
COR2	2760.0	0.430	0.190	0.120	1.33	0.090	325	3.28
COR3	2511.0	0.120	0.250	0.200	1.0	0.197	333	3.17
BEL	4480.0	0.190	0.160	0.110	1.14	0.096	290	3.98
COM	2880.0	0.130	0.260	0.200	1.0	0.203	249	3.26

Table 2: Diagonal test results

Test	Section A (cm^2)	P_{du} (kN)	τ_u (N/mm^2)	σ_I (N/mm^2)	$\gamma_{1/3}$ ($\times 10^{-3}$)	$G = G_{1/3}$ (N/mm^2)
F	5160.0	83.33	0.114	0.081	0.142	285
G	5400.0	122.06	0.160	0.113	0.538	102
H	5760.0	58.99	0.072	0.051	0.661	36
I	6000.0	51.64	0.061	0.043	0.266	74

The three groups are also well distinguished by different masonry textures: HS corresponds to a good quality masonry, MS corresponds to a masonry with little internal voids, well filled by mortar and small dimension units, LS textures represent very poor assemblages of blocks and mortar, with many internal voids and facing walls weakly pinned. Shear test results on specimens A, B and E indicate little variability of τ_k using different values for b (see Table 1), whereas different interpretation schemes of the diagonal test lead to substantially different values ($f_{vk0} = \tau_u$ or $f_{vk0} = \sigma_I$, Table 2). Although the assumed set of data is obviously too poor to permit general conclusions, the values of τ_u by diagonal tests H and I are in good agreement with τ_k evaluated with the shear-compression test E (with similar texture). Average value of τ_k (based on b < 1.5 and $\tau_k = f_{vk0} = \tau_u$ for diagonal tests), G and μ here obtained are:

- for the HS texture: $\tau_k = 0.225$ N/mm^2; G = 363.4 N/mm^2; $\mu = 3.27$;
- for the MS texture: $\tau_k = \tau_u = 0.119$ N/mm^2; G = 230.2 N/mm^2; $\mu = 4.26$;
- for the LS textures: $\tau_k = 0.074$ N/mm^2; G = 140.8 N/mm^2; $\mu = 2.79$.

To furnish a more reliable evaluation of shear strength and compare the two Eqns. (5) and (6) the data of Tables 1 and 2 are used. For each group of data (HS, MS and LS) the values of average vertical stress σ_0 and average shear stress τ_u are fitted in three cases:

(1) by Eqn. (6) with b = 1.5; (2) by Eqn. (6) with b = 1.0; (3) by Eqn. (5).

Results are presented in Fig. 6, in which τ_k, f_{vk0} and tan(ϕ) are given. Data from [Chiostrini and Vignoli, 1994] are represented with " ○ " whereas data from [Chiostrini, Galano and Vignoli, 1998] are represented by "•". Both interpretation approaches of diagonal test results, as discussed in section 2.1, have been used for the fitting process. Graphs on the left are obtained employing Eqn. (2) to evaluate shear strength for panels F, G, H and I, whereas graphs on the right take into account the alternative approach of Eqn. (3).
Good results of the fitting procedure are quite evident especially for MS and LS masonry textures; for these groups of data, both linear and quadratic interpolation appears satisfactory in predicting average shear strength.
For HS masonry, linear fitting notably differs from quadratic interpolation; this is probably due to the larger dispersion of the data in this group. Average values of the ratio between G and τ_k are (b=1.5 and left graphs):

- HS: $G/\tau_k = 1425$; MS: $G/\tau_k = 1872$; LS: $G/\tau_k = 1738$.

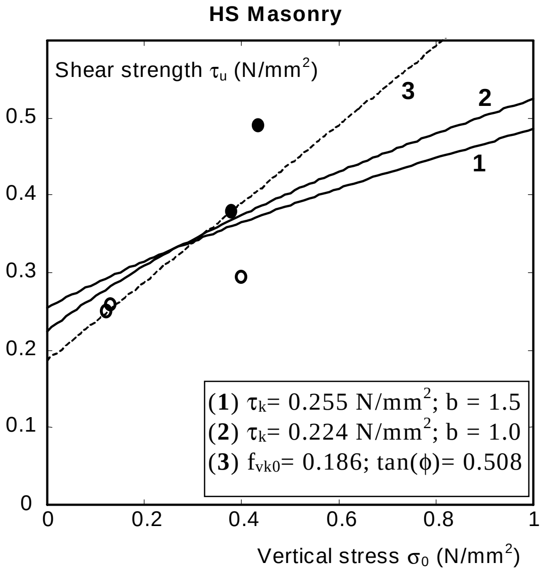
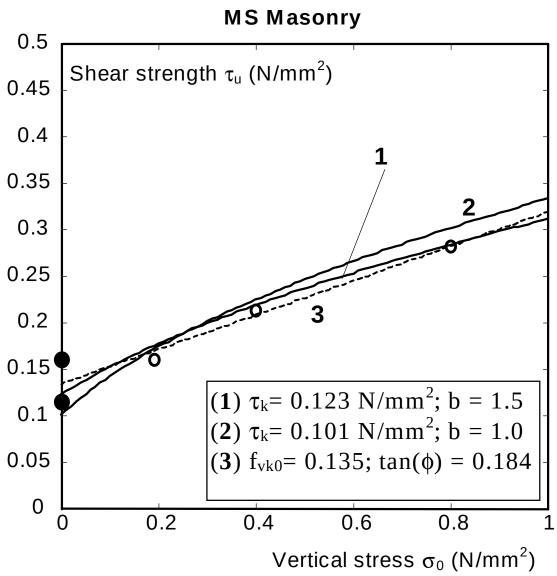
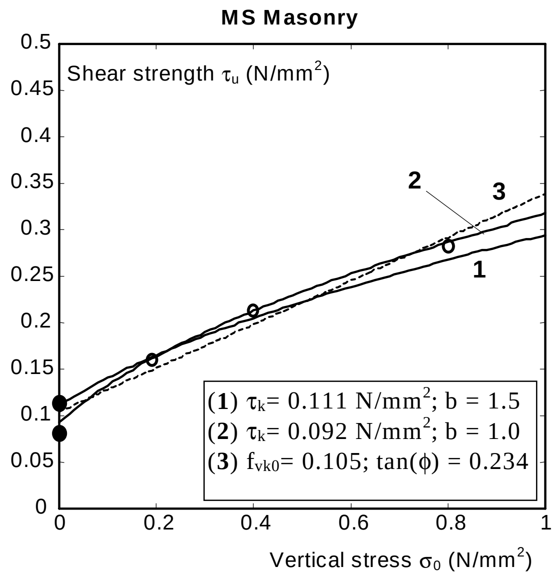
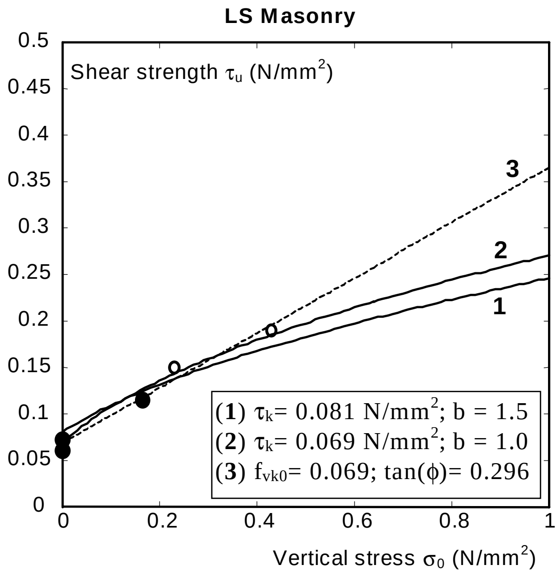
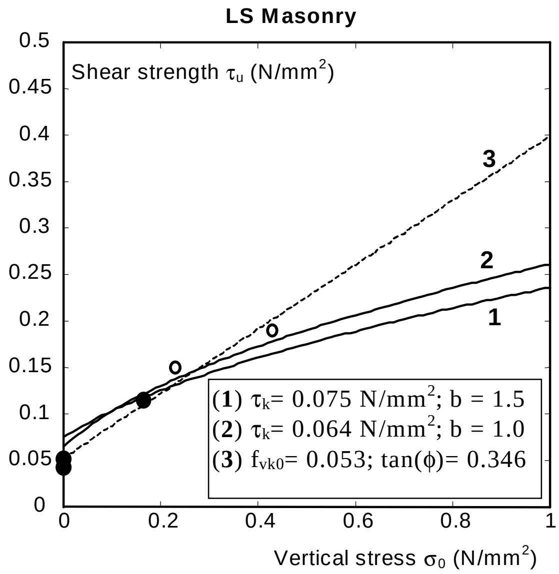


Figure 6: Best fitting of shear strength parameters for HS, MS and LS masonry. Equation of Turnsek-Cacovic with b = 1.5 (curve 1) and b = 1.0 (curve 2); Coulomb equation (curve 3, dashed). (•) Experimental data from recent tests in Lunigiana and Garfagnana; (○) data from a previous research.
Diagrams on the left: diagonal tests according with Eqn. (2), diagrams on the rigth: diagonal tests according with Eqn. (3)









These results indicate that higher values of G/τ_k ratios were obtained with respect to the value equal to 1100 suggested by Italian Standards (1981). Another conclusion is given by the high values of the ductility capacity here obtained (Italian Standards suggest a value equal to 1.5 for stone masonry shear walls). Given the τ_k values obtained from the direct fitting process is therefore possible to establish a suitable range for the referential shear strength to be used for design purposes; to do this a safety coefficient equal to 2.0 seems to be appropriate. Applying this coefficient, we obtain (using b=1.5 and the values of the left graphs):

- HS: τ_{kd} = 0.128 N/mm^2; MS: τ_{kd} = 0.062 N/mm^2; LS: τ_{kd} = 0.041 N/mm^2.

These values define the range of variation of τ_{kd} varying materials, textures and the quality of the mortar; this range is higher than that suggested by Italian Standards for existing masonry stone walls (τ_{kd} varies from 2 to 7 N/mm^2). This difference is probably due to prudential considerations that are appropriate in the rehabilitation design of ancient masonry buildings. Despite the small set of data here used the conclusions confirm the reliability of the present approach.

CONCLUSIONS

The discussion presented demonstrates: the need of extensive experimental projects to achieve larger amount of reliable data and an unified accepted interpretation approach, capable of furnishing reliable results to be used in rehabilitation designs. The paper gives a contribute in this direction, presenting some brief notes and splitting some experimental results accordingly to macroscopic characteristics of masonry walls and their overall mechanical properties. Further work is in progress to achieve a more supported position regarding this matter.

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ACKNOWLEDGMENTS. The paper was based on the results of the research project "Experimental investigation on the effectiveness of seismic strengthening techniques for historical masonry buildings in Lunigiana e Garfagnana", supported by "Regione Toscana". The M.U.R.S.T. (Ministry for University and Scientific and Technological Research) is also gratefully acknowledged.