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Experimental study on the determination of strength of masonry walls

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Abstract

The results of a research project carried out on masonry panels obtained from structures struck by the Umbria-Marchigiano earthquake of 1997–1998 are presented. The project consists of two parts: tests were performed in the laboratory, and in situ in order to determine the correct parameters describing masonry behavior. With regard to the laboratory tests, several compression tests were performed on cylindrical stone samples. Stone samples were obtained from the panels on which in situ tests had been previously carried out. Depending on the three types of in situ tests carried out (compression test, diagonal compression test, shear–compression test), different dimensions of panels were used using an appropriate cutting technique in order to leave the panels undisturbed. The shear strength and the Young and shear elastic modulus were measured. These results were compared with the values suggested by different standards. The experimental research allowed to characterize the mechanical properties of some typical masonry walls in old buildings of Umbria. These results are reported, together with an analysis of the masonry textures and sections.

Keywords: Masonry; Wall strength; Experimental

1. Introduction

Reconstruction work is now underway in the area struck by the 1997–1998 earthquake, but many difficulties could be eliminated if better technical information regarding the mechanical characteristics of soils and of masonry structures typical of this part of Italy are available. In their calculations, structural engineers and technicians have often referred to not well-identified parameters for different kinds of masonry walls found in scarce bibliography studies.

In this area, the classification and the analysis of historical masonry typologies were conducted in the past with different purposes in mind. With time, these studies supplied more and more exhaustive results. However, these contributions have rarely included an experimental part regarding the mechanical characteristics of masonry due to the effective difficulties involved in the determination of these characteristics. Masonry walls have been classified with regard to the constituent materials, section dimensions, texture and mortar types, but very rarely with regard to their shear strength and shear elastic modulus [1].

Classifications based on shear strength were first realized by Sheppard et al. [2] in Slovenia. Several shear–compression tests were carried out on panels from structures in the city of Lubiana. The compression stress was equal to that effectively hanging over the panels, but not completely well defined. In the recent past, Vignoli et al. [3] applied a similar test type on some historical buildings in Toscany. Vignoli et al. developed a test type, fixing the compression stress using oil jacks positioned over the panels.

The diagonal compression tests are clearly defined by ASTM Specifications [4], to which this experimental work refers. During the 1990s, diagonal tests were carried out in Italy by Vestroni et al., together with a number of shear–compression tests [5], on panels cut from buildings located in Abruzzo. Other similar tests were conducted by Modena [6].

All the above-mentioned experimental studies, due to the different kinds of tests adopted by the authors as

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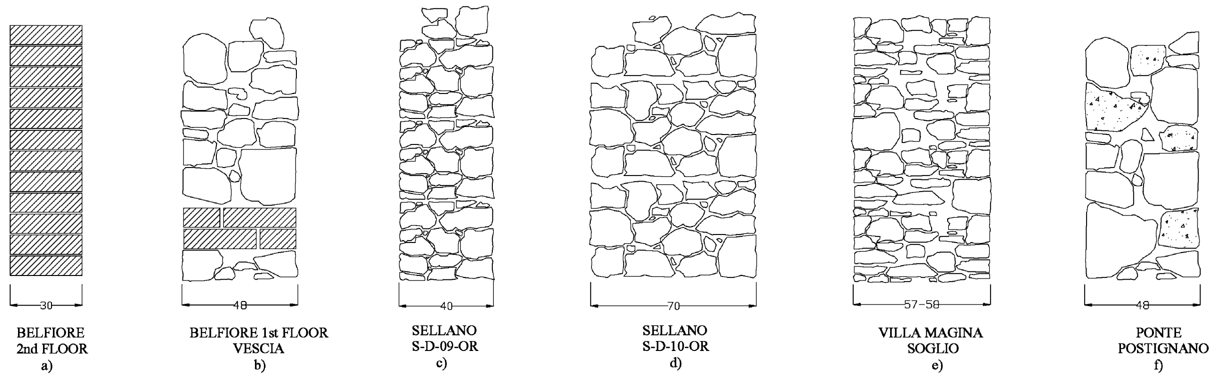

Fig. 1. Typical sections of the tested panels.

well as the different masonry types (i.e. constituting materials and masonry textures), show a large scattering of the results.

However, it must be considered that in Italy most of the seismic regions are located in the Apennines, in rural areas characterized by 'poor' masonry. In the Umbrian-Marchigiano portion of the Apennines, roughly cut stones and lime-based mortars have been used in construction for centuries. All these materials were obtained from quarries located near the urban centers. Even the dimensions of masonry sections, determined by the height of the buildings usually no more than three floors in height, are within the range of 45–60 cm. All these facts determined a substantial similarity of construction techniques, but a strong differentiation of the masonry constituent materials were linked to site availability.

The stone and brickwork, when correctly assembled, is essentially effective with respect to vertical static loads. The recent seismic events which struck Umbria and Marche in 1997–1998 have evidenced notable inadequacies of the masonry due to its almost total lack of resistance in traction. Most of the damaged buildings are located in historical centers, essentially terraced housing, in which this lack of resistance led to the heaviest damage.

This study is part of a larger research project, which also includes a report on an investigation of the strengthening techniques tested on masonry structures. This project was commissioned to the Laboratory on Anti-seismic Research, RITAM, of Terni by the Deputy Commissioner for interventions in the Umbria areas damaged by the earthquake. The research project included a series of 15 masonry panels of various dimensions subjected to compression, diagonal compression and shear–compression tests. These tests, carried out on site, allowed the evaluation of shear strength, Young and shear elastic modulus and shear strains of the masonry constituting the un-strengthened panels. The reader can refer to another paper on an investigation of the seismic-upgrading techniques employed to strengthen the masonry panels [7].

The area in which the tests were carried out is at high seismic risk: during the last 20 years it has been struck by two seismic events (1979 and 1997) with magnitudes grater than 5 (Richter scale). All the panels were obtained from seven buildings [located in Belfiore, Vescia, Soglio, Ponte Postignano, Sellano (two buildings), Villa Magina]. The buildings were chosen since they were representative of the most common masonry textures in the above reported areas. All the panels were cut from undamaged walls.

2. Characterization of masonry materials

The masonry sections of the panels is shown in Fig. 1. With the exception of two panels made with solid bricks (see Fig. 2), the remaining panels are made with roughly cut stones. These walls are composed by two weakly connected leaves (at the Vescia and Belfiore buildings two solid brick courses are interposed at intervals of approx. 80–120 cm) and the calcareous stones are white- and pink-colored. The presence of sponge travertine was observed at the Ponte Postignano

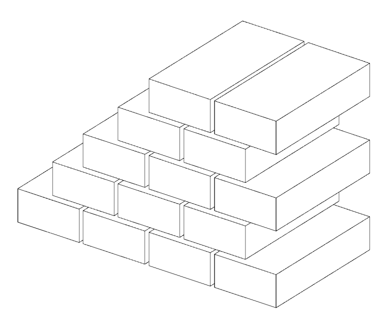

Fig. 2. The brick masonry texture of the panel located in Belfiore.

Fig. 3. One of the eight sponge travertine samples.

building, mixed together with the white and pink calcareous stones (see Fig. 3).

The dimensions of the stones vary for the different buildings from which the panels were cut. Larger stones were present in Belfiore, Vescia and Ponte Postignano (average dimension of the longest edge equal to 30 cm) while smaller stones constituted the panels at Soglio, Sellano and Villa Magina (average dimension of the longest edge equal to 15 cm)

After the panels were taken to failure by in situ tests, a series of samples were obtained from them. Fourteen cylindrical samples made of the stones (two samples of white calcareous stone, four of pink calcareous base stone, eight of sponge travertine stone) were tested in compression. The compression load was applied at a speed of 0.30 MPa/s using a Metrocom PIP 300V machine. The test consists in a monotonic loading up to the point of failure.

The results show a significant scattering in the data of the tests carried out on the sponge travertine. This depends on the high un-homogeneity of the stone due to the presence of large and frequent voids. If sample number 8 (weight 10.50 N) and number 13 (with large voids; weight 6.00 N) are not considered, the average values of the density and of the compression strength are respectively equal to 13.35 kN/m^3 e 2.66 MPa (see Table 1).

The results obtained from the tests carried out on the calcareous stones, show that, even if the density of these stones is sufficiently constant (average values equal to 23.30 kN/m^3 for the pink color one and 24.85 kN/m^3 for the white color one), the values of the strength depend significantly on the presence of inclusions, in random orientation, inside the samples. These caused a compression strength decrease of up to 60% compared to the highest values measured. Regarding the pink color calcareous base stone, the compression strength of sample number 3 is equal to 90.3 MPa while the average value is 57.5 MPa. The two white-colored stone samples show an average compression strength of 36 MPa.

The mortars are all lime-based in consideration to the absence of portlandite and of silicates of calcium and aluminum. The chemical analysis shows that the main differences are in relation to the period of construction of the buildings: the Ponte Postignano mortar has a high weight ratio of cement/aggregates. The other buildings, constructed before Ponte Postignano one, have a smaller value of this ratio and the mortars have small quartz traces removed by the erosive action of water.

3. The in situ experimental work

The walls were tested under compression, diagonal compression and shear–compression. These tests involved the use of panels of two different dimensions: 120×120 cm^2 for the diagonal compression tests and 90×180 cm^2 for the compression and shear–compression tests. All panels were cut using the diamond-wire technique and isolated from the remaining masonry walls in order to leave the panels undisturbed.

The panels are identified by a four index code, in which the first indicates the location of the structure from which the panels were obtained (B = Belfiore, V = Vescia, G = Soglio, and P = Ponte Postignano); the second, the type of test (D = diagonal compression, T = shear–compression, C = compression); the third, the identification number of the panel, while the fourth index indicates the type of intervention carried out (in this case the fourth index is always OR because this paper reports only the results on un-strengthened panels, with the exception of tests identified by codes V-T-07-IN and V-C-07-IN in which the strengthening technique using preventive injections resulted as not effective).

3.1. Compression tests

The compression tests were carried out on panels of 90×180 cm^2 dimension, with maximum section thickness of 65 cm. The test is non-destructive and consists in three cycles of loading and unloading with increasing

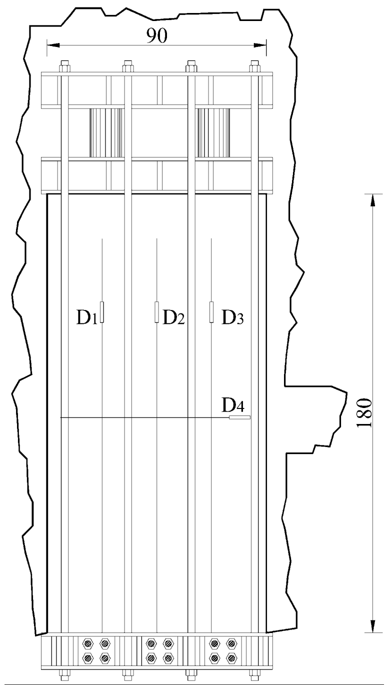

Fig. 4. Position of the inductive transducers during the compression test on both sides of the masonry panel.

maximum values of the vertical compression stress of, respectively 0.1, 0.2 and 0.3 MPa. The compression tests were designed to determine the Young modulus of elasticity. These values were also necessary in order to elaborate the data of the shear–compression tests. The test mechanism is composed of two metallic plates positioned over the panel and two hydraulic jacks, interposed in parallel between the plates, in order to permit that the panel be subjected to uniformly distributed compressive stress. The first plate was positioned over the two jacks and it was rigidly connected, by means of eight steel rods, to the base of the panel, where two metallic elements were anchored to each side of masonry wall. The second plate was instead placed under the two jacks and rested directly on the panel on a bed of mortar. During the loading, the two jacks compress the two plates: the first one is impeded to translate and it acts as a base for the two jacks, which compress the panel through the second plate. Each side of the panel was instrumented with three vertical inductive transducers. A horizontal inductive transducer was positioned on the center line on each of two sides, for a total of 11 channels of acquisition (displacements of the eight inductive transducers, pressure at the two jacks, time) (see Fig. 4).

3.2. Diagonal compression tests

The diagonal compression test, as well as the shear compression test, was designed in order to evaluate the shear strength, the shear elastic modulus and the ductility of the masonry. The diagonal test was carried out on panels 120×120 cm^2 with a maximum cross-section thickness of 70 cm. The panel remained anchored to the rest of masonry wall through a part of the 70 cm of the lower horizontal edge. The remaining three edges and a part of the fourth were cut and isolated from the rest of the masonry wall.

The test mechanism is composed of a set of metallic elements fixed at the two corners of a diagonal of the panel. A jack, placed at one corner, is interposed between two metallic elements which permit it, on the one hand, to act directly on a corner of the panel, while at the same time resulting in a rigid connection to an analogous metal element located at the opposite corner. A closed system is obtained in which the jack compresses the panel along one of two diagonals (see Fig. 5).

Both diagonals of the panel were instrumented on both sides with inductive transducers. The total number of the channels of acquisition was six (displacements of the four inductive transducers, pressure at the jack, time). The test consisted in equal couples of cycles of loading and unloading, with increases of 10 kN, up to the point of failure (see Fig. 6).

During the project phase of the compression test, particular attention was directed to the problem of the load distribution along the corners in order to avoid an excessive concentration of compression stresses at these surfaces. The metallic element, interposed between the jack and the corner, was carefully designed. Consequently, failure of the panels never occurred due to excessive compression stress at the corners.

The analysis of the results of the diagonal compression tests is the object of interpretation differing from author to author. This test was introduced to simulate a pure shear stress state. In these conditions the Mohr circle of the stress state is centered in the origins of σ–

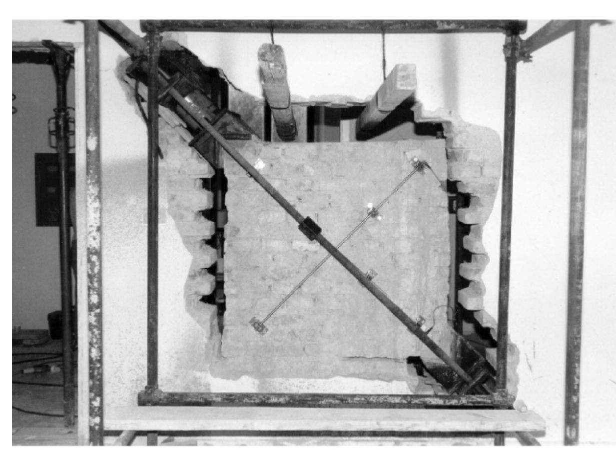

Fig. 5. Layout of the diagonal compression test on the brick panel.

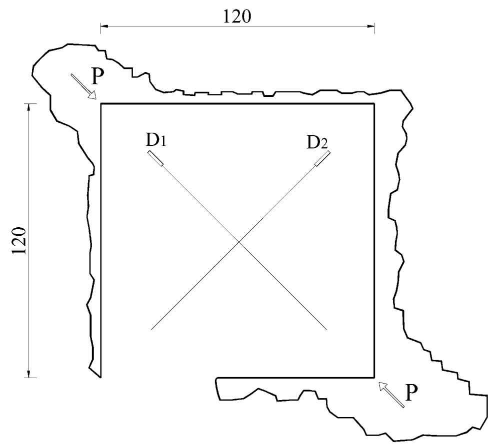

Fig. 6. Position of the inductive transducers during the diagonal compression tests.

τ axis and the value of the average shear stress τ, equal to the principal tensile stress σ_I, is given by:

$$S_s = \sigma_I = \frac{0.707P}{A_n} \tag{1}$$

in which P is the diagonal compression load and A_n net area of the panel, calculated as follows:

$$A_n = \left(\frac{W+h}{2}\right) tn \tag{2}$$

where W = width of specimen, h = height of specimen, t = total thickness of specimen, and $n=1$ (% of the cross area of the unit that is solid, expressed as a decimal).

According to this interpretation, which is that more frequently used [2], the shear strength is evaluated as:

$$\tau_{k,nom}^{diag} = S_s^{\max} = \frac{0.707P_{\max}}{A_n} \tag{3}$$

where $P_{\max}$ is the maximum load applied by the jack, A_n is the net area of the panel, and $S_s^{\max}$ is the shear strength for a diagonal compression test according to ASTM E 519-81 specifications

$$f_{vk0,nom}^{diag} \cong \tau_{k,nom}^{diag} \tag{4}$$

where $\tau_{k,nom}^{diag}$ is the nominal shear strength according to Circolare [10], and $f_{vk0,nom}^{diag}$ is the nominal shear strength according to Italian Standard [9] and Eurocode 6 [8] specifications.

3.3. Shear–compression tests

The shear–compression test was carried out on the same panels previously used for the compression tests. Considering that the compression test is non-destructive (see Fig. 7), the same test apparatus (plates, steel rods, jacks) was used to give the compression stress to the panel. For the duration of these tests, a constant vertical compression stress of 0.3 MPa was applied to the panel.

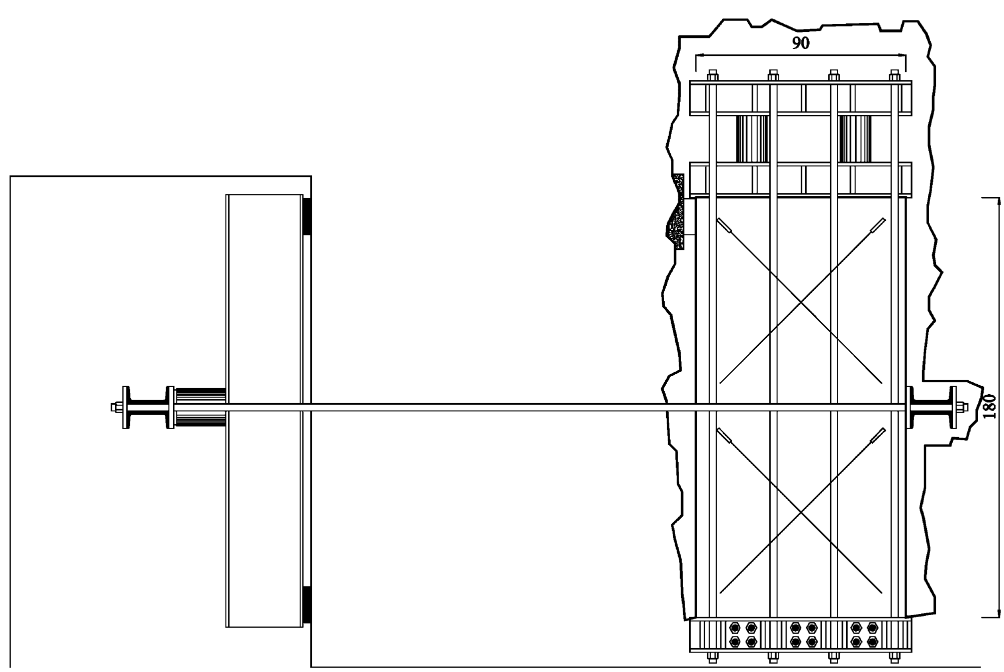

Fig. 7. Layout of the shear–compression test.

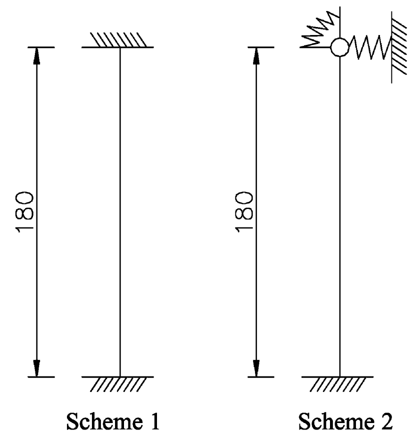

Fig. 8. The two schemes used for the elaboration of the data of the shear–compression tests.

The shear load was applied by two steel rods which acted on a special metal element made of two C shapes, coupled with plates welded to the webs, positioned at the center line of the panels. This metal element has the function of distributing the shear force through out the panel thickness. The two steel rods were connected, on the one hand, to the metal element and on the other hand, to an analogous element. A hydraulic jack was interposed between these two elements. During the loading (monotonic up to the point of failure) the jack acts on the second metal element and then on the two connected steel ties, thus resulting in traction.

The upper half of the panel was contrasted by means of a few 100-kN jacks positioned horizontally in order to avoid flexural failure mechanisms. This couple also allowed measurement of the horizontal reaction at the top of the panel. The presence of the apparatus overhanging the panel was not enough to constitute a perfect constraint. The upper half of the panel was able to translate and rotate while the lower half, connected to the rest of the masonry, could be considered as a perfect constraint. This caused a lack of symmetry in shear distribution between the upper and lower halves of the panel, which was taken into account during the elaboration of the data. As a consequence of this lack of symmetry, the lower half of the panel was always more stressed and failure always occurred here. Two different schemes were used in the elaboration of the data (see Fig. 8).

Eight inductive transducers were positioned along the diagonals of the four halves obtained by subdividing each of the two vertical sides of the panel in two equal parts. Six other traducers were positioned along each side of one vertical edge (at the base, the center point, the top of the panel) and two more transducers were placed on one side to measure vertical movements on the edge of one side of the panel and eventual rotations at the top of the panel. In addition to the displacements of the 16 inductive transducers, measurements were also acquired of the time and pressure in the two vertical and three horizontal jacks, for a total of 21 channels of acquisition (see Fig. 9).

During the shear–compression test, the failure condition occurs when the principal tensile stress σ_I in the center of the panel is equal to the tensile strength of the masonry. In order to evaluate the shear strength of the masonry, the well-known Turnsek and Cacovic formulation is assumed:

$$\tau_{\max} = \tau^{sc}_{k,nom}\sqrt{1+\frac{\sigma_0}{b\tau^{sc}_{k,nom}}} \tag{5}$$

where

$$f^{sc}_{vk0,nom} \cong \tau^{sc}_{k,nom} \tag{6}$$

and in which σ_0 is the vertical compression stress equal to 0.3 MPa, and $\tau_{\max}$ is the maximum shear stress defined as:

$$\tau_{\max} = \frac{T_{\max}}{A} \tag{7}$$

where $T_{\max}$ is the maximum shear load in the lower half of the panel, A is the horizontal cross-section of the panel, and b is a shape factor that takes into account

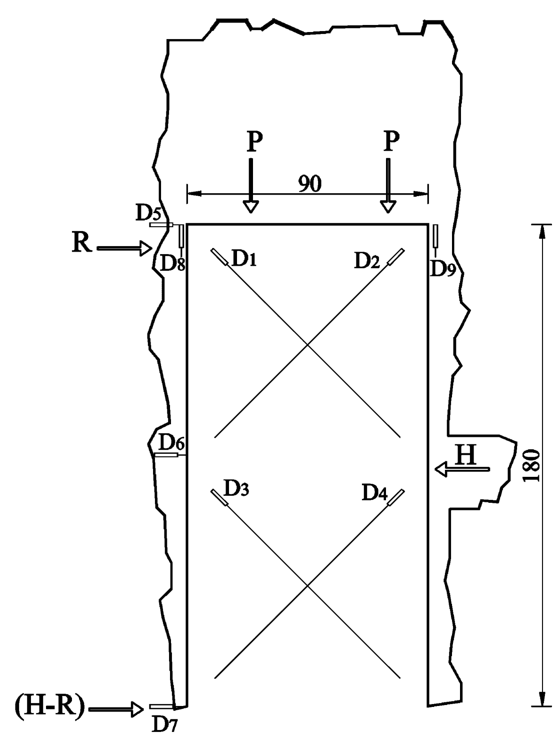

Fig. 9. Position of the inductive transducers during the shear–compression tests on both sides of the masonry panels.

the variability of the shear stresses on the horizontal section of the wall. This parameter is assumed by the Italian Standards and the well-known POR method to be equal to 1.5. The variable $\tau^{sc}_{k,nom}$ is the nominal shear strength for a shear–compression test according to the Turnsek and Cacovic formulation, and $f^{sc}_{vk0,nom}$ is the nominal shear strength according to Italian Standard [9] and Eurocode 6 [8] specifications.

3.4. Determination of shear strength

The passage from the nominal values of shear strength to the characteristic ones is not well defined by the actual standards. The Italian Standard D.M. 20.11.1987 [9] imposes that the determination of the shear strength must be obtained from at least six tests carried out on panels and it is given by:

$$\tau_k \cong f_{vk0} = k f_{vm} \tag{8}$$

where

$$f_{vm} = \frac{1}{6}\sum_{i=1}^{6} f^{i}_{vk0,nom} \tag{9}$$

$$k = 0.7 \tag{10}$$

In which $f^{i}_{vk0,nom}$ is the nominal shear strength of sample i.

Another Italian Standard [10], with no reference to the number of tests to be carried out, assumes for the k coefficient:

$$k = \frac{1}{2.5} = 0.4 \tag{11}$$

Some Authors in a previous research project [3], adopted $k = 0.5$.

During the elaboration of the data, the k coefficient was assumed equal to 0.6 considering that the number of panels similar in texture submitted to diagonal compression test was six while the number of shear–compression tests was instead three (considering also the injected panel number seven in which the strengthening technique was not effective).

4. Evaluation of the results

4.1. The Belfiore building

Five panels were cut off at the first building, located in Belfiore (a hamlet in the Foligno Commune). Three of them were tested without any kind of strengthening technique: two were submitted to a diagonal compression test (one on the first floor and the other on the second) while the remaining one (on the first floor) was subjected to the compression test and then to a shear–compression test. The masonry texture of this building, built at the beginning of 20th century to host the elementary school of the village, is made, for the first floor, of stone double-leaf walls with double solid brick courses interposed at intervals of 120 cm. The stones just roughly cut were calcareous and the double-leaf wall, weakly connected, had a thickness of 48 cm (see Fig. 1b). The mortar is lime-based. The masonry texture of the second floor is instead one-leaf made only of solid bricks (brick dimensions: $30 \times 15 \times 6.5$ cm^3) (see Fig. 2).

The results show significantly different values depending on the type of test carried out on the panels. Considering only the double-leaf walls, nominal shear strengths $\tau^{sc}_{k,nom}$ and $\tau^{diag}_{k,nom}$ was 0.130 MPa and 0.072 MPa, respectively, for the shear–compression test and for the diagonal compression test (see Figs. 10 and 11 and Tables 2 and 3).

Regarding the solid brick panel, it is significant to note that the particular brick texture caused a nominal shear strength $\tau^{diag}_{k,nom}$ of 0.069 MPa (see Table 3). After the failure of the panel, the cracks occurred only in the mortar courses, all the bricks were all undamaged due to the fact that the bricks are 6.5 cm high and the vertical joints are positioned at intervals of 15 cm, determining an angle of approximately 45° between brick vertical and horizontal joints. The typical shear cracks also have a slope of 45°, so they find a preferential direction of propagation inside the lime-based mortar joints.

4.2. The Vescia building

The masonry texture of the building located in Vescia is the same as the one at the first floor in Belfiore (double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–90 cm) (see Fig. 1b). One panel was strengthened with the preventive injection technique, but due to the absence of a sufficient number of voids, the strengthening technique failed in terms of the increment of strength. Considering the similar values obtained from a comparison with the ones in Belfiore, the data of this panel were reported together with those of unstrengthened panels.

4.3. The Villa Magina and Soglio buildings

The buildings of Villa Magina, from which one panel was cut, and of Soglio, from which two panels were cut (only one was tested unstrengthened), have a texture made of double-leaf roughly cut stone masonry walls (see Fig. 1e). Both panels were tested under diagonal compression. The masonry components are lime-based mortar and white calcareous stones at the Soglio build-

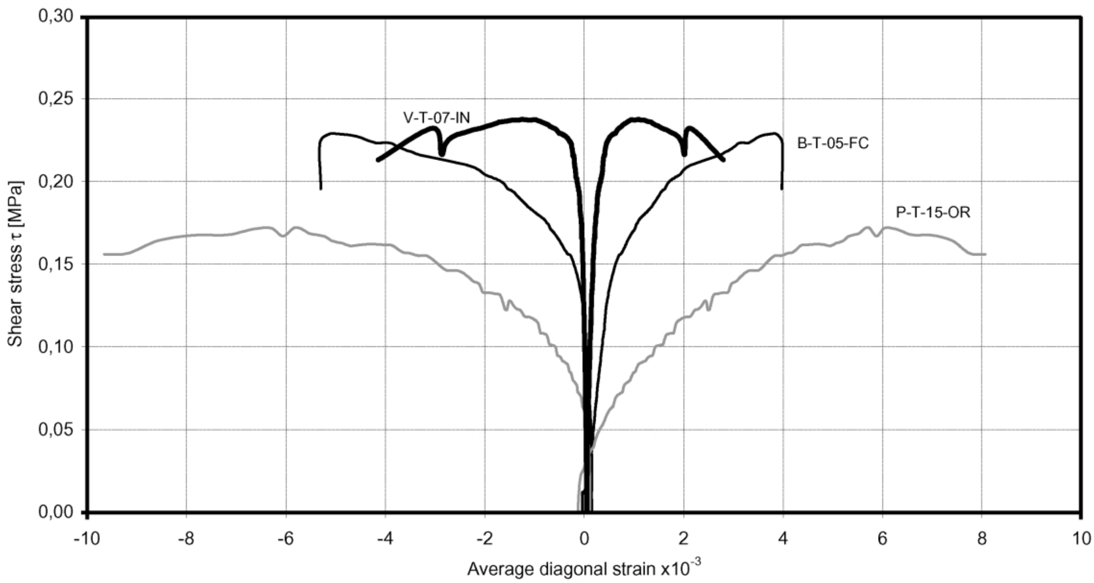

Fig. 10. Shear stress vs. average diagonal strain (shear–compression tests).

ing. In Villa Magina the pink calcareous stone is in place of the white one. The nominal shear strength $\tau_{k,nom}^{diag}$ obtained was 0.047 and 0.053 MPa, respectively, for the Villa Magina and Soglio panel. The shear modulus of elasticity $G_{1/3}$ measured at 1/3 maximum load, was equal to 19 and 26 MPa (see Table 3 and Fig. 11).

4.4. The Sellano buildings

The medieval village of Sellano is located on the Apennine mountains in center of Italy. The panels were cut off from two buildings in the center of the village. It was not possible to find the construction date of these two buildings. However, from the data of the land register of 1825 the buildings already existed.

A panel for a diagonal compression test was cut from the first building. This building is located in the main square of the village and the height of the building is approximately 4 m. The masonry texture is made of double-leaf roughly cut stone walls (see Fig. 1c) and the thickness of the panel is 40 cm (identification number of the panel = 9). The masonry components are lime-based mortar and white calcareous stones. The

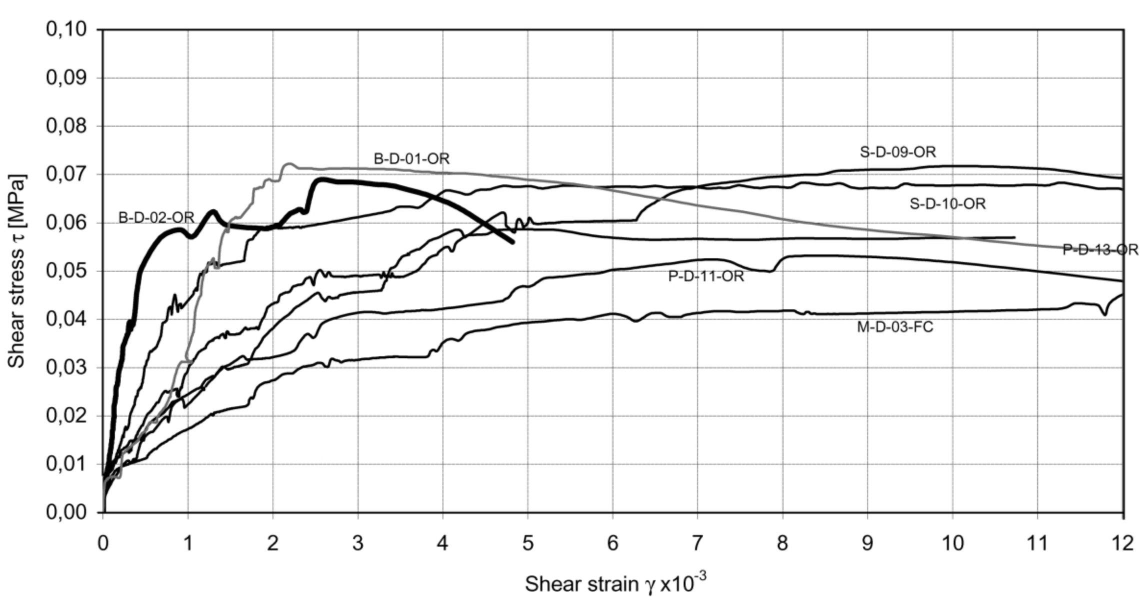

Fig. 11. Shear stress vs. shear strain (diagonal compression tests).

Table 1
Results of the compression tests carried out on stone cylindrical samples

N	Stone type	Weight (N)	Sample dimension (Φ = diameter, h = height) (mm)	σ_{max} (MPa)
1	White colored calcareous stone	13.0	133	37.37
2	White colored calcareous stone	11.0	$\Phi=71$ $h=111$	35.52
3	Pink colored calcareous stone	14.2	$\Phi=71$ $h=150$	90.30
4	Pink colored calcareous stone	14.1	$\Phi=72$ $h=150$	35.60
5	Pink colored calcareous stone	14.3	$\Phi=72$ $h=150$	54.70
6	Pink colored calcareous stone	14.4	$\Phi=72$ $h=150$	49.30
7	Sponge travertine	8.1	$\Phi=71.7$ $h=150$	3.00
8	Sponge travertine	10.5	$\Phi=72$ $h=150$	8.10
9	Sponge travertine	7.9	$\Phi=70.7$ $h=150$	2.00
10	Sponge travertine	8.0	$\Phi=71.1$ $h=150$	3.30
11	Sponge travertine	7.5	$\Phi=71.7$ $h=150$	1.75
12	Sponge travertine	9.0	$\Phi=71.9$ $h=150$	3.70
13	Sponge travertine	6.0	$\Phi=71.3$ $h=150$	1.90
14	Sponge travertine	7.5	$\Phi=70.8$ $h=150$	2.20

Table 2
Results of the shear–compression tests

Index code	Masonry texture	Section (cm)	τ_{max} (MPa)	G (MPa) Scheme 1	G (MPa) Scheme 2	$\tau^{sc}_{k,nom}$ (MPa) $b=1.50$
B-T-04-OR	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	48	0.219	546	328	0.130
V-T-07-IN	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	48	0.225	450	308	0.149
P-T-15-OR	Double-leaf roughly cut stone masonry	48	0.172	216	–	0.136

Table 3
Results of the diagonal compression tests

Index code	Section (cm)	Masonry texture	P_{max} (kN)	$\tau^{diag}_{k,nom}$ (MPa)	$G_{1/3}$ (MPa)	$\gamma_{1/3}\times10^{-3}$
B-D-01-OR	48	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	58.81	0.072	30	0.791
B-D-02-OR	30	One-leaf solid bricks masonry	34.31	0.069	131	0.136
M-D-08-OR	58	Double-leaf roughly cut stone masonry	45.80	0.047	19	0.824
S-D-09-OR	40	Double-leaf roughly cut stone masonry	48.96	0.072	25	0.942
S-D-10-OR	70	Double-leaf roughly cut stone masonry	80.93	0.068	60	0.370
G-D-11-OR	57	Double-leaf roughly cut stone masonry	51.14	0.053	26	0.642
P-D-13-OR	48	Double-leaf roughly cut stone masonry	47.66	0.059	37	0.533

second building was built over the defending walls of the village. As a consequence the thickness of the panel is approximately 70 cm (identification number of the panel = 10) (see Fig. 1d). The masonry texture is again made of double-leaf roughly cut stone walls with the same components of panel 9. A diagonal compression test was carried out on the panel cut from this building. The nominal shear strengths $\tau^{diag}_{k,nom}$ for panels 9 and 10 are, respectively, 0.072 and 0.068 MPa (see Table 3 and Fig. 11).

4.5. The Ponte Postignano building

As the two Sellano buildings, the one located in Ponte Postignano is made of double-leaf roughly cut stone masonry walls (see Fig. 1f). This building, built in the 1950s following traditional construction techniques, has a cross-section thickness of 48 cm. Compared to the other examined structures, there is also a notable percentage of sponge travertine (20–30% of the panel surface). Two unstrengthened panels were tested at this

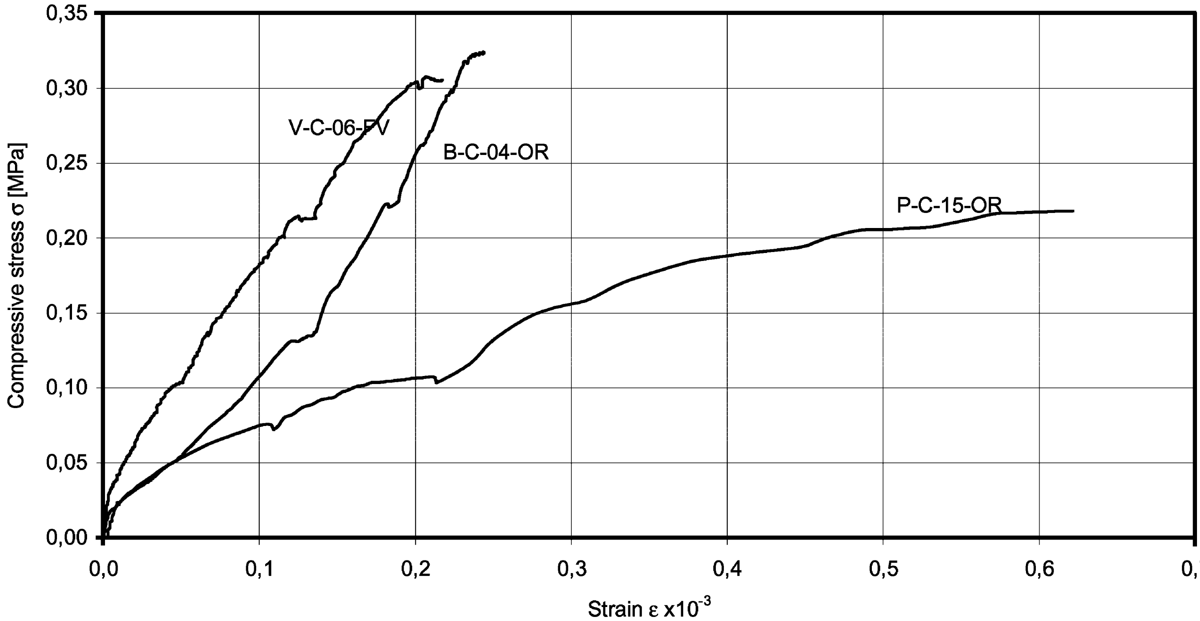

Fig. 12. Compressive vertical stress vs. vertical strain (this graph derives from the envelope of the loading and unloading cycles).

structure (one for a diagonal compression test and the other for compression and shear–compression tests).

The results obtained show values of nominal shear strength $\tau^{sc}_{k,nom}$ of 0.136 MPa for panel no. 15, submitted to a shear–compression test, and of nominal shear strength $\tau^{diag}_{k,nom}$ of 0.059 MPa for panel no. 13 (diagonal compression test).

4.6. Evaluation of the stiffness

Concerning the shear elastic modulus G, the results obtained are very variable, depending on the type of test. High values were measured in the case of shear–compression tests while smaller values of $G_{1/3}$, measured at 1/3 maximum load (varying for stone double-leaf panels from 19 to 60 MPa) were obtained for diagonal compression tests. The $G_{1/3}$ modulus of the brick texture panel was instead higher (131 MPa) compared to values obtained from the analogous type of test carried out on stone double-leaf panels. The different static schemes adopted for the evaluation of the shear elastic modulus G for shear–compression tests did not determine high G variations. The average value is in fact equal to approximately 370 MPa, independent of the scheme adopted.

The results of shear strain $\gamma_{1/3}$ measured at 1/3 maximum load is more scattered compared to the results obtained for the strength. The values are between 0.370×10^{-3} and 0.942×10^{-3} for diagonal compression tests and the average value is 0.680×10^{-3}. The result of shear strain $\gamma_{1/3}$ of the diagonal test carried out on the brick panel is 0.136×10^{-3}, significantly smaller with respect to the values obtained from tests on the roughly cut stone double-leaf wall panels.

The average value of Young's modulus of elasticity measured during the compression tests, carried out on the same panels submitted to shear compression tests, was 1124 MPa. The results showed a significant differentiation between the double-leaf stone panels with or without the double solid brick courses positioned at intervals of 80–120 cm (see Fig. 12 and Table 4). The panel of Ponte Postignano without solid brick courses was approximately 60% less stiff compared to the

Table 4
Results of the compression tests

Index code	Texture	Section (cm)	E (MPa) 1st cycle	E (MPa) 2nd cycle	E (MPa) 3rd cycle
B-C-04-OR	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	48	917	1105	1333
V-C-07-IN	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	48	1814	1644	1652
P-C-15-OR	Double-leaf roughly cut stone masonry	48	471	766	415

Table 5
Comparison of the results from the diagonal compression and the shear–compression tests

Index code	Test type	Texture	$r=\tau^{sc}_{k,nom}/\tau^{diag}_{k,nom}$
B-D-01-OR	Diagonal compression	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	1.81
B-T-04-OR	Shear–compression		
B-D-01-OR	Diagonal compression	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	2.07
V-T-07-IN	Shear–compression		
P-D-13-OR	Diagonal compression	Double-leaf roughly cut stone masonry	2.32
P-T-15-OR	Shear–compression		

average value measured for the panels of Vescia and Belfiore. The Young's modulus of elasticity was calculated for this panel considering only the first cycle of loading and unloading. In fact, it was not possible for this panel to reach the vertical compressive stress of 0.3 MPa because the masonry compressive strength of the panel was smaller.

4.7. Comparisons

It appears evident that the results are strictly connected to the type of test used for testing the panels. As a matter of fact the diagonal compression tests and the shear compression tests led to different values of strength, Young and shear elastic modulus and strains. It is therefore possible, using both tests in the very same building, to evaluate the ratio $r=\tau^{sc}_{k,nom}/\tau^{diag}_{k,nom}$ between the results of nominal shear strength. In our case, this comparison may be done on the first floor at the Belfiore building, at the Vescia building and at the Ponte Postignano building. Working with three couples of results related to the three above-mentioned buildings, the average ratio obtained is equal to 2.06 (see Table 5). A similar value of the ratio $(r=\tau^{sc}_{k,nom}/\tau^{diag}_{k,nom}=2.16)$ has been found at the building of Ponte Postignano testing two different panels repaired by the injection technique.

Surely, due to the very few number of tests carried out, the above quoted correlation must be investigated by a bigger number of tests. However, the emerging line seems quite correct and hence, they allow the following considerations.

Once it is assumed that a ratio exists between the results of the two tests, we face the problem of choosing the one more representative of the real behavior of masonry walls stressed by horizontal loads typical of seismic actions. Diagonal compression tests allow the panel a free deformation, since its four sides are free from any kind of constraints, with the exception of the small portion of masonry that permits the connection between the panel and the rest of the masonry wall. Numerical calculations demonstrated its lack of influence, and the panel can be considered completely unconstrained. On the contrary, during the shear–compression test, the two square halves resulting from the division of the panels in two parts have therefore a common edge. This causes an effect of confinement from one half to the other. These are also constrained by the presence, on the upper part of the panel, of the apparatus overhanging the panel (steel plates, jacks, rods) and, by the bottom part, of the remaining masonry constituting the wall.

The most common seismic verifications for buildings, consisting of 2–3 floors with walls characterized by a low slenderness ratio, assume the vertical masonry elements between adjacent openings as infinitely stiff. In analogy with the shear–compression tests, the failure of these elements occurs when the shear strength is not able to absorb the seismic loads. The strains along the vertical edges of masonry elements are free, while an effect of confinement is produced by the remaining overhanging part of the masonry wall.

A behavior such as this is easy to verify from the analysis of damages to constructions struck by the earthquake, in which the failure condition occurs when the tensile strength in the center of the masonry panel is achieved.

A comparison between the Italian Standards [10] and the results obtained from shear–compression tests is not easy to realize. The Italian Standards [10] define the following values of shear strength τ_k for stone masonry: 0.02, 0.04 and 0.07 MPa, respectively, for 'stone masonry in bad conditions', 'triple-leaf stone masonry' and 'roughly cut stone masonry'. The particular texture tested, one of the most diffused in Italy, made of double-leaf roughly cut stone masonry walls is not mentioned by the standards. The average values obtained ($\tau_k=$ 0.084 MPa for double-leaf roughly cut stone wall with double courses of solid brick interposes at intervals of 80–120 cm; $\tau_k=0.082$ MPa for double-leaf roughly cut stone wall) (see Table 6) are comparable to the highest values reported by the Italian Standards. However, without some experimental data, the numerical verifications for this masonry texture is usually made considering the

Table 6
Shear strength values: values obtained from the diagonal compression test were multiplied by the coefficient $r = \tau_k^{5c}/\tau_k^{diag} = 2$

	Masonry texture	τ_k(MPa)	Test type
One panel	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	0.086	Diagonal compression
One panel	One-leaf solid bricks masonry	0.082	Diagonal compression
Five panels	Double-leaf roughly cut stone masonry	0.072	Diagonal compression
One panel	Double-leaf roughly cut stone masonry	0.082	Shear–compression
Two panels	Double-leaf roughly cut stone masonry with two solid brick courses at intervals of 80–120 cm	0.084	Shear–compression

0.02 or 0.04 values of the standards. It is evident that these values are approximately 1/3 of the results obtained from the present experimental work.

5. Conclusions

The experimental work allowed an evaluation of the values of shear strength and of the shear and Young modulus of elasticity for some typical masonry walls of the Umbrian areas struck by the earthquake. In particular, the masonry made of double-leaf roughly cut stone walls was analyzed. The high number of tests carried out on panels made of this texture, produced a large number of data from which the mechanical characteristics of this texture were deduced.

The Italian Standards were taken into consideration, but it was not possible to make a comparison between the values of the standards and those obtained from the tests due to the fact that the Italian Standard does not consider the particular textures tested. However, the highest values reported by the standards for stone walls ($\tau_k = 0.07$ MPa) are smaller compared to the experimental results obtained, highlighting the fact that Italian Standards undervalue the shear strength of stone masonry.

Considering the notable variations in results typical of tests on masonry, the scattering of the shear strength results obtained with this experimental work is significantly not very high. On the contrary, the values obtained for Young and shear elastic modulus are very different from one test to another. However, the main differences depend on the types of masonry textures and the types of tests carried out. With regard to similar masonry texture and test type, the variations in results are much smaller.

In some cases, the diagonal compression test and the shear–compression test were carried out on the same buildings made of the same masonry textures. This allowed to identify a significant differentiation between the results obtained from the two test types. It was noted that the ratio between the results of shear strength $r = \tau_{k,nom}^{sc}/\tau_{k,nom}^{diag}$ for the two tests is almost constant, highlighting the problem of choosing the test which best simulates to the real behavior of the masonry when stressed by a seismic actions.

In particular, from a comparison of the shear strength results of the two different test types, not only for the unstrengthened panels but also for some panels strengthened with differing techniques, the r coefficient can be assumed equal to 2. However, it will be necessary to realize other tests, now in the planning phase, to fix this coefficient.

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