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Behavior and modeling of strengthened three-leaf stone masonry walls

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ABSTRACT

The application of three different intervention techniques on three-leaf rubble stone masonry walls are discussed here. Injections, repointing, and the placing of ties connecting the two external whytes were considered, both singularly and in combination. Lime-based products were chosen for injection grouts and repair mortars, to ensure better compatibility with the original materials. The experimental tests, performed on seventeen large scale samples under compressive loads, showed that: (i) injections are very effective to improve the mechanical characteristics of the walls; (ii) the other techniques have less influence on the strength but can operate in avoiding 'brittle' failure modes (ties placing) and in improving the durability of the masonry (repointing); (iii) the combination of the techniques ensures the enhancement of the global behavior of the walls. The integration of the experimental results with data available in literature allowed the calibration of an analytical model able to predict the compressive strength of injected walls, based on parameters given by simple experimental tests.

RÉSUMÉ

Dans cet article, nous allons observer l'application de trois différentes techniques d'intervention sur des murs de trois couches en maçonnerie de pierres. Les injections, le remplissage des joints horizontaux de mortier et le positionnement des épingles qui relient les deux couches extérieures ont été pris en considération à la fois individuellement et ensemble. Des produits à base de chaux ont été choisis pour les injections de coulis et les mortiers de réparation, afin d'assurer une meilleure compatibilité avec les matériaux d'origine. Les tests expérimentaux effectués sur dix-sept échantillons à grande échelle sous l'action de poids comprimant, ont montré que : (i) les injections améliorent considérablement les caractéristiques mécaniques des murs ; (ii) les autres techniques influencent moins la résistance mais elles peuvent opérer en évitant les modes d'échec 'fragile' (avec le positionnement des chaînes) et en améliorant la durabilité de la maçonnerie (remplissage des joints) ; (iii) la combinaison des techniques assure l'amélioration globale du comportement des murs. L'intégration des résultats expérimentaux avec les données disponibles en littérature ont permis de calibrer un modèle analytique capable de prévoir la force de compression des murs injectés, fondée sur les paramètres résultant de simples tests expérimentaux.

1. INTRODUCTION

The use of stone masonry is very common in many historic constructions, both architectural monuments and whole urban centres (especially in Europe), many of which are characterized by medium-high seismic hazard. This masonry is generally made of various and very poor materials (different type of stones, low strength lime mortars), arranged into irregular morphologies, often represented by multi-leaf wall structures, having little or no connection through the thickness. In particular, three-leaf masonry walls are characterized by a possible substantial presence of voids, often concentrated in a loose internal core [1]. They are particularly sensitive to brittle collapse mechanisms, which happen by the detachment of the layers and the out-of-plane expulsions, both under vertical and horizontal loads [2-4].

Editorial note
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Nevertheless, the knowledge of the mechanical behavior of multi-leaf masonry walls is still limited, as well as the availability of standards and codes of practice for the proper design and control of the interventions. Recent seismic events in Italy (1997 Umbria-Marche earthquake) revealed, in fact, that the rehabilitation of masonry buildings is often performed without any feasibility study [5], with serious consequences on their structural safety.

In the conservation perspective, an increasing sensitivity is nowadays directed to the choice of consolidation materials mechanically, physically and chemically compatible with the original ones. In such ambit, lime-based mortars and grouts are more and more considered (rather than cement or resins) to pursue both effectiveness and durability of the intervention.

As support for the design phase, some analytical approaches, both for original and strengthened (by injections) masonry are available in literature [2, 4]. They are based on simplified formulations, which depend on parameters easily detected by in situ survey and laboratory tests. Until now, for injected walls, their calibration is available only for cement based grouts and needs more experimental evidence for generalized applications.

A comprehensive experimental study on the behavior of three-leaf stone masonry walls has been recently performed at the University of Padua. Seventeen large scale walls have been subjected to compression laboratory tests in different strengthening conditions: (a) injection with two types of hydraulic-lime based strengthening admixtures, (b) repointing of the mortar joints, and (c) the transverse confinement with steel ties. Such techniques were applied both singularly and in their possible combinations, in accordance with the different masonry deficiencies detectable on site.

The aims of the research are: (i) to characterize the behavior of the masonry under different conditions of strengthening and repair, (ii) to define the practical procedure of the interventions, with particular attention on the control of the execution phase, and (iii) to calibrate the available simplified analytical methods for the prediction of the compressive strength of injected walls.

In the paper, the main experimental results and the more significant aspects of the application of the techniques are described. However, as useful contribution for the design, the analysis and calibration of the model is more widely considered.

2. TEST SPECIMENS

To reproduce a masonry sufficiently representative of the existing typologies, the design of the specimens was based on the data collected by the Polytechnic of Milan on more than 250 walls [1], and on the direct analysis of more than 70 existing walls described in literature [6]. The wall samples were 0.80 m long, 0.50 m thick and 1.40 m high. The two external layers, approximately 18 cm thick each, consisted of rough-shaped limestone blocks (their highest dimension is about 25 cm), bonded in sub-horizontal courses, with mortar joints from 10 to 40 mm thick. The thickness ratio between external whytes and internal core of 1:0.78 (a typical ratio collected in the north-eastern part of Italy) is the average between the values detected in literature among existing walls (1:0.55) and laboratory specimens (1:1). The internal core, having a thickness of about 140 mm, was built with mortar and rough-cut limestone pieces (remains from rough-shaping of the stones), poured into uncompacted layers between the two external layers. This allowed the creation of an adequate amount of voids to make the wall potentially injectable, mainly concentrated in the inner core [1]. In particular, the walls were characterized by a proportion of 68% of stones, 22-17% of mortar and 10-15% of voids (on the volume of the whole wall). To reproduce the worst wall conditions, no transverse connection (*i.e.* no interlocking of stone blocks) was provided through the layers. The specimens were obtained by the cutting with a wire saw of a 13.60 m-long wall (see Fig. 1). This allowed the examination of the plane transverse sections as to proportion of stones, mortar and voids, which confirmed the design assumptions.

Fig. 1 - View of some specimens after cutting.

The limestone used for the test specimen was from a quarry in the north-eastern part of Italy; its compressive strength, measured on cubes (71 mm for each side), was approximately of 160 MPa. The mortar was composed by a binder of natural hydraulic lime and lime putty (ratio 1:3); lime/sand ratio was equal to 1:3, and water/lime ratio was 0.5 (all ratios are in volume). Its compressive strength measured after 28 and 60 days on 40x40x160 mm prisms was 1.57 and 1.64 MPa, respectively.

3. EXPERIMENTAL PROGRAM

The test program is summarized in Table 1. The specimens were labelled with progressive numbers and alphabetic indexes referred to the strengthening techniques: injection of two different admixtures (I1, I2), repointing (R), transverse tying (T), and their combinations. Walls 1 through 9 were tested under compression loads after 28 days of curing, then repaired (except for the wall 4, which was seriously damaged during the first phase of loading). They were therefore re-tested about 60 days after

Table 1 - Walls subjected to compression test

Strengthening technique	Before/after repair	After strengthening
Injection 1	5I1, 6I1	13I1
Injection 2	1I2, 8I2	16I2
Repointing	3R, 7R	15R
Steel Ties	2T, 9T	11T
I1 + R	-	14I1R
I1 + T	-	12I1T
R + T	-	10RT
I1 + R + T	-	17I1RT

construction, under a first loading cycle correspondent to the maximum strength achieved before repair and then monotonically until collapse. Walls 10 to 17 were tested only after strengthening, under the same load history of the repaired ones (the limit at the first cycle was the average of the strengths detected before intervention on the walls repaired with the same techniques). Among them, walls 10, 12, 14 and 17 were interested by combined techniques. For the strengthened walls, techniques and compression tests were executed simultaneously to the repaired ones, *i.e.* 28 and 60 days after construction, respectively.

All the tests were performed under force control, using a 10 MN Amsler machine, at a load increment rate approximately equal to 0.25 kN/s. Six displacement transducers were adopted: two W20 and two W10 (20 and 10 mm of maximum deformation respectively) were applied on the main façades of the wall to record the vertical and the horizontal displacements, respectively, whereas an horizontal transducer W10 was placed on each transverse section (see Fig. 2). Applied loads and corresponding displacements were recorded with a frequency of 5 Hz.

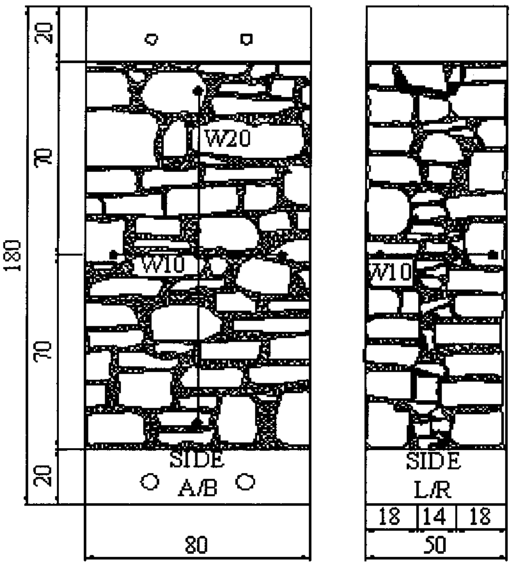

Fig. 2 - Geometry of the walls and transducers position.

4. STRENGTHENING TECHNIQUES

In the following, a short description of the three different repair techniques used in the study is given. They are specifically aimed to solve the main structural deficiencies of three-leaf existing masonry walls, which are: (a) the weakness of the internal core, (b) the deterioration of the mortars and (c) the lacking of the connection among the layers. In particular, the simultaneous application of all the three techniques can lead to a possible "integrated intervention", by means of the consolidation of the inner layer (by grout injection), extended to the external surfaces (by repointing), and by the improvement of the bond between the layers (by tying and the injection of grout).

4.1 Injections

Injections were executed on nine walls. The target was to fill the voids in the inner core and to improve its adherence to the external layers. Two different grouts (I1 and I2), both based on a natural hydraulic lime binder, were selected through physical, chemical, rheological, mechanical and injectability tests on cylinders [7, 8] (30 cm high and 15 cm on diameter), filled by the loose material taken from the inner leaf of the walls [9]. I1 is a basic admixture with a superplasticizer additive (0.25% on the binder weight), whereas I2 is a product ready for use. They have similar physical and rheological characteristics but different mechanical strength (see Table 2, where f_{gr} and f_{cyl} are the compressive strength of the grouts and of the injected cylinders respectively, measured after 28 days of curing). Therefore, as the grouts have the same injectability properties, possible variations in the mechanical behavior of the walls could be attributed to the different strength of the consolidation materials.

Table 2 - Characteristics of grouts and injected cylinders

Grouts	Water/lime ratio	Density (kg/m^3)	Fluidity(*) (s)	Compressive strength (MPa)	
				f_{gr}	f_{cyl}
I1	0.5	1.8	13	5.10	2.07
I2	0.5	2.0	13	3.20	0.81

(*) Marsh cone [9].

Injection holes were drilled on the mortar joints only at one side of the walls, following approximately a scheme of equilateral triangles. They were spaced of about 25-30 cm one to the other, where plastic tubes (having 9/12 mm as internal/external diameter) were introduced. As a consequence, a distribution of 11-12 holes per m^2 was obtained. This diffusion can assure with high probability the complete injection of the voids; during the intervention, only about the 70% of that holes were really injected. The other ones acted very usefully as checking holes, to control the diffusion path of the grout. To the same purpose, some checking holes were positioned also on the other three sides of the panels. Before injection, the lateral sections were roughly sealed with plaster to avoid excessive leakages of grout. To reproduce the real absorption of an ancient and dehydrated mortar, no preventive injection of water was done. The grout was injected at low pressure (around 0.5 atm) starting from the bottom of the walls. The control of the injection phase suggested that the entire wall can be

Table 3 - Quantity of injected grouts and voids amount estimation

Wall	V_{gr} (l)	V_{gr}/V_{inf} (l/m^3)	V_{gr}/V (l/m^3)	Derived voids (%)
1 I2	85	542	152	15.2
5 I1	85	542	152	15.2
6 I1	85	542	152	15.2
8 I2	75	478	134	13.4
12 I1T	85	542	152	15.2
13 I1	65	415	116	11.6
14 I1R	95	606	170	17.0
16 I2	75	478	134	13.4
17 I1RT	75	478	134	13.4
Average	81	517	144	14.4

considered as fully injected. Thus, the survey of the quantity of the used grout can provide a gross estimation of the injected percentage of voids. In Table 3 the injected grout amounts (V_{gr}), normalized on the inner core and on the whole wall volumes (namely V_{inf} and V respectively) are given. Again, by assuming the complete injection of the wall, the results confirmed the initial design hypothesis (percentage of voids) of the physical model. According to their similar physical and rheological characteristics, both the grouts showed a good penetration into the voids of the walls. Inspections after the walls failure also revealed a good adherence with the existing mortar.

4.2 Bed joints repointing

The repointing technique was aimed to restore the external mortar joints and to improve the bond among the stones. It is performed by the removal of the external layers of the joints and then the refilling by new mortar [10]. In such study, the technique was applied on six walls, on both sides of them. A natural hydraulic lime mortar, premixed with sand, was used as repointing material (compressive strength after 28 days was 1.72 MPa). Due to the irregularity of the joints, the existing mortar was removed to a depth varying from 2 up to 8-9 cm. Particular attention, during the execution, was paid to the depth of the groove of the existing mortar and to the accuracy of filling the joints and pressing the new mortar.

4.3 Transverse tying

The application of transverse tying through the thickness of the walls was aimed to improve the connection between the layers and to reduce the transverse deformation. Six walls were strengthened with a number of 4 steel ties per m^2. Two different types of bars were used: reinforcing steel and threaded rods, both having a tensile strength of about 600 MPa. The steel bars were simply introduced into through holes, previously drilled, and bent from the outside into a mortar joints previously excavated and then refilled with new mortar. The threaded bars were introduced into the holes and fixed with washer and nut, hidden into the joints as well. During the execution phase, particular attention was paid to the bending of the bars, due to the anchorage length, and to the covering of nuts with mortar, with aesthetical aims.

5. TEST RESULTS FOR ORIGINAL WALLS

Before strengthening, the maximum compressive strength of the walls varied between 0.99 and 1.97 MPa, but evident cracks pattern already started at a stress level varying from 0.55 to 1.09 MPa. The mean value of the secant modulus of elasticity, calculated between the 30% and the 60% of the maximum strength, is equal to 1857 MPa. As confirmation of the correct design and execution of the physical model, the obtained values are very close to experimental values collected on real historical walls [6]. Similar values were detected for the final vertical and horizontal deformations (variations from 0.2 to 6.5 ‰, absolute values), whereas final transverse strains were much higher, varying from 1.3 to 19.6‰. The cracks had a vertical or sub-vertical pattern, mostly located in the transverse sections, at the interfaces of the different layers. Therefore, as expected, the compressive failure was due to the high dilatancy of the wall, which caused the out-of-plane detachment of the layers.

6. TEST RESULTS FOR STRENGTHENED WALLS

6.1 Injected walls

After grouting and testing again, the mean value of the maximum strength was about 2.5 MPa, showing an increment of about 40%, compared to the mean value of original walls (the anomalous result of wall 8I2 was due to some problems occurred during the injection phase). Table 4 resumes the obtained results: $f_{wc,0}$ and $E_{wc,0}$, and $f_{wc,s}$ and $E_{wc,s}$ are the compressive strength and the modulus of elasticity for original and strengthened walls, respectively. Despite the different compressive strength of the grouts I1 and I2 (their strength ratio is about 1.6, see Table 2), the final strength of the injected walls was approximately the same. The modulus of elasticity was calculated between the 30% and 60% of the strength reached during the first loading cycle. After injection the Young modulus varies between 1223 MPa and 3992 MPa, showing an average increase of about the 30%. The final values of stiffness are anyway still compatible with the elastic characteristics of existing stone masonry walls [6]. A typical stress-strain diagram before and after injection is shown in Fig. 3.

Table 4 - Mechanical tests results of injected walls before and after injection

Wall	Compressive strength (MPa)		Modulus of elasticity (30÷60% f_{wc}) (MPa)	
	Before $f_{wc,0}$	After $f_{wc,s}$	Before $E_{wc,0}$	After $E_{wc,s}$
5I1	1.45	2.49	2390	2273
6I1	1.95	2.49	2029	3093
13I1	--	2.54	--	3992
1I2	1.97	2.57	1450	3449
8I2	1.91	1.82	1559	2367
16I2	--	2.48	--	1223
12I1T	--	2.59	--	1336
14I1R	--	2.14	--	1617
17I1RT	--	3.06	--	1772
Average	1.82	2.46	1857	2347

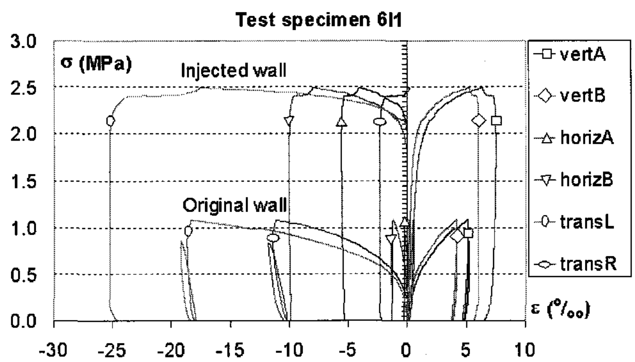

Fig. 3 - Stress-strain diagrams before (at first loading cycle strength) and after (up to final strength) injection.

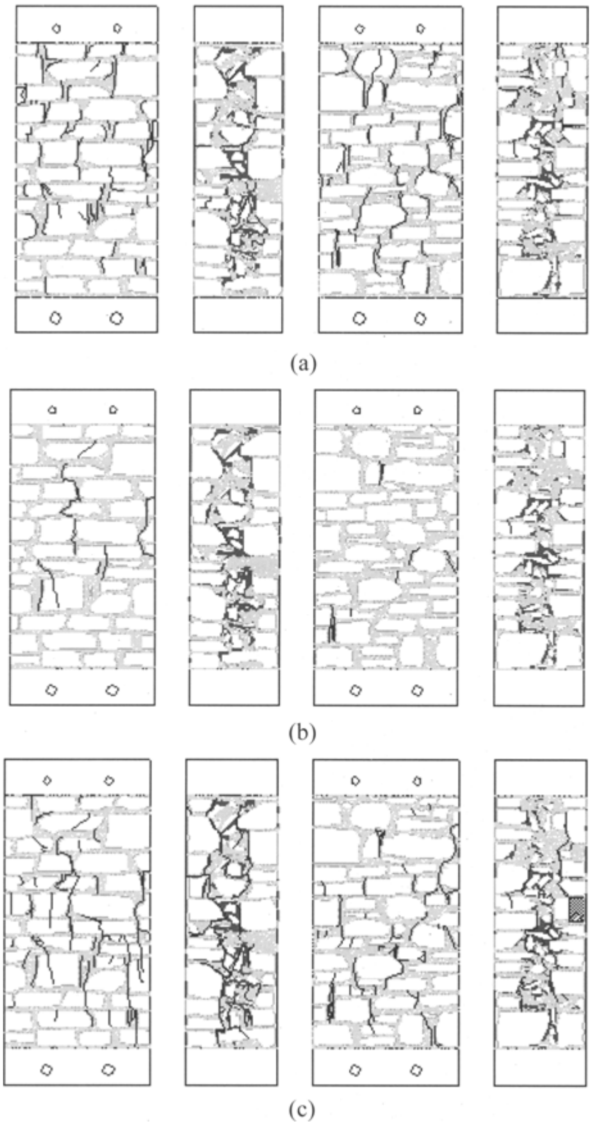

Fig. 4 - Typical evolution of the cracks pattern for an injected wall: at maximum strength in original conditions (a), after repair at the same stress level (b), and at the ultimate load (c).

The visual inspection before and after intervention at the end of the first loading cycle (correspondent to the maximum strength of walls before repair) revealed a strong reduction of the cracks pattern (see Figs. 4a and 4b). After intervention, the maximum vertical and horizontal strains are higher then before, varying from 4.2 to 10.7‰ (absolute values); on the contrary, transverse strains are in general lower after grouting (see Table 5). This reveals the efficiency of grout injections in homogenizing the wall and preventing the whytes detachment. In such connection, significant dilatancy of the wall occurs at a stress level even ten times higher than on the original condition. In Table 5 the vertical and transverse strains, before and after grouting, are given (positive and negative values indicating shortening and dilatancy, respectively).

Table 5 - Deformations for injected walls at various stress levels

Wall	Vertical strain (‰)			Transversal strain (‰)		
	Before at $f_{wc,0}$	After at $f_{wc,0}$	After at $f_{wc,s}$	Before at $f_{wc,0}$	After at $f_{wc,0}$	After at $f_{wc,s}$
5I1	3.63	0.49	7.26	-4.49	-0.004	-9.17
6I1	4.57	0.36	5.71	-18.35	-0.003	-17.13
13I1	--	0.55	9.91	--	-0.12	-27.67
1I2	6.21	0.58	6.25	-7.93	-0.32	-7.34
8I2	6.22	0.73	7.20	-11.77	-0.08	-9.90
16I2	--	1.07	10.72	--	-0.22	-18.92
12I1T	--	0.78	8.18	--	-0.04	-15.88
14I1R	--	0.71	8.21	--	-0.01	-19.56
17I1RT	--	0.63	8.24	--	-0.001	-21.91

6.2 Other techniques

The walls repaired or strengthened by the bed joints repointing and the transverse tying techniques, both applied singularly and in combination, did not show any significant increase of compressive strength and stiffness in comparison with the original conditions, unless when they were combined with injections. On the other hand, the features of the walls (the weakness of the internal core, in particular) led to consider the consolidation of the inner layer as crucial. Nevertheless, a general decrease of the deformations after repair at the same stress level of the maximum initial strength was detected (Table 6). As for the repointed walls, horizontal and transverse strains start at a stress level higher than for the original walls. More clearly, the transverse tying of the walls strongly reduced both vertical and transverse strains at the peak stress. In particular, the transverse strain showed an average reduction of about the 50% at the peak stress, and of about the 90% at the maximum strength before intervention (see Table 6).

Table 6 - Deformations for repointed and tied walls

Wall	Vertical strain (‰)			Transversal strain (‰)		
	Before at $f_{wc,0}$	After at $f_{wc,0}$	After at $f_{wc,s}$	Before at $f_{wc,0}$	After at $f_{wc,0}$	After at $f_{wc,s}$
2T	6.55	0.71	4.05	-19.57	-2.02	-8.29
9T	4.12	0.80	3.04	-10.61	-1.10	-6.34
11T	--	3.04	7.59	--	-2.57	-8.45
3R	3.19	2.78	10.45	-9.84	-0.24	-13.45
7R	4.39	2.23	5.10	-5.81	-2.32	-10.35
15R	--	2.36	7.90	--	-0.50	-14.01

From a practical point of view, the considered techniques can combine one another their own effects, allowing an enhancement of their feasibility in the execution phase. Thus, the excavation of the joint during the repointing is effective in improving the injection procedure, as it makes easier the hole drilling and the placement of the hoses; on the other hand, the filling of the joints can prevent the leakage of the grout. In fact, the highest performances have been obtained for the wall strengthened by the "integrated intervention", that is by all the three techniques (see Table 4).

7. MODELING OF THE BEHAVIOR OF MULTI-LEAF WALLS

7.1 Background

According to different studies available in literature, the compressive behavior of a multi-leaf wall can be investigated through a "multi-material" model [11, 2-4]. Assuming some simplifying hypothesis (elastic behavior of the layers, plane connection among them, transverse strains negligible), the ultimate strength of the system depends on the compressive strength and the volumetric contribution of the layers [3], as given by Equation (1):

$$f_{wc,0} = (V_{ex}/V)\Theta_{ex} f_{ex,k} + (V_{\inf}/V)\Theta_{\inf} f_{\inf,0} \quad (1)$$

where V_{ex}/V and V_{inf}/V are the volumetric ratios of external layers and internal core to the entire wall, $f_{wc,0}$ is the compressive strength of the unstrengthened wall, $f_{ex,k}$ is the characteristic compressive strength of the external layers, $f_{inf,0}$ is the compressive strength of the infill. Finally, Θ_{ex} and Θ_{inf} are corrective factors which take into account the influence of the mutual interaction between the external layers and the internal core in the global behavior of the wall. In the case of a multi-leaf wall with a weak infill (both in terms of strength and stiffness), experimental works confirmed that the external layers apply a restraint action on the internal core and, at the same time, the dilatancy of the infill apply a lateral thrust to the external layers [2]. Therefore, the core is under a compression "more" than uniaxial ($\Theta_{inf}>1$), whereas the outer whytes are in a "less" than uniaxial condition ($\Theta_{ex}<1$). The collapse is due to the failure of the external layers under combined bending and compressive stresses. At that stage, the ratio between the axial stresses carried by the internal and external layers is estimated around 1/5 [3]. Vice versa, when the internal core is stiffer than the outer layers, a brittle collapse of the wall can occur, caused by the compressive failure of the infill with the consequent sudden failure of the outer layers. At collapse, the ratio between the compressive stresses carried by the internal and external layers is estimated around 7/3 [3].

Expression (1) is proposed for the evaluation of the compressive strength of existing masonry, based on in situ slight-destructive tests; in particular, the strength of the internal core can be approximated by the compressive strength of corings (having a proper slenderness) drawn from the masonry [2].

According to Egermann's results [2, 3], Vintzileou and Tassios [4] assumed that, for an injected wall, the compressive strength of the original wall (*i.e.* before the intervention) is mainly due to the external layers, so the internal core contribution is negligible (see Equation (2)). Similarly, the strength increase after injection is mainly due to the infill consolidation, so the contribution of the external layers can be neglected. The general expression of such approach is given by Equation (3), where: $f_{wc,s}$ is the compressive strength of the strengthened wall, and $f_{ex,c}$ and $f_{inf,s}$ are the compressive strength of the external layers and of the grouted infill, respectively:

$$f_{wc,0} = (V_{ex}/V)\, f_{ex,c} \quad (2)$$

$$f_{wc,s} = f_{wc,0} + (V_{\inf}/V)\, f_{\inf,s} \quad (3)$$

To make clear the parameters of such relationship, Formula (3) was calibrated by a series of experimental tests performed on three-leaf stone masonry walls injected by cement grouts. As in [2], the strength of the consolidated internal core was approximated by the compressive strength of cylinders filled as in the real walls, then injected and tested in laboratory. Moreover, a correlation between the so detected infill strength and the compressive strength of the grout was found, as given in Equation (4), where f_{gr} is the compressive strength of the injected grout:

$$f_{\inf,s} = f_{cyl,s} = 2.5 f_{gr}^{0.5} \quad (4)$$

Nevertheless, compared with the experimental results, the introduction of such relationship in the general formula (3) causes an overestimation around the 50% of the contribution of the strengthened infill. To fit the experimental results, the final proposed formulation was empirically corrected, as in Equation (5):

$$f_{wc,s} = f_{wc,0}[1 + 1.25(V_{\inf}/V)(f_{gr}^{0.5}/f_{wc,0})] \quad (5)$$

7.2 Analytical model calibration

The over described approach is very interesting because the main parameters for the prediction of the compressive strength of multi-leaf injected walls can be evaluated by simple in situ and laboratory tests. In fact: (i) double-flat jack tests carried out on the original masonry can rather well estimate the external layers strength, (ii) the survey of the geometrical characteristics of the section of the wall (by visible portions available, corings and/or endoscopies) allows to define the volumetric parameters, (iii) simple laboratory compressive tests on the injection admixture provide the reference strength for the consolidation material. This can be very useful in the design phase of the intervention, and can point the choice of the injection material.

Nevertheless, to verify the general reliability of the proposed model deeper investigations are necessary. The contribution in literature of similar experimental works are very scarce and incomplete (consistent tests results on walls, cylinders and grouts are necessary), and are almost entirely referred to the use of cement grouts, *i.e.* material having compressive strength much higher than the original masonry walls to repair.

On the contrary, as demonstrated in the following, it is possible to establish an upper bound for the strength of the consolidation material, which allow to validate a suitable formulation without any empirical correction, for the prediction of the compressive strength of three-leaf injected masonry walls. To that aim, Equation (4) was properly re-calibrated, on the basis of the results collectable from the literature and of the present experimental work.

The whole data suitable for the calibration of the abovementioned relationship are given in Table 7. They are referred to the following materials: cement grouts [4], lime-

Table 7 - Experimental average values for the model calibration

Grouts		f_{gr} (MPa)	f_{cyl} (MPa)	E_{cyl} (MPa)	$f_{wc,0}$ (MPa)	$E_{wc,0}$ (MPa)	$f_{wc,s}$ (MPa)	$E_{wc,s}$ (MPa)
Hydr.-lime	I1	5.10	2.07	1360	1.70	2210	2.55	2347
	I2	3.23	0.81	294	1.94	1506	2.53	2336
	I3	3.65	1.38	1253	-	-	-	-
	I4	3.21	1.43	1499	-	-	-	-
	I5	3.35	1.55	2017	-	-	-	-
Lime-pozz-cem	cb.0	14.60	--	--	2.10	4400	3.30	4500
	13b.0	5.20	--	--	2.65	5550	3.40	2950
Cement	F1	30.0	13.4	20450	2.07	5625	3.83	8853
	F3	13.0	9.50	19800	1.35	5890	3.53	5886

pozzolana-cement grouts [12, 13] and hydraulic-lime products [9, 6]. In particular, the present experimental research allowed to extend significantly the potential field of application of the model, adding data correspondent to low-strength grouts.

By comparing the available results carried out on injected cylinders, the general trend of the cylinders versus the grout strengths is described by Equation (6):

$$f_{cyl,s} = 0.31 f_{gr}^{1.18} \tag{6}$$

Therefore, the general Equation (3) can be rewritten in the following one:

$$f_{wc,s} = f_{wc,0} + 0.31(V_{\inf}/V) f_{gr}^{1.18} \tag{7}$$

By applying Equations (5) and (7) to all the experimental results available on the walls it can be noticed that both can predict the ultimate compressive load, making an error smaller than 20-25% (see Figs. 5 and 6). But, Equation (5) shows a very good agreement (and often underestimates the walls strength) when applied to walls injected with low-strength grouts (about $f_{gr} \leq$ 4-5 MPa) whereas, in the case of high-strength grouts (about $f_{gr} \geq$ 14-15 MPa), it leads to rougher estimations than Equation (7).

Therefore, it can be said that a simplified model based on the evaluation of simple geometrical and mechanical characteristics, can be very appropriated for ratios $f_{gr}/f_{wc,0}$ not higher than about 4 (see Fig. 7). By using higher strength

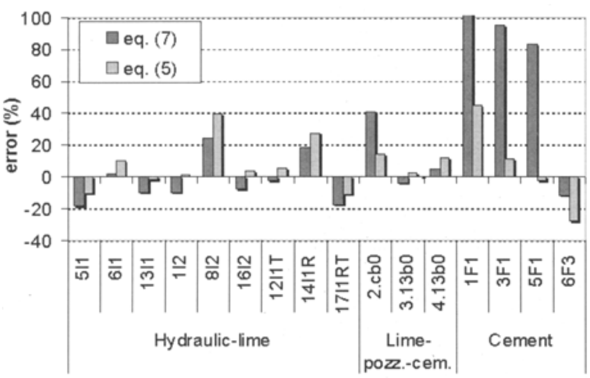

Fig. 5 - Errors for predicted and measured strength of the injected walls.

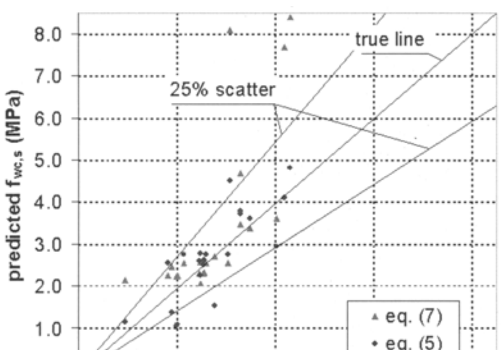

Fig. 6 - Comparison between predicted and measured strength of the injected walls.

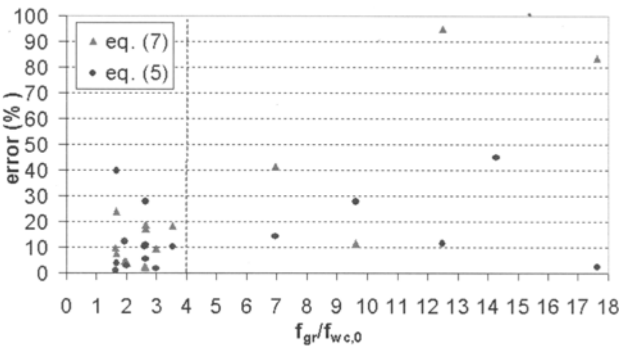

Fig. 7 - Errors related to $f_{gr}/f_{wc,0}$.

grouts, the contribution of the injected core needs a drastic reduction, which is however not rigorously justifiable.

An estimation of that contribution, based on the analysis of the available data, is given in the following. The analysis of the influence of the strength of the grout on the grouted infill and the whole wall strengths showed that the use of high-strength grouts (compared to the strength of the original wall) has a very low influence on the increase of the ultimate load capacity of the wall. On the contrary, the compressive strength of injected cylinders (which can be assumed, following the basis hypothesis of the model, as the strength of the infill) increases more significantly. The trend of the cylinders strength and of the increment of the strength of the walls versus the strength of the grout is given in Fig. 8: both increase when high-strength grouts are used, but this happens very slowly for the walls, compared to the cylinders case. As a consequence, it is not correct to assume that all the strength of the infill is implemented by the wall, especially when $f_{gr}/f_{wc,0}$ ratio exceed a value of about 4. Such value, regardless the strength of the grouts, has here been found as upper bound for the increment in strength of injected walls.

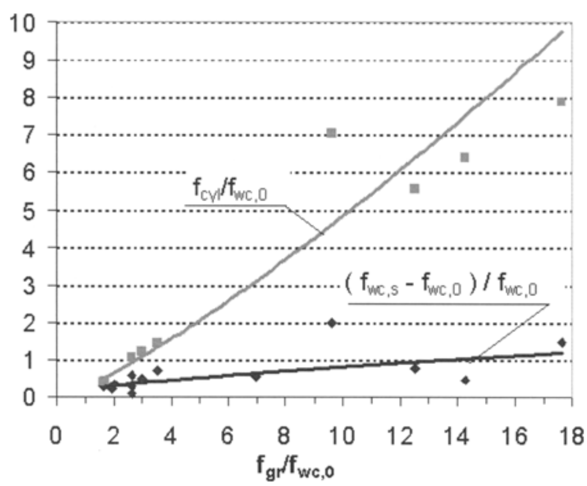

Fig. 8 - Normalized cylinders strength and walls strength increase versus grout strength.

To validate such statement, an "a posteriori" analysis, based on the strength values obtained by the experimentally tested walls ($f_{wc,0(exp)}$ and $f_{wc,s(exp)}$), allowed to estimate the actual quota of the infill strength ($f^*_{inf,s}$) utilized by the wall up to failure:

$$f^*_{\text{inf},s} = (f_{wc,s(\exp)} - f_{wc,0(\exp)}) / (V_{\text{inf}} / V) \qquad (8)$$

The ratio $\eta = f^*_{\text{inf},s} / f_{cyl}$ between the strength effectively implemented by the wall and the whole infill strength (assumed in the model as representative for the infill contribution) can be considered as an estimation of the injection effectiveness. The correlation between the real "effectiveness" of the infill strengthening (average values) and the grout strength is shown in Fig. 9. A clear decreasing trend of that parameter when the strength of the grout increases can be noticed: in particular, for $f_{gr}/f_{wc,0}$ higher than about 4 the utilization of the infill strength is reduced ($\eta<1$). Therefore, the ratio η is not generally definable as 0.5 (as in Equation (5)), but such value is reached only for $f_{gr}/f_{wc,0}$ ratios around 14-15. This ratio corresponds, in fact, as in [4], to cement grout strength (values around 30 MPa, see Table 7).

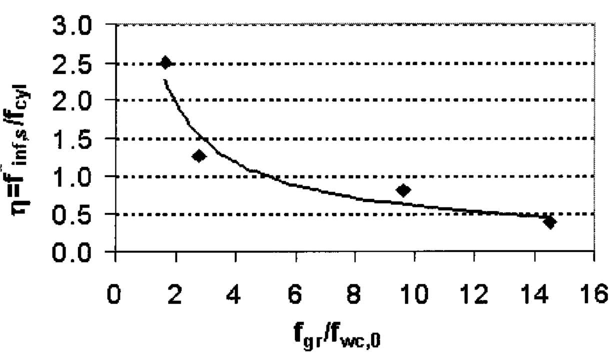

Fig. 9 - Estimation of the efficiency of the infill strengthening for different grout strengths.

As the wall cannot implement a quota of the infill strength exceeding the infill strength itself, the presence of values of $\eta>1$ in the diagram of Fig. 9 can be ascribable to the interaction between the external and the internal layers. As shown in Table 7, in fact, the cylinders injected by admixtures more compatible with the existing materials (hydraulic-lime based) have mechanical characteristics (both compressive strength and modulus of elasticity) lower than the original walls, in comparison with the cement-based ones. In the ambit of the simplifying hypotheses of the model it means that the infill has lower mechanical characteristics than the external layers. On the contrary, when high-strength grouts are used, the infill is stiffer and more resistant than the outer layers (see the direct comparison in Table 8).

According to the experimental evidence, it is possible to say that, when low-strength grouts are used, a more uniform distribution of the vertical stresses on the loaded sections is achieved, with a consequent general improvement of the behavior of the wall. As confirmed in [3], in fact, in the strengthened conditions, being the external layers still slightly stiffer than the infill, and due to the transverse connection improvement caused by the injection itself, a triaxial state of stresses acts in the inner layer, so the ultimate load capacity of the wall is increased. On the contrary, when high-strength grouts are used, the stiffer internal core carries a higher portion of the normal stresses than the external layers, and a uniform distribution of loads is not achieved. In such case, a brittle collapse of the system has to be expected, due to the crushing of the infill and the consequent thrust to the external layers. Such behavior has been confirmed in [4], where cement-based grout injected specimens showed a failure without any significant compressive strain values.

Table 8 - Average values of the mechanical characteristics of grouts and cylinders on the original walls ones

Grouts		$f_{gr}/f_{wc,0}$	$f_{cyl}/f_{wc,0}$	$E_{cyl}/E_{wc,0}$
Hydraulic-lime	I1	2.80	1.22	0.62
	I2	1.65	0.42	0.20
Cement	F1	14.52	6.47	3.64
	F3	9.63	7.04	3.36

8. CONCLUSIONS

An experimental research on the compressive behavior of three-leaf masonry walls strengthened by different techniques has been presented.

The injection of grouts mechanically compatible with the original materials proved to be the most effective intervention in homogenizing the layers, increasing the ultimate load capacity (even more than the 50% of the original strengths), improving the bond among the layers, and the failure mode. It is able to provide for the main deficiencies of such typology of walls without modifying significantly their stiffness, so it is ascribable as a compatible intervention, also for seismic zones. Repointing and transverse tying have revealed their efficiency mostly in terms of reduction of horizontal deformations, and in completing the improvement of the behavior when combined with injections. Nevertheless, to identify correct design and execution procedures, particular attention has to be paid to significant parameters and critical aspects of each single phase of the techniques.

As for injected masonry, the integration of the obtained results with the data available in literature showed that the strength of the grouts does not influence significantly the final compressive strength of the walls. In fact, due to the morphology of the walls itself, when high-strength grouts are used (in comparison with the strength of original walls), the masonry is not able to utilize their whole contribution in strength. Moreover, compared to the cases with use of low-strength grouts, the walls injected by grouts with high characteristics, have shown a more brittle collapse, evidenced by very low values of the compressive strains just before failure.

The analysis showed that a better exploitation of the intervention is limited to ratios between the strengths of the grout and of the walls lower than 4. Within such range of values, it is possible to predict the ultimate strength of injected three-leaf stone masonry walls by a simplified analytical model based on parameters evaluated by simple survey and in situ MDT applications (flat jacks, in particular) and laboratory tests (compressive tests on the grout after 28 days of curing). Beyond that ratio the model is not sufficiently reliable. In such connection, hydraulic lime-based grouts can sum the advantages of a rather full compatibility (mechanical, physical and chemical) with the complete exploitation of its mechanical strength on the walls.

Further developments of this study are focused on the influence of the geometrical characteristics of the layers on the behavior of the whole wall and of the percentage of voids filled by grout on the formulation for the prediction of the strength.

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