In-situ test for the shear strength evaluation of masonry: the case of a building hit by L'Aquila earthquake (Italy)

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SUMMARY:
The assessment of historical masonry mechanical parameters, in particular of the shear strength, is one of the most critical issues related to historical buildings preservation and safety evaluation in seismic area: this is due to the difficulty in performing reliable tests without making an excessive impact on the structure. Many doubts related to L'Aquila masonry quality, arisen during post-earthquake rehabilitation and reconstruction, might be solved by destructive in-situ tests. To this aim, a masonry building severely damaged by the 2009 L'Aquila earthquake was chosen as able to execute an in-situ shear tests. The building has been selected by considering different aspects, like as: safety of worker; accessibility; representativeness of masonry for typical historical buildings in the Abruzzo region. This paper focuses on the procedures to carry out the experimental campaign, the analysis of results and their comparison with reference ranges proposed in the Italian Code for Structural Design.

Keywords: in-situ shear test, masonry, in-plane behaviour, strength parameters

1. INTRODUCTION

A proper characterization of historical masonry mechanical parameters, with particular attention to shear strength, is one of the most critical issues related to historical buildings preservation and safety evaluation in seismic area. In fact, a reliable seismic assessment requires on the one hand, a proper selection of the modelling strategies to be adopted, and, on the other one, an adequate definition of the mechanical parameters they are founded on. To this latter aim, although a direct characterisation of strength parameters by destructive tests is of course desirable, particularly in case of monumental assets, the need to limit the impact on the structure has to be taken into account. Thus, the possibility to address to reference values proposed in the literature becomes essential: these values have to be able to represent the wide variety of masonry types with all their specific constructive and technological features.

As an example, in case of masonry existing buildings, the Italian Code for Structural Design (named in the following as NTC 2008) and its Instruction document (Circolare n.617 2009, Table C8A.2.1) propose different reference values of mechanical properties as a function of different types of masonry. Starting from some values referred to "basic" condition of masonry, they might be corrected through appropriate coefficients (Circolare n.617 2009, Table C8A.2.2) if some "rules of art" are observed (e.g. related to the presence of good quality mortar, good interlocking, etc.); these coefficient aim to increase or reduce the abovementioned basic values of strength and stiffness. However, these corrective factors cannot be exhaustive of all the specific features related to technical local rules of construction: thus, in general, for each specific area it should be desirable some in-depth characterization. This is a very important task in case of L'Aquila post-earthquake rehabilitation and reconstruction, because of many doubts related to local masonry quality as highlighted after the 2009 earthquake.

To this aim, a building in the historical centre of L'Aquila, which has to be demolished except for the ground floor, was chosen as able to execute the in-situ destructive test (Fig. 1.1). This choice has been determined by considering different aspects like as: safety of workers; accessibility; representativeness of masonry for typical historical buildings in the Abruzzo region. Different panels have been selected to perform tests addressed to analyse both the in-plane and out-of-plane response of masonry. This paper focuses only on the in-plane mechanical characterization of masonry; the results of the out-of-plane tests are described in Borri et al. (2012).

In particular, a panel has been identified to perform an in-situ shear test. It is characterized by a rubble masonry, built with calcareous stones (medium size and rounded) and air lime mortar (mixed with a significant earthy component). The mortar joints are quite thick. The cross section is composed by two leaves plus some smaller stones and mortar infill, without a inner core. The two external leaves are not connected through systematic stone headers (named *diatoni* in Italian), which act are transversal shear keys; however, the random presence of some stones a little bit bigger than others seems to provide a not so negligible cross section connection. These characteristic are quite common in masonry types of Abruzzo Region; Fig. 1.2 shows two cross-sections of the examined building.

Figure 1.1 View of the selected building and identification of masonry panel subjected to in-plane shear test.

Figure 1.2 Masonry cross-sections in the examined building.

The test addressed to evaluate the shear strength is a "force-control" test, with the possibility to apply shear action with alternate direction. The masonry panel, whose dimensions are 3.3 metres wide, 2.4 metres high and 0.6 metres thick, is located at the first floor of the abovementioned building and was not damaged, apart from few airline cracks, after the seismic event of 6th April 2009. Due to the specific conditions of the building, interested in some parts by several earthquake damages and collapses, a specific test setup have been designed. First of all, the panel has been taken down from the not collapsed floors, in order to make it free on the top; thus, the static scheme can be approximated as a cantilever. An axial vertical load has been applied in order to induce a preliminary compressive state whose resultant has been keeping constant during the test; it aims to simulate the action of operating gravitational loads. The masonry panels next to the tested one have been reinforced and used to give an adequate contrast to the jacks used to transfer horizontal actions.

Finally, additional tests (double flat jack test and direct and indirect sonic tests) have been performed in order to integrate the results obtained by the in-plane shear test and to ensure a more accurate mechanical characterization of the examined masonry.

This paper focuses on the procedures to carry out the experimental campaign, the analysis of results and their comparison with reference ranges proposed in the Italian Code for Structural Design (2008).

2. IN-PLANE SHEAR TEST

2.1. Description of the experimental setup

Differently from the setup usually adopted in similar laboratory campaigns (e.g. as illustrated in Galasco et al. 2010), due to the actual conditions of the building, a specific setup has been designed. As abovementioned, it concerns a “force-control” shear test aimed to apply alternate loading and unloading cycles. In particular, the final setup is summarized in Fig. 2.1 and it is made up of:

- an horizontal actuator of 500 kN, in order to apply the alternated loading conditions to the panel which is free to roll on the top. A specific system made of HE steel beams has ensured the actuator’s connection to the panel while the loads have been applied to the sample through a steel beam (see Fig. 2.2);
- a contrast system realized by using the panels next to the tested one which have been previously reinforced;
- a system made by two couples of steel pre-stressed bars to a load of 18 kN, that induces a compressive stress in the panel and are kept uniform during the test, in order to apply the axial load designed to simulate the effects of the operating gravitational loads and because it is compatible with the predominant shear mechanism for diagonal cracking. This system has been also realized in one of the masonry panels of the contrast system where the double flat jack test has been performed.

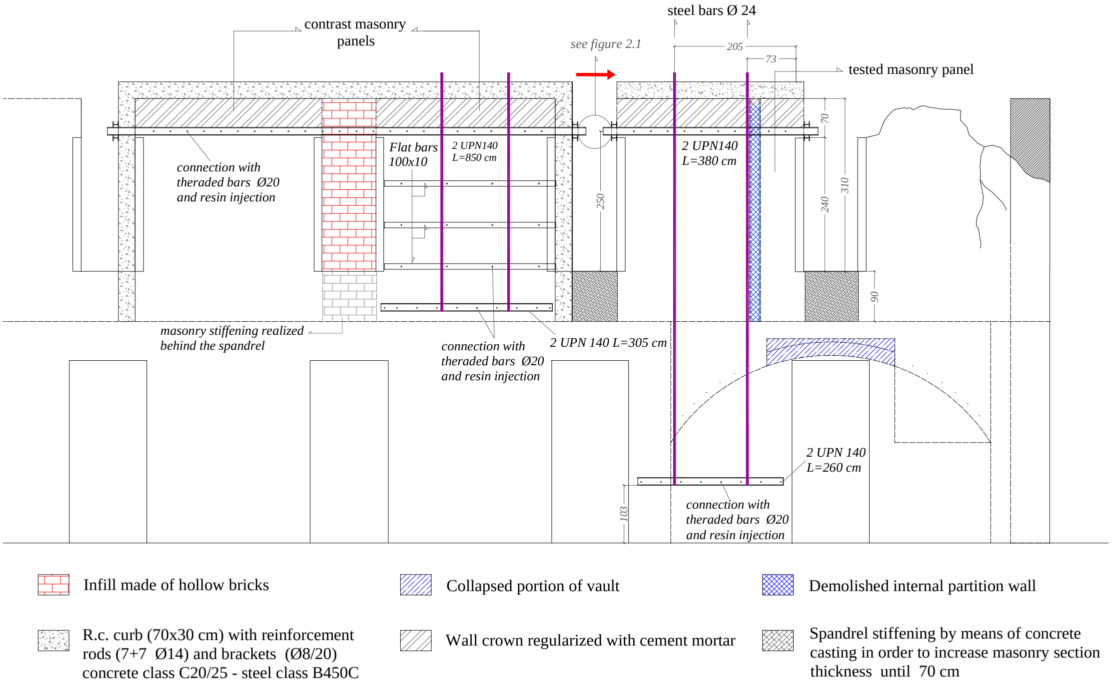

Figure 2.1 Test setup draft (the red arrow indicates positive direction of the applied load action)

Other preliminary operations carried out before the test have been: i) the reinforcement of the two spandrels next to the tested panel by means of a concrete casting in order to increase the restraint offered at panel base; ii) the demolition of a partition internal wall and of some unsafe floor portions; iii) the realization of a concrete curb on the top of the panel.

Figure 2.2 On the left, system of the actuator's connection; on the right, connection of the steel pre-stressed bars to the UPN steel profile.

Regarding the data system acquisition (load and displacements) the transducer location is illustrated in Fig. 2.3. An inclinometer has been further placed on the top of the panel.

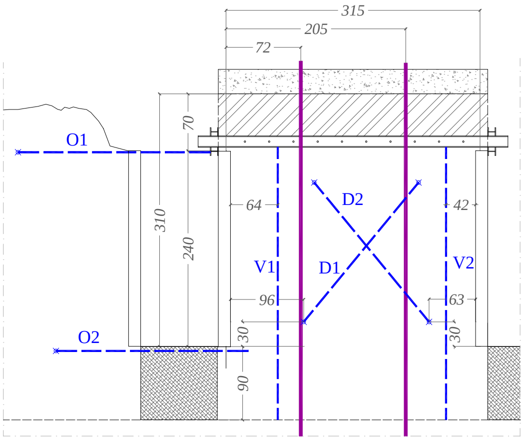

Figure 2.3 Draft of the transducers location on the masonry tested panel.

2.2. Description of the results

Two monotonous "force-control" tests have been performed on the masonry panel, first with a positive and then a negative direction (see reference system shown in Fig. 2.1). Once the axial load has been applied, a preliminary positive load sequence (horizontal shear action equal to 5 kN) has been executed in order to verify the effectiveness of test setup. Then, the test has been carried on with steadily increasing shear action with positive direction. Although the setup enabled to perform a cycling test, during the test performance the actuator was out of control and the action increased until to the failure of masonry panel. At this point, a second monotonous test has been directly executed along the negative direction.

The prevailing failure mode occurred in the masonry panel was the diagonal cracking shear mode one. Moreover, in the panel's extremities some less serious cracks came out, mainly related to the flexural response (see Fig 2.4 on the right).

Figure 2.4 The first two figures starting from the left represent the panel's damage at a the end of the test (positive and negative direction, respectively). On the right, cracks in the panel's extremities due to a flexural response.

Fig. 2.5 shows the shear-drift curve calculated from the horizontal displacements recorded by the sensors for both tests. Both curves are represented in the first quadrant to better compare their trend. In the diagram related to the negative direction test, it is possible to underline:

- a first stroke (1'-2) consequent to the progressive stiffness recovery because of the closing of the residual cracks at the end of the positive direction test (due both to diagonal cracking, in particular in the centre of the panel, and to joints' opening related to flexional behaviour);
- a second stroke (2-3) consequent to the actual panel's behaviour in the examined direction; it is worth nothing that this slope is not significantly lower than the positive direction test one, even if the masonry panel was already previously damaged. The maximum strength obtained in this second test (250 kN) is obviously a bit lower than the one determined from the first loading curve (293 kN).

A not so significant stiffness reduction may be noticed in the curve related to second test.

On the right of Fig. 2.5, an ideal diagram of the cyclic response of a masonry panel is illustrated; even if two monotonous tests have been performed in this case the diagram simulates the actual beginning condition of the panel (point 1') while the second test started. In this case, the shear-drift curve has been calculated by evaluating horizontal displacements minus the remaining ones recorded at the end of the first test. In Fig. 2.5, the last stroke are traced by a dashed line because some problems occurred during the acquisition of the results; however, it has been possible to estimate this part in both curves with other less accurate systems.

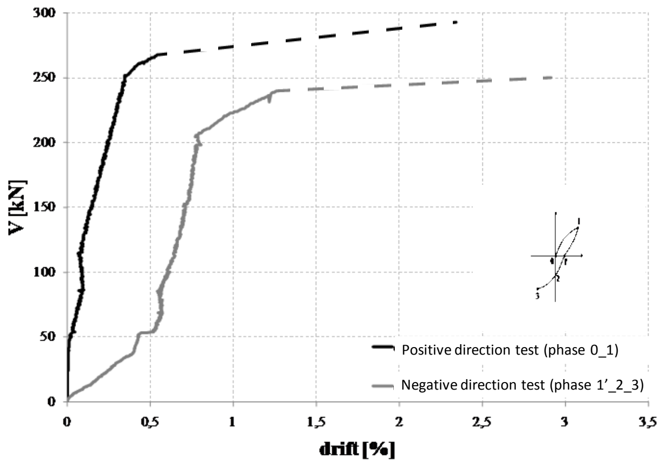

Figure 2.5 Shear-drift curve of the two in-plane shear tests and the ideal diagram of the cyclic response.

3. ADDITIONAL IN-SITU TESTS

Besides the in-plane shear test, some additional non-destructive and minor-destructive tests were performed in order to obtain more data and information to qualitatively characterize the examined masonry. In particular, the following tests have been adopted: sonic tests (direct and indirect), performed before the racking test execution; double flat jack test performed on the masonry panel used as contrast of the tested one (characterized by the same type masonry). In particular, these tests have been performed in order to: i) investigate the panel homogeneity and the cross-section quality of connection between two external leaves, in case of direct sonic test; ii) evaluate quantitatively and qualitatively the Young modulus E, through the double flat jack test and the indirect sonic test, respectively.

The sonic tests have been performed on a portion of the masonry tested panel, whose dimensions are 1,2 x 1 metres. The results are illustrated in a map representing the obtained sonic pulse velocities (Fig 3.1). Results highlighted that there wasn't evidence of detachments between the two external leaves by supporting a quite good connection between them.

As regard the double flat jack test, it has been performed according to recommendation of codes ASTM C1196-91 and RILEM Lum 90/2 Lum.D.2. Fig 3.2 illustrates post-processed results on the stress-strain diagram. The value of Young modulus E equal to 856 Mpa has been evaluated starting from the first stroke of the curve (up to the stress value of 0.9 MPa, which correspond to linearity loss). Since the stress-strain curve doesn't reach an evident plateaux, only a rough estimation of the masonry compressive strength has been obtained: it results about 2.1 MPa (which is a conservative value).

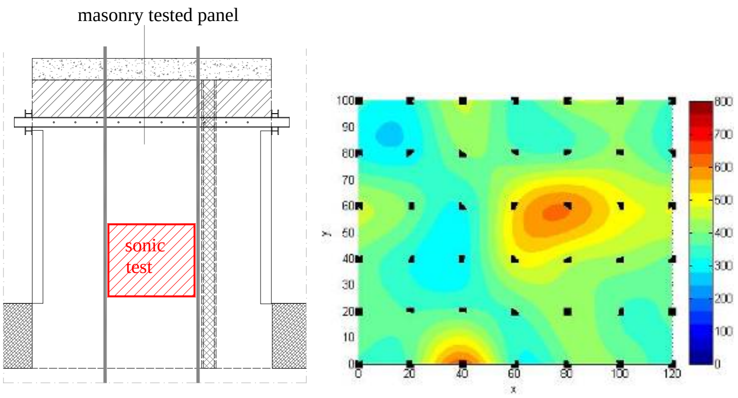

Figure 3.1 Masonry panel where sonic tests have been executed and map of the sonic pulse velocities.

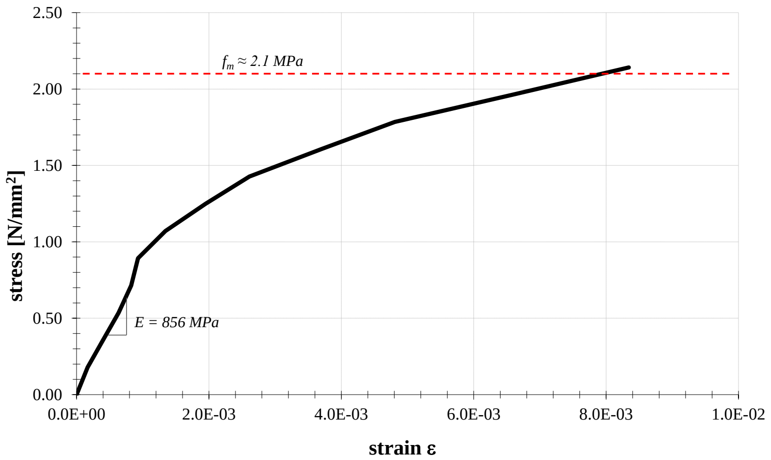

Figure 3.2 Medium strain-stress curve obtained by performing double flat jack test.

4. DISCUSSION ON RESULTS OBTAINED

In the following, some reprocessing of the results obtained is illustrated. Main aim is to evaluate the mechanical properties of the examined masonry starting from results of the in-plane shear test and then to compare them with the values proposed in the NTC 2008 (Table C8A.2.1).

First of all the mechanical properties of stiffness will be analysed in terms of Young modulus E and shear modulus G. From the results of the in-plane shear test, a first evaluation of the shear modulus G can be obtained by assuming the masonry panel's behaviour according to the beam's theory. Thus, by considering both the flexural and the shear contribution and by considering an ideal static scheme of a cantilever, it may be computed from Eqns. 4.1 and 4.2 as:

$$K = \frac{V}{u} = \left(\frac{3EJ}{h^3} + \frac{GA}{1{,}2h} \right)^{-1} \tag{4.1}$$

$$G = \frac{K \cdot 1{,}2 \cdot h}{A} \left(1 + \frac{4}{1{,}2} \frac{h^2}{l^2} \frac{G}{E} \right) \tag{4.2}$$

Where: *K* is the beam's stiffness, *h* and *l* are masonry panel's height and length respectively, *J* and *A* are panel's moment of inertia and cross section area respectively, while *V* and *u* have been obtained from the shear-drift curve for each test in correspondence of the clear loss of linearity (for the negative direction test by considering the curve after the recovery of stiffness occurred during the phase 1-2'). The resultant range for the shear modulus G is equal to 55-75 Mpa (by considering the results of the both performed monotonous tests); it has been calculated by supposing a ratio E/G of 3. Concerning this first evaluation, it may be stated as follows. This is in accordance to the assumption of a "fixed" restraint at the base of the panel: it has been hypothesized on basis of the reinforcement provided to this portion by infilling the spandrels next to the tested panels until reaching the same thickness. Indeed, it cannot be considered as infinitely rigid: in fact, at the end of the test, some damages occurred at the base of masonry panel (see Fig. 2.4). By taking into account of the limited stiffness of the constrain, the evaluation of G may be improved obtaining a range of about 80-120 Mpa.

In the following, the results are processed in terms of strength. As testified by the damage survey, the prevailing failure mode occurred in the masonry panel has been a diagonal cracking shear mode. According to the examined masonry type, it seems consistent referring – among the different criteria proposed in the literature - to the criterion proposed by Turnsek and Cacovič (1971) to interpreter the shear failure occurred. A more detailed discussion on the more suitable criteria to be adopted as reference as a function of different masonry types is illustrated in Calderini et al. (2010). Table 4.1 shows the strength criteria adopted to interpret the flexural and the shear failure modes. The flexural response is calculated on the basis of the beam theory, neglecting the tensile strength of the material and assuming an appropriate normal stress distribution at the compressed toe (according to criteria adopted also in Eurocode 8 – part 3 and NTC 2008).

Table 4.1. Resistance criteria adopted for pier in the NTC 2008.

Equ.	**Failure Mode**	**Strength criterion**	**Notes**
(4.3)	**Combined compressive and bending stress**	$M_u = \frac{lt\sigma_v}{2}\left(1 - \frac{\sigma_v}{0{,}85 f_m}\right)$	f_m masonry compressive strength, *l* section length, *t* thickness;
(4.4)	**Shear**	$T_u = lt\frac{1.5\tau_o}{b}\sqrt{1 + \frac{\sigma_v}{1.5\tau_o}}$	τ_0 masonry shear strength, *b* reduction factor depending to slenderness (Turnsek e Cacovič, 1971)

Fig.4.1 shows the comparison between the strength criteria and the experimental results.

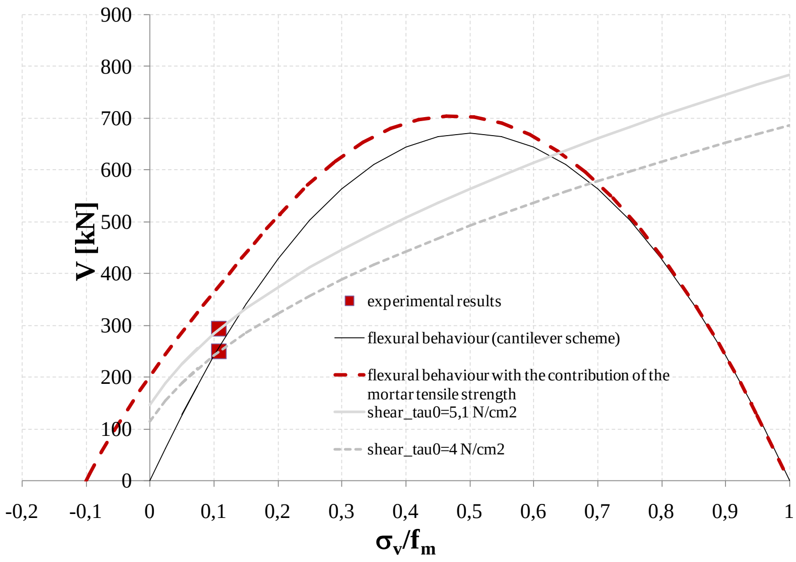

Figure 4.1. Comparison between resistance domains proposed in the NTC 2008 and experimental results obtained by racking test.

It is important to specify that: vertical stress σ_v has been calculated in the central section of the panel, by evaluating the loads of panel's weight and load transferred from the steel pre-stressed bars; the masonry compressive strength f_m has been precautionary assumed equal to 2.1 MPa, has shown in the double flat jack test's results; the coefficient b has been assumed equal to 1, because the panel's slenderness is equal to 0.75; in the combined compressive and bending domain, the correspondence between M_u and V_u has been obtained by considering a cantilever scheme without considering strength's reduction due to the hypothesis of stress-block (0.85). The domain related to the diagonal cracking shear failure mode have been plotted by imposing that Eqn. 4.4 passes through the experimental obtained values ($V_{u,test1}$ = 293 kN; $V_{u,test2}$ = 250 kN); as a consequence the estimate shear strength τ_0 is equal to 5.1 and 4 N/cm^2 respectively for the two performed monotonous tests.

It is important to notice that, next to the panel's compressive level (corresponding to the point σ_v/f_m=0.1), the strength prediction evaluated through the Eqn. 4.3 is a bit lower than the real masonry panel's one. Indeed, this formulation does not consider some contribution related to the mortar joints' tensile strength. For this reason, the compressive and bending domain evaluated by considering a minimum contribution associated to this effect has been plotted too, as shown in Fig. 4.1 by a dashed red line; for example, mortar joints' tensile strength has been generally assumed as 1/10 of masonry compressive strength.

5. COMPARISON WITH VALUES PROPOSED IN THE ITALIAN CODE FOR STRUCTURAL DESIGN

In this section the experimental results obtained from tests above discussed are compared with the values proposed in the NTC 2008 and its Instruction document (2009).

Indeed, the masonry typical of Abruzzo Region is not strictly classifiable within an exact type among those proposed in NTC 2008. Thus, firstly those with the more similar characteristics with the tested one have to be selected. In particular, those representative of an "Irregular masonry" (Type A) and an "Uncut stone masonry with facing walls of limited thickness and infill core" (Type B) have been assumed as reference. Table 5.1 shows the experimental results compared with the values proposed by the abovementioned code for both Types A and B selected.

Table 5.1. Comparison between experimental results and reference values proposed in the NTC 2008

Masonry type	f_m [N/cm^2]	τ_0 [N/cm^2]	E [N/mm^2]	G [N/mm^2]
A	100	2,0	690	230
	180	3,2	1050	350
B	200	3,5	1020	340
	300	5,1	1440	480
Experimental results	210 (from double flat jack test)	4 – 5.1 (from in-plane shear test)	856 (from double flat jack test)	80-120 (from in-plane shear test)

From Table 5.1 it may be stated as follows.
Concerning E and G parameters, the Young modulus - obtained from the double flat jack test - shows a good agreement with the ranges of NTC 2008 (especially with those proposed for Type A), whereas the range estimated from experimental shear tests for the shear modulus are lower than those of both types selected. Indeed, the range obtained by considering the constrain in the masonry panel's footing as not flexible, seem more representative of those evaluated in a cracked condition: for example, these values could be obtained from those proposed in NTC 2008 by multiplying them for 0.5 (as suggested by various codes such as Eurocode 8 and NTC 2008). In addition, as regard the G evaluation, it is important to point out that other experimental campaigns (e.g. that concerning in-situ diagonal compression tests illustrated in Brignola et al. 2009) highlighted an higher scatter of G values than shear strength for the same masonry type too.

Concerning strength parameters, the shear strength obtained (τ_0) is a bit higher than the mean reference values proposed in NTC 2008 for both types A (2.6 N/cm^2) and B (4.3 N/cm^2), whereas the compressive strength (f_m) values agree to the type B range.

As regard the ranges summarized in Table 5.1 as carried out from NTC 2008, it is important stressing that they refer to the "basic" condition of masonry, that is without the application of any corrective factors (associated to the good mortar quality, cross- section interlocking, etc.); moreover, both masonry types A and B are not able to completely reflect the features of the examined masonry. For example, type B refers to the presence of an infill core, not present in case of the examined masonry. Thus, this comparison has to be intended in some way conventional.

6. CONCLUSIONS

The paper shows the results of an in-situ experimental campaign addressed to provide a mechanical characterization of a typical masonry of the Abruzzo Region; in particular, it is focused on the direct evaluation of the shear strength. These values – hopefully corroborated in the future through additional experimental campaigns – could represent an useful reference for the definition of mechanical parameters to be adopted in the seismic assessment.

The values obtained testified a quality not particularly poor in comparison with some reference range proposed in the Italian Code for Structural Design (2008) related to similar masonry types.

In order to deepen the mechanical behaviour of this masonry, it seems important to further investigate the role of the cross-section interlocking and the quality of mortar joint (including its thickness and the actual contact among blocks). Concerning the first factor, even if the two external leaves are not connected through the systematic presence of transversal shear stone keys, masons used the few available bigger stones in order to ensure a sufficient connection; indeed, it is interesting to notice that detachments of external leaves didn’t occur at failure (as also testified in case of out-of-plane tests illustrated in Borri et al. 2012).

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