INVESTIGATION ON COMPRESSIVE BEHAVIOUR OF TUFF MASONRY PANELS

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ABSTRACT

Existing buildings and cultural heritage in the Mediterranean area are commonly composed by tuff masonry made by squared or roughly squared tuff stones. The Neapolitan yellow tuff, in particular, is a high porous volcanic stone that has been widely used as a building material to forge traditional and monumental architecture in the Campania Region.
Recent building codes focus their attention on the quantitative evaluation of the performance of existing structures under different limit states. In this context, it is evident that the availability of experimental data is of paramount importance for the vulnerability assessment and performance upgrading of existing tuff constructions. This paper reviews the experimental research carried out on medium-large yellow tuff masonry panels with single and multiple-leaf cross sections. The main target of the current work is to develop an extensive database on material data and mechanical properties of tuff masonry, in the light of recent test results. Based on the collected data, the reliability of available empirical-based relationships for estimation of strength and elastic stiffness of base materials and masonry was investigated. Moreover, the reference values of compressive strength and Young's modulus given by the Instructions to the Italian Technical Code, 2009 for soft stone masonry, have been compared against available data, and the main results are presented.

KEYWORDS: yellow tuff, masonry, compressive behaviour, seismic guidelines, existing buildings

1. INTRODUCTION

A large number of existing buildings located in different European countries are composed of tuff masonry. Significant examples can be found, for example, in Italy, the Netherlands and Germany (Nijland *et al.*, 2006; Hees *et al.*, 2004). In Italy, in particular, tuff was mainly originated as pyroclastic flow by volcanic eruptions, and can be found over Central and South regions, even for many kilometres away from the site of eruption (De Casa & Lombardi, 2007; Jackson & Marra, 2006; De Vivo, 2006).
Other types of tuff stones include those produced by a sedimentary process such as travertine and, "tufo bianco pugliese", which can be easily found within the regions of Lazio - Central Italy, and Puglia - South Italy, respectively (Stella, 1991; Evangelista *et al.*, 1990).
The Neapolitan yellow tuff has been the primary volcanic building stone used to forge the traditional and monumental architecture in Naples and surroundings, because it is the major pyroclastic tuff deposit in Phlegrean Field area, covering at least 500 km^2 (Orsi *et al.,*1992). It was the product of a huge eruption in this area dated about 12,000 years B.P. (Scarpati *et al.,* 1993).
The yellow tuff is a highly inhomogeneous material with a vesicular feature containing unevenly distributed cavities, pumices obsidian fragments, crystals and lithis embedded in an ashy matrix (Vanorio *et al.*, 2002; de' Gennaro *et al.*, 1996). It is basically a weak rock, characterized by low values of bulk density, compressive strength, and a quite high porosity ranging between 40 % and more than 60 %, which affects its durability (Ottaviani, 1988; Evangelista, 1980; Pellegrino, 1967; Nicotera & Lucini, 1967). Moreover, physical and mechanical properties of tuff vary widely depending on the quarry location and depth of extraction, and are highly sensitive to the degree of

saturation (Marcari, 2005; Ceroni *et al*., 2004; Evangelista & Pellegrino 1990; Bernardini *et al*., 1984; Evangelista, 1980).
In very populated urban areas around Mt. Vesuvius, masonry buildings have been traditionally built by using yellow tuff stones, laid in a running bond with low-strength pozzolana-based mortar, which is mix of ground lime and volcanic ash.
Load bearing masonry walls were characterized by weakly connected external leaves made of roughly squared tuff blocks (multiple-leaf wall), or stones placed through the entire thickness of the wall (single-leaf wall). In the former, the internal core was generally filled with mortar and small stones pieces remaining from cutting.
Different masonry textures were used throughout the centuries, and a common classification is as follows: "muratura listata", built from VI to XI centuries D.C., "muratura a cantieri" from XVI and XVIII centuries D.C., and "muratura a filari" built in XVIII-XX centuries D.C. (Calderoni *et al.,* 2007; Fiengo & Guerriero 1999; Burattini *et al*., 1994). Nowadays, the Italian Seismic Code NTC 08 (2009) prescribes the use of regular tuff stones, regular textures, stone and mortar with compressive strength higher than 5 MPa, in areas characterised by medium-high seismic risk.
Heterogeneity of existing tuff masonry, and the high seismic vulnerability of non-engineered masonry structures (Spence *et al*., 2004; D.P.C. 1999, 2000; Zuccaro, 1998), calls for a rational approach to assessment, well supported and validated by experimental research.
Most of the experimental works carried out in recent decades has been traditionally concentrated in the compressive behaviour of tuff masonry panels in the direction normal to bed joints (Romano, 2008; Augenti & Parisi, 2009; Prota *et al*., 2006; Cesi 2005; Calderoni & Lenza, 2004; Marcari, 2005; Faella *et al.,* 1991; Benedetti & Benzoni, 1985).
Special attention was paid to the softening response, which requires advanced displacement-controlled tests (Lourenço, 2002). In these works, medium to large-size wall panels were built with different dimensions and textures according to the scheme illustrated in Table 1.
Limited research on tuff masonry under eccentric loading is available (Faella *et al*. 1992), essentially aimed at investigating the effectiveness of traditional strengthening interventions. Experiments on masonry wall panels subjected to cyclic loading and to compressive loading in the direction parallel to the bed joints can be found in Bernardini *et al.,* 1984 and Augenti & Parisi, 2009, respectively, but data are rather limited on masonry of poor mechanical characteristics.
Attempts to develop analytical stress-strain relationships based on fitting of experimental data were made by Calderoni *et al.,* 2007, 2006, while in Augenti & Parisi, 2009, a relation based on probabilistic data processing was proposed. Comparisons between constitutive laws derived for brick masonry or reinforced concrete, and test data collected on yellow tuff masonry, can be found in Faella *et al.,* 1993.
Energy-based data, as well as the tensile and biaxial behaviour, are issues not yet considered in the literature. Such a lack of knowledge is clearly critical when vulnerability assessment is concerned, because numerical modelling of masonry would lead to inaccurate prediction of seismic capacity and failure mechanisms.
Within this context, the paper presents the state-of-the-art of the literature on representative yellow tuff masonry panels subjected to compression, with particular emphasis on recent published results. The aim is to share the effort made within the Research Project DPC-Reluis 2005-2008 (Magenes & Lagomarsino, 2009; Augenti & Romano, 2007), aiming at developing an extended database on tuff masonry panels. Both single and multiple-leaf panels have been accounted for, with different specimen sizes and block arrangements. The main results of experimental characterization on base materials (i.e. yellow tuff stones and pozzolana-based mortars) are also presented.
Since the selection of masonry mechanical parameters plays a key role in any rehabilitation design process, attention is focused on the range of reference values in compression for soft stone masonry typology, provided by the recent Instructions to the Italian Technical Code (NTC 08, February 2009). Comparisons with available test data allowed to assess reliability of the proposed range of values, with respect to yellow tuff masonry.

2. MATERIAL CHARACTERIZATION

A large number of experimental studies carried out to characterize the compressive behaviour of yellow tuff stones was found in the literature, and the reader is referred elsewhere for a more comprehensive discussion (e.g. Ceroni *et al.*, 2004; Di Pasquale *et al.,* 1992; Evangelista & Pellegrino, 1990; Ottaviani, 1988; Rippa & Vinale, 1983; Evangelista, 1980; Pellegrino, 1967; Dell'Erba, 1923).

In the following, attention is paid to the properties of tuff stones and mortar used to build the test panels in Tab. 1 (Calderoni *et al.*, 2009; Romano, 2008; Calderoni *et al.*, 2007; Prota *et al.*, 2006; Marcari, 2005; Cesi, 2005; Faella *et al.*, 2004; Faella *et al.*, 1991; Benedetti & Benzoni, 1985; Bernardini *et al.*, 1984).

The mean compressive strength (f_b) of stones ranges between 2 MPa and 5.73 MPa, with a mean value of 3.63 MPa (stand. dev. = 1.05 MPa; c.o.v. = 29.1 %). Information about experimental stress-strain curves in compression is still quite limited, due to the difficulty of carrying out stable softening test on tuff stones (Calderoni *et al.*, 2006).

In order to evaluate the tensile strength of stones (f_t), the most common indirect method used was the bending test. The tensile strength ranged between 0.30 MPa and 1.42 MPa, with a mean value of 0.63 MPa (stand. dev. = 0.48 MPa; c.o.v. = 77 %). In the work of Ottaviani, 1988, the ratio between the tensile and compressive strength values ranged from 0.09 to 0.19, with a mean value of 0.14.

A very low correlation between the tensile and compressive strength of yellow stones is observed. The average value of the ratio f_t/f_c approached 0.16, with a standard deviation of 0.10 and c.o.v. = 62 %. This trend is confirmed even when other data collected from literature survey (e.g. Asprone *et al.*, 2009; Evangelista & Pellegrino, 1990; Di Pasquale *et al.*, 1992; Stabilini, 1965) are accounted for. However, a linear interaction between the compressive and tensile strength with the porosity of stones (n) was found (Ottaviani, 1998). In particular, f_c tends to decrease as the porosity increases, with the law $f_c = 4.00 - 0.58\ n$ (MPa), where n is the porosity in percentage.

A relevant contribution on the experimental characterization of yellow tuff stones under dynamic tensile loading was given by Asprone *et al.*, 2009, but further research efforts are required in this area.

With respect to the Young's modulus E_b of tuff stones, the values range between 800 MPa and 3000 MPa, in agreement with results available in the literature (Croce & Pellegrino, 1967). Comparing the data set of f_b and E_b given by Calderoni *et al.*, 2006; Marcari, 2005, Paparo & Pellegrino, 1980; Pellegrino, 1967; Stabilini, 1965, no significant correlation between compressive strength and the modulus of elasticity is observed. In fact, E_b was found to vary between 200 and 950 times f_b, with an average value given by $E_b \approx 500\ f_b$.

Although experimental results on the elastic modulus of pozzolana-based mortars are rather scarce, the values resulted about 800 times the mortar compressive strength f_{mor}, which are consistent with the commonly accepted values for soft mortars, i.e. $E_{mor} = 800 - 900\ f_{mor}$ (Brooks & Abu Baker, 1998; Tassios, 1988).

The tensile strength of mortar was generally obtained performing standard flexural tests on prisms 40 x 40 x 160 mm, and the corresponding compressive strength has been obtained from standard tests carried out in the two halves of the prisms, according to UNI-EN 2000; 2001.

The ratio of tensile to compressive strength f_t/f_c has been found to vary between 0.2 and 0.5, with a mean value of 0.30. Regarding the effects of pozzolana materials over strength and capillary water absorption of mortar specimens, the reader is referred, for example, to Ekşi Akbulut & Aköz, 2006. However, it is noted that there is still a lack of knowledge about the uniaxial stress-strain behaviour of pozzolana-based mortars.

3. MASONRY BEHAVIOUR UNDER COMPRESSION

Analysis of the experimental data on medium-large size panels made possible the evaluation of strength and deformation behaviour. Detailed results can be found elsewhere (Augenti & Parisi, 2009; Romano, 2008; Calderoni *et al.*, 2007; Prota *et al.*, 2006; Marcari, 2005; Cesi, 2005; Faella *et* *al.,* 2004, Faella *et al.,* 1991, 1992; Benedetti & Benzoni, 1985; Sparacio, 1989; Bernardini *et al.*, 1984).

The multiple-leaf panels were characterized by two leaves of stones with the inner core filled with smaller stones and mortar. These leaves were generally weakly connected by using transversal stones. Often, transversal stones were placed only at the edges of the cross section of the wall.

The tests were performed in force control or in deformation control, but no harmonized test methods were used. The tests were made by using a stiff beam placed on a layer of mortar at the top base of the panel, in order to ensure a uniform distribution of vertical loading. Moreover, the load was applied through a spherical hinge that ensured that the applied load was centred and vertical.

3.1. Compressive strength

A large scatter characterizes strength, stiffness and post peak-behaviour, due to differences in terms of boundary experimental conditions and specimen dimensions (Mann & Betzler, 1994; van Mier, 1984). However, some tentative conclusions can be proposed for design and assessment purposes.

The compressive strength of multi-leaf panels was generally based on the gross area of the walls, neglecting the presence of multiple leaves, or possible differences in strength between the inner core and the surrounding masonry. Even though significant literature works on load-transfer mechanisms in multi-leaf masonry are currently available (Binda *et al.*, 2006; Oliveira *et al.,* 2006; Anzani *et al.*, 2005), a simple approach seems to be, the only feasible when addressing existing masonry, due to the difficulty of recognizing geometrical and mechanical properties of the leaves (FEMA 356, 2000; Instructions to the NTC 08, 2009).

Results from tests on single-leaf masonry panels showed a mean compressive strength ($f_{m,exp.}$) equal to 2.80 MPa, associated to a standard deviation = 1.14 MPa and c.o.v. = 40.3 %. As for multiple-leaf panels, an average compressive strength equals to 1.76 MPa was obtained, with a standard deviation = 0.88 MPa and c.o.v. = 50.4 %.

Good agreement in terms of global behaviour is found between masonry panels built with comparable materials strength and masonry layout (see Marcari, 2005, and Faella *et al*., 1991). Moreover, the masonry compressive strength could result lower than those of masonry constituents (see Romano, 2008; Marcari, 2005; Faella *et al*., 1991).

The data gathered allowed to compare the characteristic compressive strength f_k calculated according to the NTC 08 and EC6 (2005). The NTC 08 proposes a specific table to predict the characteristic strength of masonry when a comprehensive number of tests to determine f_k is not available. It is based on components characteristics: mortars are classified according to their mean compressive strength, while stones are characterized according to their characteristic compressive strength given by $f_{bk} = 0.75\, f_b$, where f_b is the stone mean compressive strength. The EC6 defines an empirical relation for the characteristic strength of masonry built with general-purpose mortar, with adjustment for unit proportions and wall characteristics. It can be pointed out that the NTC 08 gives on average values lower than those predicted by EC6 of about 20 %. A comparison study between characteristic strengths and EC6 predicted values can be found in Faella *et al.*, 1991.

3.2. Elastic modulus

The experimental elastic modulus $E_{m,exp.}$ varied from 680 MPa to 3000 MPa for single-leaf, and from 560 MPa to 1950 MPa for multiple-leaf panels.

In the case of single-leaf specimens, the average value of $E_{m,exp.}$ over the whole set of the specimens resulted equal to 1560 MPa (stand. dev.= 589 MPa; c.o.v = 37 %).

Regarding the multiple-leaf panels, the average Young's modulus approached 1130 MPa (stand. dev. = 419 MPa; c.o.v = 37 %).

European masonry codes (NTC 08, 2009; EC6, 2005) use empirical relationships to predict masonry characteristic strength f_k as follows: $E_m = 1000\, f_k$. Although such a relation is appropriate for new masonry, some useful comparisons against experimental data obtained for tuff masonry made with poor materials, can be found in Faella *et al.,* 1991. It has been shown that the proposed

formula led to an overestimate the experimental modulus especially when multi-leaf wall panels were considered.
Several attempts to develop more accurate methods of prediction of Young's modulus are available in the literature. This parameter is, however, rather variable even for nominally identical specimens, and a prediction of its value is not simple. In this paper attention is focussed on the ratio of the modulus of elasticity to mean compressive strength E_m/f_m, according to the technical literature (FEMA 356, 2000; ACI 530-08, 2008, Brooks & Abu Baker, 1998).
Single-leaf panels showed a mean value $E_{m,exp.}/f_{m,exp.}$ = 607, with a standard deviation = 282 and c.o.v. = 46 %.
Multiple-leaf approached the average value $E_{m,exp}/f_{m,exp}$ = 708 (stand. dev.= 245 ; c.o.v.= 34 %).
Consequently, the empirical relationship E_m/f_m= 800 proposed by Faella *et al.*, 1991 provides values higher than the experimental results in the case of single-leaf panels, while it seems reliable for multiple-leaf masonry.
Results allowed to estimate the ratio $E_{m,max}/E_{m,exp.}$, with $E_{m,max}$ the secant modulus of masonry at maximum stress. This is certainly of interest when analytical stress-strain relations available in the literature are used to predict the masonry response under compression (see Faella *et al.*, 1993). This ratio varies between 0.33 and 0.74 with a mean value of 0.73 for single-leaf, and between 0.59 and 0.74 with a mean value of 0.59 for multiple-leaf panels. Therefore, the suggested value for the ratio $E_{m,max}/E_{m,exp.}$ is 0.6 for single and multi-leaf walls.

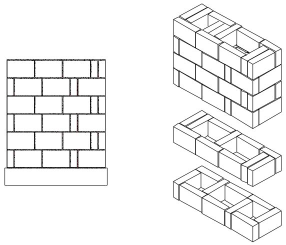
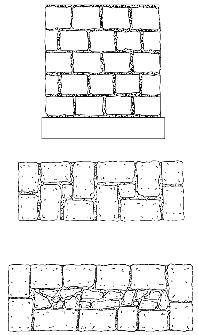
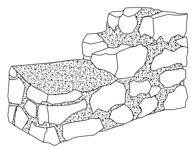
3.3. *The Poisson's coefficient*

The Poisson's coefficient v was investigated by Augenti & Parisi, 2009; Prota *et al.*, 2006; Marcari, 2005; Cesi, 2005; Faella *et al.*, 1991. This coefficient was generally calculated as the horizontal strain ε_h to vertical strain ε_v ratio within the range 0 – 30 % of the peak strength. In average, the single and multiple-leaf panels showed v = 0.13, associated to a standard deviation equal to 0.07 (c.o.v. = 52 %) and 0.04 (c.o.v. = 33 %), respectively.

3.4. *Maximum and ultimate strains*

The values of the maximum and ultimate strain, as well as the ductility ratio μ, have been investigated.
Maximum strain values $\varepsilon_{m,max}$ exhibt a large scatter. With reference to the single-leaf panels, the values vary in the range 0.24 % to 0.88 %, with a mean value of 0.39 % (stand. dev. = 0.16 %, c.o.v. = 41 %). About 67 % of data are in the range (0.2 – 0.4) % with a mean value of 0.31 %, and 33 % are higher than 0.4 %, with a mean value of 0.57 %. Data about multiple-leaf panels vary from 0.15 % to 0.61 %, with a mean value of 0.30 % (stand. dev. = 0.12 %; c.o.v. = 40.3 %). About 90 % of values are within the range (0.2 – 0.4) % with a mean value of 0.27 %, and 10 % are higher than 0.4 %.
The ultimate strain ε_u has been calculated as the strain corresponding to the ultimate stress.
Due to the uncertainty on the residual strength values, the ultimate strength is here assumed to be 85 % of the masonry compressive strength. The single-leaf panels exhibited higher ultimate strains than the multi-leaf ones. In fact, panels showed ultimate strains > 0.6 %, with an average of 1 %. The multiple-leaf walls, instead, showed values < 0.6 % in about 80 % of cases, with a mean value of 0.3 %.
The deformation capacity under compression is stressed by the ductility ratio μ, that was calculated as $\varepsilon_u/\varepsilon_{m,max}$. The single-leaf panels ranged between 1.40 to about 2.5, with an average μ = 1.90 (stand. dev. = 0.37; c.o.v. = 20.2 %). The multiple-leaf ranged between about 1.0 to 2.10, with an average μ = 1.50 (stand. dev. = 0.38; c.o.v. = 25.3 %).
It is also interesting to compare the ductility provided by EC6 (μ_{EC6} = 1.75) and the experimental results. Although the EC6 ductility holds for new masonry, it seems to be reliable for single-leaf masonry, but tends to overestimate the deformation capacity of multiple-leaf of about 20 %.

Table 1.A. Tests in medium-large tuff masonry panels available from the literature.

REFERENCE TESTS	MASONRY TYPE	DIMENSIONS (B x H x s) (cm)	MASONRY LAYOUT
Marcari, 2005	Multiple-leaf Panels W1 and W2	148 x 157 x 53	
Faella *et al.* 1991	Multiple-leaf Panels 1T1-2T1-3T1-4T1	130 x 125 x 50	
	Single-leaf Panels 5T2 and 6T2	130 x 125 x 50	
Calderoni *et al.*, 2007	Single-leaf - Cantieri Panel C1	125 x 90 x 67	
	Single-leaf - Cantieri Panel C2	120 x 95 x 65	

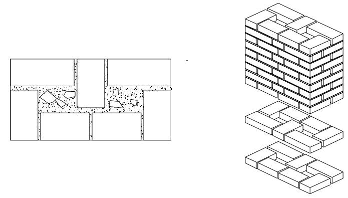
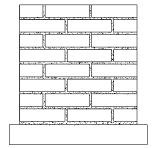
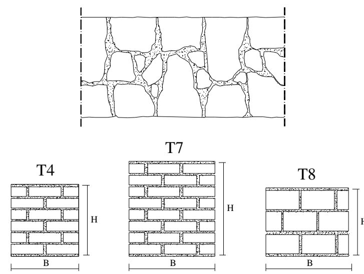
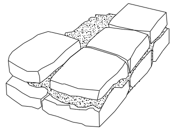
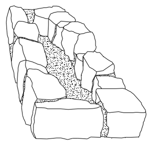
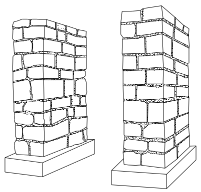
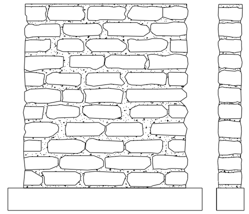
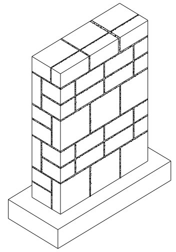
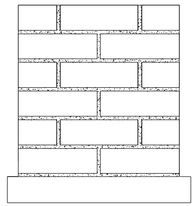
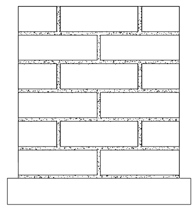
REFERENCE TESTS	MASONRY TYPE	DIMENSIONS (B x H x s) (cm)	MASONRY LAYOUT
Cesi, 2005	Multiple-leaf	60 x 76 x 30	
Prota *et al.*, 2006	Multiple-leaf	103 x 103 x 25	
Bernardini *et al.*, 1984	Single-leaf Panels TT4	82 x 86 x 25	
	Single-leaf Panels T7	104 x 104 x 25	
	Single-leaf Panels T8	100 x 83 x 12	

Table 1.B. Test in medium-large tuff masonry panels available from the literature.

REFERENCE TESTS	MASONRY TYPE	DIMENSIONS (B x H x s) (cm)	MASONRY LAYOUT
Calderoni *et al.*, 2007	Single-leaf Bozzette Panel B1	100 x 82 x 55	
	Single-leaf Bozzette Panel B2	100 x 86 x 55	
	Multiple-leaf Panel S1	133 x 91 x 42	
	Multiple-leaf Panel S2	120 x 88 x 42	
Benedetti & Benzoni, 1985	Single-leaf - good texture	80 x 80 x 50	
	Single-leaf - bad texture		
	Multiple-leaf - good texture		
	Multiple-leaf - bad texture		

REFERENCE TESTS	MASONRY TYPE	DIMENSIONS (B x H x s) (cm)	MASONRY LAYOUT
Sparacio, 1989	Single-leaf (bad text=bed joints only)	80 x 80 x 43	
	Single-leaf (good text.=head and bed joints)		
Faella *et al.*, 2004	Single-leaf	116 x 116 x 38	
Romano, 20082	Single-leaf	61 x 65 x 15	
Augenti & Parisi, 2009	Single-leaf	61 x 65 x 15	

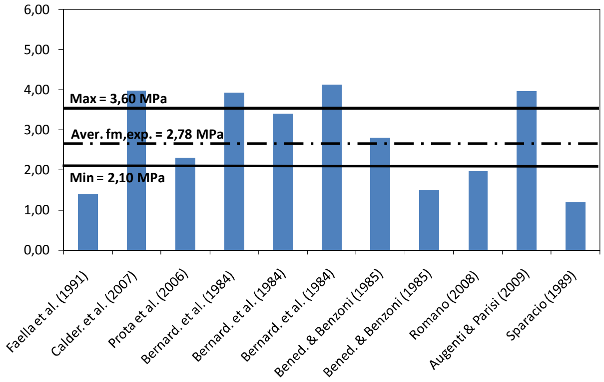
4. EXPERIMENTAL VS. INSTRUCTIONS CODE VALUES: COMPARATIVE ANALYSIS

The current section presents a comparative analysis aimed at investigating the reliability of the reference values given by the Instructions to the NTC 08 (2009), with special emphasis on the mechanical parameters in compression of yellow tuff panels.
The test masonry panels in Tab. 1 have been associated to "soft stone" masonry typology, as defined by the code. The corresponding range of average strength f_m and elastic modulus E_m values (maxima and minima) are summarized in Tab. 2.
It is worth noting that "soft stone" masonry encompasses a broad variety of stone masonry typologies made, for instance, with yellow tuff, grey tuff, tufo bianco pugliese, pietra leccese, calcarenite, limestone etc. Such materials are characterized by different mechanical properties as outlined, for example, by the works in Aiello & Sciolti, 2006; Ceroni *et al.,* 2004, so that different masonry response would be expected.
The range is given for masonry characterized by poor mortar quality, by the absence of layering with regular courses, by wall leaves merely placed together and badly connected, or with an inner core thinner than the outer leaf, and by loose stones. It can be also used for masonry with roughly squared blocks, namely "a conci sbozzati", with the inner core of good mechanical properties. Moreover, the elastic modulus in Tab. 2 is set up for uncracked masonry.

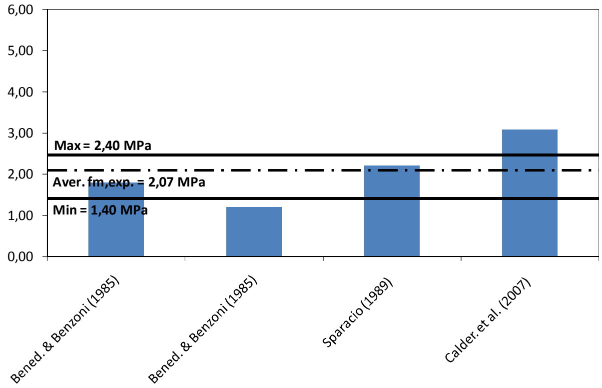
Table 2. Reference average values of the compressive strength and Young's modulus for tuff stone masonry (Instructions, 2009).

MASONRY PROPERTIES	f_m (N/mm^2)	E_m (N/mm^2)
Min value	1.4	900
Max value	2.4	1260

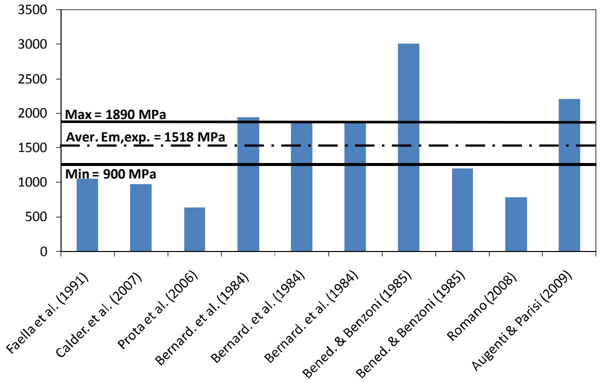
For masonry in good or fair condition, the mechanical parameters are modified using the values given in Tab. 2 and the following correction factors (Instructions, 2009): good mortar = 1.5; transverse connection = 1.5. The factor = 1.5 related to transverse connections should not be applied to the Young's modulus values given in Tab. 2. Moreover, any factor related to the presence of stone courses or ashlar borders is not provided by the code for soft stone masonry.
The multiple-leaf panels in Tab. 1 were designed with poor or large inner core, so that the corrective factor = 0.9 has also been used for this type of masonry, in compliance with the Instructions. For what concerns the quality of mortar, it has been assumed that the compressive strength not larger than 2.5 MPa corresponds to a mortar of poor characteristics. The value of 2.5 MPa is, in fact, the lower bound of the mortar compressive strength prescribed by the NTC 08 for the design of new masonry structures.
Comparative analyses are illustrated in Fig. 1 and Fig. 2. Examining the values suggested by the Instructions and comparing them with the experimental data, it is pointed out that good agreement between experimental and code values is generally found, and the corrective factor for good quality mortar seems to be well calibrated, mainly for single-leaf masonry (Figs. 1a, 1c). Differences between the mean elastic modulus and the range of code values were found for the case of panels made of poor mortar. The results presented in Fig. 1d, in fact, indicate that the mean response was beyond the range. It is worth noting that the panels considered in Fig. 1d showed a mean elastic modulus comparable to that of panels with good mortar (Fig 1d), which is not consistent with the generally accepted compressive behaviour of masonry. Therefore, more experimental study is required on this topic, accounting for different masonry textures and weak mortars.
The compressive strength of multiple-leaf panels with good mortar was found to be lower than the proposed range (Fig. 2a and Fig. 2b), but close to the lower bound, especially in absence of transversal connections (see Fig. 2b). However, the limited sample size requires more experimental outcomes to provide assessment of this trend. Furthermore, multiple-leaf arrangements made with good mortar showed an average Young's modulus very close to the lower-bound value, as illustrated in Fig. 2c.



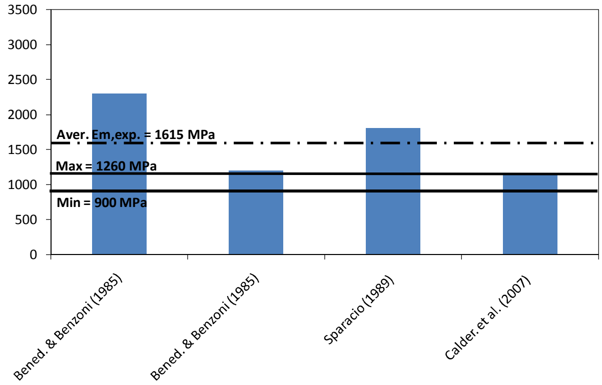
a) Good Mortar: *Compressive Strength*



b) Poor Mortar: *Compressive Strength*

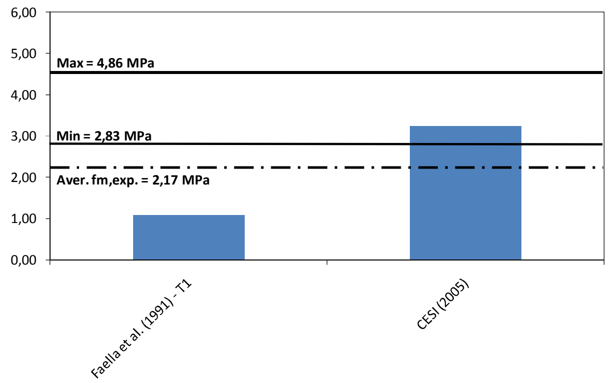


c) Good Mortar: *Elastic modulus*

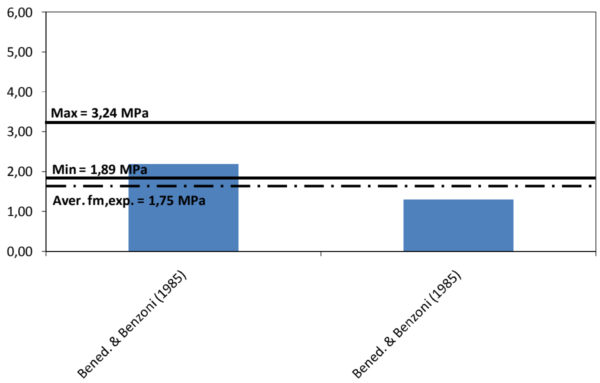


d) Poor Mortar: *Elastic modulus*

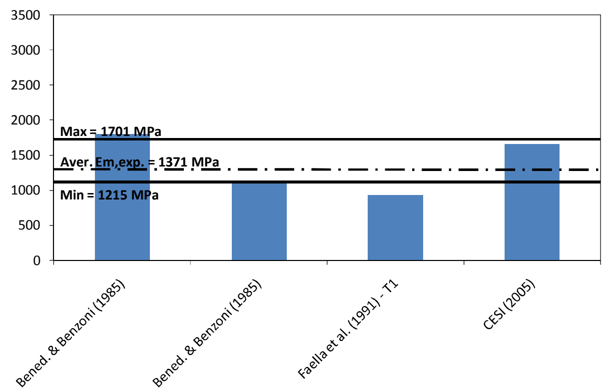
Figure 1. Single-leaf panels. Comparisons between the experimental values and the code range.



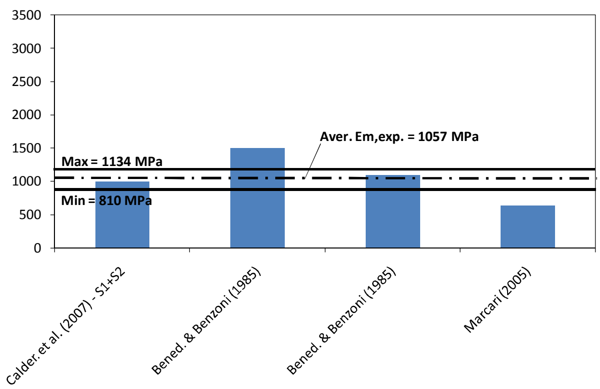
a) Good mortar + Yes trasv. conn.: *Compressive Strength*



b) Good mortar + No trasv. conn.: *Compressive strength*



c) Good mortar: *Elastic Modulus*



d) Poor mortar: *Elastic Modulus*

Figure 2. Multiple-leaf panels. Comparisons between the experimental values and the code range.

CONCLUSIONS

An extended review of the state-of the art on experimental research on yellow tuff stone masonry under compression, has been carried out. The collected data provided a database of results on both single and multiple leaf wall panels, and allowed to explore the reliability of the ranges of mechanical parameters given by the Instructions to the NTC 08 for soft stone masonry.

Even if the level of uncertainty in the data requires a statistical approach to analyse available results, some concluding remarks can be drawn from the analysis. The ranges of the compressive strength and elastic modulus in the Instructions, as well as the corrective factors, have been shown to be fairly appropriate for yellow tuff masonry. Moreover, the results indicate that the compressive strength and the elastic modulus in compression tend to the lower-bound values given by the Instructions, mainly when multiple-leaf are concerned. Finally, efforts to improve laboratory results through a comprehensive in situ characterization are strongly recommended, in order to develop reliable guidelines on assessment of existing tuff masonry performance.

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