MECHANICAL BEHAVIOUR AND STRESS DISTRIBUTION IN MULTIPLE-LEAF STONE WALLS

1. ABSTRACT

L. Binda¹, A. Fontana¹, G. Mirabella²

A proposal for the study of the mechanical behaviour under dead and live loads of multiple leaf stone walls was presented by the Authors in previous papers. Starting from an in-situ survey of wall sections, some tests were performed on small specimens and the results were simulated by a simplified linear elastic finite element analysis. In the present work a further step is therefore proposed with the assumption of a non-linear model of the joint behaviour. Moreover, the experimental work is extended to specimens of tuff masonry, which exhibit more regular shape of the stones and joint disposition compared to the masonries previously studied. Also for tuff masonry numerical analysis will be performed in order to appreciate the bond strength resources of the connection between the leaves, and to propose simplified models for the overall joint behaviour.

2. INTRODUCTION

A great variety of building techniques were used in the past for load bearing stone-masonry walls: they were different from one region of the country to the other, according to the quality of the local stones, the available facilities and the skillness of the workers. This variety can be found everywhere, in Italy as in other European countries, but an extensive classification of the different types has never been attempted so far. The analytical determination of the carrying capacity of load bearing walls under the heaviest loading conditions is especially important in seismic areas; several Italian regions fall under these circumstances. Nevertheless, the numerical analysis is particularly difficult in the case of multiple leaf masonry walls.

The first problem arises in the definition of the geometrical and technological characteristics of the wythes and of the joint system. A method for facing this problem was presented by the authors in [1], starting from an in-situ survey for a classification of the wall sections. A subsequent difficulty concerns the mechanical interpretation of the wall behaviour, owing to the construction technique and the type of connection between the wythes, which influence the load distribution within the section. Further difficulties arise in the definition of constitutive laws interpreting the local mechanical behaviour of the different material components and in the implementation of mathematical models reliable for a representative numerical analysis. Obviously these difficulties can be

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reduced when the material characteristics and the quality of the wall allow for the
homogenization of the masonry section and for the assumption of overall mathematical
models for the structural analysis. Unfortunately these circumstances very seldom occur
in the case of multiple leaf stone-masonry wall in poor constructions. These difficulties
were shown in [1] [2] following an extended in-situ survey and a laboratory experience.
Starting from the interest of the national research community on this subject, the authors
have set up in 1990 a research program to study the multiple leaf walls [1]; through the
experimental activity and the calibration of commercial codes with appropriate
constitutive laws, the interpretation of the complex reality of non-homogeneous masonry
is searched for.

The first phase of the above mentioned program already developed in [1], [2] and [3],
based on the proposal of linear models calibrated on stone-masonry with multiple wythes
section, only partially answers to the questions. In fact the proposed models, based on
the hypothesis of elastic-brittle behaviour, cannot either follow the inelastic evolution
of the structural system under increasing load beyond the elastic limit or give a prevision of
the value of the ultimate load-carrying capacity of it.

The proposed new phase of the research concerns therefore the experimental study of the
transfer of the shear stress between the wythes of the walls under vertical loads and the
consequent control of analytical nonlinear models. The results are also intended to be a
contribution to the national code on masonry, which at present gives very poor
consideration to the problems of repair and retrofitting of such masonry structures.

3. CHARACTERISTICS OF THE MULTIPLE LEAF SECTIONS

An extensive survey carried on by the authors on stone-masonry walls of some Italian
regions (Friuli, Liguria, Basilicata) have shown the complexity of the problem. The
surveyed sections can be subdivided into three main types:

a) single wythe walls, either with regularly or irregularly cut stones with sufficient
overlap to act as a single wythe (Fig. 1a);

b) two wythes, either separated by a continuous mortar joint (collar joint) or by a nearly
empty joint (Fig. 1b), or made with offset stones slightly overlapped (Fig. 1c);

c) three wythe walls, made with two external wythes of more regular units and an
internal one made with different types of rubble wall (Fig. 1d).

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Fig. 1 - Section classification.
From the complexity of cases described above, it is difficult to preview the stress distribution in the wall under vertical and horizontal loads, although it is clear that it will be strongly dependent on the morphological and mechanical properties of the wythes. Nevertheless the range of occurrences can be bounded within two limits:

1) the wall is a single wythe or the wythes are horizontally connected by stiff horizontal elements which can transfer the load from one wythe to the other. Their flexural and shear behaviour allows them to carry out the task and the stress distribution can be analytically determined through simple models;

2) the connection between the wythes is realized through vertical mortar joints, or there is a slight overlapping of the stones at each course, so that no continuous vertical joint is realized. The redistribution of the vertical forces is only possible through the bond between the stones or the dowel effect of the overlapping stones at each course. Even if a great deal of repair of stone-masonry walls by injection or jacketing has been done in Italy after the major earthquakes, the real distribution of stresses and strains within this kind of structures has remained practically unknown, so as unknown were the characteristics of the repaired sections until two or three years ago, when this one and few other surveys were started [4] [5]. Some experimental and numerical work has been done in these last years on small or small scale models [6] [7]. The work carried on at Karlsruhe University (Germany) was mainly concerning the case c) indicated above and was dealing with small scale models; from now on, there will be a tight connection between Karlsruhe and Milan research groups.

4. SYMPLE PHYSICAL MODELS AND EXPERIMENTAL RESULTS.

The cases b) and c) mentioned in the above Sec.3 were considered by the authors as the most interesting ones for the detection of the wall behaviour under vertical loads. A simplified representation of these cases is given in Fig.2a) and b), where the stones are considered regular.

In that case the amount of load transfer from one wythe to the others is at present unknown; nevertheless this is in practice treated as a case of non collaborating wythes, and jacketing or reinforcement are used for retrofitting. No attention is usually paid to the original material properties and to the possible incompatibilities between these materials and the repair.

As mentioned above, the shear bond between wythes is the only way to distribute the vertical loads between them. Experimental data are needed for implementing numerical models, in order to obtain reliable states of stress and strain simulating the wall behaviour. It is practically impossible to reproduce in laboratory full or small scale models of the walls, due to their unhomogeneity and to the characteristics of the aged mortars; nevertheless simplified models can be used to investigate the local behaviour.

Fig. 2 - Simplified representation of cases b) and c) of Fig. 1.
Two different types of stones, a sandstone (S1) from Toscana, and a soft calcareous stone (S2) from Matera, were used to study the shear behaviour of the wythes. An hydraulic lime mortar was prepared to represent the collar joint mortar; its composition was 1:3 lime/sand and the tests were carried out only after ten days curing, so that the strength of the vertical joints could simulate the effective weakness of the mortar found inside the walls. The mechanical properties of the materials are reported in Table 1.

<table>
<thead>
<tr>
<th>composition</th>
<th>f_k N/mm²</th>
<th>E N/mm²</th>
<th>Water Abs. %</th>
</tr>
</thead>
<tbody>
<tr>
<td>mortar 1:3 (l/s)</td>
<td>1.50</td>
<td>2,400</td>
<td>-</td>
</tr>
<tr>
<td>S1 sandstone</td>
<td>106.03</td>
<td>25,000</td>
<td>2</td>
</tr>
<tr>
<td>S2 calc.stone</td>
<td>5.94</td>
<td>6,147</td>
<td>21</td>
</tr>
</tbody>
</table>

Table 1 - Mechanical and Physical properties of the materials.

Four specimens were built, two with stone S1, two with stone S2; they simulate the two situation of the case b) of the previous paragraph if only one half of them is considered. The specimens were subjected to the shear bond test normally used for shear strength of mortar-brick joints[8], only providing a weak confinement in the normal direction. The horizontal and vertical displacements were measured as shown in Fig. 3; transducers 1 and 2 measured the differential vertical movements between the side stones and the central stone, while transducers 3 and 4 measured the differential horizontal movements between the central and the lateral stones.

Figs 4 and 5 report the plot of vertical load versus vertical displacement in the four cases investigated: weak joint and joint with offset overlapped stones for stone S1, S2. It can be seen that the weaker stone S2 gives a higher shear bond than S1, probably due to a better cohesion between mortar and stones, owing to the different porosity of the two stones (See table1). Fig. 5 shows clearly that when a small overlapping of the offset stones is realized, the joint becomes stiffer and the shear load is primarily transferred by the stones. Nevertheless the crack patterns indicate that, while in the case of S1 the peak load is higher due to a real collaboration of the stones, in the case of S2 the upper stone fails in compression before the load is transferred to the others. This result probably means that even the weak mortar used for building the specimen was too far strong for
the soft stones. In fig.5 the two peaks represent the subsequent failures of the two vertical joints. The mean values of $\tau$ at the collapse for the two weak joints are respectively 0.68 and 0.29 N/mm$^2$.

4. SHEAR BOND BETWEEN STONES AND MORTAR.

Triplets were built with the two types of stones following method suggested in [8] for brick masonry, to investigate the shear behaviour of the mortar-stone joints under vertical loads. The difficulties of choosing a good test method for the shear bond strength of masonry were described in [9]; the triplet test was considered by the authors as the most reliable for the purpose of this research.

The test was carried out as shown in Fig.6, under displacement control at a rate of displacement of 10 $\mu$m/s. In order to know the value of the cohesion and the value of the friction angle between mortar and stones, the triplets were tested under different levels of the normal load, calculated as nominal mean stress, as reported below:

<table>
<thead>
<tr>
<th>stone</th>
<th>applied normal stress [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>0.125 0.40 0.80 1.125</td>
</tr>
<tr>
<td>S2</td>
<td>0.0    0.125 0.40 0.80</td>
</tr>
</tbody>
</table>

In Fig.7 the applied shear load is plotted versus horizontal displacement in the case of the sandstone S1, for the four levels of the normal load; Fig.8 shows the behaviour of the calcareous stone S2.

The following comments can be made on the results obtained (see also Figs.13 and 14):

a) the value of the cohesion was lower in the case of the stone S1 (=0.33 N/mm$^2$); in fact the test at zero normal load could not be realized in this case;

b) a brittle failure takes place at the contact surface between stone and mortar for S1 (Fig.9);
c) in the case of stone S2 a fairly high cohesion (0.58 N/mm²) was detected, due to the good adhesion realized in the joint between mortar and stone (Fig.10); d) for S2, when the chosen highest value of normal stress was applied, corresponding to 0.80 N/mm², the failure occurred as a compression failure of the stone instead of a bond failure (see Fig.11), showing that perhaps the mortar used was even too strong for this particular stone; e) in both cases the percentage difference between the peak value of the shear stress $\tau_0$ and the residual one $\tau_\tau$ tends to decrease as the normal load is increasing; f) in the case of soft stones S2, the value of the peak shear stress $\tau_0$ and the value of the friction angle tend to be lower than for stone S1.

As a conclusion it seems clear that a good cohesion can take place between mortar and stones in the case of soft stones, even better than in the case of hard brittle stones rocks, due to the different porosity. Confinement to a high extent may not be useful for soft stones.
5. NUMERICAL INTERPRETATION

On the basis of the above mentioned experimental results a numerical model was implemented in order to interpret the local response of the material and the bond between stones and mortar. A finite element model was assumed in plane stress; the mesh is formed by 8 nodes elements of smaller dimensions where concentrated states of stress have to be represented. Special interface elements are used in order to simulate contact and friction between stones and mortar. Some results of the numerical analysis for stone S1 and S2 are shown in Fig.12. The shear vs. normal stress relationship in the interface element is represented by a Mohr-Coulomb friction law, similar to the one proposed in [10].

In the $\tau$-$\sigma$ plane the limit surface is represented by the line (compression positive):

$$\tau_0 = c_0 + \sigma \cdot \tan \phi_0$$

for $\sigma \geq 0$

a parabolic cut off is given by the condition:

$$\tau = 2f_r \cdot \tan \phi \cdot (\sigma/f_r + 1)^1$$

for $\sigma < 0$

where $f_r$ is the normal tensile strength in the joint, and $c_0 = 2f_r \tan \phi_0$.

The limit surface has an evolution according to an exponential law of decay as a function of the accumulated plastic slip $\gamma$. The law is assumed as follows:

$$\tau(\gamma) = \tau_0 [\alpha + (1 - \alpha)e^{-\gamma/\gamma}]$$

being $\alpha = \tau_\alpha/\tau_0$, and $\tau(\gamma) = \tau_\alpha$.

The local relationship was deduced from the tests described in sec. 4, carried out at different normal load for the 57
two types of stone; the lines representing the upper and lower bounds are indicated in Fig. 13, 14.

The results of the numerical analyses allow for some comments as follows:
- high concentration of stresses in the area where the load is applied, with calculated peak values even greater than the local strength of the material. This is confirmed by the experimental evidence: in the case of the S2 stone, as already mentioned, in the laboratory test conducted with a nominal stress of 0.8 N/mm² normal to the joint, the failure occurred in the central stone before the sliding of the joint;
- non-uniformity of the normal stresses is nevertheless detected in the joint, together with the presence of tensile stresses at the end of the joint itself which lead to a separation at the interface before reaching the sliding of the contact surfaces;
- as a consequence, a non-uniform distribution of shear stresses along the joint is realized, which causes the formation of sliding surfaces propagating from the side under tension.

6 CONCLUSIONS

The experimental results obtained at this stage of the research allow to draw the following remarks on the mechanical behaviour of the models representing the shear bond between the wythes:

a) when no normal stresses (e.g. confinements) are acting in the transversal direction of the wall, the law $\tau - s$ of the connection under vertical loads can be represented as elastic-softening stable in the sense that $(d\tau/ds) < 0$ after that $\tau$ has reached its maximum value $\tau_0$; this law can be represented as a bilateral, increasing up to the peak value and then decreasing;

b) when a confinement exists, represented by stresses normal to the joint, then the mechanical law in shear $\tau - s$ can be represented by a linear branch increasing up to a value $\tau_0$ corresponding to the peak stress and then a softening branch that reach asymptotically the value $\tau_u$. The ratio $(\tau_0 - \tau_u)/\tau_u$ is dependent on the value of the applied normal stress and tends to decrease as this stress increases.

c) when the value $\tau_u$ is reached, the friction, assured by the normal stresses, makes the behaviour of the joint very similar to a perfectly plastic material;

d) lower friction angle and higher cohesion seem to be typical of highly porous materials (like soft rocks) compared to the others;

e) the experimental tests confirm the existence of: (i) "weak" vertical joints when the transfer of the vertical loads from one wythes to the others occurs through shear stresses, (ii) "strong" vertical connections when loads are transferred mainly by a compression-flexural mechanism;
f) the shear strength of the vertical joint depends mainly on the mechanical characteristics of the stones in the case of "strong" joints;
g) the value of the ratio between the peak and the residual stress is a significant parameter for design or retrofitting of buildings situated in seismic areas: in fact, it allows to evaluate the capability of ductile behaviour of the joints.

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