Cyclic behaviour of brick masonry walls

G. Magenes & G.M. Calvi
Dipartimento di Meccanica Strutturale, Università degli Studi di Pavia, Italy

ABSTRACT: An experimental program on the seismic behaviour of old brick masonry walls is presented. Shear and compression tests on full scale walls have followed a broad investigation on the basic material mechanical parameters. The experimental results are discussed with reference to some of the existing models for the estimation of the strength of structural walls. Different failure modes can be predicted on the base of the properties of bricks and mortar. The relevance of the aspect ratio on the shear strength of a wall is discussed.

1 INTRODUCTION

The evaluation of seismic vulnerability of existing buildings has been recognized as a major problem either because of the large number of buildings constructed before the development of rational seismic codes and because of the lack of knowledge about the properties of the material used and about the seismic response of single elements and whole complex structural systems.

Effective models for the numerical simulation of such buildings need to be more refined in the constitutive relations of the structural elements and of their connections rather than to be able to represent the exact geometrical configuration.

The experimental testing of masonry elements is the only viable method to investigate the conditions under which different events may take place, and to quantify the variation of the basic mechanical properties. Only on this basis rational and effective numerical models could be developed.

2 OBJECTIVES AND METHODS

The main objective of the research presented in this paper is the experimental evaluation of the resistance and ductility of masonry walls, as a function of material properties, aspect ratio and load combinations.

The experimental results are a support for the evaluation of the effectiveness of existing structural models, and to suggest needed improvements.

A further objective consists in the evaluation of the suitability and reliability of simple tests for the estimate of the fundamental global mechanical properties which are used in the models.

The experimental program has included five full scale tests and preliminary tests on bricks, mortar, mortar joints and wallets. The capabilities of several models to predict the correct failure mode and shear strength have been checked and discussed. The overall cyclic behaviour as a function of the failure mode is of interest.

3 EXPERIMENTAL RESULTS

3.1 Preliminary tests

A number of preliminary tests had been performed on bricks, mortar and small masonry assemblies, to define the main mechanical properties of the constituent materials. All specimens had been tested according to the Rilem Guidelines (1988). The mortar was prepared with hydraulic lime and sand, in volumetric ratio 1:3.

The main results are presented in table 1. The wallets tested in compression were 250 (thickness, two unit breadth) x 765 x 680 (height) mm.

Particular care was used in testing brick triplets to evaluate the apparent cohesion and friction coefficient between bricks and mortar. It is believed that the best estimate for the relation between shear strength and normal stress is given by equation (1), which is represented by the straight line in figure 1:

\[ \tau = 0.206 + 0.813 \sigma \] (1)

The mechanical properties of the masonry were therefore similar to those of a very good quality old masonry.

3.2 Full scale cyclic tests

A total of five specimens were prepared, all having a width \( d = 1.5 \) m and a thickness \( t = .38 \) m, with a height \( h \) of 2 m (three walls) and 3 m (two walls). The purpose was to perform one preliminary monotonic test, and to explore two values of aspect ratio and two values
Figure 1: Results of the shear tests on mortar joints (triplets).

of nominal vertical compression stress $\sigma_m$ (0.4 and 1.2 MPa). The reasons for these choices have been discussed by the same authors (1991) and will not be discussed again here.

The tests have been performed according to the scheme presented in figure 2: the vertical force was applied first and the horizontal cyclic load was then applied keeping the valves of the vertical jacks closed. No rotation was therefore theoretically possible, and a double bending condition was present. As a consequence of the testing mode the vertical load increased together with the horizontal load, depending upon the stiffness degradation of the wall. The axial load increment was percentually more significant for a lower nominal value, but the forces in the jacks were monitored with continuity.

The results of the tests are summarized in table 2 and in figures 3 to 6, some comments are presented in what follows.

Table 1: Results of the preliminary tests; $E$ is the secant Young's modulus at $\sigma = 0.33 f_u$.

<table>
<thead>
<tr>
<th>test type</th>
<th>n. of specimens</th>
<th>mean (MPa)</th>
<th>c.o.v.</th>
</tr>
</thead>
<tbody>
<tr>
<td>compression on brick $f_b$</td>
<td>28</td>
<td>19.72</td>
<td>8.82%</td>
</tr>
<tr>
<td>splitting on brick $f_{mt}$</td>
<td>29</td>
<td>1.26</td>
<td>20.28%</td>
</tr>
<tr>
<td>compression on mortar $f_m$</td>
<td>15</td>
<td>4.33</td>
<td>1.84%</td>
</tr>
<tr>
<td>splitting on mortar $f_{mt}$</td>
<td>14</td>
<td>0.66</td>
<td>8.44%</td>
</tr>
<tr>
<td>mod. of rupt. on mortar $f_{mt}$</td>
<td>15</td>
<td>1.59</td>
<td>3.47%</td>
</tr>
<tr>
<td>direct tension on mortar joint $f_{ji}$</td>
<td>13</td>
<td>0.073</td>
<td>10.53%</td>
</tr>
<tr>
<td>compression on masonry $f_u$</td>
<td>5</td>
<td>7.92</td>
<td>20.2%</td>
</tr>
<tr>
<td>$E$</td>
<td>4</td>
<td>2991</td>
<td>15.1%</td>
</tr>
</tbody>
</table>

Wall MI1m and MI1, $h = 2$ m, $\sigma_m = 1.2$ MPa.
Wall MI1m was tested monotonically, wall MI1 cyclically. The maximum horizontal load corresponded to the first diagonal crack, and decreased rapidly to a lower value. The failure mode concerned mainly the mortar beds, with slight damage in the bricks.

Wall MI2, $h = 2$ m, $\sigma_m = 0.4$ MPa.
The first failure was due to a shear sliding mechanism located at the top mortar layer, with an apparent friction coefficient between 0.57 and 0.65. Thanks to the axial load increment the horizontal load also increased up to the formation of diagonal cracks. The post peak behaviour was similar to case MI1.

Wall MI3, $h = 3$ m, $\sigma_m = 1.2$ MPa.
The failure mode involved sub-vertical cracks started in the central area of the panel, with extensive brick damage. The cracks extended slowly, cycle after cycle, with a correspondent gradual strength deterioration.

Wall MI4, $h = 3$ m, $\sigma_m = 0.4$ MPa.
The mortar joints collapsed, allowing the formation of two wide diagonal cracks. The increment of the vertical load was in this case critical to avoid a flexural failure.

The overall cyclic behaviour seems to be strongly affected by the different failure mechanisms.

4 FAILURE MODELS

The dominant failure modes in the shear and compression tests are associated to the biaxial tension-compression state of stress at the center of the panel. A linear elastic finite element analysis was considered to be appropriate to evaluate the state of stress in the central area of the panels at the onset of the first shear crack. A no-tension condition at the top and bottom boundaries.

![Diagram](image_url)
Figure 3: Test on wall MI1 and MI1m (dashed line); $h = 2$ m.

was assumed, to simulate the presence of flexural cracks in the bed joints, which were detected in the tests.

A first interpretation of the results was tried considering the referential tensile strength criterion (Turnsek and Sheppard 1980), which assumes that shear cracking happens when the maximum principal tensile stress reaches a critical value $f_{tu}$. Table 2 shows how the maximum tensile stress in each wall at first cracking is considerably variable, and seems to assume two roughly constant values for the two different values of aspect ratio. It has to be noticed that this seems to be true even when the orientation of the principal stresses is approximately the same. This result instills some doubt on the accuracy of shear failure models based on the tensile strength of masonry assumed as a constant parameter, although these models have the advantage of being of simple practical use.

The f.e. analyses also showed that the walls with $h/d = 1.34$, in comparison with the walls with ratio $h/d = 2$, were characterized by higher values of the ratio $X = \sigma_x/\sigma_z$ at the center of the panel, where $\sigma_z$ is the normal stress in the vertical direction (i.e. perpendicular to bed joints) and $\sigma_x$ the normal stress in the horizontal direction (i.e. perpendicular to head joints). The influence of this parameter on the strength of masonry subjected to biaxial states of stress was pointed out by Dialer (1991), considering the results of a series of tests on biaxially loaded masonry panels. Although the influence of this parameter is rather evident from the experimental point of view, it is still not clear whether this influence has to do with an increased performance of the head joints, which takes advantage of the compression $\sigma_z$, or with other factors which still have to be detected.

A comparison with the failure criterion developed by Mann and Muller (1988) was done, where the strength of the head joints is also taken into account by means of a cohesion $c_{xz}$ and a friction coefficient $\mu_{xz}$, in addition to the strength of the bed joints which is characterized by the cohesion $c_{xz}$ and the friction coefficient $\mu_{xz}$. The criterion defines three failure conditions, namely friction failure in the bed joints, tensile cracking of bricks, and compressive failure of masonry. The latter condition is not relevant to our experimental results, while the crack pattern of the walls may be associated to the first two types of failure (or a combination of them). The criterion in its extended version assumes that at ultimate condition the head joints develop the maximum shear strength which is compatible with their failure domain. The failure conditions for these two mechanisms are given by the following equations:

$$\tau = \tau_j = \frac{c_{xz} + \mu_{xz}\sigma_z + (c_{xz} + \mu_{xz}\sigma_z)\Delta \mu_{xz}}{1 + \Delta \mu_{xz}}$$

(2)
for bedjoint failure, where $\Delta_z$ is the height of the brick and $\Delta_x$ is the width of the brick, and by:

$$\tau = \frac{t_b}{\alpha} \left( \frac{1}{f_{tb}} \right) \sqrt{ \frac{c_{xz} + 2c_{xx} + \sigma_{xz}}{f_{tb}^2} + \frac{\sigma_{xz}}{f_{tb}^2} }$$

for brick tensile failure, where $\alpha = 2.3$, $f_{tb} =$ brick tensile strength.

Following a procedure similar to the one presented by Mann and Muller (1988), the normal stresses $\sigma_x$ and $\sigma_z$ obtained in the f.e. analyses were substituted in equations 2 and 3 to calculate the shear stress $\tau$ at failure, and then the shear stress at failure was compared with the shear stress $\sigma_{xz}$ given by the f.e. analyses. The ratios $\gamma_1 = \sigma_{xz}/\sigma_{xx}$ and $\gamma_2 = \sigma_{xz}/\sigma_{xx}$ were defined, so that a value equal or less than one would imply that the failure condition for the considered failure mode is met. The values of the input parameters used in the criterion are shown in table 3, where it can be seen that the strength parameters for the head joints are assumed to be less than or equal to the values obtained from the triplex tests. It was found that the best agreement between experiments and numerical application is obtained when the cohesion in the head joints $c_{xz}$ is assumed to be zero in the walls with $h/d = 2.0$, and when it is assumed to be equal to $c_{xz}$ in the walls with $h/d = 1.34$. This could be interpreted as if the local friction failure in the head joints in these walls developed prematurely, before the attainment of the failure condition given by bed joint failure or brick tensile failure, because of the low value of compressive stress $\sigma_x$. This hypothesis is plausible, but the experimental observations do not allow its verification. With these hypotheses, the calculated values of $\gamma_1$ and $\gamma_2$ are given in table 4. According to the failure criterion the failure mechanism for each wall is given, by the lowest value of $\gamma$. In the cases where the values of $\gamma_1$ and $\gamma_2$ are close, a mixed failure mode can be supposed. The agreement with the observed experimental failure modes is very good. More details of this numerical applications have been presented by Mågenes (1991).

5 CONCLUSIONS

The experimental shear-compression tests on full scale walls have shown a prevalence of failure modes triggered by stress situation in the center of the panels, with two possible out comes:

- frictional failure of the mortar joints, typical of a lower axial action, and
- tensile cracking of bricks, typical of a higher axial action.
Mixed mode failures were also reported. The influence of the failure mechanism on the post-peak hysteretic behaviour (strength and stiffness degradation) has proved to be very significant.

The more slender walls (h/d = 2) have shown apparently a shear strength sensibly lower than the squatter walls (h/d = 1.34), for the same axial action. Some f.e. analyses have shown that this fact corresponds to different values of the horizontal compression stress, i.e. the compression stress perpendicular to the vertical mortar joints. A model proposed by Mann and Muller has proved to be able to predict the correct failure mode and to estimate with reasonable accuracy the failure load if different contribution of the vertical mortar joints are considered. Other models, based on the evaluation of the tensile strength of masonry do not seem to be effective in all the tested cases, even if the limited number of tests suggests caution in drawing definitive conclusions.

The possibility of estimating the masonry strength on the base of the properties of the constituent material seems to be promising.

The evaluation of the role played by vertical mortar joints deserves further experimental and analytical work.

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The Bricks used for the manufacture of the specimens were kindly provided by R.D.B. Nord, Pontenure (PC), Italy.
Table 2: Actions at first shear cracking and stresses at the center at the panels according to f.e. analyses.

Table 4: Minimum calculated values of the coefficients \( \gamma_f \) and \( \gamma_h \) for each wall

<table>
<thead>
<tr>
<th></th>
<th>( \gamma_f )</th>
<th>( \gamma_h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>MI1m</td>
<td>1.012</td>
<td>1.006</td>
</tr>
<tr>
<td>MI1</td>
<td>1.080</td>
<td>1.12</td>
</tr>
<tr>
<td>MI2</td>
<td>0.920</td>
<td>1.096</td>
</tr>
<tr>
<td>MI3</td>
<td>1.350</td>
<td>1.165</td>
</tr>
<tr>
<td>MI4</td>
<td>1.101</td>
<td>1.249</td>
</tr>
</tbody>
</table>


Table 3: Values of the parameters used in the failure criterion

<table>
<thead>
<tr>
<th></th>
<th>( \gamma_f )</th>
<th>( \gamma_h )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \Delta_s ) (cm)</td>
<td>12</td>
<td>25</td>
</tr>
<tr>
<td>( \Delta_s ) (cm)</td>
<td>5.5</td>
<td>5.5</td>
</tr>
<tr>
<td>( f_H ) (MPa)</td>
<td>0.84</td>
<td>0.84</td>
</tr>
<tr>
<td>( c_{es} ) (MPa) (bed joints)</td>
<td>0.206</td>
<td>-</td>
</tr>
<tr>
<td>( \mu_{es} ) (bed joints)</td>
<td>0.813</td>
<td>-</td>
</tr>
<tr>
<td>( c_{es} ) (head joints)</td>
<td>0 - 0.206</td>
<td>0 - 0.206</td>
</tr>
<tr>
<td>( \mu_{es} ) (head joints)</td>
<td>0.406 - 0.813</td>
<td>0.406 - 0.813</td>
</tr>
</tbody>
</table>

**REFERENCES**