



STRENGTH AND BEHAVIOUR OF HISTORIC MASONRY UNDER LATERAL LOADING

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Abstract

The paper describes an experimental investigation done to evaluate the shear strength and behaviour of typical ancient Italian brick masonry. The failure of historic masonry in shear is due to the failure of interface between the brick and mortar. The shear strength of historic masonry increases with pre-compression up to a limit and becomes constant at high pre-compression. The experimental data give useful information on the response of historic masonry in combined compression and shear. It appears from the results that the EC6 under estimates the strength of this type of historic masonry in combined compression and shear. The effective rigidity of the wall increases with increase in pre-compression. The distribution of normal strain and thus the stress is non-linear along the length of the shear wall even at very low shear stress.

Key Words

Historic masonry, shear strength, models, pre-compression.

1 Introduction

Although, the historic masonry structures are not considered to be resistant to earthquake, many have resisted the effect of seismic actions without any damage. Of those that suffered, the most frequent cause of damage was due to shear. To conserve and to assess the vulnerability of our heritage masonry structures, it is therefore fundamental to understand their behaviour and strength in shear.

Since the development of modern structural masonry in 1950, numerous studies (Benjamin and Williams, 1958; Sinha, 1967; Sinha and Hendry, 1969; Hendry and Sinha, 1971; Turnšek and Cačovič, 1970; Mann and Muller, 1980; Yokel and Fattal, 1976; Drysdale et al., 1979; Riddington and Ghazali, 1988; Atkinson et al, 1989) have been done to define the rules to evaluate the strength in shear, but very little is known about the historic masonry subjected to combined compression and shear.

Therefore, the authors have adopted an experimental technique to investigate the behaviour of a typical Italian historic masonry subjected to shear.

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The experiments were designed taking into consideration the following factors:

- The studies of earthquake damage of historic buildings have shown the influence of construction technique and the material properties, such as unit and mortar strength, hence similar material and construction technique were used for the construction of the model walls.
- The failure of panel happens primarily due to shear with little or no vertical tension developing to initiate failure, a point often missed in many shear tests.

2 Experimental investigation

2.1 Construction of the test structures

During the renovation of an 18th century Italian building, a few full-scale solid bricks became available. Hence, the test structures were built of bricks in 1/3rd scale obtained from these full-scale bricks. The dimensions of model bricks were 100 x 50 x 17mm. The average compressive strength of the model clay bricks was 34.3 N/mm². A low strength mortar (2.5 to 4.6 N/mm²) was used for the construction of the walls. Due to the scarcity of the 18th century bricks, the flange was made with mortar. A steel plate was used as a slab on the top of the wall. The plate was joined with epoxy resin so that the failure in shear happens in the wall and not at the interface of slab and wall.

The historic model test structure is shown in Figure 1.

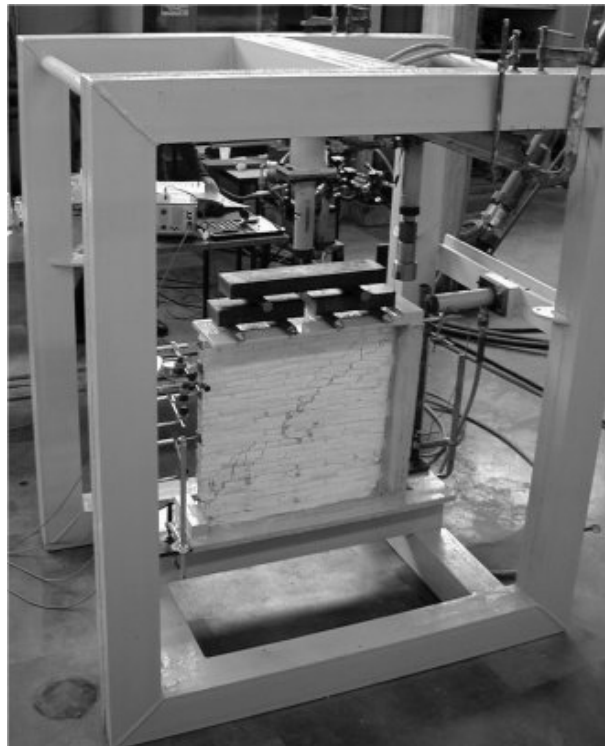


Figure 1 – Test structure and the loading frame.

2.2 Shear test arrangements and instrumentation

Seven single-story structures designated as P1-P7 were built and tested in a special frame (Figure 1). The pre-compression varied from 0.3 to 2.98 N/mm². The dimensions

of the panels tested were 630 x 630 x 50 mm (l x h x t). Three independent jacks; two for applying vertical compression and the third for shear load were used (Figure 1). The applied loads were monitored by three load-cells.

Five electrical transducers numbered 1 to 5 were used to measure the deflection at various levels of the structure as shown in Figure 2. A 100 mm 'deformmeter' was used to measure the vertical strains at four positions along the length designated by letter A to H on both faces of the shear wall as shown in the Figures 2. The principal stains were measured by strain rosette in the centre of the wall on both faces.

Before the application of shear load, full pre-compression was applied to the flange and the web of the wall. The shear force was applied in stages till failure and deflections and strains were measured at various stages till it became unsafe to take the readings.

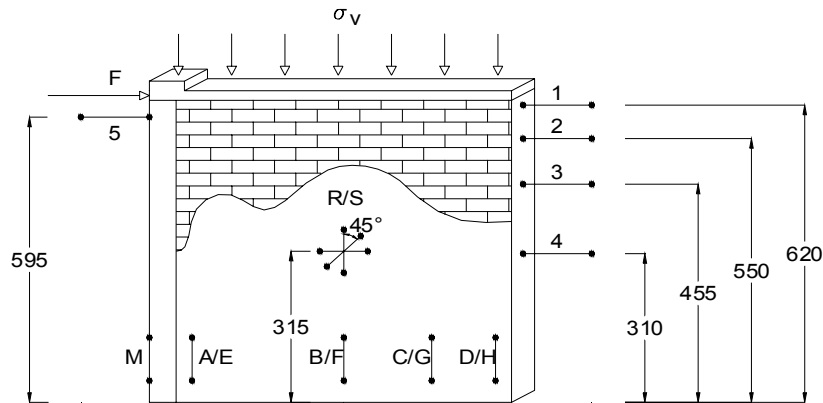


Figure 2- Shear test arrangement and Instrumentation.

3 Experimental results

The result of the shear tests is shown in Table 1.

Table 1- Results of the historic masonry shear tests at various pre-compression

Type of shear model	Pre-compression σ_y (N/mm ²)	Shear strength τ (N/mm ²)
P1	0.50	0.66
P2	0.75	0.68
P4	0.30	0.54
P5	1.15	1.43
P6	2.25	1.90
P7	3.00	1.90

A typical failure of panel in shear at high pre-compression is shown in Figure 3.

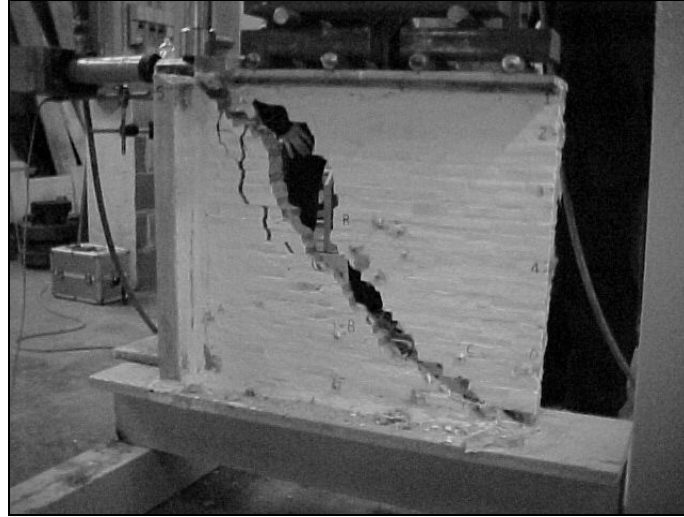


Figure 3 – Showing failure in shear at high pre-compression (Model P5).

3.1 Ultimate shear stress

The result given in Table 1 follows the well-established Coulomb criteria:

$$\tau_u = v_{b0} + \mu \cdot \sigma_v \quad (1)$$

where, v_{b0} initial bond shear stress at zero pre-compression, μ the friction coefficient and σ_v normal stress.

The shear stress increases linearly with increasing pre-compression up to a limit; this limit was 2.25 N/mm^2 of pre-compression in this case. The above relationship is not valid beyond this limit. The ultimate shear stress remains constant between pre-compressions 2.25 N/mm^2 to 3.0 N/mm^2 .

On critical examination of the test results, it appears that the initial bond shear reduces to zero at higher pre-compression, the limit being 3.0 N/mm^2 for the current work. This phenomenal behaviour is similar to those found by Sinha and Hendry (1971) in controlled shear box tests.

From their tests, the general criterion for shear failure was formulated as follows:

$$\begin{aligned} \tau_u &= v_{b0} + \mu \cdot \sigma_v & \sigma_v &\leq \sigma_{y1} \\ f_t &= -\frac{\sigma_v}{2} + \sqrt{\left(\frac{\sigma_v}{2}\right)^2 + (\tau)^2} & \sigma_{y1} &\leq \sigma_v \leq \sigma_{y2} \\ \tau &= \mu \cdot \sigma_v & \sigma_{y2} &\leq \sigma_v \leq f_{\text{comp}} \end{aligned} \quad (2)$$

where f_t and f_{comp} are, respectively, the tensile and compression strength of masonry.

According to them, there will be no increase in shear stress between pre-compression σ_{y1} and σ_{y2} .

The value of σ_{y1} and σ_{y2} for the current work appears to be 2.25 N/mm^2 to 3.0 N/mm^2 respectively.

Figure 4 compares the results of shear strength with the provisions of EC6. The initial bond strength obtained in the present test was higher than given in the EC6.

In this code the upper limit for the shear strength is 1.4 N/mm^2 , which is also lower than the test results of the historic masonry.
 From the results, it can be concluded that the provisions of EC6 will under estimate the shear strength of this type of historic masonry.

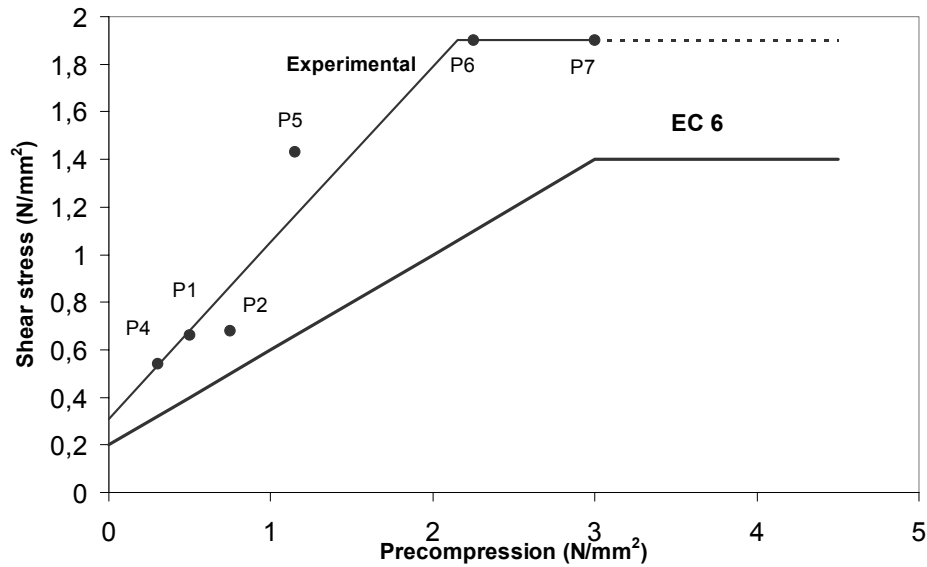


Figure 4 - Comparison of test results with the provisions of EC6

3.2 Deflection

The horizontal displacements recorded during the shear tests for three models (P1, P5 and P6) subjected to pre-compression from 0.5 to 2.25 N/mm^2 are given in the Tables 2 to 4.

Table 2 - Horizontal displacements - **P1**

$\sigma_v = 0.5 \text{ N/mm}^2$	1	2	3	4	5
F (KN)	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}
6.0	164	115	92	61	-103
9.0	286	180	153	95	-185
12.0	456	285	228	138	-282
13.5	555	340	265	158	-331
15.5	758	445	366	214	-430
17.5	1177	730	611	393	-614

Table 3 - Horizontal displacements - P5

$\sigma_v = 1.15 \text{ N/mm}^2$	1	2	3	4	5
F (KN)	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}
1.0	24	8	7	4	-14
1.5	33	14	15	6	-33
2.5	51	30	15	14	-46
7.5	120	93	65	46	-108
10.0	164	131	98	68	-131
15.0	229	190	148	98	-220
20.0	308	261	206	135	292
25.0	390	332	263	171	-385
27.0	428	366	291	188	-414
32.0	570	483	436	257	-

Table 4 - Horizontal displacements – P6

$\sigma_v = 2.25 \text{ N/mm}^2$	1	2	3	4	5
F (KN)	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}	mm 10^{-3}
5.0	63	57	46	30	-47
15.0	219	202	162	102	-170
25.0	398	371	309	197	-333
35.0	613	574	460	291	-493
45.0	942	882	709	446	-775
55.0	1270	1193	964	609	-1078
60.0	1600	0	0	0	0

Figures 5 shows the relationship between the lateral load and the deflection at the top of the structures. The lateral deflections are of the unloaded side for P1, P5 and P6 models (Points 1) which were subjected to pre-compression $\sigma_v = 0.5 \text{ N/mm}^2$, 1.15 N/mm^2 , 2.25 N/mm^2 respectively.

The results clearly show that the apparent stiffness increases with the increase of pre-compression (Fig. 5).

Although, there is some apparent increase in the stiffness between the structures subjected to pre-compression of 1.25 to 2.25 N/mm^2 , this increase is more pronounced between low (0.5 N/mm^2) and high pre-compression (2.25 N/mm^2).

Figure 6 shows the distribution of the normal strain along the length of the walls at the bottom for the lateral load only. The normal strain and thus the stress distribution along the shear wall appears to be non-linear even at very low shear load. With the superposition of the pre-compressive strain, the entire wall section will be under compression.

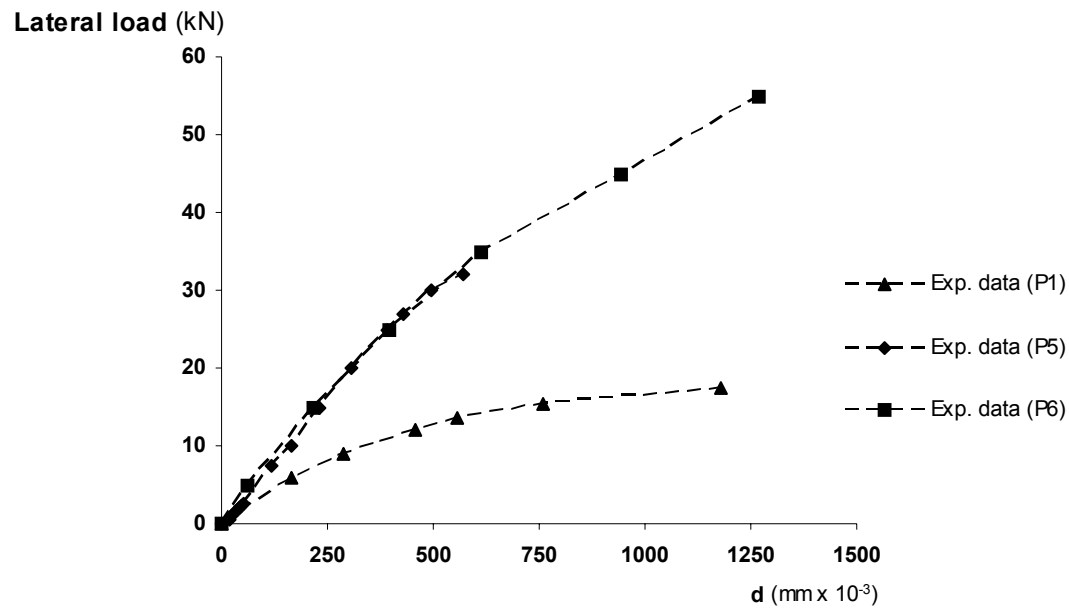


Figure 5 – Comparison of the deflections at the top of the structures for the various stages of the lateral loading.

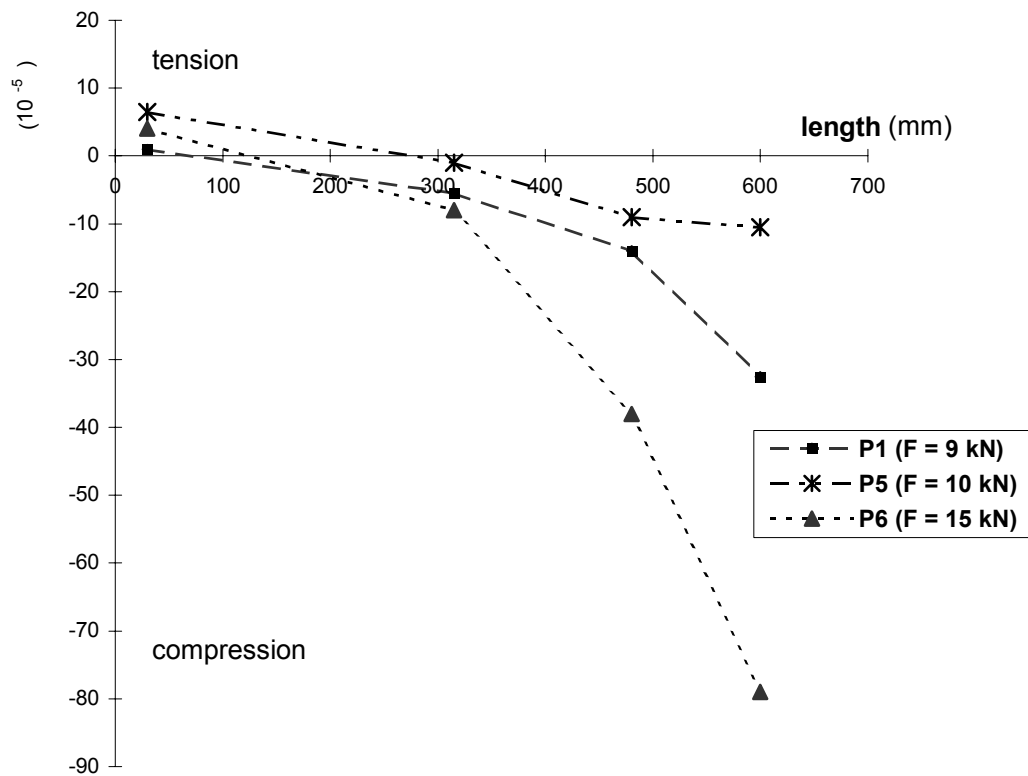


Figure 6 - Vertical strain (a) P1 at F=9kN and (b) P5 at F=10kN and (c) P6 at F=15kN

4 Conclusions

On the basis of this investigation, following conclusions can be drawn:

- i) The shear strength of historic masonry increases linearly with pre-compression to a limit. This limit being 2.16 N/mm^2 in this case. Beyond this limit, the shear strength remains constant.
- ii) The EC6 underestimates the shear strength of the historic masonry of this type.
- iii) The apparent stiffness of the structure increases with the pre-compression. The distribution of normal stress along the shear wall is non-linear even for low shear stress.

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