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Research Article

Shear Failure of Historic Masonry Walls

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Abstract

Historic masonry walls have showed excessive weeknesses and very low shear capability whenever subjected to seismic loads. The behavior of historic unreinforced masonry (HURM) walls under combined compression and shear loading plays a essential position in the seismic confirmation of masonry structures. Experimental research on historic masonry were taken out by the author on shear walls and triplets developed with total clay bricks in scale 1:3rd to determine the shear behavior of HURM walls. On the foundation of experimental investigation, a shear criterion for HURM has been presumed. It appears that the shear strength of masonry raises with the pre-compression up to a limit and turns into constant at higher pre-compression. Lastly, the experimental load-deflection connection and cracking distribution alongside the surface of the shear walls have been investigated with the hypothetical outcomes attained by a non linear FE analysis of HURM walls and a good co-relation was discovered between them.

Keywords: Shear behavior, masonry, unreinforced masonry walls, seismic loads.

1. Introduction

The preservation of the architectural heritage presents one of the important challenges in civil engineering due to the complexity of the geometry of the structures, the variability of the materials used and the loading history of the buildings. Further, majority of un-reinforced historic masonry structures are in seismic zones. Although, these historic masonry structures are not considered to be resistant to earthquake loading, many have resisted the effect of seismic action without any damage. Of those that suffered in the past, one common cause of damage was due to shear. (Bruneau M, 1994; Celep, 2005; Pagnini, et al., 2011; Syrmakezis et al., 2008).

Many investigators have examined the behavior of modern masonry wall with different shear test methods (Benjamin and Williams, 1958; Riddington and Ghazali, 1988) but very little is known about the historic masonry subjected to combined compression and shear (Capozucca and Sinha, 2004; Capozucca and Sinha, 2005), Hence, the knowledge of strength and behavior of the historic un-reinforced masonry (HURM) becomes essential for assessing the safety of the structure or for the planning of structurally compatible.

When lateral loads are applied to URM walls, several mechanisms may develop although damage mechanisms observed in masonry buildings may be divided in two following categories: mechanisms of 1st mode, where walls are stressed orthogonally to their main plane; mechanisms of 2nd mode, where walls resist to lateral loads in their own plane. Mechanisms of 2nd mode include 3 failure modes: sliding or friction failure, when slipping of upper part of the wall along a horizontal mortar joint may generally happen due to low vertical pre-compression and/or low bond; flexural failure, characterized by horizontal tensile cracks at the panel toe on the loaded side; *shear failure*, when diagonal cracks crossing both bricks and mortar joints appear (Magenes and Calvi, 1997). Since the main non linear effect of URM under in-plane lateral load is due to progressive cracking and strength degradation, it is important to develop a representative failure criterion. Most masonry walls subjected to in-plane loads are in biaxial stress state. Many researchers have investigated the behavior of URM under biaxial stress state (Turnešec and Cačovič, 1970; Hendry, 1978; Page et al., 1980; Hamid and Drysdaleü, 1981). The general failure of isotropic materials in plane stress can be defined in terms of principal stresses.

Most of the masonry structural elements have one common characteristic: the units are stacked in a periodic way. Due to this special future, the homogenization technique of periodic media is very popular among researchers (Lourenço, 2007), as a tool for structural analysis of large scale structures, rather than a detailed micro-modeling approach. Macro and micro-modeling approaches departs from the scale of >$10^{-2}m$, where the materials properties of mortar, brick and interface represent the macro properties of these components obtained in laboratory tests. However, it is known that the mechanical performance of materials depends on their morphology and phase constitution, in case of a composite.

Therefore, it is correct to assume that the structural behaviour of masonry on engineering scale is ruled by the phenomena existing on scales much lower than >$10^{-2}m$, which are related with microstructure of mortar, clay brick and eventually theirs interface.

Conventional concrete failure criteria have been also adopted by investigators with slight modifications. Von Mises yield surface is used to predict a crushing type of failure, whereas a tension cut-off surface is

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used for tension-tension or tension-compression type of failure (Chen and Saleeb, 1982).

Masonry is a heterogeneous composite in which brick units are held together by mortar. Brick units can be made from clay, compressed earth, stone or concrete. Mortar can be lime or a mixture of cement, lime, sand and water in various proportions. Consequently, masonry properties vary from one structure to the next depending on the type of brick units and mortar used. For each type of brick units and mortar, their properties depend upon the properties and composition of the constituents. Other factors contributing to the variability of masonry properties include anisotropy of units, dimension of units, mortar joint width, arrangement of bed-joints and head-joints, arrangement of brick units and workmanship. Nevertheless, bricks and mortar being the most visible components still determine the performance of masonry (Mosalam et al., 2009).

So, masonry is a material that exhibits distinct directional properties of weakness and bed mortar joint orientation should be considered (Samarasinghe et al., 1981).

Experimental analysis of masonry wallets under biaxial stress are suggested by RILEM (RILEM, 1996) that may be used to define a shear failure criterion on a plane shear stress vs compressive stress. Specimens as triplets may be used in shear tests to obtain the shear strength of masonry as suggested by Hendry. Hendry (Hendry, 1978) proposed a non-dimensional equations based on principal tensile stress as:

$$\frac{\tau_u}{f_t} = \sqrt{\left(\frac{\sigma_v}{f_t} + 1\right)}$$

where: f_t=principal tensile stress and τ_u=the average shear stress at failure.

It was found that a wide variety of test results of masonry loaded in combined compression and shear were consistent with the above relationship. Further, it was assumed that the principal tensile stress is affected by the pre-compression normal to the bed-joint and may be represented by:

$$f_t = f_{t0} + \alpha \cdot \sigma_v \tag{2}$$

where: f_{t0} is the value of f_t at zero pre-compression and is equal to the ultimate shear stress τ_0 in a pure shear and α is a coefficient. Substituting the value of f_t from equation (2) into equation (1) gives:

$$\tau_u^2 = \tau_0^2 + \tau_0 \cdot \sigma_v + \alpha \cdot \sigma_v^2 + 2 \cdot \alpha \cdot \tau_0 \cdot \sigma_v + \alpha^2 \cdot \sigma_v^2 \tag{3}$$

The value of α was taken as 0.05 (Hendry, 1978), which reduces the equation (3) to:

$$\tau_u^2 = \tau_0^2 + 1.1 \cdot \tau_0 \cdot \sigma_v + 0.053 \cdot \sigma_v^2 \tag{4}$$

This relationship requires knowledge of only one parameter, i.e. the strength of masonry in pure shear. This can only be done properly in a shear box test.

The authors have decided to do the simple triplet test to obtain shear strength with varying pre-compression considering the HURM (Capozucca and Sinha, 2005). Moreover, since seismic action can be reasonably represented by in-plane horizontal actions (Tomazevic et al., 1996), experimental tests on HURM walls in scale 1/3rd are done to explore the behaviour under both unidirectional and cyclic shear. In laboratory a typical Italian HURM was investigated (Capozucca and Sinha, 2004). A comparison between experimental data – triplet tests on wallets and unidirectional shear and cyclic tests on walls – has been shown with EC6 criterion (European Committee for Standardisation,1995), to obtain further information as reference for restoration.

2. General Considerations

Masonry constructing symbolizes a box-type structural system constructed of vertical structural elements - walls - and horizontal structural elements - floors and roofs. Vertical loads are transmitted from the floors, behaving as horizontal flexural members, to the bearing walls, and from the bearing walls, behaving as vertical compression members, to the foundation system.

As identified, when a building is exposed to an earthquake motion, the inertia force, relative to the masses of the structural system has to be taken into account. Those action affects depend on various variables, such as the mass and the stiffness of the structure and their distribution, the magnitude of the enforced actions, the range of cycles of the earthquake motion, the characteristics of the foundation soil, etc…

Since the ground motion is in general three-directional both vertical and horizontal inertia forces will be behaving on the structure, causing displacements, changing in-time (both in magnitude and in sign), ending in the three-dimensional vibrations of the building.

Horizontal inertia actions are transmitted from the floor structures, which must act as rigid horizontal diaphragms, into the bearing walls, creating shearing and bending influences, and from the bearing walls into the foundation system. Furthermore, due to the distributed mass of wall elements, distributed inertia forces are induced resulting in out-of-plane bending of walls.

Experimental tests and observation of damage modality of real structures have demonstrated that masonry walls are much less resistant to actions perpendicular to their medium plane (out-of-plane actions) than to actions parallel to this plane (in-plane actions). In the first case, the stiffness of the wall is far less than in the other. For a good load bearing behavior, all walls of a masonry building should withstand actions parallel to them, preventing inflection and overturning. This philosophy considers the behavior of the building as a box. The walls must be linked, by stiff constraints to the floor, because the floor should be able to spread the seismic actions between the walls as a function of their stiffness.

It is generally identified that a acceptable seismic behavior is obtained just if out-of-plane collapse is avoided and in-plane strength and deformation capability of walls can be fully exploited.

The seismic vulnerability of masonry buildings relies on several parameters, such as in-plane and/or in-height irregularity, discontinuity of walls/piers along the height of the building, alteration of the initial structural scheme throughout the lifetime of the building, inadequate interventions after previous seismic events, low quality construction type of masonry and/or low quality of materials, inadequate connections among vertical elements or between horizontal and vertical elements, lack of any diaphragm action of horizontal bearing elements, etc.

It has to be mentioned that masonry walls exhibit enhanced vulnerability to out-of-plane bending (low bending moment capacity mobilized under limited imposed inflexion). This pronounced vulnerability is negatively affected by all the above mentioned conditions that limit the box action of buildings, as well as by the poor quality of construction type of masonry and the poor quality of building materials. Needless to say that previous non-repaired damages, lack of maintenance, decay of materials, etc… further aggravate the effects of a seismic event.

The observations of masonry buildings when subjected to earthquakes have shown that the behavior is strongly dependent on how the walls are interconnected and anchored and to floors and roofs. In old structure the unfavourable effect of insufficient anchorage between walls and between walls and floors was often observed. Irregular structural layout in plan, large openings and lack of bearing walls in both directions often caused severe damage or even collapse. A good quality of the connections between floors and walls, between roof and walls and between perpendicular walls is also crucial to reach a good global seismic behavior of the building. Good quality connections will drive the collapse of the construction to a configuration that requires a stronger seismic action, (Borri, 2009).

The analysis concerns a range of peak ground accelerations between 0.08g to 0.40g and masonry tensile strength ranging from 0.05 MPa to 0.55 MPa. Failure results refer to a percentage of the overall failure.

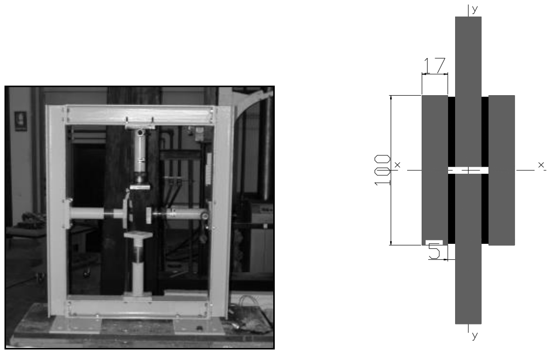
The analyses were run combining the seismic action along one principal direction with 30% of that along the transverse direction.

It should be noted that the structural conditions to be satisfied for the ultimate and the serviceability limit state are defined in terms of internal forces (shear loads and bending moments) and displacements (inter-story drifts), respectively.

3. Experimental Investigation

3.1. Triplet test

RILEM (RILEM, 1996) recommends the use of triplet to obtain the shear strength of masonry. A few full-scale solid bricks became available during the renovation of an 18th century Italian building, hence test specimens were built in 1/3rd scale utilising these.



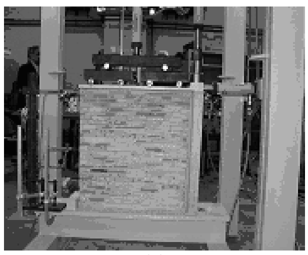
(a) (b)

Figure 1. (a) Experimental apparatus; (b) triplet specimen

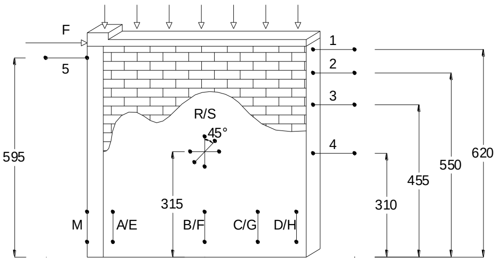
The dimensions of the model bricks were 100x50x17mm obtained from sawing the full-scale bricks. The average compressive strength of the model bricks was 34.3 N/mm². 1:1:5 (cement: lime: sand) mortar was used in the construction of specimens. The average compressive strength of mortar varied from 2.5 to 4.6 N/mm². Figure 1 shows the experimental set up that carried out triplets test. Pre-compression and shear loads were applied by two independent jacks connected to separate pumps. The pre-compression was applied by jack placed horizontally monitored by a load cell. The pre-compression was kept constant during each test.
The shear load was applied by the jack placed vertically as shown in Figure 1, the load was measured by a load cell and the applied shear load was increased at stages till failure.

3.2. Racking tests on HURM wall models

Six single storey structures designated as W1-W6 were built and tested in a special frame (Fig. 2a). A series of preliminary tests were done on small wallet specimens of HURM to obtain the compressive strength and the modulus of elasticity in two orthogonal directions for theoretical analysis (Tab.1). The flange of the T-section was made from mortar. A steel plate was used as a slab on the top of the wall. The flange and the steel plate on the top of wall were glued with epoxy resin. This was done to avoid failure at the interface of slab and the wall. The historic model test structure is shown in Figure 2(a) and (b).



(a)



(b)

Figure 2. (a) Wall model and apparatus; (b) dimensions and point of measure

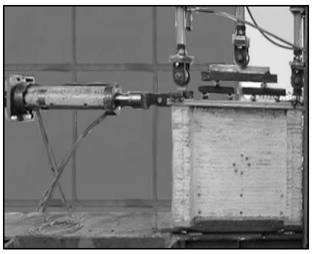
Table 1. Mechanical values of HURM by compression tests

Average values (N/mm^2)	Direction of test	Strength Young's modulus
$\mathbf{f_y}$	Normal to	**13.5**
$\mathbf{E_y}$	mortar joint	**8.76**$\cdot 10^3$
$\mathbf{f_x}$	Parallel to	**10.4**
$\mathbf{E_x}$	mortar joint	**6.90**$\cdot 10^3$

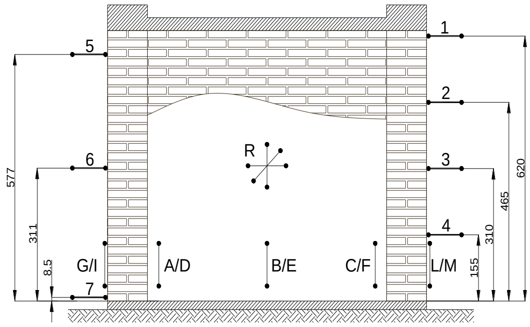
Three load cells measured the applied loads. Before the application of shear load, the pre-compression was applied to both the web and flange of the wall and kept constant throughout the test. The shear load was applied at stages till the failure and deflections and strains were also measured at various levels. In Figure 2(b) the instrumentation to measure deflections in five points (1...5) and strain gauges (M; A/E;B/F;C/G;D/H – Rosetta: R/S) in the middle of panel is shown.

3.3. Cyclic shear tests on HURM wall models

Two single story wall structures designed as C1 and C2 were built and tests under compression and cyclic shear horizontal force. Values of constant compressive stress were 1.15 and 1.50 N/mm^2 respectevily for C1 and C2 wall. The walls were built with same historic clay units in scale 1/3rd of racking tests.



(a)



(b)

Figure 3. (a) Apparatus of cyclic tests; (b) points of measure of wall.

Each wall model was as double T-section to avoid the failure by bending under shear test. Dimensions of web was equal to the walls test under unidirectional shear force: 630x630x50mm. The two flanges were in plane 105x27mm and height equal to 630mm. Vertical pre-compression was applied by three jacks. Horizontal cyclic force was applied by a jack with double effect as shown in Figure 3(a). Cyclic shear load was applied at stages till the failure; deflections and strains were measured at the steps by instrumentation shown in Figure 3(b).

4. Experimental Results

The test results of triplets are shown in Tables 2. A different behaviour between the specimens relative to the pre-compression value may be noted (Capozucca and Sinha, 2005). At low pre-compression the failure was at the interface between brick and mortar while at high pre-compression the brick failed.

Table 2. Results of triplet test

Triplet	**T12**	**T1 T2**	**T3**	**T9**	**T4**	**T10**	**T5 T11**	**T7**	**T6**	**T8**
Specimens	2	4	2	2	2	2	3	1	2	2
σ_v(N/mm^2)	0.0	0.29	0.75	1.0	1.13	1.5	2.0	2.24	2.3	2.87
τ_u(N/mm^2)	0.30	0.46	0.62	0.83	0.90	1.21	1.60	1.62	1.80	2.15

The results of shear tests on the walls are summarised in Table 3. The experimental lateral load-F vs deflection-δ diagrams for point 1 on the top lateral side of wall are shown in the Figure 4.

Table 3. Experimental shear strength by racking tests

Wall	W1	W2	W3	W4	W5	W6
σ_v (N/mm^2)	0.50	0.75	0.30	1.15	2.25	3.00
τ_u (N/mm^2)	0.66	0.68	0.54	1.43	1.90	1.90

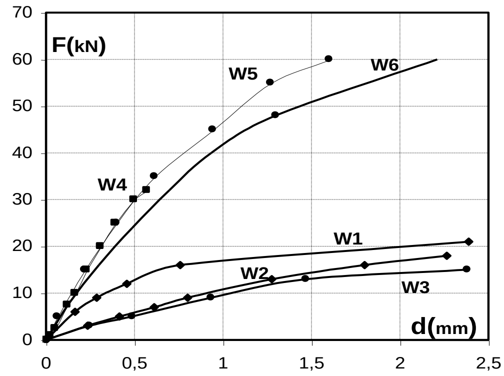


Figure 4. Experimental horizontal force vs displacement at the top of walls

In Table 4 experimental data obtained from cyclic tests on C1 and C2 specimens are shown. Displacements are measured at the top of wall at the ultimate step of loading: in the case of C1 values are referred to deflection towards only one side of wall. Ultimate value of horizontal force was equal to 52.5 kN and 56 kN, respectively, for C1 and C2.

Table 4. Data of displacement - C1 and C2 wall

Load F (kN)	**C1**	0	20	30	40	42.5	45	47.5	50	52.5
point 1. (mm)	VI cycle	0.13	-0.05	-0.14	-0.23	-0.27	-0.36	-0.58	-0.96	-1.68
Load F (kN)	**C2**	0	-20	-40	-50	-55	0	40	50	56
point 1. (mm)	VIII cycle	-0.11	0.60	1.42	1.96	2.80	1.44	-0.38	-0.86	-1.50

In Figure 5 the cyclic diagram horizontal load vs displacement at the top of wall is shown referred to the 5th experimental cycle of shear loading for C1 specimen.

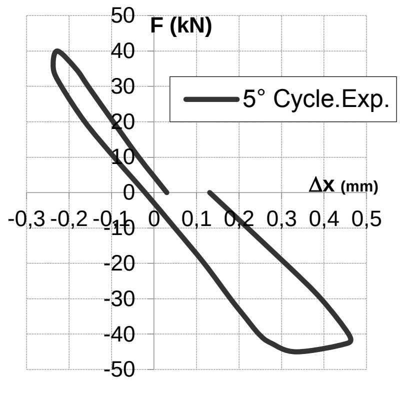
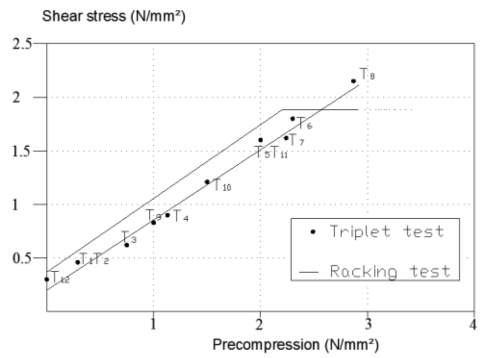


Figure 5. Cyclic experimental diagram at 5th cycle of loading (C1)

4.1. Comparison between experimental results

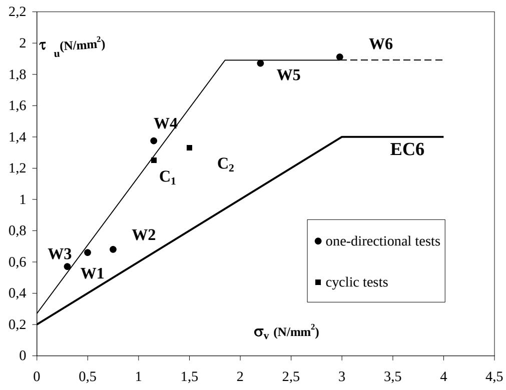
The results of triplet tests are compared in Figure 6 with the results of racking test by unidirectional shear force on single storey structures built earlier in 1/3rd scale.

The historic masonry shear strength as obtained by triplet tests is almost equal to those obtained by single storey structures built with the same material although triplet results are lower. The results of wall test are also compared (Fig.7) with failure criterion of EC6 (European Committee For Standardisation,1995). The EC6 underestimates the shear strength of the historic masonry of this type.



(a)

Figure 6. Comparison of triplet results with racking tests results



(b)

Figure 7. Comparison of exp. results on shear walls with EC6 criterion.

5. Conclusions

As a summary of the damage that has been found in the structure, it includes cracks, wall disconnections, deterioration of mortar or stone, masonry disruption, traces of moisture, wear and damage of linear elements, deterioration of elements.

The first purpose of these analyses was the estimation of the sensitivity of the global structural behavior to changes of some parameters typical of masonry constitutive laws, especially in terms of stiffness in the elastic field and ductility in the post-elastic field.

For compression, a linear behavior is assumed until reaching 50% of the strength, and then the stiffness reduces according to a parabolic law up to the strength.

The experimental investigation has been carried out these main conclusions:

- triplet tests are available to determine the shear strength of historic un-reinforced masonry;

- experimental shear strength values obtained by cyclic tests are almost equal to data determinated by uni-directional wall tests;
- shear values suggested by European code criterion (EC6) underestimates shear strength of HURM.

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