Structural Analysis of Historical Constructions – Jerzy Jasieńko (ed)
©2012 DWE, Wrocław, Poland, ISSN 0860-2395, ISBN 978-83-7125-216-7

EXPERIMENTAL SEISMIC BEHAVIOUR OF WALL-TO-HALF-TIMBERED WALL CONNECTIONS

Susana Moreira[1], *Luís F. Ramos*[2], *Daniel V. Oliveira*[3], *Raquel P. Fernandes*[4], *João Guerreiro*[5], *Paulo B. Lourenço*[6]

ABSTRACT

The out-of-plane failure mechanisms due to seismic action observed in masonry walls of historical buildings are, in many cases, a direct result of poor connections between structural elements, which are incapable of assuring proper load transmission. Therefore, the need to prevent its occurrence raises critical importance around their unstrengthened and strengthened behaviour. A typical example is the connection between external and internal walls or between walls and floors. The wall-to-half-timbered wall connections are representative of connections existent in 'Pombalino' and 'Gaioleiro' buildings, constructed during the post-earthquake (1755) reconstruction of downtown Lisbon. In order to better understand the seismic performance of masonry wall-to-half-timbered wall connections and to increase information found in literature, a series of 5 tests were carried out in representative real scale specimens in which injected anchors were installed in an irregular stone masonry wall in order to improve the seismic response of the connection. Monotonic and cyclic pull-out tests were performed on representative connections in order to assess their performance and allow their characterization. Parameters considered include: failure mode, hysteretic curve, strength degradation and total energy. The results contribute to the existent body of knowledge on the topic, by expanding the experimental database. For further development, the experimental data allows the calibration of representative numerical models, enabling parametric studies of material properties and formulation of backbone curves. This data allows idealized behaviour to be established and utilized for the construction of global building models.

Keywords: *Wall-to-timber frame connection, Injected anchor, Pull-out test, 'Gaioleiro'*

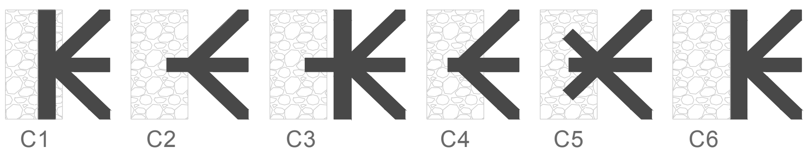
1. INTRODUCTION

Recent seismic events brought to light the importance of effective connections between structural elements. Deficient connections disturb the energy path, influencing the seismic response of horizontal and vertical structural elements, and consequently the global structural performance. Thus, it is of great interest to investigate the role of connections – both of traditional and innovative type – between structural elements in the dynamic behaviour of masonry buildings.

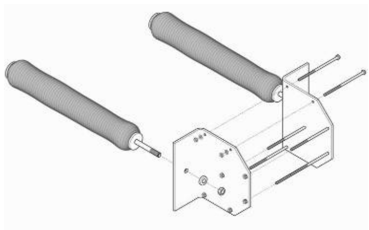
The out-of-plane mechanism of walls has been identified as one of the main failure modes in masonry buildings when subjected to seismic actions, as observed in Azores 1998, L'Aquila 2009 and Christchurch 2011. This mechanism is especially seen in presence of weak connections. It is of critical importance to study the behaviour of structural connections to prevent or minimize out-of-plane failures with the aim of saving human lives and minimizing economical losses. This research focuses particularly in wall-to-half-timbered wall connections (WT) found in buildings dating from the

[1] PhD candidate, ISISE, Dep. of Civil Engineering, University of Minho, smoreira@civil.uminho.pt
[2] Assistant Professor, ISISE, Dep. of Civil Engineering, University of Minho, lramos@civil.uminho.pt
[3] Associate Professor, ISISE, Dep. of Civil Engineering, University of Minho, danvco@civil.uminho.pt
[4] MSc., Grupo STAP, raquelpaula@stap.pt
[5] PhD candidate, DE Civil, Instituto Superior Técnico, jg@civil.ist.utl.pt
[6] Professor, ISISE, Dep. of Civil Engineering, University of Minho, pbl@civil.uminho.pt

reconstruction of downtown Lisbon after the 1755 earthquake. Half-timbered walls are part of a flexible timber cage, the so-called 'gaiola', which was an engineering innovation designed to decrease seismic vulnerability [1]. A three-dimensional oak timber cage – consisting of vertical, horizontal, and diagonal members in a St. Andrews cross pattern – increases resistance to horizontal loading and effectively dissipates energy. The half-timbered walls are then completed with brick or rubble masonry infill. External walls and building partition walls are rubble masonry with constant thickness, usually 0.50 m and 0.60 m respectively, and timber elements from the cage are embedded close to the internal face [2]. This description corresponds to the original construction, the 'Pombalino' building, which in time evolved to a much poorer type of structure, the 'Gaioleiro' building, with higher seismic vulnerability. Both types of buildings and their variations use masonry (stone and brick) and timber as main structural materials. However, as constuction continued in the decades after the earthquake, the concept of the three-dimensional cage and proper connections degraded to the point of inexistance. Also, the thickness of masonry walls stopped being constant and started decreasing along the height of the building, reaching values at the last floor of about 0.30 m [3].
The original concept was to add a secondary flexible structure within the outer walls, allowing the overturning of the masonry external walls while avoiding total collapse. This design can be effective for buildings up to two storeys, as it was observed for buildings struck by the 1998 Azores earthquake, but many of uncertainties are raised for taller buildings [4]. Different types of floor-to-half-timbered wall connections have been described in literature, as shown in Fig. 1a. The connection varies according to the amount of timber elements inside the wall and their anchorage length, relying mainly on friction to ensure the connection (see Fig. 1a, from C1 to C5). The worst-case scenario occurs when the half-timber wall leans against the masonry wall (see Fig. 1a, C6) leaving only the pavement beams to maintain the continuity of force distribution. Therefore, there is the need to promote good connection between these two structural elements, by means of strengthening.



(a)



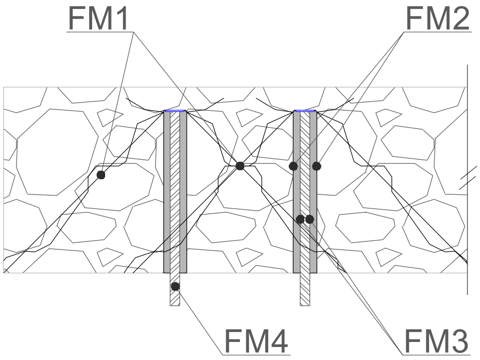
(b)

Fig. 1 Different type of wall-to-half-timbered wall connections: a) six types of connections (addapted from [5]), (b) adopted strengthening solution [6]

2. TEST SET-UP AND PROCEDURE

The specimens intend to recreate as much as possible the aspects of building construction from post earthquake Lisbon. By researching the existent literature and conducting a survey of these buildings, it was possible to gather enough information to describe the different elements in terms of material, geometry and typology. The construction technique chosen for the specimens aimed to achieve the same typologies as those of the original structure. As a result, rubble masonry panels were built using limestone from the surroundings of Lisbon and a mortar with a ratio of 1:3:10:6 (cement: hydraulic lime: sand type 1: sand type 2). This mortar ratio provided a compressive strength of 1.27 MPa (28 days) and it was chosen after performing several tests on four different mixes, assessing their workability and compressive strength at 7, 14 and 28 days. The objective was to achieve a low compressive strength mortar, between 1.0 MPa and 1.5 MPa, without using air lime.
Since connection C6 of Fig. 1a corresponds to the most unfavourable situation, the two specimens were designed to address this configuration. Therefore, the specimens were designed without the half-timbered wall, focusing exclusively on the strengthening solution. Through the consideration of existent literature, failure modes for the strengthening solution when subjected to a cyclic horizontal load were discussed. Along with the considerations of laboratory limitations, a test configuration and specimen dimensions were chosen (see Fig. 2). The failure modes expected are: cone pull-out formation (FM1), sliding at the interface between the borehole surface and the sleeve with injected grout (FM2), sliding at the interface between the steel anchor and the injected grout (FM3) and yielding of the steel anchor (FM4). The specimens have a rectangular shape and are 2000 mm long,

400 mm thick and 1600 mm high (see Fig. 3) The wall thickness was representative of a 4th floor of a ‘Gaioleiro’ building.
The strengthening solution adopted consists of two injected anchors (I) placed parallel to each other in the masonry wall and connected to the half-timbered wall by means of steel angles. Therefore, four pairs of injected anchors were installed in each of the two wall specimens, resulting in a total of eight tests.



(a)

(b)

Fig. 2 Set-up design: (a) Main failure modes expected; (b) specimen configuration

The rubble masonry wall was hand constructed using only some wires as guides for the prescribed dimensions. The complete wall was loaded vertically under compression after 28 days in order to achieve the stress state caused by the permanent loads expected for a 4th floor, shown in Fig. 2b, so that the 50 mm boreholes could be done. The distance between parallel anchors was determined considering a half-timbered wall 120 mm thick and also constructive distances of the steel angle connecting both walls.

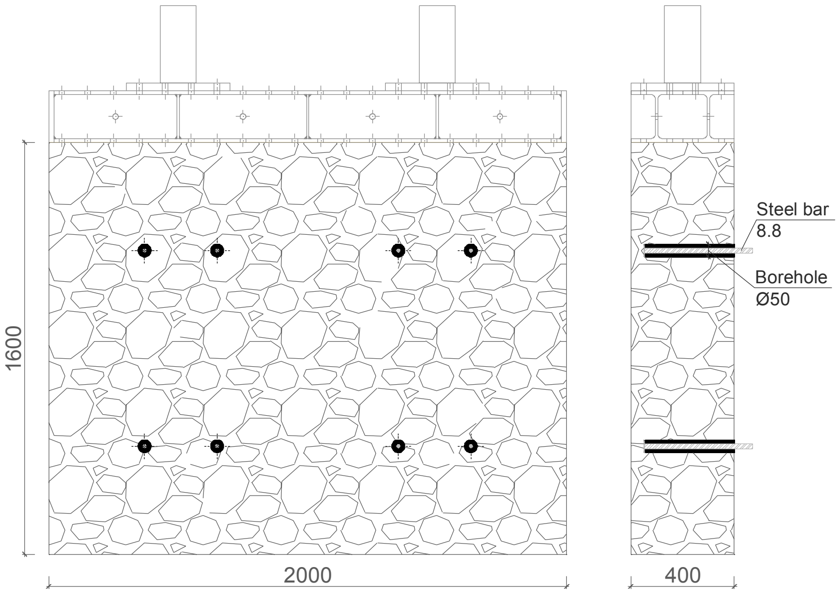


Fig. 3 Specimen configuration

To control the formation, behaviour and influence of the different failure modes on the final result, a set of 19 LVDTs and 8 strain gauges were used per test. The distribution of LVDTs followed a quadrangular grid of 150 mm around the two parallel anchors, with LVDTs measuring out-of-plane displacements of the wall and the steel bars. From the total of 8 strain gauges, one bar was instrumented with 5 and the other with the remaining 3.
Considering the laboratory limitations in terms of space as well as the size of specimens, it was possible to develop an auto-balanced set-up capable of redirecting the pull-out force back to the specimen, as shown in Fig. 4. The pull-out load, which intends to recreate the main seismic action, was applied perpendicular to the wall in order to activate the tensile capacity of the injected anchors. A hinge was used between the actuator and the specimen to accommodate small deformations. In order to perform cyclic tests, the set-up had to be anchored to the masonry wall.

As previously stated, a distributed vertical load was applied on the top of the wall to simulate the effect of permanent loads on the structure. This was achieved by placing HE200B steel profiles on top of the wall, which distributed the load provided by four hydraulic cylinders compressed against a reaction slab (see Fig. 4). The distributed vertical load was kept constant during the entire test and equal to 0.2 MPa, by using manual control. A metallic clamp was designed for this connection, rigid enough to apply the force to the specimen without interfering on the test results. The load was applied directly on the anchors, pulling the two parallel injected anchors at the same time. The monotonic tests were carried out under displacement control at a rate of 10 µm/s. From these tests, it was possible to determine a cyclic procedure comprising two repetitions per cycle, with a range of velocities between 10 and 40 µm/s. The stopping criteria adopted were: a 50% decrease in load or the propagation of cracks beyond the expected area of damage.

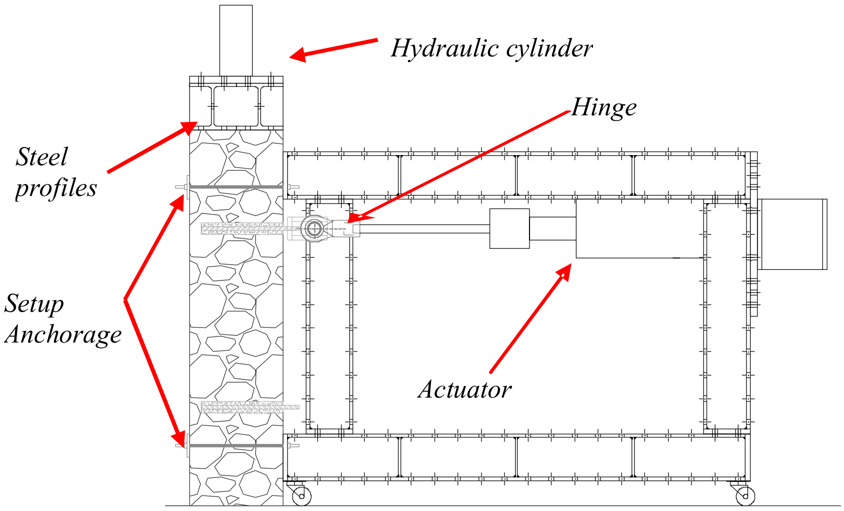


Fig. 4 Set-up for monotonic and cyclic pull-out tests

3. MATERIALS AND SAMPLES

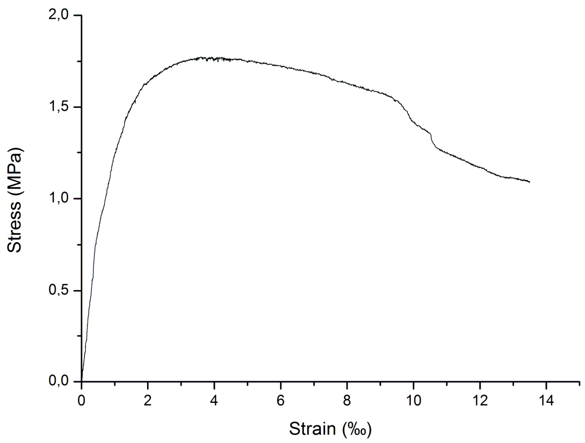
Experimental tests and subsequent numerical modelling require extensive material characterization, since material properties have a major influence on the behaviour of specimens and thus on the results. So far, Only mortar and masonry samples have the compressive strength and elasticity modulus characterized, based on EN 1015-11 and EN 1052-1 specifications, respectively [7][8]. Cylindrical mortar samples were collected during the construction of the specimens and tested at the ages of 28 days (to compare with a study performed initially) and 90 days. The results of the compressive strength tests performed on mortar specimens are presented in Table 1 for 28 days and 90 days.

Table 1 Results of the compressive strength of the mortar tested at 28 and 90 days

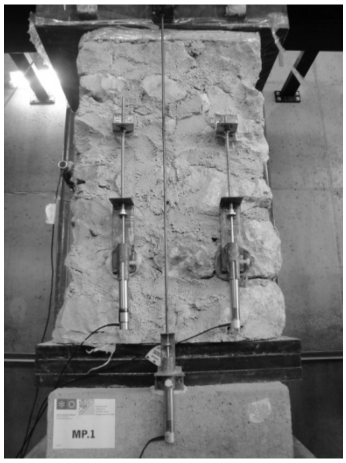
Specimen	$f_{c\,(28\ days)}$ (MPa)	$f_{c\,(90\ days)}$ (MPa)
WT.40.I.1	1.14	1.06
WT.40.I.2	1.39	1.36
Average (MPa)	1.26	1.21
CoV (%)	14.5	17.3

Table 1 shows that the average compressive strength at 28 days is basically equal to the value found before the construction of the walls (1.27 MPa), which demonstrates the rigour put in the preparation of the mortar. It also shows that the compressive strength is hardly affected by the testing age, thus allowing the testing of specimens at different ages without major consequences in terms of mortar hardening.
The masonry prisms were defined according to EN 1052-1 [8], with dimensions of 0.40 × 0.50 × × 0.80 m^3. First, a monotonic test was performed to observe failure modes and determine the

compressive strength, which was used afterwards as a boundary to perform cyclic compressive tests and consequently determine the elastic modulus. The stress-strain diagram of one of the prisms tested is presented in Fig. 5, as well as a detail of the test instrumentation. The average compressive strength of 2 prisms was around 1.60 MPa. Values found in literature range between 0.50 and 1.50 MPa, thus placing the tested specimens slightly above the interval. Specimens reached an elastic modulus approximately equal to 1000 MPa.



(a)



(b)

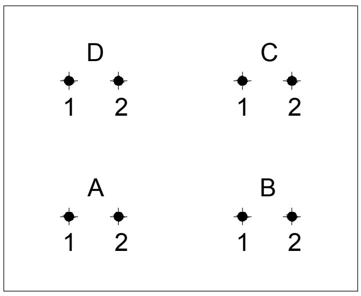
Fig. 5 Compressive test: (a) Stress-strain diagram of a masonry prism, (b) instrumentation detail

4. RESULTS

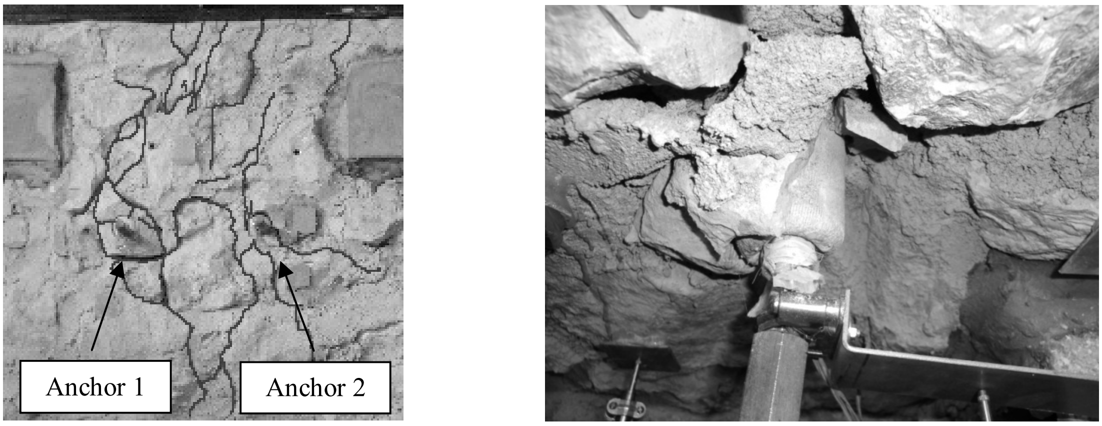
To date, only 5 tests were performed (2 monotonic and 3 cyclic). Some of the characteristics of the tests are presented in Table 2. One of the main objectives was to characterize the failure modes related with the masonry wall, so high class steel was chosen for the steel bars, in order to prevent steel yielding. As one can observe in Table 2, cone pull-out formation was the recurrent failure mode. Nevertheless, total displacement is a combination of the masonry shear cone with sliding of the interfaces (see Fig. 6).

Table 2 Characterization of the tests performed

Specimen	**Type of test**	**F_{max} (kN)**	**$d_{(Fmax)}$ (mm)**	**Failure mode**
WT.40.I.1A	Monotonic	76.93	9.16	FM1 (Anchor1)
WT.40.I.2A	Cyclic	107.21	8.52	FM1 (Anchor 2)
WT.40.I.2B	Cyclic	104.93	10.21	FM1 (Anchor 2)
WT.40.I.2C	Cyclic	74.95	7.73	FM1 (Anchor 2)
WT.40.I.2D	Monotonic	74.26	7.03	FM1 (Anchor 2)
Average	-	87.66	8.53	-
CoV (%)	-	19.2	14.5	-



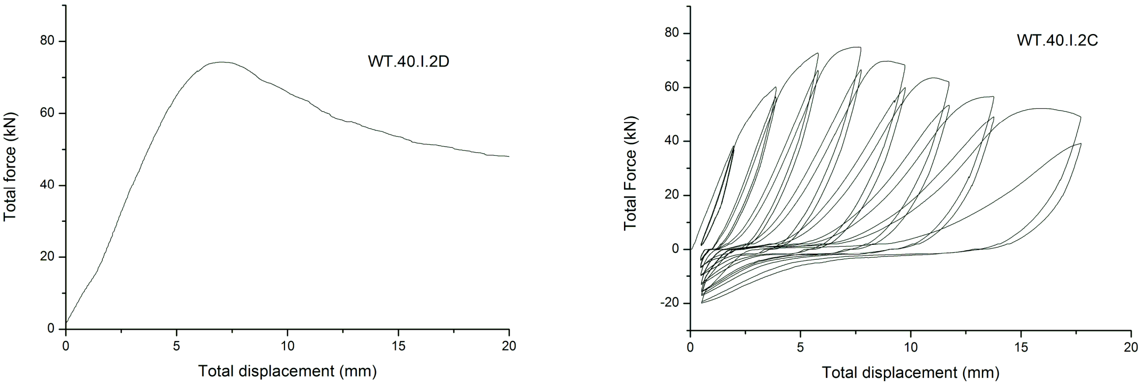
Considering all the tests, average values of 87.66 kN and 8.53 mm were obtained for the force and displacement, respectively, at the peak load. The coefficient of variation of 19% can be explained by the scatter present in this type of masonry. Additionally, the much more rigid boundary conditions at the base of the specimen might have positively influenced the lower anchors.
Shear failure of the masonry was always characterized by generalized formation of cracks around the two anchors and in some cases detachment of mortar or stones. Minor cracking was also found close to the boreholes at the end of the anchors. Further tests will allow the characterization of the contact area of the borehole, which is expected to have major importance on adhesion.



(a) (b)

Fig. 6 Failure modes: (a) formation of shear cone on WT.40.I.1C, (b) sliding and formation of shear cone

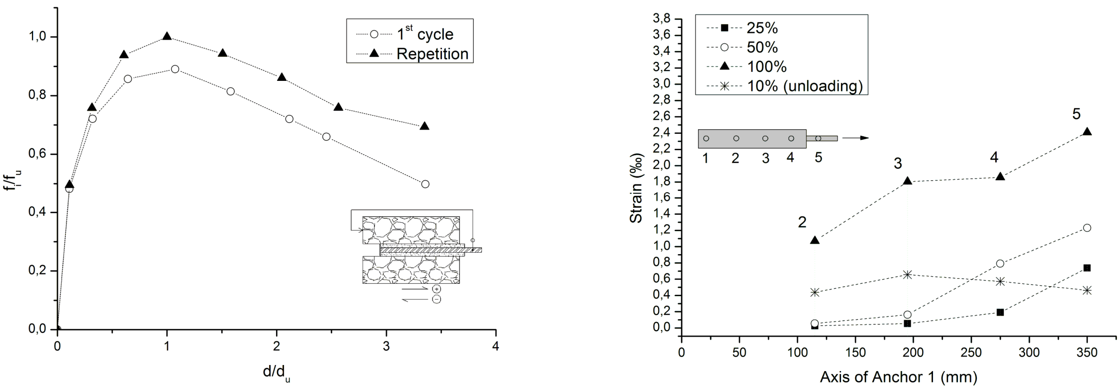
Envelope curves obtained for cyclic tests show great resamblence with the monotonic force-displacement curves, see Fig. 7, demonstrating high increase of stiffness up to 10 mm, followed by softening behaviour. For the cyclic tests, there is a visible decrease of strength between cycles, which is made clear in Fig. 8a. It's interesting to observe how stiffness of the repited cyle is similar to the one of the following first cyle.



(a) (b)

Fig. 7 Force-displacements curves of the strengthening solution (a) montonic test and (b) cyclic test

The contribution of each anchor to the final outcome was assessed by means of the strain gauges placed along the steel anchor. By using this information, it was possible to establish the amount of force taken by each anchor, and also to obtain profiles of strain distributions. As expected, strain increases from the free end to the loaded end of the anchor, assuming approximately an exponential behavior for lower forces (25%) and a linear distribution for higher values of applied force (100%), see Fig. 8b.



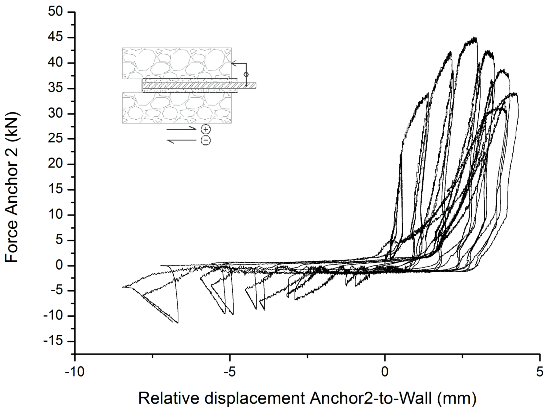
(a) (b)

Fig. 8 Cyclic pull-out of WT.40.I.2C: (a) force degradation for the relative displacement between top of Anchor 2 and back wall; (b) strain distribution of Anchor 1

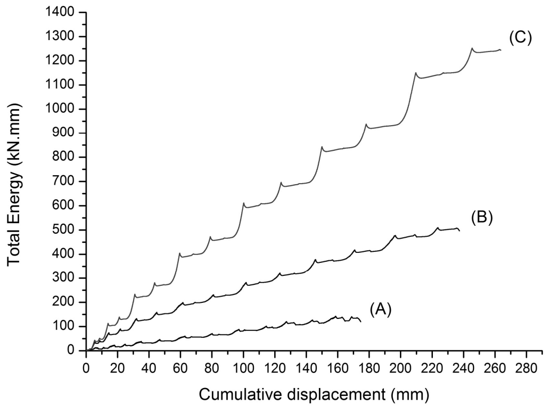
The LVDTs measured out-of-plane absolute displacements, allowing the determination of relative displacements between different parts of the specimen. The following relative displacements were considered to be of major importance due to their contribution to the failure mode and energy dissipation:

(A) loaded end to free end of the anchor;
(B) loaded end of the anchor to surrounding points in the front side of the wall;
(C) loaded end of the anchor to the back side of the wall.

The first relative displacement (A) characterizes the contribution of the anchor; the second (B) gives information about the contribution of the anchor plus the anchoring system, while the third relative displacement (C) accounts for all contributions (anchor, anchoring system and wall). Fig. 9a shows one of the force-displacement curves obtained for the relative displacement B (loaded end of anchor 2 to the front side of the wall) for specimen WT.40.I.2C, considering the force distributed to this anchor.



(a)



(b)

Fig. 9 Cyclic pull-out of WT.40.I.2C: (a) Force-displacement curve for the relative displacement between anchor 2 and front side wall; (b) different contributions of the cumulative energy of anchor 2

To better understand the contribution of each portion, their total energy was determined (see Fig. 9b). As expected, relative displacement C presents higher values since it comprises all the energy dissipated. It is also confirmed that the formation of the shear cone (difference between relative displacements C and B) is responsible for most of the dissipated energy, confirming it as main failure mode. The steel bar itself (relative displacement A) represents a lower contribution when compared to the shear cone, probably as a consequence of the higher grade of the steel used.

5. CONCLUSIONS

Tests were successfully executed, since many of the initial considerations were confirmed, but also new questions were raised. As expected, shear failure of masonry was obtained and a range of maximum pull-out forces was established, approximately from 75 to 110 kN. Nevertheless, displacements due to sliding of the interfaces contributed to the final damage distribution.
The strain profiles along the steel anchor allowed to determine the distribution of forces on both anchors and to analyze the behavior of the individual anchor, which is not visible during the test. It was observed an exponential distribution for lower forces and a linear distribution for higher values.
The analysis of the total energy of each anchor showed that the formation of the masonry shear cone is responsible for a great portion of the energy released and also enable the quantification of the contribution of each portion to the total displacement.
This research adds critical experimental information about wall-to-half-timbered wall connections, which allows a better understanding of the subject and also creates the base to develop numerical models. Further work is still required, as there are many parameters to be study, such as the role of the different interfaces and the influence of masonry properties.

ACKNOWLEDGEMENTS

This work was partially funded by project FP7-ENV-2009-1-244123-NIKER of the 7th Framework Programme of the European Commission, which is gratefully acknowledged. Authors would like to thank the technical staff of Civil Engineering laboratory from Universidade do Minho for the assistance provided.

REFERENCES

[1] Ramos, L. F., Lourenço, P. B. (2004) Modelling and vulnerability of historical city centers in seismic areas: a case study in Lisbon. *Engineering Structures* 26: 1295-1310.
[2] Mascarenhas, J. (2005). *Sistemas de Construção V – O Edifício de Rendimento da Baixa Pombalina. Materiais Básicos: o Vidro – 2.ª edição revista e actualizada (2ª ed.)*. Lisboa, Livros Horizonte.
[3] Appleton, J. (2005). *Reabilitação de edifícios Gaioleiros*. Lisboa, Edições Orion.
[4] Bento, R., Lopes, M., Cardoso, R. (2005). Seismic evaluation of old masonry buildings. Part II: Analysis of strengthening solutions for a case study. *Engineering Structures* 27: 2014-2023.
[5] Cardoso, R. Vulnerabilidade sísmica de estruturas antigas de alvenaria – Aplicação a um edifício Pombalino. (Mestrado), UTL/IST, Lisboa.
[6] Cóias, V. (2007). *Reabilitação estrutural de edifícios antigos.* Lisboa, ARGUMENTUM GECoRPA: GECoRPA.
[7] EN 1015-11, Determination of flexural and compressive strength of hardened mortar. (1999).
[8] EN 1052-1, Methods of test for masonry – Part 1: Determination of compressive strength of masonry, (1999).