Traditional timber frame walls: mechanical behavior analysis and retrofitting

G. Vasconcelos, E. Poletti

Abstract

Timber frame construction is characteristic of several historic city centres as well as of vernacular architecture in several countries around the world, either motivated by the availability of materials and construction traditions or by the need of reducing the seismic vulnerability of existing buildings, namely in south European countries. From past earthquakes, it has been seen that timber frame construction can be viewed as an interesting technology as it has exhibited a very reasonable behaviour when compared to other traditional material such as masonry.

This chapter provides an overview of the main insights on the seismic performance of timber frame buildings from the evidences of past earthquakes and provides the main results of recent research focused on the in-plane cyclic behavior of timber frame walls with distinct geometrical configurations. Additionally, the main seismic performance indexes of timber frame walls, both unreinforced and retrofitted, are presented and discussed in detail.

Keywords: Timber frame walls, experimental analysis, in-plane behavior, seismic performance, retrofitting

Graça Vasconcelos
ISISE, Department of Civil Engineering, University of Minho,
Azurém, P-4800-058 Guimarães, Portugal.
Tel: +351 253 510 200, Fax: +351 253 510 217

Elisa Poletti
NCG, Faculty of Engineering, University of Nottingham
University Park, Nottingham, NG7 2RD, UK

Contents

1 Introduction

Masonry and timber are materials used since ancient times in construction. Masonry buildings constitute an important percentage of the existing buildings and actions for their preservation should be taken since a large part of historical buildings are actually in masonry. A drawback on the use of unreinforced masonry is the low resistance to tensile stresses, leading often to an inadequate behaviour under seismic actions. A historical construction solution to improve the mechanical behaviour of ancient masonry adopted in different locations at different times, namely in seismic regions, has been the reinforcement of masonry with timber (Touliatos, 2005; Vintzileou, 2008), namely at the level of the timber floors and at additional levels (e.g. door and window lintels) by adding a ring timber beam aiming at improving the confining effect of masonry walls and thus improving its out-of-plane behaviour, adding also a more global seismic resistance to the masonry building by promoting the known box behaviour.

The particular case of traditional timber frame walls constitute an example of an important structural element of many buildings and are usually composed of vertical posts and horizontal beams with bracing diagonal elements. In Portugal, timber frame walls, known as *frontal* walls, are usually part of Pombalino buildings, which were introduced by the Marquis of Pombal, who was responsible for the reconstruction of Downtown Lisbon after the great earthquake of 1755, which partially destroyed the city. The timber-framed walls are connected to the external masonry walls by means of the timber floor beams, which are connected both to the timber-framed and to the external masonry walls (Mascarenhas, 2004) and can be beneficial to reduce the out-of-plane vulnerability of the masonry walls. The timber frame walls are also identified in several countries particularly in local vernacular architecture, due to the low cost of such structures composed of timber and several infill materials from brick and stone masonry to mud and cane.

Given the increasing interest of the research community to this structural system, it is important to promote the discussion of the main findings that can contribute to the advance on the knowledge of the mechanical behaviour of timber frame buildings to seismic actions.

Therefore, this chapter intends: (1) to give an overview of the different solutions of timber frame structures in different countries with special focus on the frontal walls characteristic of Pombalino buildings; (2) to describe some examples of the seismic behaviour of timber frame buildings in past earthquakes; (3) to summarize the experimental research carried out in recent years on the analysis of the behaviour to in-plane cyclic loading; (4) to give an overview on the possibilities of retrofitting timber frame walls and summarize some seismic indicator to assess their performance.

2 A brief overview

2.1 Traditional timber frame construction

The origin of timber frame structures probably goes back to the Roman Empire, as in archaeological sites half-timbered houses were found and were referred to as Opus Craticium by Vitruvius (Lagenbach, 2009). But timber was used in masonry walls even in previous cultures. According to (Tsakanika-Theohari, 2008; Tampone, 1996) in the Minoan palaces in Knossos and Crete, timber elements were used to reinforce the masonry. Half-timbered constructions later spread not only throughout Europe, such as Portugal (edifícios pombalinos), Italy (casa baraccata), Germany (fachwerk), Greece, France (colombages or pan de bois), Scandinavia, United Kingdom (half-timber), Spain (entramados) etc., but also in India (dhaji-dewari) and Turkey (himis) (Lagenbach, 2009; Tampone, 1996). In each country, different typologies were used, but the common idea is that the timber frame can resist to tension, contrary to masonry, which resists to compression, thus providing a better resistance to horizontal loads. Besides, the timber elements are viewed as a sort of confinement to the masonry structure, improving the mechanical properties to shear loads. In general, the cross section of the timber elements in the distinct case studies is very similar (approximately 10x12cm).

Timber frame buildings were common all over Greece in different periods, as reported by many authors (Coias, 2007; Makarios and Demosthenous, 2006; Vintzileou et al., 2007). Examples of this system are the monastic buildings in Meteora and Mont Athos, the post byzantine (Ottoman period) buildings in Central and Northern Greece and the traditional buildings in the island of Lefkas. The latter buildings consisted of a stone masonry ground floor plus one or two timber-framed masonry storeys (Figure), which represents a common disposition in timber frame buildings. Another innovation present in these buildings is the existence, at the ground floor, of timber columns stiffened by angles that constituted a secondary load bearing system in case of failure of the masonry walls, since they were connected to the timber-framed structure of the upper storeys (Tampone, 1996).

In Germany, *fachwerk* construction was very popular and several examples of timber frame constructions are present all over the country. Different timber frame styles can be found, characterized by a varying number of storeys and geometries of the timber frame. In Germany, this construction system was introduced in the 7th century and it flourished particularly in the 16th and 17th century. Three main styles can be recognized (Alemannic, Lower Saxonian and Franconian), differentiating mainly in regards of the spacing between the elements, dimensions and disposition of the framing. An example of the German constructions is presented in the lexicon by Otto Lueger (Lueger, 1894).

Another example of timber frame construction is the *casa baraccata* in Italy. After the 1783 earthquake in Calabria, authorities adopted construction methods similar to those imposed some decades before in Lisbon. The same construction technique, with slight changes, was also adopted after the Messina earthquake in 1908. In particular, Vivenzio proposed a 3-storey building with a timber skeleton aiming at reinforcing the external masonry walls, avoiding their premature out-of-plane collapse. The timber-framed walls constituted the internal shear walls, presenting a bracing system of S. Andrew's crosses, similar to what can be found in Lisbon (Copani, 2007). A difference to the Portuguese solution is the continuity of the vertical timber posts from the foundation to the roof, being anchored in the foundation (especially in the buildings built after 1908) (Tobriner, 1997).

Similar houses were also found in India and Turkey. Turkey is a prone seismic zone and is frequently subjected to strong earthquakes, meaning that the buildings need to be able to resist seismic actions. Besides, Turkey has an abundance of wood, as well as stone and clay, which promoted the growth of timber frame structures. The typical timber frame construction used in the upper floors is called *himis* and it is typically constituted of a timber frame filled with rubble or brick masonry (Gulkan and Langenbach, 2004) (Figure 1b). An alternative to masonry infill can be found in *bagdadi* constructions, where timber laths are used as infill material. This led to lightweight, seismic resistant, economical structures, but were more disposed to decay (Gulkan and Langenbach, 2004). Among India's traditional buildings, a half-timbered construction typology can be distinguished in the *dhajji-dewari* (patchwork quilt wall) system, which is a braced timber frame with masonry infill, frequently used for the upper storeys of buildings (Figure 1 c).

Timber frame construction has also been used in South America. In Peru, for example, the *quincha* presents a one-storey timber frame made of round or square wood (bamboo is often used) and filled with canes covered with earth and gypsum (Gulkan and Langenbach, 2004). This type of construction was for example proposed by Peruvian experts for the reconstruction of Haiti after the severe earthquake of 2010 (Gulhan and Guney, 2000). One of the few buildings which survived the earthquake was actually built with the construction system of *quincha*. The reconstruction proposed is being done with the improved *quincha*. The posts are grounded in a concrete foundation, the infill consists of canes covered with clay and mud and, once dried, everything is covered with a cement plaster.

2.1.1 Geometrical characterization and typologies of timber frame walls

Though the general disposition and geometry of timber frame walls is similar in most countries, variability exists in terms of timber species, cross-section of the timber elements, type of connection, type of infill, spacing between posts and presence of bracing elements.

Usually, mechanical connections such as half-lap joints or adopted were adopted, particularly when timber frames were specifically built with seismic resistant purposes (see the examples of Portugal and Lefkas, Figure 2). Generally, the diagonals were connected with simple notched connections or were simply nailed (Figure 2c).

Similarly, bracings are not always regularly used, except for Lisbon. In Lefkas they are mainly concentrated at corners. In the casa baraccata In northern Europe they are present at most at the corners of the buildings or are sometimes used as decorative elements.

Bracings disposition varies greatly, Figure 2a, as well as the dimensions of bracings and their regularity. If in frontal walls they are generally sturdy, thinner elements are used in dhajji dewari construction or even in traditional buildings found in Greece (as little as 25mm in the case of dhajji dewari). In himis construction a great number of timber frame wall typologies can be identified (Aktas, 2011), with a great variability in terms of spacing of the posts (varying from 15 to 60cm), while diagonal elements are present only at corners. Also in dhajji-dewari walls the disposition of the bracings varies greatly, going from a regular disposition (similar to that of frontal walls) to a highly irregular one, with random layouts based on the builder’s skill and the available timber elements (Langenbach, 2009).

The different geometry and connection types will then lead to different dissipative capacity in the walls, as demonstrated by the experimental results that will be presented below.

2.2 *The case of Pombalino buildings in Portugal in more detail*

In Portugal, typical half-timbered structures are known as Pombalino buildings, which are old masonry buildings constructed after the 1755 Lisbon earthquake, which destroyed Downtown Lisbon. The new buildings took their name from the prime minister of the time, the Marquis of Pombal, who encouraged the reconstruction of the city. A Pombalino building is characterized by external masonry walls up to 5 storeys. The ground floor consists of stone masonry columns supporting stone arches and clay brickwork vaults and above the first floor develops an internal timber structure, named *gaiola* (cage), see Figure 3. The *gaiola* consists of horizontal, vertical and diagonal bracing members, forming a three-dimensional braced timber structure. These timber-framed walls are filled with rubble brick or rubble stone masonry and act as shear walls. The length of a typical building is 8 to 16m and the width is about 10m. The internal walls of the *gaiola* (paredes em frontal) may have different geometries in terms of cell dimensions and number of elements, as it depends greatly on the available space and the manufacturer's customs (Mascarenhas, 2004). The main horizontal and vertical elements are reasonably long, whereas the diagonal ones are very short. The timber elements are notched together or connected by nails or metal ties. Traditional connections used for the timber elements varied and could be mortise and tenon, half-lap, dovetail connections, and other types of notched connections. A wide range of sectional dimensions can be found in the elements: the diagonal members are usually smaller (10x10cm or 10x8cm), whilst the vertical studs and horizontal members are bigger (usually 12x10, 12x15cm and 14 x10cm or 15x13, 10x13 and 10 x 10cm). The sectional dimensions of the elements are usually bigger for the lower storeys, decreasing progressively with the height of the building. The frontal walls have a width of 15-20cm, with a grout thickness covering the masonry infill of about 2.5cm but it could vary up to 5cm (Mascarenhas, 2004; Jurina and Righetti, 2007). The frontal walls act as shear walls in the building but can be considered also as partition walls. The peculiarity of this type of building is that under a seismic event, it is admissible that the heavy masonry of the façades falls down, as well as the tiles of the roof and the plaster of the inner walls, but the timber skeleton should remain intact, keeping the building standing. It should be stressed that if the connections between the external masonry piers and the internal timber-framed walls are adequate, the out-of-plane collapse mechanisms of the external façades is also minimized. Some timber elements can be found in the external walls to promote the connection between the *gaiola* and the external masonry walls (Pinho, 2000; França, 1983).

3 Seismic performance of traditional timber frame buildings

From the world seismic hazard map, it is concluded that some timber frame construction in the different countries and regions mentioned in the previous sections was not motivated by seismic reasons. Timber frame construction has been used in countries such as Germany, England and Scandinavia, which present low seismic activity, being motivated by tradition and availability of the raw materials used in the construction system, namely wood. These timber frame buildings were built to carry mainly gravitational loads. However, there are examples where timber frame structures are part of the local seismic culture, as they were introduced after the occurrence of strong earthquakes. The case of Pombalino buildings with a timber frame cage in Portugal, the Casa Baraccata in southern Italy and traditional timber frame buildings in Lefkas Island in Greece are examples of the local seismic culture.

However, even if these countries are characterized as medium to high seismic zones it is also true that timber frame buildings are not spread all over the countries and appear to be local. In Portugal, examples of timber frame walls can be seen from north (low seismicity) to south (high seismicity), namely in south-eastern (Vila Real de Santo António). On the other hand, there are examples of traditional timber frame buildings which possibly were not built based on seismic motivations but have been forced to perform under these extreme natural events, as is the case of himis construction in Turkey (Aktas, 2011) or timber-laced masonry buildings and the dhajji dewari timber frame construction in India. In fact, based on the analysis carried out on damage state of traditional timber frame buildings located in high prone seismic regions after important seismic events, it has been seen that very reasonable behaviour is exhibited by this structural system in distinct countries with high seismicity (Langenbach, 2007). Timber frame structures combine the best features of masonry and timber, offering a better overall behaviour of the buildings under seismic actions. With this respect, it is important to consider that the state of conservation of the traditional timber frame buildings can influence their seismic behaviour.

After the strong earthquake in 2003 in Lefkas, a high prone seismic region, it was observed that in spite of damages developed in the traditional buildings, they were not so severe as the ones observed in reinforced concrete buildings and no collapse of traditional buildings was recorded. The damages observed included vertical and diagonal cracks and shear cracks at the interface between timber frame and masonry infill, which in a certain extent promoted the out-of-plane collapse of infill (Figure 3a), crushing of the infill masonry. Almost no damage was found in the wood elements of the timber frame (Vintzileou et al. 2007). Another example where the efficiency of timber frame structures was actually tested consists of the traditional timber frame buildings in Turkey, already described. Turkey is frequently exposed to severe earthquakes being one of the few countries with the shortest return period in earthquakes causing often loss of lives (Gulhan and Guney, 2000). Different authors have pointed out the reasonable earthquake resistance of timber frame buildings, especially with comparison to other structural systems such as masonry or reinforced concrete structures (Figure 3b), namely during the 1894 Istanbul earthquake, 1970 Gediz earthquake and more recent 1999 Marara (Kocaeli) earthquake (Gulhan and Guney, 2000). In Kocaeli-Gölcük, in the Sehitler distric, 51% of the buildings are RC buildings (up to 7 storeys), while the rest are traditional (either half-timbered or timber-laced masonry or plain masonry up to three storeys). From these, only 0.5% of the traditional structures presented heavy damages or collapsed against 7.4% of the RC structures, 0.6% of the traditional structures presented moderate damage versus 8.6% of the RC and 10% and 16.5% respectively presented light damages. In all the mentioned earthquakes, a low number of total collapses of traditional buildings was recorded, even if light to severe damage can develop depending on the conservation of the structure, on the materials, and on the structural features of the system. The typical damages in timber frame buildings under seismic actions include: (1) cracking and failure of plaster as the result of the deformation of the braced elements and posts. When reduced space of the posts exists no propagation of the cracking occurs for the masonry infill; (2) loosening and failure at the connections (Figure 3c). In fact, the connections take a central role on the seismic behavior of traditional timber frame buildings as they are the elements keeping the structure together during the earthquakes, being understandable that important deformations and damages can develop; (3) large lateral displacements, which can result from soft-storey mechanism, resulting from the changes carried out on the traditional buildings at the first floor related to the removing of timber brace elements and studs aiming at having free spaces for commercial purposes.

In addition, the earthquakes of India 2001 and El Salvador 1986 are other two examples where the timber-laced masonry buildings and the *Bahareque* timber frame buildings behaved considerably better than reinforced concrete or unreinforced masonry (Langenbach, 2007). The heavy damage and inadequacy of timber frame building under earthquakes, as occurred in Nicaragua 1936, can often be attributed to the poor condition of the connections due to inadequate conservation. More recently, during the earthquake of Haiti in January 2010, it was seen that a great number of concrete block and reinforced concrete buildings were heavily damaged, resulting in the loss of a dramatic number of human lives and in a huge economic impact in the economy (Langenbach et al., 2010). Contrarily, the behavior of traditional timber frame buildings did not exhibit so much severe damage. Both the braced timber frame and the Colombage, with more flexible, energy dissipating systems tended to perform better than the other structural systems (masonry and reinforced concrete) (Langenbach et al., 2010).

4 Experimental characterization of traditional timber frame walls

In spite of timber-frame walls being very common all over the world and having demonstrated a reasonably good behaviour during past earthquake events, very little information is available on their experimental seismic behaviour that enables to understand the resisting mechanisms under lateral loading. This type of construction system has not been taken into great consideration from the scientific research community but a great number of historical buildings are actually timber framed, which means that the evaluation of its mechanical performance, particularly to seismic actions, can be valuable. Moreover, the great variability found in these buildings in terms of geometry, materials and modifications introduced in the structures makes their seismic assessment a relevant research issue.

With this respect, only in the last decade experimental studies have been carried out in different countries for the evaluation of the in-plane lateral performance of distinct types of timber frame walls. Therefore, this section aims at giving an overview on the experimental analysis of timber frame walls under in-plane cyclic loading by presenting the main outcomes.

4.1 *In-plane cyclic behavior of "Pombalino" frontal walls*

In relation to Pombalino timber frame frontal walls, few experimental information is available until now. The first experimental work carried out at the national laboratory of civil engineering (LNEC) by Santos dates back to 1997 (Santos, 1997), in the scope of a rehabilitation program of ancient masonry buildings. Three specimens of real walls were taken from an existing building which was going to be demolished and tested under static cyclic loads. It should be noticed that no vertical load was applied. The hysteresis loops of the tested wall, shown in Figure 4a, are indicative of the good deformation capacity and energy dissipation capacity of the structure.

Cyclic tests were also carried out by Meireles *et al*. (2012) on walls similar to the ones tested by Santos (1997), see Figure 4b. The wood specie selected was pinus pinaster, a typical Portuguese softwood, and modern nails were adopted, but assembled according to what is seen in existing walls (number and positioning). For the beams and posts a cross section of 12x8cm^2 was used and for the diagonals a section of 10x7cm^2 was adopted. Half-lap connections were considered between beams and vertical posts and between diagonal bars, additionally secured with two nails. The diagonal bars were connected to the beams and posts through nails. For the infill material it was decided to use brick masonry made with low strength hydraulic lime mortar. The walls were tested under a cantilever boundary configuration, as the top of the wall could rotate. The bottom beam was fixed to the reaction structure so that uplift was avoided. The vertical load applied was of about 80kN by means of four hydraulic jacks, aiming at simulating the dead and live load of the typical three stories and the ground floor. The tests were carried out under displacement control by using the Curee loading protocol. The hysteresis loops obtained for the two *frontal* walls tested show that in-plane lateral response is characterized by a considerable non-linear behaviour, with the hysteresis loops predicting reasonable energy dissipation (Figure 4b). The response is also characterized by pinching, which is associated to cumulative damage at the connections and progressive increase on the plastic deformations, similarly to what was also recorded in the tests of Santos (1997).

In-plane cyclic tests were carried out by Poletti and Vasconcelos (2014) in timber frame walls with the same geometrical configuration and connections (half-lap connections) but in this case, the dimension of the braced diagonal cell is lower, leading to a total height and a length of the wall 8% lower. Only one nail was used in all half-lap connections and regular brick masonry was considered as infill material, even if timber frame walls without infill were also considered, see Figure 5a,b. In some specimens lath and plaster covering was adopted as infill material, see Figure 5c. The vertical load was applied directly on the posts. Two levels of vertical loads were considered, namely 25kN and 50kN per post, and the tests were carried out under displacement control following a cyclic increasing displacement history, according to the ISO protocol (ISO 21581, 2010). The typical load-displacement diagrams are presented in Figure 6 for timber frame walls with brick masonry infill and empty timber frame walls for the two levels of vertical load. From the analysis of these diagrams, it is possible to observe that: (1) the timber frames filled with brick masonry and lath and plaster covering present similar behaviour, being the predominant resisting mechanism characterized by flexure, corresponding to the uplift of the lateral posts and rotation of the wall, as can be seen also through the diagrams of Figure 6, where the vertical displacements measured at the bottom connections (post/beam) are also shown. This resisting mechanism leads to plastic deformation of the nails placed at the bottom half-lap connections, which should be responsible for the unloaded branches characterized by a plateau; (2) the timber frame walls exhibit typical shear behaviour being the force-displacement diagrams characterized by pinching resulting from the cumulative deformation observed in the walls, particularly at the connections. The failure mode is characterized by the shear collapse of the central connections; (3) the infill brick masonry and lath and plaster covering influence the resisting mechanism of the timber frame walls. The resisting shear mechanism of plane timber frame wall is replaced by flexural rocking mechanism in case of addition of infill/covering material. The infill or covering materials act as confining elements, conditioning the deformation of the connections; (4) the vertical load applied in the posts influences the lateral resistance and the overall behaviour of the walls. The increase on the vertical load results in the increase of the lateral resistance. On the other hand, higher vertical loads lead to the decrease of the vertical uplift of the posts, mainly in case of brick masonry infill, meaning that the flexural rocking mechanism that prevails in the response of the lowest vertical load is reduced. It is possible that the higher stiffness of the brick masonry used in case of Poletti and Vasconcelos (2014) results in the higher stiffening effect of the connections leading to predominant flexural behaviour, contrarily to shear behaviour pointed out by Meireles et al. (2012). This appears also to be valid for the lateral resistance, as the lateral strength obtained by the authors is higher than the one pointed out by Meireles *et al.* (2012), taking into account that the same vertical load was applied. The difference is also in part attributed to the higher number of nails present at the bottom connections, as it was seen that a higher confining of the bottom connections led to lower uplifting, therefore to predominant shear behaviour (Poletti and Vasconcelos, 2012). The predominant flexural behaviour found for the lowest vertical pre-compression level was also obtained by Gonçalves *et al.* (2012), who carried out in-plane cyclic tests in the same walls of Poletti and Vasconcelos (2014). It should be noticed that in these two works only the brick masonry was not the same. In all mentioned experimental works the timber frame detach from the masonry for increasing lateral displacements. In the tensile part of the frame the masonry does not work at all, being only active in the neighborhood of the compression strut of the opposite side. The detachment pointed out by Meireles et al. (2012) is more associated to the shear deformation of the timber frame.

4.2 *In-plane cyclic behavior of other timber frame walls systems*

The research effort aiming at getting a more clear insight on the traditional timber frame construction has motivated very recent experimental works carried out on different typologies of timber frame walls.

The work presented by Vieux-Champagne et al. (2014) focus on a typology of walls similar to the frontal Pombalino walls in terms of geometry, even if with different cross section sizes and different connections, being almost all of them nailed connections. Steel strips were used at the bottom connections. These walls intend to represent the walls built in vernacular buildings during Haitian reconstruction after the strong earthquake of 2010 aiming at reducing the seismic vulnerability of the reconstructed buildings given the reasonable seismic behaviour exhibited by the same type of buildings during the earthquake. The timber frame walls were built with and without masonry infill, namely stone masonry infill. Based on the force-displacement diagrams provided it is observed that in-plane behaviour is characterized by an early nonlinear regime and by very reasonable ability to deform during this nonlinear regime, see Figure 7a. It is seen also that the hysteresis loops present some “pinching”, which should be associated to cumulative damage and deformation developed at the nailed connections. This effect is more pronounced in case of empty timber frames, attributed to the higher flexibility of the connections due to the absence of the confining effect of the infill material. The presence of stone masonry infill results in higher values of lateral resistance and higher stiffness but it appears that low values of maximum displacement are achieved. These results are in accordance to the results pointed out by Poletti and Vasconcelos (2014).

Ruggieri and Zinno (2013) performed in-plane cyclic tests on walls typically found in baraccata buildings. The authors tested real scale specimens based on an existing building in Mileto. A timber frame was embedded in the masonry wall on both sides. The walls exhibited a predominant flexural behaviour, with uplift of the lateral posts.

The experimental work carried out by Torrealva and Vicente (2012) on quincha timber frame walls with distinct configuration of the bracing elements (diagonal and bottom small struts) and with distinct height to length ratios of 1:3 and of 1:6, reveals a great capacity of nonlinear deformation and a trend for high energy dissipation, see Figure 7b.

Aktas (2011) presents an extensive experimental work on distinct configurations of Ottomman timber frame walls most characteristic on existing himis timber frame buildings with and without masonry infill (brick and adobe masonry) and lath and plaster infill (bagdadi covering). The connections are mainly made with nails.

The test results showed that failures were always governed by the failures at the connections. Under in-plane lateral loading, the bottom connections on the right and left, as well as those at the both ends of diagonal braces are the first ones to deform and eventually fail. At each loading cycle, nails were pulled out partially and driven back to their original places, until a point where the nail gets buckled and the connection is completely lost. In spite of somehow scattered results due to the random workmanship that characterizes traditional himis houses, it was seen that the timber frame walls appeared to have high energy dissipation capacity, which is one the most important factors defining a good seismic performance. It was also stated that the load bearing capacity and lateral stiffness of a timber frame increases clearly with infill/covering material.

4.3 *Seismic performance indicators*

The seismic performance of timber shear walls submitted to in-plane cyclic loads can be evaluated in quantitative terms from parameters such as ductility, lateral drift, dissipation of energy and equivalent viscous damping.

Usually, ductility is defined by the ability of a system to deform in the nonlinear range without a significant reduction of the resisting capacity. It is quantitatively defined as the yielding displacement to the ultimate displacement ratio, being the yielding displacement calculated as the elastic displacement corresponding to the yielding lateral force and the ultimate displacement as the displacement corresponding to 80% of the resisting lateral force defined in the post-peak regime, see Figure 8a. The lateral drift is also a quantity that enables to evaluate the capacity of in-plane lateral deformation and it is defined as the lateral displacement to the height of the force application point ratio.

Another parameter that is used in the evaluation of the seismic performance is the equivalent viscous damping ζ_{eq}, given by eq. 1:

$$\zeta_{eq} = \frac{E_d}{2\pi(E_e^+ + E_e^-)} \tag{1}$$

where E_d is the dissipated hysteretic energy enclosed in the hysteretic loop of the cyclic force-displacement diagram and E_e^+ and E_e^- are the elastic energies of an equivalent viscous system calculated at the maximum displacement in each loop for the positive and negative direction of loading respectively (Figure 8b).

Equivalent viscous damping represents the energy dissipation due to the nonlinear (hysteretic) behaviour of the walls resulting for example from the friction in the connections and opening and closing of cracks and gaps.

In this section a comparative analysis is made among the different traditional timber frame walls with respect to the ductility and lateral drifts values, see Table 1.

The values of lateral drift obtained by Poletti and Vasconcelos (2014) for all walls was close to 4%, being a little higher than the value pointed out by Meireles et al. (2012) of 3.5%. It should be mentioned that the values obtained by the authors could be even higher in some walls, particularly the ones submitted to the lowest levels of pre-compression, as the maximum displacement did not correspond to the collapse of the walls. The lateral drifts pointed out by Vieux-Champagne et al. (2014) are of the same order, ranging from 3.1% to 4%. There is a trend for timber frame walls without infill material to present higher values of lateral drift, which can be attributed to the ability of the infill material to reduce the free deformation of the traditional connections. In the case of the casa baraccata system, a lateral drift of 3% was achieved, but the walls did not reach complete failure (Ruggieri and Zinno, 2013). The lateral drifts can be even higher in case of quincha walls tested by Torrealva and Vicente (2012), which mention values of lateral drift ranging from 7.5% to 9.4%, depending on the geometrical configuration of the bracing diagonals.

Based on the experimental results pointed out by Aktas et al. (2013), timber frame structures are highly ductile thanks to the energy dissipative plastic behaviour of their connections. Drift ratios of up to nearly 9% were observed. The values of lateral drift are influenced by the presence of infill masonry and covering, but it appears that covering seems to decrease the drift ratio in a higher percentage. For empty frames, the average drift ratio is 6%, and for frames with infill/covering, the average drift ratios are 5.5 and 4.9%, respectively. Moreover, based on the values, adobe and brick infill seem to increase the drift ratio by around 3%, while bağdadi and şamdolma covering seem to decrease the drift ratio by around 18% in average.

In relation to the values of equivalent viscous damping, it should be mentioned that the authors found higher values for low lateral drifts when compared to the values found for higher lateral drifts, being in average 0.1 for infilled timber frame walls and 0.12 for timber frame walls in case of high lateral drifts. These values are of the order of the ones found by Gonçalves et al. (2014), on similar traditional Portuguese *frontal* walls, which obtained values of equivalent viscous damping for low values of drift of 0.17-0.20 for infill walls and 0.19-0.20 for empty timber frame walls. However, the values then decreased to 0.11-0.13 and 0.10-0.11 respectively, confirming the trend of having higher values for low drifts. The values of the equivalent viscous damping obtained by Vasconcelos et al. (2013) for 1:2 reduced scale "frontal" walls tested under in-plane cyclic loads was about 0.15. This higher value can possibly be attributed to the distinct "frontal" walls typology and connections: additional vertical and horizontal bars in the braced cells and mortise-tenon connections between beams and posts. Casa baraccata walls had a value of damping varying between 0.06 and 0.089 (Ruggieri and Zinno, 2013). From this work it was possible to observe that equivalent viscous damping depends on the resisting mechanism, being higher when shear response predominates. In these walls lateral drifts of about 3.5% were obtained, being comparable to the values obtained in the other studies.

5 Retrofitting of timber frame walls

As previously mentioned, timber frame buildings constitute an important portion of many historic city centres in the world. Many of these buildings have known little or even no care during their life or they have been modified without taking into account the seismic response of the structure after alterations had been made, frequently causing greater damages during earthquakes than the ones observed in buildings which had preserved their original structure. For this reason, it is important to study strengthening solutions for the rehabilitation of existing buildings, guaranteeing a good seismic behaviour and preserving as much as possible the originality of the structure.

Many examples are available on restoration works done in traditional timber frame buildings, but few experimental studies have been performed in order to assess the efficiency of the strengthening techniques adopted. Usually, rehabilitation of existing timber frame buildings, such as those carried out in the last decade in Lisbon, include strengthening of the elements (timber frame to outer masonry wall connection, timber frame to timber floor connection, etc.) as well as the replacement of decayed elements (such as floor beams, timber elements or masonry infill in timber frame walls) or even alterations of the existing structure (for example, filling of existing openings). The retrofitting has been made by using FRP sheets in the connections of the frontal walls, creating a star-shaped strengthening, or damping systems linked to frontal walls and to the outer masonry walls through injected anchors and providing additional bracing (Cóias, 2007). However, it should be noticed that these interventions have been done with very little background knowledge.

From an experimental point of view, more relevant information is available on retrofitting techniques for traditional timber connections (Branco, 2008; Parisi and Piazza, 2002), which are considered also of great importance for the strengthening of traditional walls, since strengthening of timber frame walls is almost reduced to the strengthening of the connections.

5.1 *Retrofitting solutions for timber frame walls*

With respect to retrofitting techniques in timber frame walls, it should be stressed that their implementation is not a well-covered research topic among the research community. Tests performed on modern timber frame walls do not cover retrofitting techniques. On the other hand, the majority of the information existing on strengthening techniques covers the retrofitting of timber elements submitted to different loading conditions and not connections. Due to the role that connections play on the in-plane behaviour of timber frame walls and if the condition of wood is acceptable, the retrofitting of timber frame walls is much concentrated in the retrofitting of timber connections. The retrofitting aims mainly to improve the deformation capacity and to reduce the damage concentrated at the connections, even if complementarily the increase on the lateral in-plane resistance can also be achieved.

Cruz et al. (2001) performed diagonal tests on reduced scale wallets strengthened with glass fiber reinforced polymer (GFRP) rods and glass fiber fabric (GFF) sheets. The walls were retrofitted embedding two GFRP bars to the outer timber members and GFF sheets were glued to the timber elements of the central connections. Vasconcelos et al. (2013) also presented a solution for reinforcement of timber framed walls with glued FRP sheets in some of the connections of the walls in order to access the influence of this technique on the lateral resistance, stiffness and deformation.

An alternative solution to fiber reinforced polymers is the use of steel plates or bolts to retrofit the connections. These solutions are considered to be more compatible with the existing structures and, above all, are reversible, contrarily to the composite materials that need a glue base to adhere to the timber elements.

Gonçalves et al. (2012) and Poletti et al. (2014) presented some solutions for retrofitting timber frame walls with steel plates and steel bolts at the connections of timber frame walls. Examples of distinct steel plates configuration are presented in Figure 9, together with a solution for steel bolts (Poletti et al., 2014). The solution of adding pre-drilled bolts at the base of the walls is intended to prevent uplift displacements and then limit the possible rocking mechanism, mainly for low levels of load applied to the posts. This intervention should also assure that the connections work properly until failure, by reducing the damage and instabilities in the out-of-plane direction. The steel plates can be customized (star-shaped) or even commercial rectangular steel plates. The steel plates are secured with screws and linked with bolts which go through the thickness of the wall. The steel plates can link or not the diagonal bracing elements to the main elements of the connection (vertical post and horizontal beam). From experimental results it was seen that the connection of the diagonal bars can result in a considerable increase on the stiffness and resistance, leading to a possible instability for high levels of lateral force associated to high levels of compression along the diagonals.

It should be stressed that both types of steel plates require low technical equipment and non-specialized workmanship.

5.2 *Experimental behaviour of strengthening solutions*

As mentioned before, the retrofitting of timber frame walls aims to improve its in-plane behaviour under cyclic loading, meaning that higher displacements and higher energy dissipation is expected together with more controlled damage. The seismic performance of steel plates and bolts was evaluated based on in-plane cyclic loading for two distinct levels of vertical load applied in the vertical posts (25kN and 50kN per post) following the same procedures used in the unreinforced timber frame walls (Poletti et al., 2014).

Through Figure 10, a comparison between timber frame walls with brick masonry infill retrofitted with steel bolts at connections submitted to vertical load of 50kN per post (RIW50_B) and the corresponding unreinforced timber frame wall can be made. It is observed that there is no great gain in terms of ultimate capacity and stiffness. In fact, for the lower vertical load level (25kN per post), the gain in terms of maximum load was of 23.7%, whereas for the higher vertical load level the lateral resistance decreased by 5%. In terms of ultimate displacement, the walls gained 5.7% and 0.2% for the lowest and highest vertical pre-compression levels. The very low effectiveness of bolts as a retrofitting technique in timber frame walls in terms of lateral resistance can be attributed to the predominant flexural behaviour of the walls under in-plane loading. The bolts are not completely efficient in resisting the tensile stresses induced by cyclic loading at the bottom connections, and the walls experienced damages in the central connections until their failure. The nailed diagonals detached from the main frame. The central beam tore off (Figure 11a) in tension and the central post crushed due to the shear induced by the diagonals, similarly to what happened in the unreinforced tests (Poletti, 2013). In spite of this, for both load cases, the shape of the hysteretic loops experiences some changes. The plateau caused by the uplifting and recovering of the vertical post from the bottom beam is still present, but it is less pronounced and the unloading branch of the cycles is smoother. The vertical uplifting of the posts decreased by approximately 40% for both load cases in relation to unreinforced walls, resulting from the lower predominance of the flexural resistant mechanism and from the contribution of a certain shear resistant component. Even in a reduced scale, the bolts contributed to the resistance to tensile forces developing in the bottom half-lap connections, and ensured a more remarkable post-peak behaviour enabling the connections to work until failure, contrarily to unreinforced walls, where after a certain lateral drift no contact was observed between the post and the bottom beam.

The comparison between the hysteresis diagrams found in unreinforced walls and after retrofitting with steel plates can be made through the analysis of Figure 12a, where results are shown for the walls tested with the higher vertical pre-compression (RIW50_P – timber frame walls with brick masonry infill submitted to a vertical load of 50kN per post retrofitted with custom steel plates and corresponding unreinforced wall, UIW50; RTW50_P_M – timber frame wall without infill submitted to a vertical load of 50kN per post retrofitted with commercial steel plates not connecting the diagonal braces and corresponding unreinforced wall, UTW50). Walls retrofitted with steel plates experienced a similar behaviour independently on the vertical load level. For both timber frame walls with masonry infill, an important increase in terms of load capacity and stiffness was recorded: the maximum lateral load increased by 147% for the lower vertical load and by 60.4% for the higher vertical load. The initial stiffness of the walls increased by 30% when compared to the unreinforced solution for the lower vertical load level and by 14% for the higher vertical load level. The displacement imposed to the walls does not correspond to its maximum displacement capacity as it was not possible to obtain the complete failure mode of the walls. The high stiffening effect of custom steel plates, linking the main elements of the connection (post and beam) to the diagonal braces, together with the slenderness of the wall led to this out-of-plane component, even if it was considered minimal. The ultimate state would be achieved if further lateral displacements were applied. For this type of strengthening, the values of initial lateral stiffness are comparable for the two vertical load levels, meaning that for such a strong retrofitting technique, the effect on the amount of vertical load becomes secondary.

The solution of commercial steel plates connecting only the beam and posts allowed the walls to gain significantly both in terms of stiffness and load capacity, without compromising the displacement capacity, see Figure 12b. In terms of maximum load, the walls gained 183% and 35% for the lower and higher pre-compression load respectively. On the other hand, this retrofitting solution led to remarkable pinching in the timber walls. Similarly to the retrofitting with custom steel plates, the vertical load has only marginal influence in terms of maximum load, even if it influences the initial stiffness, being higher for the higher vertical pre-compression. This solution is therefore more appropriate for timber frame walls, since its stiffening effect is not overwhelming.

It should be mentioned that the lateral cyclic behaviour obtained for timber frame walls retrofitted with commercial plates linking the main members (post and beams) with the diagonals was characterized by a significant increase of the lateral resistance but on the other hand exhibited a trend for becoming unstable in the out-of-plane direction, which should be associated to the high levels of compression stresses driven by the compressed diagonal strut, promoting the development of second-order effects. From the results obtained, it appears that this type of retrofitting is too stiff and not ductile enough for timber frame walls without infill, increasing significantly the lateral resistance (over 200% for the lower vertical load level and 97% for the higher vertical load when compared to the equivalent unreinforced wall) and the stiffness of the walls (77% for the lower vertical load and 50% for the higher one).

In case of walls retrofitted with steel plates the damages observed were similar for all walls and they consisted of: (1) failure of the half-lap connection linking two diagonal members when steel plates linked the diagonals to the main frame (Figure 11b); (2) failure of the central middle connection when the diagonals were not linked to the main frame through the steel plates (Figure 11c). The failure of the half-lap connection of the diagonal elements occurred in all specimens, independently on the type of plate, because this type of retrofitting stiffened excessively the connections, not allowing free movement to the bracing elements. The strong retrofitting of the post-beam half-lap connections in combination with the increase on the stresses carried by the diagonal bars resulted in the failure of the weakest zones of the wall, which were the half-lap connection of the diagonals. No damages were observed in the main wood members of the connection.

The performance of the retrofitting with custom star shape steel plates was also evaluated by Gonçalves et al. (2014), being observed that the retrofitting resulted in the considerable increase on the lateral strength and on the energy dissipation, see Figure 12c.

Comparing the two retrofitting solutions, bolts were able to improve the overall behaviour of the wall in terms of deformation capacity and post-peak behaviour, but it is not relevant in the increase on the lateral strength. On the other hand, the appropriate steel plates configuration is able to guarantee a better seismic response of the walls both in terms of stiffness and load capacity.

5.3 *Seismic performance indicators*

Similarly to what has been discussed previously, the seismic performance of retrofitted walls can be also analysed in order to evaluate the effectiveness of the retrofitting techniques. A comparative analysis is presented here between the retrofitted walls with steel plates and with bolts.

The assessment of the ability of the timber frame walls to dissipate energy is here evaluated based on the energy dissipated at each cycle, E_D, computed by calculating the area enclosed by the loop in the load-displacement diagram and it represents the amount of energy dissipated during the cyclic loading. The energy can be dissipated through friction in the connections, yielding of nails, yielding and deformation of the retrofitting bolts and steel plates and permanent deformation accumulated in the walls as observed during the tests. According to the results found in Figure 13 (Poletti et al., 2014), all retrofitting techniques adopted were able to guarantee greater energy dissipation during the tests (RIW25_B – timber frame walls with brick masonry infill submitted to a vertical load of 25kN per post retrofitted with custom steel plates and corresponding unreinforced wall, UIW25; RIW25_P – timber frame walls with brick masonry infill submitted to a vertical load of 25kN per post retrofitted with custom steel plates; RTW25_P_M – timber frame wall without infill submitted to a vertical load of 25kN per post retrofitted with commercial steel plates not connection the diagonal and corresponding unreinforced wall, UTW25). The highest dissipative solution is provided by the retrofitting technique with steel plates linking the diagonals. Timber frame walls with brick masonry infill retrofitted with steel plates increased the total dissipated energy by 96% and 57% respectively for the lower and higher vertical load level. For the walls tested without linking the diagonals, the dissipative capacity was lower. In case of timber frame walls with this alternative steel plates configuration, the total dissipated energy increased by 132% and 38% for the lowest and highest vertical pre-compression respectively when compared to the equivalent unreinforced wall. The retrofitting solution with steel plates connecting all elements tested by Gonçalves et al. (2014) revealed also to be highly effective in the improvement of the dissipation of energy, by increasing the total energy dissipation by 254%.

The response of the walls retrofitted with bolts showed results comparable to the ones obtained in unreinforced walls for low values of lateral drift, but improved for high values of drift in case of the higher pre-compression load, given that the solution changed the failure mode of the wall, reducing the amount of pinching in the walls, guaranteeing a higher dissipative capacity of the wall (+14%).

Comparing the results of equivalent viscous damping for the walls tested (Figure 14), the influence of the vertical pre-compression load was evident only for the strengthening with bolts. In the latter case, the highest level of pre-compression leads to higher values of equivalent viscous damping than the wall submitted to the lower vertical pre-compression. In general the retrofitted walls present higher values of equivalent viscous damping. The walls retrofitted with bolts exhibit also higher values of hysteretic damping than the unreinforced walls for high levels of lateral drift in case of the walls submitted to the highest level of pre-compression. For the lower level of vertical load, the equivalent viscous damping is only higher for lateral drifts of 3%. Walls with steel plates present a constant final equivalent viscous damping of 0.12 and 0.13 for the lower and higher pre-compression level respectively, with little variation among the walls. Similar values were found for cyclic tests on bird's mouth connections (Branco, 2008). This type of connections strengthened with bolts presents a value of equivalent viscous damping of 0.11, while the connections strengthened with stirrups presented a value of 0.15.

Higher values of equivalent viscous damping were observed by Gonçalves et al. (2014) for the retrofitting of timber frame walls with steel plates, attaining maximum values of 0.29, representing in average an increase of about 22.2% in relation to the unreinforced timber frame walls.

Based on the results available, it appears that the retrofitting solution of steel plates and steel bolts applied at the connections can be considered as retrofitting solutions for timber frame walls as they simultaneously lead to an increase on the lateral strength, energy dissipation and result in higher values of equivalent viscous damping, revealing a more appropriate seismic performance. Additionally, it can be said that more controlled damage is achieved, meaning that the costs with further retrofitting can be lower in the future.

6 Concluding remarks

Timber frame construction can be seen in urban and vernacular buildings in several countries around the world, either motivated by the availability of materials and construction tradition or by the need of reducing the seismic vulnerability of existing buildings, namely in Portugal, Italy and Greece.

Timber frame construction encompasses a great variety of geometrical configurations and infill materials but it has been seen that in general during past earthquakes timber frame buildings exhibited a reasonable mechanical behaviour when compared with other structural typologies. However, it should be mentioned that the conservation state of these structures plays an important role in the appropriate seismic performance.

On the other hand, in spite of traditional timber frame construction being spread all over the world, few research has been performed on this issue both from an experimental and a numerical point of view and the majority of the known research works dates back to the past decade and has been carried out mainly in Mediterranean countries.

This chapter points out some experimental results on the in-plane cyclic behaviour of unreinforced and retrofitted timber frame walls, which are characteristic of some constructive systems. An overview of the in-plane experimental behaviour of timber frame walls as well as of the seismic parameters is given. Additionally some retrofitting solutions are presented and a discussion of its effectiveness under in-plane cyclic loading is discussed.

From the analysis carried out it was possible to conclude that timber frame walls exhibit large capacity to deform in the nonlinear regime with remarkable lateral drifts with controlled damages under in-plane cyclic loading. Additionally, timber frame walls present good capacity to dissipate energy, which makes this system behaving better under in-plane loading than unreinforced masonry walls, used in vernacular architecture in several countries with important seismicity.

The retrofitting with steel plates at the connections reveals to be appropriate as an increase on the resistance, energy dissipation and equivalent viscous damping was recorded. Besides, for the same lateral drifts, a more controlled damage was recorded, when compared to the unreinforced timber frame walls.

7 Research Studies and Future trends

From past evidences about the reasonable mechanical performance of timber frame buildings under earthquakes, it can be considered that traditional timber frame construction deserves to be conserved and can be viewed as a true alternative for reconstruction and strengthening purposes of vernacular construction. This has been already made for example during the reconstruction process after the strong earthquake that hit Haiti in 2010 and India and Pakistan in 2005, after which a manual for building Dhajji construction was created (Schacher and Ali, 2010).

However, it is important to mention that the knowledge on the mechanical behaviour of this type of structure is still limited and additional research on the in-plane behaviour and particularly on the out-of-plane behaviour is needed. In this scope, further dynamic tests on full timber frame structures can reveal with more accuracy the dynamic global mechanical behaviour, beyond the individual behaviour of timber frame walls.

Additionally, the understanding of the main vulnerabilities of this type of construction, mainly at the level of conservation state both on the connections and on wood structural elements, is very important as they can result in less appropriate seismic performance. The proposal and validation of distinct retrofitting techniques that reveals to be compatible, reversible and durable needs to be further developed as it plays an important role in the improvement of the seismic performance of this type of structures. It should be mentioned that this topic is not well covered and further investigation is important to provide sound retrofitting guidelines to different stakeholders.

The strategies for the numerical simulation of timber frame walls, and at the limit of timber frame buildings, represent also a field of investigation that needs major contributions as very few information is available. The numerical simulation is mandatory if assessment of seismic vulnerability of timber frame buildings is required (Meireles et al., 2012; Kouris and Kappos, 2012).

References

Aktaş YD, Evaluation of seismic resistance of traditional ottoman timber frame houses, PhD thesis, Middle East Technical University, 2011.

Aktaş YD, Akyüz U, Türer A, Erdil B, Güçhan NS. Seismic Resistance Evaluation of Traditional Ottoman Timber-Frame Hımış Houses: Frame Loadings and Material Tests. Earthquake Spectra, 2013. Doi: http://dx.doi.org/10.1193/011412EQS011M.

Bianco A. The baraccata house. Guide to desing and restauration site (in Italian). Rome: GB EditoriA; 2010. P. 19, 29, 60.

Branco, Influence of the joints stiffness in the monotonic and cyclic behaviour of traditional timber trusses. Assessment of the efficacy of different strengthening techniques, Ph.D. Thesis, Civil Engineering Dept., University of Minho, Portugal, 2008.

Cóias V. Structural rehabilitation of old buildings (in Portuguese). Lisbon: ARGUMENTUM, GECoPRA; 2007.

Copani P. Timber-frame buildings in Scandinavia: high deformation prevent the system from collapse. From Material to Structure – Mechanical Behaviour and Failures of the Timber Structures ICOMOS IWC – XVI International Symposium. Florence, Venice and Vicenza 11-16 November 2007. (Available at http://www.icomos.org/iiwc/2007.htm)

Cruz, H., Moura, J. and Saporiti, J. The use of FRP in the strengthening of timber-reinforced masonry load-bearing walls, Historical Constructions, P.B. Lourenço, P. Roca (Eds.), Guimarães, 2001.

França JA. Pombaline Lisbon and the illuminism (in Portuguese). Lisbon: Bertrand; 1983.

Gonçalves A, Gomes Ferreira J, Guerreiro L, Branco F. Experimental characterization of Pombalino "frontal" wall cyclic behavior, 15th World Conference on Earthquake Engineering, 24-28 September, Lisbon, 2012.

Gonçalves A, Gomes Ferreira J, Guerreiro L, Branco F. Experimental characterization and numerical modelling of the cyclic 34ehaviour of reinforced timber framed walls, 9th International masonry conference, University of Minho, 2014.

Gulkan P, Langenbach R. The earthquake resistance of traditional timber and masonry dwellings in Turkey. 13th World Conference on Earthquake Engineering. Vancouver, 2004.

Gülhan D, and Güney IÖ. The behavior of traditional building systems against earthquake and its comparison to reinforced concrete frame systems: experiences of Marmara earthquake damage assessment studies in Kocaeli and Sakarya. Proceedings of Earthquake-safe: Lessons to be Learned from Traditional Construction, Istanbul, Turkey, 2000. (Available at: http://www.icomos.org/iiwc/seismic/Gulhan.pdf)

ISO 21581. Timber structures – Static and cyclic lateral load test method for shear walls. International Organization for Standardization (ISO), Geneva, Switzerland, 2010

Jurina L, Righetti M. Traditional building in Peru. XVI IIWC International Conference and Symposium, Florence-Venice-Vicenza, 11th-16th November 2007.

Kouris L., Kappos A.J. (2012). Detailed and simplified non-linear models for timber-framed masonry structures. Journal of Cultural Heritage, 13(1): 47–58.

Langenbach R. DON'T TEAR IT DOWN! Preserving the earthquake resistant vernacular architecture of Kashmir. New Delhi: UNESCO; 2009.

Langebach R. From "Opus Craticium" to the "Chicago frame": Earthquake-resistant traditional construction, International Journal of Architectural Heritage, 1, 29-59, 2007.

Langebach R, Kelley S, Sparks P, Rowell K, Hammer M, Julien O. Preserving Haiti's Gingerbread Houses. 2010 Earthquake Mission Report. World Monuments Fund. December 2010.

Lueger O. Encyclopedia of total technology (in German). 1894.

Mascarenhas J. Construction systems – V. (in Portuguese). Lisbon: Livros Horizonte; 2004.

Makarios T, Demosthenous M. Seismic response of traditional buildings of Lefkas Island, Greece. Engineering Structures, 28, 264-278: 2006.

Meireles H, Bento R Cattari, S, Lagomarsino, S. A hysteretic model for “frontal” walls in Pombalino buildings, Bulletin of Earthquake Engineering, 10(5), 1481-1502, 2012.

Parisi, M.A. and Piazza, M. Seismic 35ehaviour and retrofitting of joints in traditional timber roof structures, Soil Dynamics and Earthquake Engineering 22, 1183–1191, 2002.

Pinho FFS. Walls of old buildings in Portugal (in Portuguese). Lisbon: LNEC; 2000.

Poletti E., Vasconcelos G. Assessment of the Seismic Behaviour of Unreinforced Traditional Half-timbered Walls. In Proceedings of 15th World Conference on Earthquake Engineering (15WCEE), 24-28 September, Lisbon, Portugal, 2012.

Poletti E, Characterization of the seismic behaviour of traditional timber frame walls, PhD Thesis, University of Minho, 2013.

Poletti E., Vasconcelos G., Jorge M. (2014) Traditional retrofitting techniques for timber frame walls: seismic behaviour through experimental results. Journal of Earthquake Engineering, 18(4): 553-579.

Poletti E., Vasconcelos G. (2014) Seismic behaviour of traditional timber frame walls: experimental results on unreinforced walls. Bulletin of Earthquake Engineering, doi: 10.1007/s10518-014-9650-9.

Poletti E, Vasconcelos G. The confining effect of masonry infill on the seismic behaviour of traditional timber frame walls, 9th International Masonry Conference, University of Minho, in USB, 2014.

Ruggieri N, Zinno R (2013) Behaviour of the borbone constructive system under cyclic loading. Preliminary report. In Proceedings of the 1st international symposium on historic earthquake-resistant timber frames in the Mediterranean region (H.Ea.R.T.2013), November 4–5, University of Calabria, Italy

Santos S. Tests of Pombalino walls (in Portuguese). Lisbon: LNEC; 1997. Nota Técnica Nº 15/97-NCE.

Schacher T., Ali Q (2010). Dhajji Construction. For one and two storey earthquake resistant houses. A guidebook for technicians and artisans. United Nations Pakistan, National Disaster Management Authority Pakistan, SUPSI.

Tampone G. The restoration of timber structures (in Italian). Milan, Libreria Tecnica Hoepli, 1996.

Tobriner S. The baraccata house: an antiseismic system in the XVIII century Calabria (in Italian). Costruire in laterizio 1997; 56: 110-115.

Torrealva D, Vicente E. Experimental evaluation of seisic behavior of Quincha walls from the historic centre of Lima – Peru, 15^{th} World Conference on Earthquake Engineering, 24-28 September, Lisbon, 2012.

Touliatos P. The box framed entity and function of the structures. The importance of wood's role. In: Conservation of Historic Wooden Structures. In International Conference «Conservation of Historic Wooden Structures", vol.1, pages 52-64. Florence 2005.

Touliatos P. Evaluation and drafting of recommendations for interventions in buildings of the historical settlement of Lefkas (in Greek). Organization for seismic designand protection, NTUA, Public Library of Lefkas Editions, Athens, Greece, 2004.

Tsakanika-Theohari E, Mouzakis H. A Post-Byzantine mansion in Athens. The restoration project of the timber structural elements. World Conference on Timber Engineering, Riva del Garda, Italy, June 20-24 2010.

Tsakanika-Theohari E. The constructional analysis of timber load bearing systems as a tool for interpreting Aegean Bronze Age architecture. In Proceedings of the Symposium 'Bronze Age Architectural Transitions in the Eastern Mediterranean: Diffusion and Diversity', 07.-08. 05. 2008 in Munich (Weilheim 2009): 127-142, 2008.

Vasconcelos G., Poletti E., Salavessa E., Jesus A., Lourenço P., Pilaon P. In-plane shear behaviour of traditional timber walls, Engineering Structures, 56, 1028-1048, 2013.

Vieux-Champagne F, Grange S, Sieffert Y, Daudeville L. Experimental analysis of seismic resistance of shear walls in traditional Haitian houses, 15th World Conference on Earthquake Engineering, 24-28 September, Lisbon, 2012.

Vieux-Champagne F, Sieffert Y, Grange S, Polastri A, Ceccotti A, Daudeville L. Experimental analysis of seismic resistance of timber framed structures with stones and earth infill, Engineering Structures, 69, 102-115, 2014.

Vintzileou E, Zagkotsis A, Repapis C, Zeris Ch. Seismic behaviour of the historical structural system of the island of Lefkada, Greece. Construction and Building Materials, 21, 225-236: 2007.

Vintzileou E. Effect of Timber Ties on the Behavior of Historic Masonry. Journal of Structural Engineering, 134(6): 961-972, 2008.

LIST OF TABLES

LIST OF FIGURES

Figure 7. Force-displacement diagrams obtained in distinct types of timber frame walls; (a) Vieux-Champagne et al. (2014); (b) Torrealva and Vicente (2012).

Figure 8. (a) Bilinear idealization of the monotonic experimental envelop; (b) dissipated and input potential energy is a hysteretic loop

Figure 9. Retrofitting solution: (a) steel bolts; (b) custom star shape steel plates; (c) commercial steel plates

Figure 10. Hysteretic force-displacement diagrams for walls strengthened with bolts, higher pre-compression load

Figure 11. Typical damages in retrofitted walls: (a) tearing off of central beam in a retrofitted wall with steel bolts (with brick infill); (b) failure of half-lap connection in the bottom cell in a retrofitted wall with custom steel plates (with brick infill); (c) failure of central connection in in a retrofitted wall with commercial steel plates (without brick infill)

Figure 12. Comparison of force-displacement diagrams between unreinforced and retrofitted timber frame walls; (a) timber frame wall with brick masonry infill custom steel plates and unreinforced timber frame wall; (a) timber frame wall without infill with commercial steel plates and unreinforced timber frame walls (Poletti and Vasconcelos, 2013); (c) retrofitted timber frame walls with steel plates (MW 6 and MW6) and unreinforced walls (MW1) (Gonçalves et al., 2014)

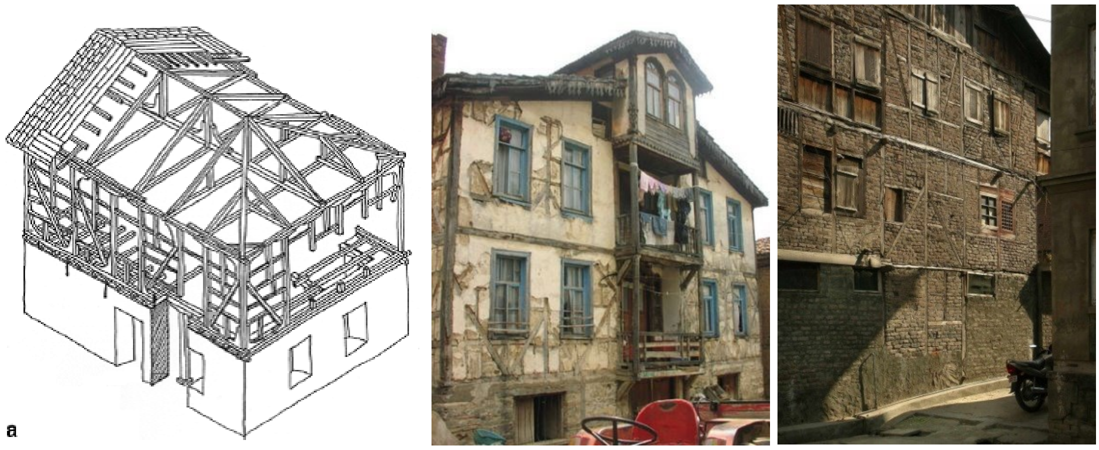
Figure 13. Cumulative dissipated energy for all walls tested by Poletti (2013)

Figure 14. Equivalent viscous damping ratio: (a) lower pre-compression level; (b) higher pre-compression level

Table 1

Seismic performance indexes

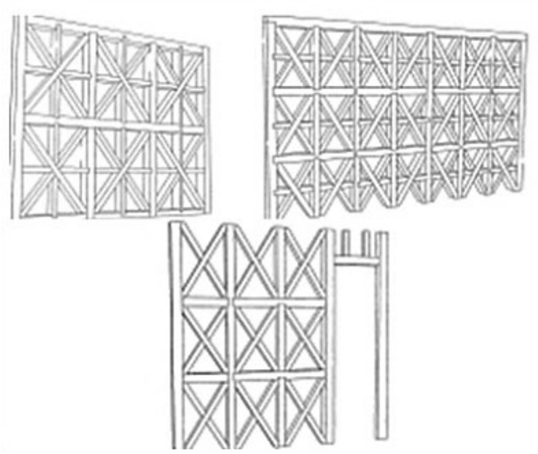
Reference	Lateral drift (%)	Equivalent viscous damping
Poletti et al. (2013)		
Timber infilled frame	4	0.10
Timber infilled frame		0.12
Meireles et al. (2012)		
Timber frame with masonry infill	3.5	
Gonçalves et al. (2013)		
Timber infilled frame		0.11
Timber frame		0.10
Vieux-Champagne et al. (2014)		
Timber frame with masonry infill	3.1-3.9	
Timber frame	4.2	
Ruggieri and Zinno (2013)		
Masonry wall with timber frame	3	0.06-0.089
Torrealva and Vicente (2012)		
Walls with citara		
Walls with diagonal	7.5	
	9.375	
Aktas et al. (2013)		
Timber frame with brick	5.5	
masonry	4.9	
Lath and plaster (bagdadi		
cladding)		



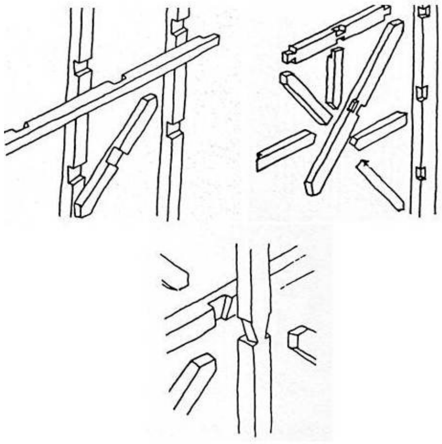
(a) (b) (c)

Fig. 1

Some examples of timber frame buildings; (a) typical house of Lefkas island in Greece, built with the local aseismic technique (Touliatos, 2004); (b) in Turkey - hatil at ground floor and himis in upper storeys (Tsakanika-Theohari and Mouzakis, 2010) ; (c) India - dhajji-dewari building in Kashmir (Langenbach, 2009)



(a)



(b)

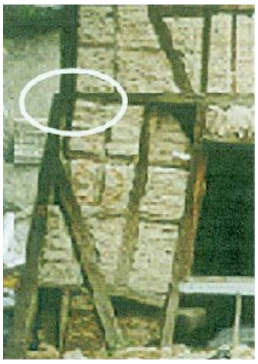
(c)

Fig. 2

Different geometries of timber frame walls encountered: (a) geometry variability in frontal walls (Cóias, 2007); (b) examples of connections used in Lisbon (Mascarenhas, 2004); (c) connections of timber frame wall in Chalkida, Greece

(a)

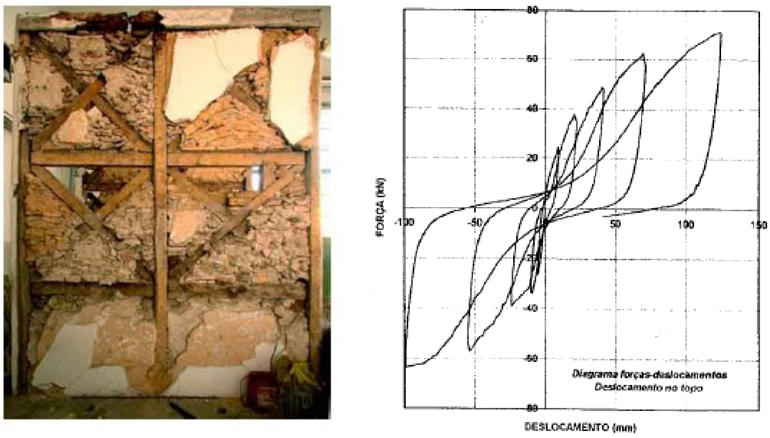
(b)



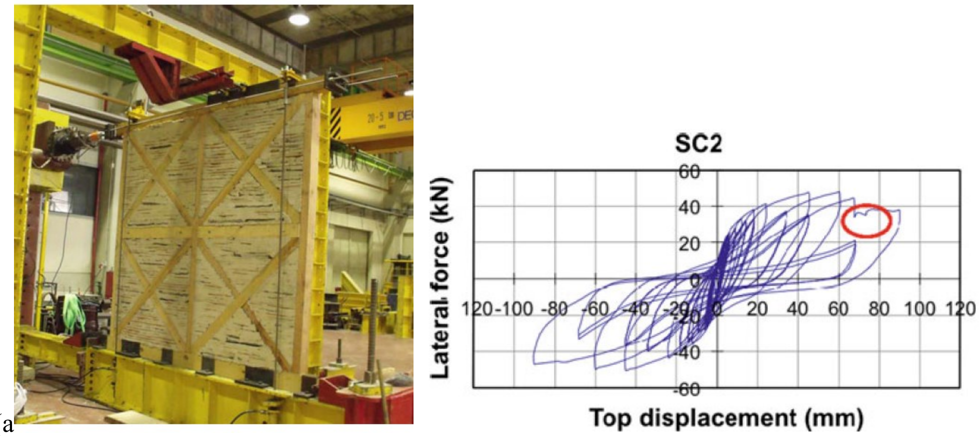
(c)

Fig. 3

Examples of damages in timber frame buildings; (a) out-of-plane collapse of masonry infill (Lefkada, Greece) (Makarios, 2006); (b) comparison of damages to traditional and modern building after the 1999 Duzce earthquake; (c) failure of connection in timber frame (1999 Kocaeli earthquake (Gülhan and Güney, 2000).



(a)



(a

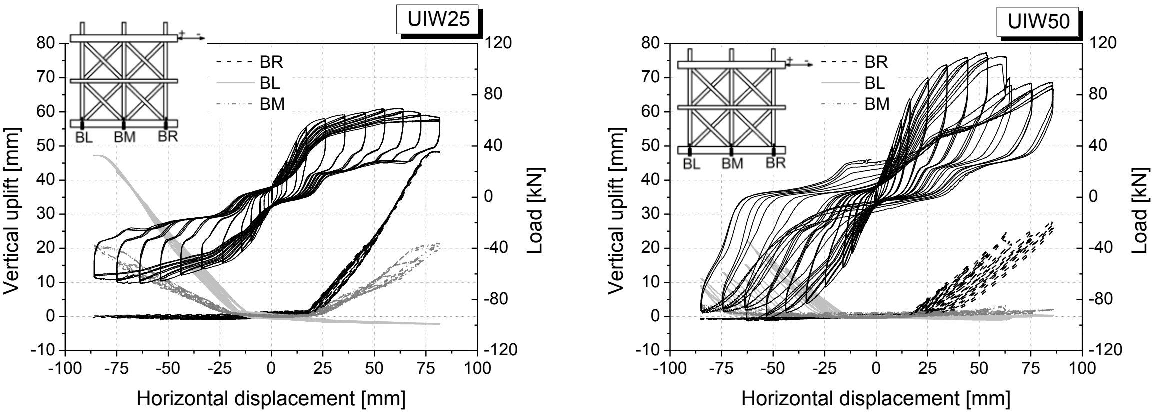
(b)

Fig. 4
Experimental testing of Pombalino frontal walls; (a) Santos (1997); (b) Meireles et al. (2012)

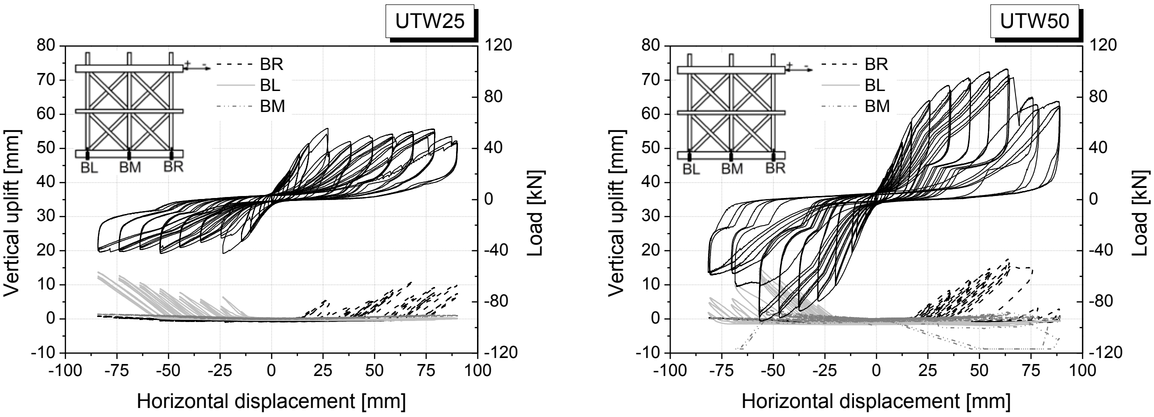
(a) (b) (c)

Fig. 5

Timber frame walls tested by Poletti (2013); (a) timber frame wall without infill; (b) timber frame wall with brick masonry infill brick; (c) timber frame wall with lath and plaster covering



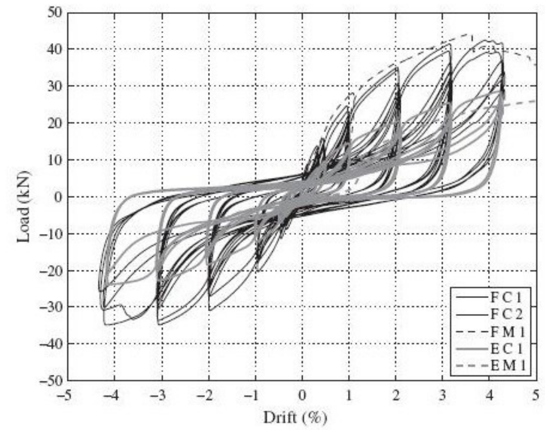
(a)



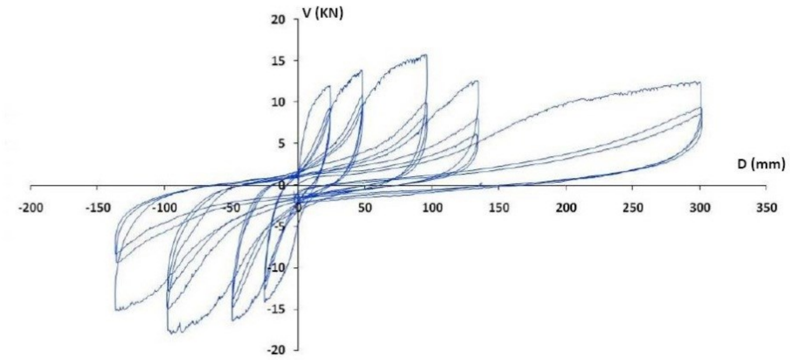
(b)

Fig. 6

Force-displacement diagrams obtained for *frontal* walls tested by Poletti (2013); (a) walls filled with brick masonry submitted to a vertical load of 25kN/post and 50kN/post; (b) empty walls submitted to a vertical load of 25kN/post and 50kN/post.



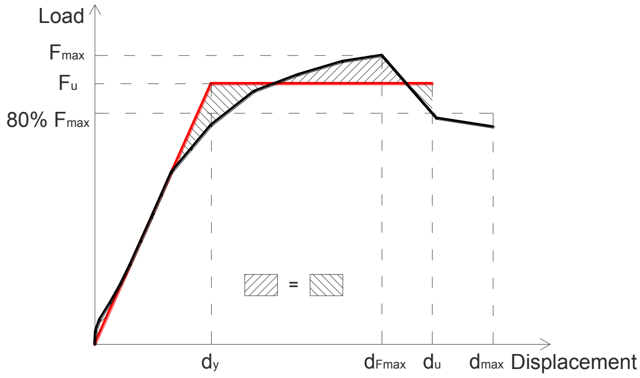
(a)



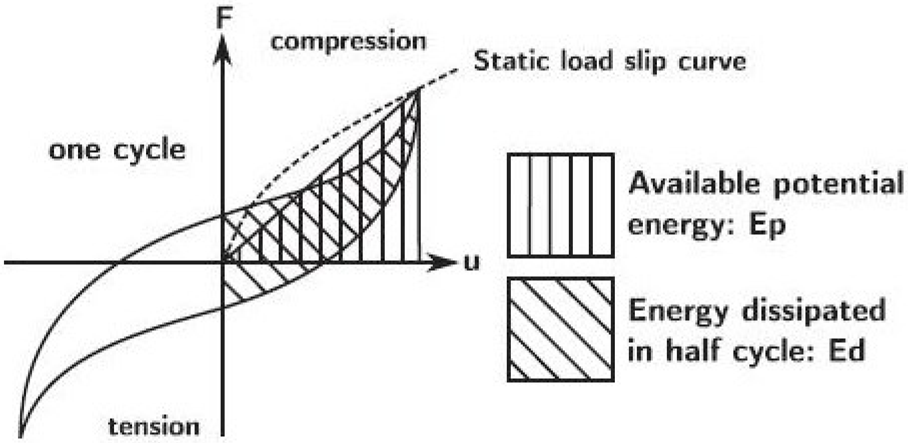
(b)

Fig. 7

Force-displacement diagrams obtained in distinct types of timber frame walls; (a) Vieux-Champagne et al. (2014); (b) Torrealva and Vicente (2012).



(a)



(b)

Fig. 8

(a) Bilinear idealization of the monotonic experimental envelop; (b) dissipated and input potential energy is a hysteretic loop

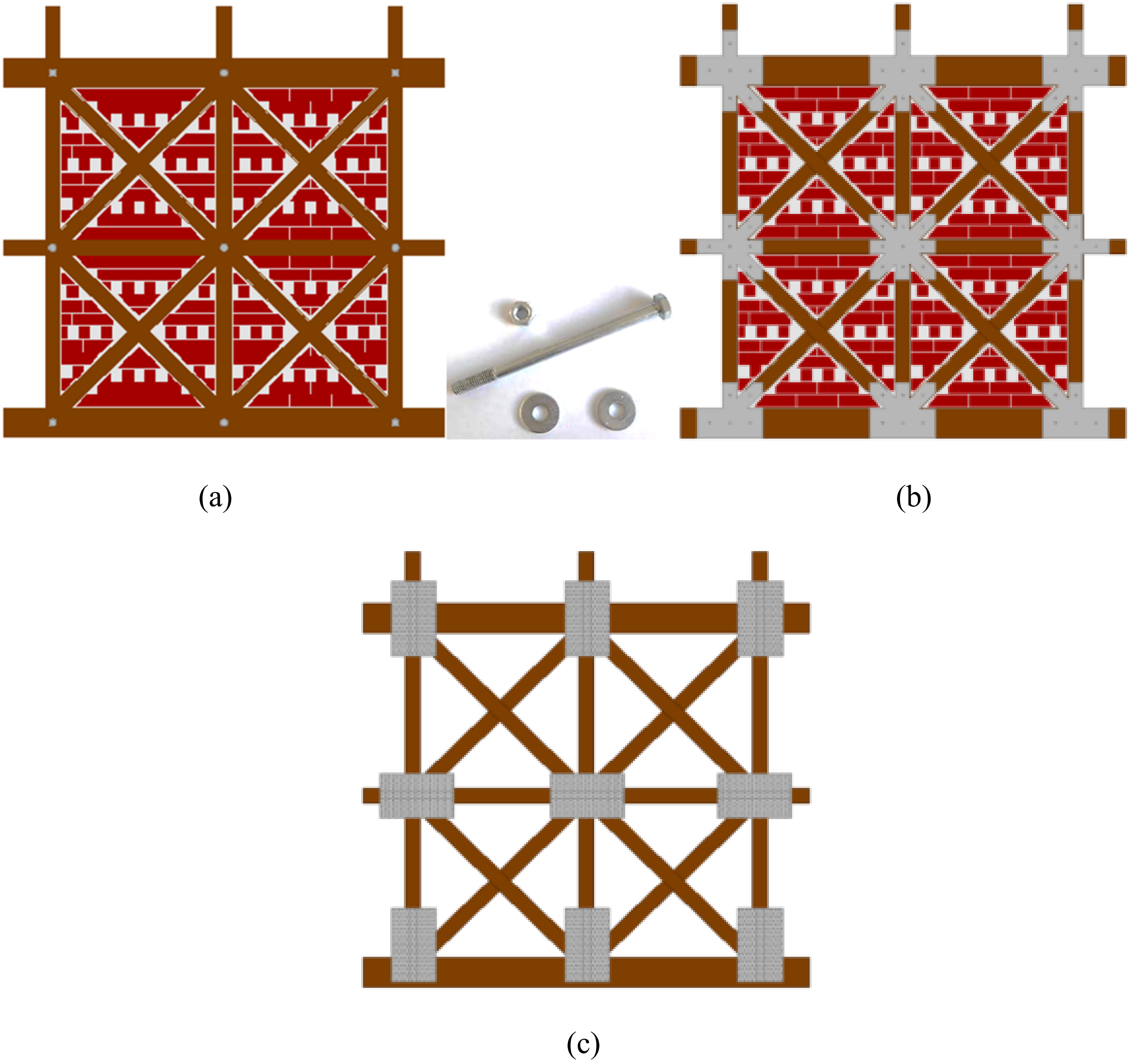


Fig. 9

Retrofitting solution: (a) steel bolts; (b) custom star shape steel plates; (c) commercial steel plates

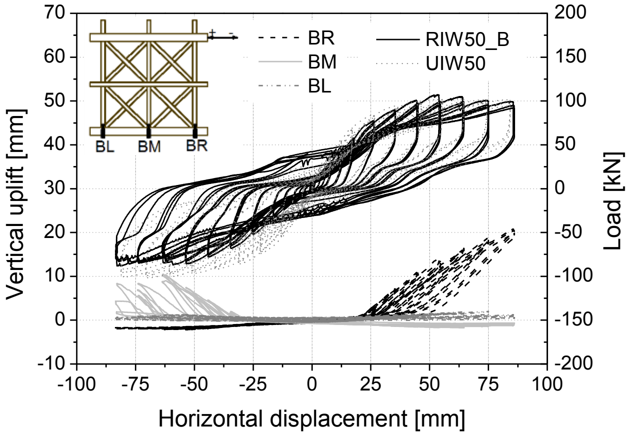
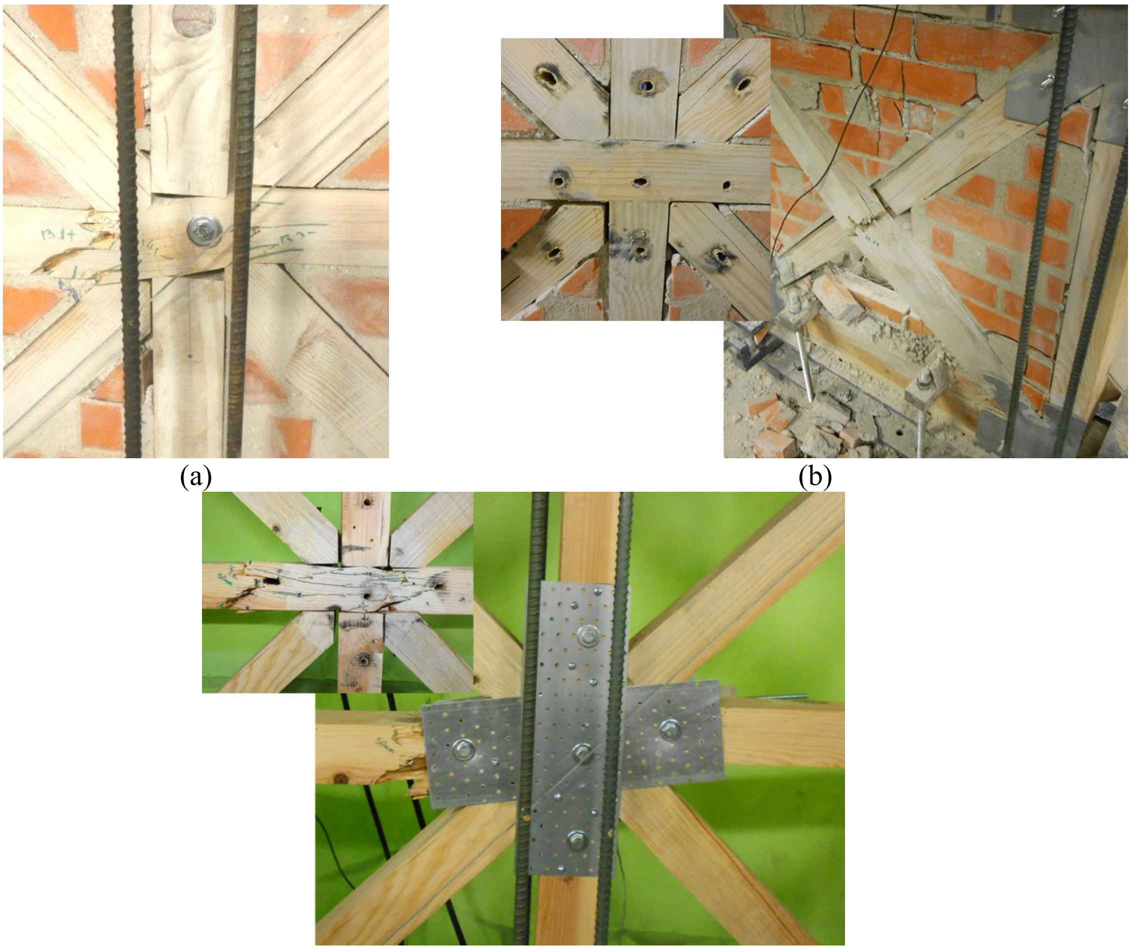


Fig. 10

Hysteretic force-displacement diagrams for walls strengthened with bolts, higher pre-compression load

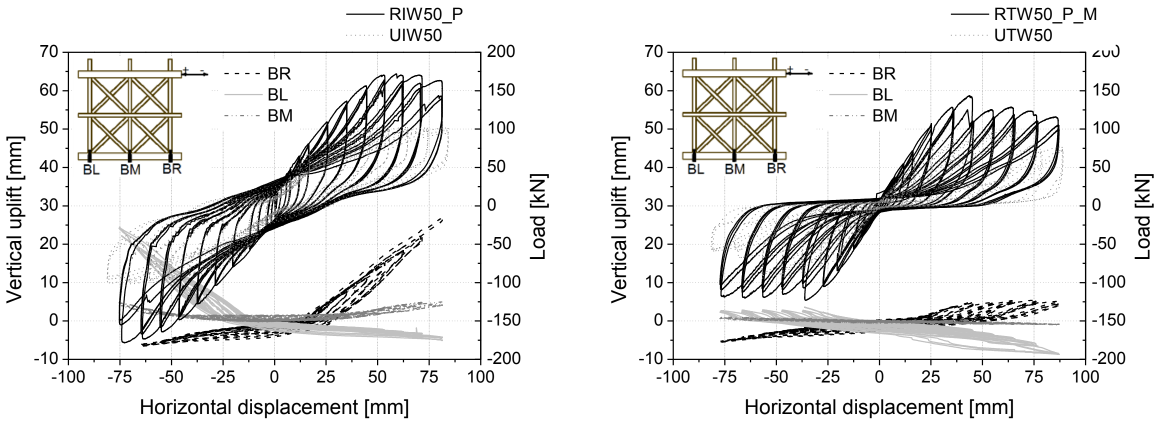


(a) (b)

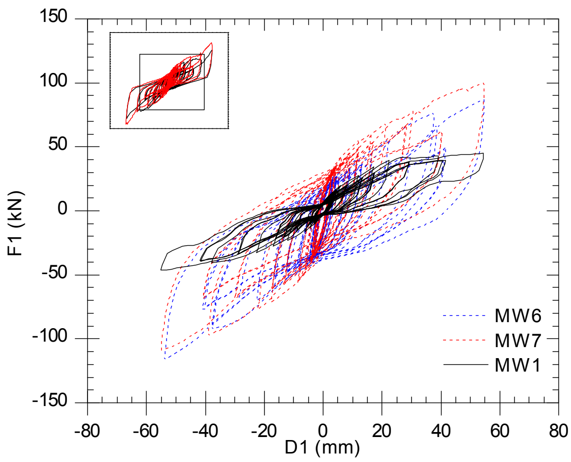
(c)

Fig. 11

Typical damages in retrofitted walls: (a) tearing off of central beam in a retrofitted wall with steel bolts (with brick infill); (b) failure of half-lap connection in the bottom cell in a retrofitted wall with custom steel plates (with brick infill); (c) failure of central connection in in a retrofitted wall with commercial steel plates (without brick infill)



(a)



(b)

Fig. 12

Comparison of force-displacement diagrams between unreinforced and retrofitted timber frame walls; (a) timber frame wall with brick masonry infill custom steel plates and unreinforced timber frame wall; (a) timber frame wall without infill with commercial steel plates and unreinforced timber frame walls (Poletti and Vasconcelos, 2013); (c) retrofitted timber frame walls with steel plates (MW 6 and MW6) and unreinforced walls (MW1) (Gonçalves et al., 2014)

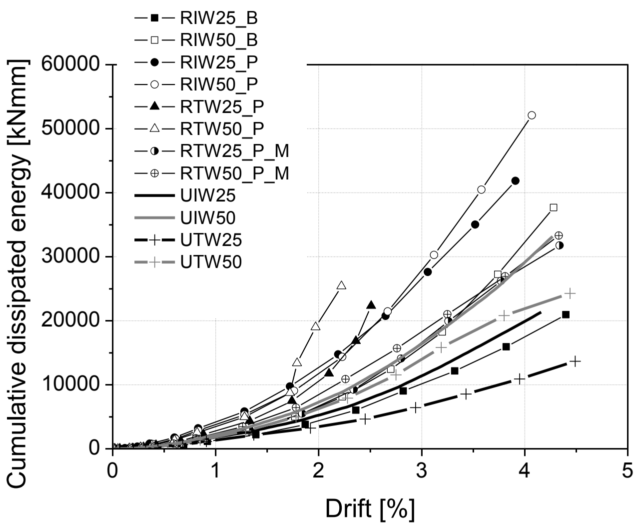
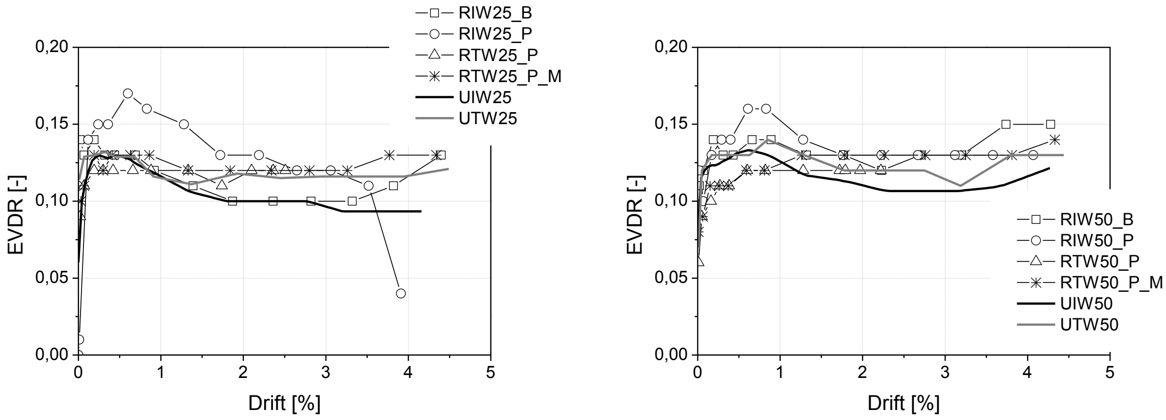


Fig. 13

Cumulative dissipated energy for all walls tested by Poletti (2013)



(a) (b)

Fig. 14

Equivalent viscous damping ratio: (a) lower pre-compression level; (b) higher pre-compression level