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Characterization of the seismic behaviour of traditional timber frame walls
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DECLARAÇÃO

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É AUTORIZADA A REPRODUÇÃO INTEGRAL DESTA TESE/TRABALHO APENAS PARA EFEITOS DE INVESTIGAÇÃO, MEDIANTE DECLARAÇÃO ESCRITA DO INTERESSADO, QUE A TAL SE COMPROMETE.

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ABSTRACT

Thesis title: “Characterization of the seismic behaviour of traditional timber frame walls”

Timber frame structures constitute an important cultural heritage of many countries, since they represent a typical anti-seismic construction adopted worldwide. Therefore, the preservation of these structures is of the utmost importance. Although recent earthquakes have pointed out the good seismic behaviour of this kind of structures, few experimental studies are available on the performance of traditional half-timbered walls and their retrofitting solutions. Aiming at filling this research gap and at better understanding the behaviour of these historic elements under seismic loads, an extensive experimental campaign has been carried out, performing in-plane cyclic tests on real scale half-timbered and timber frame walls, adopting connections and dimensions found in real structures and considering different infill types (brick masonry and lath and plaster). Moreover, considering possible damages caused by earthquakes, after the unreinforced tests, the walls were retrofitted and re-tested to compare the efficiency of the retrofitting solutions in terms of maximum load, ductility, cyclic stiffness, energy dissipation and equivalent viscous damping. Both traditional and innovative retrofitting techniques were adopted, namely bolts, steel plates and near surface mounted steel flat bars. Moreover, since the behaviour of the walls was governed by their connections, in-plane cyclic and pull-out tests were performed on traditional connections used in the walls. Additional retrofitting techniques were adopted, such as self-tapping screws and glass fibre sheets, to overcome some limitations found during the in-plane tests on the walls.

From the experimental results, an analytical hysteretic model was derived, based on the modification of existing models and calibrated on experimental results. This can be used in simplified numerical models to represent the hysteretic response of the walls.
Finally, numerical analyses were performed on timber frame walls with and without infill. The models were calibrated on the experimental results. Parametric analyses were then performed taking into account different variables.
RESUMO

Título da tese: “Caracterização do comportamento sísmico de paredes tradicionais com estrutura de madeira”

Os edifícios com paredes de madeira tradicionais constituem um importante património cultural de muitos países, uma vez que representam uma construção anti-sísmica típica adoptada mundialmente. Portanto, a preservação dessas estruturas é de extrema importância. Embora os terremotos recentes tenham apontado o seu bom comportamento sísmico, relativamente poucos estudos experimentais estão disponíveis sobre o desempenho sísmico das paredes tradicionais. Tendo como objectivo alargar a investigação e de obter uma melhor compreensão do comportamento destes elementos construtivos (paredes de frontal em edifícios Pombalinos) sujeitos a cargas sísmicas, foi realizada uma extensa campanha experimental composta por um conjunto de ensaios cíclicos no plano em escala real de paredes de frontal. Foram consideradas paredes sem preenchimento, utilizando ligações e dimensões encontradas em estruturas reais e considerando os diferentes tipos de enchimento (alvenaria de tijolo e fasquio). Por outro lado, considerando os possíveis danos causados em caso de terremoto, após os ensaios, as paredes foram reforçadas e ensaiadas novamente para comparar a eficácia das soluções de reforço em termos de carga máxima, ductilidade, rigidez cíclica, dissipação de energia e de amortecimento viscoso. Foram adoptadas técnicas de reforço tradicionais e também inovadoras, nomeadamente parafusos, chapas de aço e barras de aço inseridas ao nível das ligações. Além disso, uma vez que o comportamento das paredes é controlado pelas suas ligações, foram realizados ensaios cíclicos no plano e ensaios de pull-out em ligações tradicionais utilizadas nas paredes. Foram adoptadas técnicas de reforço adicionais, tais como parafusos auto-perforantes e folhas de fibra de vidro, para superar algumas limitações encontradas durante os ensaios cíclicos das paredes.

A partir dos resultados experimentais, foi derivado um modelo de histerese analítico, com base na modificação de modelos existentes e calibrado com base em resultados
experimentais. Este modelo pode ser utilizado em modelos numéricos simplificados para representar a resposta de histerese das paredes.

Finalmente, foram realizadas análises numéricas em paredes de frontal com e sem preenchimento. Os modelos foram calibrados com os resultados experimentais. Foram realizadas análises paramétricas tendo em conta diferentes variáveis.
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CHAPTER 1 INTRODUCTION

1.1 General framework

Timber and masonry are two of the most common materials used in construction since ancient times. These two materials appear combined as a constructive system since ancient civilizations (Tsakanika, 2006; Tampone 1996), being the masonry resisting to compressive stresses and the wood acting as ties resisting to tensile stresses. The timber elements appear embedded in masonry both at the floor levels and at different heights of the walls. Another constructive system that combines masonry and timber are half-timbered walls that can be seen in the historic centres of many cities in the world. These walls are commonly composed of a timber fame with bracing elements and with masonry infill with distinct types of materials and masonry bonds. These structures were even adopted in some countries as seismic-resistant construction, due to their good behaviour observed during past earthquakes, like in Portugal (frontal walls) in Pombalino buildings, in Greece and Italy (Mascarenhas, 2004; Touliatos, 2004; Vintzileou et al. 2007; Bianco, 2010). In particular, the frontal walls have a timber skeleton, characterized by bracings formed by St. Andrew's crosses connected to the main frame composed of posts and beams. The walls are connected to the external masonry walls and to the floors, thus giving to the buildings a greater capacity against horizontal loads, aiming at increasing their energy dissipation and ductility capacity.

After recent earthquakes (Turkey 1999, India 2005, Haiti 2010) timber frame buildings have gained some visibility and interest from the research community due to their good behaviour. They were able to withstand seismic actions sometimes better than modern reinforced concrete buildings. In India, part of the buildings rebuilt after the earthquake were timber frame ones (Langenbach, 2009), which gave the additional advantage that they could be built directly by the owner. In Pakistan, a guidebook for the construction of traditional timber-framed housing was developed by the United Nations and distributed to technicians...
and artisans (Schacher and Ali, 2010). In Haiti, a revised timber frame system was proposed to rebuild part of the constructions destroyed.

Even though witness exists of the ability of timber frame buildings or half-timbered walls to withstand seismic actions, little quantitative information is available on their real response. Moreover, many buildings have been abandoned or have been modified, altering their structure. Frequently, storeys have been added in subsequent years and openings were inserted in timber frame walls, compromising their shear resistance. A common alteration is the change of use of the structures, increasing the loads without taking into account the consequences. But these buildings constitute an important heritage of many countries and have to be preserved.

Another issue is the retrofitting of historic timber frame structures. Some examples exist of retrofitting performed without previously investigating its effect on the mechanical behaviour of the structural elements. With this respect, it is important to evaluate the performance of distinct retrofitting techniques both at the level of resistance and displacement capacity improvement, which should be based on experimental studies.

The PhD research carried out aims at filling a research gap in the characterization of the mechanical behaviour under seismic actions of timber frame walls characteristic of timber frame buildings and in particular of the Pombalino buildings which characterize Downtown Lisbon.

1.1.1 Objectives

The great motivation of this work is the need to better understand the behaviour of traditional timber frame walls used as an earthquake-resistant constructive system, particularly in Pombalino buildings. Thus, the main goal of this thesis is to acquire a better understanding of this type of elements under cyclic loading, representing in a simplified way the seismic actions. This knowledge is important to preserve the existing buildings which represent an important heritage to human kind as well as to propose a possible re-introduction of such a solution in modern construction.

In detail, the main objectives of the thesis are:

- Understanding of the different typologies of timber-frame buildings, their differences and their behaviour in recent earthquakes;
- Evaluation of the in-plane cyclic behaviour of traditional timber frame walls, based on an extensive experimental campaign;
- Evaluation of the in-plane and pull-out cyclic behaviour of traditional connections;
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- Definition of hysteretic models representing the cyclic behaviour of traditional timber frame walls to be used in detailed numerical simulation;
- Providing guidelines for numerical modelling of timber frame walls in a simplified way;
- Proposal of strengthening techniques for traditional timber frame walls.

The final goal of this work is to provide useful information on the behaviour of timber frame walls to be used both in practice and for numerical simulations.

1.2 Methodology

In order to accomplish the objectives presented above, an extensive experimental program was designed and carried out based on quasi static in-plane cyclic tests on timber frame walls.

From the experimental results, information about the seismic performance of timber frame walls based on the failure modes, lateral resistance, dissipation characteristics, ductility and viscous damping are provided. The experimental program also takes into account the analysis of the best retrofitting techniques for traditional timber frame walls. In particular, the specimen selected is an example of timber frame walls that can be found in the traditional Pombalino buildings in Lisbon. The experimental program is completed with the analysis of the behaviour of single traditional connections.

Numerical analyses have been carried out to improve the knowledge on the main parameters influencing the cyclic behaviour of the walls, to calibrate appropriate monotonic laws that enable a more accurate, but still simplified, numerical modelling of timber frame structures. The final goal of the research program is to provide guidelines for numerical analysis of Pombalino masonry buildings taking into account the influence of timber frame walls and to more accurately assess the vulnerability of these buildings.

1.3 Outline of the thesis

Besides the present introductory chapter, this thesis is divided in eight more chapters and two appendixes. The organization is as follows:

Chapter 2 presents a detailed literature review on timber frame buildings focusing on the Portuguese example but also extending to the typologies that can be found in the rest of the world. The characteristics of these buildings are described as well as their behaviour in recent earthquakes.

Chapter 3 presents the results on the characterization of the materials used for the construction and retrofitting of the wall specimens. In particular, timber, mortar, bricks and
masonry were characterized considering the appropriate tests in order to obtain important parameters, such as compressive strength, bending strength, shear strength, density and fracture energy.

Chapter 4 presents the results on the experimental campaign performed on unreinforced timber frame walls. In-plane cyclic tests were performed on real scale specimens studying the influence of the vertical pre-compression level and of the type of infill. In particular, three types of specimens were considered: timber frame with masonry infill, timber frame with lath and plaster and timber frame with no infill. A detailed analysis of the hysteretic behaviour of the walls, their deformation capacity and relevant seismic parameters, such as stiffness, dissipated energy, ductility and viscous damping, is presented. Additional information is presented in Appendix A.

Chapter 5 presents the results of tests performed on retrofitted timber frame walls. Retrofitting was applied to the walls already tested. In particular, bolts, steel plates and Near Surface Mounted (NSM) steel flat bars were used. The retrofitting solutions adopted were applied both to masonry infill walls and to timber frame walls without infill. A detailed analysis of the results is presented, the retrofitting solutions are compared with the results of unreinforced walls and advantages and disadvantages of each technique are pointed out. Additional results are presented in Appendix B.

Chapter 6 presents the results of in-plane cyclic and pull-out cyclic tests performed on traditional connections used in timber frame walls. Tests were performed on both unreinforced and retrofitted specimens. In particular, additional retrofitting solutions were experimented, such as self-tapping screws and glass fibre sheets, in order to try to overcome some limitations found during the in-plane tests on the walls at the level of the bottom connections.

Chapter 7 presents an analytical hysteretic model derived and calibrated from experimental results and based on the modification of existing models. Such a tool can be used in simplified numerical models to represent the hysteretic response of the walls. A comparison between analytical and experimental results is presented.

Chapter 8 presents numerical analyses performed on timber frame walls with and without infill. The simplified models were calibrated on the experimental results. Parametric analyses were then performed taking into account variables such as properties of timber, stiffness of the infill and wall geometry.

Chapter 9 presents a summary of the research carried out with the main conclusions and suggestions for future developments.
CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

Timber and masonry structural solutions have been used since ancient times and they are important built cultural heritage that still exists today. The conservation of ancient construction heritage is a demand of modern societies and a great research effort has been made on ancient materials and structures in order to better understand their mechanical behaviour and to propose remedial and strengthening measures.

A historic construction that is widespread around the world is timber frame construction. Timber frame buildings are characterized by a timber frame filled with an infill which is mainly rubble stones or masonry. Since this type of construction is spread throughout the world, it is important to point out the similarities and differences in order to better understand their use and behaviour. In particular, these buildings are considered to be seismic-resistant structures and, thus, attention should be given to their response to seismic actions.

In this chapter, typologies of timber frame construction at the international level are presented and their performance in recent earthquakes is discussed.

2.2 Timber frame buildings: their use and dissemination

The origin of timber frame buildings probably goes back to the Roman Empire, as in archaeological sites (Herculaneum) half-timbered houses were found and were referred to as Opus Craticium by Vitruvius (Langenbach, 2009). But timber was used in masonry walls even in previous cultures. According to Tsakanika (2006) and Tampone (1996) in the Minoan palaces in Crete (Knossos and Phestos) timber elements were used to reinforce masonry. Half-timbered constructions spread not only throughout Europe, such as Portugal (edificios pombalinos), Italy (casa baraccata), Germany (fachwerk), Greece (ksilopiki tichopiia), France (colombage), Scandinavia (bindingverk), United Kingdom (half-timber), Spain (entramados), but also in India (dhaiji-dewari) and Turkey (himis) (Côias 2007; Langenbach
Examples can be also found in Peru, in the USA, where German immigrants brought with them their know-how, and in Haiti (gingerbread houses), where French immigrants brought their traditional colombage construction.

2.2.1 The Portuguese tradition

Portuguese building tradition mainly consisted of masonry buildings, with few examples of timber buildings, such as the timber houses of fishermen near Aveiro, which are now almost fully destroyed. Timber was mainly used for trusses and floors, which can be seen all over the country (Branco, 2008). The adoption of timber as a structural material for vertical elements became common only after the destruction of Lisbon due to the strong earthquake in 1755. The map of Lisbon before the earthquake shows an irregular plan (Figure 2.1a), with narrow streets connecting the two main squares, Terreiro do Paço and Rossio. When the devastating earthquake, and subsequent tsunami and fires, hit Lisbon in 1755, the Downtown was completely destroyed (Figure 2.1b) and a reconstruction plan was put into action. The parts of the city which were rebuilt were the Baixa and Chiado neighbourhoods (Figure 2.1c). The prime minister of the time, Marquis of Pombal, appointed to engineers and military architects the elaboration of reconstruction plans of the city. Engineer Manuel da Maia presented numerous proposals, which are discussed in his Dissertation ("Dissertação" 1755-1756), divided in three parts. The new regulations adopted provided rules for urban, architectonical and structural design, such as minimal distances between buildings, typology of façades, width of roads and sidewalks, height of the buildings, orientation of the buildings, structural system and creation of blocks (Mascarenhas, 2004).

In Pombalino Cartulary ("Cartulário Pombalino" 2000), original drawings of most façades can be found, but the original plans were lost in a fire during the XIX century. The façades were very regular (Figure 2.2a), with little variability between one building and the next in the main streets (Appleton and Domingos, 2009; Mascarenhas, 2004). The buildings may have up to 5 storeys, with decreasing height of the walls. The little that had survived of Downtown Lisbon was destroyed and the new plan was put into action, building on top of the ruins in order to avoid future floods from the Tejo River.

The buildings that derived from the proposed plan, called Pombalino buildings, were characterized by external masonry walls and an internal timber structure (Figure 2.2b), named gaiola (cage), which is a three dimensional braced timber structure (Figure 2.2c), similar to many half-timbered buildings that can be found in several countries in Europe. The gaiola consists of horizontal and vertical elements and diagonal bracing members, forming the typical X of St. Andrew’s crosses, which have a dissipative function. The timber frame walls are usually filled, either with rubble or brick masonry (Figure 2.2d) or even mud and
hay. Plaster was applied to frontal walls, creating small cuts in the timber elements so that mortar could adhere better. Generally, mortar was weak and mainly lime mortar was used.

![Figure 2.1 Lisbon before and after the earthquake: (a) map of Lisbon before the earthquake; (b) destruction caused by the earthquake and subsequent tsunami and fires; (c) map of Lisbon showing area and plan of reconstruction.](image)

In the first Pombalino buildings the walls around the nucleus of the stairs were masonry walls, while in later years they were substituted by half-timbered walls. Half-timbered walls can be parallel or perpendicular to the façades, but it is most common to find them in the parallel direction since they receive the vertical load of the floors, whose beams are oriented perpendicular to the façades (Appleton and Domingos, 2009). The buildings were built in blocks, in order to offer a better structural stability.

The ground floor consists of stone masonry columns supporting stone arches and vaults made of clay bricks, above which, in the first floor, the gaiola develops (Figure 2.2b). This solution was adopted in order to prevent fire propagation to the upper floors. Early Pombalino buildings had a constant width of the stone walls of the façade, while in earlier or later buildings the width decreases along the height (Mascarenhas, 2004).

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1 http://www.skyscrapercity.com/showthread.php?t=276018&langid=6 (last accessed on 2013/08/18)
2 http://bloguehistorico6.wordpress.com/category/lisboa-pombalina/ (last accessed on 2013/08/18)
3 http://www.ordemengenheiros.pt/pl/centro-de-informacao/dossiers/historias-da-engenharia/dimensoes-e-relicas-intemporais-do-terramoto-de-1755/ (last accessed on 2013/08/18)
This construction typology was not completely new to Lisbon, as in the oldest parts of the city, near the Castle, a similar simplified construction can be also found. The innovation consists of the improvement of the system, as well as the standardization of the constructive practice in Lisbon (Mascarenhas, 2004).

Figure 2.2 - Measures adopted for new buildings built: (a) typical façade (Santos, 2000); (b) structure of whole Pombalino building (Mascarenhas, 2004); (c) disposition of frontal walls in floor plan (Cóias, 2007); (d) example of frontal wall in a Pombalino building in Lisbon

The internal walls of the gaiola (frontal walls) may have different geometries in terms of cell dimensions and number of elements (Figure 2.3a), as it depended greatly on the available space and the manufacturer’s customs. The timber elements are notched together or connected by nails or metal ties. Traditional connections used for the timber elements varied significantly in the buildings: the most common ones were mortise and tenon, half-lap and dovetail connections (Figure 2.3b). Variability exists in the sectional dimensions of the elements themselves: the diagonal members are usually smaller (10×10cm or 10×8cm), whilst the vertical posts and horizontal members are bigger (usually 12×10, 12×15cm and 14×10cm or 15×13, 10×13 and 10×10cm respectively). The thickness of frontal walls can vary from 15 to 20cm (Mascarenhas, 2004; Cóias, 2007). In some situations, the posts of the

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4 MONUMENTA http://www.monumenta.pt/
frontal wall extended for more than one floor (Appleton and Domingos, 2009), even if this solution does not appear to be the most common one. Most of the times the posts are not continuous and their position between one floor and the other may vary. The extension of the posts between two floors is made at the level of the floors by connecting them to the horizontal beam (Figure 2.3c), either adopting two independent connections between the two posts or creating a continuity with a scarf joint. This beam has a double function: it constitutes the top beam of the frontal walls and also serves as a support for the timber floor beams.

![Figure 2.3 - Details on frontal walls: (a) geometry variability in frontal walls (Cóias, 2007); (b) examples of connections used (Mascarenhas, 2004); (c) continuity of post between two floors (Mascarenhas, 2004)](image)

The frontal walls act as shear walls in the building. Other internal walls are purely partition walls (costanerias or tabiques) and are made of wooden panels.

The peculiarity of this type of buildings 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, assuring the resistance of the timber floors and keeping the building standing. It has to be considered that in Pombalino buildings a minimal timber skeleton was present also in the external masonry walls to facilitate and improve the connections with the floors and the inner shear walls (Mascarenhas, 2004). This is a construction technology that can improve considerably the out-of-plane behaviour of facades under seismic events by avoiding out-of-plane premature collapse of the façades.
The Pombalino half-timbered walls were built to provide adequate resistance of the new buildings to seismic loading. However, it should be pointed out that their seismic efficiency has never been tested under onsite real conditions, as no other great earthquake has hit Lisbon since their construction.

Approximately a hundred years after the earthquake the building practice changed, getting worse from a seismic point of view. Pombalino buildings, where a complete gaiola structure is present and it is expected to be efficient against seismic actions, were progressively replaced by Gaioleiro buildings, where the timber structure does not exist and therefore structurally represent a worse construction quality (Mendes and Lourenço, 2010).

Though this constructive system is typical of the city of Lisbon, half-timbered structures can be also found in historic city centres in the north of the country, for example in Porto, Vila Real, Braga and Guimarães, even if the seismic hazard is very low in these locations. Their use is attributed to the generalization of the Pombalino construction technology all over the country due to its popularity as a lighter and less expensive construction system. Nevertheless, in these cities the constructive system was different. The ground floor continued to consist of stone masonry, typically granite, while in the upper storeys the external walls were half-timbered (Figure 2.4a).

The internal walls were either half-timbered or they were partition timber walls, typically lath and plaster walls. The same connections found in Pombalino buildings were used (Figure 2.4b), even if in rehabilitation works, the detail given to the accuracy of the execution of the connections is not the same of the Pombalino buildings.

Considering the elements dimensions, in the buildings surveyed in Guimarães, the posts and main beams had a width of 15cm, while the diagonals and the secondary elements of 7cm. Sometimes, apart from brick and rubble masonry, hay was used as infill.

![Figure 2.4 - Half-timbered houses in the North of Portugal: (a) restoration of a historic house in Guimarães; (b) traditional connections used](image)
2.2.1 Lath and plaster (*Taipa de fasquio*)

An alternative to brick masonry infill is lath and plaster. It is a kind of infill that can be found in frontal walls as well as timber partition walls. This type of infill is typical of the north of Portugal, where timber frame walls can be found consisting of the timber frame, with or without the diagonals, over which the timber strips are nailed directly (see Figure 2.5a) (Mascarenhas, 2004). Alternatively, wooden boards are nailed on the frame, usually in two layers, one vertical and one diagonal, and subsequently the wood strips are nailed to the boards (Figure 2.5b,c,d). The latter example is typically called *taipa de fasquio* and it is a typical partition wall that can be found all over the country.

Figure 2.5 - Typical lath and plaster walls: (a) example of "frontal" wall with lath and plaster (Mascarenhas, 2004); (b) example of taipa de fasquio in Porto, prior to application of plaster⁵; (c) taipa de fasquio in Guimarães; (d) details of laths and plaster applied to partition wall⁴

Lath and plaster infill is a good alternative to masonry infill as it links together the various members of the wall, adding stiffness and guaranteeing a more compact behaviour.

2.2.2 European examples

Half-timbered buildings are particularly popular in Europe, not only in seismic regions (Greece, Italy, Portugal), but also in Northern-European countries (Germany, England,

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Scandinavia) due to the abundance of wood, see Figure 2.6a,b. The know-how was then spread throughout Europe and examples can be found even in Austria, Switzerland, Poland, Hungary, Romania, Albania, Yugoslavia and Macedonia (Figure 2.6c,d).

In Italy half-timbered buildings were present since the Roman empire, as testified by the buildings left in archaeological sites in Herculaneum (Figure 2.7a), the *opus craticium* system, which combines a timber frame with the *opus incertum*, i.e. rubble masonry. Here, the timber frame was not thought of as being seismic resistant, but only as a mean to lighten the upper storeys of the structure. Even Julius Caesar adopted this constructive method in Gaul (Gallia), as a mean to strengthen the walls with timber beams (Bianco, 2010).

Half-timbered construction became more systematic after the 1783 earthquake that destroyed Reggio Calabria. The new system, called *casa baraccata* (literally, “baracca” means shack) was adopted by the authorities by imposing 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. The term “baracca” derives from the fact that, in previous seismic events, temporary timber houses were built for the refugees (Ruggieri, 2005).

\[http://en.academic.ru/dic.nsf/enwiki/368176\] (last accessed on 2013/08/18)
Similarly to what happened in Lisbon, the government (at the time, the Bourbon dynasty was ruling the south of Italy) appointed engineers to develop rules for the reconstruction of the region. In 1784, the Royal Instructions (“Istruzioni Reali”) were emanated and they consisted of rules to be applied to the new buildings. Among the rules included, there were instructions on the exterior aspect of the buildings (it had to be simple and elegant), on height of buildings, on the width of streets, on rules on construction of balconies, on domes and bell towers and on the addition of an internal timber skeleton. Notice that these were the first official norms for seismic design.

Even though timber framing was already common in Calabria, a standardized type of half-timbered building was introduced by Giovanni Vivenzio, the court’s physicist. This choice was born by the observation of the good seismic behaviour of existing half-timbered buildings, such as the palace of Nocera, built in 1638 (Bianco, 2010). Vivenzio proposed a 3-storey building with a timber skeleton aiming at reinforcing the external masonry walls, avoiding their premature out-of-plane collapse (Figure 2.7b). The timber frames constituted the shear

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7 http://rete.comuni-italiani.it/blog/14539 (last accessed on 2013/08/18)
walls, presenting a bracing system of S. Andrew’s crosses, similar to what can be found in Lisbon (Tobriner, 1997). This system was also adopted for exterior walls in this type of buildings. Similarly to the Portuguese example, Vivenzio also proposed a construction by blocks, in this case of three buildings. The idea is that the central building has a higher height and the lateral ones act as buttresses. This disposition allows for symmetry in the two directions, ensuring a similar stiffness for the two directions. Moreover, the mass centre is lower in this way, leading to lower accelerations (Ruggieri, 2005). Vivenzio originally planned to have the timber elements visible but it was then decided to cover the timber elements with stones in order to preserve them from decay (Figure 2.7c). The timber frame was embedded in the external masonry and for partition walls it was covered with wooden strips (Bianco, 2010).

A difference to the Portuguese solution is that the pillars are not constituted of a unique timber element, but of either four or two square elements, depending on the amount of vertical load applied to that pillar (Tampone, 1996). Moreover, in these buildings the position of the vertical timber posts is constant from the foundation to the roof, being anchored in the foundation (especially in the buildings built after 1908) (Bianco, 2010).

Another important earthquake that affected the south of Italy occurred in Messina in 1908. After this event, the new standards of 1909 included rules for foundations. The standards suggested to prepare the foundation ground with a foundation slab and carry out diggings if necessary. Rock foundations or a firm soil are preferred. Moreover the posts had to be well fixed to the stone or to the foundation slab for at least 80cm. They should be burned at the extremities to prevent decay. This solution proved, in later years, not to be sufficient, as humidity problems led to the decay of timber elements in the buildings. The cross section adopted for the vertical and horizontal elements was approximately of 16×12cm and of 15×8cm for the diagonal bars. The wood had to be treated against woodworms, be without bark and burnt and the spacing of the posts ranged from 1 to 1.2m. All timber connections were nailed. The timber frame of outer walls could be embedded in the masonry, it could have 25cm of masonry from the outside and a layer of 5cm of plaster from the inside (baraccata building) or it could have masonry only as infill of the timber frame (framed building) (Branco, 2010). The diagonals were not always present: in some cases only the main frame was present (horizontal and vertical elements); in other cases St. Andrew’s crosses are present at the first floor only at the corners and at the second floor throughout the length of the wall, but only along the upper part. The standards do not give specific indications on how to build the frame, thus the great variability found in the existing structures. Often, the timber frame is in the middle of the masonry, so it is covered on both sides. The standard gave details on the connections between the horizontal members and
the masonry walls, but they were rarely put into practice. The timber frame was adopted in the baraccata house until 1914, when it was substituted by a reinforced concrete frame (Bianco, 2010).

Half-timbered and timber frame constructions can be found even in the North of Italy, due to the abundance of wood in those mountainous regions (Ceccotti et al., 2006; Ientile, 2007). Wood and stone were the only building materials available in those regions (Ceccotti et al., 2006) and the adoption of timber frame structures was quite natural. Even in these regions, variability is found in the geometry of the buildings. The structures are usually comprised of three storeys, with a stone ground floor, similarly to what is found in other countries (Figure 2.7d). Two timber frames can be distinguished: (1) the first one made with the gardiz technique, which consists of a timber grid, where the posts are driven into the ground and it is covered with mud and let dry in the sun. The frame, which is usually made of spruce, has dovetail joints and constitutes the load-bearing structure. The infill consists of lime mortar and rubble stones and a trellis of hazelnut branches is applied to improve the connection to the infill. Above the trellis, a layer of plaster is applied, reaching a final thickness of 20 cm (Ceccotti et al., 2006). Some variability is found in terms of connections (the joints can be simplified, but iron nails are added) and in terms of frame geometry (diagonal bracing elements can be introduced, stiffening the main frame); (2) in the second type of frame, the diagonal bracing elements are always present and the trellis supporting the infill is no longer present. The connections of the load-bearing frame are lapped joints and nails are used. The timber elements have a square cross section and the diagonals are not connected to the main frame but are simply inserted inside the frame. Infill consists also of rubble stones and lime mortar (Ceccotti et al. 2006).

Timber frame structures are present even in Piedmont, particularly in the Susa valley, dating back to the 17th and 19th centuries (Ientile, 2007). The buildings have three to four storeys. Also in this case, the limitation of local materials led to build using stones and wood, which were abundant. The ground floor is once again in stone masonry, while the upper storeys are in timber: semi-squared trunks were used for the frame and the infill was either gypsum and lime mortar or wooden planks (Ientile, 2007).

Another country that uses timber frame buildings as a seismic resistant solution is Greece. Half-timbered buildings were common all over Greece in different periods, as reported by many authors (Makarios et al., 2006; Vintzileou et al., 2007; Tsakanika, 2006; Touliatos, 2004). Examples of this system are the monastic buildings in Meteora and Mount Athos, the post byzantine (Ottoman period) buildings in Central and Northern Greece and the traditional buildings in the island of Lefkas.
Timber has been used together with masonry in Greece since the Minoan period (Tsakanika, 2006). Many examples can also be found of timber-tied masonry constructions in all of Greece, for example in Pilio, Zagoria, Veroia, Rhodes (Vintzileou, 2008). Generally, ties are placed on both sides of the masonry wall and tied together with timber elements that pass through the thickness of the walls (Vintzileou, 2008).

This constructive solution evolved from the use of only horizontal timber elements to tie masonry and prevent the propagation of cracks to a heavier timber frame, adopting both vertical and horizontal timber members to reinforce masonry (Tsakanika, 2006). In the island of Crete, the palace of Knossos is an example of timber-tied masonry. Through in situ inspections, Tsakanika was able to prove the presence of timber elements, indicated by the presence of “voids in the walls, mortises and horizontal beddings curved on dressed stones to provide a resting place for the connection of horizontal timber elements” (Tsakanika, 2008). Timber has been used to tie masonry adopting vertical and horizontal embedded timber elements (Figure 2.8a) and possibly they were connected with half-lap joints (Tsakanika, 2006). Two levels of horizontal timber beams tying the masonry could be distinguished, one over the lintels of doors and windows and one just underneath the floors (Tsakanika, 2006).

A similar construction can be found in the island of Santorini, where in Akrotiri a timber skeleton was inserted in the masonry structure (Tampone 1996). The buildings date back to the Bronze Age, as the ones in Crete. In Akrotiri, the need to use timber was due to the addition of a new two-storey wall to existing constructions (Tsakanika, 2006).

The half-timbered buildings in the island of Lefkas are different from those present in other regions of Greece. Here, a local structural system was developed before the 19th century and it demonstrated to be able to sustain seismic actions after buildings built with this system showed a good seismic performance during the earthquake of 1821 (Vintzileou et al., 2007). During the same earthquake, the existing masonry buildings collapsed. Based on this, British Authorities (which ruled the Ionian Islands at the time) imposed new rules, developed from the aforementioned local system (Code of construction, issued in 1827) (Vintzileou et al., 2007; Makarios and Demosthenous, 2006). The rules provided guidance on the selection of building materials, thickness of stone masonry walls (at the ground floor), storey height and distance between adjacent buildings (Vintzileou et al., 2007; Makarios and Demosthenous, 2006). It is to be noticed how these guidelines are quite similar to those adopted, some decades earlier, in Portugal and Italy. The local structural system which was used for the development of the code consisted of a stone masonry ground floor above which one or two timber-framed storeys developed (Figure 2.8b).
This disposition is typical of half-timbered buildings in seismic regions. 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 (Figure 2.8c), since they were connected to the timber-framed structure of the upper storeys (Vintzileou et al., 2007; Touliatos, 2004). Simpler timber walls are used as partition walls in the upper storeys, consisting of vertical studs that are nailed to the floor and ceiling and canes or laths are nailed to them and the void inside is filled with dried sedge. Plaster is applied on top of these walls (Porphyrios, 1971).

The geometry of the timber framed walls usually consisted of horizontal and vertical timber elements (from cypress trees) and some sparse diagonal elements (Vintzileou et al., 2007; Makarios and Demosthenous, 2006). The outer half-timbered walls are connected to the masonry walls of the ground floor through timber beams, located along the perimeter of the wall. The timber beams of the floor are connected to the walls, either half-timbered or stone masonry, through metal ties. Corner connections were usually strengthened with additional timber elements (Figure 2.8c). Moreover, the timber elements of the outer walls are over-dimensioned and studies showed that the timber columns of the ground floor can safely bear the whole vertical load even without the presence of the stone masonry wall (Vintzileou et al., 2007).

Commonly to other countries, half-timbered constructions are present in regions with highly wooded areas, as is the case of Kastoria (Tampone, 1996). But half-timbered buildings are not limited to wooded areas given that they can be found in Thessaloniki, Athens, Xanthi, Veroia and many other cities. In Athens, an example is documented (Tsakanika and Mouzakis, 2010) of a post-Byzantine mansion where the ground floor is in stone masonry with timber ties inserted in five levels of the wall, while the upper storey presented half-timbered walls with some diagonal bracings (Figure 2.8d). In Chalkida, similar buildings can be found, with a ground floor consisting of timber-tied masonry and upper floors of half-timbered walls (Figure 2.8e). In the half-timbered walls mainly notched connections and nails were adopted (Figure 2.8f) and half-lap connections were used to connect the posts to the top beam. In Thessaloniki, half-timbered buildings with masonry infill and regular diagonal bracings can be found. In Mount Athos, half-timbered buildings present a timber skeleton with regularly spaced horizontal elements and diagonal bracing elements (Ignatakis and Eftichidis, 2008).

In the rest of Europe examples of half-timbered constructions can be found mainly due to the highly available materials and the simplicity of execution. England has a great tradition of half-timbered houses, as they are characteristic of the medieval and Tudor period (Figure 2.9a). Typically, oak was used due to its abundance in English forests. These buildings are not present all over England, but only in the former timbered districts (central, western and southern England). In the north, stone was traditionally used, since it was easily available

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8 Ross D. Half-timbered houses. http://www.britainexpress.com/History/half-timber.htm (last accessed on 2013/07/06)
(Jackson, 1912). The term "half-timbering" refers to the fact that the logs were usually halved or cut down to a square section. In other European countries, such as Romania and Hungary, where no such hardwoods were available, whole logs were used\(^9\). In the early examples that still exist today, dating back to the 15\(^{th}\) century, the vertical posts are quite close together (Figure 2.9b), but with years they are placed further apart. Moreover, in subsequent years, diagonal and curved timber elements are placed, creating decorative patterns on the walls. The timber elements are all dowelled and pinned with oak pins (Jackson, 1912).

Different kinds of infill were used in England, namely wattle-and-daub, with a net of branches or canes covered by clay mud, laths and plaster or bricks. Generally, the connections used were mortise and tenon\(^8\), linking together the posts with the horizontal members.

By the 17\(^{th}\) century, wood was in short supply in England, since it had been exploited for building, heating and other uses. Wood was also greatly used in the expansions of the British merchant fleet. Moreover, the introduction of alternative construction techniques, such as brick masonry, which became easily available after the Tudor period, constituted an alternative to this construction technique, even if it is still used.

In Germany, half-timbered houses can be found all over the country and German emigrants took this tradition with them when they left the country for the USA or for Eastern European countries. Different timber frame styles can be found, characterized by a varying number of storeys and geometry of the timber frame (Figure 2.10a). Distinct types of infill materials can be used, namely brick masonry, adobe, or wooden planks (Figure 2.10b) (Bostenaru, 2004).

In Germany, this building method was introduced in the 7th century and it flourished

\(^9\) http://www.e-architect.co.uk/birmingham/half_timbered_houses.htm (last accessed on 2013/08/18)
particularly in the 16th and 17th century\textsuperscript{10}. The main styles are the Alemannic one, in the south-western parts of the country, the Lower Saxonian in the north and the Franconian style, in Bavaria and the Rhine-land, which extended to Poland and to the French Alsace\textsuperscript{11}.

![Figure 2.10 - German half-timbered tradition: (a) example of timber frame in the Lexikon der gesamten Technik (1904); (b) waddle and daub infill\textsuperscript{12}](image)

The Alemannic style is characterized by especially large spacing between the posts and by narrow strips of roofing projecting between different storeys. This style developed from the Alemannic floor-type building, which had wide column positions. The framework is heavier than in other styles because of the distance among the posts\textsuperscript{10}. In the 15\textsuperscript{th} and 16\textsuperscript{th} century more and more construction methods were adopted from Central Germany (Franconian style) becoming almost identical by the 17\textsuperscript{th} century\textsuperscript{11}.

In the Lower Saxonian the posts are nearer and are connected at right angles by beams. Few diagonal members are present. These buildings are based on the Lower German hangar-houses, which constituted a unique place where people, animals and their harvest lived in the same space. With the centuries, the differences between the styles become more difficult to recognize, due to a mixture of construction practices. In Figure 2.11a,b possible examples of these two styles are presented.

The Franconian style is similar to the Saxonian, being the posts near to each other. However, here the braces are emphasized creating very picturesque buildings, introducing also curved elements (Figure 2.11c). Just to understand the popularity of half-timbered houses in Germany, a dedicated tourist office exists\textsuperscript{11} with complete maps to visit the most

\begin{flushleft}
\textsuperscript{10} Germany’s half-timbered houses. Published by Presse- und Informationsamt der Bundesregierung, Welckerstraße, Bonn. CRS Archives Document, CRS Center, College Station, TX http://crscenter.tamu.edu (last accessed on 2010/09/01)
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\textsuperscript{11} German Half-Timbered House Road. A complete overview of the regional routes. Deutsche Fachwerk Straße. http://www.deutsche-fachwerkstrasse.de/uk/index.php (last accessed on 2013/07/06)
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\textsuperscript{12} http://www.sagen.at/fotos/showphoto.php/photo/35046/size/big (last accessed on 2013/08/18)
\end{flushleft}
beautiful examples of half-timbered construction in Germany. More than 2.5 million half-timbered buildings exist today in Germany\textsuperscript{11}. Here, half-timbered houses are still being built.

![Image](a)

![Image](b)

![Image](c)

Figure 2.11 - Half-timbered buildings in Germany: (a) 400 year old building in Ubstadt-Weiher in possible Alemannic style, with projecting roof beams\textsuperscript{13}; (b) example of construction in a possible lower Saxonian style\textsuperscript{14}; (c) building in Franconian style\textsuperscript{15}

Commonly to other central European countries, half-timbered buildings were introduced in France during the Middle Ages and were used until the 19\textsuperscript{th} century. Through the 18\textsuperscript{th} and 19\textsuperscript{th} centuries, the façades of the buildings were plastered, in order to attribute them a more modern and luxurious look\textsuperscript{16}. *Colombages* or *pan de bois* buildings present a main timber skeleton filled with various elements which present no structural purpose from a static point of view (Tampone 1996).

The timber frame was located on top of a foundation made of rubble stones. Two main techniques can be distinguished: the longwood (*bois longs*), where the posts are continuous from the first to the last storey (Figure 2.12a), and the shortwood (*bois court*), where the posts are discontinuous between storeys (Figure 2.12b), which was introduced in the 16\textsuperscript{th} century\textsuperscript{17} (Tampone, 1996). With the latter technique, the structure becomes lighter and

\textsuperscript{13} http://www.ubstadt-weiher.de/pb/Lde/642795.html (last accessed on 2013/08/18)

\textsuperscript{14} http://www.thueringen.info/impressionen/menschen-brauchtum.html (last accessed on 2013/08/18)

\textsuperscript{15} http://www.fotos.sc/PHPSESSID=51c/nav2/popi+1030189/mediafile.html (last accessed on 2013/08/18)

\textsuperscript{16} Maison à colombage http://www.techno-science.net/?onglet=glossaire&definition=6910 (last accessed on 2013/07/07)

\textsuperscript{17} Alsace: la maison alsacienne. http://www.encycopedie.bseditions.fr/article.php?pArticleId=60 (last accessed on 2013/07/07)
more rigid. The structure presents a main frame with horizontal and vertical elements (*poteaux* and *sablières*), with smaller vertical elements (*montantes*) to support the doors and frames, diagonal elements (*décharges*) to add stiffness to the frame and smaller horizontal elements (*entretoises*) to serve as lintels and add confinement to the main frame. The adoption of shorter elements led to the creation of a sort of boxed system, which even facilitated construction. The decoration level for this period is still not advanced\(^1\).  

![Figure 2.12 - Half-timbered construction techniques in France: (a) bois longs\(^1\); (b) bois courts\(^1\)](image)

In the 18\(^{\text{th}}\) century, the houses become bigger, more spacious and more detail is given to decoration. The construction method remains the shortwood one, which also allowed the development of jetties (*encorbellement*), as they allowed a different distribution of the structural elements. The construction practice remained almost unaltered until the German annexation of Alsace in 1870. After this time, mainly masonry buildings are built. In the 20\(^{\text{th}}\) century, half-timbered buildings came into vogue again, introducing restoration plans\(^1\). In terms of connections, colombages adopted half-lap, mortise and tenon and dovetail connections. Wooden pegs were used as fasteners. Even though they were not designed as seismic resistant, a record of an earthquake in 1396 in Basel shows that, though masonry buildings collapsed, the half-timbered ones remained almost intact. The peculiarity of colombages is that they are fundamentally pre-fabricated. All timber pieces were prepared in the carpentry and the assemblage only took a few days. Infill could vary from rubble masonry to straw and mud. The geometry of the timber frame could vary greatly, with various decorative patterns\(^1\).  

Various Spanish cities have examples of half-timbered buildings (*entramados*), particularly in the area of Madrid between the 17\(^{\text{th}}\) and 19\(^{\text{th}}\) century (Gonzales and Aroca, 2003). Other examples are present in Segovia, Calatahazor, San Sebastián, Burgos and its surroundings, just to mention a few (Figure 2.13a). Timber-framed elements are present only in the upper storeys, while the ground floor is in masonry (Figure 2.13b). The buildings present a timber
frame composed of vertical and horizontal elements, with few diagonal elements (Figure 2.13c) (Casas, 2006).

![Image](http://www.galeon.com/sierradefrancia/2_arquitecturar.htm)

Figure 2.13 – Half-timbered architecture in Spain: (a) historic building in Covarrubias, Spain; (b) typical disposition of building; (c) timber frame adopted in buildings (Redondo and Gonzalves, 2000).

A great variability is found in terms of infill: it can be either brick or rubble masonry, cob wall, wattle and daub, adobe or ceramic, and lime or gypsum mortars can be used (Casas, 2006). Typically the houses had 4 to 5 storeys. In Madrid, the construction system changed after a fire in 1790 destroyed more than 50 houses and only stopped when it encountered a masonry building. From then on, it was advised to build the façades of the buildings totally in masonry being the timber elements adopted for internal and partition walls. Inner timber-framed walls were built until the beginning of the 20th century (Gonzales and Aroca, 2003).

In northern European countries like Denmark, Sweden and Norway, where wood was abundant, timber has been used to build composite timber-masonry buildings, like the “long houses” (langhus). Traditionally, houses were made of trunks, but some timber-frame houses (bindingsverk) were documented in Norway during the early Middle Ages. Due to the availability of the materials (both wood and bricks), in some areas half-timbered buildings were the most common construction method (Copani, 2007). Another example of the use of timber structures in Scandinavia is given by the diffusion, from the 12th to the 14th century, of timber churches (stavkirke) (Tampone, 1996).

The half-timbered structures in Scandinavia become more articulated from the 1500s, as, with the greater experience of the carpenters comes the introduction of carved wood elements as well as the introduction of different patterns for the masonry infill. The Scandinavian half-timbered buildings do not present great alterations in terms of geometry from what was observed in the rest of Europe: a timber skeleton is present, the posts are placed at a varying distance (between 80 and 150cm), the buildings present a masonry

\(^{18}\) http://www.galeon.com/sierradefrancia/2_arquitecturar.htm (last accessed on 2013/07/13)
foundation (usually stones, with some bricks if levelling is needed) and the usual number of storeys is 2 to 3. The walls are not plastered, but the timber skeleton is visible in the façade, see Figure 2.14a,b (Copani, 2007).

![Figure 2.14 - Half-timbered constructions in Scandinavia: (a) half-timbered house in Ystad, Sweden; (b) 18th century half-timbered building in Sæby, Denmark.](http://swedenroadways.blogspot.pt/2010/10/ystad-canal-pastries-half-timber-facade.html) (last accessed on 2013/08/18)

![Figure 2.14 - Half-timbered constructions in Scandinavia: (a) half-timbered house in Ystad, Sweden; (b) 18th century half-timbered building in Sæby, Denmark.](http://www.danishnet.com/info.php/pictures/half-timbered-home-169.html) (last accessed on 2013/08/18)

Observing the Scandinavian buildings it is evident that imperfections in the wooden members of the timber frame structure (ex. torsion of a beam or deformation of the frame) do not affect the behaviour of the whole building. These buildings were particularly suitable to resist seismic action, but due to their not great rigidity some structural units achieved great deformations (Copani, 2007).

It should be pointed out that for countries not prone to seismic actions, the timber frame was not complemented with regular diagonal bracings with the aim to add stiffness to the structure, but were present only in some occasions at the corners. A regular bracing system was only adopted when the probability of the occurrence of earthquakes is great.

### 2.2.3 Asian examples

Asian countries have a great tradition of timber frame buildings. Immediately, the huge timber temples in Japan, China and Thailand come to mind. These countries have a great tradition in timber construction, but these traditional buildings are not half-timbered.

In Turkey and India, which have a high seismicity, half-timbered constructions were adopted. Both countries have known very strong seismic events in the past and half-timbered buildings have shown to resist well to such strong ground motions (Gülhan and Güney, 2000). In comparison, some modern reinforced concrete (RC) buildings did not behave as well.

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Turkey has an abundance of wood, as well as stone and clay. The typical Turkish Ottoman-style houses present jetties above a stone ground floor. The jetties strengthen the buildings, confining the lower floor (Gulkan and Langenbach, 2004).

Two types of timber and masonry constructions can be distinguished in Turkey: traditional timber-laced masonry, with horizontal timber elements embedded into bearing masonry walls (*hatıl*, which means beam), and traditional half-timbered construction, with the insertion of masonry in between the various elements of a timber frame (*himis*). It is very common to find the *hatıl* construction for the ground floor and the *himis* for the upper ones, as can be seen in Figure 2.15a,b (Gulkan and Langenbach, 2004). Himis construction was popular until the middle of the 20th century, when it was replaced by RC structures.

![Figure 2.15 - Traditional construction in Turkey: (a) typical half-timbered house with hatıl at ground floor and himis in upper storeys](http://www.turkishculture.org/architecture/houses/wood-timber-houses-278.htm (last accessed on 2013/07/16))

21 http://www.turkishculture.org/architecture/houses/wood-timber-houses-278.htm (last accessed on 2013/07/16)
22 http://www.forumgercek.com/showthread.php?t=45777 (last accessed on 2013/07/16)
23 http://motaxa.blogspot.pt/2011/03/birinci-kattaki-kaldirlacak-bagdadi.html (last accessed on 2013/07/16)
24 http://www.turkishculture.org/architecture/houses/wood-timber-houses-278.htm (last accessed on 2013/07/16)
The himis posts usually have a maximum span of 60cm and are tied at mid-storey height by other timber elements. The infill masonry is either brick or rubble stone masonry (Gulkan and Langenbach, 2004). The timber used for the frame was usually oak or yellow pine. Connections are usually mortise and tenon joints, using supplementary elements such as wrought iron nails. The traditional houses are of 2 or 3 storeys. The height of each storey was 3.5 to 3.7 m (Dişkaya, 2007). The walls in himis constructions have a thickness not greater than 10-12cm (Gulkan and Langenbach, 2004).

As an alternative infill, lath and plaster can be used (bağdadi): laths are nailed to the members of the timber frame and then plaster is applied on both sides (Figure 2.15c). This type of infill was adopted in Turkey during the 18th century (Dişkaya, 2007). This led to lightweight, seismic resistant, economical structures, but they were more disposed to decay (Gulkan and Langenbach, 2004). Another type of infill is dizeme (Figure 2.15d), which consists of wooden planks nailed to the timber frame (Doğangün et al., 2006).

Also in Turkey the half-timbered constructions were abandoned in the 19th century due to the high number of fires occurred and to the imposition of the Ottoman government to build masonry buildings. After some time, though, due to the high damages produced by earthquakes in masonry structures, the construction of timber buildings was once again allowed (Dişkaya, 2007).

Among India’s traditional constructive systems, two basic construction systems can be identified: taq (Figure 2.16a), which is a timber-tied masonry, consisting of load bearing masonry walls tied at each floor level by timber elements, and dhajji-dewari (patchwork quilt wall), which is a braced timber frame with masonry infill (Figure 2.16b). It is not known when this kind of construction was introduced, but documentation can be found dating back to the 12th century stating that wood was used for the buildings (Langenbach, 2009). The masonry piers in taq construction bear the vertical loads. The horizontal timber elements, which are embedded in the masonry at each floor level and at window lintel level (Figure 2.16c), prevent the opening of the walls. Usually the masonry piers are 45-60cm square and the bays 90-120cm wide (Langenbach, 2009).

Dhajji dewari construction was often used for the upper storeys of buildings, with taq or unreinforced masonry construction located at lower floors, similarly to what happened in Turkey or Greece. It is a good solution, as it is light and it provides a pre-compression that helps confining the bearing masonry wall of the ground floor.

The dhajji-dewari system is similar to construction systems already described in Turkey and Europe. These walls are usually one-half brick in thickness. Each storey has a separate frame, and the stiffness of the building depends on the infill masonry, since the posts are not
continuous. Moreover, the floor joists are sandwiched between the upper and bottom timber beams of the frame structure for each floor (Figure 2.16) (Langenbach, 2009).

The buildings usually have 1 to 4 storeys and the distance between the timber posts is approximately 100cm (Hicyilmaz et al., 2011). Similarly to what was found in other countries,
the size of the frame and its configuration vary greatly (Figure 2.16e). Possibly, the variation is also due to the available material, in order to reduce costs. It is possible to observe similarities in terms of geometry and frame pattern between the Indian example and the Turkish or Portuguese one.

Taq construction has been abandoned, whereas dhajji-dewari is still used today (Langenbach, 2009).

### 2.2.4 Expansion to the Americas

In North America, half-timbered constructions were introduced by European immigrants, mainly German, French, Spanish and English, evolving and merging with local traditional timber construction systems. Tradition attributes the introduction of timber frame constructions in the USA, particularly in Chicago, to architect George W. Snow in 1832 with the balloon frame, but this type of construction was not new. Timber frame constructions with mortise and tenon connections already existed. They were introduced by European emigrants and were considered an expensive and heavy construction. The balloon frame tended to eliminate the traditional connections substituting them with nailed connections, in order to speed up the construction process\(^\text{25}\). The balloon frame is so called probably because of its extreme lightness and rigidity, as well as its construction simplicity and uniformity (Audel, 1923). The balloon frame adopts lighter timber elements, eliminating bracing elements. Only the corners were strengthened with additional bracing members (Figure 2.17a,b). Moreover, the elements are standardized, clearly indicating this type of construction as a product of the industrial revolution. The wooden boards adopted have the same dimensions, but, according to requirements, two or more boards can be nailed together to create an element with a bigger cross section (Tampone, 1996). In subsequent earthquakes (Rainer and Karacabeyli, 2000; Tobriner, 2000), this kind of construction showed a good resistance to seismic actions.

In the United States, half-timbered examples are present in New Orleans and French emigrants brought the colombage to Mississippi. Moreover, in Pennsylvania, examples of half-timbered buildings derived from fachwerk construction can be found (Langenbach, 2007). In Illinois, examples of timber frame buildings with masonry infill still exist today. Infill was traditionally used up to the 1890s; masonry infill was connected to the timber frame at every fourth course with a steel anchor.

In Central America, examples of timber frame structures can be found in different countries, for example in Nicaragua and in El Salvador. In Nicaragua, traditional timber-framed

construction is known as *taquezal*, which means pocket system, while in El Salvador it is known as *bahareque* (Langenbach, 2007). In these two countries, the system is the same and it is fundamentally a wattle-and-daub construction: a heavy timber frame, with posts positioned at corners and at the intersection of walls is covered with laths or bamboo canes, which are nailed on the timber frame (Figure 2.18a), forming a sort of basket mesh (see also half-timbered construction in Germany). The voids thus formed are then filled with small stones or adobe and covered with mud or lime plaster (Langenbach, 2007). No bracing elements are present. This system was usually used for buildings of one to two storeys. It is still used today, mainly in rural areas (Figure 2.18b). Foundations are usually not present in rural areas, being the poles infixed directly into the ground or with the addition of a layer of stones or bricks as foundation. In urban areas, foundations consist of stones, clay bricks or concrete, forming a pedestal into which the posts are inserted (Lang et al. 2007).

Figure 2.17 - Evolution of timber-framed buildings, the US example: (a) Balloon frame construction (Audel, 1923); (b) visible balloon framing when siding was replaced in house26.

Other half-timbered buildings can be found in Haiti, a Caribbean country occupying a small portion of the island of Hispaniola. Here, wattle-and-daub and various examples of half-timbered constructions can be found. Half-timbered and timber frame constructions were used to build dwellings and barns, and various examples can be found in poorer areas of the island (Figure 2.19a,b). A particular type of half-timbered buildings is Haitian Gingerbread houses, which unite to the traditional half-timbering complicated decorations and turrets and are typically related to upper classes (Figure 2.20a). In fact, gingerbread, in architecture and design, indicates “elaborately detailed embellishment, either lavish or superfluous” (Encyclopaedia Britannica). The term “Gingerbread” was first used in the 1950s when an American tourist likened them to the Victorian buildings in the USA, which were similarly decorated (Langenbach et al., 2010).

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26 http://www.hereandthere.org/oldhouse/balloon-framing.html (last accessed on 2013/07/13)
Gingerbread houses first appeared in Haiti in 1881, with the construction of the Haitian National Palace. The Gingerbread Houses are an important example of post-colonial design and they were added to the 2010 World Monuments Watch in October 2009 (Langenbach et al., 2010). The structure consisted of a timber frame, with a brick masonry infill, inspired by the *colombage* style (it has to be kept in mind that Haiti was a French colony). It was adorned with carved wood on the façade and roof banks. In subsequent years, more and more buildings were built. Currently, 200 Gingerbread houses exist in Haiti, most of them in Port-au-Prince (Langenbach et al., 2010). The last Gingerbread house was built in 1910, since in 1925 the mayor of Port-au-Prince banned this type of construction, allowing only masonry, reinforced concrete and iron, to prevent fires, which repeatedly struck the city (Langenbach et al., 2010; Katz, 2011).

The materials used for these buildings were readily available on the island, even if nowadays, their forests have been decimated. Woods used for construction include Caribbean pine, fir and tropical hardwood. Clay was available to produce red bricks (largely used in Gingerbread constructions). The stone used was local, probably originated from lifted oceanic deposits (Langenbach et al., 2010).

Haitian Gingerbread houses were built using three main construction systems: (1) braced timber frame; (2) colombage; and (3) masonry bearing wall (Langenbach et al., 2010).

Braced timber frame consists of vertical timber members connected to wood sills and top plates of each storey with mortise and tenon connections with the addition of wooden pegs, which were later substituted by nails. Bracing diagonal members were inserted at the top and bottom corner of each storey, to add additional stiffness (Figure 2.20a). This technique was imported from Europe during the colonization of the American continent, but, if in Europe and North America it was abandoned during the 19th century, in Haiti it was still used during the
20th century (Langenbach et al., 2010). Some of the buildings with this kind of structural typology present a masonry infill, while those without are externally covered with horizontal lap-wood siding (shiplap) and internally sheathed with wooden boards (Langenbach et al., 2010).

![Figure 2.19 – Timber frame constructions in Haiti: (a) wattle-and-daub construction (Image credit: © Kevin Rowell)27; (b) Half-timbered construction (Image credit: © Kevin Rowell)27]

Colombage is similar to the typical French construction typology, with infill inserted in the timber frame described above (Figure 2.20b). Infill is either rubble stone masonry or brick masonry, with clay or lime mortar. Lime plaster was applied when stone masonry was used (Langenbach et al., 2010).

Masonry bearing walls were generally used for exterior walls. They consisted of either brick or rubble stone masonry or a combination of the two. For the first case, lime mortar was used and, for the second one either clay or lime mortar (Langenbach et al., 2010).

There are cases where all three structural systems are mixed; at times colombage can be used at the ground floor and braced timber frame at the upper storey, or even a ground masonry floor can be present (Figure 2.20c), with colombage and braced timber frame above (Langenbach et al., 2010).

A slightly different type of half-timbered construction is present even in South America. Traditional construction methods in Peru are earth constructions and quincha. The word quincha comes from quechua and means “about the enclosure of sticks or reeds” (Schilder, 2000). It was introduced in Peru before the Spanish colonization, improved in subsequent centuries and reached its maximum popularity during the 18th century. In subsequent years, the quincha lost its appeal and in 1993 only 4.69% of existing buildings were built with this system (Jurina and Righetti, 2009).

Figures 2.20 – Gingerbread houses in Haiti: (a) braced timber framed construction; (b) half-timbered building (©William Daniels / Panos for Time); (c) hybrid system, with load bearing masonry at ground floor, colombage at first floor and timber frame at last storey (Langenbach et al., 2010).

Quincha constructions consist of a timber frame over of which canes are nailed or secured with cords, iron wires, vegetable fibres or slender strips of animal skin. The walls are then covered on both sides with a mixture of mud and gypsum. Typically, this type of buildings (mainly used in poorer areas) had only one storey with a timber roof and the walls were 15cm thick (Jurina and Righetti, 2009; Gutiérrez et al., 2003).

Many heritage buildings in Peru have upper storeys built with quincha, being the ground floor in adobe. In 1666, quincha was used to substitute the deteriorated roofs of some churches. Quincha construction has shown a good seismic performance, at the beginning of the 18th, when the first standards appeared, it was mandatory to use quincha for upper storeys of buildings, as well as vaults and domes in churches (Schilder, 2000). This choice was motivated by the devastating 1746 earthquake that hit Lima, taking advantage of the lower weight of the structural element and its greater ductility (Lowe and Ruskulis, 2002).

http://news.bbc.co.uk/2/hi/in_pictures/8295341.stm (last accessed on 2013/07/14)
http://www.time.com/time/photogallery/0,29307,2004148_2166299,00.html (last accessed on 2013/07/14)
In Colombia, bahareque construction was present, though here it evolved with European influences reaching examples of higher aesthetic architecture.

### 2.2.5 Efficiency of half-timbered structures in recent earthquakes

Half-timbered structures have shown throughout the centuries their good capability in resisting seismic loads. One should only think of all the examples in which they have been adopted as a standard of construction, thanks to their performance in past earthquakes dating back to the 18th century.

Even if nowadays this construction technology is not commonly used, well preserved historic half-timbered buildings still exist and continue to testify their seismic capacity. In the last decades, a series of earthquakes reignited the interest in these structures. In this section, some significant examples are presented. The analysis presented should be taken with caution, since it is not possible to define in absolute terms that construction technique A is better than construction technique B. In fact, all observations should be taken with caution, since they depend on the level of maintenance of traditional structures as well as on the execution and design of modern reinforced concrete structures.

In summer 2003, a strong earthquake hit the island of Lefkas. The region was not new to seismic events, as the Ionian Islands have the highest seismicity in Greece (Makarios and Demosthenous, 2006). Damages were observed in both reinforced concrete buildings and traditional half-timbered buildings (Vintzileou et al., 2007). From a survey performed by Makarios and Demosthenous (2006) on the construction typologies for buildings, 6% of the buildings are of unreinforced masonry, 15% are wooden buildings, 34% are half-timbered and 45% are reinforced concrete buildings. The main damages observed in half-timbered

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30 http://www.caralperu.gob.pe/caralperu/civilizacion/civilizaciontecoestructiva.html (last accessed on 2013/07/14)
31 http://limamalalima.wordpress.com/2010/05/10/quincha-e-idiomincrasia-limen/ (last accessed on 2013/07/14)
32 http://www.picturesocial.com/photo/casa-del-oidor-hearer-s-house/next?context=user (last accessed on 2013/07/09)
buildings were not only due to this latest earthquake but also to previous ones and to decay due to poor maintenance. In particular, the latter had an influence on the seismic capacity of the building, since they could alter the structural system. In general, the damages observed comprised: vertical and diagonal cracks in stone masonry walls of the ground floor; cracks in interface between timber frame and masonry infill; out-of-plane collapse of infill (Figure 2.22a). Moreover, decay on several timber elements was identified, including secondary system pillars, timber beams and timber floors resulting in some additional problems during the occurrence of the earthquakes. Additionally, modifications done in the existing structure, such as removing the ground masonry walls, led to large permanent horizontal displacements, since the secondary bearing system did not have sufficient stiffness (Vintzileou et al., 2007). In fact, the damages observed were sometimes due to poor or no maintenance and to modifications done to the buildings, pointing out the importance of preservation.

Another example where the efficiency of half-timbered structures was tested is the traditional half-timbered buildings in Turkey, where there are reports of the good behaviour of timber frame construction even from earlier earthquakes. After a devastating earthquake in Istanbul in 1894, experts and citizens both were positively impressed by the good performance of half-timbered buildings and the experts who studied the damages specifically concluded that timber structures behaved better than masonry buildings, even though they were older or poorly built (Tobriner, 2000).

A big earthquake hit the country in August 1999 in Kocaeli and in November of the same year in Duzce. Reports are available on the damage percentage of half-timbered buildings compared to modern constructions, showing some visually impacting results when comparing these constructive typologies (Figure 2.22b). According to Gülhan and Güney (2000), in Kocaeli-Gölcük, in the Sehitler district, 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). Of these, only 0.5% of the traditional structures presented heavy damages or collapsed against 7.4% of the RC structures. Additionally, 0.6% and 10% of the traditional structures presented moderate and light damage respectively, whereas 8.6% of the RC presented moderate damage and 16.5% presented light damages.

In the Kavakli district, 25% of the buildings are traditional. Here, 4.4% of the traditional buildings were heavily damaged or collapsed versus 19.3% of the RC buildings, 16.8% were moderately damaged versus 26.3% of the RC ones and 19.3% versus 10.6% were slightly damaged. In the Dumlupinar district, only 18% of the buildings are traditional. Of these, 1.9% presented heavy damages or collapsed, versus 6.1% of the modern buildings, 9% had moderate damages versus 13.5% of RC buildings and 28.1% were slightly damaged versus
30.8% of RC buildings. The percentage of buildings without damage was 61% and 49.6% for traditional and RC buildings respectively (Gülhan and Güney, 2000). The same trend was observed in other districts. This is an example of how a traditional construction well done can have equal or higher performances than modern constructions, particularly if the seismic requirements for reinforced concrete buildings are not fulfilled.

Figure 2.22 - Damages to half-timbered buildings in recent earthquakes: (a) out-of-plane collapse of masonry infill in a traditional building in Lefkas, Greece (Makarios and Demosthenous, 2006); (b) comparison of damages to traditional and modern building after the 1999 Duzce earthquake (Doğangün et al., 2006); (c) failure of connection in timber frame during 1999 Kocaeli earthquake; (d) collapse of unreinforced masonry wall, while upper dhajji-dewari storey still stands after Kashmir earthquake (Langenbach, 2009); (e) soft storey mechanism due to removal of infill (Langenbach, 2009); (f) damage concentrated in masonry walls of ground floor (Langenbach et al., 2010);
Another factor that influences the performance of half-timbered buildings is the level of bracings present in the structure. Of course, the more bracings, the better, particularly if X members are present, not only diagonals. However, in some cases they were reduced, being added at the corners, where the building are more vulnerable, in order to have more openings (Tobriner, 2000).

Typical damages found in half-timbered buildings under seismic actions include failure of connections in half-timbered walls (Figure 2.22c), out-of-plane failure of masonry, separation of walls from beams, soft-storey mechanism at ground floor (mainly due to alterations made at the ground floor level, particularly regarding the removal of the stone masonry walls) (Doğangün et al., 2006).

In North America, during the 1964 Alaska earthquake, timber-frame houses did not observe heavy damages, but slid down failed slopes. During the San Fernando earthquake (1971), damages to timber-frame buildings included sliding off of foundations, collapse of add-ons and masonry chimneys and distortion of weak first storey. These damages were due to some deficiencies found in the structures, such as absence of connections between frame and foundation, absence of bracing and large openings. When the structures were well built, no severe damages were observed (Rainer and Karacabeyli, 2000).

The Great Peruvian earthquake of 1970 renewed interest in quincha construction. In 1990, after an earthquake in Alto Mayo, which destroyed many buildings, mainly those built with rammed earth, a project was approved in order to build new construction with an improved quincha, which included concrete foundations and good connections between the structural timber elements33. The following year, before the completion of the project, another strong earthquake hit the area and the buildings in quincha built up to that moment were able to resist the seismic event, while a great number of local buildings was damaged33. Projects exist that propose a new and improved quincha (Lowe and Ruskulis, 2002), taking advantage of its strong points, such as its great ductility and reduced weight (Gutiérrez et al. 2003), as well as its relatively simple construction. The posts are embedded in concrete foundation beams and the rest of the building is built in the traditional way. A manual was prepared for common people to be able to build their own house34.

In Central America, El Salvador was hit by a strong earthquake in 1986 and the only failures observed in bahareque constructions were due to poor maintenance and decay of wood (Langenbach, 2007).

34 Construyendo viviendas con Quincha Mejorada. http://www.predes.org.pe/guia–conjuyendo-viviendas-con-quincha-mejorada-tecnologia-de-mitigacion-de-riesgos (last accessed on 2013/08/15)
The good behaviour of timber-laced masonry (taq) buildings was observed during the Anantnag earthquake in 1967 in India (Langenbach, 2009). In 2005 an earthquake devastated India, hitting the Kashmir area. During this event, both taq and dhajji-dewari buildings showed a good seismic behaviour. Unreinforced masonry collapsed, whereas timber framed masonry resisted to the same seismic excitation, arising new interest in this structural solution. No percentages are available in order to compare the collapsed modern buildings versus the number of traditional ones, but the renewed interest is also due to the fact that a dhajji-dewari structure is more easily built and allows easier reconstruction plans.

A guidebook for technicians and artisans was prepared by the United Nations (Schacher and Ali, 2010) illustrating how to build one and two storeys seismic resistant houses with the dhajji-dewari system. Notice that it was suggested to apply steel plates and ties to the connections.

The Kashmir earthquake affected even Pakistan and the many new dwellings were built using timber-laced (bhatar) and timber-framed masonry, with training programs to allow common people to build their own house (Langenbach, 2009). Also in this case a manual was prepared illustrating some basic rules for construction, including rules for connections, walls and foundations (ERRA, 2007). Damages to structures included: failure of unreinforced part of building (Figure 2.22d), out-of-plane dislocation of infill, soft storey mechanism when infill was removed (Figure 2.22e) or structure modified (for example, creation of large opening at ground floor), large permanent deformations (Langenbach, 2009).

Similar observations can be made for the structures which suffered the earthquake in Haiti in 2010. Damages depended greatly on the conservation state of the buildings before the earthquake. There were many examples of structures where the wood was decayed and inappropriate alterations were made, namely the addition of concrete floor, slabs and walls (Langenbach et al., 2010). Also here, examples could be found where damage was concentrated on the masonry load bearing walls of the ground floor, while the upper half-timbered storeys had less damage (Figure 2.22f). Other damages included the falling out-of-plane of masonry infill of half-timbered walls, dislocation of timber sheathing (Langenbach et al., 2010).

From these examples it is possible to conclude that historic half-timbered constructions have a ductile behaviour, which can be exploited in seismic regions. This means that this constructive technique could be used nowadays, particularly in vernacular buildings, in which it is more common to be identified.
CHAPTER 3 MATERIALS CHARACTERIZATION

3.1 Introduction

To understand the behaviour of a structural element it is important to understand the behaviour of each of its singular components. Traditional timber frame walls are a composite structural element which combine timber, mortar, bricks and dowel type connections. These walls are used as seismic resistant shear walls in many countries, therefore it is important for each constitutive material to be known and understood. Timber plays the main role in half-timbered walls, as it constitutes the main load bearing element of the wall, having a good resistance both in tension and compression. Masonry and its components play a key role in confining the frame, so their contribution has to be analysed.

In particular, the materials analysed were timber, mortar, bricks, masonry and the materials used for strengthening purposes, namely steel plates, steel flat bars and structural timber glue. The values obtained are important because they can help calibrate numerical models aiming at reproducing the seismic response of traditional timber frame walls.

3.2 Timber characteristics

The analysis of the mechanical characterization of timber is of fundamental importance if further analyses want to be made with numerical tools. Various wood species exist with a great variability in properties. As timber is an anisotropic material, various tests would be necessary to mechanically define it, but in reality only three parameters are sufficient to obtain the whole characterization of the material. Namely, by taking into account the Joint Committee on Structural Safety model (JCSS, 2006), all mechanical parameters can be defined using the bending strength, modulus of elasticity in bending and density of timber.
Therefore, to characterize the material, timber was tested in bending and compression, considering its structural dimensions. Moreover, density and moisture content were measured. Maritime Pine (Pinus pinaster) was used for the specimens, the same one used for the construction of the walls (see Chapter 4). The specimens were kept in a controlled climatic condition (temperature of 20°C and humidity of 60%) until they reached a constant weight.

3.2.1 Bending tests

To perform bending tests, 12 specimens were prepared according to specifications given by EN 408 (2003) on structural specimens for bending. In particular, the specimens had a cross section of 16×12cm², which was the biggest cross section of the walls' elements, and a length of 256cm, which corresponds to the smallest length admitted for this test by the standard, namely 15 times the height of the element with the addition of two times the halved height for the supports (EN 408, 2003). The test setup and instrumentation are shown in Figure 3.1. In spite of standards indicating that more specimens should be chosen, a limited number of specimens was tested due to the limited availability of the testing equipment.

Before testing, all beams were evaluated using visual grading in order to assess their quality class based on their defects, namely knots, fissures, presence of pith, index of knots presence in the lateral or total section (Cruz et al., 1997). From the visual inspection made it was observed that the tested beams belonged to different classes, being classified some in E class and others in EE class, mainly due to an important presence of knots, pith presence, slope of grain and important fissures. It was decided to keep this variability in timber quality, as in practice, during construction, the carpenters often do not pay attention to timber defects, even if there are such specifications.

The bending tests were performed in accordance to standard EN 408 (2003), namely a four-point bending load configuration. For the test, bending strength and local and global modulus of elasticity were obtained for each specimen, taking advantage of the appropriate
instrumentation suggested by the standard, namely measuring the deflection of the beam for the total span (dimension l in Figure 3.1) and for the central span (dimension l₁ in Figure 3.1).

The tests were carried out under displacement control at a rate of 0.1mm/s. Table 3.1 shows the results of the bending tests, in terms of mechanical properties, namely bending strength \( f_m \), global modulus of elasticity in bending, \( E_{m,g} \), local modulus of elasticity in bending \( E_{m,l} \). The properties were calculated according to standard EN 408 (2003) as following:

\[
\begin{align*}
  f_m &= \frac{a \cdot F_{\text{max}}}{2 \cdot W} \\
  E_{m,g} &= \frac{l^3 \cdot (F_2 - F_1)}{b \cdot h^3 \cdot (w_2 - w_1) \left[ \left( \frac{3}{4} \cdot l \right) - \left( \frac{a}{l} \right)^3 \right]} \\
  E_{m,l} &= \frac{a \cdot l_1^2 \cdot (F_2 - F_1)}{16 \cdot l \cdot (w_2 - w_1)}
\end{align*}
\]  

where:
- \( a \) is the distance between a loading position and the nearest support, see Figure 3.1;
- \( F_{\text{max}} \) is the maximum load;
- \( W \) is the section modulus;
- \( l \) is the span of the beam, see Figure 3.1;
- \( l_1 \) is the gauge length, see Figure 3.1;
- \( b \) is the width of the cross section;
- \( h \) is the height of the cross section, see Figure 3.1;
- \( l \) is the second moment of area;
- \( F_2 - F_1 \) is the increment in load;
- \( w_2 - w_1 \) is the corresponding increment of deformation.

The average values and the coefficient of variation (c.o.v.) are reported. Notice that some scatter is present in the results, which depends on the quality of timber.

Failure modes varied with the quality of timber. For timber with a better quality, with less defects (Figure 3.2), failure was progressive. In spite of a sudden loss of bending load just after the achievement of the peak load, post peak the specimen was able to recover some strength, as the fibres were still keeping together the beam (see Figure 3.3).
Table 3.1 - Bending tests results

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>(E_{m,q}) [kN/mm(^2)]</th>
<th>(E_{m,l}) [kN/mm(^2)]</th>
<th>(f_m) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PNPN_01_B</td>
<td>10.90</td>
<td>12.89</td>
<td>43.76</td>
</tr>
<tr>
<td>PNPN_02_B</td>
<td>11.24</td>
<td>13.35</td>
<td>54.95</td>
</tr>
<tr>
<td>PNPN_03_B</td>
<td>10.14</td>
<td>11.38</td>
<td>36.60</td>
</tr>
<tr>
<td>PNPN_04_B</td>
<td>7.37</td>
<td>8.63</td>
<td>29.70</td>
</tr>
<tr>
<td>PNPN_05_B*</td>
<td>12.21</td>
<td>13.05</td>
<td>65.70</td>
</tr>
<tr>
<td>PNPN_06_B</td>
<td>10.32</td>
<td>10.93</td>
<td>38.81</td>
</tr>
<tr>
<td>PNPN_07_B</td>
<td>12.66</td>
<td>14.25</td>
<td>58.24</td>
</tr>
<tr>
<td>PNPN_08_B</td>
<td>12.83</td>
<td>13.16</td>
<td>53.45</td>
</tr>
<tr>
<td>PNPN_09_B</td>
<td>8.25</td>
<td>9.33</td>
<td>32.22</td>
</tr>
<tr>
<td>PNPN_10_B</td>
<td>14.16</td>
<td>16.29</td>
<td>71.33</td>
</tr>
<tr>
<td>PNPN_11_B</td>
<td>9.99</td>
<td>10.30</td>
<td>53.27</td>
</tr>
<tr>
<td>PNPN_12_B</td>
<td>9.76</td>
<td>10.65</td>
<td>36.01</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>10.82</strong></td>
<td><strong>12.02</strong></td>
<td><strong>47.84</strong></td>
</tr>
<tr>
<td><strong>C.O.V. [%]</strong></td>
<td><strong>18.02</strong></td>
<td><strong>18.30</strong></td>
<td><strong>28.38</strong></td>
</tr>
</tbody>
</table>

Figure 3.2 - Failure mode of specimen PNPN_10_B: (a) bending deformation of specimen; (b) failure initiation; (c) deformation at the end of the test

For a timber of lower quality with various defects (Figure 3.4), failure initiated in correspondence of a knot and it was brittle.

Figure 3.3 - Typical force-displacement graph of bending test
Notice that the presence of defects decreases both strength and stiffness as well as changes the post-peak behaviour. Therefore, even for the timber frame walls, attention should be paid to the presence of defects in the timber elements, as they could influence the test results.

![Figure 3.4 - Failure mode of specimen PNPN_01_B: (a) bending deformation of specimen; (b) failure initiation; (c) brittle failure initiated in correspondence of a knot](image)

### 3.2.2 Compressive tests

For the compressive tests, the specimens were cut according to recommendations given by standard EN 408 (2003), considering the section of the structural element and a height of six times the smaller cross-sectional dimension (12×16×72 cm³). The modulus of elasticity in compression parallel to the grain and the compressive strength were obtained from the tests. In this case also, 12 specimens were tested. The tests were performed in displacement control with a speed rate of 0.02mm/s. Deformation was measured on the four sides of the specimen on a length of four times the smaller cross section, i.e. 48cm (measure l₁ in Figure 3.5). The test equipment and instrumentation are shown in Figure 3.5.

![Figure 3.5 - Test setup and instrumentation for compressive test of timber](image)

Also for this batch of specimens, the quality of timber varied, influencing the test results. Table 3.2 presents the results obtained from the compression tests, in terms of mechanical characteristics, namely the compressive strength parallel to the grain, $\sigma_{c,0}$, and the modulus of elasticity in compression parallel to the grain, $E_{c,0}$. The properties were calculated following standard EN 408 (2003) as follows:
\[ \sigma_{c,0} = \frac{F_{\text{max}}}{A} \]  
\[ E_{c,0} = \frac{l_1 \cdot (F_2 - F_1)}{A \cdot (w_2 - w_1)} \]

where:

- \( A \) is the cross sectional area;
- \( l_1 \) is the gauge length, see Figure 3.5;
- \( F_2 - F_1 \) is the increment in load;
- \( w_2 - w_1 \) is the corresponding increment of deformation.

For these tests, the variation was lower in terms of strength than what observed in bending tests, whereas the same variability was recorded for the Young modulus.

**Table 3.2 - Results of compression tests**

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>( \sigma_{c,0} ) [MPa]</th>
<th>( E_{c,0} ) [kN/mm(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PNPN_01_C</td>
<td>37.89</td>
<td>9658</td>
</tr>
<tr>
<td>PNPN_02_C</td>
<td>35.67</td>
<td>9892</td>
</tr>
<tr>
<td>PNPN_03_C</td>
<td>38.02</td>
<td>11838</td>
</tr>
<tr>
<td>PNPN_04_C</td>
<td>38.62</td>
<td>10159</td>
</tr>
<tr>
<td>PNPN_05_C</td>
<td>41.07</td>
<td>12073</td>
</tr>
<tr>
<td>PNPN_06_C</td>
<td>41.04</td>
<td>12278</td>
</tr>
<tr>
<td>PNPN_07_C</td>
<td>36.01</td>
<td>11996</td>
</tr>
<tr>
<td>PNPN_08_C</td>
<td>35.98</td>
<td>9421</td>
</tr>
<tr>
<td>PNPN_09_C</td>
<td>48.08</td>
<td>15688</td>
</tr>
<tr>
<td>PNPN_10_C</td>
<td>40.30</td>
<td>11180</td>
</tr>
<tr>
<td>PNPN_11_C</td>
<td>36.87</td>
<td>10611</td>
</tr>
<tr>
<td>PNPN_12_C</td>
<td>28.58</td>
<td>7704</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td><strong>38.18</strong></td>
<td><strong>11042</strong></td>
</tr>
<tr>
<td><strong>C.O.V.</strong></td>
<td><strong>11.95</strong></td>
<td><strong>18.05</strong></td>
</tr>
</tbody>
</table>

The presence of knots can influence the failure modes. Comparing two different qualities of specimens, for example specimen PNPN_12_C, which was classified as belonging to class E, and specimen PNPN_09_C, classified as EE, different fracture modes can be observed. In specimen PNPN_12_C the failure began in correspondence of an important knot and failure spread from there, see Figure 3.6.
Specimen PNPN_09_C (Figure 3.7) instead was classified as EE and presented smaller knots and fissures. The failure initiates also in a knot, but it has lesser influence and even after failure there still are enough fibres to bear load, even though they are not perfectly vertical.

Figure 3.8 shows the typical force-displacement curve for two specimens with different timber quality. The specimen with lower quality presents lower strength. The great difference between the specimens is the post-peak regime, as, after the peak, specimen PNPN_12_C exhibits an immediate strength loss and a lower dissipative capacity when compared to specimen PNPN_09_C.

These observations should be useful when analysing failures in timber frame walls.
3.2.3 Density and moisture content

Density and moisture content in timber were calculated according to standards ISO 3131 (1975) and ISO 3130 (1975). To do this, small pieces were cut from the previously tested specimens as near as possible the failure plane. The specimens had a cross section of 20×20 mm² and a length along the grain of 25mm.

Density and moisture content were derived for specimens kept in the climatic chamber at 20°C and a relative humidity of 60%, similarly to what done for the specimens tested in compression and bending in order to better correlate the results. Moreover, some specimens were kept in laboratory conditions (same conditions which will be applied to the walls), in order to better understand the variation between the two conditions.

Table 3.3 shows the values of density ($\rho$) and moisture content (MC) for specimens taken out from the bending test specimens for both laboratory conditions (subscript lab) and climatic chamber conditions (subscript CC). Notice that the numbers correspond with those of the tested beams.

Generally, a higher density led to higher values of strength, even if this is not always true, as for some specimens the higher density was hindered by the presence of a high number of grouped knots (see specimen PNPN_01_B).

The same conclusions can be drawn from Table 3.4, which presents the same results for the specimens tested in compression.

Values of density will be important when numerical models are developed, in order to obtain the properties of timber perpendicular to the grain.

Concerning moisture content, specimens conditioned in the climatic chamber reached a value of approximately 12%, as required by standard, while specimens in the lab condition had lower values, approximately 10%.
### Table 3.3 - Density and moisture content values for bending specimens

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\rho_{w, \text{lab}}$ [kg/m$^3$]</th>
<th>$\rho_{w, \text{CC}}$ [kg/m$^3$]</th>
<th>MC$_{\text{lab}}$ [%]</th>
<th>MC$_{\text{CC}}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1B</td>
<td>606.11</td>
<td>603.86</td>
<td>10.27</td>
<td>12.77</td>
</tr>
<tr>
<td>2B</td>
<td>572.42</td>
<td>562.34</td>
<td>10.70</td>
<td>12.75</td>
</tr>
<tr>
<td>3B</td>
<td>575.23</td>
<td>579.58</td>
<td>10.35</td>
<td>12.67</td>
</tr>
<tr>
<td>4B</td>
<td>476.78</td>
<td>480.86</td>
<td>10.15</td>
<td>12.37</td>
</tr>
<tr>
<td>5B</td>
<td>604.96</td>
<td>586.72</td>
<td>10.77</td>
<td>12.97</td>
</tr>
<tr>
<td>6B</td>
<td>596.86</td>
<td>590.50</td>
<td>10.76</td>
<td>13.02</td>
</tr>
<tr>
<td>7B</td>
<td>639.79</td>
<td>637.68</td>
<td>10.80</td>
<td>13.08</td>
</tr>
<tr>
<td>8B</td>
<td>688.28</td>
<td>671.27</td>
<td>10.88</td>
<td>13.03</td>
</tr>
<tr>
<td>9B</td>
<td>518.74</td>
<td>522.05</td>
<td>10.10</td>
<td>12.57</td>
</tr>
<tr>
<td>10B</td>
<td>682.77</td>
<td>-</td>
<td>11.60</td>
<td>-</td>
</tr>
<tr>
<td>11B</td>
<td>542.79</td>
<td>543.74</td>
<td>10.20</td>
<td>12.77</td>
</tr>
<tr>
<td>12B</td>
<td>581.73</td>
<td>561.94</td>
<td>10.08</td>
<td>12.82</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td>590.54</td>
<td>576.41</td>
<td>10.56</td>
<td>12.80</td>
</tr>
<tr>
<td><strong>C.O.V.</strong></td>
<td>10.46%</td>
<td>9.08%</td>
<td>4.26%</td>
<td>1.70%</td>
</tr>
</tbody>
</table>

### Table 3.4 - Density and moisture content values for compression specimens

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\rho_{w, \text{lab}}$ [kg/m$^3$]</th>
<th>$\rho_{w, \text{CC}}$ [kg/m$^3$]</th>
<th>MC$_{\text{lab}}$ [%]</th>
<th>MC$_{\text{CC}}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1C</td>
<td>554.67</td>
<td>-</td>
<td>10.28</td>
<td>-</td>
</tr>
<tr>
<td>2C</td>
<td>553.84</td>
<td>557.11</td>
<td>-</td>
<td>12.51</td>
</tr>
<tr>
<td>3C</td>
<td>575.45</td>
<td>576.20</td>
<td>-</td>
<td>12.66</td>
</tr>
<tr>
<td>4C</td>
<td>633.34</td>
<td>635.05</td>
<td>10.54</td>
<td>13.04</td>
</tr>
<tr>
<td>5C</td>
<td>579.24</td>
<td>-</td>
<td>10.50</td>
<td>-</td>
</tr>
<tr>
<td>6C</td>
<td>573.58</td>
<td>574.28</td>
<td>9.86</td>
<td>12.54</td>
</tr>
<tr>
<td>7C</td>
<td>592.64</td>
<td>587.70</td>
<td>-</td>
<td>12.16</td>
</tr>
<tr>
<td>8C</td>
<td>625.79</td>
<td>613.56</td>
<td>10.63</td>
<td>12.61</td>
</tr>
<tr>
<td>9C</td>
<td>673.37</td>
<td>668.94</td>
<td>10.55</td>
<td>12.79</td>
</tr>
<tr>
<td>10C</td>
<td>584.84</td>
<td>583.53</td>
<td>-</td>
<td>11.97</td>
</tr>
<tr>
<td>11C</td>
<td>412.98</td>
<td>410.44</td>
<td>-</td>
<td>12.86</td>
</tr>
<tr>
<td>12C</td>
<td>521.20</td>
<td>522.55</td>
<td>-</td>
<td>12.29</td>
</tr>
<tr>
<td><strong>avg</strong></td>
<td>573.41</td>
<td>572.94</td>
<td>10.39</td>
<td>12.54</td>
</tr>
<tr>
<td><strong>C.O.V.</strong></td>
<td>11.29%</td>
<td>12.24%</td>
<td>2.79%</td>
<td>2.61%</td>
</tr>
</tbody>
</table>

### 3.3 Mortar composition and characterization

Mortar ratios were decided in advance, based on the experience of Portuguese construction companies that work in the field of rehabilitation of traditional timber frame buildings. The rehabilitation company proposed to use a mortar with a ratio in volume of 6:2:1 in terms of
sand : lime : cement, being the sand ratio of 4:2 (medium sand : fine sand). Cement CEM-II/B-L 32.5N and non-hydraulic lime were used.

The two types of sand used were analysed in terms of particle distribution (see Figure 3.9) according to standard EN 933 (2009), apparent particle density ($\rho_a$), particle density on an oven dried basis ($\rho_{rd}$), particle density on a saturated and surface-dried basis ($\rho_{s, sd}$) and water absorption ($WA_{24}$) (see Table 3.5 and Table 3.6).

![Figure 3.9 - Particle size distribution of sand used in mortar](image)

The medium sand is well distributed, whilst the fine sand is essentially of one size, as expected. Moreover, the former absorbs more water, while the particle density of the two sands is similar.

<table>
<thead>
<tr>
<th>sample</th>
<th>$\rho_a$ [Mg/m$^3$]</th>
<th>$\rho_{rd}$ [Mg/m$^3$]</th>
<th>$\rho_{s, sd}$ [Mg/m$^3$]</th>
<th>$WA_{24}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.662</td>
<td>2.644</td>
<td>2.650</td>
<td>0.251</td>
</tr>
<tr>
<td>2</td>
<td>2.664</td>
<td>2.645</td>
<td>2.652</td>
<td>0.274</td>
</tr>
<tr>
<td>average</td>
<td>2.663</td>
<td>2.644</td>
<td>2.651</td>
<td>0.263</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>0.07%</td>
<td>0.03%</td>
<td>0.05%</td>
<td>6.13%</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>sample</th>
<th>$\rho_a$ [Mg/m$^3$]</th>
<th>$\rho_{rd}$ [Mg/m$^3$]</th>
<th>$\rho_{s, sd}$ [Mg/m$^3$]</th>
<th>$WA_{24}$ [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.661</td>
<td>2.596</td>
<td>2.620</td>
<td>0.952</td>
</tr>
<tr>
<td>2</td>
<td>2.650</td>
<td>2.563</td>
<td>2.596</td>
<td>1.276</td>
</tr>
<tr>
<td>average</td>
<td>2.656</td>
<td>2.579</td>
<td>2.608</td>
<td>1.114</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>0.30%</td>
<td>0.89%</td>
<td>0.66%</td>
<td>20.57%</td>
</tr>
</tbody>
</table>
Subsequently, the water content in relation to the cement content (W/C ratio) had to be established. To do this, the mortar was studied in the lab to find the appropriate water content in terms of workability through performing flow table tests (ASTM C1437, 2007).

The objective was to obtain a value of flow table between 160 and 170mm, which from past experience (Haach, 2009) has shown to be a good compromise between workability and resistance characteristics. Table 3.7 shows the values of the W/C ratio and the corresponding flow table value measured in four diameters and the resulting average value.

<table>
<thead>
<tr>
<th>cement [g]</th>
<th>water [g]</th>
<th>W/C</th>
<th>flow value [mm]</th>
<th>average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>246.33</td>
<td>123.3</td>
<td>0.50</td>
<td>142</td>
<td>141</td>
</tr>
<tr>
<td>240.34</td>
<td>324.92</td>
<td>1.35</td>
<td>185</td>
<td>184</td>
</tr>
<tr>
<td>241.58</td>
<td>306.23</td>
<td>1.27</td>
<td>158</td>
<td>158</td>
</tr>
<tr>
<td>253.96</td>
<td>334.24</td>
<td>1.32</td>
<td>172</td>
<td>175</td>
</tr>
<tr>
<td>246.19</td>
<td>319.22</td>
<td>1.30</td>
<td>166</td>
<td>165</td>
</tr>
</tbody>
</table>

From the results, the W/C ratio adopted was 1.3 in weight, which corresponds approximately to a W/C ratio of 1.5 in volume.

After the selection of the W/C ratio, specimens were prepared to determine the flexural and compressive strength of mortar according to standard EN 1015-11 (1999). Specimens mixed in the lab were compared to those prepared by the mason during the construction of the wall. In the following table, the values of bending and compressive strength can be compared for the specified specimens.

Mortar was tested in bending and compression according to standard EN 1015-11 (1999). Specimens were prepared in the laboratory and tested at 7, 14, 28 and 56 days and others were collected from the mortar prepared by hand by a mason and used to build the walls and tested at 28 days and at the time of the test.

Figure 3.10 shows the compressive and bending strength of mortar. It can be noticed that the results found for the specimens taken from the mixtures used for the construction of the walls are significantly different from the ones prepared in the laboratory. These differences could be due to a different amount of water used in the mortar by the mason, though his work was constantly followed.

Moreover, even among the results of the same walls, the results can vary as much as 25% in specimens taken from different cells of the walls.
Materials characterization

Comparing the bending strength ($f$) and compressive strength ($\sigma_c$) values at 28 days between the specimens prepared in the lab and those taken from the walls (Table 3.8), it is clear how the former had higher values of strength. Moreover, a significant variation exists among the wall results tested at the time of the in-plane tests.

Table 3.8 - Values of compressive and bending strength of mortar

<table>
<thead>
<tr>
<th>TEST</th>
<th>28 days</th>
<th>TEST</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>specimen</td>
<td>$f$</td>
</tr>
<tr>
<td></td>
<td>lab</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td>Wall 1</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>Wall 2</td>
<td>0.92</td>
</tr>
<tr>
<td></td>
<td>Wall 4</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>average</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td>c.o.v.</td>
<td>21.08%</td>
</tr>
</tbody>
</table>

This variability notwithstanding, the results of the in-plane cyclic tests on timber frame walls with masonry infill should not be significantly influenced, since the main aim of the masonry is to stiffen the frame.

3.4 Bricks

Bricks were characterized in compression and in flexure, to obtain the mode I fracture energy. While such detailed information is not strictly necessary, it can be useful for possible detailed numerical micro-modelling. The bricks used were modern solid clay brick with a dimension of $22\times11\times6\text{cm}^3$. 
Bricks have been tested in compression according to standard EN 772-1 (2000). Cubic specimens were prepared having a side of 40mm and tested in force control with a speed of 0.3kN/s. The average compressive strength obtained was of 34.5MPa with a coefficient of variation of 15%. Moreover, compressive tests were performed considering three cubes one on top of each other, in order to not take into consideration the friction with the loading plate. The compressive strength thus calculated was of 15.9MPa with a coefficient of variation of 7%.

The fracture energy of the bricks used was calculated performing a three-point bending test on a notched brick according to standard TC50-FMC (1985). Twelve tests were performed and the mean fracture energy obtained was of 231N/m, with a coefficient of variation of 33%.

3.5 Masonry

Masonry was characterised in compression and in shear (through diagonal tension tests). Compressive tests on masonry prisms were performed according to standard EN 1052-1 (1999) while diagonal tension tests were performed according to standard ASTM E519-02 (2003).

3.5.1 Compressive strength

Compressive tests were performed on masonry prisms having dimensions of 46×12×67cm³. The masonry pattern adopted is the same one that will be used in the walls and it will be further discussed in Chapter 4.

The tests were carried out according to standard EN 1052-1 (1999) and were performed in displacement control at a rate of 10µm/s and three initial loading and unloading cycles were performed reaching a force less than half the maximum one, as specified by the standard. Figure 3.11 shows the instrumentation adopted for the test in order to obtain the compressive stress-strain diagrams and the related mechanical properties, namely the modulus of elasticity (E) and the compressive strength (σmax). In particular, LVDT 1, 2, 3 and 4 were used to obtain the modulus of elasticity, considering the mean value, and they were able to give information on the possible rotation of the specimen. Three specimens were tested. The desired properties were obtained as follows:

\[ \sigma_{max} = \frac{F_{max}}{A} \]  \hspace{1cm} (3.6)

\[ E = \frac{F_{max}}{3 \cdot \varepsilon \cdot A} \]  \hspace{1cm} (3.7)

where:

- \( F_{max} \) is the maximum load reached by the specimen;
\( A \) is the loaded cross-section area of a specimen;
\( \varepsilon \) is the mean strain, measured on four positions, of the specimen at one third of the maximum load.

![Images](image1.png)

**Figure 3.11** - Instrumentation for compressive test on masonry prism

The top of the specimens was levelled in order to have a uniform vertical load. The average compressive strength of masonry was 7.53MPa (see Table 3.9), which, compared to historic masonry encountered in traditional timber frame buildings, is a high value.

**Table 3.9 - Results of compression tests on masonry prisms**

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>( \sigma_{\text{max}} ) [MPa]</th>
<th>( E ) [MPa]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2</td>
<td>7.27</td>
<td>5427.12</td>
</tr>
<tr>
<td>C3</td>
<td>7.15</td>
<td>8005.92</td>
</tr>
<tr>
<td>C4</td>
<td>8.16</td>
<td>6801.15</td>
</tr>
<tr>
<td>average</td>
<td>7.53</td>
<td>6744.73</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>7.31%</td>
<td>19.13%</td>
</tr>
</tbody>
</table>

It should be noticed that it was decided to use modern materials, therefore with higher strength, since this is one of the current practices in rehabilitation instead of representing the traditional existing masonry. Some variation was encountered in terms of modulus of elasticity.

Figure 3.12a shows the load-displacement curves for the three specimens, pointing out the differences in terms of strength and stiffness.
3.5.2 Shear strength

Diagonal tensile tests were performed on three specimens having dimensions of 96×12×100cm³. The tests were carried out according to standard ASTM E519-02 (2003), with loading shoes at the top and bottom of the specimens and applying a displacement rate of 2μm/s. The specimens were instrumented with four LVDTs, as shown in Figure 3.13, in order to measure the vertical shortening and the horizontal extension on both sides of the specimens and thus obtain the relative strains, so that the shear modulus could be obtained.

According to standard, the shear stress and shear modulus are calculated as follows:

\[
\tau = \frac{0.707 \cdot F}{A} \quad \text{(3.8)}
\]

\[
G = \frac{\tau}{\gamma} \quad \text{(3.9)}
\]

where:

- \(F\) is the applied load;
- \(A = ((W + h) / 2) \cdot t\) is the net area of the specimen, in this case corresponding to the total area, since the units used are solid, being \(W\) the width of the specimen, \(h\) its height and \(t\) its total thickness;
- \(\gamma = (|\Delta V| + |\Delta H|) / g\) is the shearing strain, being \(\Delta V\) the vertical shortening, \(\Delta H\) the horizontal extension and \(g\) the gauge length.
Materials characterization

Table 3.10 presents the results of the tests in terms of shear strength ($\tau_{\text{max}}$) and shear modulus ($G$). It is observed that variations of both shear strength and shear modulus are low.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\tau_{\text{max}}$</th>
<th>$G$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[MPa]</td>
<td>[MPa]</td>
</tr>
<tr>
<td>D2</td>
<td>0.18</td>
<td>883.74</td>
</tr>
<tr>
<td>D3</td>
<td>0.20</td>
<td>1154.03</td>
</tr>
<tr>
<td>D4</td>
<td>0.21</td>
<td>1153.77</td>
</tr>
<tr>
<td>Average</td>
<td>0.20</td>
<td>1063.84</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>9.17%</td>
<td>14.66%</td>
</tr>
</tbody>
</table>

Figure 3.14 shows the different failure modes for the three specimens considered in the results. While in specimen D2 the crack was not perfectly diagonal, in the other two specimens the crack was more along the line of the diagonal. Specimens D3 and D4 present similar crack patterns, with a similar vertical crack aligned with the applied load.
Notice that in all specimens, the crack would follow the mortar joints, with minimal cracking of bricks.

Figure 3.15 shows the strain-stress curves of all specimens and the vertical shortening and horizontal extension in the specimens. Notice that almost no rotation is present during the test, as the measurements on the two sides of the specimens are approximately the same.

3.6 Strengthening materials

Considering the materials used for strengthening (walls and connections), which will be discussed in Chapter 5, steel plates and bars and structural timber glue were characterized in tension according to standard BS EN 10002-1 (2001). It was assumed that steel bolts followed the requirements of steel class 8.8.

3.6.1 Steel plates

Custom steel plates made of zinc-galvanized steel with a thickness of 3mm and commercial perforated steel plates made of S250GD steel with a thickness of 2mm were used to strengthen the connections of the walls (Figure 3.16a). Both types of plates were tested in tension in order to obtain their tensile strength and modulus of elasticity. The custom steel plates had a total length \( L_t \) of 260mm and a parallel length \( L_c \) of 150mm while the commercial ones had a total length of 300mm and a parallel length of 190mm (Figure 3.16b). The width of the specimen was of 20mm and of 30mm for custom and commercial plates respectively. This difference was due to the fact that commercial steel plates presented holes and a higher width was necessary in order to have two holes in the width of the specimen and take into account its influence on the mechanical properties. For both plate types, the original gauge length \( L_0 \) was of 100mm. The tests were carried out in displacement control with a rate of 0.02mm/s. A strain gauge was applied to the specimens in order to then measure the modulus of elasticity.
Table 3.11 and Table 3.12 show the tensile test results for custom and commercial steel plates respectively, namely the tensile strength ($\sigma_{\text{max}}$), the modulus of elasticity (E) and the percentage elongation after fracture (A). As it can be seen, commercial steel plates were able to guarantee a high tensile strength, 38% higher than custom plates. Even the modulus of elasticity is higher, with an increase of 33%.

### Table 3.11 - Tensile test results for custom steel plates

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\sigma_{\text{max}}$ [MPa]</th>
<th>E [MPa]</th>
<th>A [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>custom 1</td>
<td>327.16</td>
<td>200558</td>
<td>43.00</td>
</tr>
<tr>
<td>custom 2</td>
<td>317.06</td>
<td>137283</td>
<td>39.00</td>
</tr>
<tr>
<td>custom 3</td>
<td>319.99</td>
<td>176660</td>
<td>43.00</td>
</tr>
<tr>
<td><strong>average</strong></td>
<td><strong>321.40</strong></td>
<td><strong>171500</strong></td>
<td><strong>41.67</strong></td>
</tr>
<tr>
<td><strong>c.o.v.</strong></td>
<td><strong>1.62%</strong></td>
<td><strong>18.63%</strong></td>
<td><strong>5.54%</strong></td>
</tr>
</tbody>
</table>

### Table 3.12 - Tensile test results for commercial steel plates

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\sigma_{\text{max}}$ [MPa]</th>
<th>E [MPa]</th>
<th>A [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>comm 1</td>
<td>486.39</td>
<td>222564</td>
<td>3.00</td>
</tr>
<tr>
<td>comm 2</td>
<td>468.73</td>
<td>197155</td>
<td>4.00</td>
</tr>
<tr>
<td>comm 3</td>
<td>476.54</td>
<td>266177</td>
<td>3.50</td>
</tr>
<tr>
<td><strong>average</strong></td>
<td><strong>477.22</strong></td>
<td><strong>228632</strong></td>
<td><strong>3.50</strong></td>
</tr>
<tr>
<td><strong>c.o.v.</strong></td>
<td><strong>1.85%</strong></td>
<td><strong>15.27%</strong></td>
<td><strong>14.29%</strong></td>
</tr>
</tbody>
</table>

However, the percentage of elongation after fracture is noticeably lower for commercial steel plates. This was due to the fact that commercial steel plates did not experience significant hardening after the yielding point, see Figure 3.17. This difference is mainly attributed to the presence of holes in commercial steel plates. In fact, while custom steel plates have an
important elongation and deform displaying the typical necking (Figure 3.18a), the presence of holes dominates the deformation of the commercial plates.

![Figure 3.17 - Tensile test results of steel plates: (a) stress-elongation curves; (b) stress-stress curves for initial part of test](image)

Failure occurs in correspondence of a section with two holes, therefore weaker, forming necking at the two sides of the plate but then failing completely.

![Figure 3.18 - Failure modes: (a) custom steel plate; (b) commercial steel plate](image)

The influence of holes in the failure mode of the steel plates has to be kept in mind during the tests on retrofitted walls and connections.

### 3.6.2 Steel flat bars

Another strengthening material used for the walls consisted of CK45 steel flat bars. Five machined specimens were prepared and tested in tension. The cross section of the test piece had a width of 25mm and a thickness of 8mm. The original gauge length ($L_0$) was of 230mm while the total length of the test piece ($L_t$) was of 400mm. The test was carried out in displacement control at a rate of 0.05mm/s. A strain gauge was applied to the specimens in order to then measure the modulus of elasticity.
Table 3.13 shows the test results, namely the tensile strength ($\sigma_{\text{max}}$), the modulus of elasticity (E) and the percentage elongation after fracture (A). It was decided to adopt bars with a high strength and a good elongation capacity in order to use less material in the walls.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\sigma_{\text{max}}$ [MPa]</th>
<th>E [MPa]</th>
<th>A [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>677.17</td>
<td>192290</td>
<td>16.09</td>
</tr>
<tr>
<td>2</td>
<td>670.32</td>
<td>197321</td>
<td>14.78</td>
</tr>
<tr>
<td>3</td>
<td>672.07</td>
<td>192728</td>
<td>16.52</td>
</tr>
<tr>
<td>4</td>
<td>674.38</td>
<td>191871</td>
<td>16.96</td>
</tr>
<tr>
<td>5</td>
<td>670.40</td>
<td>196325</td>
<td>16.09</td>
</tr>
<tr>
<td><strong>average</strong></td>
<td><strong>672.87</strong></td>
<td><strong>194107</strong></td>
<td><strong>16.09</strong></td>
</tr>
<tr>
<td><strong>c.o.v.</strong></td>
<td><strong>0.43%</strong></td>
<td><strong>1.30%</strong></td>
<td><strong>5.06%</strong></td>
</tr>
</tbody>
</table>

Figure 3.20 shows the stress elongation curve of the specimens. After yielding, the material showed significant hardening, with minimal softening after peak load.
The failure mode for all specimens consisted of necking in the central part of the bar, see Figure 3.21.

The bars have a high strength, as required for retrofitting, but also a good deformation capacity, which is important to ensure a ductile response of the walls.

### 3.6.3 Structural timber glue

Tensile tests were performed on the structural timber glue (epoxy resin) used for retrofitting walls and connections (see Chapter 5 and Chapter 6). Twelve specimens were moulded, having a cross section of 10×5mm$^2$ and a gauge length of 50mm, but only nine specimens gave viable results. The specimens were left to cure for seven days.

Table 3.14 shows the results of the tests. A tensile strength ($\sigma_{\text{max}}$) of 13.99MPa was obtained, with some variation in the results, possibly due to problems during moulding, for example formation of voids inside the specimens. According to the technical specifications (MAPEI, 2002), the glue should have a tensile strength of 18MPa, a value 29% higher than
the one obtained, but voids could be present even in retrofitting works, so the lower mean value is considered.

Unfortunately, it was not possible to obtain the modulus of elasticity from the test, since a problem occurred with the strain gauge. Further information on the structural timber glue used can be obtained from Jorge (2010), according to whom the modulus of elasticity is of 8.11GPa.

Table 3.14 - Tensile test results for structural timber glue

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\sigma_{\text{max}}$ [MPa]</th>
<th>TYPE OF FAILURE</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>14.39</td>
<td>IG</td>
</tr>
<tr>
<td>2</td>
<td>9.46</td>
<td>OG</td>
</tr>
<tr>
<td>3</td>
<td>16.23</td>
<td>OG</td>
</tr>
<tr>
<td>4</td>
<td>14.77</td>
<td>IG</td>
</tr>
<tr>
<td>6</td>
<td>13.54</td>
<td>IG</td>
</tr>
<tr>
<td>8</td>
<td>15.35</td>
<td>IG</td>
</tr>
<tr>
<td>9</td>
<td>14.07</td>
<td>IG</td>
</tr>
<tr>
<td>10</td>
<td>16.56</td>
<td>IG</td>
</tr>
<tr>
<td>11</td>
<td>11.50</td>
<td>IG</td>
</tr>
</tbody>
</table>

**average** 13.99  
**c.o.v.** 16.23%

In relation to the type of failure, most of the specimens experienced failure inside the gauge length (IG), while for only two specimens, namely specimen 2 and 3, failure occurred outside the gauge length (OG), see Figure 3.22, where the gauge length is marked. The difference in the type of failure did not appear to influence the tensile strength results.

Figure 3.22 - Failure mode of glue specimens
3.7 Conclusions

In this chapter, all the materials used during the experimental campaign on timber frame walls have been characterized by performing the most appropriate tests.

While not all the results obtained are essential for the execution of the walls and of the strengthening, they give an insight on possible failure modes that could occur during the tests. Moreover, the mechanical properties are valuable for numerical analyses dealing with modelling of the tested walls.
4.1 Introduction

As mentioned in the previous section, half-timbered walls are often adopted in seismic regions as shear walls, in order to resist to horizontal seismic actions. The particular geometry of the walls, with St. Andrew’s crosses, is able to dissipate the energy generated by the earthquake motion and the timber structure that acts as a skeleton of the building does not encounter severe damages during the earthquake (Mascarenhas, 2004; Gülhan and Güney, 2000).

Timber-frame walls are very common all over the world but little experimental information is available on their seismic behaviour. The aim of this work is to fill this research gap and better understand how these structural elements work, as nowadays more and more rehabilitation works are carried out on historical timber frame structures, often altering significantly the original structure, without any background on what is being done.

It should be stressed that a great research effort has been made on historic materials (stone, bricks and wood) and structures (masonry and mainly timber trusses) in order to gain a better understanding of their mechanical behaviour and to propose remedial and strengthening methods for world heritage structures, but only recently there has been an interest in traditional timber frame walls. Even though this type of walls is quite common, little experimental information is available on their seismic behaviour. Various reports are available on the performance of half-timbered structures after big recent earthquakes in various countries, but usually the assessment is only qualitative (Gülhan and Güney 2000; Langenbach 2009; Dogangun 2006; Gulkan and Langenbach 2004). In fact, an important
issue is to study this kind of structures under cyclic and dynamic loads to better understand their behaviour when subjected to horizontal loads, as they represent an important cultural heritage for many countries and they have to be preserved.

Only in the last few years a few experimental and numerical works have been carried out, but they do not cover a lot of information and the research is still in progress. Meireles and Bento (2010) performed some in-plane cyclic tests on unreinforced half-timbered walls and Santos (1997) performed in-plane cyclic tests on half-timbered walls taken out of an existing building. Gonçalves et al. (2012) performed cyclic tests on traditional Portuguese timber frame walls with and without infill. Vieux-Champagne et al. (2012) studied the seismic performance of Haitian wood structures filled with natural stones and earth mortar through in-plane cyclic tests. Quinn et al. (2012) and Torrealva and Vicente (2012) investigated the seismic response of timber frame walls used in upper storeys of quincha construction in Peru, considering the timber frame with and without infill. Aktas et al. (2013) performed in-plane cyclic tests on traditional Ottoman timber frame walls typical of himis construction testing walls without infill, with masonry infill and with lath and plaster, considering different timber frame geometries and the presence of openings. Hicyilmaz et al. (2012) performed in-plane cyclic tests on traditional dhajji-dewari walls.

Thus, the main aim of this work is to contribute to the state of the art and attain a better insight on the mechanical behaviour of these structural elements under horizontal loads. Indeed, nowadays more and more rehabilitation works have been carried out on historical half-timbered structures, often altering significantly the original structure, without any background on the structural behaviour of the original timber-framed structures.

The study herein is experimentally based and reports the results on an extensive experimental campaign on the seismic behaviour of half-timbered walls. For this, and due to the available experimental facilities, quasi-static in-plane cyclic tests on real scale walls were selected. It is internationally recognised (Gerardin and Negro 2000; Henderson et al. 2003) that dynamic tests are the most appropriate tests to perform when the seismic behaviour of a structure is analysed, since they subject the specimen to a real seismic excitation. However, this kind of tests is expensive and only a few labs are equipped to perform such tests.

A most common test which is usually performed is the quasi-static in-plane cyclic test, which, by means of a reaction wall to which a loading jack is connected, an appropriate displacement history is applied at the top of the wall. Different loading protocols can be applied to perform this test. Gatto and Uang (2003) studied the effects of loading protocol on the cyclic response of real scale standard construction wood frame shear walls. They compared monotonic and four distinct loading protocols, namely CUREE protocol, CUREE
near fault protocol, the SPD (Sequential Phased Displacement) and ISO protocol (see Figure 4.1).

The standard CUREE protocol (Figure 4.1a) is characterized by a pattern of a primary cycle (defined as a cycle in which a given displacement level is reached for the first time) followed by a number of trailing cycles that are equal to 75% of the previous primary cycle and is intended to model “ordinary” ground motions typical of most far-field locations. The CUREE-Caltech near fault (NF) protocol (Figure 4.1b) is intended to model “near-fault” ground motions, which are distinguished by high-energy pulses that occur near the source of an earthquake. The SPD protocol (Figure 4.1c) involves a large number of cycles with the amplitude of each cycle based on the yield displacement while the ISO protocol (Figure 4.1d) has a smaller number of cycles with the cyclic amplitude based on the displacement at ultimate load, with stabilization cycles with the same amplitude.

The authors performed tests considering each of the loading sequences presented and they found out that the performance of wood frame shear walls is highly dependent on the loading sequence imposed on them, as clearly seen in Figure 4.2, where the loading protocols are compared with the monotonic one. A great variability was found both in terms of maximum load and of ultimate displacement.

Large numbers of cycles and equal amplitude cycle appeared to be the most demanding aspects of a given loading protocol (Gatto and Uang, 2003). The ISO protocol could lead to conservative estimates of strength and deformation capacity since it resulted demanding due to its equal amplitude cycle groups. The authors do not recommend the SPD protocol.
Experimental program: cyclic tests on unreinforced walls

because it is too conservative, even if this protocol resulted the least demanding on the framing members. With respect to the walls tested with the CUREE near fault protocol, strength tended to be higher and degradation typically occurred earlier than that with the monotonic specimens. The CUREE protocol showed the most consistent failure modes with seismic behaviour, since it was especially developed for modern wood frame shear walls.

Comparisons were made also between static and dynamic test protocols (Dinehart and Shenton, 1998). The authors tested real scale shear walls statically in accordance with ASTM E-564 (2006). The shear walls were tested dynamically using the sequential phased displacement procedure (SPD). Comparing the results of the static and dynamic tests, the ultimate load measured in the dynamic tests was comparable to the static ultimate load; however, a reduced ductility was registered, since the displacement at maximum load was significantly smaller. In regard to the load-deformation curves, the static results tracked the peak dynamic results reasonably well (Dinehart and Shenton, 1998).

In the present experimental campaign, it was decided to adopt the ISO protocol, even though it can lead to conservative estimate of strength and deformation capacity. Indeed, from studies performed on real scale modern timber frame walls it appears to be time demanding due to its equal amplitude cycle groups. The choice for this loading protocol was based on the limitations of the laboratory equipment, which has a limited number of steps in a loading procedure, thus making it difficult to apply the 75% stabilization cycles without having to stop and restart the test more times. Moreover, the CUREE protocol was thought for modern timber frame wall and, therefore, for traditional half-timbered walls it was thought to adopt a more traditional approach similar to what proposed for masonry walls (Tomaževič et al. 1996).
4.2 Experimental program

The objective of this work is to study the seismic performance of traditional half-timbered walls, which are shear-resisting walls, found in ancient buildings. With this goal in mind, an intensive experimental program was carried out performing quasi-static cyclic tests on real scale walls.

In Portugal, this kind of study is of great importance, since the majority of buildings in Downtown Lisbon are built with half-timbered walls acting as shear walls and many of those buildings are being rehabilitated. However, no real information is available on the behaviour of such structural elements, since neither any earthquakes have hit Lisbon since the devastating one in 1755, nor much experimental research has been carried out to fill the information gap on this issue. For this reason, a study on the behaviour of such traditional walls is of great importance having in mind both the better insight on the seismic behaviour of these structural elements and gather knowledge to better act in rehabilitation purposes.

4.2.1 Wall specimens

Different kinds of materials could be used both in terms of timber and in terms of masonry, but it was decided to use materials adopted in the Portuguese tradition, i.e. national pine (Pinus Pinaster) and bricks for masonry. The characterization of these materials has already been discussed in Chapter 3. It has to be noticed that the bricks used were modern ones, a choice based on cost purposes and on the fact that in rehabilitation works modern materials are usually employed.

The timber frame of the walls was built in a local carpentry specialized in rehabilitation projects by specialized carpenters and masons. Real scale dimensions for the wall specimens were considered. Thus, the sectional dimensions of all the members and the size of the cells were decided according to the dimensions of existing buildings found in literature (see Chapter 2). The top and bottom beams have a cross section of 16×12cm$^2$ and all the other members of 8×12cm$^2$. The total width of the wall is 2.42m and the total height 2.36m, resulting in a height to length ratio of approximately 1.0, even if in some cases this ratio can be higher, whilst the cells are 86cm wide and 84cm high.

The connections of the main frame are all half-lap connections, as well as the connections between each two diagonals, whereas the connection between the main frame and the diagonals is made by contact (Figure 4.3). In all of the connections a nail was inserted; the nail had a square section of 4mm and a length of 10cm for all the half-lap connections, while for the contact connections between the diagonals and the main frame the size of the nail was bigger, namely a square section of 6mm and a length of 15cm. The nails were modern.
and it has to be pointed out that their length and section is smaller than the ones found in existing buildings.

![Figure 4.3 - Construction of wall specimens: (a) the half-lap connections of the main frame; (b) half-lap and contact connections of the diagonals; (c) completed timber frame (dimensions in cm); (d) detail of the type of connections adopted](image)

As it should be expected in a manual job, the connections presented some clearances, which can have some influence on the results of the shear tests. Clearances were detected at the half-lap joints of as much as 2mm. Moreover, the timber used was already dry and drying fissures were present in the timber elements, which can, in a certain extent, promote the onset of cracking propagation and failure in the walls during testing.

After the completion of the timber frame, part of the walls was filled with distinct types of infill to obtain the traditional half-timbered walls which characterize many cities in the world. Thus, besides the walls without any infill material, two additional groups of walls were considered, namely (1) timber frame walls with brick masonry infill and (2) lath and plaster walls. The use of different types of infill aimed also to assess its influence on the cyclic behaviour of timber frame walls.

The first type of infill of the walls was in brick masonry and was carried out in the laboratory by a specialized mason. The masonry pattern was suggested by a Portuguese company from Lisbon which specializes in the rehabilitation of Pombalino buildings. The masonry pattern consists of double leaf masonry with transversal series of bricks every two rows of horizontal double leaf masonry, as detailed in Figure 4.4. It was decided to use modern materials available in the market mainly to represent what it is being done nowadays in
rehabilitation works, in order to reduce rehabilitation costs. The bricks with modern dimensions (22×6×11 cm$^3$) were cut in half along the thickness (Figure 4.4a) and then other cuts were made in order to accomplish with the adopted masonry bond. The masonry pattern adopted and the cut of the modern bricks intended to attain representative masonry existing in historical constructions.

Notice that in historical buildings, bricks were usually smaller and, moreover, rubble masonry could be used, thus having a more uniform distribution of materials even in the corners. Moreover, the bricks used had a higher strength than those used in the original buildings, so they could confine more the connections.

Additionally, two lath and plaster walls were built, using wood strips with a trapezoidal section (Figure 4.5) spaced of 2cm. As mentioned in Chapter 2, this type of infill is typical of the north of Portugal, but they can be found even in frontal walls, creating a lighter and more elastic wall (Mascarenhas, 2004). The wooden strips have a tendency to confine the wall more, since they link together the posts and the beams, preventing deformations of the walls.
The wood strips were nailed with a nail gun. The wood strips were not always continuous, but of varying length, and were nailed to each timber element they encountered with at least 3 nails. Mortar was inserted between the strips (the same mortar used to build the masonry infill) and inside the timber frame for a thickness of 1.5cm on each side of the wall. To prevent a higher mortar thickness, insulation panels were inserted in the wall to reduce the free space inside, but without providing any additional stiffness or weight to the wall.

4.2.2 Experimental setup and cyclic procedure adopted

The cyclic tests were carried out using the setup illustrated in Figure 4.6a. The application of the vertical load was done by means of vertical hydraulic actuators applied directly on the three posts of the walls and connected to the bottom steel profile through steel rods which connected the actuators to a hinge welded in the bottom steel profile, so that the actuators were able to follow the horizontal movement of the wall. The bottom steel profile was connected to the reaction floor.

The horizontal displacement was applied to the top timber beam through a hydraulic servo-actuator with a maximum capacity in terms of displacement and load of 200mm and 250kN respectively. The actuator was connected by means of a 3-D hinge to the reaction wall and a two-dimensional hinge was connecting it to the wall specimen. The top beam of the wall was confined by two steel plates connected through sufficiently stiff steel rods so that cyclic displacements could be imposed to the top of the wall. The bottom timber beam was connected to the bottom steel profile in 6 points and it was confined laterally in order to prevent any kind of movement of this element.

The out-of-plane displacements, which proved to be a delicate problem for these slender walls, were prevented by a guide created in the upper beam by means of punctual steel rollers distributed along the length of the beam on the two sides of the wall (Figure 4.6b). Though linear rollers would have been able to better confine the out-of-plane movement of
the walls, their use was not possible, due to the high number of steel rods used to obtain the reaction for the vertical actuators and to make the cyclic loading possible. Timber joists were screwed to the top of the wall in order to create an additional contact surface to guide the in-plane movement of the wall.

All walls were instrumented with linear voltage displacement transducers (LVDTs), placed in strategic positions to capture the global and local behaviour of the walls. Figure 4.7 illustrates the general position of the LVDTs. The horizontal displacements at the top and mid height beam are measured on both sides of the wall. The control displacement is applied to the top beam on the right side of the wall (front view) and an LVDT of ±150mm measures the corresponding displacement on the other side of the wall (TOP). Two LVDTs of ±75mm are placed on the two sides of the walls at mid height and measure the horizontal displacement of the central beam on the two sides (ML and MR). The vertical uplift of the three bottom connections are monitored through LVDTs BR, BM and BL, having the lateral ones a range of ±25mm and the middle one a range of ±12.5mm.

The displacement in the diagonals is measured by LDVTS DF and DB at the front and back of the wall, measuring the displacements of the different diagonals in order to appreciate the effect of the compressive and tensile cycles as well as differences on the two sides of the wall. Both LVDTs have a linear range of ±50mm.

The local opening of the half-lap and nailed connections in different positions of the walls was measured through LVDTs CR, CM, CL, DH1, DH2 and DV. In order to ascertain that the bottom beam is completely fixed during the test, LVDTs BV and BH measured the vertical and horizontal detachment from the bottom profile and the lateral confinement respectively.

The cyclic procedure adopted during the tests was based on standard ISO DIS 21581 (2009). In order to better capture the highly non-linear behaviour of the walls, additional steps were added in the procedure, considering an increment in the applied displacement of 10%
Experimental program: cyclic tests on unreinforced walls (see Figure 4.8). The displacements applied were derived from a previous monotonic test performed on the wall in order to evaluate its displacement capacity, which will be discussed further in Section 4.4 (see Appendix A, Table A.1 for the detailed displacement history applied). From the displacement of 10% of the maximum load and higher, each loading step was repeated three times. After the initial cycle, two stabilization cycles were introduced. In this way, the reduction in strength and stiffness can stabilize before increasing the applied displacement in the next loading step. Stabilization cycles allow the wall to achieve the maximum damage for a given displacement.

![Displacement history](image)

Two different test speeds were adopted: one for displacement up to 10% of the maximum displacement (namely 0.05mm/s) and one for the remaining displacement levels (namely 0.35mm/s). The first speed is the one adopted in the monotonic test, which meets standard requirements (ISO DIS 21581, 2009). The second one was adopted based on a balance between low speed and tests duration, since according to standard, cyclic tests should have the same speed as monotonic ones.

It has to be noted that the cycles used in the procedure were not linear, so the speed reported is a mean velocity. Due to limitations of the test equipment, the application of the displacement was made with a sinusoidal law. From a preliminary test no difference was observed in the response of the wall between the loading procedure with a sinusoidal law and that with a linear law.

### 4.3 Vertical loading

Two different pre-compression load levels were considered, namely 25kN/post and 50kN/post. The application of different vertical load levels aimed at assessing the influence of this variable on the lateral response, considering also that it is possible that, due to the fact that half-timbered buildings have known a great rehabilitation effort in the last years, their structure and use could have changed, being feasible that additional vertical loads can act on...
this type of walls. Notice that originally these walls are intended to act mainly as shear walls. Therefore, it is convenient to study the effect of additional loads on the structural element. The calculation of the amount of vertical load to be applied was made according to Eurocode 1 (2002). The load considered includes the self-weight of the structural and non-structural elements considered, the imposed loads and the live loads.

According to information found in literature and to tests performed on the materials used, the weights presented in Table 4.1 were adopted. According to these values, half-timbered walls have a weight of 3.54kN/m, considering an average wall height of 2.5m (usually the last storey was lower, while the lower storeys were approximately 3m).

<table>
<thead>
<tr>
<th>Element</th>
<th>Weight</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood</td>
<td>5.90kN/m³</td>
<td>Lab tests</td>
</tr>
<tr>
<td>Bricks</td>
<td>18kN/m³</td>
<td></td>
</tr>
<tr>
<td>Mortar</td>
<td>17kN/m³</td>
<td>Lab tests</td>
</tr>
<tr>
<td>Timber floor</td>
<td>0.7kN/m²</td>
<td>(Ramos, 2002)</td>
</tr>
<tr>
<td>Partition walls</td>
<td>0.17kN/m²</td>
<td>(Cardoso, 2002)</td>
</tr>
<tr>
<td>Live loads</td>
<td>2kN/m²</td>
<td>(EC1, 2002)</td>
</tr>
</tbody>
</table>

From drawings of existing buildings (Santos, 2005), it was assumed an influence length of 3.5m. The partial safety coefficients were considered to be equal to 1.0 as this is an experimental work. The load of the roof was assumed to be taken by the outer load-bearing masonry walls, as this was the original configurations of Pombalino buildings. The weight of the imposed and live loads thus obtained was of 10.05kN/m.

Considering these values, a total vertical load of 75kN was obtained. In order to consider changes in the structures and a redistribution of loads, the amount of vertical load applied was doubled and its influence was studied during the cyclic tests.

In total, ten unreinforced timber walls were tested, distributed in three distinct groups according to the type of infill: (1) walls named as UIW with brick masonry infill, i.e. half-timbered walls; (2) walls named as UFW with fasquio infill, i.e. lath and plaster walls; (3) walls named as UTW, in which no infill was considered, i.e. timber-frame walls. The number 25 or 50 used in each designation is associated to the vertical load applied in each post of the walls, 25kN and 50kN respectively. The number of specimens for each typology is indicated inside brackets (Table 4.2).
Table 4.2 - Typology of the specimens tested under cyclic loading

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vertical load</th>
<th>Type of infill</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25kN/post</td>
<td>Brick masonry</td>
</tr>
<tr>
<td></td>
<td>50kN/post</td>
<td>Lath and plaster</td>
</tr>
<tr>
<td></td>
<td>No infill</td>
<td></td>
</tr>
<tr>
<td>UIW25 (3)</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>UIW50 (3)</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>UFW25 (1)</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>UFW50 (1)</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>UTW25 (1)</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>UTW50 (1)</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

In all tests, bolts were used in the protruding part of the central beam in order to prevent the local failure and ensure the continuity of the central beam, as it would be found in an existing building.

To better recognize the walls during each test, the three UIW25 walls were named UIW3, UIW3, UIW7 and the three UIW50 walls were named UIW4, UIW5, UIW6 (and UIW8, since an additional wall was tested for strengthening purposes).

### 4.4 Preliminary tests performed on half-timbered walls

Some preliminary tests were performed in order to evaluate the efficiency of the test setup, understand how the walls would behave under cyclic loading and prevent possible problems during the tests.

A first monotonic test was performed on a wall specimen from which it resulted necessary to stiffen the structure preventing the out-of-plane movement of the wall. Due to some additional problems with the control of the test, only the third monotonic test was actually successful and the stiffness of the wall had already decreased, but the test was still significant in terms of ultimate displacement, which was found out to be 101.34mm. In Figure 4.9 the procedure and the results obtained for the monotonic test are shown. According to standard ISO DIS 21581 (2009), the test was first performed in force-control, up to 40% of the estimated maximum load, then a displacement control was used until failure. For the test, a speed of 0.05mm/s was used.

Moreover, some preliminary tests (Poletti and Vasconcelos, 2012) were performed in order to assess the adequacy of the wall specimens. It was found out that the walls were characterized by a rocking behaviour, with significant uplifting of the half-lap connections at the base beam, so two tests were performed altering the bottom connections of the wall so as to try to prevent the uplifting of the posts, which caused the rotation of the wall, and try to obtain an as pure as possible shear behaviour.
A cyclic test was performed on the wall in its original condition. As it is often found in existing timber frame walls and as planned initially, the first configuration represents the initial condition, where the bottom connections had only one nail, and the possible continuity of the post between two contiguous storeys was not considered (see Chapter 2).

![Figure 4.9](image)

Figure 4.9 - Monotonic test: (a) force-control procedure for initial part; (b) Load-Displacement diagrams of monotonic test

To better understand the influence of the level of continuity in the bottom connections, additional nails were added to the three bottom connections, inserting the nails not only perpendicularly to the cross section, but also transversally (Figure 4.10a) in order to oppose more resistance to the opening of the connections. The total number of nails in each bottom connection was now 6 nails. Traditionally, a total number of two to three nails could be found in each connection, but considering that the nails used in this study were smaller, a greater number was used. The idea was to avoid the considerable high uplift of the vertical posts.

![Figure 4.10](image)

Figure 4.10 - Alterations in the original configuration of the wall: (a) additional nails; (b) base reinforced with steel plates.

Another technique used is the adoption of commercial steel plates, which were inserted at the bottom connection on both sides of the wall (Figure 4.10b). Since the half-lap connection with the addition of two steel plates has 3 shear planes, it was chosen to modify the commercial plates inserting, apart from the screws connecting the steel plate to the timber elements, a bolt that ties together the connection, avoiding the out-of-plane movements and
helping against the uplift. In fact, using simple screws to secure the plates would not have prevented the uplift and the screws would have broken in shear, as it happened during the test, where the bolt resulted being the only element securing the steel plates.

The behaviour of the wall with the initial configuration for the bottom connections was characterized by a clear flexural resisting mechanism (Figure 4.11a). The lateral posts uplift as much as 50mm and the central one reaches an uplift of 22mm. The wall fails when the bottom connections are not able to work anymore, since they offer no resistance to the rocking mechanism developed.

For the wall tested with additional nails, all the nails had the tendency to pull-out and the hysteretic loops (Figure 4.11b) are very flat for low forces, a behaviour which is associated with pinching (Figure 4.12a). This effect is correlated to the fact that the local crushing of wood when dowel-type fasteners are used, which corresponds to the attainment of the embedment strength of wood, results in the degradation of the connection in successive cycles. The clearance created, promoted by the crushing of wood, results in sliding at the connection before the contact between wood and the fastener is achieved (Piazza et al., 2005), so that the connection is again effective, leading to the gain of resistance.

Figure 4.11 - Hysteretic curves of preliminary tests: (a) original solution; (b) additional nails; (c) reinforced base
For this wall, pinching was more evident than the original configuration due to the higher amount of nails present. In this case, failure also occurs when the bottom connections do not work anymore, with the nails not being able to secure the connection. The wall was able to reach slightly higher loads when compared to the original solution, with an increase in maximum load of 7%. The behaviour of the wall is still flexural. In fact, the posts are still uplifting, even though with at a lower grade, with a maximum uplift of 36mm, corresponding to a decrease of 30% if compared to the original solution.

The steel plates were able to secure the posts to the base of the wall, allowing a minimal uplift (5mm), since the combination of plates and bolts was preventing a free uplifting. The hysteresis loops are still characterized by pinching (Figure 4.11c), but this time the behaviour is predominantly a shear one. The constraining of the bottom connections allowed the wall to gain in terms of ultimate load, which increased by 43% when compared to the original solution. The steel plates deformed in the linear range and the bolts only experienced slight deformations, preventing the uplift of the posts. The majority of the screws however failed in shear as the heads were cut off. Moreover, with this solution, the connections at mid height were the ones more involved in resisting the shear stresses, so now the damages are concentrated at that height and the failure occurred at the lateral connection at mid height, in correspondence of the half-lap connection (Figure 4.12).

![Figure 4.12 - Damages in preliminary tests: (a) wall with additional nails; (b) wall with steel plates](image)

In terms of dissipated energy, the three walls exhibited a similar trend, while ductility resulted higher for the original solution and lower for the wall with the reinforced base (Poletti and Vasconcelos, 2012).

Considering the overall behaviour of the walls and the traditional connections found in existing buildings, and considering that the walls tested will then be retrofitted, the choice made was to maintain the original solution of the unreinforced half-timbered wall with only one nail in each half-lap connection. The behaviour of the wall is a clear flexural one, while
the walls should behave as shear walls, so their full seismic capacity is not taken advantage of. But it has to be taken into consideration that the gain in terms of ultimate load and energy dissipation (Poletti and Vasconcelos, 2012) are significant only when compared to the results of the specimen with steel plates applied to the bottom connections, but this configuration could be considered already a strengthening solution, since various existing buildings present weak connections at the base of the floor, being the posts discontinuous. So in this study, it was chosen to admit the flexural behaviour of the half-timbered walls and try to improve their performance in the strengthened configuration. But it has to be pointed out how, just by changing the connections of one level of the wall its overall behaviour alters and grants a more active participation of the wall in the shear absorption of the structure.

4.5 Experimental behaviour of half-timbered and timber-frame walls

Cyclic test results performed on both infill and timber frame walls are here presented and a discussion of their general behaviour is reported. The presentation and discussion of the results can be divided into three parts, namely: (1) discussion of the typical force-displacements hysteresis diagrams; (2) discussion of the main deformation features and typical failure modes; (3) assessment of seismic performance indicators.

4.5.1 Typical hysteretic diagrams

All unreinforced half-timbered walls subjected to the same vertical pre-compression level present a similar behaviour. Walls UIW25, tested with the lower vertical pre-compression load, present a predominant rocking behaviour (see Figure 4.13a), characterized by the S-shape of the force-displacement diagrams (Magenes and Calvi, 1997), with a significant vertical uplift of the posts. In Figure 4.13, complementary to the hysteresis diagrams, the evolution of the vertical displacements measured by LVDTs placed at the bottom connections of the walls versus the top lateral displacement of the walls is also shown. It is seen that the lateral posts were uplifting as much as 50mm, pointing out the important rotation experienced by the walls.

It should be noticed that the low vertical level applied to the wall, in conjunction with the aspect ratio of the walls (approximately 1), contributes in great extent to the predominant flexural rocking mechanism. Besides, the geometry of the connections at the base of the walls is also important for the promotion of this resisting mechanism. In fact, the half-lap connection at the base of the wall was the most vulnerable to tensile forces, given that only one nail was added and, additionally, in this part of the walls, the connection has no continuity, contrarily to the other connections of the posts. Notice that this trend for the uplift
of the wall with the detachment of the post at the base is also seen in case of other typologies of cantilever shear walls, namely masonry walls, with the opening of horizontal cracks (Magenes and Calvi, 1997).

Once the connections were completely open, the walls exhibited a low decrease in terms of strength, since the bottom connections were not working anymore. This behaviour is typical of the rocking mechanism, for which high displacements can be reached with little reduction of strength (Magenes and Calvi, 1997).

Figure 4.13 - Hysteretic curves for the different timber frame walls: (a) brick masonry infill - lower vertical load level; (b) brick masonry infill - higher vertical load level; (c) lath and plaster wall - lower vertical load level; (d) lath and plaster - lower vertical load level; (e) timber frame wall - lower vertical load level; (f) timber frame wall – higher vertical load level.
The hysteretic curves are quite flat at the origin, when there is the load inversion, which is associated to the pinching effect. The behaviour of the walls was not symmetrical in the first cycles, becoming then more symmetrical as the displacement in the walls increased, which can be attributed to some imperfection on the connections. Generally, higher loads were reached in the positive direction, while the ultimate displacement was slightly higher in the negative direction. This load asymmetry can be related to the asymmetry in the lateral load application, as only one horizontal actuator is used. The same behaviour was observed for all 3 walls tested. In Appendix A, Figure A.1a, the hysteretic diagrams of all walls are presented. It appears that wall UIW2 reached slightly higher values of load, in average 11% more than the other two walls. This could be due to a better manufacturing of the frame or of the infill.

The rocking mechanism was evident even in walls submitted to the higher level of vertical pre-compression load (walls UIW50), even if in this case a shear component was clearly present (Figure 4.13b). The uplift of the vertical posts occurred during these tests too, in a lower amount (up to 35mm), but it still conditioned the shape of the hysteretic loops. The walls exhibit a progressive loss of stiffness, even though the ultimate load does not differ greatly from the maximum one.

It should be mentioned that the boundary conditions can have an important effect on the shape of the force-displacement diagrams, particularly in the unloading regime of the response. In fact, the timber walls are tested as cantilever. Besides, it is seen that the vertical posts can detach from the bottom beam, which appears to reproduce the real condition existing and observed on site.

Indeed, if the form of the hysteretic loops is compared with the uplift of the vertical posts for the same horizontal displacement, one can notice that: (1) the change in stiffness in the loading branch starts when the lateral posts start uplifting; (2) the plateaux that occur in the unloading branch of each cycle occur when the bottom connections start closing. This same behaviour was observed in other tests performed on walls with the same geometry (Gonçalves et al., 2012). Moreover, it was seen how in the preliminary tests (Section 4.4), a lower amount of uplifting would create a smoother unloading branch. The same shape of the unloading branch is present in cyclic tests results performed on dhajji-dewari walls (Hicyilmaz et al., 2012), where the response of the wall was characterized by a significant uplift of the bottom connections, which in this case were mortise and tenon. This peculiar form is not present, for example, in similar tests conducted on modern timber walls (Lam et al., 1997; Varoglu et al., 2006) or historical half-timbered walls (Meireles and Bento, 2010; Santos, 1997). The test results found in (Santos 1997), related to similar traditional walls taken from existing structures, point out clearly distinct features of the unloading branch of the force-
displacement diagrams. In this case however, the base of the wall was concreted into a concrete slab, and thus completely fixed at the base, avoiding the possibility of detaching the posts from a bottom wood beam. Even for the higher vertical load, very similar results were observed when the test was repeated. In Appendix A, Figure A.1b, the hysteretic diagrams of all walls are presented. Even though the maximum load reached was similar for all walls, some differences can be observed in terms of dissipated energy and stiffness of the walls, which will be discussed later.

The hysteresis diagrams found for lath and plaster walls for both vertical load levels are very similar to those observed for half-timbered walls, see Figure 4.13c,d. Wall UFW25, tested under the lower vertical load, exhibited a clear flexural behaviour, characterized by an even more remarkable S-shaped diagram (see Figure 4.13c). In case of wall UFW50, tested with the higher vertical load level, a mixed flexural/shear behaviour characterizes the response, similarly to what was observed for the walls tested with brick masonry infill. The confining effect given by the timber strips assured an important lateral stiffness, limited the working of the central connections leading thus to the damage concentration at the base of the posts. The behaviour was confirmed by the vertical uplift of the posts. The uplift was higher for the wall tested with the lower vertical load, with a vertical uplift reaching up to 50mm. The walls UFW50 reached a vertical uplift of 32.43mm, still significant (Figure 4.13d). Also in this case, it is possible to notice in the unloading branch of the hysteretic loop there is a plateau for decreasing displacements, which is attributed to the closing of the connections at the base of the posts.

The higher asymmetry in the cyclic response, namely at the level of the maximum lateral strength observed in lath and plaster walls can be attributed to the asymmetry on the damage patterns of these walls (see Section 4.5.3). The asymmetry is more relevant in case of the walls submitted to the higher level of vertical pre-compression. Moreover, these walls, due to the higher stiffness they reached, experienced some out-of-plane movement, noticeable for the lower displacement capacity attained by the wall UFW50.

Timber-frame walls exhibited a different behaviour in relation to infill walls. The shear resisting mechanism predominated in the lateral response over the minor flexural component. But, as it can be noticed from the hysteretic diagrams (Figure 4.13e,f), the walls experienced severe pinching. This appears to indicate that the pinching can in a certain extent be avoided by the infill, both brick masonry and lath and plaster. Moreover, the unloading branch of the various loops is more regular, even if the plateau characterizing the post uplifting is still present.
For both vertical load levels, the vertical uplift of the posts was similar and minimal, if compared to that of filled walls (13.8mm for UTW25 and 17.51mm for UTW50). Contrarily to what happened in infill walls, the uplift was higher for the wall tested with the higher vertical load level, but this was due to the fact that the wall failed for higher levels of horizontal displacement and the posts were more stressed. UTW25 exhibited an early failure and after that the posts were stressed less, thus uplifting less.

For wall UTW25 (Figure 4.13e), once the wall failed (at an applied displacement of 30.36mm), it was possible to observe an accommodation of the wall, which recuperated strength, even though stiffness degraded, managing to reach almost its maximum load. This resulted from a stress redistribution; after the central connection failed, stresses were redistributed to other connections, allowing the wall to regain strength. The same did not happen to the wall with the higher vertical load, as failure occurred for higher values of displacement (applied displacement of 70.85mm) and it was not able to recover some strength.

As it can be deduced from the analysis done until now, the presence of infill greatly changes the response of timber-frame walls to cyclic actions. The type of infill, though, does not appear to overly influence their behaviour.

Moreover, the amount of vertical pre-compression applied to the walls greatly influences their behaviour. It changes their response to cyclic actions, since a higher pre-compression leads to a stiffening of the wall and to a greater load capacity. Half-timbered walls with brick masonry infill gained in average 64.7% in terms of maximum load, while only losing 2.8% in terms of ultimate displacement. Lath and plaster walls gained 29% in terms of maximum load, but their ultimate displacement decreased of 2.8%. In case of timber frame walls an increase of vertical pre-compression resulted in an increase on the lateral resistance of 104.6%, with a loss in terms of ultimate displacement of only 2.7%.

For all unreinforced walls, it was noticeable that, although there was strength degradation between one step and the next one, i.e. in a step (n+1) at the displacement corresponding to the maximum displacement of the previous step (n) the load reached by the wall was lower than that reached at the previous step for that displacement (Figure 4.14a), strength degradation was minimal among the three repeated cycles in one step (Figure 4.14b). The examples reported refer to the step corresponding to 70% and 80% of the maximum displacement. For UIW25 walls, the loss in terms of strength is less than 5kN. The same trend was noticeable for UIW50 walls (Figure 4.15a), even if the strength degradation between two successive steps is slightly higher, but not exceeding 10kN. The strength
degradation in the three cycle repetitions is still minimal. The same happened in lath and plaster walls for both vertical load levels.

![Figure 4.14 - Strength degradation in UIW25 walls: (a) strength degradation between two successive steps; (b) strength degradation among three cycle repetitions](image)

Even for timber frame walls, strength degradation was minimal in both cases, but for wall UTW50 failure led to an important strength loss (Figure 4.15b). The same did not occur for half-timbered walls, even if the maximum load was achieved at this step.

This behaviour is attributed to the rocking behaviour with a low level of damage of infill walls. Timber frame walls, instead, experienced severe damage, as it will be illustrated in the following sections.

![Figure 4.15 - Strength degradation between two successive steps: (a) UIW50 walls; (b) UTW50 wall](image)

### 4.5.2 Deformational features of the walls

Besides the uplift of the post analysed previously, some other deformational features are also here analysed in order to better understand the lateral behaviour of the different walls. Notice that the bottom beam did not uplift from the steel profile on which the wall was anchored. Moreover, horizontal displacement of the bottom beam was controlled and found out to be less than 0.5mm.
The behaviour of the diagonals was similar in all three UIW25 walls. The main diagonals were opening and closing according to the forces acting along the diagonal being tension or compression. The movement was generally higher when the element was in tension, which should be associated to the possibility that the contact connections between the diagonal and main frame are able to detach. If one element failed, the diagonals would experience greater displacements, since the opening of the connections would allow a higher movement to the diagonals. In general, the displacement of the diagonals in UIW25 walls was of 10mm when in compression and 18mm when in tension, with little variations among the walls (see Figure A.2 in Appendix A).

The displacement along the diagonals was higher for UIW50 walls (Figure 4.16a). This result is in correspondence with the more evident shear behaviour of the walls for this vertical pre-compression. For all walls, the diagonals reached a displacement of about 30mm, apart from wall UIW4, which reached a displacement of 41.6mm since the protruding part of the central beam on the left fell off and, since the wall opened in plane enlarging, the diagonals were able to move more (see Figure A.3 in Appendix A).

It is observed that the displacement in the diagonal of walls UTW are clearly higher than the ones recorded in the filled walls. The maximum displacement measured in the UTW walls subjected to 50kN/post is 20mm higher than in case of walls UIW submitted to the same level of pre-compression, see Figure 4.16b. This behaviour revealed the much more relevant shear resisting mechanism exhibited by UTW walls. Indeed, the failure for both UTW walls occurred when the central beam failed in shear, due to the high shear concentration induced by the diagonals. In fact, since the masonry infill was not present to stiffen the walls, the diagonals were able to move freely, inducing greater damages to the walls. In both walls, the main diagonal reached a deformation of approximately 50mm both in the positive and in the negative direction once the central beam failed, which caused the increase in the rate of displacement of the diagonals. In Figure 4.16b it is clearly visible how, once the central beam failed in shear, the diagonals started opening with a higher rate.

For filled walls, the higher movement experienced by the diagonals as well as a non-linear deformation of the vertical posts is also more relevant in case of walls submitted to higher levels of vertical load. The lower uplift allows the shear deformation to be more significant, thus elongating the diagonals. When the uplift of the walls is high, the vertical post detaches from the bottom beam and the rocking mechanism predominates, resulting in lower deformation in the diagonal. In this case, the diagonals rotate together with the wall in a more rigid way. In Appendix A (Figure A.3 to Figure A.5), the diagonal displacement of all walls is reported.
For infill walls, when the global horizontal displacement at mid height of the walls submitted to the lowest level of vertical pre-compression is compared to the top displacement of the walls, it is seen that it is approximately half of the top displacement (Figure 4.17a). This explains why posts did not experience significant bending and justify the rocking deformation of the wall. Besides, the displacement measured on the right and left sides of the walls are approximately the same, which confirms also the absence of local relative deformations at the connections and thus the predominant rigid deformation of the walls. In UIW50 walls, the horizontal displacement reached in the central beam is higher than half of the horizontal displacement reached at the top of the wall (54.3mm vs. 84.29mm for wall UIW6, for example), which means that the lateral displacement has an additional component beyond that associated to the rotation of the wall and that corresponds to the shear component of deformation. The same behaviour was observed for lath and plaster walls with the same distinctions for the two vertical load cases. In case of UTW walls, where the shear deformation is predominant, the lateral displacement at mid height of the walls in the right and left sides of the walls measured by the LVDTs MR and ML respectively are completely different when a certain loading direction is considered. Once the central connection failed in shear, the increase on the lateral deformation of the walls led to the increase of the crack opening and thus to the remarkable elongation of the diagonals. This effect is measured by LVDTs MR and ML when the wall is pulled or pushed respectively. When the wall is pushed, the displacement measured by LVTD MR is low as it is in the part of the wall not affected by the shear crack opening of the central connection. The same behaviour is recorded in the LVDT ML when the walls is pulled, confirming the symmetry of deformations under positive and negative directions of the lateral load. With the crack opening at the central connection it is not possible to have a full displacement transfer between border posts. In Appendix A (Figure A.6 to Figure A.9) the mid-height displacement of all walls is shown.
Experimental program: cyclic tests on unreinforced walls

Figure 4.17 - Displacement at mid height of the wall: (a) UIW25 wall – lower vertical load level (25kN/post); (b) UTW25 wall – lower vertical load level (25kN/post)

Notice that crack opening at the central connection increases considerably as the lateral displacement increases, see Figure 4.18, causing the differential opening on the two sides of the wall. It is clear that, after failure, the connection was opening progressively, until it was not possible anymore to register its opening, due to reaching the end of the linear range of the transducers.

Figure 4.18 - Opening of the half-lap connections at central mid height for timber-frame walls – intermediated half-lap connection CM

The movement of the diagonals could also be observed in the single connections, where the nailed connections between the diagonals and the main frame were opening various millimetres during the test. From Figure 4.19 and Figure 4.20, where the horizontal component of the displacement (DH1 and DH2) of the diagonal bars are displayed for half-timbered and timber-frame walls, it is seen that the diagonals were detaching in one direction, while in the other direction they were cutting the connections. This means that the posts separate from the diagonals when tensile forces are applied along the diagonals. The separation is increasing continuously in the case of half-timbered walls, while for timber-frame walls there is a significant increase in the detachment near failure. In the case of lath and plaster walls (see Appendix A), the wooden strips, connecting the diagonals to the main
frame, contribute to the limitation of the separation between the diagonal and the main frame due to the lateral confining effect. A similar opening was observed even in the vertical direction, reaching openings of 9mm for displacement values of 80mm.

In UIW25 walls, the movements of the diagonals are reduced, when compared to UIW50 walls (Figure 4.19). In this case, the movement is more marked in the vertical direction, due to the higher uplifting of the posts, while in the horizontal direction the detachment of the diagonals is significantly lower (see also Appendix A for further results).

For UIW50 walls, from Figure 4.19b,d it is possible to notice how the higher vertical load changed the movement of the diagonals, which is now more marked in the horizontal direction and lower in the vertical one. This trend was observed for all types of walls (see Figure 4.20 and Appendix A).

When observing the opening of the nailed connections between the diagonals and the main frame for timber frame walls (Figure 4.20b), it is seen how, after failure, the detachment of the diagonals from the main frame is higher than in infill walls, since the diagonals had a higher degree of movement capacity.
Moreover, up to failure there was some detachment of the connections of the upper level, other than the ones at the bottom; after failure, the detachment of the diagonals from the main frame increased of more than 100%. In particular, at the bottom, from visual observations at an applied displacement of 60.73mm there was a vertical uplift of the diagonal of 20mm, which after failure decreased, as happened to the vertical uplift of the posts, while the detachment of the diagonals from the top beam increased from 5mm to 30mm.

It has to be pointed out that in case of infill walls the diagonals exhibited equal uplifting to the vertical posts (see Figure 4.21), given that both masonry blocks and wooden strips were preventing an independent behaviour of the two elements. In case of timber frame walls without infill, the diagonal elements can move independently. This is also an indication of the fact that the strong flexural behaviour of infill walls was mainly associated to the presence of the infill material. In fact, in timber frame walls, the movement of the diagonals reached values of 30mm for displacement values of 80mm, measured manually during the tests due to lack of transducers in those connections.

Figure 4.21 - Comparison between vertical uplift of bottom connection and vertical uplift of adjacent diagonal for UIW50 wall
Unfortunately, it was not possible to monitor all connections with transducers, but some additional results are shown in Appendix A.

The distinct infill material influences also the horizontal opening of the half-lap connections at mid height of the distinct walls at the border posts, see Figure 4.22. It is observed that the half-lap connections between central beam and external posts are higher in case of UIW walls, when compared to UFW walls. This difference is associated to the limitation of relative displacements imposed by the application of wooden strips. In fact, this type of infill ties together the whole wall, limiting the in-plane opening of the connections. On the other hand, in UTW walls the opening of the half-lap connections was limited up until failure, when the connections were opening freely (Figure 4.22c).

For infill walls, the deformations of the half-lap connections at mid height are even lower when the lower vertical load is considered. For UIW25 walls, all openings of these connections are lower than 2mm (see Figure 4.23). For UIW50 walls, the values are higher, reaching 8mm or more in cases where one connection failed (see Figure 4.22a). Higher deformations are only registered when the connection fails.
Experimental program: cyclic tests on unreinforced walls

This points out that the vertical pre-compression has also influence on the maximum opening of these connections. To sum up, it can be concluded that in general, for infill walls tested with the lower vertical load level, the opening of connections was small and not very significant, the walls were rotating as a whole, thus allowing little possibility to local deformations. On the other hand, infill walls tested with the higher vertical load level experienced important deformations in the connections, with openings in both half-lap and nailed connections. Timber walls instead experienced, for both vertical load levels, important deformations in all connections, since the absence of infill guaranteed a prevalent shear component and free deformation capacity to the walls.

4.5.3 Typical damage patterns

The distinct deformational features of the walls discussed previously resulted from distinct damage patterns exhibited by the different walls. The typology of walls is particularly relevant in the damage patterns when infill and unfilled timber-frame walls are compared.

The predominant resisting mechanism exhibited by half-timbered walls was rocking, particularly in case of the lower vertical pre-compression load level, see Figure 4.24a. As mentioned above, when a wall rocks, it can achieve large displacements without a significant loss in strength and thus with low damage. In the case of the half-timbered walls here analysed, the damages found were not great and thus it was generally simple to reassemble the walls for retrofitting.

Since the walls tended to rotate, the posts did not experience significant bending, but they maintained their linearity, as it can be attested when comparing the displacements measured at the top of the wall and at its middle height.

The same behaviour was observed for lath and plaster walls, see Figure 4.24b. For timber-frame walls, instead, the response was mainly a shear one and the posts would bend (Figure 4.24c).
In case of brick masonry half-timbered walls, the detaching of brick masonry from the timber frame (Figure 4.24a) was evident due to rocking movement. As the confinement of the frame was loosening during the test, masonry tended to move out-of-plane, moving out as much as 2cm. The vertical posts and the diagonals were clearly uplifting (Figure 4.25a) and the nail placed in the half-lap connections offered little resistance to the tearing force provided by the post (this point will be further discussed in Chapter 6).
During the tests, the bottom connections presented a tendency to open out-of-plane, so that the post would come out (see Figure 4.25b). The movement mainly involved the lower connections, but some opening was observed even at mid-height connections, which was triggered from the bottom ones. Moreover, the cyclic movement of the walls contributes for the trend of pulling-out of the nails from the connections (Figure 4.25c). After the test, the nails at the bottom connections were severely deformed as the result of the pulling effect of the posts in correspondence to the half-thickness of the half-lap connection (Figure 4.25d).

The damages at the connections were more relevant in UIW50 walls, both in terms of masonry cracking and damages to the timber frame. In particular, crushing of the central half-lap connection was observed, a loosening of the half-lap connections at mid height, with probable damage to the connections from nail pulling of the diagonals. Figure 4.26a,b shows the typical crack pattern at the end of the test in brick masonry half-timbered walls.

Notice that for masonry infill walls, most of the damage is concentrated in the lower part of the wall. In general, masonry was not greatly damaged, even if most of the cracks were developing at the unit-mortar interfaces for walls submitted to the highest pre-compression level and only some bricks fell out mainly due to nails pulling out from the diagonals and causing the damages to masonry. Besides, the interface between masonry and timber was very weak, leading to the early detachment of the masonry from the main frame as mentioned above. Masonry tended to behave as a block, while the wall was rocking, the triangles filling the frame were moving accordingly. When the wall was uplifting significantly,
the masonry in the lower part of the wall would “fall down” and it did not offer additional stiffness to that part of the cell.

In UIW4 the protruding part on the left broke, leading to a more significant crushing of the central connection, since the beam was able to move through the notch in the post without meeting any resistance (Figure 4.27a), only that of the nail which was easily tore out of place. This led to a severe crushing of the central connection (Figure 4.27b), leading to the higher asymmetric response of the wall (as it can be seen in Appendix A, Figure A7.a).

![Figure 4.27 - Damages in walls: (a) protruding part of UIW4 fell off; (b) crushing of central connection](image)

In order to avoid this local failure in the following tests, bolts were applied in the protruding parts in order to simulate the continuity of the central beam in an existing wall and avoid these weak points in the wall.

It has to be reported that UIW8 wall had different damages than the rest of the walls. Crushing in the central connection did not occur, but instead the posts were opening out-of-plane in all three points of the post (Figure 4.28a), particularly in the central post: the top connection opened 5mm, the middle one 10mm and the bottom one 35mm. In the other walls too, the posts would move out-of-plane, but in a less severe way. The higher movement would concern the lateral posts and not the central one.

For all specimens, the walls suffered a permanent deformation (Figure 4.28b): the post is not vertical anymore and the central beam generally passed from being 2,42m long to being 2,44m. This later led to some problems in the retrofitting of the walls.

In lath and plaster walls the damages on timber connections were similar to the ones observed in half-timbered walls. The walls exhibited rocking mechanism, particularly for the lower level of pre-compression, see Figure 4.24b, and the damages were higher for the higher vertical load. It was clearly visible that no timber strips broke but bent where the post was uplifting.
Besides, in UFW50 wall, the left post failed at the base, due to the stress concentration associated to rocking, inducing also crushing of the bottom beam (Figure 4.29). This led also to remarkable out-of-plane deformation due to the lack of support. On the other side, the same failure led to a more accentuated flexural behaviour at the end of the test when the wall was being pulled (negative direction of displacement), since the connection had lost its stiffness. The damages in the mortar were more noticeable in the lower part of the wall for the lower vertical load level (Figure 4.26c), as happened for masonry infill walls. For the higher vertical load, the cracking density was higher, developing both at the timber elements borders, as in the wall submitted to the lowest pre-compression, and in the triangular cells, see Figure 4.26d.

As already mentioned, the timber frame walls presented a much more relevant shear deformation. Thus, contrarily to what happened in infill walls, the damages in the timber
frame walls were concentrated at the central beam, namely at the central connection, where diagonal elements converged.

In fact, since the diagonals were able to move freely due to the absence of the infill, and the posts experienced minimum uplifting, the shear stresses induced by the diagonals in the central beam were higher and in both cases the central connection in the beam crushed due to the lateral compression applied by the diagonals. Even the lateral connections experienced higher stresses and in case of wall UTW25 the right connection failed, due to the major shear action of the diagonals and to the stresses redistribution which occurred after the early failure of the central connection (Figure 4.30).

![Figure 4.30 - Typical damages in timber frame walls: (a) failure of lateral connection; (b) opening of lateral connection](image)

According to what was mentioned in the previous section, the cracking at the central connection promoted the elongation of the diagonal leading to the separation between the diagonals and the main frame, see Figure 4.24c and Figure 4.31a, with openings easily visible. The walls subjected to the higher pre-compression load experienced more severe damage than the walls under the lower load. Moreover, contrary to what happened in infill walls, the displacement of the diagonals was independent of that of the posts. At the end of the tests, permanent deformation was met in all diagonals (Figure 4.31b), but it was easily repairable.

Independent of the type of infill, for the higher vertical load level the failure of the walls occurred when the central connection of the central beam began crushing (Figure 4.32a). Even if some strength degradation occurred, the ultimate displacements are comparable to those of the walls tested under the lower pre-compression.

The differences in terms of damage to the central half-lap connection according to the distinct types of walls can be seen by comparing the damage patterns shown in Figure 4.32, obtained for the highest level of pre-compression. In both filled walls, some deformation of the central connection was observed but it was not enough to result in severe damage as in
the timber walls. In case of lath and plaster walls (Figure 4.32b), the central middle connection did not crush, because the damage was moved to a weak point in the central post, i.e. a knot from which cracking propagated causing the failure of the central post.

![Image](a) ![Image](b)

**Figure 4.31** - Typical damages to timber walls: (a) opening of diagonals; (b) permanent deformation after test.

For half-timbered walls (Figure 4.32c), the crushing of the central connection was concentrated on the post, and not on the beam as for timber frame walls, pointing out that for infill walls the majority of the stresses was assumed by the post, indicating a different stresses distribution for the two wall typologies.

![Image](a) ![Image](b) ![Image](c)

**Figure 4.32** - Damages in central connection: (a) timber frame walls; (b) lath and plaster walls; (c) half-timbered walls

The confining effect given by the infill guaranteed that the connection did not open at the same levels of those of timber walls. Moreover, the damage and relative opening were more progressive.
4.6 Evaluation of seismic performance

In the seismic design of new timber structures or in the rehabilitation of existing structures, including historic timber frame walls, the study of the seismic performance is of paramount importance. Since the seismic response of timber structures is very complex and time dependant, a better understating of the hysteretic factors that govern the problem is important for a safe and economical seismic design or for the adoption of the most adequate retrofitting measures. Parameters such as ductility, energy dissipation, overall cyclic stiffness, equivalent viscous damping ratio and lateral drifts characterize the behaviour of timber shear walls and are helpful in evaluating the performance of a structure under cyclic loading. In this section, the main seismic parameters are presented for the walls previously analysed.

4.6.1 Obtaining the bi-linear idealized diagrams

Aiming at obtaining the equivalent bilinear diagrams, which are a perfectly elasto-plastic representation of the actual response of the wall specimens, the monotonic envelopes for each wall tested were defined for the two load cases, see Figure 4.33. The average monotonic results are presented for half-timbered walls; the variation in terms of envelope curves for the single walls can be found in Appendix A, Figure A23a,b. The monotonic envelope curves are defined as the curve connecting the points of maximum load in the hysteresis plot in each displacement level (ISO DIS 21581 2009). According to the standard, the envelope curves should be plotted for both the initial cycle and the two stabilization ones, but, since low variations were observed during the tests among the three repetitions, the envelope curves of the two stabilization cycles were discarded.

All walls present softening behaviour, but the strength degradation is not much remarkable. Only three walls, namely UFW25, UIW4 and UTW50, had a loss in strength higher than 20%.

It is seen that in average the type of infill material has some influence on the initial stiffness and maximum lateral resistance but it depends mainly on the vertical pre-compression level. In case of low levels of pre-compression load (25kN/post), the maximum resistance is higher in case of lath and plaster walls, whereas the initial stiffness has no change. In walls with higher level of pre-compression, the maximum load is approximately the same and the initial stiffness is higher for lath and plaster walls. In case of timber walls with no infill, it is seen that they present both lower initial stiffness and lower lateral strength for both load levels, even if the differences are less marked for the higher load. This difference on the lateral resistance for the lower level of pre-compression should be attributed to the clear difference on the resisting mechanism that predominates for each typology. In fact, half-timbered walls
submitted to the lower level of pre-compression present a predominant rocking behaviour, contrarily to the timber frame wall, which behaves predominantly in shear.

When the vertical load applied increases to 50kN/post, the half-timbered walls present some rocking but it is clear that a shear component of deformation is present, which is closer to the shear behaviour of the timber frame walls.

Table 4.3 to Table 4.6 present the significant values of the envelope curves used to obtain the bilinear idealizations, namely the maximum load ($F_{\text{max}}$), the displacement at maximum load ($d_{\text{max}}$), the ultimate displacement ($d_u$), the average values (avg). All results are presented for both the positive (P) and negative (N) direction.

The envelope curves of the three specimens tested for the lower vertical load level are very similar. The maximum load varied between 69.43 and 60.62kN, the minimum between -67.49kN and -58.55kN with a coefficient of variation (C.O.V.) of 6.95% and 7.09% for the positive and negative direction respectively. The variation in terms of ultimate displacement was even less, namely 0.99% and 2.34%, respectively. Generally, the walls had a higher displacement capacity in the negative direction, possibly because of a lower uplifting, thus a lower rotation in that direction.

Moreover, from Table 4.3 it is possible to observe how the loss among initial and stabilization cycles in terms of maximum load and ultimate displacement is minimal, with a coefficient of variation which ranges from 0.8% to 3.14% for the maximum load and 0.05% to 0.22% for the ultimate displacement. Instead, there is a difference in terms of displacement at maximum load, as maximum load for the stabilization cycles was not always reached at the same displacement step as for the initial cycle. Since the walls are rotating, the ultimate displacement in both directions is affected by the rotation. In fact, the maximum applied displacement was 91.09mm, but usually approximately 10mm were lost due to clearances in the connections as well as rotation of the wall.
For UIW50 walls (Table 4.4), the values of maximum load and ultimate displacement for the walls are once again similar, with coefficient of variations all under 5%. The maximum load varied between 112.84kN and 102.35kN while the minimum between -96.87kN and -106.61kN.

The ultimate displacement varied between 80.18mm and 85.41mm in the positive direction and between -77.03mm and -84.84mm in the negative one.

Once again the variation between the initial cycle and the two stabilizing cycles is minimal. In this case too, the maximum load was not always reached at the same displacement step among the three cycles. Moreover, all walls reached higher values of maximum load in the positive direction; the ultimate displacement, instead, was higher for the positive direction, contrary to what happened for UIW25 walls.

The increase of vertical pre-compression level led to higher values of maximum load, with a gain in terms of average values of 68.76% in the positive direction and 60.67% in the negative one.

Considering lath and plaster walls, wall UFW50, tested with the higher vertical pre-compression, reached a maximum load similar to that reached from the walls with infill masonry, being only 3.6% lower than the average value of UIW50 walls. Wall UFW25 instead, reached a maximum load 10% higher of that obtained from UIW2, which had the higher maximum load, and 18.3% if compared to the average value of maximum load for UIW25 walls.

The same conclusions can be drawn for timber frame walls. Wall UTW25 experienced a low capacity, lower than what encountered for infill walls (21% in case of fatchwork walls and 14% in case of masonry infill walls). For the higher vertical load level, instead, the difference was not very noticeable (3% in case of fatchwork walls and 6% in case of masonry infill walls).

In Appendix A Figure A.23, all envelope curves are shown individually for each wall typology and for each vertical pre-compression load to facilitate the comparison.
Table 4.3 - Significant values for half-timbered walls, lower vertical load level

<table>
<thead>
<tr>
<th>WALL</th>
<th>F&lt;sub&gt;max&lt;/sub&gt; [kN]</th>
<th>d&lt;sub&gt;max&lt;/sub&gt; [mm]</th>
<th>d&lt;sub&gt;u&lt;/sub&gt; [mm]</th>
<th>F&lt;sub&gt;max, avg&lt;/sub&gt; [kN]</th>
<th>C.O.V. F&lt;sub&gt;max&lt;/sub&gt; [%]</th>
<th>d&lt;sub&gt;u, avg&lt;/sub&gt; [mm]</th>
<th>C.O.V. d&lt;sub&gt;u&lt;/sub&gt; [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>UIW2</td>
<td>69.43</td>
<td>63.56</td>
<td>60.62</td>
<td>63.98</td>
<td>54.88</td>
<td>46.92</td>
<td>81.64</td>
</tr>
<tr>
<td>UIW3</td>
<td>-67.49</td>
<td>-58.55</td>
<td>-63.42</td>
<td>-75.06</td>
<td>-55.68</td>
<td>-52.25</td>
<td>-86.07</td>
</tr>
<tr>
<td>UIW7</td>
<td>68.09</td>
<td>61.02</td>
<td>60.05</td>
<td>55.09</td>
<td>54.98</td>
<td>55.76</td>
<td>81.64</td>
</tr>
<tr>
<td>UIW2</td>
<td>-65.75</td>
<td>-56.90</td>
<td>-61.99</td>
<td>-63.72</td>
<td>-55.73</td>
<td>-52.29</td>
<td>-85.87</td>
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<tr>
<td>UIW3</td>
<td>67.39</td>
<td>59.77</td>
<td>59.66</td>
<td>55.06</td>
<td>55.02</td>
<td>55.77</td>
<td>81.57</td>
</tr>
<tr>
<td>UIW7</td>
<td>-64.89</td>
<td>-55.91</td>
<td>-61.19</td>
<td>-74.79</td>
<td>-55.76</td>
<td>-52.31</td>
<td>-85.83</td>
</tr>
<tr>
<td>avg</td>
<td>68.30</td>
<td>61.45</td>
<td>60.11</td>
<td>58.04</td>
<td>54.96</td>
<td>52.82</td>
<td>81.62</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>1.52</td>
<td>3.14</td>
<td>0.80</td>
<td>8.86</td>
<td>0.13</td>
<td>9.67</td>
<td>0.05</td>
</tr>
<tr>
<td></td>
<td>2.01</td>
<td>2.33</td>
<td>1.82</td>
<td>9.09</td>
<td>0.07</td>
<td>0.06</td>
<td>0.15</td>
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</table>
Table 4.4 - Significant values for half-timbered walls, higher vertical load level

<table>
<thead>
<tr>
<th>WALL</th>
<th>F&lt;sub&gt;max&lt;/sub&gt;</th>
<th>d&lt;sub&gt;max&lt;/sub&gt;</th>
<th>d&lt;sub&gt;u&lt;/sub&gt;</th>
<th>F&lt;sub&gt;max avg&lt;/sub&gt;</th>
<th>C.O.V.</th>
<th>d&lt;sub&gt;u avg&lt;/sub&gt;</th>
<th>C.O.V.</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>[kN]</td>
<td>[mm]</td>
<td>[mm]</td>
<td>[kN]</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; envelope curve</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; envelope curve</td>
<td>P</td>
<td>112.84</td>
<td>112.73</td>
<td>102.35</td>
<td>107.72</td>
<td>54.01</td>
<td>71.66</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; envelope curve</td>
<td>N</td>
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<td>-106.61</td>
<td>-101.83</td>
<td>-100.56</td>
<td>-62.75</td>
<td>-69.34</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; envelope curve</td>
<td>P</td>
<td>109.5</td>
<td>108.56</td>
<td>99.71</td>
<td>104.32</td>
<td>54.02</td>
<td>62.53</td>
</tr>
<tr>
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<td>N</td>
<td>-92.27</td>
<td>-102.36</td>
<td>-99.16</td>
<td>-98.06</td>
<td>-53.27</td>
<td>-59.52</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; envelope curve</td>
<td>P</td>
<td>107.92</td>
<td>106.82</td>
<td>98.28</td>
<td>102.72</td>
<td>54.06</td>
<td>62.52</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; envelope curve</td>
<td>N</td>
<td>-91.14</td>
<td>-99.96</td>
<td>-96.72</td>
<td>-95.95</td>
<td>-53.25</td>
<td>-59.67</td>
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<tr>
<td>avg</td>
<td>P</td>
<td>110.09</td>
<td>109.37</td>
<td>100.11</td>
<td>104.92</td>
<td>54.03</td>
<td>65.57</td>
</tr>
<tr>
<td>avg</td>
<td>N</td>
<td>-93.43</td>
<td>-102.98</td>
<td>-99.24</td>
<td>-98.19</td>
<td>-56.42</td>
<td>-62.84</td>
</tr>
<tr>
<td>C.O.V. [%]</td>
<td>P</td>
<td>2.28</td>
<td>2.78</td>
<td>2.06</td>
<td>2.43</td>
<td>0.05</td>
<td>8.04</td>
</tr>
<tr>
<td>C.O.V. [%]</td>
<td>N</td>
<td>3.25</td>
<td>3.27</td>
<td>2.58</td>
<td>2.35</td>
<td>9.71</td>
<td>8.95</td>
</tr>
</tbody>
</table>
Experimental program: cyclic tests on unreinforced walls

### Table 4.5 - Significant values of lath and plaster walls

<table>
<thead>
<tr>
<th>WALL</th>
<th>F&lt;sub&gt;max&lt;/sub&gt; [kN]</th>
<th>d&lt;sub&gt;max&lt;/sub&gt; [mm]</th>
<th>d&lt;sub&gt;u&lt;/sub&gt; [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UFW25</td>
<td>UFW50</td>
<td>UFW25</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; envelope curve</td>
<td>P 76.37</td>
<td>104.92</td>
<td>45.90</td>
</tr>
<tr>
<td></td>
<td>N -65.95</td>
<td>-95.42</td>
<td>-52.07</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; envelope curve</td>
<td>P 73.78</td>
<td>102.03</td>
<td>45.85</td>
</tr>
<tr>
<td></td>
<td>N -64.10</td>
<td>-93.74</td>
<td>-52.08</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; envelope curve</td>
<td>P 72.73</td>
<td>100.8</td>
<td>45.83</td>
</tr>
<tr>
<td></td>
<td>N -62.81</td>
<td>-92.3</td>
<td>-41.92</td>
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<tr>
<td>avg</td>
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<td>45.86</td>
</tr>
<tr>
<td></td>
<td>N -64.29</td>
<td>-93.82</td>
<td>-48.69</td>
</tr>
<tr>
<td>C.O.V. [%]</td>
<td>P 2.52</td>
<td>2.06</td>
<td>0.08</td>
</tr>
<tr>
<td></td>
<td>N 2.46</td>
<td>1.66</td>
<td>12.04</td>
</tr>
</tbody>
</table>

### Table 4.6 - Significant values of timber frame walls

<table>
<thead>
<tr>
<th>WALL</th>
<th>F&lt;sub&gt;max&lt;/sub&gt; [kN]</th>
<th>d&lt;sub&gt;max&lt;/sub&gt; [mm]</th>
<th>d&lt;sub&gt;u&lt;/sub&gt; [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UTW25</td>
<td>UTW50</td>
<td>UTW25</td>
</tr>
<tr>
<td>1&lt;sup&gt;st&lt;/sup&gt; envelope curve</td>
<td>P 55.73</td>
<td>102.06</td>
<td>27.44</td>
</tr>
<tr>
<td></td>
<td>N -42.11</td>
<td>-95.22</td>
<td>-23.65</td>
</tr>
<tr>
<td>2&lt;sup&gt;nd&lt;/sup&gt; envelope curve</td>
<td>P 52.87</td>
<td>97.36</td>
<td>79.18</td>
</tr>
<tr>
<td></td>
<td>N -40.91</td>
<td>-91.62</td>
<td>-54.15</td>
</tr>
<tr>
<td>3&lt;sup&gt;rd&lt;/sup&gt; envelope curve</td>
<td>P 52.02</td>
<td>94.65</td>
<td>79.17</td>
</tr>
<tr>
<td></td>
<td>N -39.57</td>
<td>-89.92</td>
<td>-54.24</td>
</tr>
<tr>
<td>avg</td>
<td>P 53.54</td>
<td>98.02</td>
<td>61.93</td>
</tr>
<tr>
<td></td>
<td>N -40.86</td>
<td>-92.25</td>
<td>-44.01</td>
</tr>
<tr>
<td>C.O.V. [%]</td>
<td>P 3.63</td>
<td>3.82</td>
<td>48.23</td>
</tr>
<tr>
<td></td>
<td>N 3.11</td>
<td>2.93</td>
<td>40.07</td>
</tr>
</tbody>
</table>

In order to obtain the bi-linear diagram, the yield displacement has to be defined. It can be obtained according to different approaches: (1) Eurocode 8 (2003) considers the yield load as the maximum load and assumes the yield displacement from the equivalence of the energy enclosed by experimental envelopes and the bi-linear diagram; (2) the Italian code (Circolare Ministeriale, 2009) assumes the failure load, necessary to identify the ultimate displacement, as greater or equal to 85% of the maximum load, calculates the yield displacement and load from the imposition of the equivalence of the energies and considers the linear stiffness as the secant stiffness calculated at 60% of the maximum load of the envelope curve; (3) Tomaževic (1999) considers the failure load as 80% of the maximum load and calculates the yield displacement from the equivalence of the areas. In this work, the latter method was used (Figure 4.34a), since (1) assuming the maximum load as yield
load increases considerably the corresponding yield displacement, which should result in the remarkable decrease on the ductility and (2) the Italian Code method has an imposition for the initial stiffness which is not suitable for timber walls, due to their highly non-linear behaviour (it has to be kept in mind that timber codes consider as secant stiffness at 40% of the maximum load). It should be pointed out that for the majority of the walls, the ultimate displacement corresponds to the maximum one obtained experimentally, since only two walls lost more than 20% of the maximum load in the degradation process (namely wall UFW25 and wall UTW50). Therefore, the ultimate displacement corresponds to the displacement reached in the last cycle imposed to the walls.

Figure 4.34b,c presents the bilinear curves used to obtain the values of ductility for all walls. Only positive values are shown, since it was decided to take the positive displacements of the envelope for the calculation of seismic parameters.

This choice was made due to the asymmetrical behaviour of the walls, since the load is applied only from one side of the wall. Moreover, clearances in the wall and the same rotation of the wall can alter the applied displacement when the wall is being pulled.
According to European Standard ISO DIS 21581 (2009), the lateral stiffness of the walls may be calculated according to eq. 4.1:

\[ K_{1,in+} = \frac{0.3F_{\text{max}}}{\delta_{40\%F_{\text{max}}} - \delta_{10\%F_{\text{max}}}} \]  

(4.1)

where \( \delta_{40\%F_{\text{max}}} \) and \( \delta_{10\%F_{\text{max}}} \) are the displacement values obtained in the envelope curve at 40 and 10% of the maximum load \( (F_{\text{max}}) \) respectively.

The consideration of the initial displacement corresponding to 10% of the maximum force should be associated to the need of overcoming some type of initial nonlinearity. It should be noticed that in this case of traditional timber-frame walls, considerable nonlinear behaviour at very small values of lateral drift were recorded, which should be associated to the accommodations that the wall connections encounter at the beginning of the tests. In this work, to overcome the initial nonlinear behaviour and to obtain a more adjusted linear branch to the monotonic envelopes, it was decided to calculate the secant stiffness taking into account the origin and the point corresponding to 40% of the maximum load, \( (K_{1,s+}) \). All values of stiffness were calculated for the initial cycle.

The values of the secant stiffness, \( K_{1,ins+} \), and \( K_{1,s+} \) are shown in Table 4.7. As expected, the values found for the secant stiffness \( K_{1,s+} \) considering a secant stiffness from the origin up to 40% of the maximum load are greater than those of the standard initial stiffness because it softens the effect of the initial nonlinearity due to the initial adjustment of the traditional walls connections. The values are nonetheless of the same order.

The three UIW25 walls had an average initial stiffness of 3.03kN/mm (c.o.v. 10.17%). The UIW50 walls experienced a higher variation, with an initial stiffness of 3.75kN/mm (c.o.v. 18.42%). It is observed that, stiffness varied among the three types of walls tested. This variation can be attributed to a varying stiffness of the timber elements of the walls, to imperfections in the assembling of the timber frame, mainly possible clearances in the connections, and to the variation in the stiffness of the masonry infill, as a great variation in terms of results was reported when characterizing the materials of the walls.

Notice that the values of stiffness for wall UIW8 are not reported, since the behaviour of the wall was different than that of the other, with very high values of initial stiffness (11.47kN/mm).

The vertical pre-compression influences the lateral stiffness. It is observed that there is an increase on the stiffness as the vertical pre-compression increases. The increase is particularly relevant in case of lath and plaster walls. This is in agreement with past results obtained also in traditional timber walls submitted to lateral cyclic loading (Vasconcelos and
The timber walls present the lower values of initial stiffness, which should be associated to the absence of infill. The values of initial stiffness are however higher than what observed in modern timber walls with sheathing panels. Dinehart and Shenton (1998) found values of elastic stiffness which varied between 1.79 kN/mm and 2.35 kN/mm; Varoglu et al. (2006) found a great range of values between 0.70 kN/mm and 6.03 kN/mm for midply wood shear walls, depending on the type of sheathing material and the load protocol applied.

<table>
<thead>
<tr>
<th>WALL</th>
<th>$K_{1,in+}$ [kN/mm]</th>
<th>$K_{1,s+}$ [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>UIW2</td>
<td>2.98</td>
<td>3.86</td>
</tr>
<tr>
<td>UIW3</td>
<td>3.36</td>
<td>4.05</td>
</tr>
<tr>
<td>UIW7</td>
<td>2.75</td>
<td>3.29</td>
</tr>
<tr>
<td>Average UIW25</td>
<td>3.03</td>
<td>3.73</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>10.17%</td>
<td>10.59%</td>
</tr>
<tr>
<td>UIW4</td>
<td>4.42</td>
<td>5.05</td>
</tr>
<tr>
<td>UIW5</td>
<td>3.04</td>
<td>3.23</td>
</tr>
<tr>
<td>UIW6</td>
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<td>4.80</td>
</tr>
<tr>
<td>Average UIW50</td>
<td>3.75</td>
<td>4.36</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>18.42%</td>
<td>22.63%</td>
</tr>
<tr>
<td>UFW25</td>
<td>3.24</td>
<td>3.93</td>
</tr>
<tr>
<td>UFW50</td>
<td>7.99</td>
<td>10.21</td>
</tr>
<tr>
<td>UTW25</td>
<td>2.14</td>
<td>2.60</td>
</tr>
<tr>
<td>UTW50</td>
<td>3.16</td>
<td>3.60</td>
</tr>
</tbody>
</table>

Comparing with other experimental works performed on traditional half-timbered and timber frame walls, Gonçalves et al. (2012) found significantly lower values of stiffness for traditional Portuguese frontal walls, namely 0.7 kN/mm for timber-frame walls with a vertical load of 75 kN and 2.2 kN/mm for masonry infill walls. The differences could be attributed to the different workmanship (which could lead to higher clearances in timber-frame walls), different materials and different infill properties.

Comparing the results with traditional half-timbered walls from Haiti (Vieux-Champagne et al., 2012), the initial stiffness of the walls is significantly lower for Haitian walls, but this is easily understandable since the connections are weaker and very few elements are continuous, the connections are guaranteed only by means of nails. It would appear (no definite results have been published at the time of writing) that the initial stiffness of timber-frame walls filled with stones is approximately 0.8 kN/mm.
In order to evaluate the degradation of stiffness experienced by the walls during the cyclic tests, cyclic stiffness was calculated for each cycle considering the average of the slopes of the line connecting the origin and the two points of loading corresponding to the maximum (positive and negative) displacements. Due to the accommodations that occur in the wall for low values of drifts already mentioned, values of cyclic stiffness calculated for drift values lower than 0.15% are not considered reliable and thus they are not represented here.

The variation of the cyclic stiffness for increasing lateral drifts is presented in Figure 4.35a. The lateral drift is calculated as the ratio between the lateral top displacement and the height at which the lateral load is applied. It is seen that the cyclic stiffness presents steep stiffness degradation for early lateral drifts. The major stiffness degradation occurs for lateral drifts lower than 1%. It is seen also that the cyclic stiffness decreases exponentially and approaches very low residual stiffness for increasing lateral drifts. In case of walls submitted to the highest levels of pre-compression load, it is possible to differentiate the values of stiffness degradation among the distinct types of walls. This is not possible in case of the lower pre-compression load for which the stiffness degradation is similar for all walls. Besides, it is noticed that the residual cyclic stiffness is comparable for both pre-compression load levels. The steep degradation of the cyclic stiffness is associated to the strong non-linear behaviour evidenced by the walls from early stages of lateral drifts.

Timber walls exhibit the lowest cyclic stiffness, but for higher values of drift the results of the three wall types are comparable. The homogenization of the values occurs after a drift of 1.5% for the higher vertical load level and after a drift of 3.5% for the lower vertical load level. However, the degradation rate is different from UTW25 walls to walls UIW25 and UFW25. In walls, the cyclic stiffness reached an almost constant value after a value of drift of 1.85%, whereas walls UIW25 and UFW25, continued to lose stiffness for increasing values of drift. UTW50 walls instead, degraded its stiffness in a way more similar to what happened for the infill walls, even though both timber frame walls exhibited an early stiffness degradation.
higher than what observed in infill walls. This was probably due to the fact that the absence of infill caused the frame to loosen more for low values of drift.

It is clearly seen that the amount of vertical load directly influences the cyclic response of the walls. The higher vertical load leads to higher values of cyclic stiffness (Figure 4.35b) up to lateral drift lesser than 3.5%, after which stiffness starts to be comparable for the two load levels. For lateral drifts up to 2.5%, the cyclic stiffness was almost the double for the highly loaded walls.

Comparing the values of cyclic stiffness for the different wall typologies, for the lower vertical load level, lath and plaster walls had an increase of 27% and 12% for low and high values of drifts respectively when compared to masonry infill walls. Contrarily, timber frame walls saw a decrease in their values of cyclic stiffness by 32% and 22% for low and high values of drift respectively when compared to half-timbered walls.

The same trend was observed for the higher vertical load level: lath and plaster walls increased their values by 64% and 6% respectively while timber frame walls decreased by 31% and 28%.

Comparing the results with similar ones performed on traditional scaled Portuguese timber-frame walls (Vasconcelos et al., 2013) the values of cyclic stiffness for low values of drift are higher (between 12 and 14kN/mm), but this could be attributed to the different geometry which presents a higher number of bracing members. The degradation of stiffness is however similar, with very low values of stiffness for higher values of drift.

**4.6.2 Evaluation of ductility**

Ductility is an important factor for the evaluation of the seismic behaviour of structures in seismic regions, as it is directly related to the ability of the structure to deform nonlinearly without significant loss of strength. Displacement ductility is defined here as the ratio between the ultimate displacement \(d_u\) and the yield displacement \(d_y\) defined in the equivalent bilinear diagram (Section 4.6.1).

Ductility \(\mu_1\) was calculated using the values of secant stiffness calculated above considering the slope of the curve between the origin and 40% of the maximum load. Once again, only the portion of the bilinear curve corresponding to positive values of displacement was considered. As far as ductility is concerned, some variation is found when the typology of the walls is considered. Timber frame walls with infill present higher values of ductility, see Table 4.8. An average ductility \(\mu_{1,\text{avg}}\) of 5.20, with a coefficient of variation (C.O.V.) of 12.81%, was found for UIW25 walls, while for UIW50 walls an average ductility of 3.62 (C.O.V. 27.82%) was obtained.
### Table 4.8 - Values of ductility for tested walls

<table>
<thead>
<tr>
<th>WALL</th>
<th>$\mu_1$</th>
<th>$\mu_{1,avg}$</th>
<th>C.O.V.</th>
</tr>
</thead>
<tbody>
<tr>
<td>UIW2</td>
<td>4.88</td>
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<td></td>
</tr>
<tr>
<td>UIW3</td>
<td>5.97</td>
<td>5.20</td>
<td>12.81%</td>
</tr>
<tr>
<td>UIW7</td>
<td>4.76</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UIW4</td>
<td>4.24</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UIW5</td>
<td>2.46</td>
<td>3.62</td>
<td>27.82%</td>
</tr>
<tr>
<td>UIW6</td>
<td>4.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UFW25</td>
<td>4.65</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UFW50</td>
<td>7.58</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UTW25</td>
<td>4.57</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UTW50</td>
<td>3.53</td>
<td></td>
<td></td>
</tr>
<tr>
<td>UTW50*</td>
<td>2.90</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Besides, it is seen that timber-frame walls presented lower values of ductility for both load cases when compared to half-timbered walls. This means that the filling of the timber frame walls leads to improvement of the ductility as it results from the change on the resisting mechanism from shear to flexure or a mixed mode of shear and flexure mode, as in case of the half-timbered walls submitted to the highest level of pre-compression. It should be noticed that the lower values of ductility found for timber frame walls are associated to considerable higher levels of damage. In all cases, the walls are repairable after the imposition of the maximum lateral drift, but in case of timber-frame walls, some timber elements need to be replaced. This appears to indicate that the presence of infill improves the seismic behaviour of the walls as improve ductility with a lower level of damage.

Apart from lath and plaster walls, it should be stressed that the ductility of the walls tends to decrease with the level of pre-compression, which results from the higher importance of the shear resisting mechanism on the lateral response of the walls. This behaviour is also characteristic of other types of walls, including stone and brick masonry (Vasconcelos and Lourenço 2009). It has to be pointed out, though, that for some walls these values are not the final ones. For example, wall UIW5, which presents a low value of ductility, only met a strength loss of 11%, meaning that the test could have continued and lower values of ductility could have been obtained. The same can be said for wall UIW6 (strength loss of 1%), UIW2 (strength loss of 6%), UIW7 (strength loss 14%) and UTW25 (7%).

When comparing the values of ductility found for these walls with other ones found for modern timber walls on which similar tests were performed, slightly higher values can be found for shear walls with large sheathing panels, with values varying from 4.9 to 6.2 (Lam et al. 1997). Toothman (2003) obtained even higher values of ductility for timber shear walls with different sheathing materials, with values ranging from 7.66 to 10.41. Similar tests
performed on reduced scale half-timbered walls (Vasconcelos et al. 2012) gave values of ductility ranging from 5.48 to 17.95, depending on the amount of vertical pre-compression. However, the geometry and connections type of these walls was significantly different from what tested here.

It should be stressed than in general the walls tested did not reached high values of degradation, meaning that the walls could undergo some additional displacements, which could result in higher values of ductility. It was not possible to reach these values due to the limitations on the imposed lateral displacement.

In any case, when values of ductility and lateral drift obtained in timber frame walls are compared to other shear walls typologies such us brick or block masonry, it is seen that timber-frame walls present considerably higher values with repairable damage, presenting considerable advantages over the typologies mentioned.

4.6.3 Assessment of the ability to dissipate energy and equivalent viscous damping

Besides ductility and lateral drifts, one major parameter used for the assessment of the seismic performance of the seismic behaviour is the ability of a structural element to dissipate energy during cyclic testing. Here, the dissipation of energy per each cycle and the cumulative energy are considered. The energy dissipated by the walls at each cycle, $E_D$, is 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 (Figure 4.36a). The energy can be dissipated through friction in the connections, yielding of nails and residual deformation in the wall panel, as observed during the tests.

![Load-Displacement Diagrams](image)

Figure 4.36 - Dissipated energy and equivalent viscous damping: (a) area enclosed in loop; (b) input energy

Figure 4.37a,b reports the graphs of the variation of the dissipated energy per each cycle for all walls tested with the two vertical load conditions with the increasing lateral drift. It is
observed that the vertical load has an important role on the amount of energy dissipated per each cycle. The energy dissipation for each cycle increases as the lateral load increases, being the increment ratio clearly higher is case of the walls submitted to the higher level of pre-compression. This can be related to the higher amount of damage found for this level of vertical pre-compression. Notice that half-timbered walls submitted to the lower pre-compression level (UIW25) behave mainly in rocking from the detachment of the vertical posts from the bottom beams, which in nature is a low dissipative behaviour mode. The increase on the dissipated energy per cycle is naturally related to the increase of damage as the lateral drift increases. In effect, the energy dissipation is always associated to the propagation of damage in a structure. Once again it can be noticed how UIW25 walls (Figure 4.37a) have similar values, with the exception of UIW2, which was able to dissipate slightly higher amounts of energy, due to its higher load capacity.

The same can be said for walls UIW50 (Figure 4.37a) which all present similar values of dissipated energy, and only UIW4 presents lower values for high values of drift due to its early failure. All walls exhibited an almost linear trend in terms of dissipated energy.

The low values of dissipated energy for timber frame walls (Figure 4.37b), compared to filled walls, can be attributed to the strong pinching present in this walls, that clearly diminishes the dissipative capacity of the walls. In fact, when comparing the different shape of the hysteretic loops for each wall type (Figure 4.38a,b) the strong pinching in timber frame walls is evident, as well as the plateau in the unloading branch of filled walls, which reduces the dissipative capacity of the walls.
If the trend of dissipated energy per each cycle is approximately linear, the cumulative dissipated energy has an exponential trend (Figure 4.39a,b). For both load levels, lath and plaster walls had a higher dissipative capacity.

When comparing the ratio between dissipated and input energy, both for the single cycle and the cumulative values (Figure 4.40a,b), it is immediately evident that the ratio increases rapidly for low values of drift and it then starts decreasing, losing a significant percentage. The walls do not have an increasing dissipative capacity due to their flexural behaviour. Since the posts are uplifting, creating the plateaux in the unloading branch of the hysteretic curve, the walls lose some dissipative capacity, thus influencing the parameters here calculated. The vertical uplift of the posts starts at values of drift of approximately 0.7%, which corresponds to the drift value for which the ratio between dissipated and input energy starts decreasing. It can be noticed that a higher vertical load, reducing the post uplifting, acts as hold downs and this appears to help the seismic performance of the walls. These observations are in accordance to what was found by other authors as well (Toothman 2003; Johnston et al. 2006; Dean and Shenton 2005).
A confirmation that infill helps in improving the seismic characteristics of the walls comes from the comparison of the trends in terms of energy dissipation. For both load cases, filled walls were able to dissipate a higher amount of energy for the same drift value.

Comparing the total cumulative dissipated energy among the different wall typologies, once again lath and plaster walls exhibited the highest values, gaining 2% on half-timbered walls for the lower vertical load level and 10% for the higher. Timber frame walls instead dissipated less energy than half-timbered walls, namely 36% and 27% less for the lower and higher vertical load level respectively.

Comparing the results obtained with those from Aktas et al. (2013), the cumulative energy dissipated by the walls is much higher. The Ottoman walls reached values of 12000kNmm, values much lower than what encountered in this study, but in that case the connections are mainly notched or nailed, therefore they dissipate less energy.

The equivalent viscous damping ratio (EVDR) is correlated to energy dissipation. Damping is the process by which vibration steadily diminishes in amplitude (Chopra 1995). Damping diminishes the energy of the structure through various mechanisms, such as, for the present case, friction in the connections and opening and closing of cracks and gaps. Equivalent viscous damping is calculated according to equation 4.2 (Magenes and Calvi 1997):

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

where $E_d$ is the dissipated hysteretic energy discussed above, $E_e^+$ 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 (see Figure 4.36b).
It should be stressed that considerable variation of EVDR among the walls (Figure 4.41a). For low values of lateral drift there is a great variation on the EVDR in walls UIW25, being lower for lateral drift greater than 3%. A similar behaviour was observed even for walls UIW50, for lateral drifts greater than 2%.

EVDR is influenced by the presence of infill and the amount of vertical load (Figure 4.41b). For the lower vertical load level, it appears that the type of infill does not influence this parameter, but the absence of infill increases the values of EVDR from a constant value of 0.10 for infill walls to an almost constant value of 0.12 for timber walls. For the higher vertical load level, this trend is not so clear. Lath and plaster walls present the higher values for low values of drift and then EVDR values decrease and become comparable to those of timber frame walls. This means that, although timber walls present lower values of maximum load, stiffness and ductility, they have a comparable damping capacity for high values of drift due to the shear behaviour. In fact, only analysing the loops of infill walls vs. timber frame walls it is possible to observe how the loops in timber frame walls are more curved, so the damping for these walls is higher.

For infill walls, there is no increase of EVDR as the damage progresses, while a minimal increase can be observed for timber frame walls. This different behaviour is connected once again to the different global behaviour of the walls. As it was observed in Magenes and Calvi (1997), for walls with a flexural behaviour the values of EVDR are almost constant, while for a diagonal shear cracking response the values are usually increasing with an increase of drift and damage accumulated.

The values of EVDR found in this work are comparable with what observed in tests on modern timber frame walls (Toothman 2003), where values varied between 0.11 and 0.17 depending on the type of sheathing material.
Gonçalves et al. (2012), on similar traditional Portuguese frontal walls, obtained values of viscous damping for low values of drift of 0.17-0.20 for infill walls and 0.19-0.20 for timber-frame walls. 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, then decreasing values with an increase in viscous damping in some cases for ultimate values of drift.

4.7 Conclusions

Aiming at analysing the lateral behaviour of timber frame walls, characteristic of ancient construction in Portugal, and that are comparable to other timber frame walls found in several countries, an experimental campaign was designed based on static cyclic tests. Distinct parameters were considered, namely typology for the infill and vertical pre-compression load. Three distinct solutions were adopted: (1) brick masonry infill; (2) lather and plaster; and (3) no infill. Besides, two vertical pre-compression levels were considered for each wall type.

From the detailed analysis of the experimental results it is possible to conclude that:

- The presence of infill changes considerably the response of the walls in terms of predominant resisting mechanism, ranging from a predominant flexural or mixed shear/flexural behaviour of half-timbered walls to a shear predominant resisting mode for timber frame walls with no infill;

- The increase on the vertical pre-compression load results also on the change on the lateral behaviour of the half-timbered walls. Higher values of vertical pre-compression results in the mixed flexural/shear resisting mechanism, whereas in case of walls submitted to the lowest pre-compression, flexural/rocking mechanism predominates. In case of timber frame walls with no infill, shear resisting mechanism prevails for both levels of vertical pre-compression.

- The vertical pre-compression level influences also the load capacity of the walls, their stiffness, ductility and energy dissipation. A higher vertical load leads to higher values of all seismic parameters. Moreover, the higher pre-compression accentuates damages in infill walls, while in timber frame walls damage patterns did not depend on the amount of vertical load applied.
Timber frame walls present lower load capacity, stiffness and ductility when compared to half-timbered wall, but the damping capacity is slightly greater, due to their distinct response;

The traditional half-timbered walls present in general a very good deformation capacity, exhibiting higher lateral drifts with controlled damage. In case of timber frame walls with no infill the same lateral drifts are associated to a high level of damage, mainly concentrated at the central connections. The absence of infill promotes the free functioning of the walls, which results in much higher shear deformations and higher damage due to shear deformation of the connections.

The application of infill in traditional timber walls can, in a certain extent, be viewed as a strengthening strategy as it improves the lateral cyclic performance and controls the damage progression at the connections due to the confining effects.
Experimental program: cyclic tests on unreinforced walls
CHAPTER 5  EXPERIMENTAL PROGRAM: CYCLIC TESTS ON RETROFITTED WALLS

5.1 Introduction

It has already been pointed out how half-timbered walls are an important historical heritage structural typology used in various countries which has to be preserved. Half-timbered buildings constitute an important portion of many historical city centres in the world. Many of these buildings have known little or no care during their life or they have been modified without taking into account the seismic response of the structure after the alterations had been made, frequently causing greater damages during earthquakes than what observed in buildings which had preserved their original structure (see Chapter 2). 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. It has to be kept in mind that no great earthquakes have hit Lisbon after the 1755 one, which had a return period of 250 years, so it is plausible to be expecting a devastating earthquake in the near future.

Many examples are available on restoration works done in traditional half-timbered buildings, but few experimental studies have been performed in order to assess the efficiency of the strengthening techniques adopted.

Numerous Pombalino buildings in Lisbon have been retrofitted using FRP sheets in the connections of the frontal walls, creating a star-shaped strengthening (Cóias, 2007), or damping systems linked to frontal walls and to the outer masonry walls through injected anchors and providing additional bracing (Cóias, 2007).
As already mentioned, little experimental information is available in literature. Cruz et al. (2001) performed diagonal tests on reduced scale wallets strengthened with Glass Fibre Reinforced Polymer (GFRP) rods and Glass Fibre 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 strengthening of timber frame walls with glued FRP sheets in some of the connections of the walls in order to assess the influence of this technique on the lateral resistance and lateral strength.

Gonçalves et al. (2012) have performed in-plane cyclic tests on half-timbered walls retrofitted with steel plates, reinforced rendering and dampers applied on diagonal bracers. All strengthening techniques were able to increase the stiffness and the dissipative capacity of the walls, with some instability problems for the damping system and an early cracking of the reinforced plaster.

More relevant information is available on retrofitting techniques for traditional timber connections (Branco, 2008; Parisi and Piazza, 2002), which are of great importance for the strengthening of traditional walls, since strengthening of half-timbered walls is almost reduced to the strengthening of the connections. However, the implementation of the retrofitting solutions in the wall is not a well-covered topic. Branco (2008) studied the cyclic behaviour of traditional timber connections (bird’s mouth connections) and appropriate retrofitting solutions, which consisted of metal stirrups, internal bolts, binding strips and tension ties, being all considered as traditional solutions. The insertion of one bolt across the joint axis and metal stirrups positioned at the two sides of the joint, bolted to the timber elements, gave the best results (Branco 2008).

Higher amount of research is available on modern timber frame walls (Lam et al., 1997; Toothman, 2003), but for these walls strengthening is not usually considered. An improvement in their seismic capacity is usually achieved through the adoption of different sheathing or through an alternative disposition of the frame (Varoglu et al., 2006). Premrov et al. (2004) studied timber frame walls coated with carbon fibre-reinforced polymers (CFRP) strengthened fibre-plaster boards.

In the scope of the strengthening of structural elements an innovative technique that has been exploited with distinct types of materials, namely structural timber beams, is the near surface mounted (NSM) technique. Examples can be found in practice in the rehabilitation of traditional timber floors (Cólias, 2007), using both steel rods or bars and FRP elements, or in experimental results on timber and glulam beams tested in bending (Jorge, 2010; Xu et al., 2012; Juvandes and Barbosa, 2012). This type of strengthening is particularly efficient in
bending, since it increases the bending strength of the timber elements, allowing greater deformations without failure. The efficiency of this retrofitting technique has also been confirmed by its use in reinforced concrete elements (Dalfré, 2013; De Lorenzis and Teng, 2007) for flexural and shear strengthening.

With respect to retrofitting techniques in half-timbered walls, it should be stressed that its implementation is not a well-covered research topic among the research community. Tests performed on modern timber frame walls rarely cover retrofitting techniques. On the other hand, the majority of the information existing on the strengthening techniques covers the retrofitting of timber elements submitted to different loading conditions.

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 half-timbered walls, etc., see Figure 5.1a) or even alterations of the existing structure (for example, filling of existing openings, see Figure 5.1b). But these interventions have been done with very little background knowledge. Moreover, some interventions altered completely the structure of the building.

![Figure 5.1 - Restoration works in existing buildings: (a) replacement of decayed elements, continuity not guaranteed (© MONUMENTA); (b) addition of new elements to close openings (© MONUMENTA).](image)

Only in recent years, studies have been carried out on the experimental behaviour of traditional half-timbered walls and their retrofitting solutions (Vasconcelos et al., 2013; Gonçalves et al., 2012).

### 5.2 Strengthening approach

The walls strengthened were the same walls that had been tested in the unreinforced condition, so they experienced some damage in the first test to which they had been subjected. The strengthening approach here adopted consists of restoring the wall to its
original condition, with appropriate solutions, in order to subsequently strengthen the wall and test it again. Notice that retrofitting solutions have been applied only to walls with masonry infill or no infill, since it was decided that strengthening lath and plaster walls would be onerous as well as representing only a small portion of existing timber frame walls.

In Chapter 4 damages experienced by the unreinforced walls are listed. In order to bring the walls back to their original condition, it has been necessary to:

- **Repair the masonry blocks.** Masonry had little damage during the unreinforced tests, but still mortar cracked and some bricks fell off. To reinstate masonry to its almost initial condition, a natural hydraulic binder was used, PROMPT (VICAT, 2003), characterized by a good adhesion to all building materials. This natural cement is also characterized by a fast setting, so the repair of masonry was done two hours before the beginning of the test. In fact, a compressive strength of 4MPa was obtained after 30min for a volume ratio of 1:1 (PROMPT mortar : sand), so it was chosen to add more sand (ratio 1:2) in order to slow down the hardening and reach this values after 90 minutes. Figure 5.2a shows a repaired wall. The dark parts correspond to where mortar was added.

- **Close the half-lap connections that opened during the unreinforced test.** A typical damage encountered was the out-of-plane opening of the half-lap connections, as the nails were pulling out. The opened connections were closed using a bar clamp (Figure 5.2b), which had to be left in place for some hours so that when taken out the connection would not open again.

- **Substitute a damaged part of a post.** Half-timbered walls tested with the higher vertical load encountered damage in the central post at the central connection, which crushed due to the shear effect of the diagonals. It was decided to repair the post by gluing a prosthesis. The damaged part of the central post had to be removed (Figure 5.2c) until healthy timber was encountered and an appropriately cut wood piece was glued (Figure 5.2d) using a structural timber glue, Mapei Mapewood Paste 140 (MAPEI, 2002). Since the central beam was deformed and the connections were not aligned anymore, the deformation was recovered pushing the beam back into position by means of a bar clamp.

- **Replace an element.** In the case of timber frame walls without infill, damages were too extensive to remove and glue only a part of the member, so it was decided to substitute the whole central post. In both timber walls tested in the unreinforced condition, the central timber beam was replaced (Figure 5.2e), since the connections had failed and the beam was in fact not continuous anymore. In case of the patchwork wall (UFW50), the central post was replaced, since it was heavily damaged. Moreover, a prosthesis was added to the left post and bottom beam of the same wall, where crushing had
occurred in the previous test (see Chapter 4). A carpenter proceeded to remove the elements and position the new elements with the same dimensions nailing them into place.

- **Close a crack in a timber element.** When a timber member was cracked in correspondence of the half-lap connection and could cause an early local failure during the test, bolts were applied (Figure 5.2f) to assure that the timber element would not break. In other cases, when the cracks were small, structural timber glue was injected in the crack.

![Figure 5.2 - Typical repairs: (a) repair of masonry blocks; (b) closing of connections with a bar clamp; (c) removal of damaged wood with a wood chisel; (d) gluing of new timber piece; (e) insertion of new beam; (f) closing of crack in timber element.](image)

Thanks to these interventions, it was possible to bring the walls back to their original condition, or as close as possible. This was done considering that the strengthening is
applied to an existing structural element, which will be repaired if necessary, but not always substituted. Either way, since the walls were repaired, it was believed that the previous test would not overmuch influence the response of the retrofitted walls.

In order to assess this statement, two timber walls were left untested and were strengthened directly in order to see if the response of the strengthened wall would change. Results are presented in the next paragraphs.

5.3 Retrofitting techniques adopted

Observing the damages on the walls after the unreinforced tests, three retrofitting techniques were chose, two more traditional and one innovative:

- **Bolts.** Since the half-lap connections were opening out-of-plane, especially at the bottom connections, it was chosen to apply this simple intervention in order to have the connections work properly until failure and reduce the uplifting of the post at the bottom connections.
- **Steel plates.** The choice of this kind of strengthening was made in order to reduce the uplifting of the posts at the bottom connections which caused the flexural behaviour of the walls in the unreinforced condition as well as strengthening the connection between post and beam at all other connections.
- **Steel flat bars (NSM).** The bars are embedded into the connections with the near surface mounted technique (NSM). This innovative technique was chosen much for the same reasons as steel plates, with the addition of being a potentially invisible solution.

Since the investigation of this thesis concerns traditional walls built with simple materials, it was opted to use simple strengthening techniques with compatible materials, such as steel, without adopting modern and more technologically advanced materials, such as FRPs. All techniques are considered to be fast to be applied and the first two are removable (reversible), which represents an advantage, mainly concerning cultural heritage structures. The last technique, though not reversible, is potentially invisible, which could have beneficial effects if plaster is not applied on the walls.

5.3.1 Bolts

The first retrofitting technique chosen was to insert bolts at the centre of each half-lap connection of the main frame, for a total of nine connections (Figure 5.3a), aiming at tying together the two elements of the half-lap connection, namely vertical post and horizontal beam. The bolts pass through the thickness of the wall. The selection of these connections was related to trend for the out-of-plane detachment exhibited in the first testing campaign, especially at the bottom connections (Chapter 4). In fact, it was observed a tendency of the
posts to detach from the bottom beam for high values of displacement, opening out-of-plane, effectively losing the contact between the two elements, and therefore the efficiency of the connection, which would not work properly until failure. Notice that with the addition of a bolt at the base, it was intended to prevent uplift displacements and therefore limit the rocking mechanism observed, mainly for low levels of load applied to the posts.

The bolts used had a diameter of 10mm and a total length of 160mm and were class 8.8 steel fasteners. They were inserted in pre-drilled holes, according to recommendations of Eurocode 5 (2004) and were adjusted with screw nuts sufficiently tightened, also according to standard. Washers were used to better distribute the stresses (Figure 5.3b). For more calculations concerning bolts strengthening, see Appendix B; notice that the calculations are approximate and qualitative.

![Bolt Diagram](image)

Figure 5.3 - Strengthening with bolts: (a) a strengthened wall; (b) bolt, screw nut and washers used.

For this type of intervention, low tech equipment and workmanship are required; moreover the intervention is removable as well as economical.

5.3.2 Steel plates

The second type of strengthening consists of applying steel plates in all connections on both sides of the wall. Two types of steel plates were used; (1) custom plates were designed with a star-shape (Figure 5.4a). The steel plates are secured with screws and linked with bolts which go from one side to the other of the wall. The steel plates can link the diagonal bracing elements to the main elements of the connection (vertical post and horizontal beam). This choice was made for both infill and timber frame walls and it led to negative consequences in terms of ductility that will be analysed in the following sections. The plates were made in zinc-galvanized steel and had a thickness of 3mm. The plates already had holes of a diameter of 10.5mm for bolts of 10mm diameter; (2) due to the high price of the custom plates, it was decided to use rectangular commercial plates for timber frame walls without infill, as shown in Figure 5.4b. For timber frame walls, it was chosen to test the walls
adopting two solutions for the steel plates, i.e. linking the diagonals to the main frame as
done for half-timbered walls and linking only the main members of the connections.
Perforated plates (Rothoblaas plates PF703085 (140×400mm) and PF703035 (80×300mm)
(Rothoblaas, 2012)) made of steel S 250 GD and having a thickness of 2mm were chosen.
The plates had to be altered, enlarging certain holes in order to accommodate the bolts.
Moreover, as one plate was not sufficient to cover one connection, plates had to partially
overlap, taking care to create the superposition along the most stressed element, in order to
provide additional strength. The bolts used are the same ones presented in the previous type
of strengthening. The screws used (type PF603550 screws from Rothoblaas (2012)), have a
diameter of 5mm and a length of 50mm. They present a round head with a cylindrical
underhead and are especially designed to be used with steel plates.

![Steel plates strengthening: (a) custom steel plates applied to half-timbered wall; (b) commercial steel plates applied to timber frame wall](image)

In order to apply this strengthening, both sides of the wall should be accessible. The plates
are secured in the right position on both sides of the wall for each connection and holes are
drilled through the plate in the wall in order to accommodate the bolts which will secure the
plates and link the elements (Figure 5.5a,b). Care should be taken when drilling the holes so
that the existing holes in the plates correspond on the two sides. The bolts are secured with
screw nuts sufficiently tightened. Screws are inserted on both sides of the wall in order to
to better distribute the stresses in the plates.

Both types of steel plates require low technical equipment and non-specialized workmanship.

Appendix B, section B.1.2 shows in detail the geometry and disposition of the plates, with the
indication of the number of bolts and screws inserted for each connection, together with the
simplified qualitative calculations adopted for such strengthening.
Figure 5.5 - Pre-drilling of holes to accommodate bolts: (a) infill walls with custom plates; (b) timber frame walls with altered commercial plates.

5.3.3 Steel flat bars (NSM)

The last intervention consists of embedding CK45 steel flat bars in the connections inserted with the near surface mounted technique (NSM).

In order to perform the retrofitting, slots were opened in the elements with a plunge router (Figure 5.6a), having a width of 12mm and a depth of 23mm to accommodate the flat bars with a section of 8×20mm\(^2\). The slots have to be bigger than the flat bar, allowing at least 1.5mm on each side so that the glue can adhere well. The cuts were then cleaned with compressed air and filled with structural timber glue (MAPEI Mapewood Paste 140) (Figure 5.6b), being then the steel flat bars inserted (Figure 5.6c). Additional glue was added if necessary to completely fill the slots while excessive glue was cleaned in order to obtain a clean final appearance (Figure 5.6d) for the connection. To achieve the maximum strength, the glue had to be left to dry for 7 days. This intervention is potentially invisible (a thin wooden strip could be used to close the slot, see Figure 5.6e), but not removable.

The NSM strengthening intervention was chosen in order to obtain the same gains of the steel plates interventions, but simultaneously avoiding some problems observed during the experimental tests (namely, the development of out-of-plane mechanisms), which can prevent the gain in deformation after post-peak. Considering what was learned from the tests with the steel plates strengthening, it was decided to apply this kind of strengthening only at the main half-lap connections, i.e. to the connections between vertical posts and horizontal beams, without linking the diagonals.
In previous studies available in literature (Cruz et al., 2001), steel rods had been used. It was chosen to adopt steel flat bars instead of steel rods based on preliminary tests performed. The tests were performed on a wall already tested with bolts retrofitting, which had severe damage, and it was decided to apply steel rods at the connections at mid height, while applying steel plates at the bottom connections. Since the damage was concentrated at mid height from the previous test, it was chosen to apply a sample retrofitting only at that height.

It was chosen to apply steel rods with a diameter of 8mm to all central connections on both sides of the wall. Unfortunately, due to the fact that the connections were already damaged, the rods applied were not able to guarantee a sufficient tensile strength to resist the stresses.
to which the connections were subjected and an early failure occurred (Figure 5.7a), with opening of the already damaged connections (see Figure 5.7b,c,d).

Even though the wall failed early and did not gain in terms of ultimate load capacity or stiffness, its behaviour was significantly better since the flexural behaviour of the wall was significantly diminished. The uplifting of the posts was lower and the wall presented a prevalent shear behaviour (see Figure 5.7a). Based on these results, it was chosen to adopt this strengthening technique, but applying reinforcement elements with a higher strength, namely CK45 steel flat bars.

The first disposition of bars was chosen considering that the main damages were observed at the mid-height connections and at the bottom connections. To guarantee a sufficient anchorage length, the bars embedded in the bottom connections were bent at 90 degrees, on the side of the wall where the posts are discontinuous (Figure 5.9a). To achieve this, they had to be cut at mid-depth, bent and the cut was filled with welding. For all the connections at the bottom and mid-height of the wall, two parallel bars were embedded in the posts and in the beams on the side where they were discontinuous, to avoid the opening of the connections, and one bar was embedded on the opposite side, where the posts and beams
were continuous, to improve the resistance of the material (Figure 5.9a). After some problems were encountered during the test (which will be discussed in the subsequent sections), it was decided to repeat the test strengthening the bottom connections with commercial steel plates (Figure 5.9b), similarly to what done for the previous retrofitting technique.

Figure 5.8 - Disposition of steel flat bars in walls tested: (a) half-timbered walls, lower vertical load level; (b) half-timbered walls, lower vertical load level, repetition

After analysing the results, it was chosen to decrease the number of steel flat bars for the half-timbered wall tested with the higher vertical load level (Figure 5.9a). Only one bar was inserted in each connection on each side of the wall in the direction where the element was discontinuous.

It is to be pointed out that, in order to save material, the connection at the top beam in the middle position was not strengthened, because it was thought that the confinement given by the test setup made it superfluous.
Figure 5.9 - Disposition of steel flat bars in walls tested: (a) half-timbered walls, higher vertical load level; (b) timber frame walls.

For timber frame walls without infill (Figure 5.9b), since the confinement given by the infill is not present, it was decided to insert in each connection cross-shaped bars welded together with a notched connection in the middle. The bottom connections were strengthened with commercial steel plates, similarly to what done for the second solution for half-timbered walls. For the detailed geometry and disposition of the steel flat bars, see Appendix B, section B.1.3.

5.3.4 Parameters to be taken into consideration

The designing of the retrofitting techniques described until here required the consideration of the following parameters:

- **cross-sectional dimensions of timber elements involved.** Limitations on the minimum distances from the borders should be followed for bolts and screws. For the specimens tested, a minimum distance of 7cm was adopted for bolts and a minimum distance of 2cm for screws;
- **presence of knots or of pre-existing drying fissures.** Bolts should not be placed near knots of fissures, since they create a preferential failure path. If pre-existing drying fissures are significant, adequate measures should be taken, for example repairing them
with the use of timber slips or replacing the element. For this reason, visual inspection is suggested before strengthening the walls;

- **tensile strength and ductility of steel plates.** It should be ensured that the plates have an adequate capacity to deform and that failure by tearing of the plate in tension does not occur. The tensile strength of the plate has to be greater than that of the component material;

- **tensile strength of component material after reduction of cross-section.** When performing cuts in the timber elements to apply the steel flat bars, attention should be paid in order not to excessively reduce the cross section of the element, thus greatly reducing its resistance.

- **tensile strength of steel flat bars.** Attention should be paid to the type of steel flat bars used in order to guarantee a sufficient tensile strength to the connection;

- **bond strength between steel flat bars and structural glue and between structural glue and component material.** The bond between the materials should be investigated in order to avoid early failure due to debonding (Jorge, 2011);

- **anchorage length.** Anchorage length has to be investigated and a sufficient anchorage length has to be provided for the flat bars in all members. Anchorage length depends on the dimensions of the flat bar. For the bars adopted, an anchorage length of 15cm was chosen, considering approximately 20 times the width of the bar (Jorge, 2011).

It should be stressed that both retrofitting techniques were applied after the specimens were previously submitted to lateral cyclic tests, meaning that the retrofitting was made for certain levels of damage induced in the walls. This situation simulates the retrofitting that can be made after the occurrence of an earthquake.

### 5.4 Test setup, procedure and instrumentation

The test setup used for the tests on retrofitted walls was the same one adopted for the unreinforced tests (see Chapter 4). Even the procedure was not altered, in order to be better able to compare the results between unreinforced and retrofitted tests, even though possibly retrofitted walls could have had a different ultimate displacement.

As for the previous tests, the walls were instrumented with LVDTs in strategic positions. Figure 5.10 shows the position of the transducers used. Notice that not all LVDTs were used for each test, as the equipment allowed a maximum of 16 transducers for each test and their position was chosen according to the strengthening being tested. Namely, LVDTs DTL, DBR, DTR and DBL, which measured the opening of the half-lap connection linking two diagonals, were used only for walls retrofitted with steel plates, since they had a not negligible displacement.
In order to understand the efficiency of the strengthening materials and their actual participation during the tests, strain gauges were positioned in strategic places, particularly on the flat bars and on the steel plates. Strain gauges were applied in the direction where the timber element was discontinuous (Figure 5.11). As an example, strain gauge MV was applied on the bar in the vertical direction on the side where the post was discontinuous, while strain gauge MHR was applied to the same connection on the opposite side where the beam was discontinuous and on the right side of the half-lap connection.

Moreover, strain gauges were applied to the timber post (TH and TL) in order to record the strains in this element.

The number and position of the strain gauges tended to vary, so their position is shown on each of their graph presented in this chapter.

As for the unreinforced walls, two vertical load levels were considered, 25kN/post and 50kN/post, similarly to what done in the previous tests.

In total, twelve strengthened walls were tested, distributed in two distinct groups, see Table 5.1, according to the type of infill: (1) walls named as RIW with brick masonry infill, i.e. half-timbered walls; (2) walls named as RTW, in which no infill was considered, i.e. timber frame walls. The number 25 or 50 used in each designation is associated to the vertical load
applied in each post of the walls, 25kN and 50kN respectively. The type of strengthening is designated according to the last letter, see Table 5.1, and is divided in four groups: (1) letter B designates strengthening with bolts; (2) letter P designates strengthening with steel plates; (3) letters P_M designate strengthening with commercial steel plates connecting only the main elements of the connection; (4) letter S designates strengthening with NSM steel flat bars.

Table 5.1 – Typology and numenclature of tested walls

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Vertical load</th>
<th>Type of infill</th>
<th>Type of strengthening</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25kN/post</td>
<td>Brick masonry</td>
<td>Custom steel plates</td>
</tr>
<tr>
<td></td>
<td>50kN/post</td>
<td>No infill</td>
<td>Commercial steel plates</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Bolts</td>
<td>NSM</td>
</tr>
<tr>
<td>RIW25_B</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RIW50_B</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RIW25_P</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RTW25_P</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RTW25_P_M</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RIW50_P</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RTW50_P</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RTW50_P_M</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RIW25_S</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RTW25_S</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RIW50_S</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
<tr>
<td>RTW50_S</td>
<td>✓</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

For all typologies, only one specimen was tested. This decision was based on the scarce availability of specimens as well as on the results obtained in the unreinforced tests. In fact, it was observed that very similar results were obtained in the distinct unreinforced specimens, allowing to consider that is reasonable to consider only a single test as representative.

5.5 Experimental cyclic behaviour of retrofitted walls

As already done in Chapter 4, the presentation and results of the cyclic tests result will be divided into three parts, namely: (1) discussion of the typical force-displacements hysteresis diagrams; (2) discussion of the main deformation features and typical failure modes; (3) assessment of seismic performance indicators.

5.5.1 Typical hysteretic diagrams

In this section the hysteretic diagrams of the retrofitted walls tested are presented, together with the vertical uplift of the bottom connections, in order to better understand the behaviour of the walls, together with the behaviour of the wall in its unreinforced condition. As saw in
the previous chapter, unreinforced walls had a strong flexural behaviour when tested with the lower vertical pre-compression load, characterized by rocking of the walls and uplifting of the vertical posts. The walls tested with the higher pre-compression load exhibited a composite flexural-shear resisting mechanism. The retrofitting techniques adopted aimed at improving the performance of the walls under cyclic loading by limiting the rocking mechanism and improving their resistance, ductility and energy dissipation.

Comparing the hysteretic behaviour of the strengthened walls with bolts at connections and the corresponding unreinforced half-timbered walls, it is observed that there is no great gain in terms of ultimate capacity and stiffness. In fact, for the lower vertical load level, the gain in terms of maximum load was of 23.7%, while for the higher vertical load level it lost 5%. In terms of ultimate displacement, the walls gained 5.7% and 0.2% respectively (Figure 5.12).

![Figure 5.12 - Hysteretic diagrams for walls strengthened with bolts: (a) lower pre-compression load; (b) higher pre-compression load](image)

However, the general behaviour of the walls changes. In fact, for both load cases, but especially for the lower vertical load level, the shape of the hysteretic loops experiences some changes. The plateau caused by the uplifting of the vertical post from the base beam is still clearly present, but it is less pronounced and the unloading branch of the cycles is smoother. In fact, vertical uplifting in the posts decreased by approximately 40% for both load cases. Moreover, the S-shape of the hysteretic loops for wall RIW25_B (Figure 5.12a) is not so evident, meaning that the flexural resistant mechanism is not so predominant and a certain shear resistant component develops. The bolts contribute to the resistance to tensile forces present in the bottom half-lap connections, when lateral loading is applied at the top of the walls, ensuring a degree of continuity to the bottom connections, which could not open freely. Moreover, since the posts cannot fall out of place when the connections close, thanks to the confining effect of the bolt, the connections are able to work until failure, fact which did not occur in unreinforced walls, where after a certain lateral drift there is no contact between the post and the bottom beam. This effect is overcome by the presence of the bolt.
The very low effectiveness of bolts as a retrofitting technique in half-timbered walls can be attributed to the important flexural behaviour. In fact, bolts are not completely efficient in resisting the tensile stresses induced by cyclic loading, being possible to observe practically the same damage patterns, i.e., tearing off of the half-lap connections. Since shear was predominant in case of timber frame walls (without infill brick masonry) and due to their failure mode, this retrofitting solution would not have been adequate for timber frame walls, therefore it was decided to retrofit timber frame walls only with steel plates.

The comparison between the hysteresis diagrams found in original walls and after retrofitting with steel plates can be made through the analysis of Figure 5.13. The retrofitted half-timbered wall submitted to the lowest level of vertical load failed at a displacement of 52.9mm, resulting from the failure of the right bottom connection due to the presence of a knot near one of the bolts securing the plate, but a great gain in terms of stiffness and strength was recorded. The premature failure resulted in a decrease on the displacement capacity of 6%, but the maximum lateral load presented an increase of about 147% in relation to the original wall. The initial stiffness of the wall increased by 30% when compared to the unreinforced solution. The behaviour is still governed by a predominant flexural mode which resulted in the failure of the bottom connections, associated to important uplifts, which are lower than in the case of the original walls but of the same size order.

The hysteretic behaviour of wall RIW50_P is displayed in Figure 5.13b. The gain in terms of maximum load is also considerable in this case, reaching an increment of 60.4%. Moreover, it is observed that the initial stiffness increased of 14% and the ultimate displacement is of the same size order. However, the displacement imposed to the walls does not correspond to the maximum displacement capacity as it was not possible to obtain the failure mode of the wall, even if, due to the high levels of lateral load reached, the wall was experiencing some out-of-plane displacement, reaching values of 6mm at mid height of the wall. The high stiffening effect of the retrofitting solution 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.

It should be pointed out that the use of star shaped steel plates, linking the main elements of the connection (post and beam) to the diagonals gives a significant additional strength and stiffness to the wall. In fact, 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 of the amount of vertical load becomes secondary.
As already mentioned, for the half-timbered walls the custom plates were adopted aiming at better accommodating the infill. The analysis of the damage of the plates after the test revealed that the deformation concentrated around the holes of the bolts and no signs of deformation on the steel plates were recorded. Moreover, for these plates, no distributed screws were used and the plates exhibited a trend to bend, following the deformation of the bolts.
Aiming at exploiting more the deformation ability of steel plates, it was decided to adopt plates with a greater theoretical capacity to deform, according to technical catalogues. However, once the plates were characterized, it was seen that the high number of holes, aiming at better distributing the stresses in the steel plate, created a preferential failure path in the plates before reaching high deformations. Notice that the higher number of fasteners in a connection is favourable from the point of view of ductility (Eurocode 8, 2004). The adoption of commercial plates intended also to reduce the costs and grant an easier implementation of the retrofitting technique in real interventions.

The lateral cyclic behaviour obtained for timber frame walls with commercial plates linking the main members (post and beams) with the diagonals for both load levels is shown in Figure 5.13c,d. The retrofitted walls exhibited a highly out-of-plane behaviour, mainly in the positive direction which resulted in the instability of measurements of the in-plane response for both pre-compression levels. In the negative direction, instability was observed mainly in the last part of the tests. The configuration of the retrofitting scheme increased significantly the connections, leading to a significant increase on the lateral strength and thus on the loading of the diagonals, promoting the development of second-order effects. In order to record the total response of the walls, at least in the negative direction, tests were stopped and restarted applying the displacement cycles only in the negative direction. Nonetheless, even in this direction, the wall was not able to reach the applied displacement and out-of-plane instabilities were significant for high imposed lateral displacements (RTW1_P reached -76.47mm for an applied displacement of -103.92mm, while RTW2_P reached -66.20mm for an applied displacement of -91.09mm). The out-of-plane behaviour was also promoted by the type of failure found for this type of retrofitting technique, which developed in the half-lap connection linking two diagonal members in a cell. When this connection failed, the diagonal destabilized, leading to the global destabilization of the wall as the result of the trend for the out-of-plane deformations. From the results obtained, it appears that this type of retrofitting is too strong for timber frame walls without infill, increasing significantly the lateral resistance and preventing severely the movement of the diagonals, which is a deformability feature of the unreinforced timber frame walls, as observed in Chapter 4.

In order to avoid this behaviour it was decided to adopt the same type of strengthening but without linking the diagonals, i.e. the steel plates are in the same position as in the previous tests, but the bolts and screws link only the main members (posts and beams) allowing the free detachment of the diagonals, allowing the diagonals to deform freely. This solution allowed the walls to gain significantly both in terms of stiffness and load capacity, without compromising the displacement capacity (see Figure 5.13e,f). In fact, in terms of maximum
load, the walls gained 183% and 35% for the lower and higher pre-compression load respectively, while lost 5% and 3.5% in terms of ultimate displacement respectively. On the other hand, this retrofitting solution led to severe pinching in the timber walls. Similarly to the retrofitting with custom plates, also in this case, 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.

To sum up, the walls retrofitted with steel plates behaved predominantly in shear. The posts are still uplifting, but the plateau in the cycles is not as visible as in the unreinforced condition. The unloading branch is now smoother, providing a greater dissipative capacity to the walls, even if the pinching effect is considerably higher in these cases. It should be noticed that the application of the steel plates to the diagonal bars promotes the considerable increase on the stiffness by confining the connections, leading to more important uplifts and more influence of the flexural resisting mechanism when compared to the retrofitting technique where the diagonals are free to deform and the walls experienced higher levels of uplift. The stiffening effect of the custom steel plates appears to be similar to the one observed in case of the addition of the brick masonry, which can be confirmed when timber frame walls are compared to brick masonry infill half-timbered walls (see Chapter 4).

Considering the walls strengthened with NSM steel flat bars, in qualitative terms, it is observed that this type of strengthening enhances the lateral behaviour of half-timbered walls, with the improvement on the lateral resistance of 62% and 30% for the walls submitted to the lowest (RIW25_S) and highest (RIW50_S) pre-compression load levels respectively, see Figure 5.14a,b. In case of the wall RIW25_S, two phases can be identified in relation to the predominant resisting mechanism. The first phase is characterized by a predominant shear resisting mechanism before the failure of the bars at the bottom connection due to insufficient anchorage length. As a consequence of this failure, the wall exhibited a flexural behaviour characterized by rocking and by the S-shape of the curve (Figure 5.14a), similarly to what happens with unreinforced walls. It is interesting to notice that in the second phase the force-displacement diagram is practically coincident with the one recorded in the unreinforced wall. In fact, after the failure of the bottom connections the remaining steel bars are ineffective and almost no contribution to the cyclic response is recorded. The failure of the bottom connections led to the natural increase of the vertical uplift, see Figure 5.14a. The anchorage configuration adopted for the NSM steel flat bars revealed not to be adequate, resulting in the opening of horizontal cracks in the bottom beams due to tensile stresses developed in the direction perpendicular to the grain, in the bent part.
As it was mentioned previously, to avoid the premature failure of the bottom connections, it was decided to use commercial steel plates at the base. Besides, in this case only one plane of NSM steel bars was considered by applying the steel flat bars in the discontinuous part of the half-lap connections. This choice was made due to the confinement given by the infill, which prevents sudden ruptures in the timber elements. The shear resisting mechanism prevailed in the lateral response of the reinforced half-timbered walls submitted to the highest level of pre-compression, which was particularly evident from the unloading branch of the force-displacement diagrams and from the limited vertical uplift of the posts (Figure 5.14b). In this case only a minimal plateau in correspondence to the closing of the connections in the unloading branch was recorded when compared to the unreinforced wall.

For timber frame walls, given the absence of the confining effect of brick masonry and the predominant shear resisting mechanism observed in the unreinforced walls, accompanied by the higher level of deformation of the connections, it was decided to strengthen all connections applying the NSM steel flat bars both horizontally and vertically at the main half-lap connections.

Figure 5.14 - Steel flat bars with NSM strengthening (a) half-timbered wall, lower vertical pre-compression; (b) half-timbered wall, higher vertical pre-compression; (c) timber frame wall, lower vertical pre-compression; (d) timber frame wall, higher vertical pre-compression.
From the comparison between the unreinforced and retrofitted timber frame walls (Figure 5.14c,d) it is clearly visible a considerable improvement of the lateral response, with the increase on the lateral resistance of about 197% and of 64% for the retrofitted walls submitted to the lowest and highest levels of pre-compression respectively. However, a reduction on the ultimate displacement capacity was also observed being of 12% in case of the wall RTW25_S and of 6% in case of the wall RTW50_S. It should be pointed out that, for the last loading steps, the walls experienced a small out-of-plane component, which should be associated to the high level of stresses to which they were subjected, reducing their ultimate displacement capacity.

A remark should be made in relation to the increase percentage on the cyclic lateral resistance of wall RTW50_S. In fact, it was observed that in no case the walls presented severe damage both at the connections and timber elements. Due to this, wall RTW50_S was submitted to a monotonic test after the cyclic test in order to characterize the failure mechanism. The lateral resistance obtained of 179kN was 121% higher than the one recorded in the unreinforced timber frame wall, which confirms that the increase on the lateral resistance recorded in the cyclic test did not mobilize all the contribution of the flat steel bars and does not correspond to the failure configuration of the wall. The lateral displacement corresponding to the failure of the wall was of about 89.36mm.

In both timber frame walls, the lateral response of the walls is governed by the resisting shear mechanism, even if the uplifting at the bottom connections was still present. A difference between the two walls is observed in the unloading branch, since RTW50_S wall presents a non-smooth unloading branch, even though the vertical uplift was lower. This could be due to a greater difficulty in the recovery of the uplift deformations due to the higher friction resistance associated to the higher levels of stress imposed in the posts, as witnessed by the higher strains registered in this wall.

All walls strengthened with steel flat bars experienced pinching, but it was more severe for timber frame walls, as observed for walls retrofitted with steel plates. It appears that pinching manifests itself more when there is less confinement, being it given by the infill or by the strengthening.

By comparing the improvement of the lateral strength between half-timbered and timber frame walls, it is observed that the retrofitting with NSM flat steel plates proved to be more effective for timber frame walls rather than half-timbered walls. This should be associated to the absence of infill allowing the timber frame to deform more and develop a more predominant shear resisting mechanism.
For all walls it is possible to observe the strength and stiffness degradation from the hysteretic curve. The latter will be discussed in the following sections. As far as the strength degradation is concerned, it is easily visible how, once the behaviour of the walls becomes non-linear, the walls experience a higher strength degradation than unreinforced walls. This topic will be further analysed in Chapter 7.

5.5.2 Deformational features of the walls

Besides the uplift of the posts analysed previously, some other deformational features are also here analysed in order to better understand the lateral behaviour of the different walls.

From the displacement measured on the diagonals (Figure 5.15) it is possible to understand the stiffening effect that each retrofitting technique has on the walls. For the retrofitting technique with bolts (Figure 5.15a), the diagonals did not exhibit considerable deformation until the maximum lateral load was achieved, similarly to what happened in infill walls (see Chapter 4). After failure of the central connection, the confinement of the diagonals was not effective anymore, greater displacements of the diagonals were measured, resulting in a greater deformation capacity of the wall. For both vertical load levels, the diagonals reached displacements of about 45mm. For all walls retrofitted with steel plates (Figure 5.15b,c), the displacement of the diagonals was considerably lower, achieving values of the order of 23mm, with the exception of wall RIW50_P. In this wall, the complete failure was due to the failure of the half-lap connection of the diagonals, similarly to the other walls with the same retrofitted solution. Consequently, the non-effectiveness of this diagonal bar resulted in the increase of the diagonal deformation, particularly when the lateral imposed displacement induced compressive stresses along the diagonal. Failure caused the connection not to work even in compression and led to higher deformations. Apart from this anomaly, the limited displacement of the diagonals confirms the high stiffening effect of the retrofitting with steel plates, particularly when the diagonals elements are connected to the main members by the steel plates.

When the bracing elements are not linked to the steel plates (Figure 5.15d) failure was associated to the progressive crushing and shear failure of the central connection, leading to the increase of crack opening and thus to higher elongations of the diagonals. The screws of the central connection that helped distribute the stresses in the steel plates all failed in shear, after which the plates were secured only by the bolts. On the other hand, the bolts exhibited high plastic deformations.
Figure 5.15 - Diagonal displacement in walls for higher vertical load level: (a) bolts strengthening; (b) plates strengthening in infill walls; (c) plates strengthening in timber walls; (d) plates strengthening in timber walls (only main connections); (e) NSM steel flat bars in infill wall; (f) NSM steel flat bars in timber frame wall.

For the NSM strengthening, in spite of the diagonals being submitted to higher levels of compressive stresses and thus induce higher shear stresses at the central connections, given that the lateral resistance is considerably higher than in case of unreinforced walls, low levels of damage in the connections were observed (Figure 5.15e,f). This retrofitting technique improved thus the shear resistance of the connections by preventing their crushing as it was observed in the unreinforced walls. In any case, the diagonal displacements are clearly limited when compared to the ones observed in the unreinforced walls. This is
particularly relevant in case of timber frame walls, as it presents more than the double of the diagonal displacements. Only after failure of the central connections due to the strengthening failure during the monotonic testing of the timber frame walls submitted to the highest level of pre-compression, see Figure 5.15f, the diagonals reached high values of displacement.

In Appendix B, Figure B.8, the graphs of all the diagonal displacements of the walls can be found.

The same conclusions can be drawn from the analysis of the horizontal displacement at mid height of the wall (Figure 5.16). For both walls retrofitted with bolts (Figure 5.16a), the two sides of the walls experienced similar displacements up to failure, at which time the displacement at mid height was approximately half of the top displacement, indicating an almost linear deformation of the posts for low values of applied displacement.

After failure, the displacements at mid height increased significantly and became asymmetrical. In fact, with the shear crack opening at the central connection it is not possible to have a full displacement transfer between border posts, so differential displacements are encountered. The same behaviour was noticed for timber walls strengthened with steel plates without linking the diagonals (Figure 5.16b).

For the stiffer solutions (Figure 5.16c,d), the displacement on the two sides of the wall was similar, with a small tendency to deform more on one side after failure occurred (Figure 5.16c). The displacements recorded were generally higher than half of the displacement applied to the top of the wall, meaning that the deformation of the walls was not the result of the rotation of the walls but results from the deformation associated to flexure and shear of the wall.

For the retrofitting with NSM steel flat bars, the displacement on the two sides of the wall is similar. There is only a trend for some distinction between the horizontal displacements measured at mid height between the displacements at both sides of the walls for higher deformations (Figure 5.16e,f). In Appendix B, Figure B.9, the graphs of all the displacements at mid height of the walls can be found.

The different deformation of the walls can also be observed in the openings recorded in the different connections, see Figure 5.17, in which distinct local deformations measured at the half-lap connections are shown. For walls retrofitted with bolts (Figure 5.17a), the half-lap connections linking the vertical posts and the central beam experience low openings throughout the tests (up to 4mm), similarly to what was experienced in unreinforced half-timbered walls (see Chapter 4). Once failure occurred in the half-lap connections in the middle beam, the opening of the connection measured by LVDT CL increased considerably and in a progressive way, resulting from the redistribution of stresses until the stress transfer
is compromised by the complete separation of the beam since the continuity of the beam was compromised and the displacement was not transferred from one side of the wall to the other. At the same time, the central connection failed, having a similar behaviour, i.e. opening progressively and causing the complete separation of the two parts of the beam (see Appendix B, Figure B.10a). The same behaviour was observed in RIW25_B, but the LVDT disposition did not allow to record these openings.

A different deformation was observed among the walls retrofitted with steel plates, linking the main elements of the connections to the diagonals (Figure 5.17b), both for half-timbered
walls and for timber frame walls, The displacement recorded at the main half-lap connections was minimal, not reaching 5mm. This was observed in all walls retrofitted with steel plates, as can be ascertained from Appendix B, Figure B.10b-j. This result confirms again that the retrofitting technique based on steel plates was effective in the prevention of the opening of the connections, promoting a more rigid response of the wall.

The use of this retrofitting technique strengthened the main connections, leading to the stresses transfer to the connections linking the diagonal elements in the cells. This resulted

Figure 5.17 - Opening of connections: (a) central left connection of wall RIW50_B; (b) central left connection of wall RIW50_P; (c) diagonal opening of half-lap connection in wall RTW50_P; (d) central middle connection of wall RTW50_P_M; (e) horizontal detachment of diagonal, RIW25_S; (f) vertical detachment of diagonal, RTW50_S
in the failure of the half-lap connection linking the two diagonals, causing a progressive opening of that connection (Figure 5.17c). Notice that in timber frame walls, even though the connection failed, the diagonals maintained their linearity, so when the failed element was in compression, the connection was still working and the walls were not subjected to great deformations, contrarily to what happened for wall RIW50_P. Notice that, when the connection of the diagonals did not fail, the opening of the connections was comparable to that of the other connections in the wall, with openings less than 5mm (see Appendix B, Figure B.12a-l). In wall RTW25_P, the connection in the right bottom cell failed and its opening is shown in Appendix B, Figure B.12g.

For walls strengthened with steel plates without linking the diagonals failure occurred in the central connection for both load levels, which started to open progressively (Figure 5.17d). Nonetheless, the connection was still able to work, since the steel plates were keeping the timber elements together. In fact, in order to see the damage level to which the wall had been subjected, it was necessary to take out the steel plates, since no damage was visible otherwise.

For all walls in which the diagonals were not linked to the main frame, the nailed connections between the diagonals and the main frame recorded significant opening, reaching a detachment of 15mm, see Appendix B, Figure B.11. This was due to the fact that the steel plates, though guaranteeing the continuity of the connections, did not prevent progressive damages in the same, which allowed a higher mobility to the diagonal elements. In general, it is noticed how these displacements are usually lower in case of infill walls, due to the confining effect of infill, which hinders the deformation capacity of the timber frame.

In NSM steel flat bars retrofitted walls, the relative vertical and horizontal displacement of the diagonal in relation to the vertical post is low due to the higher confining effect of the NSM flat steel bars in the walls, particularly in case of infill walls (Figure 5.17e). Similarly to what happened in unreinforced walls, also when retrofitted timber frame walls present higher level of detachment, even if it is 75% lower than in the case of unreinforced timber frame walls, see Figure 5.17f.

To understand the efficiency of the retrofitting techniques adopted, in particular to understand the level of activation moment and the performance of the element applied, strain gauges were applied to the steel elements used in the reinforcement techniques.

In general, strain gauges applied to the steel plates recorded small deformations in the plates; usually strains recorded were under 1.5‰ (Figure 5.18a). This was observed for steel plates in different positions in the walls (see Appendix B, Figure B.13). The main deformation
in the steel plates consisted in the ovalization of the holes where the bolts were inserted and bending of the plates which could not be prevented by the screws, since they failed in shear.

Strain gauges were positioned even on the timber elements, particularly on the lateral posts to evaluate the deformation to which the members were subjected. In general, small values of strain were recorded, both at mid height (Figure 5.18b) and at the top of the post (see Appendix B, Figure B.13), indicating that, even though the posts were deforming, their deformation capacity was not completely exploited.
Steel flat bars applied to both half-timbered and timber frame walls registered high values of strain, reaching values beyond the yielding. For half-timbered walls (Figure 5.18c), the most stressed bars were the ones embedded in the half-lap connection where the element was discontinuous. Both vertical and horizontal bars reached strains of 5‰, confirming that the flat steel plates are in the plastic regime. In timber frame walls (Figure 5.18d,e), NSM flat steel bars at the central connections exhibited the highest strains, attaining values of 6‰. Lower strains were recorded in the steel plates located at the lateral posts reaching values of 2‰.

5.5.3 Typical damage patterns

The retrofitted walls experienced severe damages after failure, contrary to what observed in unreinforced walls. Nonetheless, in most cases, the strengthening was still able to work and guarantee an adequate resistance of the wall.

The distinct deformational features of the walls discussed previously resulted from distinct damage patterns exhibited by the different walls. The typology of strengthening is particularly relevant to the damage patterns when half-timbered and timber frame walls are compared.

Walls strengthened with bolts exhibited severe damages. The walls develop damages in the central connections, which failed, and the nailed connections between the diagonals and the main frame detached. The central beam tore off (Figure 5.19a) in tension and the central post crushed due to the shear induced by the diagonals (Figure 5.19b,c), similarly to what happened in the unreinforced tests (see Chapter 4). In Figure 5.19b it is clearly visible the diagonal crack that passes through the bolt caused by the diagonals in compression. The damage developed in the post was more significant for the wall subjected to the higher vertical load level, as the left connection at mid height of the wall also failed (Figure 5.19d), causing a higher shear effect of the diagonals on the central connections, as they had a higher deformation capacity. Even though part of the central post had been substituted by gluing a new wooden element after the test of the unreinforced wall and the shear induced by the diagonals made this piece to detach (Figure 5.19c), it should be noticed that detachment did not occur in the glue interface but in the thickness of the element, which confirms that the continuity was restored.

As already mentioned, in case of walls retrofitted with steel plates the damages observed were similar for all walls and they consisted in: (1) failure of the half-lap connection linking two diagonal member in a cell for walls in which the steel plates linked the diagonals to the main frame; (2) failure of the central middle connection for walls where the diagonals were not linked to the main frame through the steel plates.
As mentioned above, failure in RIW25_P occurred early in the bottom right connection, caused by the presence of an important knot near the bolt (Figure 5.20a), proving that a weak zone can govern the failure mode. However, when the test was repeated, failure started in the diagonal element of the bottom right cell, when this element was in tension, i.e. the wall was being pulled (negative direction).

For wall RIW50_P, the failure occurred in the half-lap connection linking the two diagonal elements of the bottom left cell (Figure 5.20b). Contrarily to what happened in all other walls with this retrofitting, both elements of the connection failed, thus influencing the behaviour on both directions of loading. Moreover, the diagonal did not maintain its linearity, but was falling out. The central connection of the wall did not fail, but it is possible to see the ovalization of the holes accommodating the bolts in the diagonal elements, since the elements were particularly stressed. Moreover, for this wall, diagonal cracks appeared at the lateral bottom connections due to the shear effect of the diagonals (Figure 5.20c).
Figure 5.20 - Typical damages in walls strengthened with steel plates: (a) failure of bottom connection in RIW25_P; (b) failure of half-lap connection in bottom cell in RIW50_P; (c) shear cracking in bottom connection in RIW50_P; (d) failure of half-lap connection in bottom cell in RTW50_P; (e) shear cracking in bottom connection in RTW50_P_M; (f) failure of central connection in RTW50_P_M.

In timber frame walls with the same plates' disposition, the failure was the same, i.e. failure of diagonal element (Figure 5.20d). This failure occurred in all specimens, independently on the type of plate, because the type of strengthening stiffened excessively the connections, not allowing free movement to the bracing elements. The creation of strong retrofitted points
resulted in the failure of the weakest zones of the wall, which were the half-lap connections of the diagonals. Notice that no damages were observed in the main wood members of the connection. An ovalization of the holes for the bolts in the diagonals was observed for timber frame walls too. Also in this case, diagonal cracks were present at the lateral bottom connections, due to the action of the diagonals (Figure 5.20e).

When the diagonals are free to move, the failure occurs in the main member of the frame. In both timber frame walls tested without linking the diagonals with the steel plates, failure occurred in the central middle connection (Figure 5.20f) due to the shear action imposed by the diagonal elements. For the higher vertical load, the failure propagated along the horizontal beam in alignment to the bolt applied in this beam, due to the presence of a knot. From Figure 5.20f the rotation of the steel plate during the test is clearly visible. The rotation was associated to the non-deformation of the steel plate and to the shear failure of the screws resulting from their shear resisting mechanism against this rotation. All diagonals presented permanent clearances at the end of the test, having detached from the main frame of as much as 15mm.

For walls strengthened with NSM steel flat bars, different failure modes were encountered due to the trial and error method applied when strengthening the walls. As already mentioned, for wall RIW25_S an early failure was registered due to failure of the welding applied in the steel flat bars at the bottom connections associated to excessive tensile stresses developed at the bottom connections, see Figure 5.21a. The choice to use a bended bar was made in order to guarantee a sufficient anchorage length. Due to this early failure, no other damages were observed in the wall, which behaved as an unreinforced wall. For wall RIW50_S, since the number of bars was reduced and steel plates were used at the bottom, it is possible to observe the deformation of the bar at the central connection (Figure 5.21b), though complete failure of the bar did not occur.

A different bar disposition was applied to timber frame walls. In RTW25_S a significant deformation was observed in the bar strengthening the central connection (see Figure 5.21c) as well as cracks on the central beam. Moreover, the left post cracked, but independently from the NSM strengthening.

Since no significant damages were observed in RTW50_S wall at the end of the regular test, as already mentioned a monotonic test was carried out at the end of the cyclic one. During this test, failure occurred at the central connection, see Figure 5.21d, associated to the failure of the bar at the welding and further propagation of cracking in the wood.
With the deformations reached, the approximate strength estimated in the bars for deformation of 6‰ was of 627MPa, a value 50% higher than the yield strength. For both wall typologies, the bars deformed in the plastic regime.

Also for timber frame walls, buckling of the steel plates at the bottom connections was observed, as well as shear cracks caused by the compressive action of the diagonals.

![Figure 5.21 - Typical damages in walls: (a) welding failure in RIW25_S; (b) deformation of steel bar in central connection in wall RIW50_S; (c) bar deformation in central connection of RTW25_s wall; (d) failure of steel flat bar in RTW50_S wall after monotonic test.](image)

Concerning the deformations observed in the strengthening elements, all bolts used in the bottom connections, either as a standalone solution or together with steel plates, presented important deformations (Figure 5.22a), since the uplifting was still present in all walls.

As mentioned above, steel plates, both custom and commercial, did not encountered significant deformations during tests. Only the plates located at the bottom connections tended to buckle (Figure 5.22b) due to the elongation and compression during the test to which they were subjected. However, the holes accommodating the bolts were ovalized, particularly when the relative bolt deformed during the test.
Moreover, the bolts located in the half-lap connections that failed were significantly deformed (Figure 5.22c). It has to be noticed that for all bolts the deformation happens at a length of approximately 6cm, which is exactly the thickness of half connection, i.e. where the two elements are in contact. When analysing the connections after demolishing the walls, the holes in the timber elements of the posts, i.e. the elements that were uplifting, presented severe ovalization. For walls where the steel plates were linked to the diagonals, the higher deformations of the bolts were those which were linking the diagonals to the main frame (Figure 5.22d) and the holes of the diagonals were those which had a more important ovalization of the steel plate.

When the opening of the half-lap connections was negligible (Appendix B, Figure B.10), the deformation of the bolts and of the plates was minimal and it is not reported here.

In all half-timbered walls the damage was also observed in the masonry infill, with cracking (mostly at the unit-mortar interface), detachment of masonry from the main frame and out-of-plane rotation of the masonry blocks, see Figure 5.23. The damages were concentrated at
the bottom half of the wall, as happened in unreinforced walls (see Chapter 4), but they propagated even in the upper part of the walls, particularly for walls strengthened with bolts (Figure 5.23a). In case of half-timbered walls strengthened with steel plates the complete detachment of some masonry blocks was observed (Figure 5.23b). For walls retrofitted with NSM steel flat bars, masonry experienced minimal damages and it will not be reported here.

![Figure 5.23 - Crack pattern in half-timbered walls: (a) bolts strengthening; (b) strengthening with steel plates](image)

It has to be pointed out that, once the connections failed and deformed, masonry blocks exhibited a trend for falling into the opening, which resulted in the reduction of connection confinement and in the progressive increase on the connections opening.

### 5.6 Seismic performance parameters

Similarly to what done for unreinforced walls, seismic parameters such as initial and cyclic stiffness, energy dissipation, ductility and equivalent viscous damping were obtained for retrofitted walls. The values were calculated according to the procedures already presented in Chapter 4, so they will not be repeated here.

#### 5.6.1 Envelope curves and ductility

As happened for unreinforced walls, even though according to the standard the envelope curves should be plotted for both the initial cycle and the two stabilization ones, since low variations were observed during some of the tests among the three repetitions, the envelope curves of the two stabilization cycles will be discussed further on.

Figure 5.24a presents the curves for the walls tested with the lower pre-compression load, while Figure 5.24b presents the same results for the higher pre-compression load. For both load cases, retrofitting technique with bolts provided the lower increase both in terms of stiffness and of load capacity. In fact, with this technique it was possible to achieve displacement capacities comparable to what was achieved in the unreinforced tests, which resulted from the lower increase on the stiffness of the wall. An increase in load capacity was observed only for the lower vertical load level, while for the higher one almost no gain in load capacity was registered.
The retrofitting technique with the steel plates linking the diagonals to the main members of the wall gave similar results in terms of stiffness, load capacity and displacement capacity almost independently on the vertical load level and on the typology of the wall (either half-timbered or timber frame walls), indicating that the effect of the retrofitting technique hinders the influence of both factors on the response of the walls. On the other hand, when the diagonals were not linked to the main elements of the frame, a lower stiffness was observed in both load cases, being more evident the coupling effect on the variation of the vertical loading and the application of the retrofitting.

As seen previously, the NSM retrofitting technique was more efficient in timber frame walls. The initial stiffness offered by this solution is similar for both infill and timber frame walls, but the ultimate capacity reached is higher for timber frame walls, mainly due to the different mechanisms developing in the two kinds of walls. In half-timbered walls the flexural behaviour predominates and consequently the strengthening of the bottom connections is of paramount importance so that the uplift of the walls can be avoided. Notice that if flexural/rocking predominates, the deformations of the remaining connections are limited, which contribute for the inefficiency of the NSM flat steel bars, given that they are not exploited. In timber frame walls the connections present a higher degree of mobility, meaning that the reinforcement introduced at the connections is able to contribute more for the lateral resistance. In case of RIW25_S, the capacity of the wall was influenced by the early failure of the flat bar applied to the bottom connection.

When comparing the results obtained for the retrofitted walls with examples available in literature in relation to the strengthening of traditional connections, higher improvements are recorded for the single connections.

In the following tables (Table 5.2 to Table 5.6), the significant values of the envelope curves for all cycle repetitions and for all retrofitted walls are reported. As it can be easily deduced,
the strength degradation in the stabilization cycles is usually small. For example, for RIW50_B walls, the variation is less than 5% (Table 5.2), for RTW50_P it is less than 1% in the negative direction (Table 5.4), but the wall had not reached failure, so it is not completely representative.

In other cases, the variation can be significant; for example, wall RTW50_P_M had a 20% strength loss in the cycle repetition at the step where the maximum load was reached (Figure 5.25a). This can be easily deduced from the comparison of the three envelope curves of the walls.

Comparing the different retrofitting solutions, the highest values of resistance were obtained for walls retrofitted with steel plates, with little influence on the vertical load. However, for these walls, the ultimate displacement was lower.

Table 5.2 - Significant values for walls retrofitted with bolts

<table>
<thead>
<tr>
<th>WALL</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$d_{\text{max}}$ [mm]</th>
<th>$d_u$ [mm]</th>
</tr>
</thead>
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<tr>
<td></td>
<td>RIW25_B</td>
<td>RIW50_B</td>
<td>RIW25_B</td>
</tr>
<tr>
<td>1st envelope curve P</td>
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<td>107.00</td>
<td>87.97</td>
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<td>N</td>
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<td>2nd envelope curve P</td>
<td>72.61</td>
<td>99.70</td>
<td>76.64</td>
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<tr>
<td>N</td>
<td>-68.95</td>
<td>-83.56</td>
<td>-44.42</td>
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<tr>
<td>3rd envelope curve P</td>
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<td>97.59</td>
<td>76.75</td>
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<td>-83.22</td>
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<td>N</td>
<td>5.43</td>
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<td>11.42</td>
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Table 5.3 - Significant values for walls retrofitted with steel plates, lower vertical load level

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<th>WALL</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$d_{\text{max}}$ [mm]</th>
<th>$d_u$ [mm]</th>
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<td>curve</td>
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<td>147.13</td>
<td>104.91</td>
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<tr>
<td>curve</td>
<td>N</td>
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<td>-173.08</td>
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<td>3rd envelope</td>
<td>P</td>
<td>142.01</td>
<td>101.98</td>
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<tr>
<td>curve</td>
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<td>avg</td>
<td>P</td>
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<td>-173.91</td>
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<td>C.O.V. [%]</td>
<td>P</td>
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<td>6.17</td>
</tr>
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<td></td>
<td>N</td>
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<td>1.98</td>
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Experimental program: cyclic tests on retrofitted walls
Table 5.4 - Significant values for walls retrofitted with steel plates, higher vertical load level.

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<th>WALL</th>
<th>$F_{\text{max}}$ [kN]</th>
<th>$d_{\text{max}}$ [mm]</th>
<th>$d_u$ [mm]</th>
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<tr>
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<td>N</td>
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<tr>
<td>curve P</td>
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<td>122.44</td>
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<td>N</td>
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<td><strong>avg</strong></td>
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<tr>
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Table 5.5 - Significant values for walls retrofitted with NSM steel flat bars, lower vertical load level

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<th>( d_{\text{max}} )</th>
<th>( d_u )</th>
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<td>[kN]</td>
<td>[mm]</td>
<td>[mm]</td>
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<tr>
<td>1\textsuperscript{st} envelope curve</td>
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<tr>
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Table 5.6 - Significant values for walls retrofitted with NSM steel flat bars, higher vertical load level.

<table>
<thead>
<tr>
<th>WALL</th>
<th>$F_{max}$ [kN]</th>
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<th>$d_u$ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
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<td>RTW50_S</td>
<td>RIW50_S</td>
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<td>P</td>
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<td>N</td>
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<tr>
<td>$2^{nd}$ envelope curve</td>
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<td>127.22</td>
<td>154.67</td>
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<tr>
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<td>-153.63</td>
<td>-66.73</td>
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<td>$3^{rd}$ envelope curve</td>
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<td>150.64</td>
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<td>avg</td>
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<td>-66.73</td>
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<td>C.O.V. [%]</td>
<td>P</td>
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<td>N</td>
<td>3.75</td>
<td>3.57</td>
<td>0.33</td>
</tr>
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</table>

Notice that not in all walls the maximum load was reached at the same step for all three envelope curves. The three envelope curves present the higher differences for timber frame walls (Figure 5.25a) (see more graphs in Appendix B, Figure B.14), while for infill walls the values are usually lower than 8% (Figure 5.25b). This is another indication of the effect of infill, which hinders the strength loss of the walls. RIW50_P experienced some strength degradation, but only after failure occurred and infill fell out of the wall.

![Figure 5.25](image_url) - Envelope curves of stabilization cycles: (a) wall RTW50_P_M presented significant strength degradation during the stabilization cycles; (b) the strength degradation of wall RIW50_S during the stabilization cycles was less than 10%.

As far as ductility is concerned, the values were obtained from the bilinear idealization according to Tomazevic (1999). The bilinear curves, once again obtained only for the positive part of the envelope curve, as already explained in Chapter 4, are shown in Figure 5.26. It should be stressed that for the majority of the walls, the ultimate displacement corresponds to the maximum one obtained experimentally, since only two of the walls lost more than 20% of the maximum load in the degradation process, namely walls RIW25_P and RIW25_S due
to their early failure and consequent flexural behaviour. Therefore, the ultimate displacement corresponds to the displacement reached in the last cycle imposed to the walls.

![Bilinear idealizations of tested walls: (a) lower vertical pre-compression; (b) higher vertical pre-compression.](image)

For wall RTW50_P the bilinear idealization is not presented, since in the positive direction the behaviour was linear up to the out-of-plane problems which led the test to be inconsistent in that direction.

From the bi-linear diagrams, the values of ductility were obtained, as specified in Chapter 4. In general, values of ductility for all walls were lower than what observed in unreinforced walls (Table 5.7). This is mainly related to the increase on the lateral strength and to the consequent increase on the yielding displacement, even for higher values of lateral stiffness. On the other hand, the strengthening with bolts resulted in low values of ductility due to the decrease on the lateral stiffness. Particularly, the stiffer the strengthening applied, the lower the value of ductility for the wall.

<table>
<thead>
<tr>
<th>WALL</th>
<th>( \mu_{1+} )</th>
<th>equivalent unreinforced wall</th>
</tr>
</thead>
<tbody>
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<td>5.20</td>
</tr>
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<td>RIW50_B</td>
<td>3.02</td>
<td>3.62</td>
</tr>
<tr>
<td>RIW25_P</td>
<td>2.54</td>
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</tr>
<tr>
<td>RIW25_P*</td>
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<td>5.20</td>
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<td>3.53</td>
</tr>
<tr>
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<tr>
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<td>5.20</td>
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</tr>
<tr>
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<td>3.62</td>
</tr>
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<td>4.57</td>
</tr>
<tr>
<td>RTW50_S</td>
<td>2.52</td>
<td>3.53</td>
</tr>
</tbody>
</table>
The ductility value of wall RIW25_S was calculated even without the displacement limitation (\(\ast\)) to appreciate the difference.

Notice that the evaluation of the ductility is directly related to the stiffness and the lateral strength as the yielding displacement used in the calculation of ductility is dependent on both variables.

The values were particularly low for strengthening done with steel plates. This is due to the high increase in load capacity of the walls, leading to the increase on the yield displacement, even for higher values of stiffness. This fact, in conjunction with the limited imposed lateral displacement, which did not correspond to the collapse state of the walls, resulted in the decrease of ductility. Nevertheless, it should be stressed that the limitation of the results obtained in this work is related to the fact that the values of the maximum lateral displacement obtained for the retrofitted walls did not effectively correspond to the failure of the walls and only moderate levels of damage were found. This can justify the lower values of ductility found for the retrofitted walls, when compared to the unreinforced walls, see Table 5.7. In any case, if the lateral drifts are considered, it is seen that the retrofitted walls are able to attain the same lateral drifts with considerably lower damage, which represents a great advantage of the retrofitting of the walls.

In particular, in walls retrofitted with custom steel plates (RTW50_P) the response was still in a linear regime when the out-of-plane behaviour occurred. Besides, and many walls did not reach a clear softening in their response or, if they did, the loss of strength was less than 20%. Both walls strengthened with bolts, even though they experienced failure of the connections, could have reached higher values of ultimate displacement, as the strength degradation was not great. The same can be said for RIW50_P and for all timber walls strengthened with steel plates. Though failure occurred, the steel plates were able to guarantee the efficiency of the connections.

Moreover, it is seen that the decrease on the ductility is related to the relative increase of the stiffness and lateral resistance promoted by retrofitting. In fact, higher values of elastic displacement are found for stiff and high resistant walls, which is particularly relevant in case of timber frame walls. Taking into account that the real maximum imposed displacements were not attained, due to the limited damage found, the trend for the decrease on the ductility of retrofitted walls should be seen with care. At least, the differences should not be as high, if the collapse state of the retrofitted walls was attained.

For these reasons, the values of ductility could increase if the tests could go further in terms of lateral drifts, as it believed that the walls would assure higher levels of lateral drift. Nevertheless, it was decided to provide the ductility of the walls, even if it should be viewed
as indicative in some cases. For example, wall RTW50_P_M had a loss of strength of 19% at the end of the test, and its ductility is similar to one obtained in the unreinforced specimen, indicating that its final ductility should be of this order. Similarly, wall RTW25_P_M reached a strength loss of 17%. For the other walls retrofitted with steel plates the ductility should not be viewed as a true value since they clearly show evidences of being able to withstand higher levels of lateral displacement. For example, wall RIW50_P reached a strength loss of only 5%. In any case, it can be seen that the ductility values obtained for the retrofitted walls are associated to low level of damages. The same can be said for walls retrofitted with steel flat bars: wall RIW50_S experienced a strength loss of 7% and wall RTW25_S of 0.4%.

Cruz et al. (2001) were able to improve the ductility of the walls between 158% and 316% due to the possibility of applying greater displacements than in the unreinforced configuration. Notice that they performed a diagonal compressive test, with different boundary conditions for the specimens than what done in the cyclic tests performed here.

5.6.2 Evaluation of initial stiffness and stiffness degradation

Initial stiffness was calculated with two methods, as done for unreinforced walls. Once again, the values were calculated only for the initial cycle, since the degradation during the cycle repetition was very small.

All types of strengthening provided values of initial stiffness higher than the ones recorded for unreinforced walls (see Chapter 4), apart from walls retrofitted with bolts, as can be seen in Table 5.8. Since the walls had already been tested, the repaired walls did not reach the same condition as the initial wall. Strengthening done with bolts is considered to be a very soft intervention on the walls, and being a very basic solution, it does not improve the stiffness or the capacity of the wall, but allows a full exploitation of the connection. The loss in terms of initial stiffness was of 46% and of 21% for the lower and higher level of the pre-compression load respectively with respect to the unreinforced timber walls.

The retrofitting technique with steel plates, as already mentioned, stiffened considerably the walls, particularly in case timber frame walls, even after the previous test. Indeed, this solution increased the initial stiffness of half-timbered walls of 31% and 14% when compared to the same unreinforced walls for the lower and the higher pre-compression load respectively. The gain was of 78% and 51% respectively for timber frame walls. It is to be pointed out that the absence of confinement given by the infill in timber frame walls led to a high increase both in terms of stiffness and of lateral load capacity. This last issue led to an important out-of-plane mechanism, more noticeable in timber frame walls; the increase of the lateral capacity of the wall contributed to a higher importance of second order effects, which exasperated the out-of-plane movement. With the use of commercial solutions for the plates,
not linking the diagonals to the main timber elements of the frame, the gain in terms of stiffness decreased to 30% and 28% respectively for the two load levels in relation to the unreinforced walls.

Table 5.8 - Values of stiffness for walls tested with different retrofitting solutions

<table>
<thead>
<tr>
<th>WALL</th>
<th>$K_{i,lin}$ [kN/mm]</th>
<th>$K_{i,s}$ [kN/mm]</th>
</tr>
</thead>
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<tr>
<td>RTW50_S</td>
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<td>3.57</td>
</tr>
</tbody>
</table>

All walls retrofitted with steel plates exhibited similar values of initial stiffness, apart from wall RTW25_P_M, which was more in line with stiffness provided by bolts, as its retrofitting has a less confining effect and more dependent on the vertical load level.

The NSM flat steel bars increase clearly the initial stiffness of the walls for the lower pre-compression load (25kN/post), being of about 38% for half-timbered walls and of about 96% for timber frame walls. When the walls are submitted to the vertical pre-compression load of 50kN/post, the effect of the NSM flat steel bars on the stiffness is not the same. In fact, almost no changes can be considered; in fact, half-timbered walls decreased their initial stiffness, due to the fact that fewer bars were inserted. The timber frame wall increased its initial stiffness by 28%, while the value of secant stiffness calculated with the second method remained unaltered.

The difference on the improvement of the stiffness due to NSM retrofitting among the walls should be associated to: (1) the stiffening effect of brick masonry on half-timbered walls and the predominance of the flexural behaviour results; (2) the fact that in case of the half-timbered walls submitted to a pre-compression load of 50kN/post, the configuration adopted for the NSM flat steel bars resulted in a lower amount of steel bars; (3) the higher effectiveness of NSM flat steel bars to increase the shear stiffness of timber frame walls; (4) the absence of the confining effect on the timber frame walls that present lower values of stiffness when they are tested in the unreinforced condition.
The improvement of the initial stiffness given by NSM flat steel plates appears to be in line with results achieved by past researches. According to Vasconcelos et al. (2013), the strengthening of timber frame walls by applying GFRP sheets at the connection clearly increases their initial stiffness.

The lower values of initial stiffness for retrofitted walls that had already been tested is in accordance to what was observed by other authors who performed diagonal compression tests on traditional half-timbered panels strengthened with FRP rods and GFRP sheets, recording a loss of stiffness of over 60% (Cruz et al., 2001). Walls retrofitted with steel plates were able to increase the initial stiffness of the walls, because the confinement provided to the connection lead to the impossibility of relative displacement since early stages of loading.

Stiffness degradation was evaluated according to the procedure adopted in Chapter 4 for unreinforced walls. Due to the accommodations that occur in the wall for low values of drifts already mentioned, cyclic stiffness calculated for drift values lower than 0.15% is not considered reliable and thus they are not represented here.

The variation of the cyclic stiffness for increasing lateral drifts is presented in Figure 5.27a,b. The lateral drift is calculated as the ratio between the lateral top displacement and the height at which the lateral load is applied. For all walls a dramatic loss of stiffness is encountered for values of drift lower than 0.5%, due to the accommodations in the walls at the beginning of the test. After this stage, the decrease on the stiffness is much lower and should be associated to the cumulative damage developed in the walls.

For both vertical pre-compression loads, strengthening with bolts gave the lowest values of cyclic stiffness, as well as a lower rate of degradation. The values are similar or lower to what was observed for unreinforced walls, since this solution did not improve the stiffness of the wall, but only the local behaviour of the connections, in the sense that no detachment was allowed. For low values of drift there was a decrease of cyclic stiffness of 47% and 12% for the lower and higher vertical load level respectively. For high values of drift, RIW25_B wall had a similar stiffness to the unreinforced one, while RIW50_B decreased its cyclic stiffness by 16%.

It is clearly visible that for strengthening carried out with steel plates linking the diagonals, the values of cyclic stiffness increased significantly, particularly for the lower pre-compression level. The amount of vertical pre-compression applied has little influence on the cyclic stiffness of the walls. In general, stiffness was higher for walls with a higher pre-compression level, but the difference was minimal.
Walls RTW\textunderscore P\textunderscore M recorded the lowest values of cyclic stiffness among the walls strengthened with steel plates, while RTW\textunderscore P the highest, but as already mentioned this over-stiffening is deleterious.

![Figure 5.27 - Stiffness degradation: (a) lower pre-compression load; (b) higher pre-compression load](image)

The repetition of the test performed on wall RIW25\textunderscore P did not allow a recovery of the cyclic stiffness of the wall, but nonetheless gave an indication of a better way of applying the strengthening.

It should be stressed that for retrofitting of the traditional timber frame walls with steel plates, independently on the type, the cyclic stiffness is always higher than that of unreinforced walls, even for the highest levels of lateral drifts, which revealed the effectiveness of the retrofitting from this parameter.

For timber frame walls with NSM steel flat bars it appears that the decrease on the cyclic stiffness up to lateral drifts of approximately 0.5% is followed by an increase up to a lateral drift of 1%; the strengthening does not immediately start working, as the initial values of stiffness are lower and comparable to those of unreinforced timber frame walls.

It has to be pointed out how, once wall RIW25\textunderscore S failed at the bottom connections, its behaviour became that of an unreinforced wall; even the values of stiffness were the same as what registered in unreinforced walls. The repetition of the test with steel plates at the bottom connections was not able to recover the cyclic stiffness of the wall, since the connections had irrecoverable damage. Stiffness was initially lower than that of the unreinforced test (Figure 5.27a) and remained approximately constant until the end of the test.

For half-timbered walls retrofitted with steel plates, the increase on the cyclic stiffness for low values of drift (from 0.2% to 0.4%) was of 102% and 66% for the lower and higher vertical load respectively, when compared to the unreinforced walls. For higher values of drift, the increase in the stiffness was of 140% and 96% for the lower and higher vertical load
respectively, thus guaranteeing an important stiffness even when the walls are damaged. For timber frame walls with the alternative steel plate configuration, the increase in cyclic stiffness for initial values of drift was of 92% and 113% for the lower and higher vertical load and of 178% and 78% for higher values of drift, indicating that after failure there was still an additional stiffness given to the wall by the retrofitting.

### 5.6.3 Assessment of the ability to dissipate energy

As already mentioned in Chapter 4, one major parameter used for the assessment of the seismic performance is the ability of a structural element to dissipate energy during cyclic testing. For retrofitted walls, the energy can be dissipated through friction in the connections, yielding and deformation of the retrofitting bolts, steel plates and bars and permanent deformation accumulated in the walls as observed during the tests.

All retrofitting techniques adopted were able to guarantee a greater energy dissipation during the tests (Figure 5.28a,b). Taking into account that a further lateral displacement could be imposed to the retrofitted walls, the maximum cumulative dissipated energy would be even greater.

The highest dissipative solution is provided by the retrofitting technique with steel plates linking the diagonals. For the walls tested without linking the diagonals, the dissipative capacity was lower.

Retrofitting with bolts showed results comparable to the ones obtained in unreinforced walls, improving only 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.

It is interesting to notice that wall RIW25_S presents good levels of dissipated energy for low to medium lateral drifts, approaching the values of the energy dissipated in the unreinforced walls after the lateral drift of 2.75% is achieved, which corresponds to the collapse of the bottom connections and the further predominance of a relevant rocking resisting mechanism. This also shows the difference on the dissipation of energy between shear and flexural resisting mechanism already observed by other authors in relation to other types of shear walls (Vasconcelos and Lourenço, 2009).
The same conclusions can be drawn when comparing the cumulative dissipated energy of the walls (Figure 5.29). All walls present an exponential trend, with the steel plates having the highest values of cumulative dissipated energy and bolts strengthening being comparable to the unreinforced condition in case of the lower level of vertical pre-compression. The cumulative energy dissipated in case of the walls retrofitted with bolts and submitted to the highest level of pre-compression is higher for high amounts of drift (+14%).

Half-timbered walls 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% respectively when compared to the equivalent unreinforced wall. Half-timbered walls retrofitted with NSM steel flat bars increased the total dissipated energy by 25% and 13% respectively for the lower and higher vertical load level, while timber frame walls increased by 161% and 52% respectively.
Even though timber frame walls showed fuller loops during the tests, with smoother unloading branches and a more noticeable shear behaviour, this did not always translate in a great increase of their dissipative capacity (see Figure 5.29). This is mainly due to the shape of the loops for each type of wall and of strengthening (Figure 5.30a,b). In fact, for traditional strengthening (i.e. bolts and steel plates) and for both load cases, timber frame walls, though having fuller loops, presented a much higher pinching than infill walls, effectively decreasing the dissipated energy. This difference in the level of pinching was encountered even in unreinforced walls, as seen in Chapter 4.

For the innovative retrofitting solutions (i.e. NSM steel flat bars) it is seen that for half-timbered walls there is a trend for the decrease of the dissipated energy. This is particularly evident in case of the walls with the lower pre-compression load level due to the collapse of the bottom connection and the further development of flexural rocking behaviour, being the hysteresis loops close to an S shape, as previously seen. In case of timber frame walls, whose hysteresis loops are characterized by smoother unloading branches and a more noticeable shear behaviour (Figure 5.30), it appears that there is an increase on the dissipated energy, particularly in case of the wall submitted to the lower value of pre-compression load. In fact, in spite of the fact that pinching behaviour also characterizes the hysteresis loops of these walls, it seems that the increase of the energy dissipated due to the additional yielding of the NSM flat steel bars is able to counterbalance it.

![Figure 5.30 - Typical cycle of tested walls for a given displacement of 70mm: (a) lower vertical load level; (b) higher vertical load level](image)

The ratio between dissipated and input energy, both per cycle and the cumulative values (Figure 5.31a,b) are higher than the values observed for unreinforced walls (see Chapter 4).

The ratio is particularly high for low values of drift, due to the displacement being recorded on the opposite side of that of load application, causing a loss of initial displacement, and then tends to reach an almost constant value.
Chapter 5

Figure 5.31 - Ratio between dissipated and input energy: (a) values for each cycle, lower vertical load level; (b) values for each cycle, higher vertical load level; (c) ratio of cumulative values, lower vertical load level; (d) ratio of cumulative values, higher vertical load level

For all walls, a higher vertical pre-compression leads to higher values of energy dissipation in relation to the input energy, when considering each cycle individually (Figure 5.31a,b). The same trend is not apparent when the cumulative values are taken into account (Figure 5.31c,d), except for strengthening done with bolts. It is possible to notice three phases in the diagrams. A first phase, up to 1% of lateral drift, during which the walls do not experience any damage; a second phase, in which the values of energy dissipation in relation to the input energy are decreasing to an almost constant level, when the wall is experiencing some rocking and the first damages appear; a third phase, where the ration increases again as the walls experience heavy damage and are able to dissipate greater amounts of energy. This last phase is not apparent in wall RIW25_P, as the strengthening was effectively not contributing to the resistance and the wall behaved as an unreinforced one.

For all walls, the ratio is particularly high for low values of drift, due to the lesser difference of the lateral displacement between both sides of the walls (opposite side to the application of the load and side corresponding to the application of loading), resulting from more reduced initial adjustments at the connections. It is seen that after the peak value of the input and dissipated energy ratio, the behaviour of retrofitted half-timbered and timber frame walls is
Experimental program: cyclic tests on retrofitted walls

slightly different. There is not a clear trend in the dependency on vertical pre-compression and type of wall, as for half-timbered walls the values are similar, while for timber frame walls it would appear that the ratio is higher for the wall tested with the lower pre-compression.

5.6.4 Equivalent Viscous Damping Ratio (EVDR)

Comparing the results of equivalent viscous damping for the walls tested (Figure 5.32a,b), 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 presents 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 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 encountered for cyclic tests on bird’s mouth connections (Branco, 2008). This type of connections strengthened with bolts presents a value of EVDR of 0.11, while the connections strengthened with stirrups present a value of 0.15.

For walls tested with NSM steel flat bars, for both load levels, the values of EVDR were higher for higher values of drift, particularly for the higher pre-compression level. Half-timbered walls presented values of EVDR of approximately 0.12, while timber frame walls varied between 0.11 and 0.13, with higher values for the lower pre-compression. These values corresponds to an increase of the equivalent viscous damping of the retrofitted walls of about 20% for half-timbered walls, while the increase for timber frame walls was approximately 18% for the lower vertical load level, while for the higher one the values were
almost unaltered. It has to be noticed that for unreinforced walls there was a greater loss in terms of EVDR for higher values of lateral drift. This was due to damage propagation, as it was higher for unreinforced walls, while retrofitted walls encountered lighter damage.

Additionally, comparing the retrofitted and unreinforced walls, it is seen that the retrofitted ones present a lower variation and almost no decrease was recorded after a lateral drift of 1.5% as happens in unreinforced on the equivalent viscous damping, which should be associated to the more rational distribution of damage. The exception is wall RIW25_S, since the retrofitting failed and therefore the wall reached values of EVDR of the equivalent unreinforced wall. Nonetheless, before failure the wall had reached a stabilized value of EVDR, similarly to the other walls retrofitted with the same technique.

Comparing to similar tests conducted on strengthened concrete block masonry (Haach et al., 2010), where EVDR is increasing for high values of drift, in the case of timber frame walls the values tend to decrease or reach a constant value. Only walls strengthened with bolts encountered an increase in values when compared to the initial ones. This behaviour is due to pinching, which characterizes both timber frame and, in a smaller scale, half-timbered walls, reducing the dissipative capacity of the walls.

5.6.5 Brief comparison of the seismic performance indexes among distinct retrofitting techniques

The results of half-timbered and timber frame walls retrofitted with different techniques can be compared among them to grasp the advantages and disadvantages of each technique.

In terms of maximum values (see Table 5.9), apart from retrofitting performed with bolts, which showed from little to any increase in terms of maximum load capacity, improving only the post peak behaviour of the walls, generally steel plates and NSM steel flat bars retrofitting techniques tended to play a major role in the lateral resistance of the walls submitted to the lower vertical load, reaching an increase in terms of maximum load capacity up to almost 200%. In turn, for the higher vertical load, the increase was not greater than 70%. This phenomenon led to a greater similarity in the behaviour of the wall between the two vertical load levels for each kind of strengthening. For all kind of strengthening, the loss in terms of ultimate displacement (considering that the applied displacement was the same as that which was applied in the unreinforced walls) was usually in the order of 3% or less, apart from the walls which presented out-of-plane problems.

Concerning the deformation, the connections retrofitted with NSM steel flat bars were opening less than the walls with other retrofitting solutions for the same level of lateral load. Moreover, for the same deformation level, the damage in the walls was considerably less. The stiffening effect of this retrofitting technique can be also observed through the
deformations recorded in the single connections, when compared to the ones recorded in unreinforced walls. If for strengthening done with steel plates the connections recorded significant opening, even if the steel plates were effectively confining the connection, for the NSM retrofitting technique significant displacements were registered only after complete failure of a connection. The main connections were more rigid, thus did not allow as much movement.

Moreover, though strengthening with NSM steel flat bars resulted in an initial stiffness similar to that given from steel plates, being slightly higher for this innovative retrofitting system in timber frame walls, see Table 5.9, due to the stiffening effect given to the main half-lap connections the walls did not experience the same out-of-plane mechanism of the walls strengthened with steel plates since the deformation capacity of the wall was higher, due to the capacity of the diagonal elements to detach from the main frame.

While it is not possible to compare the dissipative capacity of walls retrofitted with steel plates between timber frame and half-timbered walls, since the former experienced problems during the test, it can be seen that for timber frame walls with steel plates without linking the diagonals it notably increased and it is comparable to that of timber frame walls retrofitted

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<th>VERT LOAD LEVEL [kN/post]</th>
<th>PARAMETER</th>
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<th>TIMBER FRAME WALLS</th>
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<td>STEEL PLATES</td>
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<td>EVDR</td>
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</tr>
</tbody>
</table>

* It was considered the value before total failure of bottom connections, which led to the wall behaving as an unreinforced wall.

Table 5.9 - Comparison among values of seismic parameters for unreinforced and retrofitted walls

172
with NSM steel flat bars, the difference being of approximately 10% for both load levels, indicating that the two solutions are very similar in terms of results.

The strengthening with steel flat bars gave a lower dissipative capacity than other types of strengthening for half-timbered walls, but this could be attributed to the fact that the infill was hindering the effect of the bars, which could not deform freely. On the other hand, this technique guaranteed a slightly higher dissipative capacity for the timber frame walls, particularly so for the lower pre-compression level, even when compared to other retrofitting solutions. It is believed that the energy dissipation performance could be higher for this retrofitting technique when compared to the others. This should be associated to the fact that the total displacement capacity of the walls were not exploited as seen previously, which resulted in lower damages for similar displacement levels. It is supposed that by imposing larger lateral displacement to the retrofitted walls, higher damage could develop leading to higher values of cumulative dissipated energy.

Considering the values of EVDR, the values obtained for the retrofitted walls are similar for all retrofitting solutions, pointing out that an innovative retrofitting technique could be a comparable alternative to a traditional one when approaching a strengthening problem.

Lastly, a note should be made concerning the cost of the different retrofitting techniques. While it is clear that retrofitting done with bolts is the cheapest solution (approximately 12€ per wall) and the easiest and less time-consuming to implement, the cost of retrofitting done with steel plates amounts to approximately 130€ per wall and it takes one day to retrofit one wall; lastly, retrofitting performed with NSM steel flat bars costs about 100€ per wall, therefore less than steel plates, but it takes 8 days to retrofit one wall (1 day for opening the slots and 7 days to apply the glue and let it dry). Moreover, opening the slots requires a precise workmanship.

Therefore, when choosing which retrofitting technique should be used, one should also consider costs and the time required to perform the retrofitting.

5.7 Conclusions

Aiming at gathering a better insight on the influence of distinct retrofitting techniques applied on timber frame walls on the lateral behaviour of timber frame walls, characteristic of ancient construction in Portugal, and on the improvement of their seismic performance, an experimental campaign was designed based on static cyclic tests. Distinct parameters were considered, namely typology of the wall and vertical pre-compression load. Three distinct retrofitting solutions were adopted: (1) bolts, (2) steel plates and (3) NSM steel flat bars. In
case of steel plates, two distinct geometrical configurations were adopted. Besides, two vertical pre-compression levels were considered for each wall type.

From the detailed analysis of the experimental results it is possible to conclude that:

- The presence of infill still influences the behaviour of the retrofitted walls, but not in the same level as in unreinforced tests, since the retrofitting solutions play a predominant role on the lateral behaviour and hinder in certain extent the influence of other factors, such as the vertical pre-compression.

- The increase on the vertical pre-compression load does not overmuch influence the behaviour of the retrofitted walls, mainly for the solutions that significantly change the stiffness of the walls and particularly for timber frame walls. A dependency on the vertical load was observed only for the simplest and less intrusive retrofitting technique with bolts. For the other techniques, walls reached similar values of load, displacement and stiffness independently of the vertical load. Infill walls retrofitted with NSM steel flat bars showed a dependency on the vertical load, since the confinement given by the infill hindered the efficiency of the NSM retrofitting solution.

- The retrofitting with bolts improved the behaviour of the walls in the sense that it allowed to exploit the full capacity of the connections, changing the failure mechanisms and improving mainly the dissipative features of the walls, particularly for increasing lateral drifts. Additionally, it should be mentioned that this technique is the cheapest and the less intrusive when compared to steel plates. In any case, all retrofitted techniques have the advantage of being reversible.

- The retrofitting technique with steel plates greatly increased the stiffness of the walls, particularly when the diagonal elements were linked to the main frame. Besides, the use of this retrofitting technique led to an important increase on the lateral resistance of the wall. The increase on the compression loads of the diagonal elements results in a trend for these walls to show some out-of-plane instability. This can indicate that this retrofitting technique can be redesigned to be applied in certain key connections instead of applying it in all the connections. In any case, the solution proved to be efficient for
infill walls and for timber frame walls with the alternative solution. The steel plates were able to guarantee a good behaviour of the walls even after peak stress.

- The NMS steel flat bars retrofitting solution proved to be more appropriate for timber frame walls, since the deformation of the timber elements was not hindered by the infill and the exploitation of the flat bars was greater. Nonetheless, it revealed to be efficient even for infill walls. Moreover, it can be considered as an invisible intervention, which can be of particular interest for historic structures, if aesthetics are an issue.

- The level of energy dissipation by using NSM technique is similar for half-timbered walls and slightly higher in case of timber frame walls, when compared to other retrofitting techniques. It should be noticed that the level of damage found for the walls retrofitted with this solution is lower, which should be associated to the possible further exploitation of the lateral displacements. Notice that, contrarily to the walls retrofitted with steel plates, the walls retrofitted with NSM technique did not experience significant out-of-plane displacements.

- There is a trend for the retrofitted walls techniques to present a decrease of ductility for all walls. However, it should be stressed that but for a great number of specimens the ultimate displacement capacity of the walls was not reached, indicating that it could be possible to obtain higher results.

- All retrofitting techniques ensured a higher dissipative capacity for the walls, with similar values for half-timbered and timber frame walls. The higher dissipative character of the retrofitted walls is revealed by (1) energy dissipated at each cycle; (2) the cumulative energy dissipated for a given lateral drift and (3) equivalent viscous damping.

- Based on the simplicity of the retrofitting technology and on results obtained, it should be stressed that NSM retrofitting technique revealed in fact as a valid alternative to more traditional retrofitting solutions with the advantage of diminishing the visual impact.
Experimental program: cyclic tests on retrofitted walls
6.1 Introduction

As it can be easily seen from the experimental campaign presented in Chapter 4 and in Chapter 5, the characterization of half-timbered structures depends essentially on the resistance of the connections, since they represent the crucial point of the structure, where stresses accumulate. Concerning the global rigidity of the structure it is fundamental to understand how the connections work, in particular the allowed movement of the various parts. The possibility of having relative displacements and rotations among the elements of the wall contributes to confer ductility to the timber structure. Additionally, the connections with the horizontal diaphragms are equally important. Moreover, for structural systems where half-timbered walls are adopted together with load bearing masonry walls, it is also important to analyse the connection between the two types of walls.

The behaviour of the connections depends on the material properties, the quality of their execution (clearances in notches could alter the response of the connection) and, for existing structures, on the decay of wood.

Commonly, traditional timber connections are assumed to be rigid or hinged. As they have significant rotational stiffness, even when unreinforced, they could be better classified as semi-rigid and friction-based (Branco, 2008) instead of hinged. Traditional timber connections depend greatly on compression and friction among their elements. Due to the production process, i.e. the manual work of the carpenter, there can be some irregularities.
and clearances which influence their performance, therefore commonly the contact between the members is improved by strengthening the connections with metal elements. Traditional timber connections rely mainly on notches, wedges, bearing faces, mortises, tenons and pegs, while metal fasteners are less present, though nails can be inserted to improve the connection. On the other hand, metal fasteners constitute an important tool in rehabilitation works.

Mechanical connections are defined as those where fasteners penetrate the wood and are usually divided into two categories: dowel and bearing (Soltis, 1996). Dowel type fasteners, such as nails, screws and bolts, transmit either lateral load, by bearing stresses developed between the fasteners and the elements of the connections, or withdrawal load, which are axial loads parallel to the fastener axis transmitted through friction or bearing to the connected materials. Metal connector plates combine lateral load action of dowel fasteners and the strength properties of the metal plates (Soltis, 1996). Bearing type connections transmit only lateral loads, for example shear plates that transmit only shear forces through bearing on the connected materials (Soltis, 1996).

In literature, it is possible to find experimental results on traditional timber connections. Various studies are available on bird’s mouth connections, typically used for roofs (Branco, 2008; Parisi and Piazza, 2002) and on mortise and tenon connections regarding pull-out, bending and shear tests (Shanks and Walker, 2005; Descamps et al., 2006; Koch et al., 2013), as well as on dowel-type connections (Xu et al., 2009). Moreover, studies exist on the characterization of specific traditional connections, for example on the rotational performance of traditional Nuki joints in Taiwan (Chang et al., 2006), rotational and pull-out tests on historic Dou–Gon joints in Taiwan (D’Ayala and Tsai, 2008), tensile tests of Japanese Kama Tsugi and Okkake DaisenTsugi joints, which are traditional notched joints (Ukyo et al., 2008), pull-out tests on steel-wood nail connections used in timber structures in Haiti (Vieux-Champagne et al., 2012). Due to the very great variability that exists as far as connections type is concerned, usually specific tests have to be carried out for each type of connection.

In this Chapter, the behaviour of traditional connections adopted in Portuguese half-timbered walls is analysed. In particular, in-plane cyclic and pull-out tests were carried out on representative joints and results are here presented in terms of strength and displacement capacity, stiffness, energy dissipation capacity and viscous damping. Moreover, retrofitting solutions are analysed, considering those used in the walls previously tested as well as new proposals.
Chapter 6

6.2 Experimental campaign on unreinforced traditional connections

An experimental campaign on traditional connections used in Portuguese timber frame walls was carried out in order to better study their behaviour, since they are the key elements of the walls. In order to do this, a significant connection of the wall tested was selected. The lateral bottom half-lap joint was adopted, since it was seen that, for unreinforced walls, it was governing the behaviour of the walls, as it was preponderant for the rocking movement of the wall.

The evaluation of the deformation patterns and damage progress assists in the further understanding of the mechanical behaviour and in the selection of the most appropriate retrofitting solutions for traditional timber frame walls. As already mentioned, various experimental results are available on traditional connections of other timber structural elements, such as roofs (Branco, 2008; Parisi and Piazza, 2002). However, very little information is available in literature on the behaviour of traditional connections used in timber frame walls. Therefore, the research presented here is important as a starting point for the study of such joints.

For this, 14 specimens have been tested for the mechanical characterization of the traditional connections in timber frame walls. Pull-out and in-plane static cyclic tests have been performed. In the following section, details about the geometry of the specimens, the test setup and procedure are presented. Moreover, the results of the tests will be discussed in detail.

6.2.1 Specimens

The specimen selected has the same geometry of the bottom joint in the wall. An adequate height of the post was adopted, in order to apply the load at a height that would allow to have a representative deformation of the post. It was considered to apply the load at approximately two thirds of the cell. Figure 6.1 shows the geometry of the specimen. Unfortunately, it has not been possible to take into account the effect of the diagonal on the connection during the cyclic test and the specimens were tested considering only the simple half-lap joint. The bottom beam was anchored on both sides of the connection, as done in the walls, at the same distance from the connections that was used during the wall tests.

It was decided not to take into account the influence of the masonry infill in these tests. From what was observed during the tests on the walls (see Chapter 4 and Chapter 5), infill has an important confining effect on the timber frame and adds stiffness and strength to the frame.
The non-consideration of the infill represents the most unfavourable condition, since the connection is weaker without infill.

![Figure 6.1 - Geometry of specimens tested: (a) half-lap joint and nail inserted; (b) dimensions of the connection (in cm).](image)

The specimens were built in the same period of the timber frame walls with the same type of wood, *Pinus Pinaster*. A nail was inserted in the centre of the connection, similarly to what was done in the connections of the walls (see Chapter 4). The specimens were stored in the same laboratory conditions of the walls.

### 6.2.2 Test setup, procedure and instrumentation

Two types of tests have been performed on traditional connections: (1) pull-out tests and (2) in-plane static cyclic tests. This choice is justified by the fact that, during the tests performed on the walls, there was a tendency for the bottom connections to uplift, particularly in infill walls. Therefore, it was decided to test the uplifting capacity of the connections in the unreinforced and retrofitted condition. To do this, the beam of the connection was anchored to a steel profile which was linked to the reaction floor. The post was pulled-out by means of an hydraulic actuator which was linked through a hinge at the top of the post by means of a U profile gripping the post with four 12mm rods (Figure 6.2a). Notice that, in order to prevent failure at the top gripping device, GFRP sheets were glued to strengthen the zone. The actuator used had a load capacity of 150kN and a displacement range of 200mm.

In order to compare the cyclic behaviour of the walls with the main characteristics of the mechanical behaviour of the connections, in-plane static cyclic tests have been performed on the connection, with a setup similar to that used for the walls (Figure 6.2b). The bottom beam was anchored to a steel profile which was in turn anchored to the reaction floor. A constant vertical load was applied to the post by means of a hydraulic jack with the same system of hinges and rods used for the tests on the walls (see Chapter 4). The same actuator used for the pull-out tests was placed in a horizontal position and anchored to the rigid steel frame.
linked to the reaction floor. Two 2-dimensional hinges were placed, one between the steel frame and the actuator and one between the actuator and the steel plate linking it to the specimen. This was made to allow rotations during the test. A more detailed description of both test setups can be found in Koukoviki (2013).

Similarly to what was done for the walls, the in-plane cyclic tests were performed considering two levels of vertical pre-compression, namely 25kN and 50kN.

The procedures used for these tests were adapted from standard EN 12512 (2001). For both pull-out and in-plane cyclic tests, a preliminary monotonic test was carried out in order to obtain the yield and ultimate displacement.

For the pull-out test, it was assumed as ultimate displacement 50mm. This value, obtained from the monotonic test (see Figure 6.3a), confirms what was observed experimentally in the walls, for which the maximum uplift of the posts was approximately 50mm. Since no standard is available for pull-out tests, keeping in mind this ultimate displacement, it was decided to adopt 6 steps going from 10% to 100% of the ultimate displacement. From the step corresponding to 40% of the ultimate displacement, two stabilization cycles were applied after the main one (see Figure 6.4a), in order to obtain the maximum damage for that value of displacement and stabilize the strength and stiffness degradation. This procedure was adapted keeping in mind the one adopted for the walls. In order to minimize the time demand, two displacement rates were considered for this procedure: (1) a first displacement rate of 0.15mm/s was used for displacements up to 20% of the ultimate displacement; (2) for the remaining steps, a rate of 0.3mm/s was used. For the detailed description of the applied procedure, see Koukoviki (2013).

Four pull-out tests were performed on unreinforced half-lap connections. The specimens were designated as URTx, standing for unreinforced timber connection followed by the number identifying the specimen.
Traditional connections: experimental tests on unreinforced and retrofitted connections

For in-plane cyclic tests, a yield displacement ($d_y$) of 19.9mm was obtained from the preliminary monotonic test, see Figure 6.3b, considering yield values for a load-displacement curve without two well-defined linear parts (EN 12512, 2001), assuming an initial secant stiffness and a secondary one with slope corresponding to 1/6 of the initial stiffness. Ten displacement steps were applied, from 25% up to 275% of the yield displacement, as illustrated in Figure 6.4b. Also in this case, two rates were considered: (1) a first rate of 0.1mm/s was used for displacements up to 75% of the yield displacement; (2) a rate of 0.3mm/s was used for the displacements between 75% and 275% of the yield displacement.

All specimens were monitored with linear voltage displacement transducers (LVDTs) in strategic positions in order to record the most significant deformation of the connections, as shown in Figure 6.5.

For the pull-out tests (Figure 6.5a), LVDT V1 measures the vertical uplift at the back of the connection, i.e. the separation between beam and post, LVDT OOPR measures the out-of-plane opening of the connection on the right side and LVDT OOPL measures the same opening on the right side.
In the in-plane cyclic tests (Figure 6.5b), the horizontal displacement of the connection was measured at two heights, one at the load application level (HT) and one at mid height of the post (HM). The out-of-plane opening of the connection was measured with LVDT OOP, whereas LVDTs VR and VL measured the vertical uplift of the connection on the right and left side respectively. Additionally, the uplift of the beam from the steel profile to which it was anchored was monitored.

6.2.3 Results on pull-out tests

From the results of the pull-out tests it was observed that all connections behaved in a similar way (Figure 6.6a), being the response characterized by out-of-plane opening when the post was pulled out and deformation of the nail (Figure 6.6b). The connection stopped working when the nail was not effective, as it pulled out completely from the beam.

Figure 6.7a presents a typical force-displacement diagram characterizing the pull out response of the connection. It is seen that the diagram is characterized by a high initial stiffness and an early non-linear behaviour. In the unloading branch, the connection has an immediate loss of strength and then acquires compression forces. This is associated to the reaction to the re-entering of the post to its original position in the beam, due to the plastic deformation developed in the nail. On the other hand, the reloading branches present a high amount of pinching, caused by the crushing of the wood surrounding the nail and consequent increasing clearances (see Figure 6.6d). Important strength degradation is observed during the tests. This phenomenon is not only observed between two successive steps, but also in the stabilization cycles. These two characteristics (pinching and strength degradation) were observed in the cyclic behaviour of timber frame walls without infill, as the behaviour of the connections affect the response of the wall. In timber frame walls, the unloading branch was characterized by the same high strength degradation observed in the connections.

Considering the out-of-plane opening of the connection (Figure 6.7b), it is observed that the connection remains closed as the post is being pulled (Figure 6.6c), being the existing...
clearances also eliminated. During unloading, due to the difficulty for the post to reach its original position due to the plastic deformation of the nail and its impossibility to re-enter the beam, the connection exhibits increasing opening values for increasing vertical uplift levels, and thus for a higher deformation of the nail.

![Image](a)

![Image](b)

![Image](c)

![Image](d)

Figure 6.6 - Deformational features of connections during pull-out tests: (a) uplifting of post; (b) out-of-plane opening of connection and deformation of nail when the post re-enters; (c) no out-of-plane opening occurs while the post is uplifting; (d) final deformation of nail and damage to wood.

Residual or permanent out-of-plane displacements can be observed for very high values of displacement as well as important nail deformation. At the end of the test a permanent opening was observed in the connections, being usually higher at the bottom of the post and lower at the top of the post, indicating its rotation. The values of opening varied between 20 and 45mm, depending on the level of vertical uplift reached and the level of clearances.

Even though the general behaviour of the specimens was similar, it was noticed that the maximum load and stiffness of the connection depended greatly on its level of interlocking. It was observed that for high clearances present in the connection, the load capacity of the connection decreased and the out-of-plane opening progressed more rapidly. The latter issue can be easily seen when comparing the out-of-plane opening of specimens URT3,
which had a very good interlocking, and URT2, which presented a poor interlocking (Figure 6.7b and Figure 6.7c respectively). For the same values of vertical uplift, specimen URT2 presents significantly higher values of out-of-plane opening; URT2 had a residual opening of 10mm at the step of 20mm while URT3 at the same uplift opened only 4mm.

Comparing the envelope curves of the tested specimens (Figure 6.7d), it can be noticed that specimen URT3 reached a maximum load of 5.07kN, whereas URT2 only reached 2.79kN, meaning that the resistance decreased by 45%. Moreover, the two specimens with the higher clearances (URT2 and URT4) failed earlier, since the nail pulled out of the beam at the first cycle of the step of 50mm, contrarily to what happened in the other specimens.

For all specimens, no uplifting of the beam from the steel profile was recorded.

When the flexural behaviour predominates in the lateral response, it can be concluded that the non-linear behaviour of the bottom connections influences the overall behaviour of the walls: (1) the unloading of the walls is influenced by the difficulty of the post to recover its original position due to the plastic deformation of the nail, being the unloading branch of the force-displacement diagrams characterized by a plateau; (2) the same deformational features were observed in the walls and, as the vertical uplift increased, the stiffness of the
wall decreased, stressing its rocking behaviour; (3) besides, the pinching behaviour of the walls can in a certain extent be influenced by the pinching shown by the bottom connections, as the initial loss of strength in the connections leads to their not immediate response in the walls.

6.2.4 Results on in-plane cyclic tests

The specimens behaved in a similar way for both vertical load levels. Vertical cracks were observed in the notched part of the post (Figure 6.8a), associated to shear stresses developed during the cyclic test, as the side of the notched part of the post not in compression slides and cracks. Moreover, cracks perpendicular to the grain developed at the interface between the post and the beam, as the beam cuts the post during its cyclic movement (Figure 6.8b). The level and shape of cracking depended greatly on the grain alignment and the presence of knots (Figure 6.8c,d).

![Figure 6.8 - Deformational features of in-plane cyclic tests: (a) vertical cracks in post; (b) a clear horizontal crack developed at the interface between post and beam; (c) diagonal cracking in connection; (d) cracking influenced by presence of knot.](image)

Usually, damages were greater and developed more rapidly for the connections tested with the higher pre-compression level. The nails did not deform, as the vertical uplift was minimal and they were mainly behaving as a hinge around which the post would rotate. As for the pull-out tests, the post opens in the out-of-plane direction, being the grade of opening dependent on the presence of clearances and on the vertical load level.

Figure 6.9a presents a typical hysteretic curve of a specimen with the lower pre-compression level. The connection has a linear initial response and then non-linearity appears nearer to the maximum load capacity. Strength and stiffness degradation was observed. From the comparison of the envelope curves of all tests performed (Figure 6.9b), differences in the response regarding mainly the maximum load capacity were attributed also in this case to the quality of the connection and of the material, as the presence of knots or of grain
misalignment led to lower values of load. The average maximum load is 5.53kN while the minimum -6.61kN, with variations of 19%.

![Figure 6.9 - Results of in-plane cyclic tests, lower vertical load level: (a) typical load-displacement diagram; (b) envelope curves for all specimens; (c) typical out-of-plane opening of connection; (d) typical vertical uplift of connection.](image)

The out-of-plane opening of the connection varied among the tests depending on the level of interlocking. However, the rate of opening was similar. In Figure 6.9c, where the progressive opening of the connection versus the top horizontal displacement is shown, it can be seen that the increase in the opening of the connection is almost linear and the opening becomes significant only for displacement levels higher than the yield displacement, equal to 19.9mm.

Comparing the horizontal displacement measured at the top of the connection with those recorded by LVDT HM at mid height, it can be concluded that the post maintained its linearity throughout the test, as the displacement measured at mid height was approximately half of that measured at the top of the connection.

During the test, the post uplifted from the bottom beam, accompanying the rotation, which resulted in the compression stress concentration at the opposite side to the application of the lateral load (see Figure 6.9d). Even though the cyclic test on just one connection did not represent the dual cyclic-pulling out action exercised on the connection during the test of the wall, it is still representative in terms of local damage developed in the connections of the
walls (at least at the bottom corners). Particularly, this behaviour is approximately what is observed during the wall tests in the compressed post, since they had a tendency to rotate around a point and simultaneously cracks developed.

The walls tested with the higher vertical pre-compression present a higher degree of asymmetry, mainly attributed to the asymmetry in their damage distribution, since cracking for these connections tended to be heavier on one side. From the typical hysteretic diagram (Figure 6.10a), it is observed that the connections are in the elastic range up to the maximum load, after which strength and stiffness degradation occurs. Also for this pre-compression level the results varied in terms of maximum load capacity and initial stiffness (see Figure 6.10b), depending on the quality of the connection and the presence of defects. The variation for these connections is lower than in the lower pre-compression, with an average maximum load of 6.48kN and a minimum of -5.98kN and a variation of 10% and 14% respectively.

Both the out-of-plane opening (Figure 6.10c) and the vertical uplift (Figure 6.10d) were lower than the recorded ones in the tests with the lowest vertical pre-compression, but the trend was the same. The compression stresses in the post are higher for the highest level of pre-compression, resulting in the trend for the post to penetrate into the beam. This is confirmed
by the vertical LVDTs measuring the uplift of the post that record negative values in the opposite side to the application of lateral load.

6.3 Experimental campaign on retrofitted traditional connections

After the unreinforced tests, the specimens were repaired and retrofitted in order to (1) understand the influence of the retrofitting solution on the behaviour of the connection and (2) to compare different retrofitting solutions understanding their strong points and shortcomings. The retrofitting solutions were mainly the ones adopted for the walls, but some alternate solutions were considered.

6.3.1 Retrofitting solutions adopted

Since the half-lap welded connection for the steel flat bars used in timber frame walls seemed to work well, it was decided to try to use this strengthening technique for the bottom connections. Due to the unavailability of the same steel flat bars used in the walls because of problems with the supplier, steel rods of class 8.8 with a diameter of 10mm were used instead. In order to obtain the same tensile strength of the bars, two parallel steel rods were used. During the walls tests it was seen that bending and welding the bar did not guarantee sufficient anchorage length. Therefore, it was decided to use straight rods and increase the anchorage length by welding a horizontal rod to the two vertical ones as it was done in timber frame walls (see Figure 6.11a). The slots opened had a depth of 15mm and a width of 12mm. The same structural timber glue used for the walls was adopted (MapeWood Paste 140 (MAPEI, 2002)).

Another alternative solution was to strengthen the connections with GFRP sheets. A uni-directional fibre glass fabric (MapeWrap G UNI-AX (MAPEI, 2012)) was used together with an appropriate epoxy resin (MapeWrap 31 (MAPEI, 2013)) for impregnation of the fibre with a dry system. Notice that two sheets were applied: (1) the first one linking the two non-continuous elements, i.e. a vertical sheet linking post and beam on the side where the post is discontinuous and (2) a horizontal sheet linking post and beam on the side where the beam is discontinuous. Besides, the second sheet, orthogonal to the first, was glued over it to increase the anchorage of the first sheet (Figure 6.11b). The idea is that the GFRP sheets should be able to prevent both the uplift of the post and the out-of-plane opening of the connection.

Another alternative solution was the adoption of self-tapping screws (Figure 6.11c). This solution was only adopted for pull-out tests, to evaluate its effect on the connection against uplifting forces, but not against a cyclic lateral action. Four screws were inserted in the connection, two with an angle of 60° inserted from the post and intersecting the beam and
two with an angle of 45° screwed from the beam and intercepting the notched part of the post. Each two pairs were inserted one from each side of the connection. The screws, having a length of 19cm and a diameter of 8mm, were type VGZ9200 (Rothoblaas, 2012). Notice that to perform this kind of retrofitting in a wall, the diagonals should have to be taken out or the position of the screws should be modified.

Finally, steel plates were applied in the same way as done in the walls (see Chapter 5), positioning two steel plates, one horizontally and one vertically, on each side of the wall and connecting them with bolts and screws (Figure 6.11d).

Specimens had to be repaired before applying the retrofitting. For the execution of the pull-out test, the only intervention necessary was that of taking the nail out of the post, repositioning the post inside the notched beam and then applying each strengthening.

Table 6.1 presents the nomenclature used for each specimen adopted for the pull-out tests together with the associated retrofitting technique.
Table 6.1 - Nomenclature of specimens adopted for pull-out tests.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>TYPE OF STRENGTHENING</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>GFRP</td>
</tr>
<tr>
<td>GFRP_T</td>
<td>✓</td>
</tr>
<tr>
<td>SCREWS</td>
<td></td>
</tr>
<tr>
<td>STEEL PLATE</td>
<td></td>
</tr>
<tr>
<td>NSM</td>
<td></td>
</tr>
</tbody>
</table>

For the in-plane cyclic tests, various cracks were present in the notched part of the posts and, in two occasions, total failure of that part occurred. Since applying the strengthening on such damaged part could alter the efficiency of the strengthening, it was decided to cut out the damaged part of the post and replace it with a prosthesis. It was decided not to substitute the entire post, as in an existing structure this is not always possible. To apply the prosthesis, different solutions could be adopted (Figure 6.12); for example, only the minimal damaged part of the post could have been taken out and replaced with an equivalent one by gluing it (Figure 6.12a).

![Possible solutions for adoption of prosthesis in post: (a) simply glued prosthesis, (b) prosthesis with embedded steel rods; (c) adopted prosthesis with S-shape and screws.](image)

As an alternative, and in order to improve the continuity, apart from gluing the prosthesis, steel rods could be inserted in the prosthesis, linking it to the pre-existing post (Figure 6.12b). This technique is used in restoration projects of historic structures (Tsakanika and Mouzakis, 2010). This solution, though, is highly invasive and already a strengthening per se. Moreover, it makes the application of some of the retrofitting solutions foreseen impossible, such as NSM steel rods or even steel plates, since the screws and bolts could come into contact with the rods.
The adopted solution (Figure 6.12c) consisted of a glued prosthesis, taking out a bigger part of the damaged post and creating an S shape contact connection by gluing the two pieces. Moreover, to improve the adherence, two screws were used to better link the two elements of the post. The idea was to re-establish the continuity of the post. In this way, it was possible to apply all retrofitting techniques. The timber used to build the prosthesis was the same one used for the original specimens, while the structural timber glue used was the same adopted for all other applications (MAPEI, 2002).

Notice that to assess the influence of the prosthesis on the strengthening, GFRP retrofitting was applied both to a damaged specimen (suffix Da) and to a specimen with prosthesis (suffix Pr) for the in-plane cyclic tests for each vertical load level.

Table 6.2 presents the nomenclature used for the specimens adopted for the in-plane cyclic tests together with a description of the retrofitting technique used, the vertical load applied and the type of specimens used (with prosthesis or damaged).

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>LOAD</th>
<th>TYPE OF SPECIMEN</th>
<th>TYPE OF STRENGTHENING</th>
</tr>
</thead>
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<td>Prosthesis</td>
<td>Commercial steel plates</td>
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<tr>
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<td>Damaged</td>
<td>GFRP</td>
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6.3.2 Testing procedures

For the pull-out tests, since even the simplest retrofitting solution greatly increased the stiffness and strength of the connection when compared to the unreinforced one, the cyclic procedure had to be updated. Very small initial steps were considered, starting from uplifts of 0.10mm up to 30mm. Stabilization cycles were used starting from vertical uplifts of 0.40mm. This was done in order to capture the whole non-linear behaviour of the connection, as some retrofitting solutions would fail for low values of uplift. Figure 6.13 shows the new procedure adopted for the pull-out tests.

For a detailed description of the new procedure adopted, see Koukouviki (2013).
6.3.3 Results on pull-out tests

Results on retrofitted pull-out tests demonstrated that even the simplest retrofitting technique can help in decreasing the level of uplifting of the connection. Depending on the type of strengthening, the connections showed a great increase of initial stiffness and maximum load capacity, and failure modes changed completely, sometimes being extremely brittle.

Retrofitting performed with GFRP sheets had a very high initial stiffness and reached its maximum capacity for a low value of vertical uplift (Figure 6.14a). The diagram presents two peak values: (1) the first one (15 times greater than that of the unreinforced specimen) was reached when the first failure occurred, namely the sudden debonding of the vertical sheet that was strengthening the connection on the side where the post is discontinuous (Figure 6.15a, detail 1); (2) after this point, the strengthening on the other side of the connection ensured some strength and stiffness, and it was actually able to recover some strength. Debonding of the horizontal sheet began until total failure of the fibres (Figure 6.15a, details 2 and 3). After failure, the strength of the connection was lower than the one obtained for the unreinforced specimen, indicating that this solution is viable only if the maximum strength of the connection is not reached, since its post-peak behaviour is not performing.

Strengthening executed with self-tapping screws (Figure 6.14b) was the less invasive on the connection, being able to greatly improve its strength (6 times over) and stiffness without showing a brittle failure, but actually being able to ensure a post-peak softening behaviour and therefore a great capacity to dissipate energy. For this retrofitting solution, failure was mild, since the damage was progressing throughout the test, with pulling out of the screws, causing slight damages to the beam. After the peak load the screws were responsible of grain disorganization, which eventually became wood dust (Figure 6.15b). Moreover, at the end of the test, plastic deformations of the screws were observed. Notice that this test is characterized by severe pinching, typical of dowel-type connections. This is particularly
noticeable after the peak load, since the clearances resulting from the crushing of the wood due to the contact with the nail were more relevant for increasing lateral displacements.

Considering the test performed with steel plates (Figure 6.14c), the maximum load capacity exceeded the maximum load recorded in the unreinforced tests by over 46 times. Moreover, the stiffness of the connection increased greatly and a good post-peak behaviour was observed, since the steel plates were able to ensure a good residual strength even after peak load. For this case pinching plays an important role, particularly after peak load is attained. In fact, elongations of the steel plates were observed during the test (Figure 6.15c, detail 1), meaning that the bolts inserted in the connection ovalized the hole. Similarly to what happened with GFRP strengthening, the plate on the discontinuous part of the post failed first in tension with a clearly horizontal rupture in correspondence with a line of holes (Figure 6.15c, detail 2), given that they represent the weakest point of the plate. Notice that the same type of failure was observed in the characterization in tension performed on the steel plates in Chapter 3. The plates on the other side were able to withstand the maximum uplift imposed, even though the horizontal plate applied on the beam where it was
discontinuous had actually failed (Figure 6.15d). At the end of the test, most of the screws on the post failed in shear, whereas the bolts deformed plastically (Figure 6.15).

The connection retrofitted with NSM steel rods had a very high initial stiffness (Figure 6.14d) and no deformations were observed on the steel rods. Unfortunately, it was not possible to finish the test, as a problem occurred with the control LVDT and it was decided to cancel the test on the connection in order to preserve the equipment. Nonetheless, the solution showed a good potential for it to be used in walls without failing due to insufficient anchorage length.

Concerning other displacements recorded during the tests, it was seen that the uplift of the beam from the steel profile was less than 2mm for steel plates and GFRP, and 5mm for the NSM technique due to the excessive stiffness of the strengthening. For screws strengthening an uplift of 0.15mm of the bottom beam was recorded, meaning that it was not too demanding on the connection. After failure, the GFRP connection opened in the out-of-plane direction of about 4mm, whereas no out-of-plane displacements were observed for the other connections.
If the results of the different retrofitting solutions adopted are compared (Figure 6.16), two clear trends can be seen: (1) one where a very high initial stiffness is guaranteed by the strengthening, namely GFRP and NSM and which led to a brittle failure (even if for NSM it was not reached) and (2) one where the increase in stiffness was not as severe (but still great) and a more ductile behaviour was observed in the connection. The latter was assured by self-tapping screws and steel plates, which constitute solutions characterized by a greater dissipative capacity. Brittle failure should be avoided, but it is possible that in a wall, where all connections are participating, the behaviour can be different. A further investigation would be required.

6.3.4 Results on in-plane cyclic tests

The analysis of the results obtained in the in-plane cyclic tests on the connections show that their mechanical behaviour is greatly influenced by the condition of the prosthesis. It is observed that for example in the strengthening performed with GFRP, the results varied greatly between the damaged connection and the one with the prosthesis. For both vertical load levels, the damaged specimens were not even able to ensure the restoring of the initial condition of the connection. In Figure 6.17a,b the results are compared to the unreinforced test performed on the damaged connection and to the average results of the unreinforced tests (monotonic envelopes CYC25 and CYC50). Notice that the load and stiffness were always lower than those of the unreinforced tests.

Considering the damages of retrofitted specimens, it is observed that the same patterns develop when compared to unreinforced specimens (Figure 6.15a). The failure of the sheets occurs in the direction perpendicular to the fibres at the height of the previous damage already developed in the wood. It has to be pointed out that, due to limited availability, the glass fibre sheets used were uni-directional. It is possible that, by applying multi-directional GFRP sheets, failure could have been prevented or at least postponed. The same type of failure was observed for both vertical pre-compression levels.
In average, the strengthening applied to specimens with prosthesis was able to increase both initial stiffness and maximum load capacity for both vertical pre-compression levels by 43% and 52% respectively. However, this behaviour was observed only up to a certain value of drift, after which the prosthesis failed, the lower part of the post remained vertical, whereas the upper part was rocking around the screws (Figure 6.18b). This occurred for both load levels and pointed out the weakness of the prosthesis adopted. In fact, it presented a lower stiffness than that of the retrofitting solution applied and was not able to promote the continuity of the post.

Analysing the vertical uplift of the post, it is possible to understand when the prosthesis stopped working (Figure 6.19a). For the retrofitting applied in the damaged specimens without considering the prosthesis, it is seen that the vertical uplift increased and was more significant than what observed during the unreinforced tests. On the other hand, the specimens with prosthesis had a slightly lower uplift when the post was continuous, but when the prosthesis started failing the uplift decreased until reaching zero values, meaning that the lower part of the post did not present any movement.
In relation to the displacement recorded at mid height of the post (and for which LVDT HM was in contact with the prosthesis), it was observed that after the failure of the prosthesis, almost no displacement was recorded since only the upper part of the post was moving at the end of the test. The mid height displacement on the damaged retrofitted specimen was linear and half of the displacement measured at the top (Figure 6.19b).

In case of retrofitting with steel plates submitted to lowest pre-compression load level an increase of 40% was observed for the maximum load (Figure 6.17c). When the prosthesis failed (Figure 6.18c) the upper part of the post was simply rocking. However, for the higher level of pre-compression, this trend was not observed. In this case the post bended and deformations related to buckling were also observed in the steel plate (Figure 6.18c). Moreover, cracks opened laterally on the post, similarly to what happened in the unreinforced tests, but the steel plate was still able to promote a higher dissipative capacity. Analysing the vertical uplift (Figure 6.19c) and displacement at mid height for both vertical load levels, it is seen that the uplift is clearly higher in the specimens with the highest vertical pre-compression. In case of the specimen submitted to the lowest vertical load the uplifts are
close to zero, confirming the low effectiveness of the prosthesis in the latter case. When steel plates were used and the highest vertical load level was applied, the whole post experienced deformation and important rotation at the bottom was observed. There was an increase in lateral load of 21% and even stiffness increased, though in a lower degree.

Given this behaviour, it was decided to test the specimens retrofitted with NSM steel rods directly with the higher vertical load. Nevertheless for this kind of strengthening the prosthesis failed even for the higher pre-compression level (Figure 6.17d). After experiencing an increase in load of 34%, the lower part of the post once again stopped contributing for the resisting mechanism (Figure 6.18d, detail 1). This was probably due to the fact that NSM retrofitting stiffens the post more and the high difference in stiffness between the two parts of the post caused the prosthesis to fail even for the higher vertical load.

Aiming at preventing this undesirable behaviour, commercial steel plates were screwed laterally to the post, linking the two parts in order to guarantee continuity. With this procedure, a better continuity was obtained, even if the post was still not monolithic (Figure 6.18d). Comparing the hysteretic diagram with the previous one (Figure 6.17d), the
maximum load increased by 45% in the positive direction and by 24% in the negative one, whereas the initial stiffness remained approximately the same, since the addition of the steel plates influenced only the continuity of the post, but not the stiffness of the connection. Some vertical cracks were observed between the rods, but the opening did not occur at the glue interface, but in timber. The uplift of the post was similar to what observed for the steel plates strengthening (Figure 6.19d).

Comparing the different retrofitting solutions adopted (Figure 6.20), uni-axial GFRP on damaged specimens was not able to guarantee any strengthening to the connection. On healthy connections steel plates and GFRP sheets could improve significantly the performance of the connection, even if the prosthesis influenced and reduced the efficiency of the strengthening. For the higher vertical pre-compression level (Figure 6.20b), results are clearer and it appears that GFRP and NSM are able to guarantee the highest stiffness while the connection with steel plates experienced the less degradation in terms of strength.

6.4 Seismic parameters of traditional connections

For the tested connections it was decided to evaluate the same seismic parameters calculated for the walls, namely initial and cyclic stiffness, energy dissipation and equivalent viscous damping ratio. A comparison between unreinforced and retrofitted specimens for both types of tests is also presented in this section.

6.4.1 Initial and cyclic stiffness

Initial and cyclic stiffness were already defined in Chapter 4. In this section the normal and lateral stiffness are calculated based on the pull-out and cyclic tests.

6.4.1.1 Pull-out tests

All unreinforced connections presented low values of initial stiffness ($K_i$), see Table 6.3. Additionally, it is observed some variation among the results, being the average value of
stiffness of 3.94kN/mm with a coefficient of variation (C.O.V.) of 24%. As already mentioned, this is due to the level of interlocking in the connection, since some specimens presented important clearances, which depends greatly on the workmanship of the carpenter.

All retrofitted specimens presented very high values of initial stiffness. With this respect, it should be noticed that the specimen strengthened with the NSM technique cannot be considered as the final one, since the maximum load was not reached. Steel plates presented the lowest value of initial stiffness (Table 6.3) followed by self-tapping screws.

Table 6.3 - Values of initial stiffness for pull-out tests.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$K_{in}$ [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>URT1</td>
<td>3.32</td>
</tr>
<tr>
<td>URT2</td>
<td>3.40</td>
</tr>
<tr>
<td>URT3</td>
<td>5.31</td>
</tr>
<tr>
<td>URT4</td>
<td>3.72</td>
</tr>
<tr>
<td><strong>average</strong></td>
<td><strong>3.94</strong></td>
</tr>
<tr>
<td><strong>C.O.V.</strong></td>
<td><strong>24%</strong></td>
</tr>
<tr>
<td>GFRP_T</td>
<td>128.53</td>
</tr>
<tr>
<td>SCREW</td>
<td>67.96</td>
</tr>
<tr>
<td>STEEL PLATE</td>
<td>14.96</td>
</tr>
<tr>
<td>NSM*</td>
<td>165.74</td>
</tr>
</tbody>
</table>

* test did not reach failure

Self-tapping screws are driven into the timber elements and they tighten the connection, since they create their own precisely fitted hole, therefore possible clearances between the post and the beam, which influence the vertical uplift, are eliminated and the contact of the horizontal interface is improved, as well as the friction of the vertical interfaces. In the case of strengthening with steel plates holes have to be drilled to accommodate the bolt, therefore small clearances could be created decreasing the initial stiffness of the connection. This is one of the advantages of self-tapping screws, since they allow a direct entrance of the element ensuring a perfect adherence to the material. The strengthening with FRP sheets led to a high initial stiffness, but its failure was quite brittle.

As far as stiffness degradation is concerned, all unreinforced connections (Figure 6.21a) presented a similar trend. In this case, specimens with higher clearances exhibited the highest degradation in stiffness, pointing out the importance of a good interlocking in the connection.

Apart for the specimen retrofitted with steel plates, all the other retrofitted connections had a similar trend in terms of strength degradation (Figure 6.21b). Even though the specimen retrofitted with steel plates presented the lower initial stiffness, its degradation was the
slowest and at the end of the test its residual stiffness was almost the double of the solution adopting self-tapping screws.

Figure 6.21 - Stiffness degradation found for pull-out tests: (a) unreinforced specimens; (b) retrofitted specimens.

### 6.4.1.2 In-plane cyclic tests

Connections subjected to in-plane cyclic tests presented lower variations in terms of cyclic stiffness for the unreinforced specimens (Table 6.4 and Table 6.5), even if for some specimens an asymmetry existed between the two directions, mainly attributed to the asymmetry in damage patterns. Moreover, similar values of stiffness were obtained for both vertical load levels, contrarily to what observed in timber frame walls, for which a higher vertical pre-compression resulted into a higher initial stiffness.

**Table 6.4 - Values of initial stiffness of unreinforced specimens during in-plane cyclic tests, lower vertical load level.**

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>DIRECTION</th>
<th>$K_{in}$ [kN/mm]</th>
<th>$K_{in,mean}$ [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1_25</td>
<td>+</td>
<td>0.29</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.33</td>
<td></td>
</tr>
<tr>
<td>C3_25</td>
<td>+</td>
<td>0.26</td>
<td>0.27</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.29</td>
<td></td>
</tr>
<tr>
<td>C4_25</td>
<td>+</td>
<td>0.44</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>C5_25</td>
<td>+</td>
<td>0.31</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.39</td>
<td></td>
</tr>
<tr>
<td>average</td>
<td>+</td>
<td>0.33</td>
<td>0.33</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.32</td>
<td></td>
</tr>
<tr>
<td>C.O.V.</td>
<td>+</td>
<td>24%</td>
<td>13%</td>
</tr>
</tbody>
</table>

All types of strengthening increased the values of initial stiffness of the connections with the exception of already damaged specimens (Table 6.6). The strengthening with GFRP increased the initial stiffness by 45% and 44% for the lower and higher vertical load, while
steel plates increased the initial stiffness by 103% and 58% for the lower and higher vertical load respectively. NSM retrofitting increased the value of initial stiffness by 75%.

Table 6.5 - Values of initial stiffness of unreinforced specimens during in-plane cyclic tests, higher vertical load level.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>DIRECTION</th>
<th>$K_{in}$ [kN/mm]</th>
<th>$K_{in,mean}$ [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>C2_50</td>
<td>+</td>
<td>0.36</td>
<td>0.36</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.37</td>
<td>0.36</td>
</tr>
<tr>
<td>C6_50</td>
<td>+</td>
<td>0.31</td>
<td>0.39</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>C7_50</td>
<td>+</td>
<td>0.36</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.34</td>
<td></td>
</tr>
<tr>
<td>C8_50</td>
<td>+</td>
<td>0.27</td>
<td>0.25</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.22</td>
<td></td>
</tr>
<tr>
<td>average</td>
<td>+</td>
<td>0.33</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.35</td>
<td>0.35</td>
</tr>
<tr>
<td>C.O.V.</td>
<td>+</td>
<td>14%</td>
<td>19%</td>
</tr>
</tbody>
</table>

Table 6.6 - Values of initial stiffness of retrofitted specimens during in-plane cyclic tests.

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>DIRECTION</th>
<th>$K_{in}$ [kN/mm]</th>
<th>$K_{in,mean}$ [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>GFRP_T_Da_25</td>
<td>+</td>
<td>0.34</td>
<td>0.35</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>GFRP_T_Da_50</td>
<td>+</td>
<td>0.35</td>
<td>0.31</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>GFRP_T_Pr_25</td>
<td>+</td>
<td>0.48</td>
<td>0.48</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.48</td>
<td></td>
</tr>
<tr>
<td>GFRP_T_Pr_50</td>
<td>+</td>
<td>0.55</td>
<td>0.49</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.43</td>
<td></td>
</tr>
<tr>
<td>StPl_Pr_25</td>
<td>+</td>
<td>0.69</td>
<td>0.67</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.65</td>
<td></td>
</tr>
<tr>
<td>StPl_Pr_50</td>
<td>+</td>
<td>0.45</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.62</td>
<td></td>
</tr>
<tr>
<td>NSM_Pr_50</td>
<td>+</td>
<td>0.62</td>
<td>0.59</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>0.56</td>
<td></td>
</tr>
<tr>
<td>NSM_StPl_50</td>
<td>+</td>
<td>0.68</td>
<td>0.60</td>
</tr>
</tbody>
</table>

Taking into consideration the cyclic stiffness degradation, the unreinforced specimens presented a similar trend in stiffness degradation: (1) for the lower vertical load, the specimens with the highest strength and initial stiffness, namely CYC01_25 and CYC04_25, had the lower degradation (Figure 6.22a), stabilizing at values of cyclic stiffness of
approximately 0.1kN/mm. The other two specimens had a higher level of degradation and they both stabilized at a value of cyclic stiffness of approximately 0.03kN/mm; (2) for the unreinforced walls tested with the higher vertical load level, the same trend was observed, as connections CYC06_50 and CYC08_50 presented the lower degradation of stiffness (Figure 6.22b), even if in this case the final values of cyclic stiffness were nearer, between 0.5 and 0.8kN/mm.

The stiffness degradation of retrofitted connections was heavily influenced by the prosthesis and by the previous damage. For the lower vertical load level, the damaged connections had the same cyclic stiffness of the unreinforced specimen throughout the test (Figure 6.22c). Only the steel plates retrofitting was able to increase the values of cyclic stiffness, even if once the prosthesis failed, the stiffness became similar to the one recorded in the unreinforced specimen. The GFRP strengthening applied on a specimen with prosthesis increased the values of cyclic stiffness only slightly prior to the failure of the prosthesis.

For the higher vertical load level, only the damaged connection (GFRP_T_Da_50) showed significantly lower values of cyclic stiffness when compared to the unreinforced test (Figure 6.22d).
6.22d). All other solutions had an increase of cyclic stiffness, even if it quickly degraded as the prosthesis became ineffective. The specimen retrofitted with NSM rods and additional steel plates to make the prosthesis effective, showed higher values of cyclic stiffness and a higher residual stiffness indicating that, with an appropriate prosthesis or even in an undamaged connection, the retrofitting solution should behave appropriately. The other connection where the prosthesis partially worked (steel plates) showed a similar slope of degradation. For all connections, the degradation trend was approximately linear.

6.4.2 Energy dissipation and equivalent viscous damping ratio

The dissipative capacity of the connections was also analysed. This issue is of great importance since the walls dissipate energy mainly through their connections.

6.4.2.1 Pull-out tests

The dissipated energy during pull-out tests carried out on unreinforced specimens was minimal, especially for connections with an inadequate interlocking, meaning that friction does not play a role in the dissipation of energy. Figure 6.23a shows the cumulative energy dissipated by unreinforced specimens. Notice that specimens URT1 and URT3 had values of dissipative energy more than two times higher than the other connections.

![Figure 6.23](image)

Figure 6.23 - Cumulative dissipated energy during pull-out tests: (a) unreinforced specimens; (b) retrofitted specimens.

The dissipated energy increases considerably once strengthening against uplifting is applied (Figure 6.23b). The stiffer solutions highly increased the dissipative capacity of the connections. The more dissipative strengthening technique consists of the steel plates solution, which increases the energy dissipation by over 8 times. It is not possible to give indications on NSM retrofitting, since it was not possible to complete the test. Retrofitting with screws was able to promote a good dissipative capacity of the connection, with an increase of 175% in relation to unreinforced specimen. The strengthening with GFRP leads to a good dissipative capacity for low values of uplift, but after failure the energy dissipation is lower.
than the value found for the equivalent unreinforced connection. Therefore, it is evident that brittle failures should be avoided, since the strengthening becomes inefficient.

Similar conclusions can be drawn on the equivalent viscous damping ratio measured for the connections (Figure 6.24). Values of damping of unreinforced connections are influenced by their level of interlocking, while for retrofitted connections pinching plays an important role. In fact, connections retrofitted with screws presented the higher values of damping, as the effect of pinching was less severe than what observed for example in case of retrofitting with steel plates, for which energy dissipation was diminished by pinching, decreasing its ratio with input energy.

The retrofitting with GFRP sheets presented the lower values of damping ratio (the last two values of drift were not considered since the connection had already failed and the only energy dissipation was in compression).

Notice that retrofitted connections tended to have higher values of damping than the ones in the wall tests, particularly for retrofitting performed with screws. It is however difficult to associate these results to walls, since the dissipative capacity of the walls depends on all connections.

6.4.2.2 In-plane cyclic tests

In case of the connections tested under in-plane cyclic lateral loading the unreinforced specimens with higher strength were able to dissipate more energy (Figure 6.25a,b). As was observed for the values of cyclic stiffness, this difference is more evident in the lower pre-compression level tests, also due to the higher variation in terms of results for this load level. In general, specimens tested with the higher pre-compression level were able to dissipate more energy, in average 29% more.
Considering the retrofitted connections, as was also observed for the cyclic stiffness degradation, specimens which were already damaged dissipated less energy than the unreinforced ones for both load levels, whereas energy dissipation was greater for the specimens where the prosthesis was more efficient (Figure 6.25c,d).

For unreinforced specimens there is not a clear trend in the viscous values (Figure 6.26a,b). For both load cases, values are higher for connections which after failure present a clear rocking behaviour, while connections with a marked softening behaviour have lower values of damping.

Even retrofitted connections presented peculiar results (Figure 6.26c,d), since the rocking behaviour of the upper part of the post led to high values of viscous damping. These values, though, are not really representative, since the failure obtained was not that of the connection per se, but of the prosthesis, which should be avoided.

Contrarily to the walls, the connection presents increasing values of the viscous damping ration.
Traditional connections: experimental tests on unreinforced and retrofitted connections

Figure 6.26 - Equivalent viscous damping ratio during in-plane cyclic tests: (a) unreinforced specimens, lower vertical pre-compression; (b) unreinforced specimens, higher vertical pre-compression; (c) retrofitted specimens, lower vertical pre-compression; (d) retrofitted specimens, higher vertical pre-compression.

6.5 Comparison between wall and joint behaviour

It is difficult to correlate the results obtained on the connections to the global behaviour of a wall, since it is comprised of various joints of different type interacting with each other. Notice that the tested connection is only representative of the bottom connections of the walls. The central connections should also play an important role on the overall behaviour of the walls, particularly in case of timber frame walls, where the flexural rocking behaviour is not as relevant as in the case of timber frame walls with masonry infill. However, some observations can be made.

One of the first things that can be pointed out is the similarity on the crack patterns of the bottom connection of the timber frame walls and the tested connection. In timber frame walls vertical cracks appear on the posts at the bottom connections (Figure 6.27a), leading up to the nail, even if in this case there is even the influence of the diagonal, that causes some diagonal cracking.
Some similarities are also observed between the damages found in the connection of the walls and the tested connection when steel plates are applied. In both cases cracks appeared laterally on the posts of the walls (Figure 6.27b), even if in the wall there was the additional shear action of the diagonal with horizontal crushing of the connection.

Pull-out tests instead can be compared to the cyclic tests on timber frame infill walls, where the post in tension is subjected to important uplifting. In this case, when the post is in tension, the diagonal does not influence the uplifting. There is however the lateral component of the movement of the post. In half-timbered walls, the out-of-plane opening of the post was observed, with plastic deformation of the nail and at the end of the test the connections would remain open. This uplifting caused by the rocking response of the walls needs an intervention to prevent it, keeping always in mind that the retrofitting has to be compatible with the lateral cyclic movement.

![Figure 6.27](image)

Figure 6.27 – Similarities between damages observed in walls and connections: (a) cracking of bottom connection in timber frame wall; (b) cracking in connection retrofitted with steel plates.

The retrofitting with GFRP sheets can be a possibility to improve the behaviour of the bottom connection of the walls under uplift movements. This technique exhibited good behaviour in terms of resistance, but it is considered that bi-axial sheets should be used in order to avoid brittle behaviour. This means that brittleness of the failure should be taken into consideration when designing the strengthening.

Comparing the strengthening with steel plates in walls and connections, it is seen in both cases that compressive stresses cause buckling of the plates. In tension, plastic deformation of the bolts and tensile failure of steel plates in correspondence to the holes was observed. Moreover, it was always observed ovalization of holes in timber and steel plates.

From the results obtained in the connection it is seen that this solution is very good in terms of ability to dissipate energy. Of course it was seen that, in case of timber frame walls,
diagonals should not be linked, but this was not studied at a connection level. Further studies should be made in order to take into consideration the action of the diagonals.

Considering the results on the NSM technique applied in the connections, it should be stressed that configuration of the steel rods should be able to solve the insufficient anchorage length problem encountered in infill walls. The welded cross for the bottom connection guaranteed a good dissipative capacity during the in-plane cyclic test (the prosthesis’s influence notwithstanding) and it should be able to ensure an adequate strength against uplifting actions. As a downside, the excessively high initial stiffness registered during pull-out tests could have a negative effect on the out-of-plane behaviour of the walls, particularly timber frame ones, but, as in the case of steel plates retrofitting, the free movement of the diagonals should limit this problem. Similarly, the application of bi-axial GFRP sheets to timber frame connections linking the diagonals to the main frame could lead to similar problems.

To complete the comparison and correlation between walls and connections, additional tests would be necessary, in particular on the central connection of the walls and its interaction with the diagonals, since it was observed that this connection was the most important one and the most stressed one in the walls.

6.6 Conclusions

Whichever intervention done on a structure has to be made keeping in mind that an interrelation exists on interventions at all levels, from the whole structure, to the structural elements, to the individual joints.

In the present work, to better understand the behaviour of traditional timber frame walls, it is important to understand their correlation with the key factor influencing their structural response, i.e. the connections. To do this, pull-out and in-plane cyclic tests have been performed on unreinforced and retrofitted traditional connections to study the joint in more detail and try to correlate its behaviour with the behaviour of the walls. From the results obtained, the following conclusions can be made:

- Pull-out tests on unreinforced connections were mainly influenced by the quality of interlocking in the connection, which would increase the load carrying capacity of the nail;
- The deformational features of the connections were in accordance with the ones found in the walls, where connections would open out-of-plane due to the asymmetry in thickness of half-lap connections;
In-plane lateral behaviour of unreinforced connections was influenced by construction defects (clearances) and typical damage consisted of vertical and horizontal cracks in the lower part of the post;

- Retrofitting applied to specimens for pull-out tests highly increased the initial stiffness of the connection presenting also a higher energy dissipation capacity. However, attention should be paid to the desired level of strength, since GFRP sheets strengthening led to brittle failure. Retrofitting performed with screws and steel plates was able to guarantee a post-peak softening behaviour. The NSM strengthening appeared to be excessively stiff and further tests should be carried out to obtain more clear conclusions about its effectiveness in terms of resistance and post-peak behaviour;

- The good response of self-tapping screws to uplifting forces should be complemented by a study of its response to in-plane cyclic actions, possibly with a different disposition of the screws, such as an application from the front of the connection. The current application, though very efficient during pull-out tests, would require the removal of the diagonals from a wall;

- Uni-axial GFRP sheets are not appropriate for undergoing shear forces, due to their early rupture and debonding;

- The prosthesis solution adopted was inappropriate for the strengthening solutions used, since it created a weaker section in the post than some of the retrofitting techniques adopted in the actual connection. It would be more appropriate to create a well strengthened prosthesis, for example with embedded rods, and apply additional strengthening, if needed, around it or use a bigger prosthesis with more screws;

- The NSM configuration adopted for the bottom connection appears to be able to guarantee sufficient anchorage length to the rods, without early failure of the welding. This solution could possibly be used even at the bottom connections of the walls, but due to limitations of the testing equipment, the final strength and dissipative capacity of the retrofitting solution was not obtained;

- Additional information on the influence of the diagonal on the cyclic response of the connections is needed.
Traditional connections: experimental tests on unreinforced and retrofitted connections
CHAPTER 7  ANALYTICAL ANALYSIS OF THE HYSTERETIC BEHAVIOUR OF TIMBER FRAME WALLS

7.1 Introduction

The hysteretic behaviour of timber shear walls subjected to cyclic loading is highly nonlinear and its description can be very complex if a very high accuracy is required. However, simplified models can give equally good results without using much complex equations or parameters. All hysteretic models available in literature to describe the in-plane lateral behaviour of structural elements are based on experimental results, since it is essential to accurately represent the real nonlinear behaviour of shear walls in a numerical model. It should be stressed that an analytical model of the main structural elements resisting to lateral forces is of great importance for a simplified numerical analysis of a building. In fact, it is possible in this way to model the walls in a simplified way, introducing their hysteretic response by means of applying the hysteretic rule to an appropriate finite element model. This tool can help in the design of restoration measures of historic structures as it can help in the description of the global behaviour of the building.

Important factors that have to be incorporated when modelling the hysteretic response of a timber shear wall are strength and stiffness degradation and pinching, since they greatly influence the hysteretic behaviour. Mechanical degradation might lead to progressive weakening and total failure of structural elements and therefore it is important to consider it.

Based on the experimental information on the cyclic behaviour of timber frame walls, an analytical model describing its hysteretic behaviour has been derived. This model is based
on the modification of existing models and is calibrated on static cyclic tests performed on unreinforced and retrofitted timber frame walls. The model is able to predict the lateral load-displacement response of half-timbered walls considering both a flexural and shear predominant behaviour. Appropriate parameters were introduced in order to define the key points of the non-linear behaviour of the walls. Stiffness and strength degradation were also taken into account.

7.1.1 Brief revision on existing hysteretic models

In literature it is possible to find various hysteretic models for different materials. A classic model for concrete was proposed by Takeda et al. in 1970, who developed a trilinear backbone curve which included a segment for uncracked concrete. This model was then used as a starting point for later models.

Various hysteretic models exist for masonry. Among the most notorious models, the one proposed by Modena (1992) can be pointed out (Figure 7.1a), based on a symmetrical envelope curve on which the cycles are built, which was modified in subsequent years by other authors (da Porto et al., 2009).

As far as timber walls are concerned, various examples are available of models developed for modern timber shear walls. Various authors developed their hysteresis models using different paths for the envelope curve. Stewart (1987) idealized pinching and stiffness degradation using a set of force-history rules (Figure 7.1b), considering both pinching and strength degradation. Ceccotti and Vignoli (1990) developed a hysteresis model for moment-resisting semi-rigid wood joints which also models pinching and stiffness degradation (Figure 7.1c) and introduced it in a subroutine which was incorporated in the nonlinear dynamic analysis program DRAIN-2D. The diagrams represent a linear simplification of cyclic test data from which the different stiffness values and line slopes at various stages of displacement are derived.

Pang et al. (2007) introduced an evolutionary parameter hysteretic model for wood shear walls using exponential damage-dependent functions for the parameters of the model. As a backbone curve, they used the one developed by Stewart (1987). Dolan (1989) divided the hysteresis loops into four segments, defining each one by an exponential equation with four boundary conditions. Folz and Filiatrault (2001) developed a model based on Stewart’s (1987) envelope law and a series of path-following rules to reproduce the response of the wall (Figure 7.1e).

Foliante (1995) separated the source of nonlinearity in the hysteretic system from any linear components, obtaining in this way a system with a hysteretic element added to a mechanical model with a linear viscous damping and a linear spring.
Ibarra et al. (2005) developed simple hysteretic models based on existing ones, which include strength and stiffness deterioration properties. Moreover, they incorporated an energy-based deterioration parameter controlling four cyclic deterioration modes.

Figure 7.1 – Examples of existing hysteretic models: (a) masonry model developed by Modena (da Porto et al., 2009); (b) hysteretic model developed by Stewart (1987); (c) hysteretic model developed by Ceccotti and Vignoli (1990); (d) unloading branch of evolutionary parameter model developed by Pang et al. (2007); (e) hysteretic model developed by Folz and Filiatrault (2001); (f) hysteretic model developed by Meireles et al. (2012) calibrated on half-timbered walls.
Considering traditional timber frame walls, some recent works can be found. Meireles et al. (2012) developed a model for traditional Portuguese half-timbered walls modifying existing models for modern timber frame walls, consisting of an exponential envelope, an exponential unloading and a linear reloading (Figure 7.1f). Existing models were generally suitable, but did not consider some peculiarities of half-timbered walls. The authors used an existing model to represent the first loading and envelope curve and adopted an evolutionary parametric model for unloading and reloading branches.

These are only a few of the numerous models that can be found in literature on the hysteretic modelling of timber shear walls based on experimental observations. The aim is to point out that obtaining a hysteretic model based on experimental results is essential to accurately represent in a numerical macro model the non-linear behaviour of timber shear walls.

### 7.2 Proposed analytical hysteretic model

The main features describing the hysteretic behaviour of traditional timber frame walls, whose experimental results were discussed in previous chapters, are the important uplifting of the bottom connections due to the predominant flexural rocking resisting mechanism, pinching, stiffness degradation and energy dissipation. The main issue regarding the walls with predominant flexural behaviour consists of more complex unloading branches with various alterations of stiffness depending on the activation of the bottom connections. This means that the model to be used to reproduce the hysteretic behaviour has to focus on the unloading branches.

After analysing the experimental results obtained in Chapter 4 and Chapter 5, an analytical model was proposed based on the stiffness of the timber frame walls in the various phases of loading. The same model, appropriately simplified, can be applied to the retrofitted half-timbered walls.

#### 7.2.1 First loading and envelope curve

An envelope curve is the curve that connects the points of maximum load for a corresponding displacement for each load step in the hysteresis plot. In order to describe the envelope curve for each wall, it was decided to adopt the equation (Equation 7.1) first proposed by Foschi (1974), which was later included in the program CUREE:

\[
\begin{align*}
F &= sgn(\delta \cdot \Delta \delta) \cdot (F_0 + r_1 \cdot k_0 \cdot |\delta| \cdot (1 - e^{-k_0 |\delta| / F_0})) & |\delta| \leq |\delta_m| \\
F &= sgn(\delta \cdot \Delta \delta) \cdot F_m + r_2 \cdot k_0 \cdot [\delta - sgn(\delta \cdot \Delta \delta) \cdot \delta_m] & |\delta_m| < |\delta| \leq |\delta_{ult}| \\
F &= 0 & |\delta| > |\delta_{ult}|
\end{align*}
\]

(7.1)

\(k_0\) is the tangent stiffness of the envelope curve and values will be higher than the secant stiffness calculated in Chapter 4; \(\delta\) is the general displacement and \(\Delta \delta\) is the gradient of
displacement used to define the sign (sign) of the load; \( \delta_{\text{ult}} \) is the ultimate displacement, whereas \( \delta_m \) is the displacement corresponding to the maximum load \( F_m \). The exponential loading of the envelope curve is defined through stiffness \( r_1 \cdot k_0 \), which corresponds to the slope of the curve immediately before reaching the maximum load \( F_m \). \( F_0 \) is the load at which the line with slope \( r_1 \cdot k_0 \) intersects the vertical axis.

The curve also takes into account the softening behaviour of the wall by means of stiffness \( r_2 \cdot k_0 \), corresponding to the slope of the curve after the displacement corresponding to the maximum load, \( \delta_m \), until reaching the ultimate displacement \( \delta_{\text{ult}} \) (see Figure 7.2).

![Figure 7.2 – First loading and envelope curve](image)

The envelope curve thus described is able to well represent what was observed experimentally. To reduce the number of parameters, it was decided to adopt mean values between the positive and the negative direction. This could lead to some differences between the experimental and numerical models, particularly when the test was not perfectly symmetrical, as observed in some of the walls tested. Notice that it is believed that the asymmetry found in the experimental tests can be associated in part to the asymmetrical loading configuration.

### 7.2.2 Unloading branch

The unloading branch of the hysteresis loop for half-timbered walls is characterized by varying stiffness due to the flexural rocking response resulting in the closing of the middle and corner bottom half-lap connections that uplift in the loading part of the test. For a detailed representation of the unloading branches, five main phases can be distinguished, being each one described through simple linear relations connecting the points characterizing each phase (see Figure 7.3a): (1) between \( \delta_{\text{ult}} \) and \( \delta_{\text{up,5}} \), displacement at which the connections start to close, there is a great loss in strength for a small decrease in displacement. The loss is almost instantaneous, thus the values of stiffness \( r_3 \cdot k_0 \) are high; (2) between \( \delta_{\text{up,5}} \) and \( \delta_{\text{up,6}} \) when the middle bottom connection closes and the curve is characterized by a stiffness
Analytical analysis of the hysteretic behaviour of timber frame walls

\( r_4 \cdot k_0 \); (3) between \( \delta_{\text{up,f}} \) and \( \delta_{\text{cl}} \) when the corner bottom connection also closes and the stiffness \( r_5 \cdot k_0 \) characterizes the loop; (4) once all connections are closed, their stiffness is regained and stiffness increases to \( r_6 \cdot k_0 \) until the load reversal that occurs at displacement \( \delta_{0,\text{un}} \); (5) in order to take into account pinching, stiffness \( r_7 \cdot k_0 \) is considered between the points \( (\delta_{0,\text{un}}; 0) \) and \( (0; F_r) \), where \( F_r \) is the residual force in the cycle when the displacement is 0. It has to be pointed out that a certain consistency should be adopted when defining the target displacements, which are however easily identifiable. Thus, the unloading branch is described according to the expressions given in equation 7.2:

\[
\begin{align*}
F &= F_{\text{ult}} + (\delta - \delta_{\text{ult}}) \cdot r_3 \cdot k_0 & |\delta_{\text{up,s}}| < |\delta| \leq |\delta_{\text{ult}}| \\
F &= F_{\text{up,s}} + (\delta - \delta_{\text{up,s}}) \cdot r_4 \cdot k_0 & |\delta_{\text{up,f}}| < |\delta| \leq |\delta_{\text{up,s}}| \\
F &= F_{\text{up,f}} + (\delta - \delta_{\text{up,f}}) \cdot r_5 \cdot k_0 & |\delta_{\text{cl}}| < |\delta| \leq |\delta_{\text{up,f}}| \\
F &= F_{\text{up,cl}} + (\delta - \delta_{\text{cl}}) \cdot r_6 \cdot k_0 & |\delta_{0,\text{un}}| < |\delta| \leq |\delta_{\text{cl}}| \\
F &= (\delta - \delta_{0,\text{un}}) \cdot r_7 \cdot k_0 & 0 < |\delta| \leq |\delta_{0,\text{un}}|
\end{align*}
\]

where:

- \( F \) is the generalized force;
- \( F_{\text{ult}} \) is the force corresponding to displacement \( \delta_{\text{ult}} \), described above;
- \( F_{\text{up,s}} \) is the force corresponding to displacement \( \delta_{\text{up,s}} \), described above;
- \( F_{\text{up,f}} \) is the force corresponding to displacement \( \delta_{\text{up,f}} \), described above;
- \( F_{\text{cl}} \) is the force corresponding to displacement \( \delta_{\text{cl}} \), described above.

For timber frame walls without infill (Figure 7.3b) the central post does not uplift, therefore it is assumed that \( \delta_{\text{up,s}} = \delta_{\text{up,f}} \) and the slope between \( \delta_{\text{up,s}} \) and \( \delta_{\text{cl}} \) is \( r_4 \cdot k_0 \). Besides, in case of retrofitted walls, the behaviour is also described through expressions of Equation 7.2 but only considering three phases corresponding to three slopes. In this case no uplifting is evident in the hysteretic curve (see Chapter 5), and it is considered that for the unloading branch, \( \delta_{\text{up,s}} = \delta_{\text{up,f}} = \delta_{\text{cl}} \). This simplification could be used even for the reproduction of the hysteretic curve of modern timber shear walls, which present a smooth unloading branch.

To describe pinching, a residual force, \( F_r \), is used for zero displacement. A different trend in terms of residual force is observed between half-timbered and timber frame walls and between different vertical load levels. For UIW25 walls, \( F_r \) can be assumed constant (see Figure 7.4a), while values of displacement corresponding to a zero force, \( \delta_{0,\text{un}} \), vary with a linear law of the ultimate displacement of cycle i (see Figure 7.4b).
For the walls subjected to the higher vertical per-compression, values of residual force were increasing with the lateral displacement. The walls exhibited a constant value of residual force up until reaching the maximum load, after which the residual force increased as a result of damage accumulation (see Figure 7.4c). Values of $F_r$ can be therefore expressed by means of a bilinear, considering a constant value for displacements lower than $\delta_m$, (which is already a variable that has to be input) and a linear law for displacements higher than $\delta_m$. For UIW50 walls, values of residual displacement $\delta_{0,un}$ when load reversal occurs vary with a linear law (Figure 7.4d).

With these values it is possible to obtain the values of parameter $r_7$, which defines pinching. Lower values of $r_7$, therefore higher values of $\delta_{0,un}$, the more the wall is characterized by pinching.

When comparing these trends with those of timber frame and retrofitted walls, a different behaviour can be noticed. Values of residual force present the same trend, namely an approximately constant value for timber frame walls (both unreinforced and retrofitted), whereas retrofitted infill walls present increasing values of residual force (Figure 7.4e). Timber walls present more severe pinching, therefore the parameters defining it have different trends than what observed for infill walls (Figure 7.4f). For both vertical load levels and even for retrofitted walls, the trend is approximately exponential. Therefore, it is immediately apparent that the analytical hysteretic curves have to be defined for each wall typology as well as for each vertical load.

The other unloading parameters were also defined based on the experimental results obtained for the walls. Parameter $r_3$, representing the steep slope of the unloading branch after the maximum displacement, describes the almost instantaneous loss of strength and is defined based on the values found in the distinct types of walls. To calculate the parameter,
the strength loss and displacement $\delta_{up,s}$ are necessary. The drop in strength is shown for half-timbered walls in Figure 7.5.

For timber frame walls and for retrofitted walls a linear behaviour was (Figure 7.5c), having values significantly greater than unreinforced half-timbered walls, leading to a less pronounced plateau, as there is less uplifting.
For both vertical load levels, a linear relation can be assumed in relation to the maximum drift of each cycle. $\delta_{up,s}$ can be given in relation to the ultimate displacement of each cycle considering a simple ratio. In fact, the relation between the two displacement points is linear (see Figure 7.6a), namely $\delta_{up,s,i} = 0.97\delta_{ult,i}$ for the lower vertical load level and $\delta_{up,s,i} = 0.99\delta_{ult,i}$ for the higher vertical load level. Therefore, $r_3$ can be calculated as the slope between the ultimate displacement and point $\delta_{up,s}$. For this parameter, the vertical load level does not have a great influence and a mean value of 0.98 could be assumed.

![Figure 7.5 - Unloading branch parameters: (a) initial strength loss in UIW25 walls; (b) initial strength loss in UIW50 walls; (c) initial strength loss for all wall types.](image)

The values of $\delta_{up,s}$ can be assumed equal to that of the ultimate displacement to simplify the model, but in some tests this was not the case. In timber frame walls, the correlation between the two displacements is linear, with a relation of 0.96 for both vertical load levels, whereas for retrofitted walls it can be assumed as 0.98 of the ultimate displacement for infill walls and 0.81 for timber frame walls. Once again, a clear different can be noticed between half-timbered and timber frame walls.

In a similar way, the other points characterizing the unloading branch can be defined. Even displacement $\delta_{up,f}$, corresponding to the point where the middle connection closes
completely, has a linear relation with the ultimate displacement, namely $\delta_{up,f,i} = 0.70 \cdot \delta_{ult,i}$ for the lower vertical load level and $\delta_{up,f,i} = 0.73 \cdot \delta_{ult,i}$ for the higher vertical load level. For both load levels, the load at which the middle bottom connection closes can be assumed constant (Figure 7.6b). Displacement $\delta_{cl}$ represents the displacement at which all connections are closed and contact between posts and bottom beam is re-established. A linear relation can be obtained between $\delta_{cl}$ and the ultimate displacement. For this parameter, the vertical load level influences the behaviour, but notice that the values can be assumed approximately half of each other.

Even for these parameters, a similar trend was observed for timber frame and retrofitted walls, but with different values, confirming the need to calibrate the parameters for each wall.

Figure 7.6 – Unloading branch parameters: (a) correlation between $\delta_{up,s}$ and $\delta_{ult}$ for calculation of $r_3$ in UIW25 walls; (b) values of load corresponding to $\delta_{up,s}$ in UIW25 walls; (c) correlation between $\delta_{up,s}$ and $\delta_{ult}$ for calculation of $r_3$ in UIW50 walls; (d) values of load corresponding to $\delta_{up,s}$ in UIW50 walls

Having defined these key points in the behaviour of the wall during a cyclic test, it is possible to define even parameter $r_5$ and $r_6$. Even though some approximations were made, a good correlation between the variables $r_i$ calculated from experimental results and those calculated using the described simplifications was found.
The parameters vary depending on the level of vertical pre-compression. Possibly, with the analysis of experimental results available in literature with different pre-compression levels, a correlation could be found between these parameters and the vertical load applied. Moreover, different trends and values were observed when comparing timber frame and infill walls and unreinforced and retrofitted walls, pointing out that a specific calibration for each type of wall is necessary.

### 7.2.3 Reloading branch

The reloading branch can be described with four expressions (see Equation 7.3), being well defined for all walls by the position of two points:

\[
\begin{align*}
F &= F_r + \delta \cdot r_{7i-1} \cdot k_0 & 0 < |\delta| \leq |\delta_p| \\
F &= F_p + (\delta - \delta_y) \cdot r_y \cdot k_0 & |\delta_p| < |\delta| \leq |\delta_y| \\
F &= F_y + (\delta - \delta_b) \cdot r_b \cdot k_0 & |\delta_y| < |\delta| \leq |\delta_b| \\
& \text{see Eq. 7.1} & |\delta_b| < |\delta| \leq |\delta_{\text{max},i}|
\end{align*}
\]  

(7.3)

Hysteretic curves for timber frame walls are characterized by pinching. Therefore, when reloading the wall, the first part of the curve presents the same slope as the last part of the previous cycle \((r_{7i-1})\), since the response presents pinching up to the displacement \(\delta_p\) (see Figure 7.7) and relative force \(F_p\). Subsequently, stiffness is restored in the wall and a linear part can be recognized up to displacement \(\delta_y\) (and relative force \(F_y\)), characterized by the slope \(r_y \cdot k_0\). After the displacement \(\delta_y\), the wall behaves in the plastic regime and the stiffness decreases to \(r_b \cdot k_0\), which describes the stiffness of the curve until the envelope curve is reached.

![Figure 7.7 - Reloading branch](image)

When reloading a wall, stiffness and strength degradation occur. Thus, it is important to take these two factors into account when calculating the reloading curve for subsequent cycles. In order to achieve this, the percentage of strength degradation is calculated for the cycle \(i\)
Analytical analysis of the hysteretic behaviour of timber frame walls

Considering the slope $r_b$ of the curve connecting the force corresponding to the displacements $\delta_y$ with that corresponding to $\delta_{\text{max}_i-1}$, being the latter the displacement corresponding to the maximum displacement of cycle $i-1$. In this way, the strength degradation $\Delta F$ is taken into account for cycle $i$. With this slope, the reloading branch reaches the envelope curve at a displacement $\delta_b$ from which the expressions in Equation 7.1 are used to define the remaining part of the reloading branch.

When analysing each cycle individually, displacements $\delta_p$ and $\delta_y$ are easily recognisable and a correlation can be found with the ultimate displacement of each cycle also for these reloading parameters (Figure 7.9a,b). In particular, $\delta_p$ can be assumed linear, whereas $\delta_y$ has a linear variation with the ultimate displacement of the cycle. The corresponding load values can be assumed constant (Figure 7.9c,d).

For timber frame and unreinforced walls, both values of displacement varied linearly, whereas, also in this case, the corresponding loads were approximately constant.

Strength degradation $\Delta F$ can be assumed constant, since the force corresponding to displacement $\delta_{\text{max}_i-1}$ has a linear correlation to the maximum force of cycle $i-1$, leading to a
constant strength degradation throughout the test for both vertical load levels (Figure 7.9a). For timber frame and retrofitted walls, strength degradation $\Delta F$ is increasing throughout the testing, confirming the necessity to calculate the parameters for each wall typology (Figure 7.9b).

![Diagram](image)

Figure 7.9 - Parameters for reloading branch: (a) values of degraded strength; (b) values of strength degradation for all walls.

### 7.3 Validation of proposed model

Based on the equations presented above, the analytical hysteretic envelope was obtained for the walls tested. Here, some examples are shown, concerning both unreinforced and retrofitted walls to show the adequacy of the model.

#### 7.3.1 Unreinforced half-timbered walls

When applied to the unreinforced half-timbered wall results, the model developed showed a good approximation with what is observed experimentally. The hysteresis loops are well represented by the model, as can be seen in Figure 7.10.

![Diagram](image)

Figure 7.10 - Analytical vs. experimental results: (a) hysteric curve of UIW25 walls; (b) hysteric curve of UIW50 walls

The difference in the loops for this type of wall, which has an asymmetrical response, is mainly due to the decision of assuming the mean values between the two loading directions.
in order to reduce the number of parameters. This behaviour was observed for both vertical load levels.

The model is able to represent well the plateau observed in the unloading branch of the hysteretic response of the walls caused by the uplifting of the bottom connections, particularly in walls UIW25 (Figure 7.10a).

For the higher vertical load, the same good approximation was observed (Figure 7.10b), with errors depending mainly on the asymmetric response of the walls. In terms of dissipated energy, both walls had a good approximation, with an error close to 10%.

7.3.2 Unreinforced timber frame walls

In case of timber frame walls without infill, the model could be simplified, since for these walls the uplifting interests only the lateral posts, so it was possible to assume the simplification \( \delta_{up,l} = \delta_{s} \) and not consider parameter \( r_5 \). Figure 7.11 shows the comparison between the analytical and experimental hysteretic curve for the two vertical load levels. The model is able to well represent the response of the wall until peak loading, being the error more significant in the post-peak branch due to asymmetry recorded in the walls. It should be mentioned that for timber frame walls, a tri-linear envelope curve could be more appropriate, since the exponential envelope is not able to fully capture properly the response of the wall.

Even though some adjustments could be made, the results are nonetheless satisfactory.

7.3.3 Retrofitted walls

The same model has been applied to retrofitted walls, both half-timbered and timber frame without infill. If retrofitted half-timbered walls exhibit a general behaviour similar to the unreinforced walls, with well-defined steps where the various stages are easily recognized (see Chapter 5), retrofitted timber frame walls present very low values of uplift and it can be
assumed that $\delta_{\text{up,s}} = \delta_{\text{up,f}} = \delta_{\text{cl}}$. Thus, the unloading branch can be divided in only three parts, considering three different slopes. With this approximation, the analytical model is able to represent in a good way the experimental results (see Figure 7.12a). Once again, the main cause of error is the assumption of mean values for the parameters, which leads to approximate results. For higher values of displacement, the analytical hysteresis loops in the positive direction do not reach the same displacement observed experimentally, leading to a loss in terms of energy and increasing the prediction error to the analytical model.

Also for retrofitted half-timbered walls (Figure 7.12b) the asymmetry in the response of the wall caused some disturbance on the approximation in the results, even if the comparison between experimental and predicted energy dissipation revealed to be close.

### 7.3.4 Adjustment to other wall types

As mentioned above, it is believed that this model can easily describe the hysteretic behaviour of other timber wall typologies. Since it is based on the different slopes of the hysteresis loops in different phases of the loading, one can only assume that a phase does not exist and simplify the unloading or reloading path, obtaining thus a model similar to what proposed by other authors (Stewart, 1987; Ceccotti and Vignoli, 1990).

It should be stressed that an analytical model is of extreme importance for a simplified numerical analysis of a building. This model can be introduced to represent the hysteretic response of half-timbered walls in a finite element model by applying it to simplified elements, either introducing the parameters with laws derived from the experimental results or fitting the parameters manually.

Further developments include a comparison with other existing models, as well as finding a clear correlation between different vertical load levels and different wall types.
7.4 Conclusions

In this chapter an analytical hysteretic model able to describe the cyclic behaviour of traditional half-timbered walls observed experimentally is presented. The proposed model is an update of existing hysteretic models for timber walls. From the comparison between the experimental results and the analytical model it is possible to conclude that:

- The model is able to describe with a good approximation the hysteretic response of half-timbered walls where a rocking mechanism is present, reproducing their peculiar unloading path;
- Some errors are found when the response of the wall is highly asymmetrical, since it was decided to assume a mean value for the parameters between the two loading directions;
- A simplified version of the model is able to represent with a good approximation the response of a wall where no uplifting is present, i.e. with a smooth unloading branch;

This analytical model is presented in order to be implemented in the future in a finite element software to simulate the hysteretic response of the wall.
CHAPTER 8  NUMERICAL MODELLING OF TIMBER FRAME WALLS

8.1 Introduction

Numerical modelling is an important tool for the analysis of the behaviour of structural elements. Since it is not always possible to obtain experimental results, a numerical approach can represent a good alternative. However, the models should be calibrated on existing experimental or practical results.

Concerning timber frame structures, and walls in particular, various numerical studies are available on the behaviour of modern timber frame shear walls. Fewer results are available on traditional ones. Kouris and Kappos (2012) performed non-linear numerical analyses on traditional half-timbered walls. Based on the test results of Santos (1997), the authors first performed a detailed modelling of a wall, considering non-linear properties for the materials and modelling the contacts in timber elements. Subsequently, they created a simplified model using beam and link elements, which was used to simulate a building façade. Quinn and D'Ayala (2013) modelled traditional Peruvian timber frame walls (quincha) adopting semi-rigid spring elements to model the mortise and tenon connections of the wall.

Another approach to the modelling of timber frame walls is the use of analytical hysteretic models calibrated on experimental results to a simplified numerical wall model. Ceccotti and Sandhaas (2010) performed such an analysis on a traditional timber frame wall creating a numerical model consisting of lumped masses and rotational springs, where the rigid elements represent the timber elements, while the global cyclic behaviour of the wall is given only by the springs. The springs could model both rotations and vertical uplifting. This model was then extended to model a whole building. A similar approach was used by Meireles (2012) introducing an analytical hysteretic model calibrated on experimental results in the finite element software, modelling a traditional Pombalino wall representing the connections.
Numerical modelling of timber frame walls

by means of spring elements. A macro-element was developed to represent the Portuguese frontal walls inside a masonry building. Hicyilmaz et al. (2012) developed a numerical model of a dhajji dewari building, taking into account parameters such as vertical load, shortening of the diagonal bracings and removal of nails from the wall.

Numerical models are available even for traditional timber connections. Hong and Barrett (2008) developed a three-dimensional finite element model for Japanese post and beam connections (basically a mortise and tenon joint), modelling the nails and steel plates of the connection and the crushing of timber that occurred at the mortise-tenon interface. Branco et al. (2006) modelled the behaviour of traditional timber roof connections using nonlinear moment-rotation laws and hysteretic rules in order to represent the cyclic response of the joint. Considering modern connections, Tannert et al. (2010) studied the strength prediction of rounded dovetail connections considering size effects. Xu et al. (2009) developed a 3-D numerical model to predict the mechanical behaviour of steel to timber joints.

In this chapter, a numerical model has been validated to simulate the mechanical behaviour observed in the experimental results of cyclic tests on traditional timber frame walls. Firstly, the model was calibrated considering only the timber frame and subsequently infill was added. A parametric study was then carried out considering some variables such as post continuity, type of infill and wall geometry.

8.2 Numerical model developed

The numerical model of the timber frame was created using software DIANA (2009). The anisotropic behaviour of timber was taken into account and the connections were modelled using interface elements with Coulomb friction properties.

Two models are here presented, a 3-D model and a simplified 2-D model. When modelling timber connections, the choice of the type of finite element model to be created is always a challenge, as the representation of all the planes of the connection plays an important role, but, on the other hand, results in a complicated model that can be computationally expensive.

8.2.1 Geometry

The geometry of the model follows accurately the walls tested in the Laboratory of Structures at University of Minho. Due to the type of connections of the wall, which are half-lap joints, a 3-D model was created using 20 nodes brick elements (CHX60). In this way, the real behaviour of the connection was taken into account (see Figure 8.1a).
The half-timbered wall was modelled considering all the connections between the posts and the beams. Only the half-lap connections in the diagonals were not modelled, as it was seen during the experimental campaign that they have little influence on the behaviour of the wall.

Plane quadrilateral, 8+8 nodes interface elements (CQ48I) were used to simulate the timber-timber contact and the timber-masonry contact. Different properties were assumed for each plane. The nails in the connections were not modelled, as this would have required a too high computational effort for a low improvement in the results. Figure 8.1b shows a detail of the interface elements of a half-lap connection and the contact connection between the diagonals and the main frame. The interfaces of the two types of connections are independent and work separately, as it happens in a real wall.

The masonry infill was modelled as a macro element. Each triangle of masonry in the wall is considered as a block. This choice was based on the experimental results, as it was observed that masonry actually behaves like a block and cracking of masonry appeared not to influence significantly the response of the wall. It is considered that the main contribution of masonry is for the addition of stiffness to the wall, but it was seen that a damaged masonry is still very much able to contribute to the stiffness of the wall. The wall behaviour is dominated by the timber connections, so a simplified modelling of masonry is admissible.

Even if this model would have well represented the behaviour of the wall, and the first results were satisfactory, it was computationally demanding and the calibration of the interface elements was challenging, considering that different properties had to be applied to different connections. Moreover, to perform parametric analyses, such a complicated model would be too demanding when changing the geometry. Another issue is that, taking into account that a future development could be the implementation of the wall model in a simulation of a whole building, a simplified model would be more suitable.
Therefore, a simplified 2-D model of the wall was created, using plane stress elements to model the timber members and 3+3 nodes CL12I line interface elements to model the connections. The elements used for timber were eight-node quadrilateral isoparametric plane stress element CQ16M (Figure 8.2a) based on quadratic interpolation and Gauss integration, whereas the interfaces used were line-based elements with 3+3 nodes (Figure 8.2b). For the interface elements, a 3-point Lobatto integration scheme was used.

![Figure 8.2 - Finite elements used: (a) eight-node quadrilateral plane stress element (DIANA, 2009); (b) 3+3 nodes line interface element (DIANA, 2009); (c) six-node triangular plane stress element (DIANA, 2009).](image)

To take into account the uplift observed during the experimental tests, interface elements were introduced between the posts and the bottom beam (Figure 8.3a). The horizontal interface elements should simulate the bottom half-lap joint. Moreover, interface elements were introduced between the diagonals and the main frame to represent the nailed connections between the two elements. It was important to represent these connections, since it was seen during the experimental campaign that the action of the diagonals had an important influence on the walls, particularly when infill is not present. Moreover, using a rigid connection would have stiffened the wall in an unrealistic way.

Notice that in this simplified model, it was assumed that the half-lap connections of the diagonals and of the middle and top beam of the wall were rigid. To simulate the crushing observed in these connections, mainly at the central post, different materials properties were considered in this region.

![Figure 8.3 - Plane stress model of wall: (a) timber frame wall; (b) infill wall.](image)
In a subsequent phase, infill was introduced in the model. Six-node triangular isoparametric plane stress elements were used (CT12M), based on quadratic interpolation and area integration (Figure 8.2c). Masonry infill was assumed as a continuous and isotropic material and the brick joints were not represented. Interface elements (CL12I) were introduced between the masonry infill and the timber frame (Figure 8.3b) to represent the connectivity between the two materials.

The boundary conditions applied to the model reproduce those applied during the tests, i.e. the bottom beam is fixed at the base. In terms of load application, the conditions of the test were also replicated. A uniform vertical pressure was applied on each post and a horizontal displacement was applied to the right section of the top beam. The solution algorithm used for the non-linear analyses performed was the full Newton-Raphson method, where the stiffness matrix is updated after each iteration. The convergence criterion was energy based. Moreover, a line search algorithm was used to scale the incremental displacements in the iteration process automatically and improve convergence.

A comparison between the detailed 3-D and simplified 2-D model is shown in Figure 8.4. Both models are able to represent the rocking behaviour characterizing the wall. The 3-D model is able to better capture the deformation of the half-lap joints, but this problem is partially overcome by adopting different material properties in the 2-D model.

![Figure 8.4 - Deformation and stresses distribution of timber frame wall model: (a) 3-D model; (b) 2-D plane stress model.](image)

**8.2.2 Material properties**

Non-linear properties were assumed for timber and for the timber-to-timber interfaces. For the model of masonry infilled timber frame walls, both masonry and timber-to-timber interfaces were assumed non-linear.

The orthotropic behaviour of timber is taken into account by using the Rankine-Hill Anisotropic model (Lourenço, 1996), which captures different strengths and softening characteristics in orthogonal directions (Figure 8.5a). In order to model the anisotropic
Numerical modelling of timber frame walls

behaviour of the material, the model considers a Hill yield criterion in compression and a Rankine yield criterion in tension.

The variables of the anisotropic material model include modulus of elasticity parallel and perpendicular to the grain ($E_0$ and $E_{90}$), compressive and tensile strength parallel and perpendicular to the grain ($f_{c,0}$; $f_{c,90}$; $f_{t,0}$; $f_{t,90}$), their corresponding fracture energies ($G_{fc,0}$; $G_{fc,90}$; $G_{ft,0}$; $G_{ft,90}$), factor $\alpha_r$ that determines the shear stress contribution to tensile failure (default for Rankine is 1.0), factor $\alpha_t$, which relates the equivalent length of the element to the area of the finite element and is 1 for quadratic elements, factor $\beta$, which couples the normal compressive stresses and is typically equal to -1, factor $\gamma$, which controls shear stress contribution to compressive failure, and $\kappa_p$ factor, which is the equivalent plastic strain corresponding to the peak compressive stress. The linear properties for timber include Poisson's ratio $\nu$, shear modulus $G_v$ and material density $\rho$.

For interface elements, a Coulomb Friction model was used (Figure 8.5b), combining the friction criterion with a gap criterion, in order to allow the detachment of the diagonals, and assuming a Mode-II constant shear retention model after gap appearance. This means that if the tensile traction normal to the interface exceeds the tensile strength of the connection ($f_t$), a gap will arise. After gap formation, the tensile strength of the interface is reduced to zero immediately. The other variables for this material model are cohesion ($c$), the tangent of the friction angle ($\tan \phi$), the tangent of the dilatancy angle ($\tan \psi$) and the reduced stiffness of the interface for the constant shear retention model after gap appearance. The linear variables of the model consist of the normal stiffness ($k_n$) and tangent stiffness ($k_t$) of the interface.

![Image](image_url)

Figure 8.5 - Material properties models used: Rankine-Hill Anisotropic (DIANA, 2009); (b) Coulomb friction (DIANA, 2009); (c) total strain fixed crack model, exponential softening in tension (DIANA, 2009); (d) total strain fixed crack model, parabolic compression (DIANA, 2009).
Infill was assumed isotropic and a total strain fixed crack model with exponential softening in tension and parabolic in compression was used, see Figure 8.5c,d. The linear variables of the model consist of the Young modulus (E), Poisson’s ration (ν) and density (ρ). The non-linear parameters in tension are the tensile strength of masonry (f_t) and the Mode-I fracture energy G_f^I. For the compressive behaviour, compressive strength (f_c) and compressive fracture energy (G_c) are required. For the shear behaviour, it was assumed a constant shear retention, being the shear retention factor (β) assumed to be equal to 0.01.

8.3 Model calibration

8.3.1 Timber frame walls

The model was calibrated based on the experimental results obtained, considering the material properties derived from the tests carried out in Chapter 3 and comparing the results with the envelope curves of the in-plane cyclic tests of the walls, taking also into account the deformation patterns and stress distributions. Table 8.1 shows the material properties adopted for timber. The additional information necessary which was not derived from experimental results was obtained using the JCSS (2006) probabilistic model code.

As mentioned above, in order to take into account the influence of the half-lap connections of the main frame, different material properties were applied to the timber elements in the position of the joints (Table 8.2). In fact, it is assumed that the region of the connection cannot be considered completely homogenous and this simplified procedure to take into account the discontinuity was considered to be reasonable. Higher values of stiffness perpendicular to the grain were used, to take into account the contribution of the post in the half-lap joint, while all strength properties were reduced, since the connection is the weak point of the beam, as the material is not continuous and defects, such as fissures and clearances, are concentrated there.

<table>
<thead>
<tr>
<th>Material properties used for timber.</th>
</tr>
</thead>
<tbody>
<tr>
<td>f_{t,0}</td>
</tr>
<tr>
<td>f_{t,90}</td>
</tr>
<tr>
<td>E_0</td>
</tr>
<tr>
<td>E_90</td>
</tr>
<tr>
<td>f_{c,0}</td>
</tr>
<tr>
<td>f_{c,90}</td>
</tr>
</tbody>
</table>

For the interface elements, different properties were adopted for the bottom connections and the diagonal ones. Notice that the half-lap bottom connections should present higher stiffness due to a certain level of interlocking, which does not exist in case of diagonal
elements, mainly as regards the shear stiffness. Thus, for the half-lap joints, higher values of stiffness, cohesion and tensile strength were assumed (Table 8.3) in order to represent the contribution of both nail and interlocking of the half-lap joint.

Table 8.2 - Reduced timber properties for half-lap joints.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{t,0}$</td>
<td>15 MPa</td>
</tr>
<tr>
<td>$f_{t,90}$</td>
<td>5 MPa</td>
</tr>
<tr>
<td>$E_0$</td>
<td>11000 MPa</td>
</tr>
<tr>
<td>$E_90$</td>
<td>5000 MPa</td>
</tr>
<tr>
<td>$f_{c,0}$</td>
<td>2 MPa</td>
</tr>
<tr>
<td>$f_{c,90}$</td>
<td>3 MPa</td>
</tr>
<tr>
<td>$G_v$</td>
<td>700 MPa</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>9</td>
</tr>
<tr>
<td>$\rho$</td>
<td>590 kg/m$^3$</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1</td>
</tr>
<tr>
<td>$\beta$</td>
<td>-1</td>
</tr>
<tr>
<td>$\kappa_p$</td>
<td>0.001</td>
</tr>
<tr>
<td>$G_{H,0}$</td>
<td>70 Nmm/mm$^2$</td>
</tr>
<tr>
<td>$G_{H,90}$</td>
<td>50 Nmm/mm$^2$</td>
</tr>
<tr>
<td>$G_{fc,0}$</td>
<td>130 Nmm/mm$^2$</td>
</tr>
<tr>
<td>$G_{fc,90}$</td>
<td>70 Nmm/mm$^2$</td>
</tr>
<tr>
<td>$E_0$</td>
<td>11000 MPa</td>
</tr>
<tr>
<td>$E_90$</td>
<td>5000 MPa</td>
</tr>
<tr>
<td>$f_{c,0}$</td>
<td>2 MPa</td>
</tr>
<tr>
<td>$f_{c,90}$</td>
<td>3 MPa</td>
</tr>
<tr>
<td>$G_v$</td>
<td>700 MPa</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.3</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>9</td>
</tr>
<tr>
<td>$\rho$</td>
<td>590 kg/m$^3$</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1</td>
</tr>
<tr>
<td>$\beta$</td>
<td>-1</td>
</tr>
<tr>
<td>$\kappa_p$</td>
<td>0.001</td>
</tr>
<tr>
<td>$G_{H,0}$</td>
<td>70 Nmm/mm$^2$</td>
</tr>
<tr>
<td>$G_{H,90}$</td>
<td>50 Nmm/mm$^2$</td>
</tr>
<tr>
<td>$G_{fc,0}$</td>
<td>130 Nmm/mm$^2$</td>
</tr>
<tr>
<td>$G_{fc,90}$</td>
<td>70 Nmm/mm$^2$</td>
</tr>
</tbody>
</table>

In particular, tangential stiffness was assumed higher, as the penetration of the post inside the beam guarantees a high stiffness in this direction. A lower normal stiffness was assumed for the bottom connections, as the pure nail withdrawal did not give a high resistance, with friction not always participating.

Table 8.3 - Material properties used for interface elements of bottom connections.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_n$</td>
<td>1.25 N/mm$^3$</td>
</tr>
<tr>
<td>$k_t$</td>
<td>5 N/mm$^3$</td>
</tr>
<tr>
<td>$\rho$</td>
<td>0.15 N/mm$^3$</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1</td>
</tr>
<tr>
<td>$\beta$</td>
<td>-1</td>
</tr>
<tr>
<td>$\kappa_p$</td>
<td>0.001</td>
</tr>
</tbody>
</table>

For the interface elements between the diagonals and the main frame, the mechanical properties adopted are shown in Table 8.4. It was assumed a lower cohesion and friction angle in relation to half-lap joints, since no interlocking was present. Moreover, the tangential stiffness was lower, as the only contribution is given by the presence of the nail and of friction. For the diagonal interfaces only one friction surface exists, whereas as in half-lap joints, three friction surfaces are mobilized in shear.

Table 8.4 - Materials properties used for interface elements of diagonals.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_n$</td>
<td>1 N/mm$^3$</td>
</tr>
<tr>
<td>$k_t$</td>
<td>1 N/mm$^3$</td>
</tr>
<tr>
<td>$\rho$</td>
<td>1.3 MPa</td>
</tr>
<tr>
<td>$\alpha$</td>
<td>1.5</td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.9</td>
</tr>
<tr>
<td>$\kappa_p$</td>
<td>0.1</td>
</tr>
</tbody>
</table>

In relation to the properties in the normal direction, it should be noticed that only a slightly higher value of normal stiffness was assumed for the half-lap joints (see Table 8.4 ad Table
8.3) as the nail diameter and length was smaller than in the diagonal, not justifying a significantly higher value of normal stiffness.

Assuming these values, the model of the timber frame was calibrated, applying a monotonic load to the top of the wall in displacement control. Figure 8.6 shows the fitting of the numerical load-displacement curve to the experimental monotonic envelope.

It is observed that a reasonable fitting between experimental and numerical results was obtained both in terms of stiffness, lateral resistance and maximum displacement. It should be noticed that the same material properties were assumed for the two vertical loads, and only the pressure on the posts was varied.

In terms of initial stiffness, the numerical model exhibits a value of 3.8kN/mm for both load levels, which is in good accordance with the experimental results for the higher vertical load, with an increase of only 6%. For the lower vertical load the error is higher, with an increase of stiffness of approximately 45%. In terms of maximum load, both load levels exhibited a good approximation, with a decrease of 9% and 15% respectively for the lower and higher vertical load.

In terms of deformation patterns and damages, the model presented some limitations. The assumption of having the half-lap joints rigid at the middle and top beam did not allow the opening of those connections due to the shear action of the diagonals, as it happened during the experimental tests. Even though stresses concentrated at those connections, particularly in the central one, the impossibility of having separation between the elements led to a highly flexural response of the model (Figure 8.7), with higher uplift at the bottom connections.

The uplift of the lateral connection for the lower vertical load level reached 65mm, a value comparable with what observed experimentally in half-timbered walls, where the infill did not
allow the connection to collapse, as it was confined by the timber frame. However, the model was still able to represent, in a simplified way, the response of the wall.

In terms of stresses distribution, both walls presented important stress concentrations at the central connections (Figure 8.7c,d), with detachment of the diagonals for the higher vertical load, as occurred during the experimental tests. Given the continuity of the central connection, it is not possible to see the actual S-shape deformation of the connection, but the distribution of stresses presents a similarly shaped path. The interfaces of the diagonal allow penetration of the element in the connection, indicating that there is a shear action of the diagonal on the beam. The compressive strength of wood at the central connection though was not reached.

Stresses concentration also occurred at the left bottom connection, resulting sometimes in crushing of the post.

Figure 8.7 - Deformation and stress distribution of numerical model: (a) lower vertical load level; (b) higher vertical load level; (c) central connection, lower vertical load level; (d) central connection, higher vertical load level (values in MPa).

### 8.3.2 Timber frame walls with masonry infill

To calibrate the numerical model of the timber frame walls with masonry infill, non-linear properties were attributed to both masonry (Table 8.5) and timber-to-masonry interfaces (Table 8.6). The properties of the timber frame were kept constant, as it was assumed the same calibration adopted for the model without masonry infill.
The properties assumed for masonry were slightly lower than those obtained from the experimental tests on masonry prisms carried out in Chapter 3, since a variability in terms of mortar resistance and in terms of quality of construction of the walls was observed. Therefore, to take into account a possibly weaker masonry for the walls, the values were reduced. For the interface properties, values similar to those attributed to the diagonal interfaces were adopted.

Table 8.5 - Material properties assumed for masonry infill.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$</td>
<td>5500 MPa</td>
</tr>
<tr>
<td>$f_t$</td>
<td>0.05 MPa</td>
</tr>
<tr>
<td>$f_c$</td>
<td>5 MPa</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.2</td>
</tr>
<tr>
<td>$G^d$</td>
<td>2 N/mm</td>
</tr>
<tr>
<td>$G_c$</td>
<td>20 N/mm</td>
</tr>
<tr>
<td>$\rho$</td>
<td>1800 kg/m$^3$</td>
</tr>
<tr>
<td>$\beta$</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Table 8.6 - Material properties assumed for timber-to-masonry interface.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k_n$</td>
<td>1 N/mm$^3$</td>
</tr>
<tr>
<td>$k_t$</td>
<td>0.5 N/mm$^3$</td>
</tr>
<tr>
<td>$f_t$</td>
<td>1 MPa</td>
</tr>
<tr>
<td>$c$</td>
<td>0.8</td>
</tr>
<tr>
<td>Reduced stiffness</td>
<td>0.9 N/mm$^3$</td>
</tr>
<tr>
<td>$\tan \psi$</td>
<td>0.2</td>
</tr>
</tbody>
</table>

In Figure 8.8 the load-displacement curves obtained in the static cyclic experimental tests (average envelope) and in the numerical analyses are presented. It is observed that a reasonable approximation is shown both in terms of initial stiffness and in terms of maximum load, with an increase of both values, 65% and 45% increase of initial stiffness for the lower and higher vertical load level respectively and 23% and 7% increase of maximum load for the lower and higher vertical load level respectively.

For both load levels, the model exhibited a flexural rocking response (Figure 8.9a), with a clear compression strut along the main diagonal. The masonry infill did not present high
values of stress. Stress concentrations occurred at the connections, in particular at the left bottom connection (Figure 8.9b) and at the central connection (Figure 8.9c), a trend that was also observed during the experimental tests.

Figure 8.9 – Deformation and stress distribution of numerical model of infill wall: (a) higher vertical load level; (b) left bottom connection, higher vertical load level; (c) central connection, higher vertical load level (values in MPa)

### 8.4 Parametric analyses

Having appropriately calibrated the models, it was decided to carry out parametric analyses in order to better understand the influence of different parameters on the lateral response of the wall. In fact, parametric analyses are an important tool in numerical modelling since they allow to study the influence of different parameters on a calibrated model, a feat which would otherwise require extensive experimental testing. In the parametric studies performed on timber frame walls, it was decided to evaluate the influence on the response of the walls of the following parameters:

- Normal stiffness of the interface simulating the bottom half-lap connection of the wall, in order to take into account different levels of continuity of the post and possible retrofitting of this zone as studied previously;
- Material properties of timber, considering different values of elastic modulus and of strength in both directions of the grain;
• Material properties of infill, considering different values for elastic modulus and strength of the infill as well as considering different levels of connectivity between infill and timber frame by changing the properties of the interface elements;
• Geometry of the wall, considering different height to length ratio by adding cells to the wall model.

8.4.1 Post continuity
During the experimental campaign carried out in Chapter 4 it was seen that the continuity of the post, given by the limitation of uplifting of the bottom connection, greatly influences the overall response of the wall, both in terms of stiffness and of maximum load (Poletti and Vasconcelos, 2012).

To capture the influence of such a parameter, the normal stiffness $k_n$ of the interface elements linking the bottom beam to the posts was varied, considering values ranging from 25% to 500% of the value used in the calibrated model. The tangential stiffness was kept constant, as it does not influence the uplifting of the post. This parametric study was carried out on the timber frame wall model with the lower vertical load level.

Figure 8.10a shows the load-displacement curves for different values of normal stiffness. Notice that both initial stiffness and maximum load are influenced by the post continuity. For an increase of normal stiffness higher than 500% the parameter loses its influence, as the connection becomes rigid. For high $k_n$ values, the tensile stresses in the connections of the central beam reached the maximum tensile strength value assumed for those elements, as the response of the wall was predominantly in shear.

![Graph](image-url)

**Figure 8.10** - Influence of post continuity on lateral response of the wall: (a) load-displacement curves; (b) variation of initial stiffness.
Numerical modelling of timber frame walls

In terms of initial stiffness $K_{in}$, it increases with increasing values of normal stiffness of the connection, $k_n$, until a constant value is reached, after which other parameters, such as timber strength and connection to the diagonals should have more relevance (Figure 8.10b).

Comparing the deformatonal features and stress distributions in relation to the normal stiffness of the bottom interfaces (Figure 8.11), a weak bottom connection leads to a rocking behaviour of the wall, with stresses concentration on the left post, while a stronger bottom connection leads to a shear deformation of the wall.

Figure 8.11 - Influence of post continuity on deformation of timber frame wall: (a) weak connection, 0.50x; (b) strong connection, 5.00x (values at same displacement level, in MPa).

8.4.2 Material properties

Different wood species are used in traditional timber frame walls, therefore a study of the influence of the material properties of timber is relevant when analysing their response. Moreover, the type of infill also presents a considerable variation in terms of materials and masonry bonds, and either brick or rubble masonry could be found and in some cases even weak masonry materials can be used. Another case is that of infill performed with lath and plaster, which had a different connectivity to the timber frame, since laths were nailed directly on the frame. Even for masonry infill, nails could be inserted in the frame in order to create a sort of ties between the frame and the infill, thus improving the connectivity between the two materials.

For these reasons, a study of the influence of timber and masonry properties is here presented.

8.4.2.1 Influence of the mechanical properties of timber

Timber properties were varied in an integrated manner, i.e. applying a multiplying factor to all properties, namely Young modulus, density, shear modulus, tensile and compressive strength, fracture energies. The scale factor was applied to all timber elements. The properties varied between 50% and 200% of the values assumed for the calibrated timber
frame model with the lower vertical load level. Figure 8.12a shows the load-displacement curves obtained varying the timber properties.

It is observed that both stiffness and lateral strength of the walls are dependent on the mechanical materials of wood, being the lateral strength more affected than the lateral stiffness. Higher mechanical properties result in more rigid and more resistant walls. However, the differences are not so much significant, with a variation of 9% for the stiffness and of 5% for the resistance. The most visible difference is observed in the post-peak behaviour, as a wall with higher properties appears to have less strength loss. The variation of initial stiffness $K_{in}$ with infill properties is approximately linear (Figure 8.12b).

![Figure 8.12](image)

**Figure 8.12 - Influence of timber properties on lateral response of wall: (a) load-displacement curves; (b) variation of initial stiffness with timber properties.**

### 8.4.2.2 Influence of infill properties

The influence of infill was studied for the lower vertical load level considering varying properties. The study was carried out varying infill material properties between 25% and 200% of the values used for the calibrated model, whereas properties of the interfaces varied between 50% and 200% in order to study the influence of the infill connectivity to the timber frame, keeping the infill properties constant, namely those assumed during the calibration process. Notice that the increase on the mechanical properties implies to change the type of infill adopted for the walls.

Infill properties appeared not to influence the initial stiffness of the wall (Figure 8.13a), which is governed by the timber frame and by the bottom connections. The load capacity of the wall increased with increasing values of infill properties, even if the variation was relatively small (10%). In the experimental results, for the lower vertical load level, the type of infill (brick masonry or lath and plaster) did not have a great influence on the initial stiffness of the walls. Notice that lath and plaster infill not only intervenes in the infill properties, but also in the infill connectivity, therefore it represents a combination of the two parametric studies.
An important factor in the response of the wall turned out to be the connectivity between infill and timber frame (Figure 8.13b). Although the influence on the initial stiffness is less significant, with a variation of 16%, the maximum load capacity varied of 40%, indicating that, when the infill is well connected to the frame it can significantly increase its strength. On the other hand, the decrease on the mechanical properties of the masonry-timber interface leads to an important decrease on the lateral resistance.

In order to better understand the influence of infill on the resistance of the wall, two analyses was carried out considering (1) linear properties for infill and (2) weak properties of the masonry infill. It was seen that, even though the linear infill contributed to the stiffness of the wall, it did not add any additional strength, as the ultimate load was similar to the one recorded in the wall without infill (Figure 8.13c). Moreover, considering an extremely weak infill with extremely weak connections (limit case in which properties are almost zero), the response of the infill wall was, as expected, similar to the response exhibited by the timber frame without infill (Figure 8.13c), pointing out the role of the infill on the overall behaviour of the wall.

From the results obtained it can be concluded that the presence of infill is very important for the response of timber frame walls, particularly as concerns the resistance of the walls. This
confirms what observed in the experimental results, where infill walls had higher stiffness, load capacity and dissipative capacity.

### 8.4.3 Wall geometry

The last parameter to be taken into consideration is the geometry of the wall. Timber frame walls have a great variability in terms of geometry, namely height to length ratio of the wall, height to length ratio of a single cell, sectional dimensions of the timber elements, positioning of the diagonals. In this study, only the first issue will be discussed, keeping as a future development the analysis of the other parameters.

Three alternative wall geometries were analysed, namely a wall with three cells along the length and two cells along the height (3×2), a wall with two cells along the length and three cells along the height (2×3) and a wall with three cells along the length and three cells along the height (3×3). With these three configurations it was possible to vary the height to length ratio.

The general behaviour of the models in terms of stresses paths and deformation was similar to what observed for the original model with a 2×2 configuration (Figure 8.14), with similar stresses concentrations in the connections and at the compressed post.

![Figure 8.14 - Wall geometries studied, minimum principal stresses: (a) 3x2 wall; (b) 2x3 wall; (c) 3x3 wall (values in MPa).](image)

The response of the walls changed in terms of initial stiffness and load capacity (Figure 8.15). For the same height, a higher width of the wall, corresponding to a lower height to length ratio, leads to higher initial stiffness and load capacity. For example, model 3×2 has an initial stiffness 80% higher than that of model 2×2, while model 3×3 has an increase of 92% on the stiffness when compared to model 2×3. In terms of maximum load, the increase is of 76% for two cells in height and of 98% for three cells in height.

On the contrary, for the same width, a higher height (i.e. an increasing height to length ratio) leads to smaller values of initial stiffness and load capacity, as the higher slenderness of the wall leads to a lower resistance. It should be stressed that these results are comparable to what is found in masonry walls, where the height to length ratio takes an important role both
Numerical modelling of timber frame walls

on the lateral resistance and on the resisting mechanism, being the low values of height to length ratios associated to predominant shear behaviour (Haach et al., 2011).

Both initial stiffness and load capacity had an approximately exponential variation with the height to length ratio (Figure 8.15b).

![Figure 8.15 - Influence of the height to length ratio on the lateral response of the wall: (a) load-displacement curves; (b) initial stiffness and load capacity of walls](image)

### 8.5 Further developments

The numerical analyses carried out in this work are only a small part of what could be analysed when studying traditional timber frame walls. Among the future developments that could be mentioned, the most interesting ones are: (1) the implementation of the hysteretic model derived in Chapter 7 into a finite element software in order to model the cyclic response of the wall. This would require the implementation of a subroutine or the alteration of an existing one and it would allow to capture the cyclic response of the wall in a simplified way, for example creating a model using beam elements and spring elements to represent the connections, to which the hysteretic properties would be applied; (2) the simulation of the seismic behaviour of a whole timber frame structures through the execution of pushover and dynamic analyses on a simplified model. This is the further step after the implementation of the hysteretic model. In this way, a clear response of a traditional timber frame building could be analysed, taking into account different wall geometries in the buildings and different levels of connectivity among timber frame walls but also between timber frame and masonry walls.

Other studies that could be considered are the assumption of different types of timber frame walls. This thesis focuses on traditional Portuguese walls, but a great variety of walls exist and in recent years experimental tests have been carried out on walls of different origins (Vieux-Champagne et al., 2012; Aktas et al., 2013; Hicyilmaz et al., 2012) being possible to validate the application of the proposed hysteric models based on results available in literature and assess the differences among the walls. Another issue is the differences found
in the whole buildings, as in Pombalino buildings the external walls are masonry ones, while in Greece and Italy a timber frame is present. A comparison of the response at the whole building level would also be interesting.

8.6 Conclusions

In order to be able to represent the wall in a simplified way, a 2-D model was preferred to a detailed 3-D model, even simulating the connections in a simple way. A good correlation was found between the experimental results and the calibrated numerical model, for both timber frame and infill walls. Material non-linearity was considered for both timber and masonry as well as for timber-to-timber and timber-to-masonry interfaces.

A parametric analysis was carried out pointing out the following points:

- The greater influence on the behaviour of the wall is given by the continuity of the post as well as the connectivity between timber and masonry. This seems in accordance with experimental results as the response of the wall was governed by its connections;
- The geometry of the wall influences its stiffness and load capacity, as a higher height to length ratio led to lower values of these parameters;
- Future developments on this field should be important, since only a few studies currently exist.
9.1 Conclusions

This thesis aimed at obtaining a better understanding of the response of timber frame structures during seismic events, since they constitute an important cultural heritage in many countries. It was seen that timber and masonry have been used together since ancient times both as a strengthening solution and as an independent structural system. Moreover, timber frame buildings have been adopted specifically as seismic-resistant constructions and still today their good response to earthquakes can be witnessed.

Due to the important rehabilitation effort that was observed in various countries in the last decade, a study of retrofitting solutions was also carried out in order to provide a better insight for appropriate interventions.

This thesis focuses mainly on experimental results, with additional numerical simulations to confirm and further develop the study. Since conclusions were already presented for each topic, some brief and final remarks are reported here.

9.1.1 Experimental campaign on traditional timber frame walls and connections

Tests performed on unreinforced timber frame walls pointed out the dominant influence of the connections on the global behaviour of the wall as well as the influence of infill.

Connections represent the dissipative mechanism of the walls as well as their weakest point. Failure occurs at the connections, but, thanks to the geometry of the wall, once a connection fails a stress redistribution occurs and the wall still has a remaining bearing capacity.
Final remarks and future developments

An influencing factor was the bottom half-lap joint of the wall, which links the post to the bottom beam. Since it was a Tee halving joint, uplifting of the bottom connections occurred, leading to a flexural response of the wall, particularly for the lower vertical load level. This issue represents the problem of the level of continuity of the post from one floor to the next. The level of connectivity between the post and the beam varied, which occurred not only in traditional Portuguese timber frame buildings, but also in systems found in other countries, such as in Colombages constructions. This parameter has to be taken into account when intervening in existing buildings or when building new timber frame houses (notice that in the manuals created for reconstruction purposes, additional nails and steel straps are added).

The second factor that greatly influences the response of timber frame walls is infill. The infill increases both the stiffness and the load carrying capacity of the wall, having a confining effect on the connections, promoting the flexural response of the wall with the detachment of the bottom connection from the bottom beam. On the other hand, infill is able to guarantee a greater dissipative capacity to the wall as well as a greater ductility. During the cyclic tests, infill can dissipate energy and limit the damages to the timber frame.

It should be noticed that the type of infill does not have a great influence on the response of the wall, at least if the stiffness is not much different. By comparing the performance of masonry infill and lath and plaster, it is observed that the variations in terms of stiffness, load capacity and seismic parameters are minimal, even if lath and plaster gave a greater confinement to the timber frame, as they were directly linking the timber elements together without compromising their deformability. In fact, in existing timber frame walls different types of infill are adopted, but all have shown a good behaviour during earthquakes.

Timber frame walls without infill have a predominantly shear response but lower load capacity as well as lower values of seismic performance parameters. Without infill, the walls dissipated less energy and had lower values of ductility. The absence of infill reduced the stiffness of the frame, which deformed more and experienced heavier damages. In practice, infill, or at least wooden planks or boards, are always present to allow for some confinement.

For the three types of walls used in the experimental campaign, an increase of the vertical pre-compression level led to an increase of stiffness, load carrying capacity and dissipative capacity, meaning that the vertical load acting on this type of walls influences the lateral behaviour, being an important factor to be considered in a possible change of use of the buildings where they are present.

The presence of infill still influences the behaviour of retrofitted walls, but not in the same level as in unreinforced tests, since now the retrofitting has a prevailing role. Retrofitting is
applied at the level of connections and thus the alteration of their strength and stiffness governs the response of the wall.

Retrofitting with bolts was able to restore half-timbered walls to their original condition and improve the post-peak behaviour, but it did not increase the strength and stiffness of the walls. This kind of retrofitting is appropriate for a wall with infill, but in a simple timber frame it will not be effective, as it would not prevent the failure modes observed in unreinforced walls, i.e. tearing off of the connections.

The retrofitting technique with steel plates greatly increased the stiffness of the walls, particularly when the diagonal elements were linked to the main frame. In case of timber frame walls, this solution led to out-of-plane deformations and, thus, it is not recommended to link the diagonals to the main frame. In case of timber frame walls with masonry infill, it constitutes an appropriate strengthening solution, both simple and reversible. It is easily implemented and it successfully prevents the rocking behaviour of the walls. Steel plates and hold down anchors are commonly used in the connections of modern timber frame walls and in reconstruction works of traditional timber frame houses.

On the other hand, NSM steel flat bars strengthening was best suited for timber frame walls, since infill hinders the deformation capacity of the bars. Among the different configurations, the adoption of two crossed bars welded with a half-lap joint was the most appropriate. This was also confirmed by the connections tests, which showed that properly linking the two bars can even overcome anchorage length problems. The solution is invasive and not reversible, but it is potentially invisible. It is currently being used for strengthening of timber beams in modern structures, but its extension to walls is also possible.

All retrofitting techniques ensured a higher dissipative capacity for the walls, with similar values for half-timbered and timber frame walls. Moreover, the retrofitting solutions adopted hindered the influence of the vertical load level.

Studying in detail the behaviour of a single connection, in particular the half-lap joints of the bottom beam, it was seen that they present little to no resistance to vertical uplifting forces. Different solutions can be adopted, with varying stiffening effects on the connections. Even during the tests on the walls, steel plates proved to be able to successfully strengthen the connections, guaranteeing a good ductility to the connection. NSM steel flat bars with the crossed bars configuration is another successful solution. Even a simple solution such as self-tapping screws proved to be highly efficient without excessively increasing the stiffness of the connection, being useful if only a local intervention to prevent uplift is to be performed. Retrofitting with GFRP sheets can be also an alternative solution to prevent uplifting, but possibly multi-axial sheets should be used to improve their response to lateral actions.
In terms of repair of damages to timber elements, in order not to substitute an entire element, the damaged part can be taken out and substituted with a prosthesis. The choice of the prosthesis has to be made with care. The prosthesis adopted for connection tests was not able to restore the continuity of the element and it created a weak point in the post, transferring the failure position. A bigger prosthesis would have been necessary, with a longer overlapping and a higher number of screws in order to prevent the creation of a hinge during the tests. Otherwise, a prosthesis with glued-in steel rods could have been adopted, as normally done in restoration works.

9.1.2 Analytical and numerical modelling

An analytical hysteretic model calibrated on the obtained experimental results has been proposed. With the calibrated parameters, the model is able to well represent the cyclic response of traditional timber frame walls, being able to take into account its possible flexural response, as well as pinching and strength and stiffness degradation. This model can be implemented in a finite element software in order to model the hysteretic response of the walls.

In order to be able to represent the wall in a simplified way, a 2-D model was preferred to a detailed 3-D model, even simulating the connections in a simple way. A good correlation was found between the experimental results and the calibrated numerical model, for both infill and timber frame walls. The response of the model was mainly governed by its connections, in particular the stiffness of the bottom connections.

From the parametric analyses carried out it was found that post continuity played a major role on the response of the wall, as a stiffer bottom connection led to higher values of strength and stiffness.

Infill properties and connectivity between infill and timber also constitute an important influencing parameter. Masonry infill increases the initial stiffness and the strength of the wall. Moreover, if the connectivity between infill and timber is improved, higher values of strength and stiffness are obtained relatively to the case where lower resistance of the interface is adopted.

Concerning the geometry of the wall, an increasing height to length ratio leads to lower values of strength and stiffness for the wall.

9.2 Future developments

Aiming at better characterizing the seismic performance of timber frame walls, further developments can include:
• Experimental study of timber frame walls with different types of cells and with openings. It was seen how a great variety of geometries and cross-sectional dimensions exist. Further experimental studies would capture the influencing parameters as well as would be able to allow a comparison between timber frame walls of different countries;

• Execution of shaking table tests on a gaiola structure, in order to capture the real seismic response of the structure;

• Experimental study of traditional connections taking into account the influence of the diagonal elements, as they play an important role in the response of the wall;

• Experimental study of strengthened connections with an improved prosthesis, in order to improve the obtained results;

• Implementation of the hysteretic model proposed into a finite element software in order to model the cyclic response of the wall;

• Further parametric analyses in order to compare additional variables, such as cross-sectional dimension of timber elements and type of strengthening and vertical load;

• Simulation of the seismic behaviour of Pombalino buildings taking into account the presence of the timber frame walls through nonlinear static (pushover) and nonlinear dynamic analyses. This analysis can be extended also to timber frame buildings.
REFERENCES


Audel T. (1923). Audel's Carpenter's and Builder's. 72 Fifth AVE, New York, USA


EN 408 (2003). Timber structures – Structural timber and glued laminated timber – Determination of some physical and mechanical properties. CEN, European Committee for Standardization, Brussels


Gerardin M., Negro P. (2000). The European Laboratory for Structural Assessment(ELSA) and its Role for the Validation of European Seismic Codes. In Proceedings of


References


References


APPENDIX A  EXPERIMENTAL PROGRAM:
DETAILS ON CYCLIC TESTS
ON UNREINFORCED WALLS

A.1 Test setup, instrumentation and procedure

Table A.1 - Displacement history applied to wall specimens.

<table>
<thead>
<tr>
<th>Step</th>
<th>Nº of cycles</th>
<th>Displacement</th>
<th>Amplitude</th>
<th>Frequency</th>
<th>Period</th>
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<td>91.09</td>
<td>0.0010</td>
<td>1041.02</td>
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</table>
A.2 In-plane cyclic tests on unreinforced half-timbered walls: hysteretic response

Figure A.1 – Test repetition of half-timbered walls: (a) lower vertical load level; (b) higher vertical load level.

A.3 In-plane cyclic tests on unreinforced half-timbered walls: deformation capacity

Figure A.2 - Diagonal displacement in all UIW25 walls: (a) UIW2; (b) UIW3; (c) UIW7
Figure A.3 – Diagonal displacement of UIW50 walls: (a) Wall UIW4; (b) UIW6; (c) UIW8

Figure A.4 - Diagonal displacement of lath and plaster walls: (a) lower vertical load level; (b) higher vertical load level.
**Experimental program: details on cyclic tests on unreinforced walls**

Figure A.5 - Diagonal displacement of timber-frame wall, lower vertical load level.

Figure A.6 - Horizontal displacement at mid height of UIW25 walls: (a) UIW3; (b) UIW7
Figure A.7 - Horizontal displacement at mid height of UIW25 walls: (a) UIW4; (b) UIW5; (c) UIW6; (d) UIW8

Figure A.8 - Horizontal displacement at mid height of lath and plaster walls: (a) lower vertical load level; (b) higher vertical load level
Experimental program: details on cyclic tests on unreinforced walls

Figure A.9 – Horizontal displacement at mid height of timber frame walls: (a) displacement vs. force higher vertical load level; (b) Top displacement vs. mid height displacement lower vertical load level; (c) top displacement vs. mid height displacement higher vertical load level.

In the case of UIW25, some alterations were made in the positions of the LVDTs. In Figure A.10 the results are shown for individual connections with the indication on the graph of the altered position of the LVDT.

Figure A.10 - Opening of central connections for UIW25 walls: (a) UIW2; (b) UIW2
Figure A.11 – Vertical opening of contact connections for UIW25 walls: (a) UIW2; (b) UIW3; (c) UIW7
Experimental program: details on cyclic tests on unreinforced walls

Figure A.12 – Horizontal opening of nailed connections for UIW25 walls: (a) UIW2; (b) UIW7; (c) UIW7
(a)

(b)

(c)

(d)

(e)

(f)

(g)

(h)
Figure A.13 - Opening of central connections for UIW50 walls: (a) UIW5; (b) UIW5; (c) UIW6; (d) UIW6; (e) UIW6; (f) UIW8; (g) UIW8; (h) UIW8

Figure A.14 – Horizontal opening of connections for UIW50 walls: (a) UIW6; (b) UIW6; (c) UIW4; (d) UIW8; (e) UIW8
Figure A.15 – Vertical opening of contact connections for UIW50 walls: (a) UIW5; (b) UIW6; (c) UIW4; (d) UIW4
Experimental program: details on cyclic tests on unreinforced walls

Figure A.16 – Vertical opening of half-lap connections in central beam for UIW50 walls: (a) right connection; (b) middle connection; (c) left connection
Figure A.17 - Opening of central connections for lath and plaster walls: (a-b) lower vertical load level; (c) higher vertical load level
Experimental program: details on cyclic tests on unreinforced walls

Figure A.18 – Horizontal opening of contact connections for lath and plaster walls: (a-b) lower vertical load level; (c) higher vertical load level
Figure A.19 - Horizontal opening of contact connections for lath and plaster walls: (a) lower vertical load level; (b) higher vertical load level

Figure A.20 - Opening of central connections in timber frame walls: (a-b) lower vertical load; (c-d) higher vertical load
Figure A.21 – Horizontal opening of contact connections in timber frame walls: (a-b) lower vertical load; (c) higher vertical load

Figure A.22 - Opening of connections in timber frame walls: (a) lower vertical load; (b) higher vertical load
A.4 Envelope curves

Figure A.23 - Envelope curves of tested walls: (a) UIW25 walls; (b) UIW50 walls; (c) lath and plaster walls; (d) timber frame walls
Experimental program: details on cyclic tests on unreinforced walls
APPENDIX B  EXPERIMENTAL PROGRAM: DETAILS ON CYCLIC TESTS ON RETROFITTED WALLS

B.1 Strengthening calculations

In the first part of this annex the calculation of the retrofitted connections with the bolts, steel plates and steel bars (NSM technique) is presented and it aims at: (1) comparing the resistance performance of each technique; (2) understanding in qualitative terms the impact of the retrofitted connections in the lateral resistance of the walls, where the techniques are applied.

In Eurocode 5 (2004), section 8 gives the guidelines for the calculation of the resistance of connections with metal fasteners. Here, indications can be found both for timber-to-timber connections with metal fasteners (for example bolts) and for steel-to-timber connections, as can be the case of connections with steel plates.

B.1.1 Bolts strengthening

Section 8.5 of EC5 takes into consideration bolted connections, giving specifications for the characteristic values of the yield moment, embedment strength as well as minimum spacing and end/edge distances for the bolts.

Considering the dimension of the bolt (10mm diameter) and the dimensions of the timber elements (80mm and 160mm), it was decided to apply one bolt in each connection, even in the bottom ones, where the sectional dimensions were greater. This choice was made because, according to EC5, the following specifications had to be followed: (1) minimum spacing in parallel and perpendicular directions to the grain. Considering Table B.1, which indicates the minimum distances according to EC5, and Figure B.1, which explains the terminology of the afore mentioned table, minimum spacing in the direction parallel to the
grain should be assured, namely \( a_1 = (4 + |\cos \alpha|) \cdot d \), where \( \alpha \) is the angle between the force and the grain direction and \( d \) is the bolt diameter. In this case the value of \( \alpha \) is 0°, therefore \( a_1 = 50mm \). The spacing perpendicular to the grain, with an angle of 90°, would require a spacing of \( a_2 = 4 \cdot d = 40mm \), which corresponds to half of the sectional dimension of the timber elements, thus it is possible to have only one bolt along the direction perpendicular to the grain in the posts and in the beams at mid height of the wall; (2) apart from spacing parallel to the grain, edge and end distances are also important. In the case studied, the edge distances are superfluous, since, as already seen, from the minimum distances for spacing, only one bolt will be used at the centre of the connection and, thus, even considering the maximum edge distance required by EC5 \( (a_{4,t} = 4 \cdot d = 40mm, \) considering \( \alpha = 90^\circ \) ) is verified. For end distances, when considering a loaded end, the required distance is \( a_{3,t} = \max(7 \cdot d; 80mm) = 80mm \), while for the unloaded end \( a_{3,c} = 4 \cdot d = 40mm \), considering \( \alpha_{3,c} = 180^\circ \). These distances apply both for the bottom and top connections.

Table B.1 - Table 8.4 of EC5: Minimum values of spacing and end and edge distances for bolts

<table>
<thead>
<tr>
<th>Spacing and end/edge distances (see Figure 8.7)</th>
<th>Angle</th>
<th>Minimum spacing or distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a_1 ) (parallel to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( (4 +</td>
</tr>
<tr>
<td>( a_2 ) (perpendicular to grain)</td>
<td>( 0^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 4 \cdot d )</td>
</tr>
<tr>
<td>( a_{3,t} ) (loaded end)</td>
<td>(-90^\circ \leq \alpha \leq 90^\circ )</td>
<td>( \max(7 \cdot d; 80mm) )</td>
</tr>
<tr>
<td>( a_{3,c} ) (unloaded end)</td>
<td>( 90^\circ \leq \alpha &lt; 150^\circ )</td>
<td>( 4 \cdot d )</td>
</tr>
<tr>
<td></td>
<td>( 150^\circ \leq \alpha &lt; 210^\circ )</td>
<td>( \max(4 \cdot d; 6 \cdot \sin \alpha \cdot d; 4d) )</td>
</tr>
<tr>
<td></td>
<td>( 210^\circ \leq \alpha &lt; 270^\circ )</td>
<td>( \max(4 \cdot d; 6 \cdot \sin \alpha \cdot d; 4d) )</td>
</tr>
<tr>
<td>( a_{4,t} ) (loaded edge)</td>
<td>( 0^\circ \leq \alpha \leq 180^\circ )</td>
<td>( \max(2 \cdot \sin \alpha \cdot d; 3d) )</td>
</tr>
<tr>
<td>( a_{4,c} ) (unloaded edge)</td>
<td>( 180^\circ \leq \alpha \leq 360^\circ )</td>
<td>( 3 \cdot d )</td>
</tr>
</tbody>
</table>
Considering the distances required by EC5 and above mentioned, it was decided to apply only one bolt in each half-lap connection. In fact, applying two bolts at the bottom connection would require a spacing of 50mm between each other, an edge distance to the beam of 40mm and an end distance of 80mm when the element is in tension, which makes the use two bolts impossible. Figure B.2 illustrates the solution adopted for strengthening with bolts.

The bolted connection thus obtained has a single shear plane and could behave according to the failure modes given in Figure B.3. The characteristic load carrying capacity per shear plane and per fastener is calculated, according to EC5, as:
The characteristic load-carrying capacity per shear plane per fastener is denoted as $F_{v,Rk}$, the timber or board thickness or penetration depth as $t_i$, the characteristic embedment strength in timber member $i$ as $f_{h,i,k}$, the fastener diameter as $d$, the characteristic fastener yield moment as $M_{y,Rk}$, and the ratio between the embedment strength of the members as $\beta$. The characteristic axial withdrawal capacity of the fastener is denoted as $F_{ax,Rk}$.

The axial load-bearing capacity and withdrawal capacity of a bolt should be taken as the lower value of (1) the bolt tensile capacity and (2) the load-bearing capacity of the washer. The values in Table B.2 have been assumed for the calculations. The thickness $t_1$ is the thickness of the post in the connections, whose grain has an angle of 0° with the load direction, while $t_2$ is the thickness of the beam in the connections, whose grain has an angle of 90° with the load direction.
Table B.2 - Values assumed for calculations

<p>| | | | | |</p>
<table>
<thead>
<tr>
<th></th>
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<th></th>
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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$t_1$</td>
<td>60</td>
<td>mm</td>
<td>$f_{h,1,k}$</td>
<td>43.53</td>
</tr>
<tr>
<td>$t_2$</td>
<td>60</td>
<td>mm</td>
<td>$f_{h,2,k}$</td>
<td>29.03</td>
</tr>
<tr>
<td>$\alpha_1$</td>
<td>0</td>
<td>°</td>
<td>$\beta$</td>
<td>0.67</td>
</tr>
<tr>
<td>$\alpha_2$</td>
<td>90</td>
<td>°</td>
<td>$M_{y,Rk}$</td>
<td>95545.72</td>
</tr>
<tr>
<td>$d$</td>
<td>10</td>
<td>mm</td>
<td>$F_{ax,Rk}$</td>
<td>11.66</td>
</tr>
</tbody>
</table>

Thus, the characteristic load carrying capacity per shear plane and per fastener for the bottom connections, considering mainly the uplifting forces in order to enable the evaluation of the load angle, is, according to EC5:

$$F_{v,Rk} = \min\left\{ \frac{26.12}{a}, \frac{17.42}{b}, \frac{11.88}{c}, \frac{12.82}{d}, \frac{11.35}{e}, \frac{12.30}{f} \right\} = 11.35 kN$$

This is an indicative value, as only the uplifting forces have been taken into account. In any case, it can give an idea about the resistance of the bottom connection to resist against the uplift displacements. It should be noticed that the response of the wall with bolts retrofitting is a combination of flexure and shear.

**B.1.2 Steel plates strengthening**

The spacing and end and edge distances calculated above for the bolts are also valid for the steel plates strengthening, to which the bolts are applied. However, sometimes the limitation of the geometry of the steel plates and of the wall did not make possible to always comply with the regulations.

According to section 8.7 of EC5, for smooth shank screws with a diameter smaller than 6mm (as is the case of the screws used for this retrofitting technique), the rules for nailed connections apply. According to table 8.2 of EC5, the minimum spacing of screws should be $7d = 7 \cdot 5mm = 35mm$ for spacing perpendicular to the grain. For spacing parallel to the grain, considering an angle of application of load of 0° in relation to the grain, which is admissible for the members, since we can consider the elements of the frame as trusses, a minimum distance of $15d = 15 \cdot 5mm = 75mm$ is recommended. This minimum distance was not always respected, due to the small sectional dimensions of the elements, both timber and steel plates. At the first test, the minimum distances were respected, but it was seen that, nevertheless, most of the screws failed in shear and at the end of the test no screws were present in the most stressed connections. Therefore, the number of screws was increased, keeping in mind the effective number of screws. In the case of commercial steel plates, minimum distances were not always respected in order to avoid the need of using three
combined steel plates for one connection, instead of two. The use of two connected steel plates, could give continuity problems, and thus three plates were avoided.

Steel plates strengthening constitutes a typical steel-to-timber connection and its characteristic load-carrying capacity depends on the thickness of the steel plates (EC5, 2004). A steel plate is considered thin if its thickness if equal or less than \(0.5d\), which in the case in exam is verified, since \(t_p = 2mm < 0.5 \cdot 10mm = 5mm\). Even if the diameter of the screws is considered (5mm), the relation above still verifies the thin plate condition.

Figure B.4 shows the disposition of bolts and screws in the connection strengthened with steel plates. Notice that for custom steel plates no screws were applied (Figure B.4a). After buckling of the plates was observed during the tests, it was decided to additionally use screws in order to better distribute the stresses. Thus, commercial steel plates were chosen and the holes adapted to accommodate the necessary bolts (Figure B.4b,c,d). Two plates were juxtaposed and rotating according to necessities for each connection.

Figure B.4 - Steel plates strengthening: (a) bolts disposition in custom steel plates; (b) bolts and screws disposition in commercial steel plates, central connection linking the diagonals; (c) bolts and screws disposition in commercial steel plates, bottom and top connection linking the diagonals; (d) bolts and screws disposition in commercial steel plates, central connection without linking the diagonals.
For the case in study, thin steel plates as the outer members of a double shear connection (see possible failure modes in Figure B.5a), the characteristic load-carrying capacity for screws and bolts per shear plane per fastener is given by the minimum value found equation 8.12 of EC5:

\[
F_{V,Rk} = \min \left\{ 0.5 f_{h,2,k} t_2 d, \frac{1.15}{1.2} \sqrt{2 M_{y,Rk} f_{h,2,k} d} + \frac{F_{ax,Rk}}{4} \right\}
\]

To apply this equation, it has to be assumed that only the bolts are considered for the calculations (the screws do not connect one plate to the other) and the post is considered continuous. In this case, since the bolts link the two elements together, it is a reasonable assumption, since the bolt allows the connection to behave in a monolithic way.

![Figure B.5 - Failure modes for steel-to-timber with thin steel plate: (a) outer members of double shear connections; (b) in single shear](image)

To perform the calculations, the following values were assumed:

- \( f_{h,2,k} \): 43.54 N/mm²
- \( d \): 10 mm
- \( t_2 \): 120 mm
- \( M_{y,Rk} \): 95545.72 Nmm
- \( F_{ax,Rk} \): 11663.16 N
- \( p_k \): 590 kg/m³
- \( f_{u,k} \): 800 N/mm²
- \( k_{90} \): 1.5 (depends on type of wood, see eq. 8.33 of EC5)
- \( \alpha \): 0 rad
- \( n_{ef} \): 2.54
- \( N \): 3 (number of bolts in parallel row)
- \( k_{ef} \): 0.85 (Table 8.1 of EC5)
- \( t_{min} \): 147.5 mm
- \( t_{plate} \): 2 mm

To calculate the contribution of axial load on the bolts, as done in section B.1.1:
Experimental program: details on cyclic tests on retrofitted walls

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{t,bolt}$</td>
<td>62831.85</td>
<td>N</td>
<td>Tensile capacity of bolt</td>
</tr>
<tr>
<td>$F_{u,washer}$</td>
<td>11663.16</td>
<td>N</td>
<td>Load bearing capacity of washer</td>
</tr>
<tr>
<td>$f_{c,90,k}$</td>
<td>5.5</td>
<td>N/mm$^3$</td>
<td>Characteristic compressive strength of timber, perpendicular to the grain</td>
</tr>
<tr>
<td>$d_w$</td>
<td>30</td>
<td>mm</td>
<td>Diameter of contact area of washer</td>
</tr>
</tbody>
</table>

With these values, the characteristic load-carrying capacity per bolt and per shear plane is:

$$F_{v,Rk} = \min \left\{ \frac{26.13}{13.41}, \frac{j}{k} = 13.41kN \right\}$$

Considering the effective characteristic load-carrying capacity parallel to the row:

$$F_{v,Rk} = n_{ef} \cdot F_{v,Rk} = 34.11kN$$

To perform the same calculations for the contribution of the screws, each steel plate on the two sides of the wall separately should be considered, considering a steel-to-timber connection with a thin steel plate in single shear (Figure B.5b). For such a connection, the characteristic load-carrying capacity for screws per shear plane per fastener is the minimum value given by the following equation:

$$F_{v,Rk} = \min \left\{ \frac{0.4f_{h,k}t_zd}{1.15}, \frac{2M_{y,Rk}f_{h,k}d}{1.15} + \frac{F_{ax,Rk}}{4} \right\}$$

To perform the calculations, the following values were assumed:

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Value</th>
<th>Unit</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{h,k}$</td>
<td>29.85</td>
<td>N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>$d$</td>
<td>5</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td>$t_1$</td>
<td>50</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td>$M_{y,Rk}$</td>
<td>7879.583</td>
<td>Nmm</td>
<td></td>
</tr>
<tr>
<td>$F_{ax,Rk}$</td>
<td>97024.67</td>
<td>N</td>
<td></td>
</tr>
<tr>
<td>$\rho_k$</td>
<td>590</td>
<td>kg/m$^3$</td>
<td></td>
</tr>
<tr>
<td>$f_u$</td>
<td>400</td>
<td>N/mm$^2$</td>
<td></td>
</tr>
<tr>
<td>$n_{ef}$</td>
<td>5.01</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>$n$</td>
<td>10</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>$k_{ef}$</td>
<td>0.7</td>
<td>-</td>
<td>(according to Table 8.1 of EC5)</td>
</tr>
<tr>
<td>$t_{min}$</td>
<td>51.625</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td>$t_{plate}$</td>
<td>2</td>
<td>mm</td>
<td></td>
</tr>
<tr>
<td>$n_{tot}$</td>
<td>20</td>
<td>-</td>
<td>(total number of nails on each side)</td>
</tr>
</tbody>
</table>

To calculate the contribution of axial load on the bolts, as done in section B.1.1:

$$f_{ax_{-},k} = 51.59 \text{ N/mm}^2$$
f_{ax,k} \quad 51.59 \quad \text{N/mm}^2 \\
\alpha \quad 1.571 \quad \text{rad (angle between withdrawal direction and grain)} \\
l_{ef} \quad 45 \quad \text{mm (effective screw length)} \\
l \quad 50 \quad \text{mm (screw length)} \\
n_{ef} \quad 14.82 \quad \text{(effective number of screws for axial loads)}

With these values, the characteristic load-carrying capacity per screw and per shear plane is:

\[ F_{v,Rk} = \min \left( \frac{2.99}{38.15} a \quad b = 13.41 \text{kN} \right) \]

Considering the effective characteristic load-carrying capacity parallel to the row:

\[ F_{v,Rk} = n_{ef} \cdot F_{v,Rk} = 14.96 \text{kN} \]

Once again, these values have to be taken with care, simplifications have been made. Moreover, notice that the screws of the discontinuous element always failed in shear, with their heads cut-off, so their contribution was not present up to failure of the connection.

It should be noticed that the connections with steel plates give considerably more strength, even only considering the bolts for the calculations, than the bolted connections, particularly if the contribution of the bolts and screws are simultaneously considered. This confirms the lower uplift experienced by the walls, because of the additional strength and stiffness given by the strengthening.

### B.1.3 NSM steel flat bars strengthening – estimation of the tensile strength

For NSM techniques the EC5 does not apply directly, but usually the application of strengthening with this technique is based on experimental results from literature. NSM strengthening has been applied to timber only in recent years. Studies have been performed by Jorge (2010), studying the bond behaviour between glulam and FRP and then applying FRP strips with the NSM technique to continuous double span glulam slabs and testing them. From the analysis of the tests, the author suggested that an anchorage length of 15d should be used. The same anchorage length is suggested by other authors. For FRP strips, good performances were found for bond length of 7.5 times the height of the strip (De Lorenzis and Teng, 2007) for concrete structures.

For the bars used, it was decided to adopt an anchorage length of at least 20cm from the half-lap connection. In order to have sufficient strength, a high performance steel was used, CK45, with a tensile strength between 600 and 800MPa, and from tensile tests performed in Chapter 3, a value of 672.87MPa was obtained. The bars had a section of 8×20mm², so the ultimate tensile force that they could withstand was of 108kN per bar.
Experimental program: details on cyclic tests on retrofitted walls

For the first walls tested two layers of bars were used, in order to assure a high strength to the connections. Two elements were embedded when the element was discontinuous and one perpendicular element was inserted as a second layer (Figure B.6a) in a deeper slot (the depth was approximately 23mm for the first layer and 45mm for the second layer). For the bottom connections, as there was no sufficient anchorage length for the vertical bar (being the height of the beam 16cm) and since it was difficult to cut a slot till the bottom of the beam (the slots were opened with the wall in a vertical position and the plunge router could cut starting from a height of 6cm from the bottom), it was decided to bend the bar and fill the corner with solder (Figure B.7a).

![Figure B.6 - Disposition of flat bars in walls: (a) infill walls with double depth disposition; (b) timber frame walls with notched bars disposition](image)

The choice of having two bars when the element was discontinuous (Figure B.7a,b) was made to guarantee a continuity to the element in order to avoid openings of the connections.

As after the first tests, the welding at the bottom connection failed it was decided to apply at the bottom connection steel plates, similarly to what was done in Figure B.4d, and reduce the number of bars, in order to have greater deformations in the wall. In particular, it was decided to apply only one layer of bars and only one bar in each connection, in the surface where the element was discontinuous (Figure B.7c).

For timber frame walls, since the absence of infill led to a greater deformation of the timber member, it was decided to use a bar for each element of the connection on both sides of the wall. In order to avoid to have two layers of bars, they were instead linked together creating a half-lap connection (Figure B.6b) and then welded together (Figure B.7d) to improve the anchorage length.
Figure B.7 - Dimensions of NSM strengthening adopted for each wall: (a) bottom connection of wall RIW25_S, non-continuous post side; (b) mid height connection of RIW25_S wall, non-continuous beam side; (c) mid height and top connections of wall RIW50_S, discontinuous post side; (d) mid height and top connections of walls RTW_S.
B.2 In-plane cyclic tests on unreinforced half-timbered walls: deformation capacity

Figure B.8 - Diagonal displacement in retrofitted walls with lower vertical load level: (a) RIW25_B; (b) RIW25_P; (c) RTW25_P; (d) RTW25_P_M; (e) RIW25_S; (f) RTW25_S.
Figure B.9 - Horizontal displacement at mid height of retrofitted walls: (a) RIW50_B; (b) RTW25_P_M; (c) RIW25_P; (d) RTW50_P; (e) RIW25_S; (f) RTW25_S.

In the following figures, the openings of the central connections are shown. For the walls not presented, the openings were either less than 0.5mm or the LVDT did not work properly (as is the case of wall RIW25_B).
Experimental program: details on cyclic tests on retrofitted walls

(a) RIW50_B
(b) RIW25_P
(c) RIW25_P
(d) RIW50_P
(e) RIW50_P
(f) RTW25_P_M
(g) RTW25_P_M
(h) RTW25_P_M

Horizontal opening [mm] vs Horizontal displacement [mm]

RIW50_B
RIW25_P
RIW25_P
RIW50_P
RIW50_P
RTW25_P_M
RTW25_P_M
RTW25_P_M
Figure B.10 - Opening of central connections: (a) RIW50_B; (b) RIW25_P; (c) RIW25_P; (d) RIW50_P; (e) RIW50_P; (f) RTW25_P_M; (g) RTW25_P_M; (h) RTW25_P_M; (i) RTW50_P_M; (j) RTW50_P_M; (k) RIW50_S; (l) RTW50_S; (m) RTW50_S; (n) RTW50_S.
Experimental program: details on cyclic tests on retrofitted walls
Figure B.11 – Detachment of diagonals from main frame: (a) RIW25_B; (b) RIW25_B; (c) RTW25_P_M; (d) RTW25_P_M; (e) RTW50_P_M; (f) RTW50_P_M; (g) RIW25_S; (h) RIW50_S; (i) RIW50_S; (j) RTW25_S; (k) RTW25_S; (l) RTW25_S; (m) RTW50_S; (n) RTW50_S
Experimental program: details on cyclic tests on retrofitted walls

(a) RIW25_P
(b) RIW25_P
(c) RIW50_P
(d) RIW50_P
(e) RIW50_P
(f) RIW50_P
(g) RTW25_P
(h) RTW25_P
Figure B.12 – Opening of half-lap connections in diagonal elements: (a) RIW25_P; (b) RIW25_P; (c) RIW50_P; (d) RIW50_P; (e) RIW50_P; (f) RTW25_P; (g) RTW25_P; (h) RTW25_P; (i) RTW50_P; (j) RTW50_P; (k) RTW25_P_M; (l) RTW50_P_M.
Experimental program: details on cyclic tests on retrofitted walls

(a) 
(b) 
(c) 
(d) 
(e) 
(f) 
(g) 
(h)
Figure B.13 – Strain gauges in retrofitted walls: (a-f) RTW25_P; (g) RTW25_P_M; (h-l) RTW50_P_M; (m) strain in bottom steel plate, strain gauge applied where steel plate buckled in wall RTW50_P_M
B.3 In-plane cyclic tests of retrofitted walls: envelope curves

(a) RIW25_B

(b) RIW50_B

(c) RIW25_P

(d) RIW50_P

(e) RTW25_P

(f) RTW50_P
Figure B.14 - Envelope curves of stabilization cycles: (a) RIW25_B; (b) RIW50_B; (c) RIW25_P; (d) RIW50_P; (e) RTW25_P; (f) RTW50_P; (g) RTW25_P_M; (h) RIW25_S; (i) RTW25_S; (j) RTW50_S
Experimental program: details on cyclic tests on retrofitted walls