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A COMPLEX ASSESSMENT OF HISTORIC ROOF STRUCTURES

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Abstract

Roof structures are an important part of the built heritage all around the world. In recent years, the interest for historic roof structures has risen all around Europe which led to the development of assessment methodologies which evaluate the current state and value of those structures.

However almost all historic roof structure studies, are focusing on either the structural features of the structure, their state of conservation or their aesthetic value, leaving the connection between all these features out of sight and ignoring the link between the roof structures and the buildings they are connected to. Therefore, the research is proposing a more complex analysis of historic roof structures. The study is starting from 7 buildings from a block placed on the outer side of the main historic centre of Timisoara, Romania, built at the end of the 20th century. In Timisoara, due to the lack of interest in reusing the available space hidden inside historic roof structures, they can be found mostly in their initial conformation, with only a few strengthening interventions.

The study is starting from a general point of view, trying to identify how the roofs are perceived from the outside, how they are shaping the historic block and how they are linked to each other. Furthermore, the study is shifting to a more detail based analysis that tries to identify how those principles influence the choice of structural typology and characteristics and how the chosen structural typology is influencing the general perception of the historic building.

Ultimately, the main purpose of the study is to highlight the high complexity of roof structures and that a link between all their features needs to be identified to properly assess their value.

1 INTRODUCTION

Compared to many European cities, where the value of historic roof structures is not fully appreciated [1] and roof structures are being replaced by complex, contemporary structures, in Timisoara they are mainly still intact. The main reason is the lack of interest of investors for the use of the additional space available in the city center. Therefore, Timisoara can still offer a great diversity of structural typologies in a good state of conservation, which can present new views concerning the study of historic roof structures.

Many other researchers and local authorities all around Europe acknowledged the value of historic roof structures in recent years [1–5]. The main purpose of the studies is the general identification of used typology, their history and evolution and to raise the attention of local authorities towards the hidden heritage of the cities. While the analyzed features of the roof structures may seem different in every research, all of them share the respect for this type of historic structures and the desire to preserve them for the future.

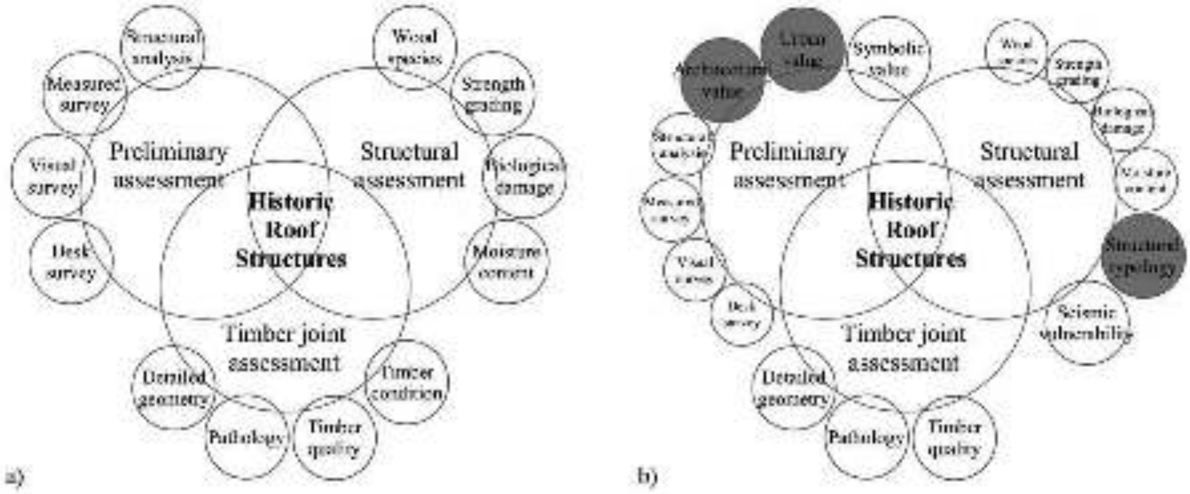

Figure 1: Assessment of historic timber roof structures – a) existing assessed criterions; b) proposed new criterions

Besides the history and typological based studies, other researchers try to define assessment methodologies, suitable for historic roof structures, that would help to properly evaluate and preserve this structures [6, 7]. Assessment methodologies are generally organized in 3 main sectors: preliminary assessment, structural assessment and timber joints assessment, with various subsections in every one of these sectors (Figure 1a). Since these assessment methodologies are only trying to determine the value and state of conservation of the roof structure without taking into consideration their connection with their context, this research is proposing additional criterions that should be included in roof structure assessment methodologies (Figure 1b).

Starting with the year 2014, various roof structures from the historic part of Timisoara, built between the end of the XVIIIth and middle of the XXth century were surveyed, as part of a complex project of the Faculty of Architecture and Urban planning. The main goal of the study is to further develop roof structures assessment methodologies by adding new criterions strongly related to the context of the roof structure and the way of thinking of craftsmen [8, 9].

Together with 5^{th} year students all the steps of existing roof structure assessment methodologies were taken into consideration while a special attention was given also to the new proposed assessment criteria: architectural and urban planning principles shaping the roof.

2 COMPLEX ASSESSMENT OF ROOF STRUCTURES IN TIMISOARA

2.1 Historic context

Timisoara is a medium sized city located in the western part of Romania. Its history begins in the XIIth century. Due to its good strategic position, the city was modified and reorganised in time by different civilisations: Hungarian, Ottoman, Habsburg, Austrian, Austro-Hungarian, Serbian and later on Romanian [10]. In the XVIIIth century, the city was conquered by the Austrian Empire and the old Ottoman fortress rebuilt in the Vauban style. The interior of the new fortress was organised after a rectangular grid of blocks with 3 main squares, with distinct functions located, between them. The main characteristic of this blocks is that the buildings were placed on the outer side of the grid, forming a continuous façade and a complex shaped inner courtyard.

Figure 2: Location of the analyzed building block

In order to properly assess how architectural and urban planning principles are influencing the choice of roof structure typology, a block located at the outmost side of the old fortress was chosen, a block that suffered continuous changes until the beginning of the XXth century (Figure 2). The block respects main characteristics of historic building blocks of the city, being formed by buildings, placed around a central courtyard, forming a continuous exterior façade. The block suffered many changes in time, changing its shape and functions according to various needs of the time [13].

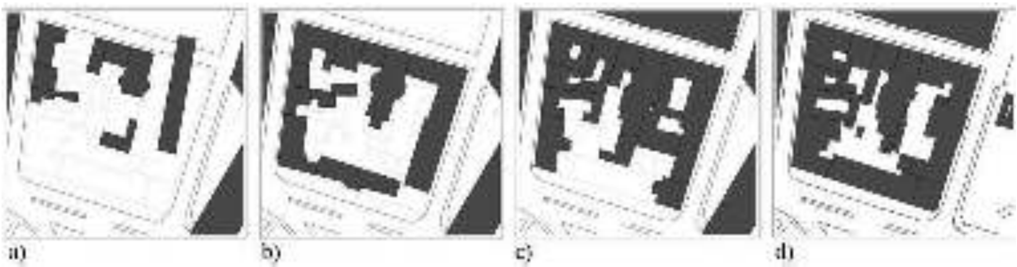

Figure 3: Urban evolution of the block: a) early XVIII century; b) middle of the XVIIIth century; c) middle of the XIXth century; d) today

The first building that was built in this block was the St Kathrine church at the middle of the XVIIIth century, built in the northern part of the block. Additional 2 other buildings were built, on the northern and eastern side, a boy's school and Sisters of Notre Dame School, both directly connected with the church (Figure 3a). Soon after that, the other sides of the block were filled up with other buildings (Figure 3b), with various functions, from residential to public, and the block was soon completely closed. The biggest and final changes of the block occurred at the end of the XIXth century, when the old fortress was turned down so that the old city could connect with the neighbourhoods developed in its proximity. Therefore, major changes were made in the block: the church was rebuilt, the boys school and Sisters of Notre Dame School were demolished (Figure 3c) and a new school was built in the same place (Figure 3d).

2.2 Roof structures with urban value

2.2.1 Case study 1 – Hotel Victoria

Built on the north-western corner of the grid, in the middle of the XVIIIth century, the building used to be an inn, without closing the western front of the block entirely. Later, at the beginning of the XXth century the inn was torn down, and a new building in the 1900 Secession style, specific for Timisoara, was built (Figure 4). The main feature of the building is the round tower highlighting the corner of the building and of the block. Above this tower a cone-shaped roof was placed. The building has a "U" shape with 2 wings towards the street, covered by a gable roof, and a secondary wing facing the inner courtyard, covered by a shed roof.

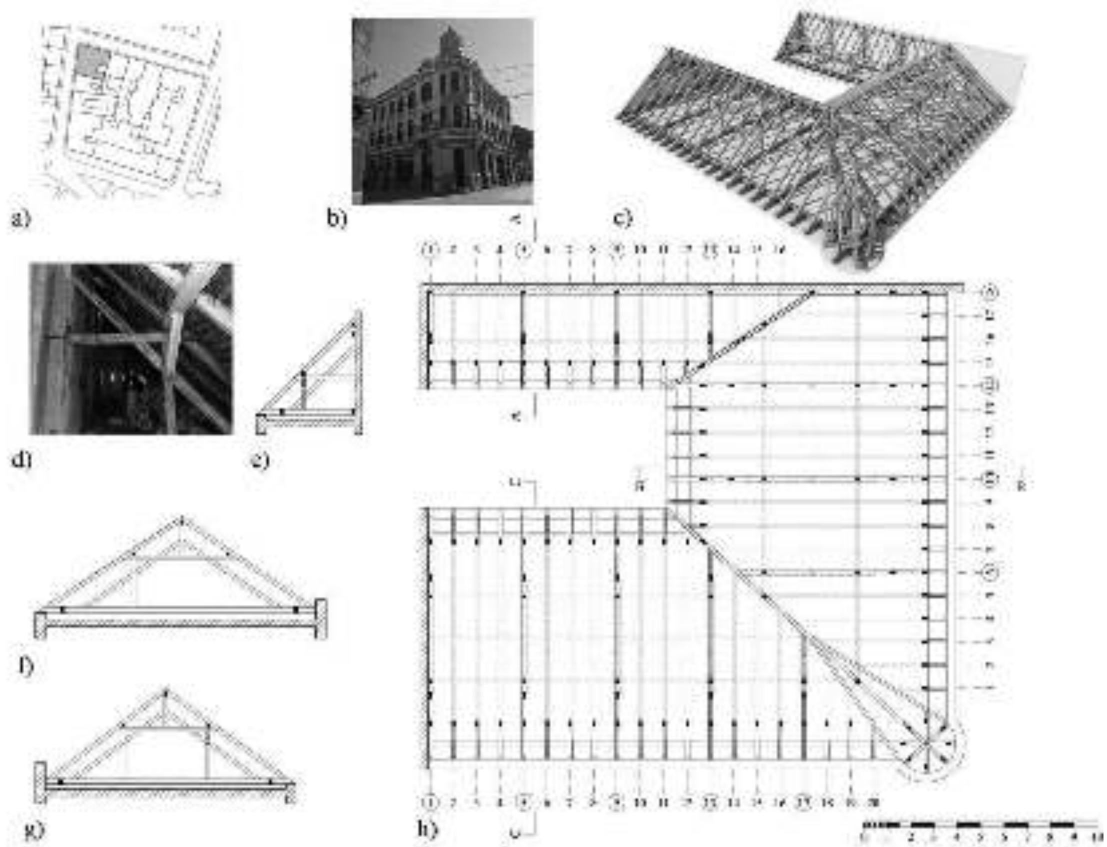

Figure 4: Case study 1 – Hotel Victoria: a) position of the building in the block; b) street view; c) axonometric view; d) interior view; e) A-A cross section; f) B-B cross section; g) C-C cross section; h) plan view.

Unfortunately, in this case, the space inside the main roof structure was partially transformed into rooms. Therefore, only a measured survey and complete assessment of the shed roof could be made. Elements of the main frame of the gable roof were still visible so they were assessed also, and the general typology of the roof structure could be identified. The used roof structure typology represents an eclectic king and queen post roof truss. The truss is therefore divided into two parts, with a superior king post truss and inferior queen post truss with rafters, purlins and a complex strutting-hanging device with tie-beam, straining beam and hanging post. Additional compound rafters can be found also, connecting the two main parts of the truss. For the shed roof, the same eclectic gable roof truss typology was adapted and the hanging post was moved in the ridge area. Above the corner tower of the building the same roof structure was adapted and turned around a central axe.

This roof structure shows how the same structural typology was used and adapted in various positions and shapes of the roof in order to obtain a certain image.

2.2.2 Case study 2 – I. C. Bratianu high school (Wing A)

Placed on the south-western corner of the building block, this building has one of the strongest relations to its surrounding. Despite its plain facades, without any type of decoration, the 3-storey building has a high urbanistic value due to its position, towards one of the main squares of the city and the historic "Huniade" castle. Built at the end of the XVIIIth century as an office building, it is one of the few buildings of the block which didn't suffer major changes in time, except for functional changes, the building being today a wing of the I. C. Bratianu high school (Figure 5). Because the building can be seen from far away, the roof structure has a significant role also. Since the building has 2 facades towards the street, the roof is a one-sided hip roof, with a hip end placed towards the southern façade. Still neither the building, nor the roof is standing out, when viewed from the outside.

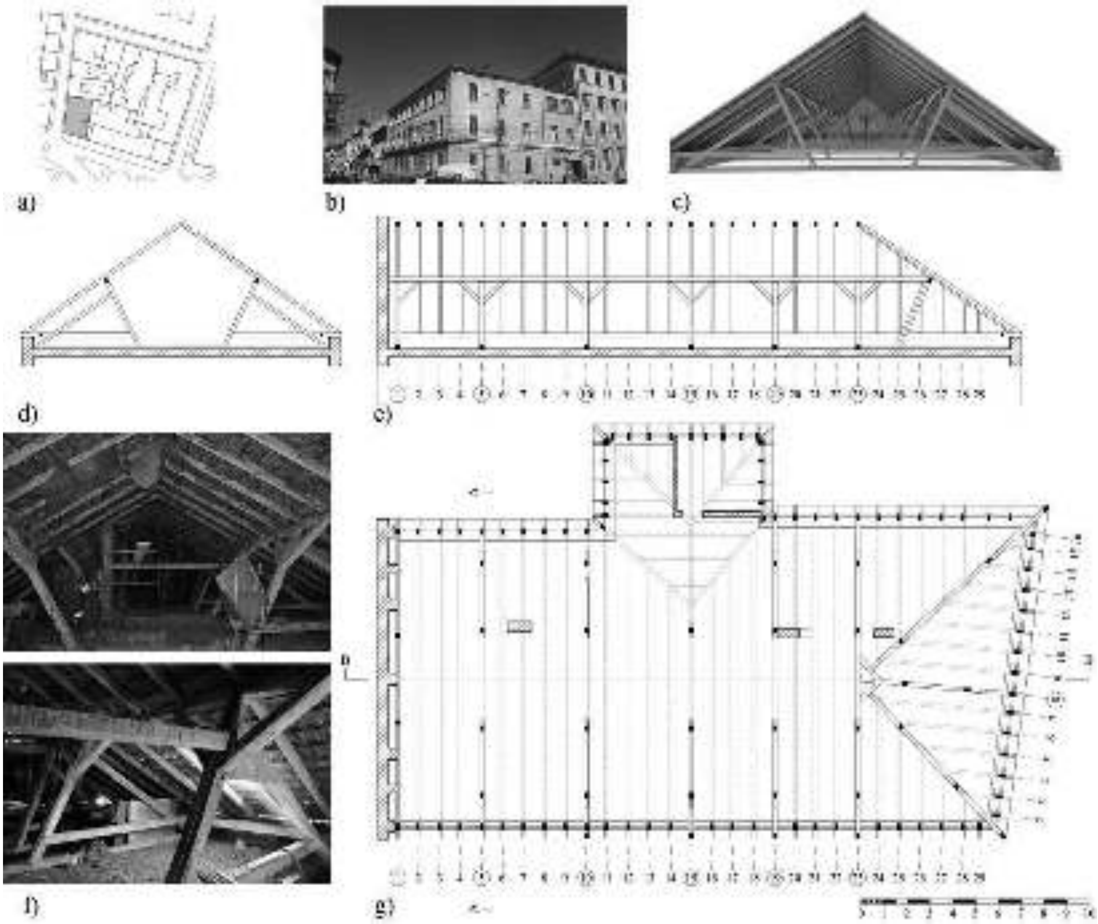

Figure 5: Case study 2 - I. C. Bratianu high school (Wing A): a) position of the building in the block; b) street view; c) axonometric view; d) A-A cross section; e) B-B cross section; f) interior view; g) plan view.

On the inside on the other hand, the roof structure is one of the most impressive roof structures of the block, because of the chosen truss typology and of the complex way to solve the hip end of the roof. The main gable roof is a queen struts purlin roof structure. The structure seems to be a transitional typology between the Romanic, angle brace rafter roof structure and the eclectic strut roof structure. One the one hand, the position of main structural elements is exactly like those of a Romanic angle brace structure. Still, the angle brace was replaced with a strut. On the other hand, elements specific for the 20th century strut purlin roof can be identified, like the compound rafter and the header. On the other hand, an additional horizontal beam connecting the rafter, strut and compound rafter was used, which is not common for neither typology. The used structural elements show that the structural typology specific for the strut purlin roof wasn't fully developed at that time of construction and craftsman were still adapting existing, known construction principles. The hip end of the roof is one of the key areas from both structural and aesthetic point of view. The hip end is connected with the last main frame of the gable roof by a complex system of tie-beams in the inferior part of the structure. In order to transfer the loads from the hip end to the gable roof, the roof structures of the two areas should be connected in the upper part also [11]. Because struts are used in this area too, no additional connection in the upper part is necessary, the loads being directly transferred to the tie-beams.

Since hip ends of historic roof structures present one of the most complex technical solutions, this structure is an example of adaptation of a known structural typology in order to comply to the shape and position of the building.

2.2.3 Case study 3 – I. C. Bratianu high school (Wing B)

Originally built in the middle of the XVIIIth century, as a 2-storey office building in an impressive baroque style, the building was torn down at the end of the XIXth century. The new building, a 4-storey building with an eclectic façade was built as a girl school of the Sisters of Notre Dame order. Today the building is still used as a school, being the main wing of the I. C. Bratianu high school (Figure 6). Like the corner building analyzed before, due to its position in the block and its height, the building is one of the key-perspective points of the area, the roof highlighting its importance.

The building is divided in two sectors, the difference between the two of them being visible even on the façade. The same point marks a change of the used roof structure typology also. The left side of the roof structure represents an eclectic king and queen post roof structure. The inferior part of the structure is a queen post like roof structure, with main characteristic elements like hanging post, straining beam and compound rafter. But there are also features that differentiate this structure from a typical queen post roof structure like the use of tongs, instead of a tie beam to connect the hanging post with the compound rafters and rafter. Hanging posts are not connected with each other in the inferior part. Because of the significant height of the roof, in the upper part a king post roof structure was used.

Additional mortise cavities mainly in the hanging posts and the compound rafters show that this roof structure used to be of a different typology but was adapted in time. It seems that additional longitudinal elements used to reinforce main frames of the structure, thus placing the original roof structure in the XVIIIth century.

The right side of the roof represents a typical example of an eclectic king and queen post roof structure with all characteristic structural elements: hanging post, straining beam, tie beam, angle braces, rafters and purlins. Like in the left part of the roof, an additional king post was inserted in the upper part of the structure, connecting the two compound rafters and forming a second hanging device.

The structure presents the way a common structural typology had to be adapted in order to satisfy different type of needs, in this case the need to raise the height of the roof and highlight the silhouette of the building.

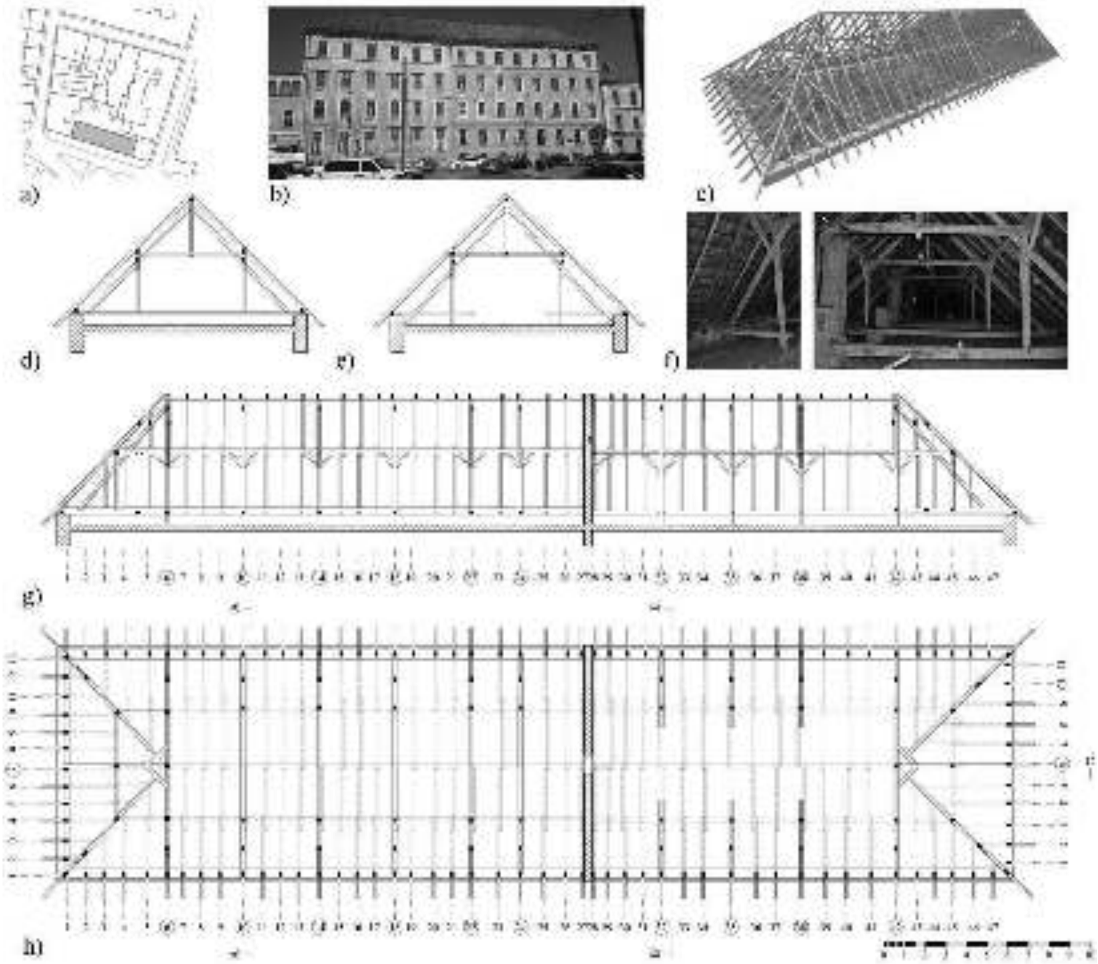

Figure 6: Case study 3 - I. C. Bratianu high school (Wing B): a) position of the building in the block; b) street view; c) axonometric view; d) A-A cross section; e) B-B cross section; f) interior view; g) C-C cross section; h) plan view.

2.2.4 Case study 4 – I. C. Bratianu high school (Wing C)

The third building of the I.C. Bratianu high school is quite similar to the first wing of the school. Placed on the south-eastern corner of the block, the building is a key perspective point when going towards one of the main squares of the city. Built at the beginning of the XIXth century, the building suffered minor changes in time, keeping more and less the same shape, although neighboring buildings kept changing. The building is a 3-storey building, with an almost square plan and a slightly chamfered corner (Figure 7). The chamfer of the corner is highlighting the importance of the building in the surrounding urban space, thus creating a strong relation between architecture and the street. The same eclectic style of the main wing of the high school can be identified here also. The roof is completely subordinated to the building, highlighting the features of the building without drawing any attention to itself.

Despite the square plan of the building, the roof structure is L shaped, resulted as the intersection between a gable roof, alongside the southern façade and a one-sided hip roof, parallel with the eastern facade. The intersection between the two structures, is one of the most complex structural solutions of the building block, mainly due to the presence of the chamfered corner.

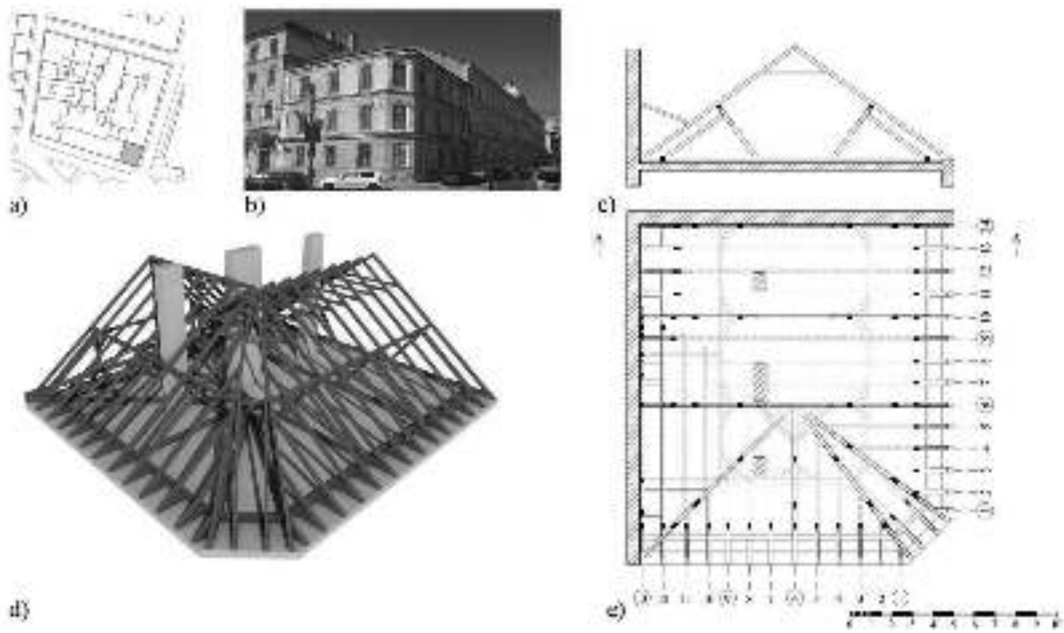

Figure 7: Case study 4 - I. C. Bratianu high school (Wing C): a) position of the building in the block; b) street view; c) A-A cross section; d) axonometric view; e) plan view.

The used structural typology is similar to the one used at the fist corner building of the high school. The same transitional typology between the Romanic, angle brace rafter roof structure and the eclectic strut roof structure was found here. The Romanic general shape of the roof structure was kept here also with additional structural elements of the 20th century strut purlin roof structure like the compound rafter and the header. In the upper part of the roof structure an upper collar beam was used instead of the typically eclectic angle brace. No additional elements were introduced in the roof structure presenting a more developed structural typology of the one used in the south-western corner building.

The intersection between the two roof structures presents typical hip end technical solutions adapted for this specific case. Tie beams and upper collar beams are connected in a complex system thus permitting the load transfer from a structure to another. The roof structure above the chamfered corner presents a solution usually used for towers or rounded church roof structures. In this case, a half of the main frame was turned 15° around a central axe, thus creating the chamfered corner roof structure.

This roof structure and all its technical solutions, present very clear how various know structural typologies had to be adapted to create a certain desired appearance.

2.3 Roof structures with architectural value

2.3.1 Case study 5 – Residential building

The analyzed building, is one of the oldest of the block. It was built as part of the old Franciscan Monastery, being located adjacent to the St. Kathrine church. The building changed its functions in time, from monastery, to normal school, until today, when the building is used as an apartment building (Figure 8). Some of the apartments still belong to the church being used as offices.

The building is in the middle of the northern side of the block, forming together with the neighboring buildings a continuous façade. Still, the building has only two floors compared to the other buildings, with 3 floors, being one of the smallest of this block.

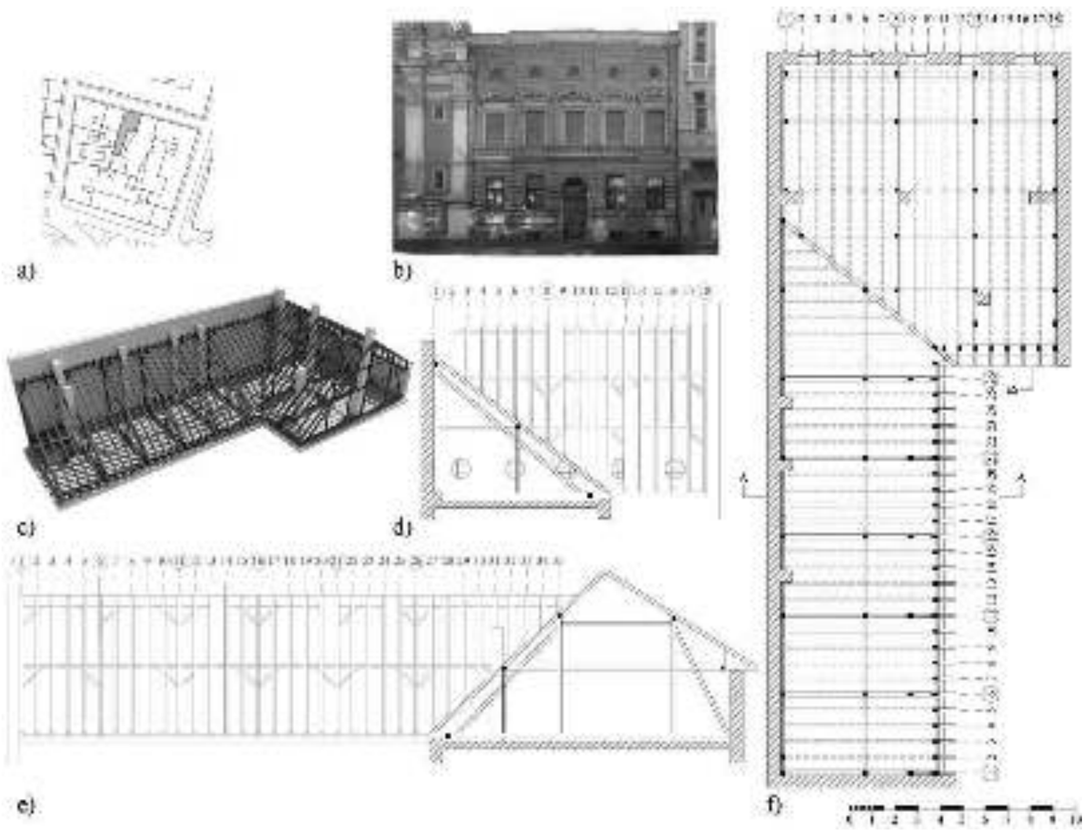

Figure 8: Case study 5 - Residential building: a) position of the building in the block; b) street view; c) axonometric view; d) A-A cross section; e) B-B cross section; f) plan view.

Above the last floor of the building, an oculus was introduced in slab, with the purpose to introduce natural light to the interior stairway. Additional windows were introduced therefore in the attic wall facing the street. This architectural feature had the biggest influence on the chosen roof structure typology. The roof structure had to be adapted so light could get into the attic.

Despite presenting clear eclectic queen post roof structure elements: rafters, compound rafter, intermediate purlins and hanging device composed of tie-beam, straining beam and hanging post, other elements can be identified also. An additional intermediate post, also known as a princess post, was introduced between the queen post and the eaves, due to the long span of the roof frame. In this way, a secondary strutting device was formed. The straining beam of the secondary hanging device is also connected with the ends of the rafters of the exterior façade where another intermediate post was introduced.

This roof structure typology and the way a usual typology is adapted in order to comply to architectural needs, is a clear example that roof structures cannot be assessed without their link with the building they are belonging to.

2.4 Roof structures with architectural and urban value

2.4.1 Case study 6 – Faculty of Chemistry

The Chemistry Faculty building, is one of the biggest buildings of the block and one of the oldest buildings of the Politehnica University of Timisoara. Built in the middle of the XVIIIth century as part of the Franciscan monastery, the building was soon transformed into a school. Since that moment, the building kept continuously changing its shape, but is maintaining is educational function until today.

The current 3-storey building has a complex shape, covering almost the whole eastern façade of the block and part of the norther façade (Figure 9). The façade presents eclectic features, classic with local neo-baroque elements. Due to its position in the block and the presence of a park opposite of the building, the Chemistry Faculty has today a significant urban role, being visible from various points around the block.

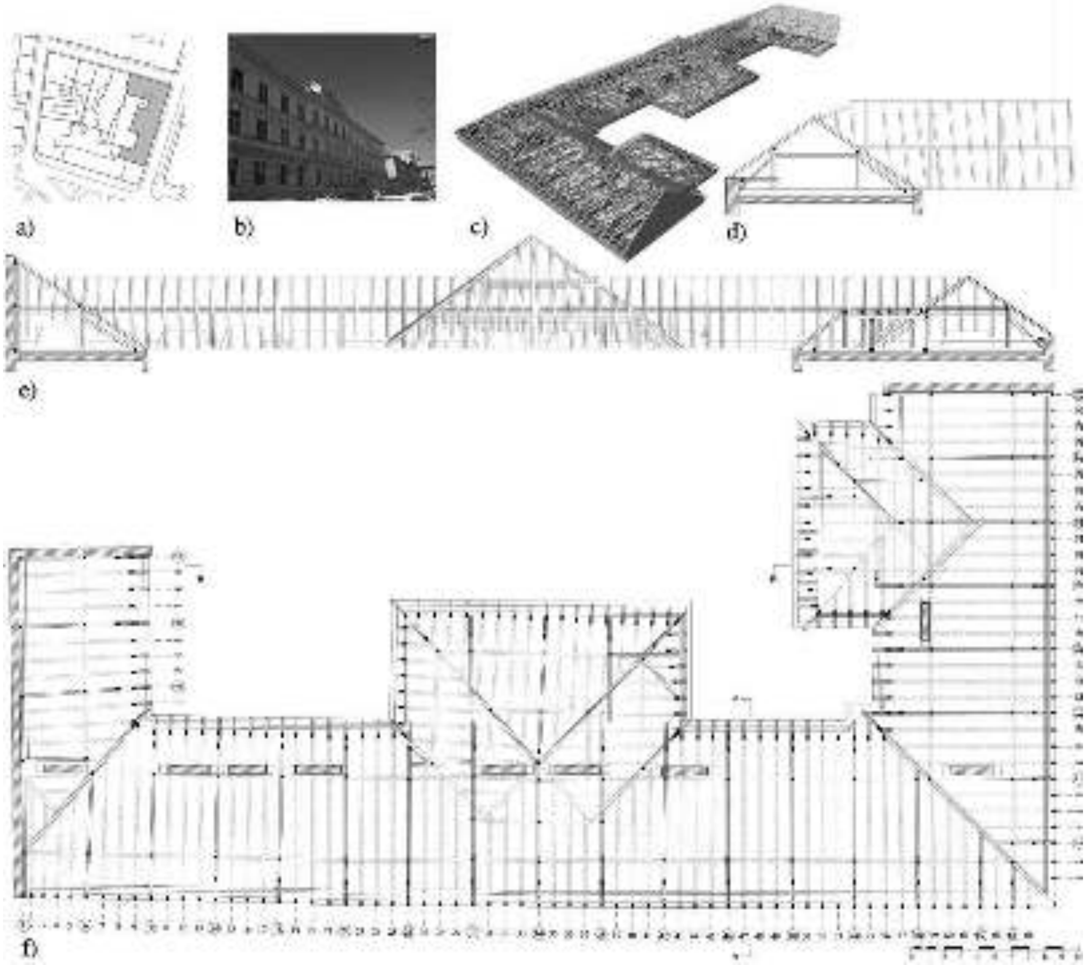

Figure 9: Case study 6 – Faculty of chemistry: a) position of the building in the block; b) street view; c) axonometric view; d) A-A cross section; e) B-B cross section; f) plan view.

Due to the shape of the building, the roof structure has a very high complexity, adapting and combining various roof shapes in order to cover the whole building. The roof combines a shed roof, on the interior wing of the building, two intersected gable roofs, above the main wings towards the street, and two hip roofs, towards the inner courtyard. Despite the complexity of the roof shape, the chosen structural typology is almost the same all over the building, with slight differences due to the technical needs of each shape. The roof structure is mainly eclectic with characteristic elements: rafters, compound rafters, intermediate purlins and hanging device composed of tie-beam, straining beam and hanging post. Due to the increased height of the main façade walls, all the roof structure of the gable roofs had to be adapted. Therefore, the end of the rafter was placed on the raised wall and connected with the compound rafter and the queen post with an additional horizontal beam. The hip roofs present the most complex technical solutions, in order to transfer the loads to the gable roofs. The two structures are connected by a complex system of tie-beams and straining beams.

This roof structure is a good example of a composition of various roof shapes, strongly related to the architectural and urbanistic needs and the way structural typologies had to be adapted in order to create a desired image.

2.5 Roof structures with no architectural and urban value

2.5.1 Case study 7 – Residential building

The building is a 3-storey residential building, placed in the middle of the western side of the block. Like most of the buildings in this block, the building has a main rectangular shape building placed towards the street and a secondary, wing towards the interior courtyard. Compared to the other buildings of the block, since the building was a residential building since the XVIIth century, in the middle of the western front and taking the reduced width of the street, no special attention was given to the shape or height of the roof (Figure 10). The main purpose of this building is to close the exterior of the grid.

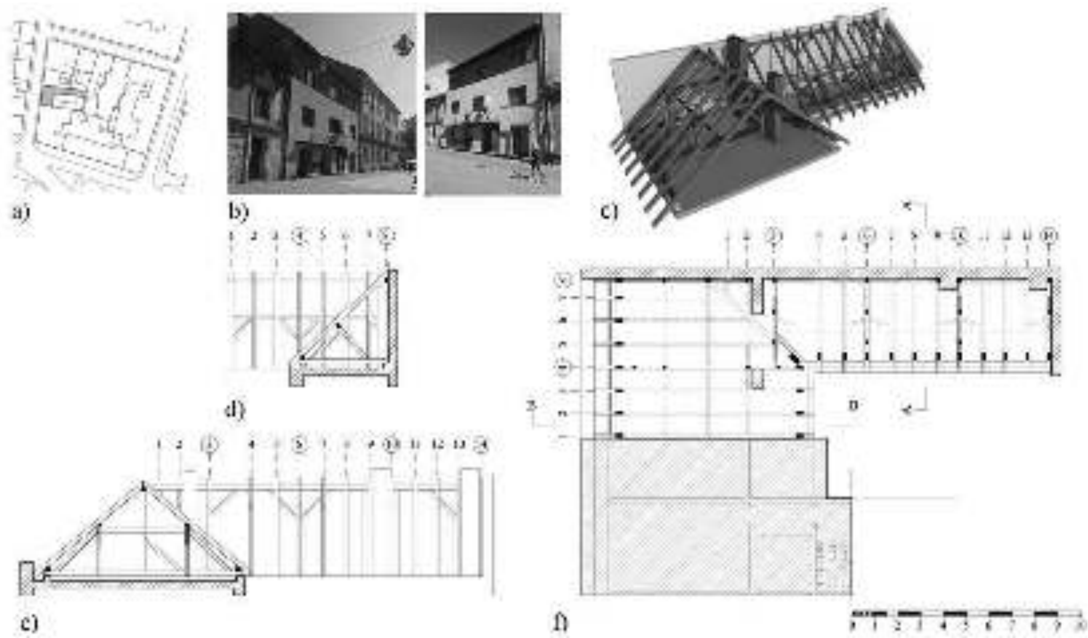

Figure 10: Case study 7 – Residential building: a) position of the building in the block; b) street view; c) axonometric view; d) A-A cross section; e) B-B cross section; f) plan view.

The roof frame of the gable roof in the front part of the building, represents a clear example of a queen post eclectic roof structure. The main frame consists of rafters, compound rafters, purlins (intermediate, ridge and eaves purlin), and a hanging device composed of tie-beam, straining beam and hanging post. Additional brick elements were placed under every tie-beam to lift the whole structure from the slab. The courtyard wing of the building presents another typology of roof structure, being a queen struts purlin roof structure with a vertical post placed in the ridge area. The struts are placed at an 60° angle to the tie-beam and are connected by an intermediate purlin. An additional compound rafter is connecting the tie-beam to the strut.

The roof has in this case no special architectural or urban value, but the roof structure is still adapted to the traditional shape of the building, with gable roof in the front and shed roof facing the courtyard.

3 CONCLUSIONS

This study brings forward that the value of a roof and its structure is closely related to its context, both architectural and urban, and highlights the need to add these new, complex criterions in roof structure vulnerability assessment methodologies.

The study of the buildings and their roof structures of a block in Timisoara that was reshaped continuously over time brought forward interesting facts that are worth to be studied further. On the one hand, the study presents how according to their position in the block, the roofs of the buildings are treated with more or less attention. On the other hand, it highlights the importance of the context of the building, the urban planning and architectural principles for the assessment of historic roof structures. Their shape, chosen structural typology and many technical features are strongly connected to those principles, typically used structural typologies being adapted to obtain a desired image. The result, are unique structural typologies, which have a great value from both technical and aesthetical point of view.

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CHAPTER III

TIMBER FRAMED STRUCTURES

SHATIS'17
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TIMBER 2.0: RESILIENCE AND VULNERABILITY OF WOOD CONSTRUCTION IN EARTHQUAKES AND FIRES

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Keywords: Turkish House, Timber Frame, Stud Frame, *hımış, hatil,* half-timber, nails, earthquake, fire.

Abstract

This paper traces how both earthquakes and fires have shaped the history of timber construction in what is now Turkey and in the much more recently established city of San Francisco, California. It also touches upon how the historical spread of both the Ottoman and Persian empires influenced the adoption of timber construction technologies not only in Eastern Europe but as far east as Kashmir and other parts of India. The paper will describe the effects of earthquakes on traditional timber and masonry buildings, with comparisons to the performance of modern structures of reinforced concrete which are often found adjacent to the traditional wooden structures.

The paper then takes on the subject of urban fires in Istanbul neighborhoods with timber buildings, and the impact this has had on the ultimate prohibition of new timber construction, and the gradual decline and abandonment of the historic houses in urban core of the city. This history, both of earthquakes and fires, is then compared with that of San Francisco, with its much shorter history, but with its earthquakes in the mid-19th century and its most famous earthquake in 1906, which spawned the fire that burned over a large part of the city – a far larger area than that which is reported as the largest of the fires which destroyed a section of Istanbul.

The paper concludes with a more hopeful look at how timber construction, with mitigations against the incidence and spread of fires, could become again a safer alternative to rein- forced concrete in earthquake areas subject to non-engineered construction and failures to comply with building codes or acceptable construction practices. It closes with the point that the "Turkish House" is something more than just a structure that is safe in earthquakes. It is an evocative and socially meaningful icon of Turkey's heritage and art.

1 FROM CONSTANTINOPLE TO ISTANBUL: THE EARTHQUAKE OF 1509

One of the largest earthquakes of the many that have struck what is now Anatolia over the centuries occurred in 1509, only 56 years after Constantinople was captured by the Ottomans in 1453 in the reign of Sultan Mehmet II. This earthquake resulted in the issuance of an Imperial Edict which declared a state of emergency in what was then the "*entire country*," requiring that *"all able bodied males be available to serve the rebuilding activities."* The edict also bestowed 20 gold pieces to every family whose home had collapsed. What also stands out "*is that it forbade new construction near coastal areas where damage was heaviest and made stone block construction illegal, demanding timber-frame buildings"* [1].

Traditional Turkish timber residential construction for the last several hundred years can be divided into three typologies. First, there is horizontal timber lacing in masonry bearing walls, each of which is called a *hatil* and the plural is *hatillar*. This consists of a timber ring beam, shaped like a ladder but laid horizontally in the wall. The historical precursor to this involved laying rings of brick masonry in ancient Roman concrete and stone walls, including the Theodosian walls in Istanbul [2].

The second is a timber frame with masonry infill known in Turkey as *hımış*, (in English called "half-timber"). Here, the masonry is usually a single "wythe" or layer, often with the bricks laid at angles to fit between the studs, or alternatively if stone is used, random rubble set into thick layers of mortar. Depending on the locale the mortar is commonly a lime mortar, but in some rural areas a mud mortar may be used. In *hımış,* the combined timber and masonry wall is only as thick as the timbers – meaning only plus or minus 12cm (5 inches), and thus, when one considers how thin the walls are it seems counterintuitive that this system would be safe and resilient in large earthquakes - but the frame confines the masonry making it an integral part of the structural system. From a structural engineering standpoint, trying to calculate this structure is ultimately frustrating because it comes in many variations, all of which are non-engineered and thus based on intuition rather than rules or codes (Figure 1).

Figure 1: house with *hımış* on the 1st and 2nd floors, and *hatil* on the ground floor in Safranbolu.

Figure 2: Stud framing typical in central Istanbul without infill masonry.

Figure 3: San Francisco 19th century house with timber framing exposed.

The third timber structural system most resembles construction as is common in the USA and Canada – 100% timber with closely spaced studs and joists covered on the exterior with sawn wood cladding and on the interior with lath and plaster over a hollow wall.[3] (Figure 2&3+18-20+24-26) Except for wood-abundant countries such as the U.S. and Canada, buildings built entirely of wood are quite rare. Turkish *hımış* construction has thinner walls than masonry bearing-wall buildings, and is thus easier and more economical to construct. It also serves as overburden weight on masonry walls below for earthquake resistance (Figure 1).

Today this system of using sawn wood siding nailed to the framework would seem to be more simple to construct than quarrying and dressing the stone and making and placing the mortar to form the walls between the timber posts and beams, but before the age of power saws and the invention and manufacture of wire-cut nails, the seeming ease and practicality of working with wood siding was elusive. The saws that did exist made the precision cutting of the lightweight framework possible, but cutting of a massive number of boards to cover the walls remained impractical (Figure 4). Returning to the second typology, *hımış* construction is a variation on a shared construction tradition that has existed through history in many parts of the world, from Elizabethan England to 19th Century Central and South America. In Britain, for example, it would be referred to as "half-timbered," in Germany as "*Fachwerk*," in France and Haiti as "*colombage*" (which did well in the 2010 Haiti quake, as seen for example in Figure 5), in Kashmir, India as "*dhajji-dewari*," and in El Salvador as "*bahareque*" [4]. Ancient Roman examples have been unearthed in Herculaneum, several involving interior partitions, but one involving an entire two-story row house [5]. The palaces at Knossos have been identified as having possessed timber lacing of both the horizontal and the infill frame variety. This history sets the date of what can be reasonably described as timber-laced masonry construction back to as early as 1500 to 2000 BC [6].



Figure 4: Hand cutting timber planks in Nepal, 2016. **Figure 5:** *Colombage* (Half-timber) house with almost no damage from the 2010 earthquake in Haiti. **Figure 6:** Half-timber 3 story house with almost no damage in 2001 earthquake in Ahmedabad, Gujarat.

One likely reason why this building tradition can be found in much of eastern Europe and in South Asia is because of the reach and influence of both the Ottoman and Persian Empires, which together extended into modern-day Pakistan and directly influenced the Mughal Empire, which controlled large parts of India from the 16th to the 19th centuries. *Hımış* was a characteristic form of construction in many parts of Ottoman Turkey and has continued in common use up until it was rapidly displaced by reinforced concrete frame with hollow clay block infill construction beginning in the middle of the 20th Century.



Figure 7: URM stone house in Bhuj after 2001 Gujarat earthquake. **Figure 8:** This man survived but his mother didn't, and he is cremating her, and holds her picture. **Figure 9:** Another view of the ruins of both stone houses and concrete houses in Bhuj.

Observing the places where this method did not become a predominant building type supports this hypothesis. For example, in India, the Gujarat earthquake of 2001 revealed that the Mughal city of Ahmedabad in its historic walled city core had similar timber-laced construction (Figure 6), while the historically Hindu city-states of Bhuj (Figures7-9), Anjar, Morvi, and Jamnagar did not share this construction tradition. In the 2001 earthquake the Hindu cities were devastated, while in Ahmedabad the occupied and maintained houses in the Walled City survived with very little or no damage. As proof that Ahmedabad was also well within the damage district, many reinforced concrete buildings surrounding the city collapsed [7].

In the case of Kashmir, the architecture and construction traditions came to the region with the influx of Muslim Sufis preachers from Central Asia and Persia, beginning in the early14th century, and the subsequent invasions and migrations of people from Persia, who also brought the handicrafts and carpet weaving for which now Kashmir is famous. In fact, *dhajji dewari,* which means “patchwork quilt wall,” comes from ancient Persian [8].

2 KOCAELI AND DÜZCE EARTHQUAKES 1999

We now return to Turkey, not for ancient or medieval history but because of two catastrophic earthquakes that occurred in 1999. The first of these, the Mw 7.6 Kocaeli (or Marmara or İzmit) earthquake, was on August 17, 1999, followed on November 12th by the Mw 7.2 Düzce earthquake. The first quake caused approximately 20,000 fatalities, and the second added another 1,000. Views of collapsed buildings were all over the news at that time, but little was broadcast about timber-laced masonry buildings. For unreinforced masonry, there was the arresting aerial photograph taken in Adapazari showing a stone mosque standing with all of its minarets intact, completely surrounded by reinforced concrete (RC) buildings that had pancake collapsed (Figure 10).

I knew that construction techniques existed in Turkey that were similar to those I had documented in Kashmir, but I did not know if such buildings existed in the 1999 earthquake affected cities. When I spoke to a returning reconnaissance team member I asked if he had seen any timber with infill masonry buildings, and he said he did see some and they were still standing. Then when I asked if he had taken any photos of them, he replied: “*no*”. Despite the fact that they might hold evidence of a key to resilience in this and future quakes, the vernacular buildings were to him neither relevant nor of value. It was at that moment that I decided that I must go to Turkey to explore this situation more fully. Thus my research on traditional timber-laced masonry construction moved from a study in the British Library about the 1885 earthquake in Kashmir to a view of the damage in 1999 from under a hard hat (Figure 11).



1999 EQ photos: Figure 10: Mosque and minaret still standing in Gölcük with collapsed RC buildings around it. (Al Jazeera) **Figure 11:** *Hımış* house next to an entire row of collapsed RC buildings in Adapazzari. **Figure 12:** *Hımış* house next to destroyed RC building. (Adem Doğangün)

In some areas of Gölcük and Adapazari, the first of the two earthquakes destroyed more than a third of all housing units, almost all of them in reinforced concrete buildings. I soon found that there were clusters of *hımış* houses still standing in the heart of these districts, mostly dating from the early part of the 20th century. They were thus significantly older than the reinforced concrete apartment blocks surrounding them that had collapsed [9].

Two Turkish engineering researchers (Demet Gülhan, and Inci Özyörük Güney) conducted a detailed statistical study in several areas of the damage district, and they found a wide difference in the percentage of modern reinforced concrete buildings that had collapsed, compared to those of traditional construction. In one district in the hills above Gölcük where 60 of the 814 reinforced-concrete four-to-seven-story structures collapsed or were heavily damaged, only 4 of the 789 two-to-three-story traditional structures collapsed or had been heavily damaged. The reinforced concrete buildings accounted for 287 deaths compared to only 3 in the traditional structures. In the heart of the damage district in Adapazari, where the soil was poorer, their research showed that 257 of the 930 reinforced concrete structures collapsed or were heavily damaged and 558 were moderately damaged. By comparison, none of the 400 traditional structures collapsed or were heavily damaged and 95 were moderately damaged. For more discussion of this and to see the graph, please go to this footnote link [10].

These statistics reveal that the difference between the traditional and the modern systems is not the materials used or the size of the buildings. Ironically, it is because *hımış* is a non-engineered traditional building technology, while reinforced concrete is dependent on being an engineered building system. When reinforced concrete is used for non-engineered construction and where both design and construction departs from correct building practices, the risk of collapse in earthquakes is significantly increased. This is not a problem for a traditional technique such as *hımış*, as it is intended to be a non-engineered building system, and as demonstrated by the statistics it is more forgiving. Variations in quality and methodology are inherent in this system, just as commonly occur in traditional construction in general, but the occupants are protected by its inherent redundancy and flexibility (Figure 12).

3 THE ORTA (ÇANKIRI) EARTHQUAKE OF JUNE 6, 2000 (MW 6.0)

In June 2000, less than a year after the 1999 earthquakes, an earthquake measuring 5.9 on the Richter scale occurred near the rural town of Orta, 100 km north of Ankara. This earthquake has provided an opportunity to evaluate the performance of *hımış* construction in a smaller earthquake in a rural setting, together with other construction types including unreinforced rubble stone and modern reinforced concrete. The rubble masonry was used primarily for barns and it fared the worst, with a number of farm animals killed by collapsing walls. The reinforced concrete construction, however, was, with a few exceptions, only slightly damaged. What was particularly interesting to find was that many of the examples of *hımış* construction appeared to be damaged to about the same degree as found in the *hımış* houses subjected to the much larger 1999 earthquakes. There was some cracked and fallen plaster with some dislodgement of the masonry infill, but collapses were limited to long abandoned structures with rotted timbers. See [11] for more details and photo-documentation.

The 2000 Orta earthquake illustrates the problem of comparative analysis of earthquake performance of existing buildings. Looked at superficially it would appear that *hımış* suffered significant damage, but this fails to take into account the mechanism by which traditional construction resists earthquakes – flexibility and energy dissipation rather than strength and stiffness. Its survival in the much larger and longer 1999 earthquake illustrates that the *hımış* is capable of maintaining stability over many cycles. To do this, however, the deflection of the structure and friction in the infill must begin at the onset of shaking. Thus the shedding of the plaster in both the larger 1999 Kocaeli earthquake and much smaller Orta earthquake was similar. By comparison, although only lightly damaged in this and other smaller earthquakes, the non-engineered concrete buildings often exhibited a rapid and catastrophic degradation of strength in larger earthquakes, often leading to collapse. This happened because they are inflexible and lack the reserve capacity that has been found to exist in both *hatıl* and *hımış* construction. The brittle hollow tile block infill walls in the concrete frame buildings are initially stiff, and then, once cracked, tend to collapse, leading rapidly to a soft story failure [12].

Figure 13: Yuva family in damaged house.

Figure 14: Yuva from top of minaret, with new village visible on hill above.

Figure 15: Elden old village below and new village above.

Figure 16: Elden old village.

Another important lesson to be learned from the Orta earthquake involves the failure to properly account for two important concerns at a government level when undertaking disaster recovery: (1) the need for the government personnel, particularly the engineers and inspectors, to understand that cracked plaster does NOT mean a traditional *hımış* house is damaged beyond repair, and (2) the cultural and social factors that must be understood and respected in an affected community.

The fates of two rural farming villages near Orta, Turkey demonstrate these points in a sad and tragic way. Yuva and Elden suffered damage in the 2000 earthquake. Rather than helping the residents repair their houses the government recommended the villages be demolished and relocated to what was described by their geologists as safer ground. The residents voted to accept the government's proposal because it came with the promise of new houses in exchange for their old ones. The new sites were selected by a geologist rather than by an agricultural expert or social scientist. They were remote from all that is necessary for human agricultural settlements – water, trees, fertile soil, and protection from the wind. No provision was made for barns for the animals, nor for a community center, general store, or even a mosque. As a result, In September 2004 – more than 4 years after the quake – the new construction was still not finished, and only a handful of the houses had been occupied. Even more inexplicable is the fact that the houses that were built were not earthquake safe, but instead, were constructed of hollow clay tile block with heavy reinforced concrete roofs, as evidenced by one that had not yet been plastered in Yuva New Village [13] (Figures 13-16).

4 2005 KASHMIR EARTHQUAKE

On October 8, 2005 an earthquake devastated the mountainous area of the Pakistan section of Kashmir, killing over 80,000 and rendering most of the local survivors homeless. On the Indian side of the border the damage was much less, but another difference was noticeable: on the Pakistan side of the border where there was a massive death toll, the traditional construction as described above was quite rare.

On the Indian side, however, the performance of the timber-laced traditional construction confirmed earlier findings. Professors Durgesh Rai and C.V.R. Murty reported: *"In Kashmir traditional timber-brick masonry [dhajji-dewari] construction consists of burnt clay bricks filling in a framework of timber to create a patchwork of masonry, which is confined in small panels by the surrounding timber elements. The resulting masonry is quite different from typical brick masonry and its performance in this earthquake has once again been shown to be superior with no or very little damage."* They cited the fact that the "*timber studs…resist progressive destruction of the…wall…and prevent propagation of diagonal shear cracks…and* *out of plane failure*." They went on to recommend that: *"there is an urgent need to revive these traditional masonry practices which have proven their ability to resist earthquake loads*" [14].

The most impressive example of a new acceptance of traditional construction for earthquake hazard mitigation to date is in Pakistan. There, a year after the 2005 Kashmir earthquake after the recommendations by a number of creative leaders in UN-HABITAT and other NGOs working in Kashmir, the Government of Pakistan approved *dhajji* construction as 'compliant' for government assistance. *Dhajji dewari* is the Kashmiri version of half-timber or *hımış* construction. A year after that, they also approved *bhatar* (a timber–laced bearing wall masonry construction). Now, nearing a decade after the earthquake, there are more than 150,000 new homes in this region of Northern Pakistan constructed in either of these two traditional typologies [15].

From a hazard mitigation perspective, the example of the re-adoption of these traditional local technologies represent a potentially sustainable approach to housing construction in many parts of the developing world, as an alternative to the now ubiquitous use of RC frames. While it will not entirely displace the continued construction of RC frame structures, it can perhaps displace what would otherwise inevitably be the most collapse-prone of them. Thus, it can help provide the basis for establishing a better balance where not every building must be in concrete. Hopefully, the local knowledge of the risks of RC frames done badly will become better known, moving people away from the notion that this is the only way to have a modern house. In truth, the exclusive embrace of concrete as "modern" has been very destructive of the architectural traditions and itinerant craft traditions in many parts of the world. A re-adoption and re-learning of the kind of crafts needed for the reemergence of local vernacular architecture can help preserve other aspects of the traditional culture of a community as well [16].

5 FIRE IN 1865 IN ISTANBUL

We return to the city where we started. Three and a half centuries after the Ottoman Sultan issued the edict that mandated that the reconstructed houses after the earthquake of 1509 be made of wood, a new disaster struck on September 18, 1865. This time it was not an earthquake but fire. It was the biggest fire in Istanbul's history, covering one third of the Sultanahmet peninsula, a larger area than any of the many other fires that had plagued the city for centuries [17]. This fire was named the "*Hocapaşa* Fire," and otherwise became known as the *harik-i kebir*, or "Big Fire." [18].

Just as occurred after the earthquake in 1509, the fire affected construction standards in the city, except this time it was a move away from building houses out of timber. Andrew Finkel, Journalist for the Guardian in Turkey observed: *"Istanbul was a city plagued by fire ...but this was not because the houses were made of wood but because they were close together."* One can even make comparisons to the narrow twisty streets that were a cause of the spread of the Great Fire of London in 1666, almost exactly two centuries earlier [19].

Thirty years prior to the fire, in 1836, the Sultan had established guidelines to mitigate against such disasters, but it was not until after the fire that both the motivation and opportunity existed to apply them over a large area. A report was prepared after the fire which focused on two provisions defined in 1836: (1) that new buildings should be of "*kârgir*," that is of cut stone or brick and mortar rather than of timber, and (2) the streets should be regularized and widened to get rid of the *"crooked, narrow holes* (cul-de-sacs) *with abrupt ascents and descents"* to allow room for evacuation and fire equipment access [20].

Figure 17: Houses with firewalls.

The "*Hocapaşa* Fire was preceded by only nine years in 1856 by a fire in Aksaray, and followed by another in 1870 called the Pera Fire, both of which were also influential on the future town planning in Istanbul [21]. After the Pera fire, brick and stone con- struction became mandatory in certain zones in 1875, and in other secondary zones timber was occasional- ly allowed, provided that masonry firewalls were constructed between the wooden buildings [22]. Be- cause masonry was more costly than timber, this served to make reconstruction unaffordable for many except in certain areas. (Figure 17)

Over time, these developments began the long process of deterioration and gradual abandonment of many of what had been fairly upscale wooden houses with their iconic wide projecting rectangular bays. This deterioration also served to create a more apparent division between the wealthy and poorer residents in Istanbul. This happened in spite of the fact that all taxes on brick and mortar were eliminated, and a government commission for road improvement, the *Islahat-i Turnk Komisyonu* (I.T.K.) even set up its own factories to produce these building materials at an economically favorable rate [23].

Reinforced concrete made its entrance into the Turkish construction market in the early 20th century, and now the majority of buildings are made of reinforced concrete with brick infill walls. In fact, what proved to be particularly vulnerable in the 1999 earthquakes was the fact that the brick used for the infill walls was what is called *tula* block, which is an extruded hollow clay tile brick that is initially quite strong, but very brittle – a dangerous combination in earthquakes.

Over the course of the last century the wooden houses of Istanbul have gradually fallen into disrepair, even as many were still lived in, but usually by tenants without an interest in their maintenance. Many have been demolished and replaced with stone and concrete apartment houses under a process which has come to be called "*yap-sat*" (build-sell) where an owner sells his property to a builder-developer, who then builds a condominium form of ownership apartment house and the previous owner is given one of the units as compensation for the sale of the property (Figures 18-20).

Figure 18, Figure 19, Figure 20: Deteriorated wooden houses in Sultanahmet, Istanbul, with modern concrete apartment house visible to the left in #18.

French author Théophile Gautier (1811–1872) had written "*In four months I have seen six great fires*." In his own life growing up in Istanbul, he reported that he saw *"wooden buildings burned by greedy owners who wanted to live in larger modern concrete apartment blocks*" [24]. Prof. Carel Bertram, in her book "*Imagining the Turkish House - Collective Visions of Home,* described how this process has continued in the middle of the 20th century: "*In 1950, the country was taken over by the political populism of Adnan Menderes, who set into motion a process of easy and cheap construction (yap-sat) that allowed the destruction of the remaining traditional fabric of most urban areas….The old houses and konaks that had not been destroyed or relegated to slums found themselves surrounded by concrete"* [25].

6 SAN FRANCISCO

There is one city on the other side of the globe similarly at risk of earthquakes that is also filled with wooden houses, which resembles that of Istanbul to a remarkable degree. Even the architecture in Istanbul with its horizontal parapets, square bay windows and shiplap siding resembles that of San Francisco's shiplap clad balloon framed Victorian buildings. It is almost as if a ship filled with designers and carpenters from Istanbul had set sail from Istanbul and docked in San Francisco and commenced construction. This story of San Francisco begins like Istanbul's described above – with an earthquake. This is not the 1906 earthquake that the city is famous for, but of October 8, 1865, only 20 days after the Great *Hocapaşa* Fire. This was followed only three years later with even larger one on October 21, 1868.

San Francisco in 1865 was a rapidly expanding post-gold rush frontier town in the American west. These earthquakes were large enough to cause widespread damage, particularly to large masonry courthouses and commercial buildings. In fact, in a paper published in 1930 in the "*Bulletin of the Seismological Society of America*," San Francisco consulting engineer, Walter L. Huber states: "*Contrary to popular opinion, these earlier earthquakes were at least comparable in intensity to that of 1906*" [26].

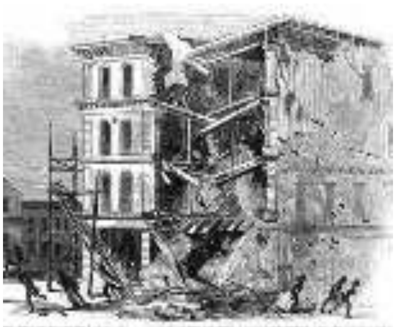

Figure 21: 1865 EQ. (web)

Figure 22: 1868 EQ. (Bancroft)

Figure 23: 1906 EQ. (Genthe)

More recently, in an article published in 2016, Jack Boatwright, a geophysicist at the USGS states that his research has shown that "*The 1868 quake was twice as big as the standing model we had of it*." The San Jose Mercury newspaper the day after the quake reported "*buildings and trees seemed to pitch about like ships in a storm at sea*" [27], and Mark Twain's commentary on his own experience of the 1865 earthquake is one of the most informative and graphic descriptions of experiencing an earthquake ever written [28]. Thus in this case, the effect of these was not unlike that of the earthquake in Istanbul of 1509 – and like that earthquake, they encouraged a continuation of what was already the widespread construction of multi-story stud-framed wooden buildings (Figures 21-23).

7 WOODEN BUILDINGS IN NORTH AMERICA

One important difference between San Francisco and Istanbul, however, is that, while the timber frame buildings were in decline and even banned for new construction in Istanbul, in San Francisco they were being built rapidly with increasingly ornate and colorful detailing by rich and poor alike. But then, *what about fire?* The Turkish author Orhan Pamuk has said that in Istanbul there was a *"tradition of watching fires*" [29]. Did this not also occur in San Francisco with all these wooden buildings? Huge urban fires did happen across the United States, including the famous Chicago Fire of 1871, but instead of the repetition of many fires, these great fires that spread over a large area were usually not repeated more than once in the same city.

For San Francisco its time would come, and it came following another earthquake that caused as many as 50 to 60 fires to break out. It was the Great Earthquake of 1906. The fires coalesced into several great fires that burned for days, consuming the entire central business district plus the most of the splendid wooden mansions on Nob Hill as well as other residential areas on the eastern edge of the city. But, wait! Isn't San Francisco famous for its wooden Victorian houses today? (Figures 24-26)



Figure 24, 25: San Francisco Victorians, 1940 (HABS), and today. **26:** The "Painted Ladies" (B. Mittal)

This is where it gets interesting, because these now famous wooden houses did survive the fires. There were, of course, an equal or greater number of wooden buildings that burned, but a large part of the destroyed area was the central business district with brick and stone buildings with interior wood floors, some of which were even equipped with fire shutters. However, what makes this particular fire remarkable is that not only did it get into the brick buildings which then were left as collapsing shells; it also burned out every downtown highrise building of fireproof construction. These were first generation skyscrapers of skeleton steel frame construction with brick floors of jack-arch construction, a structural technology that had its first origins in Chicago only two decades before. Despite their fireproof construction, it was the contents of the buildings that were flammable – just as occurred in the World Trade Center buildings in New York on 9/11 almost 100 years later. Despite the best efforts of the fire department the fire could not be stopped from getting into the buildings. Interestingly, despite the fact that they had suffered both earthquake damage and were completely burnt out, most of these buildings were repaired and many are still extant today, over a century later [30].

However, this story does not explain why the areas of continuous blocks of side-by-side wooden houses survived, even though they lack the firewalls that were mandated in mid-19th century in Istanbul. This question has no simple answer, and to a large degree the reason may simply be good luck. The streets were wider and on a regular grid, in contrast to the medieval layout of pre-19th century Istanbul. In addition, San Francisco possessed the best firefighting equipment available at that time, and also had installed cisterns under certain intersections of the city streets holding water specifically to fight fires. However, much of this system failed from earthquake damage. Also the dynamiting of buildings to try to create firebreaks proved instead to spread the fire [31]. In the end, salvation came with the wind off of the ocean from the west. For the first two days of the fire this normally prevailing wind was absent, but on the third day it finally gathered enough strength to arrest the westward progress of the fire across the rest of the city.

Figure 27: Google Map of San Francisco, with 1906 Fire to scale with Istanbul 1865 Fire, and on
Figure 28: Lisbon 1755 earthquake & Fire and Tokyo 1923 earthquake and Fire.

As can be seen in (Figure 27 & 28), the San Francisco fire area was vastly larger than Istanbul's 1865 "Big Fire," and Europe's most famous earthquake and fire, the 1755 Lisbon earthquake, fire and tsunami. However, one only needed to wait another 20 years before another earthquake started a fire that burned an area vastly larger than that in San Francisco – the Great Kantō earthquake, which was followed by a fire that wiped out the center of Tokyo.

The lesson of a fire devastating the interiors and contents of fireproof steel and masonry buildings, while leaving vast numbers of wooden buildings untouched to be enjoyed for the more than a century since that tragic day, is that while wood buildings are particularly vulnerable to fire because they serve as the fuel for a blaze as well as suffer resulting destruction, their destruction is not inevitable. Now in the modern day, with electricity and safer sources for heating and cooking as well as sprinklers and other prevention technologies, as well as plaster board and intumescent paint, the risk of the starting and spreading of fires is now much less than the history and folklore of such has been in the history of Istanbul.

Figure 29: Wooden houses on Howard street, only one of which remains still centered on its foundation. Most interesting is that they managed to hold the line against the fire with these houses facing ones burning. (The Atlantic, 4/11/16) **Figure 30:** John Shultz House after the 1889 Johnstown dam burst, a remarkable visual testament to the resilience of 19th century stud frame construction. (Bettmann/Getty Images)

8 EARTHQUAKES AND WOODEN STRUCTURES

I now leave fires and return to the question of earthquake risk and performance of wooden structures. There is strong evidence that the current prevalence of timber construction for both houses and large multi-story apartment buildings on the West Coast is a product of the earthquake risk. Certainly, even in recent earthquakes, not every wooden structure has survived without serious damage, but there are almost no instances where the structural failure of wooden buildings has resulted in fatalities.

When they were in the Schultz House (Figure 30) that was washed almost a half mile downstream all six family members inside survived. In floods wooden houses float and in earthquakes their lightness is both an asset and a problem that needs to be consciously addressed. They need to be secured to the foundation, or the rocking from the earthquake can literally make them "walk off" the foundation or collapse the weak cripple wall framework that supports the house below the ground floor level, as seen in (Figure 29). The modern introduction of garages under houses and multi-family structures without lateral resisting moment frames or shear walls have led to soft-story collapses and near-collapses in the most recent earthquakes in California such as the Loma Prieta earthquake of 1989, which affected the Bay Area, and the Northridge earthquake, which affected the Los Angeles area.

It is crucial to note that timber stud-frame structures do not resist earthquakes as frames, but as membrane structures where all of their walls are part of their lateral force resisting system. This is true for both the structures with nailed horizontal wood siding - in both San Francisco and in Istanbul - and is also true for the masonry infilled *hımış* construction as well. In fact, it is most likely that the almost universal use in both of these cities of the shiplap siding, which is nailed along the top and bottom of each board flush to the studs instead of traditional clapboards, may be because of its effective contribution to lateral strength and stiffness.

Now, as traditional and vernacular forms of construction have gained increasing interest by students and professors alike, a number of engineering papers indicate that the masonry infill is largely missing from the engineering analysis except as dead weight. There is, for example, much emphasis on the need for diagonal timbers within the frame to act as lateral braces. In actual practice, there are many traditional infill frame structures - for example in the Vale of Kashmir and in Pakistan – which do not have diagonals, but which have proven to be as resilient as the others in the 2005 earthquake. This was also true with some *hımış* buildings in Turkey during the 1999 earthquakes (Figure 31).

Figure 31: Srinagar, Kashmir *dhajji* structure without diagonals

Figure 32: House in Düzce, Turkey after 1999 EQ, showing the 'working" of *himis* wall

Figure 33: Collapsed RC building in 1999 showing shear failure of hollow clay tile infill wall.

In recent decades, engineers have struggled to figure out a way to analyze and do calculations on reinforced concrete frames with infill masonry, and one of the important methodologies that has been developed is to include what is now called the "*equivalent diagonal strut*." This is the compression strut that occurs as the building's frame deforms and transfers compression load onto the infill masonry. In an earthquake, this causes the characteristic "X" cracks. However, one almost never sees an "X" crack in a *hımış* or *dhajji* structure. This means that there is a very important and beneficial difference between the traditional infill systems, and the infill walls found in modern concrete frame buildings (Figures 32 & 33). The most counterintuitive difference is that the mortar in traditional construction is most often either mud or weak lime mortar, in contrast to cement mortar used in modern RC construction. The second is that the infill panels are much smaller than those found in modern RC structures. This is linked to a need to rediscover what had been traditional knowledge, not unlike what was observed after the 1885 earthquake in Srinagar by Arthur Neve, in the quote referenced above: *"...wood is freely used, and well jointed; clay is employed instead of mortar, and gives a somewhat elastic bonding to the bricks, ... If well built in this style the whole house, even if three or four stories high, sways together, whereas more heavy rigid buildings would split and fall.*"[32]

9 CONCLUSION

Finally, one must focus on the most important purpose for our interest in timber construction of all types. Keeping us safe through the 15 or 30 seconds of the next earthquake is of course important, but it is the meaning that our homes have to us over the years between two earthquakes that is of equal importance. In her book *Imagining the Turkish House,* Carel Bertram opens by describing her experience as a Fulbright Scholar of Islamic History traveling to remote towns in Turkey when people asked her what she was doing. When she responded with the "*simple answer*" that she "*had come to do research on the Turkish house*," she reports that "*What I found, to my surprise, was that I had only to utter those three words to have my interlocutors' faces become positively beatific. In fact, I came to expect a glow, as they repeated the words with a reverent love:* 'the Turkish house'*"*[33]. Perhaps by coming to understand how these houses can keep one safe for the essential 15 seconds, they can also once again be embraced, loved, lived in, and created anew for the enjoyment of lives today and yet to come.

Figure 34: Safranbolu, Turkey

[1] Ergünay,O and Gülkan, P. 1993, Land-use Planning as Instrument of Earthquake Hazard Mitigation, in Comprehensive Approach to Earthquake Disaster Mitigation, Andreas Vogel, ed, Vieweg & Sohn Verlag,Wiesbaden. P246.

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INSIGHT INTO THE TRADITIONAL TIMBER FRAME WALLS: HERCULANEUM EVIDENCE VERSUS BRACED FRAME STRUCTURES IN PORTUGAL AND IN ITALY

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Keywords: traditional timber frame wall, brace timber frame heritage, Herculaneum

Abstract

In recent years there has been an increasing interest in the traditional timber frame constructions in earthquake prone areas, due to their good earthquake resistance and the remarkable architectural features.

The purpose of this paper is to provide an insight into the historic timber frame walls and the reasons underpinning their use and dissemination within three different contexts, i.e. Herculaneum (until 79AD), Portugal and South of Italy (XVIII century).

During the Roman Age, the large use of timber frame walls corresponds to a proactive approach to working within a project's external constrains. Population growth and the need for fast and low-priced structures spurred an upgrade of construction techniques, especially in Herculaneum. Aftermath of the macro-earthquakes in Portugal and in Italy, instead, a similar construction system, based on three-dimensional timber frames, is expressly employed for its seismic resistance. This diachronic comparison based on literature review and on-site analysis aims to clarify the influence of the timber frame walls on the architectural layout and on the building behaviour in these three case studies.

1 INTRODUCTION

The timber-framed structure can be interpreted as a constructive archetype that correlates *genius loci* with *genius artis* and *genius materialis*. Nardi introduces these concepts to define the technical acts a generator of archetypes where the construction and cultural instances merged and became the essential points of building system [1].

The persistence of the half-timbered system spans centuries and civilizations with various local declinations according to each environment. However, the archaeological evidence is scarce, especially within very strongly inhabited areas. The majority of the walls made of timber framework were lost after the structures had fallen into disuse.

Exceptions are those rare examples in which special conditions have fostered their conservation: in the European temperate climate, in cases where the wooden elements were baked (e.g. remnants of a fire), or in specific conditions of water saturation; or in the dry climates of North and East Africa [2]. In addition to the perishability of the components, there was previously little interest in analysing and preserving these structures, as demonstrated in the destructive early excavation campaigns in the Vesuvian area [2, 3].

Leaving aside the debate on its provenience, the timber-framed structure represents the most common type among the mixed construction practices employed by Romans. The Romans spread this mixed construction system throughout Gaul to the provinces of their jurisdiction, implementing the non-indigenous construction experience with their distinctively pragmatic, shrewd, and standardized work in order to increase the rapidity of construction.

Accordingly, the analysis of the Roman timber framed walls can be a starting point for discussing the genesis of this construction system in the Mediterranean basin.

Within the considerable literature on the Roman building techniques, an essential overview was addressed by Lugli in 1957 and by Adam in 1984 [4,5]. More recently, Giuliani re-examines these materials in order to overcome the risk of historical-archaeological simplifications through an accurate investigation in terms of metrological, material, and structural features, as appreciated by Bacchetta [2,6]. In addition, Ulrich provides a clear overview of the ancient sources and of the main questions surrounding the woodworking tools, examining recent discoveries within the Roman Empire in the Italian provinces (e.g. Lavinium) and in Britain (e.g. Verulamium) [7].

In this paper, a brief analysis of the timber frame walls at Herculaneum is carried out based on literature review and on-site analysis. Firstly, the reasons for the relevant presence of wooden components and the difficulties in gathering evidence are underlined. A remarkable case study, *Casa a Graticcio*, is described as precious yet controversial physical evidence. Its speculative reconstruction, often overlooked in the literature, is discussed to raise issues related to its value, conservation, and public appreciation (Figure 1).

Figure 1: Reconstruction works of *Casa a Graticcio* (1929) [8]

Secondly, the authors compare a few archaeological findings at Herculaneum with the internal structural skeleton of the buildings in Lisbon downtown and in the quake-damaged towns of South of Italy (XVIII-XIX century).

A considerable literature on these construction systems respectively aftermath of the macro-earthquakes in Lisbon (1755), and in less than thirty years in the South of Italy can be found [9,10,11]. However, further studies are still required in order to devise proper safeguarding strategy.

2 PHYSICAL EVIDENCE OF TIMBER FRAME WALLS AT HERCULANEUM

2.1 Unlocking the reasons for the relevant presence of wooden findings at Herculaneum

Regardless of several adverse factors that prevent the comprehensive acquaintance of its historical backdrop, the ancient Herculaneum includes unparalleled cases study for a deeply understanding of the traditional half-timbered system.

The wider diffusion of this technique in this mid-sized town compared to other Roman settlements (e.g. Pompeii, Stabiae, Oplontis, Boscoreale) depends on: (i) a contingent cause, due to the intervention works along the Roman expansions and after the 62 AD earthquake; and (ii) the historical event, i.e. the eruption of Mount Vesuvius (79 AD) that allows the well preservation of the timber components. These two key points are briefly analysed.

Firstly, the strong demand of new constructions and the need of heavy repairs aftermath of the 62 AD earthquake may have influenced the diffusion of thin and cheap half-timbered walls [12,13]. The traditional timber frame system made use of available materials and did not require specific preparation of the materials or highly skilled labour. The pioneering and systematic use of mixed technique stemmed from the Romans' ability to structure their spaces according to the paradigms of their contemporary life, with a consequence of redefying the boundaries of the intimacy [14].

After seventeen years of the post-earthquake reconstruction, Herculaneum was overwhelmed and submerged by volcanic material flows at very high temperature (400ºC), which carbonized the organic components, encapsulating them in an anoxic environment, and determined its unique conservations conditions [3,12].

Unfortunately, many of the carbonized wooden artefacts have not been maintained. The fragility of the charred woods and the difficulty to extrapolate its among a solidified volcanic ash, as well as the low interest in these components in the early archaeological campaign, explain the absence of references in the excavations reports and also the heavy on-site destructions in 1780s [15,16].

2.2 Overlapping historical layers: *Casa a Graticcio* (*Insula III*, 13-15)

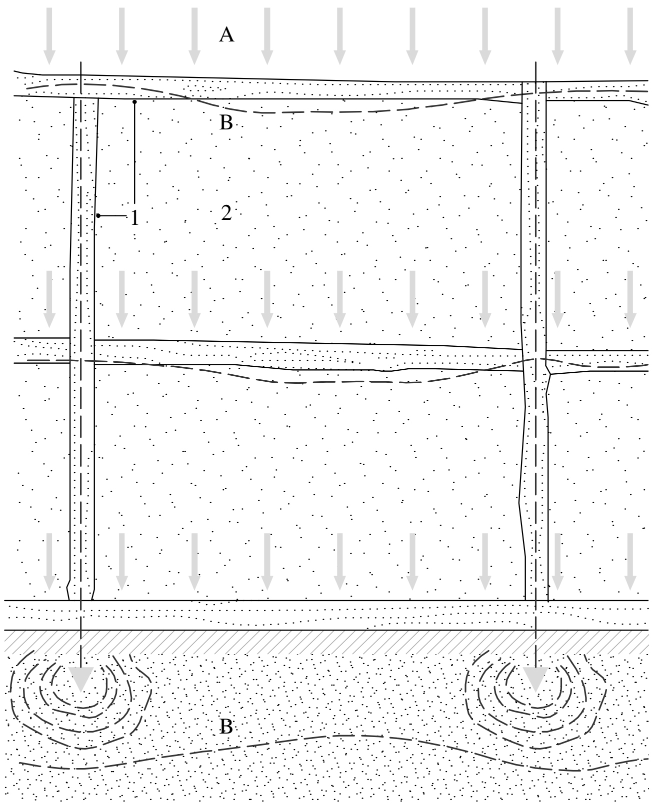
Among many dwellings discovered and restored at Herculaneum, *Casa a Graticcio* (*Insula III*, 13-15) is a remarkable evidence of the socio-economic organisation of the last building phase of the city.

In terms of typological features, it represents the transition from the single-family atrium houses to multi-family houses with courtyard. It is also representative of humble Roman houses in the narrow hybrid *insulae* (grid of rectangular blocks) that included commercial trading on the ground floor and rental apartments in upper floors.

From the early findings it was clear that the building layout is a result of a post-earthquake reconstruction. Monteix underlines the changes to the original layout through evidence of an early Samnite settlement of pre-Roman Age [8,16]. However, the construction solutions were likely selected for the cost-effectiveness, the thinness (possibility to split efficiently the interior space), rather than for the seismic resistance of the timber frame system. The internal construction system of the ground floor was probably chosen for the speed of execution and low thickness. In fact, the timber frame walls would have implied the possibility of easy dismantling. In its original conception, the construction system was characterized by a timber skeleton filled with heterogeneous components including straw, clay, and fragments of roof tiles. Divided into regular frames of 0.60cmX0.80cm of carbonised wooden components, the framework walls were locked within pillars in brickwork or tuff rocks and brick.

When *Casa a Graticcio* was discovered in 1927, only the structure on the ground floor still was integrally remaining, not including two thirds of the height of the three brick pillars that were later reconstructed by Maiuri. Occupying the public sidewalk, these pillars were executed with the specific aim to support the timber floor and the upper wall. Maiuri declares that the original timber framed structure was conserved only on the south side of the ground floor [16].

By being fully supported by the brick pillars, the external facade in (restored) timber framework acts as a non-loading system. The structural behaviour depends also on the boundary conditions, i.e. the adjacent houses which form a compact urban agglomeration. Due to their high flexural strength, the internal frame structures are presumed to collaborate with the external to support the weight of the roof structure. As underlined by Giuliani, this follows a structural model consisting of a load-bearing skeleton of linear elements appropriately spaced, which take and transfer all the stresses of the imposed loads vertically downwards [6] (Figure 2).



1. structural skeleton A. compression
2. filling B. combined compressive and bending stress.

Figure 2: Distribution of loads in a structure frame from Giuliani [6] (reprocessed by the author)

The 1930s interventions on the architectural vestiges of *Casa a Graticcio* implied heavy reconstructions of the internal structures and of the volume overhanging the main street (Figure 1). Even in the absence of a dataset from the daybook record, Monteix discovers several incongruities in terms of the external configuration as well as regarding items of furniture (e.g. bronze statuettes, positions of the wooden beds). Archive photographs allow interpretation of the recent works, especially the one related to the reconstruction of the external wall structure and the balcony; the different phases may be identified in time through the date of the snapshot incised on the negative [16].

As occurs in other buildings at Herculaneum, the type and the regularity of the replaced elements expose the modernity of this intervention that aimed at a semantic unity. In addition, the high degradation of the materials induced by these improper and incompatible restoration works is clear evidence of the low authenticity of this building following its reconstruction. On the external facade, the detachment of the coating layer shows two L-shaped steel beams distanced by hollow bricks that support the balcony and bear on the aforementioned pillars (Figure 3). The square frames of the external upper walls, whose timber components were replaced by new elements of the same size, were filled by blocks of yellow tuff stone [16], which is a foreign material to the local construction culture.

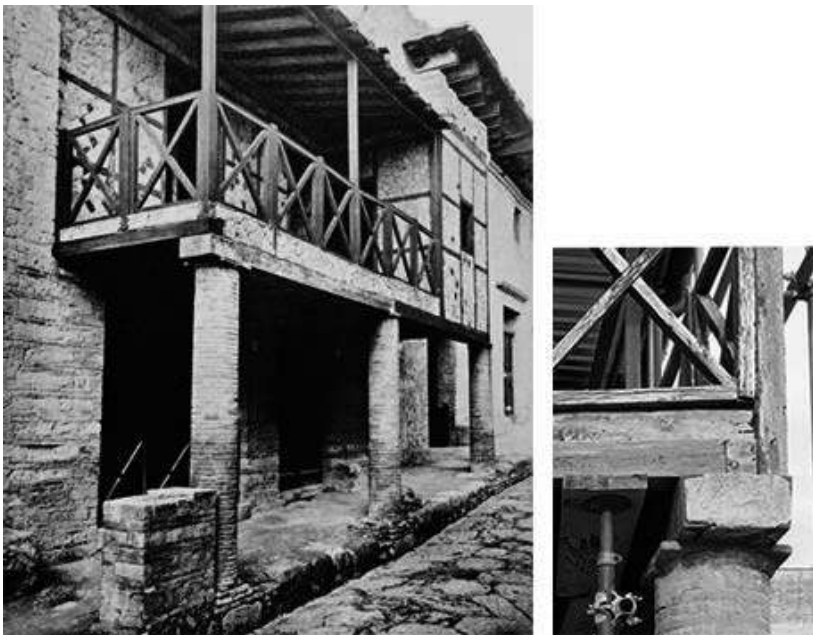


Figure 3: *Casa a Graticcio* (*Insula III*, 13-15, Herculaneum): main elevation after the 1930s restoration [16] and detail of L-shaped steel beams (2016, photo by the author).

Leaving aside the controversial aspects the structure inherited in the 1930s, this case exemplifies the persistence of human experimentalism in the ex-post earthquake intervention.

Casa a Graticcio may be the most relevant historical example of the proactive approach to working within a project's external constrains, an attitude that has since become the basis of modern engineering.

3 DIACHRONIC COMPARISON OF TIMBER FRAME WALLS

3.1 Localization in the building

Craticii parietes (i.e. timber frame wall) at Herculaneum can be divided into two groups depending on their localization: (i) the external walls on upper level, whereas the ground floor is in masonry or brick works; and (ii) the internal partitions on the ground floor and upper floors, directly supported by the floor structure, often coated with *opus sectile* (i.e. type of mosaic work).

When the external facade is made of timber framework (i), the lightness of this construction system allows an increase in the height of the upper-floor housing by ensuring a proper distribution of additional loads. It demands an alteration of the formal composition of the building, by squeezing the proportion of the facade through an inversion of the ratio between the height of the ground floor and the upper floor (e.g. *Casa di Nettuno e di Anfitrite*), or by lowering the overall elevation (e.g. *Casa a Graticcio*).

In cases of internal half-timbered walls (ii), the thinness of the partitions allows the limited floor space to be more easily organized while still providing some insulation. These partitions divide either secondary or principal spaces (e.g. *Casa del Larario di Achille*): the wall structure was completely hidden by figurative and coloured frescos in a such way that this construction solution was not distinguishable from a brick or masonry wall.

Regardless of their internal or external localization, the original load-bearing capacity of the frame walls at Herculaneum is completely lost due to the on-site burial, the damage caused by the excavations since the 1780s, and the works of the 1930s. In the latter phase, metallic structures, new infill, modern timber frame with some original casting pieces, and consolidation with paraffin were widely employed [15].

In the literature, Papaccio pointes out the Roman system's structural effectiveness under seismic forces [13]. On the other hand, Laumain underlines its structural fragility when compared to the medieval French timber frame buildings. Even in the presence of a regular structural grid that allows good load distribution, the slenderness of the timber framework and the limited cross-section of the components leads us to suppose that the Romans were not aware of the potential strength of this system [17].

Focusing now on the timber framework walls in Portugal (so-called *frontal* walls), this technique is a structural permanence before and after the 1755 earthquake, as can be seen in the overhanging dwellings in Lisbon [19]. The timber frame walls, whose seismic resistance has been fully documented on the basis of experimental tests, are the key components of a three-dimensional system above the first floor (Figure 4). Its use was systematised and largely employed during the reconstruction of Lisbon downtown after the 1755 earthquake, with a remarkable improvement of the earthquake-resistance performance of the so-called *Pombalino* multi-storey buildings [9, 10].

In contrast to the uniformity and standardization of the *Pombalino* system, several construction layouts and geometrical can be found in the braced frame heritage of the South of Italy. In these buildings, the timbered walls are filled in rubble masonry (external walls) or with horizontally arranged reeds between regularly spaced vertical posts (internal walls), as shown in Figure 5 [10]. As proven by recent experimental campaign, the timber framework would provide to the masonry wall an additional tensile strength under dynamic action [18].

Figure 4: Internal timber frame wall: *Pombalino* building in Lisbon versus Bishop's Palace in Mileto (Calabria region) (2015, photo by author).

2.2 Connections and materials characterization

This analysis focuses on the main construction features (i.e. connections, diagonal members, frames) and the materials characterization of the timber frame walls in the three mentioned case studies.

- *Connections:* Physical evidence shows how this type of internal wall at Herculaneum was executed after the construction of the floor and roof, as simple partitions in order to subdivide the interior space. As an example, in *Collegio degli Augustali* (VI, 21), the absence of technical devices (e.g. metallic fasteners and interlockings) to connect the half-timbered wall to the orthogonal structure (the external masonry wall) demonstrates that this was a later alteration of the original layout. The timber frame wall was executed in a second phase for a subsidiary function, detracting from the decorative scheme and compromising the inner spatial layout based on blind arches (Figure 5). In other cases, vertical posts were executed next to the orthogonal wall without any transversal connections, whose anchor devices remain unclear due to the surface finish applied in the 1930s.

In *Pombalino* buildings, a vast number of timber elements are lumped together by iron or metal ties [9,10]. The most common carpentry joints are half-lap joints (cogged half-lap joint or dovetailed or wedged dovetail) and mortise-tenon joints. These joints together to the *Trait de Jupiter* can be found also in the Herculaneum findings.

Figure 5: *Collegio degli Augustali,* Herculaneum (2016, photo by author).

This latter joint is commonly employed when the wooden beams are not available in the required length. It can be found also in a Bishop's Palace built in 1784 at Mileto (Calabria region). In this building, the internal timber frame walls are connected to the floor beams by half-lap joints (Figure 6).

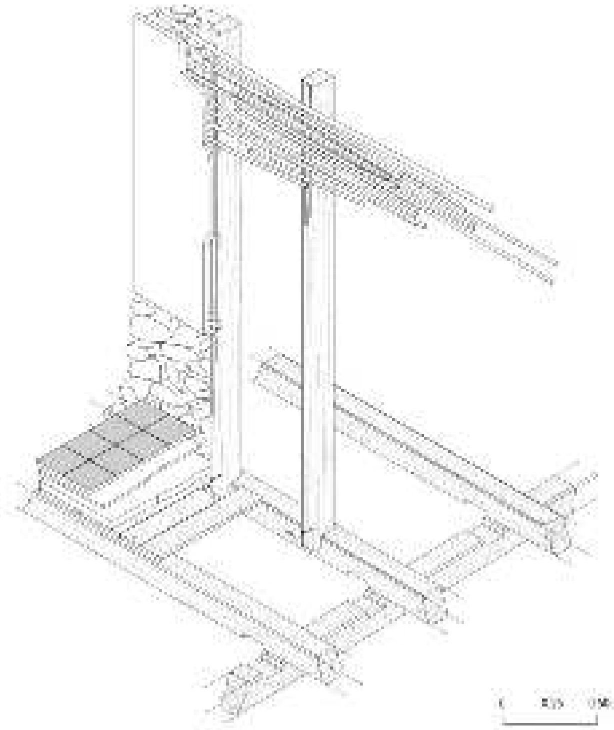


Figure 6: Internal half-timbered wall: Bishop's Palace in Mileto (Calabria region)(by author).

- *Diagonal members*: While stating the incompleteness of the archaeological excavations in Vesuvian towns, Adam argues that the use of bracing components (so-called "*tra-*

versa sbieca" or "*saetta*") is rare within the Roman vestiges [5]. As shown by the low percentage of findings, Romans did not commonly execute diagonal components. The execution of bracing diagonals increases the time of construction and the degree of its difficulty. However, in terms of structural performance, the frames assume the behaviour of non-deformable triangles and they are then able to withstand the seismic effects related to 45° cracking. In the only exception so far discovered, timber diagonal elements can be found in the internal wall on the upper floor of *Villa di M. Arrius Diomedes* at Pompeii, although the wooden components were later replaced by plaster (Figure 7).

Figure 7: *Villa di M. Arrius Diomedes* at Pompeii by Adam [5]

Regardless of a less accuracy during the latest construction process, the *Pombalino* system until 1880 (when this type of construction was abandoned) always included most of timber frame wall reinforced by cross bracing components. In the Italian cases, instead, the diagonal components were not always employed, e.g. a building in Filadelfia described by Tobriner [11] versus Bishop's Palace in Mileto [20].

- *Frames:* The timber frame walls are subdivided into squared frames. There is a dimensional correspondence between the width of the timber frame and the width of the doors (e.g. *Casa del Bel Cortile*). The same principle can be found in the other systems in analysis.

The regular geometry, implying an increase of the structural reliability under dynamic loads, was lost in the Portuguese timber frame walls during the Middle Ages, where unequally-sized squares prevailed [19]. Instead, after the 1755 earthquake, the regularity and the standardization of the construction process was respected [9,10].

- *Materials characterization*: The most commonly employed wooden species in Herculaneum was the same recommended by Vitruvius, the *Abies alba* (*De Architectura* 2, 8-9, 11-12), used both for building construction and for interior furniture, as confirmed by ancient sources and recent laboratory testing [21, 22].

Various types of wooden species and genus may be found in a single *Pombalino* building, i.e. *Pinus pinaster*, *Castanea sativa*, *Quercus ilex*, *Pinus silvestris*, *Pinus palustris, Quercus robur*, and *Quercus suber* [8,9]. *Calabrian Chestnut* is the most frequently used in the buildings of Calabria region.

The timber frameworks were commonly filled with heterogeneous materials, whose differences of the thermal capacity and hygroscopicity with the wood would cause the crumbling of the surface finish. In addition to the rising damp and the high ignitability, this is one of the most important drawbacks relating to this traditional technique.

4 CONCLUSION

The authors provide a comparison between Herculaneum's findings and the buildings executed aftermath of the late XVIII century earthquakes in Lisbon and in the South of Italy. It is shown that, regardless of their relevant construction differences, these systems involve similar structural principles and techniques that influence the buildings behaviour under static and cyclic loadings. The structural reliability of these constructions is based on the reduction in the weight from the lower to the upper floors, the ductile behaviour of the carpentry joints, and the high strength-to-density ratio of timber.

However, the building stock of the late XVIII century embodies an awareness of the seismic resistance of the timber frame system that it cannot be identified in the houses at Herculaneum.

This field of research should be further deepened (e.g. detailed survey of timber frames in the Vesuvian dwellings) in order to underlook the main features of this traditional technique.

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HISTORIC TIMBER FRAME STRUCTURES: A COMPARISON OF DIFFERENT CONSTRUCTIVE SYSTEMS AND THEIR RESISTANCE TO SEISMIC ACTIONS

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Keywords: timber-frame, seismic capacity, infill, Pombalino, Borbone system, Kay peyi

Abstract

Timber-framed architecture is a popular construction type popular all over the world. Timber frames are commonly adopted as a structural element in many countries with specific characteristics varying locally in terms of geometry and materials. Their diffusion in Southern European countries is linked to their good seismic resistant capacity, but only in the last decade interest has grown for this structural typology and studies have been performed to better understand their behaviour.

This paper wants to analyse the origin and development of timber frame structures around the world, their evolution and diffusion as well as their specific adoption as seismic resistant structures. In particular, a comparison will be drawn and analysed between typical timber-framed structures present in Portugal (edificios Pombalinos), Southern Italy (casa baraccata) and Haiti (Kay Peyi). In-plane cyclic tests have been performed in representative traditional timber-framed walls of all these construction typologies. The walls were built in real scale and adopting the original geometry and material properties. The idea was to represent the historical timber-framed walls.

A similar mixed rocking-shear response was observed in all walls, with varying uplifting of the vertical posts. Depending on the quality and stiffness of the masonry infill (which varied from brick masonry to rubble masonry), the walls presented varying degrees of initial stiffness, maximum load capacity and energy dissipation capacity. This comparison points out the great variability existing in the quality and performance of timber-framed structures, keeping in mind that strength and stiffness are not always efficient in seismic events and that most of these structures have proved to perform well during seismic events.

1 INTRODUCTION

Timber frame buildings are a common traditional construction present in many countries, particularly in local vernacular architecture, and they constitute an important cultural heritage worth preserving. Different examples can be found all over the world. For the purpose of this paper, timber-framed architecture present in Portugal, Italy and Haiti will be considered.

After the devastating earthquake in 1755 that destroyed Downtown Lisbon, a reconstruction plan was put into action by the Prime Minister of the time, Marquis of Pombal, who appointed engineers and military architects to elaborate reconstruction plans of the city. The new regulations adopted pro-vided 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 [1]. The new buildings designed took into account the seismic capacity of the structure, safety against fire as well as a standardization of the structural elements, in order to achieve a cheaper and faster construction.

The buildings that derived from the proposed plan, called Pombalino buildings, were characterized by external masonry walls and an internal timber structure, named gaiola (cage), which is a three-dimensional braced timber structure, similar to many half-timbered buildings that can be found in several European countries. 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 [2]. The timber frame walls are usually filled, either with rubble or brick masonry or even mud and hay. Plaster was applied to frontal walls, creating small cuts in the timber elements so that mortar, generally lime-based, could adhere better. The adoption of a weak mortar and infill allowed flexibility to the wall joints, which could dissipate a higher amount of energy in case of an earthquake.

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, reaching up to 5 storeys. 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 [1]. The typical width of the external masonry walls was of 90 cm at the ground floor. A simplified internal timber structure was embedded into the external masonry walls on the inner side, to facilitate and improve the connections with the floors and the inner timber frame walls. The connections between the external masonry walls and the internal timber frame walls varied and it depended mainly on the number and length of timber elements embedded in the external walls (Moreira et al., 2014).

Additionally, masonry walls perpendicular to the façades divided the buildings and avoided fire propagation in adjacent buildings.

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 found. The innovation consists of the improvement of the system, as well as the standardization of the constructive practice in Lisbon [1].

The internal walls of the gaiola (*frontal* walls) may have different geometries in terms of cell dimensions and number of elements, 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.

In Italy, after the 1783 earthquake that destroyed Reggio Calabria a new system, called casa baraccata (literally, "baracca" means shack) was adopted by the authorities by imposing standardized construction methods. The same construction technique, with slight changes, was also adopted after the Messina earthquake in 1908. 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. The rules included instructions on the exterior aspect of the buildings, on the 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. These rules constituted 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 [3]. Vivenzio proposed a 3-storey building with a timber skeleton aiming at reinforcing the external masonry walls, avoiding their premature out-of-plane collapse. The timber frames constituted the shear walls, sometimes presenting a bracing system of S. Andrew's crosses. 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. Different dispositions were adopted for the timber frame, from a double bonded frame with masonry in the middle (used for public buildings) to a single frame embedded into the wall at different depths [4].

In Haiti, traditional timber-framed buildings were present in the island, both in the form of local vernacular architecture and in the form of architectural styles brought by emigrants, e.g. Gingerbread houses. The Kay peyi structure analysed here is part of a reconstruction project [5].

1.1 Existing experimental studies on timber-framed architecture

Studies regarding the seismic performance of timber-frame structures are rather scarce and most of them are based on the qualitative assessment of structures in disaster areas [6,7]; only recently some experimental studies have been carried out on the seismic behaviour of timber-frame walls of different typologies, namely the Portuguese gaiola, the Turkish himis, the Italian casa baraccata, the Haitian Kay peyi, the Indian Dhajji-Dewari [4,5,8,9,10,11]. All these works analyse the cyclic behaviour of timber-frame walls, with few correlations between distinct typologies or a structured analysis method.

Given the cultural heritage importance of traditional timber-frame structures around the world and the rough information of the mechanical seismic performance, it is of fundamental importance to have further and specialised research addressing the analysis of the mechanical behaviour of this type of structures, from experimental, numerical and safety assessment viewpoints, taking into account different typologies available and their variables.

In this paper, a comparison of different experimental works on timber-frame walls will be carried out. The results will be analysed in terms of influencing parameters, such as presence and type of infill and type of connection. Moreover, seismic parameters such as stiffness, ductility and energy dissipation will be analysed. This work wants to point out the importance of timber-frame structures and the need for further and collaborative studies, particularly in the Mediterranean area, which offers a great number and a great variety of such structures.

2 COMPARISON OF EXPERIMENTAL RESULTS

2.1 Specimens and loading protocols adopted

Timber-frame structures are highly non-linear and complex structures, distinguished by their high energy-dissipation capacity. There are many types of traditional timber-frame structures, built without engineering-considerations, which generally form the bulk of heritage structures in many countries. These structures are known to have survived many seismic disasters, but only recently an effort has been made to better understand their behaviour in a quantitative manner.

Most experimental studies concerning timber frame walls have been performed recently and a great variability exists in terms of specimens adopted (considering different sectional dimensions, different infill types and different geometrical configurations, including the presence or absence of bracing elements) and in terms of loading protocols adopted, varying from European to American protocols (ISO 21581 and CUREE protocol).

All walls tested were based on examples found in real buildings, adopting real cross-sections. In the case of Portuguese timber-framed walls from Pombalino buildings, cross sections were between 16×12cm^2 and 8×12cm^2. For “Kay peyi’’ construction, the walls presented smaller cross-sections than what encountered in other countries, namely 10×10 cm^2 for angle post, 5×10 cm^2 for post and top and bottom beam and 2.5×10 cm^2 for bracing elements and remaining beams. Also the wall reproducing the Borbone system, real dimensions were used for the elements. In this case, no diagonal bracing members were present. For the tests of Pombalino walls without bracing elements [11], 12×12cm^2 elements were used. An encompassing experimental work on timber frame walls traditional of the Turkish culture has been performed by Aktas et al. [9].

A great variability exists in terms of wood species used: maritime pine (Pinus pinaster) for Portuguese frames [8,12], Calabrian chestnut for the Borbone walls [4], Pinus sylvestris [11] and many others for other works presented in the introduction.

2.2 In-plane cyclic tests results on unreinforced walls

Timber frame walls are characterised by a mixed flexural-shear response. In particular, 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 infill timber frame walls (Figure 1) to a shear predominant resisting mode for timber frame walls with no infill (Figure 2) [4,8]. In general the presence of infill is able to increase strength and stiffness of the walls, while not compromising their displacement capacity.

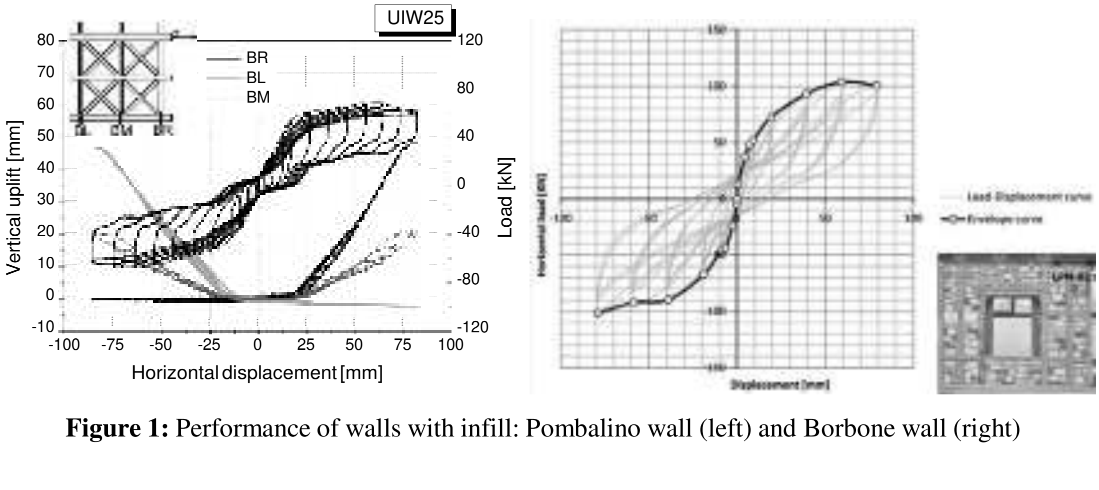


Figure 1: Performance of walls with infill: Pombalino wall (left) and Borbone wall (right)

For all different types of timber frame walls tested, the absence of infill led to lower values of maximum load, while the displacement capacity was maintained (Figure 2). The decrease was particularly relevant for the last two wall types, as the timber frame presented weak bracing and weak horizontal members in case of kay peyi walls and no bracing was present in case of Borbone wall.

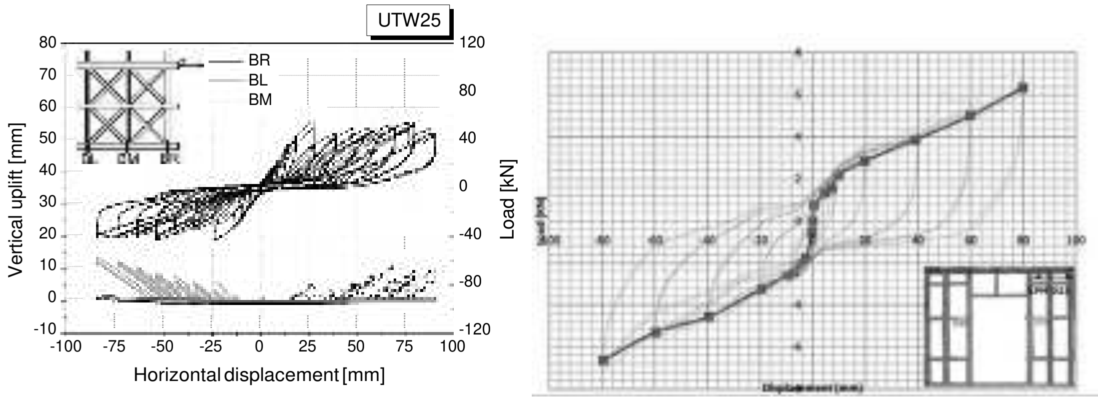
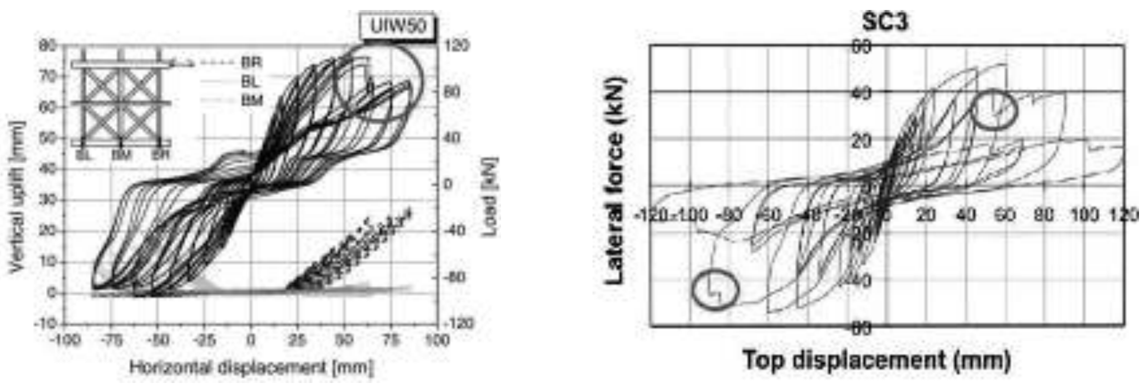


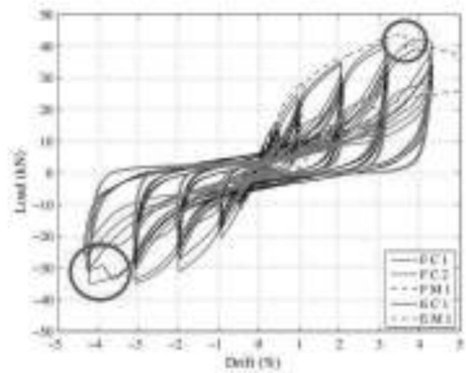
Figure 2: Performance of walls without infill: Pombalino wall (left) and Borbone wall (right)

For infill walls, a rocking response is clearly visible, with uplifting at the bottom corner, which led to failure of the bottom connection in case of kay peyi walls. A clear shear behaviour is instead observed for walls without infill, with greater deformations in timber elements and in some cases rupture of timber members.

Connections influence the global behaviour of the wall. Therefore, it is of the utmost importance to design ductile connections in order to prevent a brittle failure, even if local. Regarding frontal walls, the central joint prevents the rotation of the central beam leading in some cases to its brittle failure (see Figure 3a). In other examples encountered in Lisbon, frontal walls can be fixed by large nails in regard to the cross section of the wood that can lead to splitting of the wood (Figure 3b). In the case of Haitian shear walls, the uplift of the connection led to tension in the steel strips and to the pull-out of the nails. In some cases, a quite brittle failure is obtained, as depicted in Figure 3c, due to the fact that the head of the nail is cut off by the steel strip or the nail is pulled-out due to the too weak distance left from side of the wood.



(a) (b)



(c)

Figure 3: Typical response of timber walls: a) frontal wall [8]; b) frontal walls with large nails [12]; c) kay peyi wall [5].

An interesting study consists of understanding the importance of the diagonal bracing members. The absence of diagonal bracing members provides a lower ductility when compared to a similar braced wall [11], but the presence of a strong infill still provides still provides a good dissipation capacity and a good deformability of the wall. Therefore, bracing elements are important, but the quality of the infill (both mortar and bricks) should be addressed in order to accurately evaluate the capacity of a shear wall.

2.2 Seismic parameters

It has to be pointed out that a quantitative comparison of results is difficult given the different geometries adopted, different stiffness for infill materials as well as different vertical loads. However, a qualitative analysis is possible among results for all walls. Different parameters allowing to assess the seismic vulnerability of shear walls are here analysed for each specimen, namely maximum drift, stiffness, strength and stiffness degradation, dissipated energy and viscous damping.

Considering the values of initial stiffness for different types of walls, frontal walls gave high values of initial stiffness, with an average value of 3.03kN/mm for infill walls and 2.14kN/mm for walls without infill. These values depend on the quality of the infill material, since a stronger infill leads to higher values of stiffness [11].

The initial stiffness of kay peyi walls is significantly lower [5], 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. In fact, the initial stiffness of timber-frame walls filled with stones is approximately 0.8kN/mm.

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.

Frontal walls were able to dissipate great amounts of energy, with reported values of 21000kNmm for brick masonry infill and 14000kNmm for walls without infill [8]. Kay peyi walls, having weaker connections and no vertical load applied, reached values of dissipated energy of 11000kNmm for infill walls and of 7900kNmm for walls without infill [5].

Ruggieri et al. [4] reported a value of dissipated energy for the final cycle in the positive direction (drift of approximately 3%) of 3,982kNmm for infill timber frame walls of the baraccata system, which assuming an approximately symmetrical behaviour, is comparable to what found for frontal walls [8].

In terms of viscous damping, similar results were obtained for different types of timber frame walls.

Gonçalves et al. [13], on 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.

Vieux-Champagne et al. [5] obtained 0.15 for infill walls and 0.17 for timber frame walls with no infill with values approximately constant throughout the test. Dutu et al. [11] obtained values varying between 0.8 and 0.16, with values increasing with damage accumulation.

2.3 Experimental results vs. in-situ observations

Experimental results have shown the good seismic potential of timber framed structures with different geometries and different infill materials. But in many cases, no subsequent seismic events took place to verify the actual behaviour of such buildings (e.g. the return period for the Lisbon earthquake is 250 years and no big events occurred after the 1755 earthquake).

Half-timbered structures have shown throughout the centuries their good capability in resisting seismic loads. There are also several post disaster observations reporting poor performance of timber structures, generally related to poor maintenance, timber decay, structural alteration of the original building and improper connections.

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.

The Greek timber-framed buildings in the island of Lefkas withstood a strong earthquake in summer 2003. Damages were observed in both reinforced concrete buildings and traditional half-timbered buildings [6]. From a survey performed by Makarios and Demosthenous [14] 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.

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.

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. According to Gülhan and Güney [15], 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.

Many such examples are available, testifying that such a vernacular architecture, which uses local know-how and materials, is able to successfully respond to societal needs and is able, even today, to behave well in comparison to more modern and engineered architectures. Because of their good performance, timber frame buildings have been used for reconstruction plans of vernacular buildings in rural areas hit by catastrophes, such as Haiti and Pakistan.

3 CONCLUSIONS

- Timber frame buildings have shown a good seismic response in recent events worldwide and have been chosen for the reconstruction of rural areas.

- A great variability exists in terms of strength and stiffness capacity of timber-framed structures depending on their geometry (presence of diagonal bracing members and cross-sectional dimension of timber elements) as well as the quality of the infill material.
- Traditional vernacular timber-framed architecture proved to be able to withstand seismic actions and behaves appropriately even when compared to engineered architectures.

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SEISMIC ANALYSIS OF TIMBER FRAMES WITH INFILLS IN ROMANIA

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Keywords: Timber Frames, Masonry, Seismic analysis

Abstract.
Like in many places around the world, before using reinforced concrete, in Romania, most buildings were made of timber frames and different types of infill. As the technology evolved, this type of structure has not been used so much anymore and construction method is rarely remembered correctly. But recently, traditional houses' architecture started to draw attention for owners, and moreover, the fact that it involves natural materials makes it even more attractive.
At this moment, in Romania, for this type of structures there is no specific design method specified in the national Code P100-1/2013, and also no evaluation procedures for this existent type of building. This fact motivated the present study and after a field study near Vrancea seismic source area, three categories of structures based on timber frames and different types of infills (masonry, wattle and daub, earth) were found and focused on.
This paper presents the results of a simple modeling using a finite element program ETABS, calibrated to an existing experimental test, previously conducted in Japan, on a timber framed masonry wall. The same wall was evaluated according to the existing regulations in Romania, to check the accuracy of the evaluation results. The comparison can be made because the general mechanical principles were of the wall configuration studied in Japan, are also valid for the Romanian traditional houses.

1 INTRODUCTION

In Romania, in the last years the studies on earthquakes produced an increase in the awareness of the population and authorities. For example, the most seismic exposed cities from Romania are Bucharest and Iasi. In Bucharest, according to seismic code P100-92 [1] the ag (maxim expected seismic ground acceleration) was 0.20g and today, according to code P100-1/2013 ag is 0.30g (50% increasing) and for Iasi city, it was also 0.20g and increased to 0.25g (25% increase).

After the two major earthquakes that occurred in Romania on 10 November 1940 and 4 March 1977, there is not much information about traditional buildings with timber frame and masonry infill or other infills which suffered complete collapse or major damages. Thus people generally assume that traditional residential houses behaved well during seismic events.

Today, more owners want to build such traditional houses with infilled timber frame structure, because they are easy to build, relatively cheap, ecologic, aesthetic and, the most important, as the recent studies have shown, they have a satisfactory seismic resistance and especially a high ductility, aspect also revealed by the past seismic events. In this moment, in Romania, for this type of structures there is no specific design method specified in the national Code P100-1/2013, and also no evaluation procedures for this existent type of building.

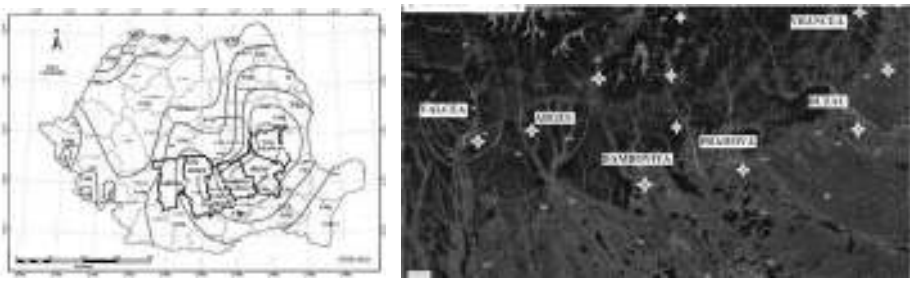
2 TRADITIONAL HOUSES WITH TIMBER SKELETON AND VARIOUS INFILL FROM ROMANIA

Romanian is one of the most earthquake prone country in Europe. The Romanian seismic source is a point source in Vrancea region, which was geographically defined as 40x80 km^2 [2] (Figure 1a) and is located at the confluence of three main tectonic plates as: East-European Plate, Intra-Alpine (subduction) Plate and Moesic Sub-Plate (Figure 1b) [3]. Also, Vrancea source is located at the influence of active deformation areas Adriatic, Aegean and Vrancea, through ALCADI Panonic system (Figure 1c) [3]. The Vrancea source can generate major earthquakes with very large focal energies released, around 2-3 seismic events/ century, that could have magnitudes on Richter – Gutenberg scale of $M_{G\text{-}R}=7\div7.5$.

a) b) c)

Figure 1: – a) The epicenters distribution of Vrancea source **[2]**; b) Tectonic dynamic of Romanian territory **[3]**; b) Deformations transfer through ALCADI Panonic system **[4]**.

As worldwide, especially in seismic countries (i.e. Greece, Portugal, Turkey, Italy, China, Myanmar etc.), the traditional houses with timber frames and various infills can be found also in Romania. The previous studies shown that generally the traditional houses with timber skeleton and various infill were built mainly in seismic regions nearby the material's sources (wood, stone, clay), such as the mountain regions (where there are forestry and quarries) or hill regions. Thus, in order to study the seismic behavior of the traditional houses in Romania, some regions were selected, located near the Vrancea source (Figure 2a) and nearby mountain and hill regions in Buzau county, Vrancea county, Dambovita county, Prahova county, Arges county and Valcea county (Figure 2b).



a) b)

Figure 2: a) The seismic location of the investigated regions; b) The geographic location **[5]**.

A number of 129 traditional houses were investigated and the following statistics were obtained: 80% of them are with timber skeleton and brick masonry infill structure (Figure 3a); 15% are with timber skeleton and strips applied at 45^o and clay plaster (Figure 3b); 5% with timber skeleton and wattle & daub; additionally, 73% of them are between 60-90 years old; 14% of them are older than 100 years (Figure 4a) and 13% are younger than 60 years old (Figure 4b). Even though they aren't traditional ones, houses with timber skeleton and AAC (autoclaved aerated concrete) masonry infill was found.

a) b) c)

Figure 3: a) Traditional house with timber skeleton and brick masonry infill from Dumitreştii de Sus village, Vrancea county b) Traditional house with timber skeleton and strips applied at 45o and clay claster from Mustăţeşti, Argeş county; c) Traditional house with timber skeleton and wattle & daub from Băbeni, Vrancea county

a) b)

Figure 4: a) Traditional house with timber skeleton and wattle & daub over 100 years, from Băbeni, Buzău county; b) Traditional house with timber skeleton and brick masonry infill less then 60 years (~20 years), from Buzău county

A particular issue was observed during field investigations, all traditional houses are only with one story, except one (Figure 5a), unusual for Vrancea county, because it is built in “Fachwerk” style, just like Peleş Castel from Sinaia (Figure 5b).

a) b)

Figure 5: a) Traditional house with timber skeleton and masonry infill (only at 1st story and attic), from Chiojdeni, Vrancea county; b) Peleş Castel from Sinaia, Prahova county

Other important technical aspects are: the foundations are made only from stone (river rocks – Figure 6a) with or without earth mortar; always the base of timber frames is done from hardwood (oak tree, locust tree, etc.), able to sustain the moisture conditions, and the other are generally from softwood (pinewood); the timber structure is not embedded in foundations, it is only simply supported; the timber frame structure is built by vertical and horizontal elements and bracings, which are positioned at the corner’s and intersections’ structure, but not always in coherent distribution (Figure 6b); the joints are cross-halved (Figure 6c); steel clamps are added to increase the resistance and stiffness of the joint; almost all traditional house investigated have veranda; the roof covering of a traditional house, in traditional solution was made by wood shingle, but now most of them are replaced because they were damaged (biological decay).

a) b) c)

Figure 6: a) Stone foundations (river rocks); b) bracings positioned at the corner’s house; c) cross-halved joints

From locals’ testimony (personal communication) the past earthquakes didn’t affect seriously this type of traditional houses. Some damages occurred after earthquakes, which were repaired immediately, but the serious damages were caused by the xylophage bacteria attack and moisture (lack of drainages, roof damages). Most of them are abandoned and they aren’t maintained properly; also, the geotechnical phenomena (settling, landslides, etc.) mainly caused cracks and walls overturning. Anyway, traditional houses withstand over years due to their reduced size dimensions (one story and maximum ~7 x15 m^2), remarkable constructive details, but also due to their ecological feature (natural material: stone, wood, earth). Today they are still building, but more rarely, because they are not promoted properly and the people are losing more and more the traditional construction method.

3 EVALUATION ACCORDING TO THE REGULATIONS IN ROMANIA

This chapter will present the evaluation of one wall according to the regulations in Romania. We choose to evaluate a timber framed wall with masonry infill, tested in Japan in a static cyclic regime. The analysis is compared with the experimental results [6], so later the evaluation can be adapted for the Romanian specific timber frames with infills. The present research is a continuation of the one conducted in Japan. The dimensions of the specimen are presented in Figure 7. The bricks were made in Japan whit the dimensions 210x100x60mm and the mortar recipe is 1:2:6 (cement:lime:sand). According to material tests the average compressive strength (fm) and Young's Modulus of mortar is 8.35 MPa respectively 13.01 GPa and the compressive strength and Young's Modulus of bricks is 57.6 MPa, respectively, 16.8 GPa. Young's Modulus for both mortar and bricks was calculated as the secant stiffness corresponding to one third of the maximum strength. There were also prism compression tests. The results are presented in Table 1 and Table 2 [6].

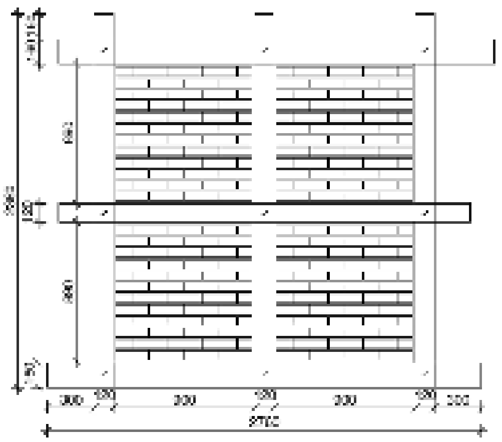


Figure 7: Timber-framed masonry wall [6]

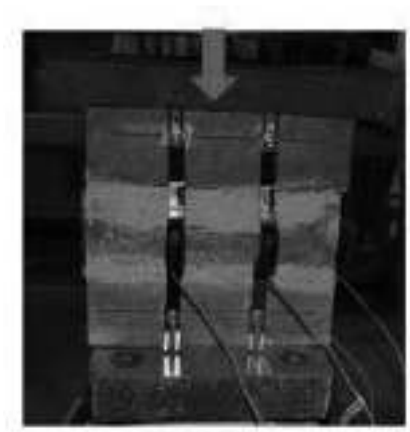


Figure 8: Prism compression setup **[6]**

Table 1 : Test Results for materials

Material	Strength (f_m) [MPa]	Young's Modulus [GPa]
Mortar	8.35	13.01
Bricks	57.6	16.8

Table 2 : Prism Compression Test Results

Specimen	Compression strength [MPa]	E masonry [GPa]
1	42.6	1.7
2	31.4	1.3
3	36.4	2.6

3.1 CR6-2006 [7]– Design code for masonry structure in Romania

Below are summarized the equations proposed by this regulation.

$$\boldsymbol{f_d = m_z * \frac{f_k}{\gamma_m}} \tag{1}$$

where: f_d = design compressive strength, m_z = coefficient of working conditions, f_k = characteristic compressive strength, γ_m = safety factor for material

$$\boldsymbol{f_{vd} = m_z * \frac{f_{vk}}{\gamma_m}} \tag{2}$$

where: f_{vd} = design shear strength, m_z = coefficient of working conditions, f_{vk} = characteristic shear strength, γ_m = safety factor for material

Taking into account the low number of tests performed on the materials, we will consider the lowest value of the characteristic compressive strength obtained from experimental test. According to Table 2 this value is f_k = 31.4 MPa.

Design compressive strength is calculated by reducing the characteristic compressive strength by the coefficient of working conditions and by the safety factor for material.

m_z depends on the brick section and mortar recipe. In this case, m_z was consider = 0.85 because the brick section is less than 0.30 m^2.

γ_m depends on brick type, mortar recipe and limit state. For ULS (ultimate limit state), γ_m is 2.5.

Characteristic shear force (f_{vk}) is obtained from standard compressive strength, and initial shear strength considered 0.3 N/mm^2 and normalized axial force. Axial force was considered =60 kN.

This code does not require the calculation for diagonal tension cracking.

Figure 9 and Figure 10 present the results given by CR6-2006.

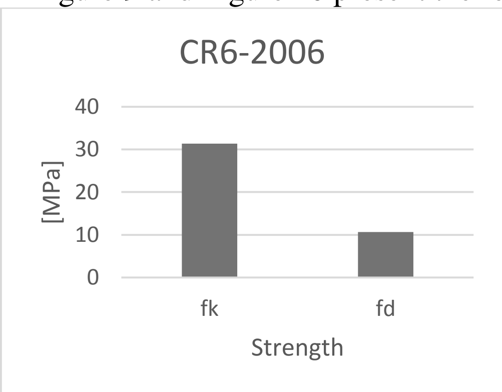


Figure 9: Compressive strength according to CR6-2006

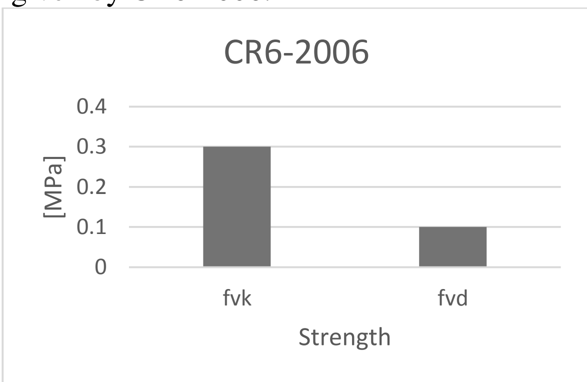


Figure 10: Shear strength according to CR6-2006

Design shear force capacity V_{Rd} is calculated with the following equation:

$$V_{Rd} = f_{vd} * t * l_c \tag{3}$$

where: V_{Rd} = design shear force capacity, f_{vd} = design shear strength, t = wall thickness l_c = the length of the compressed area of the wall.

Because of the length of the compressed area of the wall, V_{Rd} depends on the axial load. Because the whole wall is compressed, l_c = 2.16 m. It results that V_{Rd} = 216 kN

3.2 CR6-2013 [8] – Design code for masonry structure

This regulation replaces CR6-2006 and brings some modifications.

Beside the design compressive strength (noted f_d) and shear sliding in the joint (noted f_{vdl}) calculated as above (Equation 1 and Equation 2), in this case, CR6-2013 proposes the calculation of diagonal tension cracking (noted f_{vdi}). Its equation is found below.

$$f_{vdi} = m_z * \frac{f_{vki}}{\gamma_m} \tag{4}$$

where: f_{vdi} = diagonal tension cracking strength, m_z = coefficient of working conditions, f_{vki} = characteristic tension cracking force, γ_m = safety factor for material, and here is 2.2.

Characteristic tension cracking force (f_{vki}) is obtained from masonry tensile strength and normalized axial force. For both shear sliding and diagonal tension cracking force, axial load was considered = 60kN.

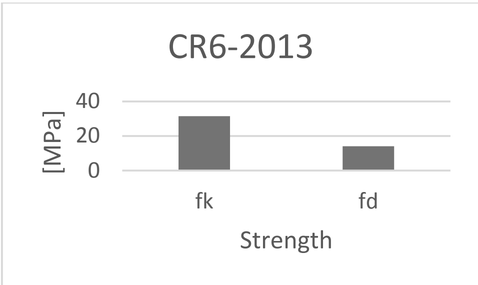


Figure 11 : Compressive strength according to CR6-2013

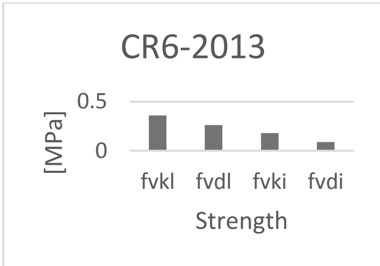


Figure 12 : Shear strength according to CR6-2013

Design shear force capacity V_{rd} is minimum between shear sliding and tension cracking strength. Design shear sliding strength $V_{rd,l}$ is calculated with the following equation:

$$V_{Rd,l} = f_{vd,l} * t * l_c \tag{5}$$

where: $V_{rd,l}$ = design shear sliding force capacity, $f_{vd,l}$ = design shear sliding strength, t = wall thickness, l_c = the length of the compressed area of the wall

Because of the length of the compressed area of the wall, $V_{Rd,l}$ depends on the axial and lateral load.

Diagonal tension cracking force capacity $V_{rd,i}$ is calculated with the following equation:

$$V_{Rd,i} = \frac{A_w}{b} * f_{vd,i} \tag{6}$$

where: $V_{rd,i}$ = diagonal tension cracking force capacity, $f_{vd,i}$ = diagonal tension cracking strength, b = coefficient depending on wall dimensions, A_w = area of the wall section.

In this case b=1 and $V_{Rd,i}$ = 18.36 kN

3.3 P100-3/2008 [9] – Seismic evaluation of existing buildings

This seismic evaluation code uses as a reference point the mean value of compressive strength to calculate the standard compressive strength (Equation 7). As test showed, this value is 36.8 MPa (Table 2).

$$f_b = 0.85 * f_m \tag{7}$$

where: f_b = standard compressive strength, f_m = mean value of compressive strength

This time, in order to obtain the design compressive strength, one should divide the mean value of compressive strength by two factors. (Equation 8).

$$\boldsymbol{f_d} = \frac{\boldsymbol{f_m}}{\boldsymbol{CF*\gamma_m}} \tag{8}$$

where: f_d = design compressive strength, f_m = mean value of compressive strength, CF = confidence factor, γ_m = safety factor for material

Regarding CF, we can choose from three confidence factors depending on how much we know about the building. If tests were done on materials and also exact measurements on site, it can be considered that there is a full knowledge over the building and CF = 1.

The safety factor for material, according to P100-3/2008, depends on the age of the brick and mortar recipe. In our situation, we can choose γ_m = 2.75. This value corresponds to old buildings (1900 to 1950) and mortar with lime and cement.

To evaluate the shear strength, we should determine the failure mechanism: shear sliding or diagonal tension cracking.

Equation 9 and Equation 10 describe the calculation method for shear sliding, respectively, diagonal tension cracking.

$$\boldsymbol{f_{vd}} = \frac{\boldsymbol{f_{vk}}}{\boldsymbol{CF*\gamma_m}} \tag{9}$$

where: f_{vd} = shear sliding, f_{vk} = characteristic shear force, CF = confidence factor, γ_m = safety factor for material

f_{vk} (characteristic shear force) is obtained from standard compressive strength, initial shear strength considered 0.045N/mm^2 and normalized axial force. Axial force was again considered 60 kN.

$$\boldsymbol{f_{td}} = \frac{\boldsymbol{0.04*f_m}}{\boldsymbol{CF*\gamma_m}} \tag{10}$$

where: f_{td} = tension cracking, f_m = mean value of compressive strength, CF = confidence factor, γ_m = safety factor for material

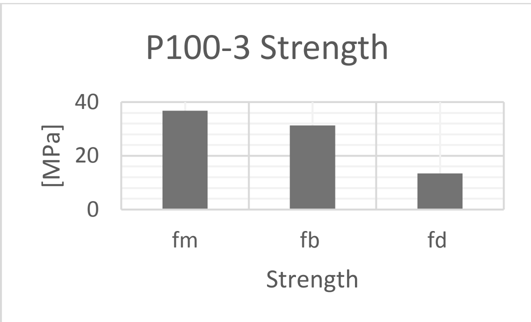


Figure 13 : Compressive strength according to P100-3/2008

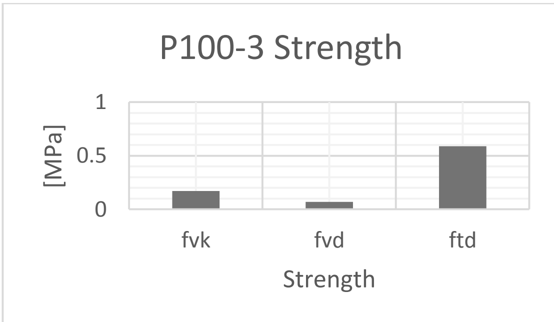


Figure 14 : Shear strength according to P100-3/2008

Design shear force capacity V_{Rd} is minimum between shear sliding and tension cracking strength.

Design shear sliding strength V_{f21} is calculated with the following equation:

$$\boldsymbol{V_{f21}} = \frac{\boldsymbol{1.33}}{\boldsymbol{CF*\gamma_M}}\left(\boldsymbol{f_{vk,0}} * \frac{\boldsymbol{l_{ad}}}{\boldsymbol{l_c}} + \boldsymbol{0.7} * \boldsymbol{\sigma_d}\right) * \boldsymbol{t} * \boldsymbol{l_c} \tag{11}$$

where: V_{f21} = design shear sliding force capacity, CF = confidence factor, γ_m = safety factor for material, $f_{vk,0}$ = initial shear strength considered, l_c = the length of the compressed area of the wall, l_c = adhesion length, σ_d = normalized axial force, t = wall thickness

If $l_{ad} \leq 0$ than V_{f21} will have the following value:

$$V_{f21} = 0.93 * \frac{N_d}{CF * \gamma_M} \tag{12}$$

where: V_{f21} = design shear sliding force capacity, CF = confidence factor, γ_m = safety factor for material, N_d = axial force

All these terms, except adhesion length have been discussed above. This term depends also on axial and lateral force.

Tension cracking strength V_{f22} is calculated with the following equation:

$$V_{f22} = \frac{t * l_w * f_{td}}{b} \sqrt{1 + \frac{\sigma_0}{f_{td}}} \tag{13}$$

where: V_{f22} = tension cracking force capacity, t = wall thickness, l_w = wall length, f_{td} = tension cracking strength, b = coefficient depending on wall dimensions, σ_d = normalized axial force. In this case V_{f22} = 38.27 kN.

3.4 Comparison between CR6-2006, CR6-2013 and P100-3/2008

As it can be seen in the charts below, evaluation of existing building code uses a compression strength higher than the design codes for new buildings. It is an expected outcome taking into account the for new buildings we follow the condition that at least 95% of the test to be above the value of f_k and for evaluating existing building we use the mean value of compressive strength which is higher.

One can also notice the difference between design codes for new masonry structures in terms of design strength. CR6-2013 [8] uses a safety factor with a lower value than CR6-2006 [7] resulting in a higher design strength. The new design code, from 2013 gives higher design strength for shear and uses a value for tension cracking.

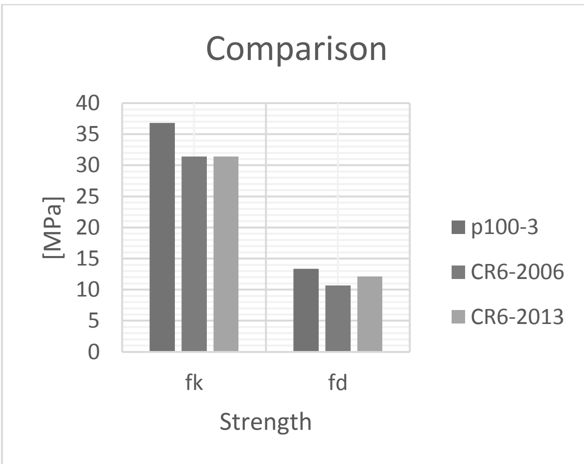


Figure 15 : Comparison of compressive strength

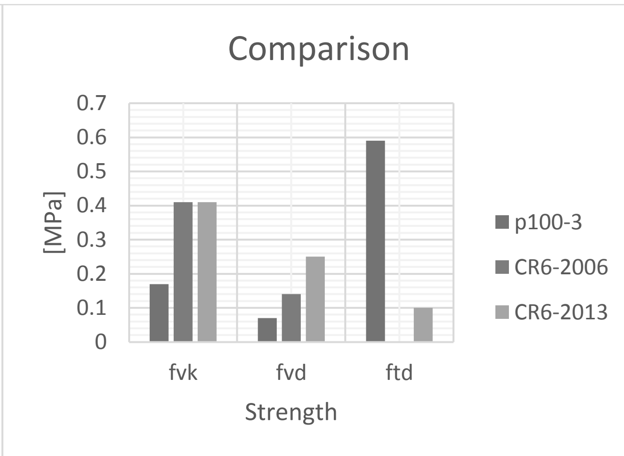


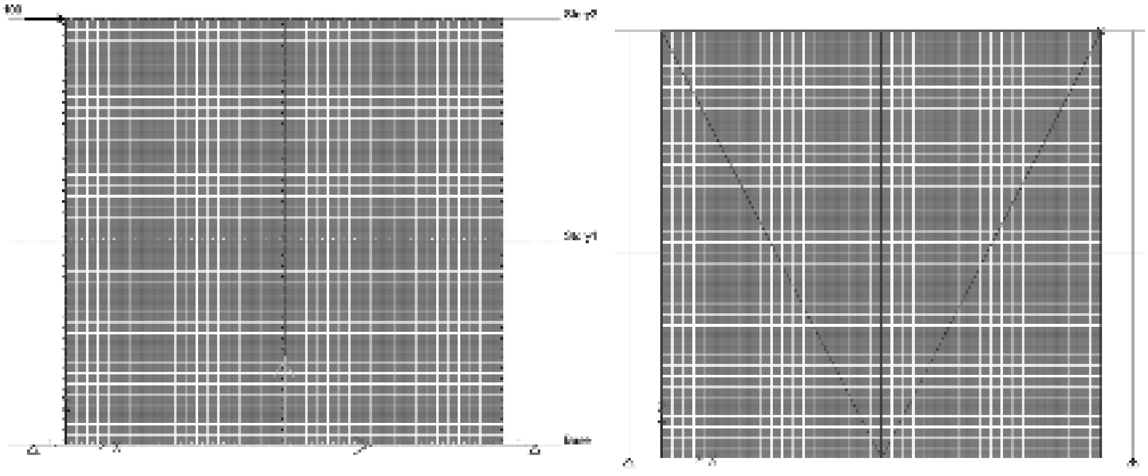
Figure 16 : Comparison of shear strength

4 FINITE ELEMENT MODELING

For numerical model, finite element software ETABS was used. The aim was to reproduce the test model's behavior whit a minimum computational level as everyone uses in current design.

Timber frame (beams and columns) were modeled as linear elements that intersect each other in their central axis. For simplicity, timber was defined as an isotropic material with Young's modulus obtained from test (11.92 GPa). The masonry panels were modeled using shell element with Young's modulus corresponding to characteristic compressive strength (1.3 GPa). Fixed support conditions were assumed as the experimental set-up of the wall [6].

Figure 17 a) presents the model made in CSI ETABS for the Japanese specimen and Figure 17 b) shows a conformation found in traditional Romanian houses. For more accurate results, masonry panels were meshed. The force was applied in the top left corner. For Romanian model ere used the elasticity characteristics for a mud masonry brick, determined through tests, and the minimum elasticity characteristics for timber since these tests were not made yet.



a) b)

Figure 17: ETABS Model for a) Japanese specimen, b) Traditional Romanian house

Figure 18 shows the envelope of the experimental result against the ones obtained using Romanian designing codes and the numerical models for both Japanese and Romanian conformation. It can be noticed that using the full stiffness for timber and masonry as an input for the finite element program, we obtain significantly lower comparing with the experimental behavior. To calibrate it to the test results, the ETABS model was modified by using only a fraction of stiffness for each type of element. The aim was to obtain a numerical model with an output that approximates the test results.

The best option in this case was the use of 10% of the actual stiffness of the elements. This input helps to model a wall with approximately the same stiffness as the one tested. The slope of the Top Force-Displacement chart is close to the real curve.

Similar results were presented in [10], where it is also presented a comparison between test result and numerical modelling, in this case using SAP2000 software. The situation described above (lower displacements comparing with experimental behavior) was the same, the authors reduced the geometrical stiffness of the timber frames and masonry panel by a factor of 37 to make the overall model stiffness the same as the experimental target value [10].

Regarding the values of the capable shear forces, calculated according to the Romanian codes, one can observe major differences between the three of them and between them and the real curve. However, it should be kept in mind that these regulations provide an approximate global calculation with coverage coefficients to avoid both design and execution errors.

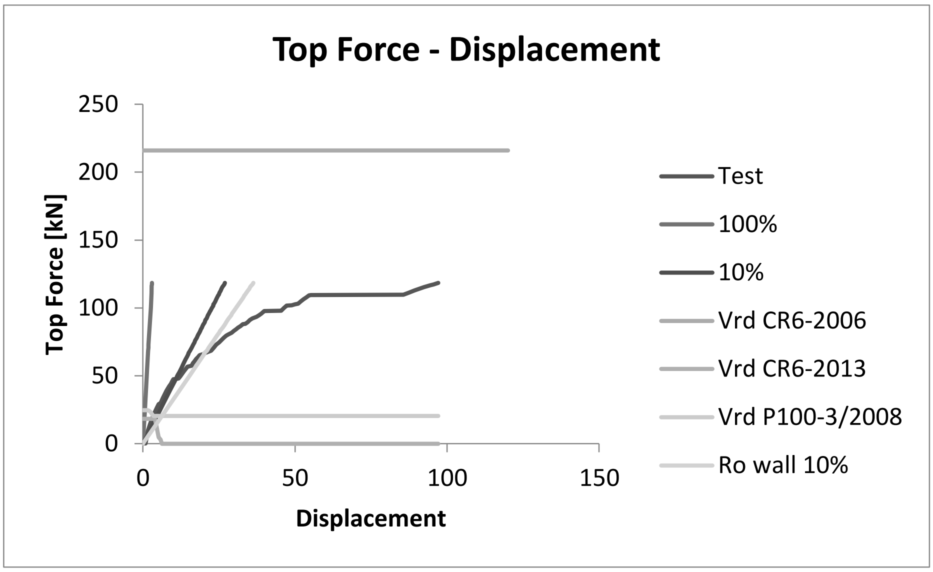


Figure 18: Top Force-Displacement for different fractions of rigidity

These low values could have various explanations, but the most important is that the masonry infill detaches quickly from the timber elements. In this case, the frame and shell do not act like a unified wall anymore. The masonry will be only an infill, and not a structural element.

It is also interesting to see which is the appropriate fraction of stiffness to be use for different types of frames. For this rectangular frame, 10% of initial stiffness was obtained, while for a frame with St. Andrew's crosses, 2.7% was obtained [10].

5 CONCLUSIONS AND FUTURE DIRECTIONS

As it was presented at the beginning, timber frames with various infill houses has been and still are quite used around the world. Although it may seem an outdated method, the rustic look, the low price and the ease of purchasing the materials turn them into a desired home.

In Romania, the current regulations exclude such houses from the beginning. Materials in this combination, are considered too weak to meet the resistance and stability requirement. The characteristic values of compressive strength and shear force are far inferior to the materials commonly used during this period.

It is easy to see the evolution of the legal regulation regarding the new constructions but also the existing ones. CR6-2013, the newest code for masonry buildings, comes with more requirements (the calculation of diagonal tension cracking) but also with lower values of the coefficients for limit states. P100-3/2008, the only regulation for existing buildings, refers to CR6-2006 for equations and coefficients, even if is not used anymore. The major difference between these two is that P100-3/2008 uses as a reference point the mean value of compressive strength to calculate the standard compressive strength the same way we use now for nonlinear analysis for new buildings.

For the finite element modeling, there is also a discrepancy between real behavior and computational modelling. Software commonly used for design do not capture all the small phenomena, so in order to achieve an output closer to reality, it is necessary to use a more advanced program, in which each element to be modeled separately, and especially the interaction between them. However, for regular engineers, simplicity of modeling is very important.

6 ACKNOWLEDGEMENT

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CONSTRUCTION SYSTEMS OF TIMBER STRUCTURES IN CHINA AND ITALY: A FIRST COMPARISON OF CONSERVATION APPROACHES

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Keywords: Construction Systems, Timber, Comparison, Conservation Approaches

Abstract

Timber is one of the construction materials widely used in historical structures all over the world. Particularly in China, main historic building and architectural heritage is made of timber structure, the traditional system is a pure wooden framework, which two main ones are Tai-liang and Chuan-dou. They are composed by vertical columns that begin at ground level, short vertical supports as struts, and horizontal beams or lintels connecting the struts and columns. In Italy, instead, timber components, as floor and roof, are included in the building structural system, commonly made of masonry.

In the paper, the main aspects concerning the two construction systems are compared, in particular, the following categories are taken into account: Typologies of construction systems, are mainly demonstrated the framework in macroscopic way and other minor carpentry work in microscopic way; Composition of construction systems, which include main structural composition and method of elements' connection; With the great different and distant timber technical traditions between two countries, aiming at illustrating the basic requirement of conservation approaches.

The comparison among the various constructive systems between Italy and China, according to the analyzed aspects, constitutes a preliminary but fundamental step in the process of preservation of historical materials and structural behavior of timber components. It represents a crucial step for the reliability of investigation and experimental methods.

1 INTRODUCTION

To understand conservation approaches of traditional and historic timber constructions require the preliminary and fundamental knowledge of construction system. With the long history of Chinese and Italian timber structural architectures, both of them have developed a distinctive construction system to support the whole building.

From the perspective of configuration of the building, Chinese traditional architecture has developed a more notable character from configuration, is divided into 3 parts, the roof, the body and the base (Fig.1), the distinctive configuration generated on account of function, structure and artistic level, which are mutually combined. The wooden framework was adopted in traditional construction system, which contains the roof and body, is comprised of longitudinal column and transversal beam. The area of enclosed 4 columns make one "Jian", the standard area of a room in traditional way, a building is composed by several "Jian". The different way of overlapping and jointing on columns, beams and other elements generate two main frame systems in China, which is Tai-liang and Chuan-dou, which will be described in following context. The weight-bearing frame provides flexibility for the placement of walls, windows, and doors. Non-load-bearing walls can be made of many different materials, such as brick, clay, wood, bamboo, or even corn or cotton stalks.

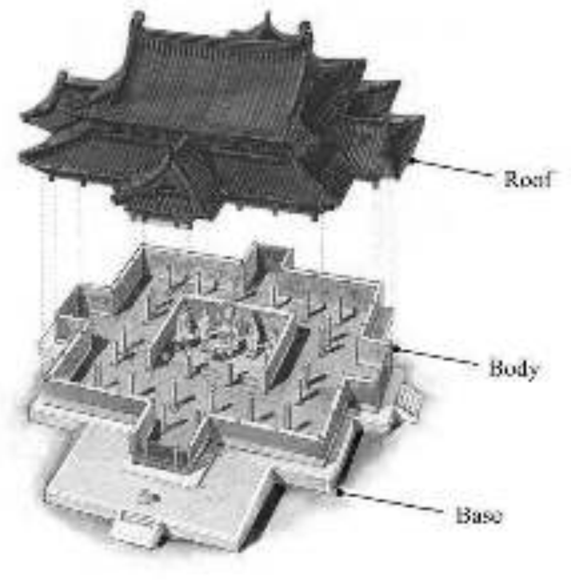

Figure 1: Configuration of Chinese ancient building

Compared with configuration of Italian building, the situation is too complex to make a typological conclusion, however the roof can be discussed, which the presumably of the simplest type is made of two inclined struts, king post and horizontal tie beam. Its distance covering from 5 to 15m, with the most basic prototype, it can be transformed into various complex types. Regardless of the basic simple truss and transformations, the composed truss, both of them are the most important structural part of the whole building.

2 TYPOLOGIES OF CONSTRUCTION SYSTEMS

2.1 Framework based upon roof

Following the Chinese traditional timber categorization methodology, namely the "major carpentry" viewpoint which is based upon the structural techniques of the roof, are divided into two main types: “Tai-liang” and “Chuan-dou” structure. The difference between these two kinds of framework is specifically the way of connection between the beam (lintel) and columns (Fig.2). Tai-liang means there is a big beam between two peristyle columns, on which another beam support purlin, the horizontal elements that support the rafters, are positioned along the stepped shoulders of the skeleton, such as The Hall of Supreme Harmony (Taihedian) at the center of the Forbidden City in Beijing (Fig.3). The second type of structure is “Chuan-dou”. In this kind of structure, the entire skeleton of the building consisted of columns, purlins, struts, lintels. The purlins are located on the top of the columns or struts, which are connected by the horizontal lintels and architraves. A typical example is the traditional dwellings in Southern China (Fig.4). Tai-liang was mostly built in North area of China, especially prevalent applied in the official or royal building. And Chuan-dou is usually built in South area of China, and because of climate, it tends to be more ventilated and lighter. In addition to the two main types illustrated above, there is another type, Jing-gan. In this technique, wall could be obtained by using rows of logs placed horizontally and without column and beam structures. The construction systems can be met diverse requirements of building types, ranging from royal palace, temple, dwelling, garden building to tower and bridge. The existing typical cases, such as the great hall of Foguang temple, Wutai mountain, in Shanxi; Guanyinge of Dule temple, in Jixian, Tianjin; wood pagoda, in Ying xian of Shanxi, etc. As well as Ling’en Palace, Chang Tomb of Emperor Yongle, the biggest palace hall in China ancient time, whose area is nearly 2,000 m^2.

Dong drum tower, is one of most important and charismatic public buildings in the architecture of Chinese Dong minority, the existing drum tower were mostly built from early Qing dynasty to fairly recent years[1]. Timber is the unique material used in constructing the Dong drum towers, furthermore, it contains both Tai-liang and Chuan-dou types. Taking Yan Lan drum tower as an example, is a typical Tai-liang drum tower (Fig.5), which was erected in A.D. 1764(Qing Dynasty) in Yan Lan village, Tongdao county of Hunan province. What’s more, it is combined with another auxiliary building such as village gate and temple, thus forming a complex with special shapes along with a variable public space within the village.

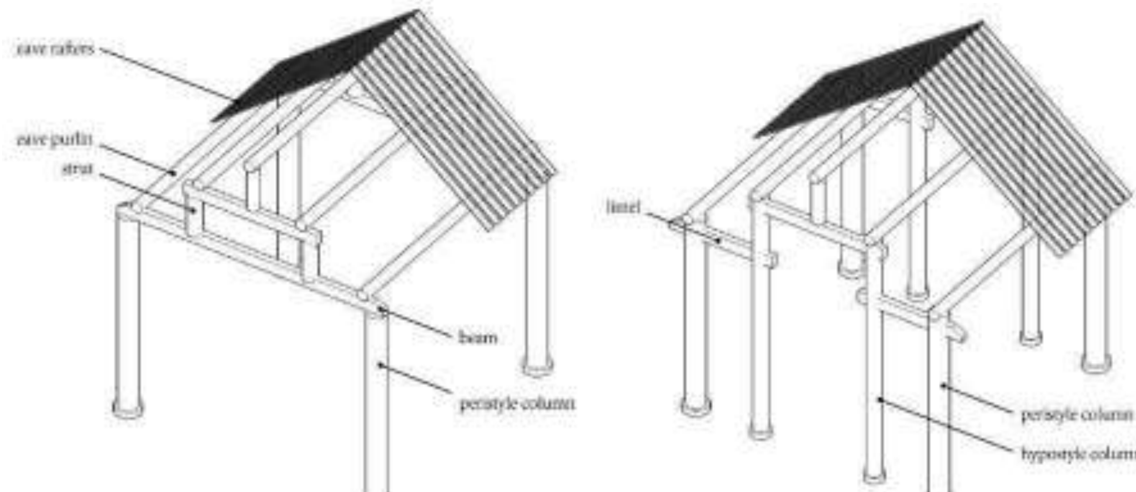

Figure 2: Tai-liang(the left one) and Chuan-dou(the right one) structure

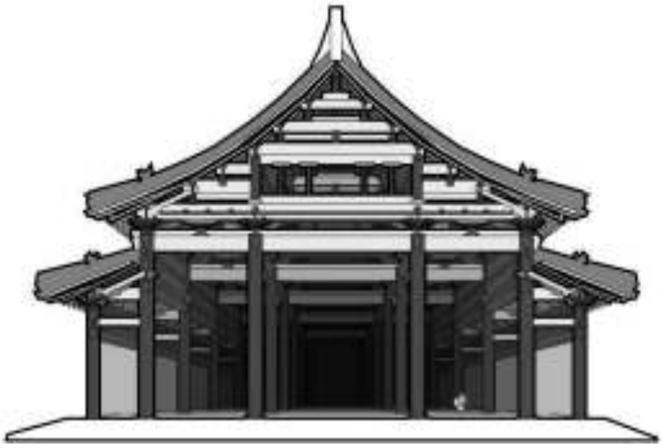

Figure 3: The Hall of Supreme Harmony (Taihedian) in Beijing, China (A.D. 1695)[1]

Figure 4: Traditional dwelling, Fujian, China[2]

Chuan-dou applied in Dong drum tower exiting a subtle difference with Han nationality's official one, the plan of Chuan-dou drum towers are usually a regular polygon. There is a central post (touching ground) or a king post (suspended above the ground) being located in the geometric center of a regular polygonal plane, rising up to the top. Using this kind of construction technique, the roof style of drum tower is multi-eave pyramidal roof makes the style is symbol of Dong minority. The Dong Chuan-dou drum towers are generally more than five stories or eaves (Fig.4). The drum towers take full advantage of this type of construction by expressing the creativeness and development of the traditional timber structure of China.

[1] Re-drawn from http://chounamoul.exblog.jp/15878540/
[2] From Qianlang Li. 2009. *Through the Wall*. Guangxi Normal University Press, Naning, China.

(a)

(b)

Figure 5: Yang lan drum tower

(a) 3D model; (b) Appearance

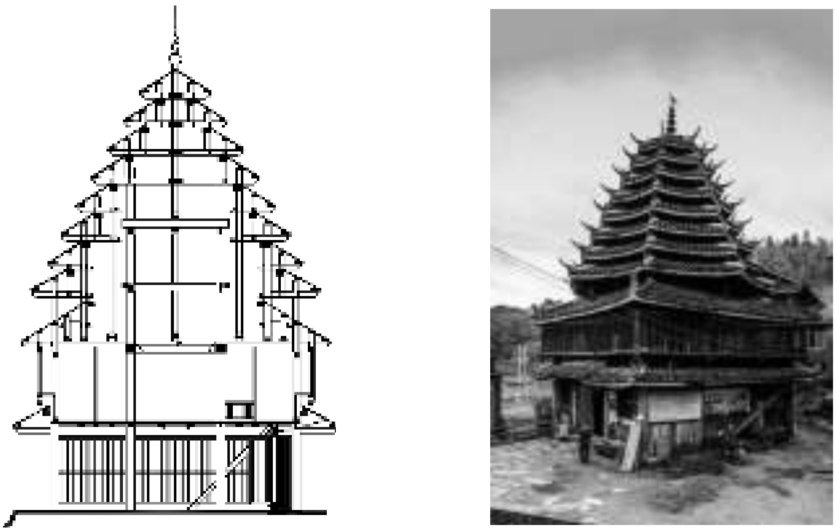

Figure 6: Gao sheng drum tower

On the other hand, there are so many kinds of roof structures in Italy that the precise classification on typology of roof cannot be concluded directly, however the most common one is a triangle truss composed by a horizontal beam, on which a vertical king post and two inclined struts connected both beam and king post with iron ties (Fig.7). Others can be regarded as the transformations, such as hammer beam truss, or even a structure with only rafters and inclined beam. There are two UNESCO Word heritages in Italy, i.e. Basilica of St. Apollinare in Classe (Fig.8), Ravenna, Emilia-Romagna Region, was erected in beginning of 6th century, another one is Patriarchal Basilica of Aquileia (Fig.9), Aquileia, Udine, Friuli-Venezia Giulia

Region, which was erected in A.D.1031, then rebuilt in A.D.1379. Both are covered by a timber structural roof, despite the diverse types of construction techniques.

Figure 7: Simple truss (prototype)

Figure 8: Basilica of St. Apollinare in Classe [3]

Figure 9: Patriarchal Basilica of Aquileia[4]

Some significant examples can be found in Veneto region, as follows.

The "Arsenale" of Venice in Italy, is a complex of former shipyards and armories. It transformed more and more complex from years to years. The building complex under consideration is formed by the roofs, very simple timber trusses at the beginning (first half of the 14th century), large span, huge and complex spatial timber structures during 16th century, engineered mixed steel-timber and steel trusses in the second half of the 19th century [2]. The most original and simple type of truss indicates in this paper was constructed in "Corderie" and "Isolotto" (Fig. 10, 11). The examples characterize the structural technique that widely used in the roof constructions.

In the case of San Fermo, a XI century's church located in Verona of Italy. The wooden roof is a distinctive structure that composed by a large series of struts and puddled iron ties, which is one of the types of structures with complex truss, to be more precisely, the hammer beam truss (Fig.12). The roof is a composition of many timber beams with a large series of elements like struts, decorative parts, puddled iron ties and many other connection devices, it is a sort of arch-braced structure, then the vertical load force upon the lateral walls of the church. It's an open truss with a span of 18m that not far from the widest one of Westminster Hall [3]. After analyzing above, utilizing hammer beam truss may widen the span the truss.

Both of Tai-liang and Chuan-dou are mortised mutually by horizontal and vertical structural elements, the load is transmitted upward through struts and beams to the top of roof, then the gravitational forces downward out through from the framework to the ground. Thus, a circulatory construction system is generated. Therefore, the elements consume the seismic energy to insulate and reduce the vibration by their interaction. According to the theory of modern vibration control [4], the construction system is a passive reduce-seismic system. Compared with the construction system of China, Italy also using a distinctive system that timber com-

[3] From https://arsartisticadventureofmankind.wordpress.com/2014/10/24/golden-age-of-byzantine-art-iii-churches-of-ravenna-santapollinare-nuovo-santapollinare-in-classe-and-san-vitale/

[4] From https://en.wikipedia.org/wiki/Aquileia

ponents included in the structural system of masonry constructions. The timber structure is not the only material of the construction system, but it still plays an important structural role.

Figure 10: “Corderie” Truss[5]

Figure 11: “Isolotto” Truss[6]

Figure 12: The wooden roof- San Fermo

2.2 Other typologies: minor carpentry work

In addition to the framework, there still are other minor carpentry works, including door, window, ceiling, caisson, floor, etc.

The door and window in China may be classified into one unit, because both of them utilizing the similar method of work, which is made of frame and casement. And the method of work on door and window is different depending on different level of architectures. The “Jian-chuang (balustrade window and door)” (Fig.13) is used in high level or significant architecture, like royal temple. The “Zhi-zhai (removable window and door)” (Fig.14) is used in secondary architectures, like folk dwellings.

The carpentry method of caisson in China and Italy is similar, is made of grid shape structure by wooden kneel, on which is a ceiling. Chinese caisson tends to be decorated in more palatial way (Fig.15[7], 16[8]).

Figure 13: Jian-chuang

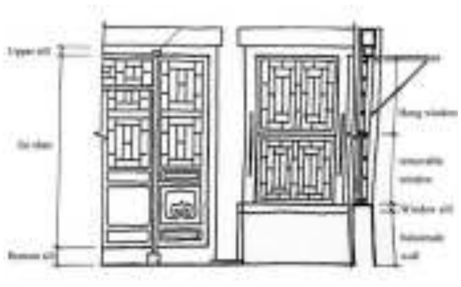

Figure 14: Zhi-zhai

[5] From http://www.artmoorhouse.com/artists/antonio-martinelli-gallery.php

[6] From http://www.artribune.com/attualita/2015/05/biennale-di-venezia-il-padiglione-dellamerica-latina-raccontato-da-alfons-hug/

[7] http://tiscsvr.tbroc.gov.tw/photo.asp?phrfnbr=11205

[8] From the PowerPoint “La diagnosi delle strutture lignee in opera: criteri, mezzi, operatività” of Massimo MANNUCCI. Pp23.


Figure 15: Caisson in China **Figure 16:** Caisson in Italy

3 COMPOSITION OF CONSTRUCTION SYSTEMS

3.1 Main structural elements

There are tremendous variety of structural elements, the most typical and primary are concerning below.

3.1.1 Column or strut

Wooden column in China is classified into peristyle columns, hypostyle columns and central column, what's more, the strut is another kind of column, is post on the beam rather than touching ground, the structural function is similar with column. The columns depicted in following Figure.

3.1.2 Beam or lintel

Beam and lintel both are the transverse elements, is support on the column or wall along the direction of span. And lintel is inserting into the column rather than posting on it, on which are the struts.

3.1.3 Braced-arch

Wooden braced-arch is an arch of timber, having a truss-like framework maintaining rigidity under a variety of loads, such as Palazzo della ragione, in Padova of Italy (Fig.17).

Figure 17: Indoors view of Palazzo della ragione

3.1.4 Tou-kung

Tou-kung is a distinctive structure in Chinese traditional building, in simple terms is a series of brackets and blocks broken from a whole into hundreds of smaller components and used joints to produce a larger component. This system allow for the distribution of forces and transmission of weight across the joints [5]. It is used to stand on the column and support the roof tier. Due to the different position of Tou-kung, it has Zhu-tou ke (Tou-kung sets on columns), Ping-shen ke (Tou-kung sets between columns), Jiao ke (Tou-kung sets on the Corner) these three classification. Tou-kung is composed of Tou, Ang, Qiao, Sheng, Kung and some other elements (Fig.18).

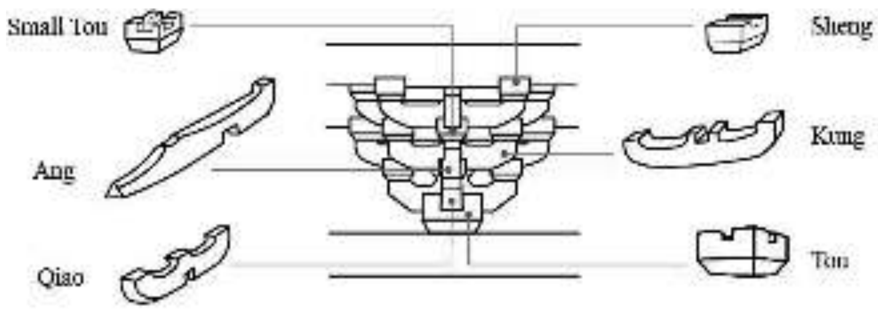

Figure 18: Constitution of Tou-Kung

3.2 Method of elements' connection

In China, the "Sun-mao" elements (tenon and mortise joints) were stared using in Hemudu Site for overhead timber structure six to seven thousand years ago. The main body of timber structure matured and additional forms of tenon and mortise joints were highly developed in Han dynasty. Along with the use of Sun-mao structures in Ming and Qing dynasties, over a thousand architectural forms of concise style present a liberal yet solemn air. The largest timber structure architecture first built in Ming Dynasty in China, the Forbidden City.

The most basic mortise and tenon comprises two components: the mortise hole and the tenon tongue, is a mean of joining two pieces of woodworking at an angle (usually 90°) to each other. There are still many joint variations based on the simplest one (Fig.19).

The tenon and mortise in Italy is different. Rather than using the timber as a unique material, glue is used to secure the pieces together and variations on the joint. The clamping wedge and bracket metal sealing is also common used to lock the joint in place (Fig.20).

It is not easy to check the inner structural connection from the outside of Sun-mao. Furthermore, tenon and mortise variations are highly-diverse from carpenters who hold different inherited techniques, therefore, analysis of traditional timber construction systems in scientific way requires not only the research of specific nodes, but also the acquaintance and comparison of various tenon and mortise craft-working, even the cultural diversity between East Asia and Europe [6].

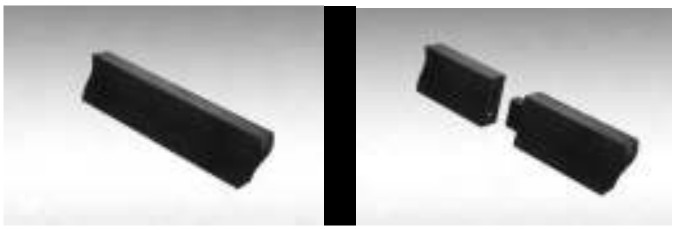

Figure 19: "Sun-mao" (tenon and mortise joint)

Figure 20: Tenon and mortise (Castello Estense, Ferrara, Italy)

4 CONSERVATION APPROACHES AND PRINCEPLES

The part aimed at defining basic and universally applicable principles and practices for the conservation of historic timber structures with due respect to their cultural significance [7]. However, due to the status that there are as great different in timber construction systems as there are species of trees, differences in climate and contrasts of terrain as well as the conservation legislation system between China and Italy.

For example, the repair and maintenance of Chinese ancient buildings must be followed the principle of the availability of historic building of original condition, which refers to the all the historical significance of heritages in individual or complex architectures. No.2.0.2 of Chinese national code [8] read the content should be preserved when conservation and restoration implements:

1. The original architectural form, which includes the layout of plan, facade, construction character and aesthetic context;

2. The original architectural structure;

3. The original architectural material;

4. The original traditional method.

The original condition not only includes the condition prior to any conservation interventions but also the condition after having been subjected to treatments, adaptations, or reconstructions during the course of its history and which interventions are judged to have significance, including a ruined state that reveals important historical attributes. In complex situations, scientific investigation should be undertaken to determine the historic condition. When a historic building preserves fabric or techniques from several periods, the values should be identified and conserved so that all the elements of significance are retained.

In a word, the primary aim of conservation is to maintain the historical authenticity and integrity of the cultural heritage. Traditional and ancient timber constructions refer to all types of building regardless of they are wholly or partially in timber that have cultural significance, for the purpose of conservation of these structures, the principles [7]:

Take into account the great diversity of historic timber structures; take into account the various species and qualities of wood used to build them; recognize the vulnerability of structures wholly or partially in timber due to material decay and degradation in varying environmental and climatic conditions, caused by humidity fluctuations, light, fungal and insect attacks, wear and tear, fire and other disasters; note the Venice Charter, the Burra Charter and related UNESCO and ICOMOS doctrine, and seek to apply these general principles to the protection and preservation of historic timber structures.

Refers to each procedure, are as following,

1. Inspection and diagnosis

The state of historic construction and its surrounding environment ought to be recorded and analyzed carefully before any intervention, aiming to acquire and catalogue the historical information, including traditional carpentry skill or characteristic samples of structures as well as the materials used in previously treatments. In accordance with the Article 16 of Venice Charter [9]and the ICOMOS Principles for the Recording of Monuments Groups of Buildings and Sites [10]. A thorough and accurate diagnosis of the condition and the causes of decay and structural failure of the timber structure should precede any intervention.

2. Monitoring and maintenance

A regular monitoring and maintenance is vital to conservation of historic timber buildings. It can find and manage the problem timely and prompt, for the purpose of earliest and minimum intervention to the historic building or cultural heritage. Skilled technicians and professional device is the important method. Rather than the most advanced device, the optimum and suitable method should be adopted [11].

3. Intervention

Any proposed intervention should follow traditional means and be reversible, if technically possible, or at least not prejudice or impede future preservation work whenever this may become necessary; and not hinder the possibility of later access to evidence incorporated in the structure. And the historic structure should be considered as a whole, including structural members.

4. Repairing and replacement

New members or parts of members should be made of the same species of wood with the same, or, if appropriate, with better, grading as in the members being replaced. Where possible, this should also include similar natural characteristics. The moisture content and other physical characteristics of the replacement timber should be compatible with the existing structure. Craftsmanship and construction technology, including the use of dressing tools or machinery, should, where possible, correspond with those used originally.

5. Contemporary and materials and technologies

Contemporary materials should be chosen and used with the greatest caution, and only in cases where the durability and structural behavior of the materials and construction techniques have been tested in advance and satisfactorily proven over a sufficiently long period of time.

5 CONCLUSIONS

Due to the primary state of the research on comparison of construction systems, the conclusions are still under a qualitative stage. Therefore, the conclusions below are the general comparison in most common situation.

1. Timber structural position, which is the biggest difference between China and Italy, the former one is built in the whole timber structural framework meanwhile it is a load-bearing system itself, conversely, in most situation, the main timber structure only built in roof or floor and as a part of the whole building for the latter one.

2. Main typology of framework based on roof in China, from ancient time to nowadays, however, in any case, there were built in Tai-liang and Chuan-dou. Compared with Italian one, the typology was unidentified yet, the classification cannot be defined precisely. However the most basic and common one is the simple truss composed by king post, inclined struts and tie beam. Others are the transformation based on it. Rather than there are mainly horizontal and vertical elements in Tai-ling and Chuan-dou, the braced arch is common used in Italian building, etc. That makes the span of building more flexible than Chinese one, for example, to reach the same span of building, Tai-liang or Chuan-dou takes more woods than the construction with braced arch.

3. There are other typologies of the construction system in microscopic way. Door & window is another complicated carpentry work in China; The method of caisson is different between China and Italy, which the Chinese one possesses a more complex-structure than Italian one, as well as the aesthetic expression; In most situation, floors and some balconies within the buildings are built in wooden structure in Italy.

4. The tenon-mortise joints between these two countries are totally independent system, refers to the connection way or even structural behavior [6] is different. However the comparison can make the knowledge of timber structure more clear and integrated.

5. Considering the differences background of China and Italy, however an essential requirement in the conservation of a historic building or cultural heritage is to preserve its values, authenticity and integrity as they have evolved during the course of its history. Through

good conservation practice, a site's historic and cultural context and its cultural traditions are preserved and retained for the future.

ACKNOWLEDGEMENTS

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AN EIGHTEENTH CENTURY POSTAL PALACE OF AUGUST III IN KUTNO – ITS ORIGINAL ARCHITECTURE AND HALF-TIMBERED FRAME CONSTRUCTION

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Keywords: Kutno, postal palace, August III, half-timbered frame construction, architectural research

Abstract

In the years 1750-1752 in Kutno there was built a postal palace for August III, who was then the king of Poland and the prince elector of Saxony. The palace was designed as a half-timbered frame construction by Johann Martin Walter, the builder. In the following decades and centuries, it moved from one owner to another and was rebuilt for several times. For several years the palace has belonged to the Regional Museum in Kutno, whose authorities are planning to renovate it and convert into the Museum of Saxony Art.

Due to intended adaptation, in 1993 and 2003 some historical and architectural research was conducted here. Its results, however, contradicted each other with respect to the initial architectural design and the structure of a half-timbered frame construction of the palace. This has led to the resumption of another research in 2015. Its aim was both to analyze the original spatial layout and initial building technique applied here. What was taken into account was the analysis of the supporting walls and trusses, carpentry joints, processing of the timber and the system of carpentry assembly signs. These studies were accompanied by archaeological, dendrochronological and thermoluminescent research, the results of which allow us to conclude that in the middle of the eighteenth century a spatial arrangement was implemented in accordance with Walter's design. Also, the supporting structure, with a few exceptions, corresponds to his plan. The structure of the walls is characterized with a simple truss created by horizontal and vertical elements, the rhythm of the posts, the reduction of the stiffening elements – the passing braces – only to the necessary ones, and the lack of decoration. When it comes to carpentry joints, the most common are mortise and tenon joints or cogged joints. Lap joints and but joints are quite rare. All the elements were traditionally processed with an axe and then with a broadaxe. In order to denote them while placing in the actual construction there were used carpentry signs based on the Roman signs as well as several additional signs to differentiate transverse and longitudinal elements notched angularly or triangularly.

Studies have determined the contradictions between previous architectural studies and enriched the knowledge of the building technique used to construct the palace. They are the basis for planned reconstruction and adaptation. Preliminary comparative analysis has shown that timber frame architecture and construction technique applied here are not distinguished from other implementations and the theoretical knowledge included the eighteenth-century textbooks

1 INTRODUCTION

In the years 1750-1752 in Kutno there was built a postal palace for August III, who was then the king of Poland and the prince elector of Saxony (Fig. 1). It was erected as a half-timbered frame construction, according to the design of the builder Johann Martin Walter and served as a stop station (so called Nacht-Lager) on the way from Dresden to Warsaw and back until the death of August III in 1763. In the following centuries the palace often changed hands and was frequently transformed. Ultimately, it was destroyed by fire in 2003 [1].

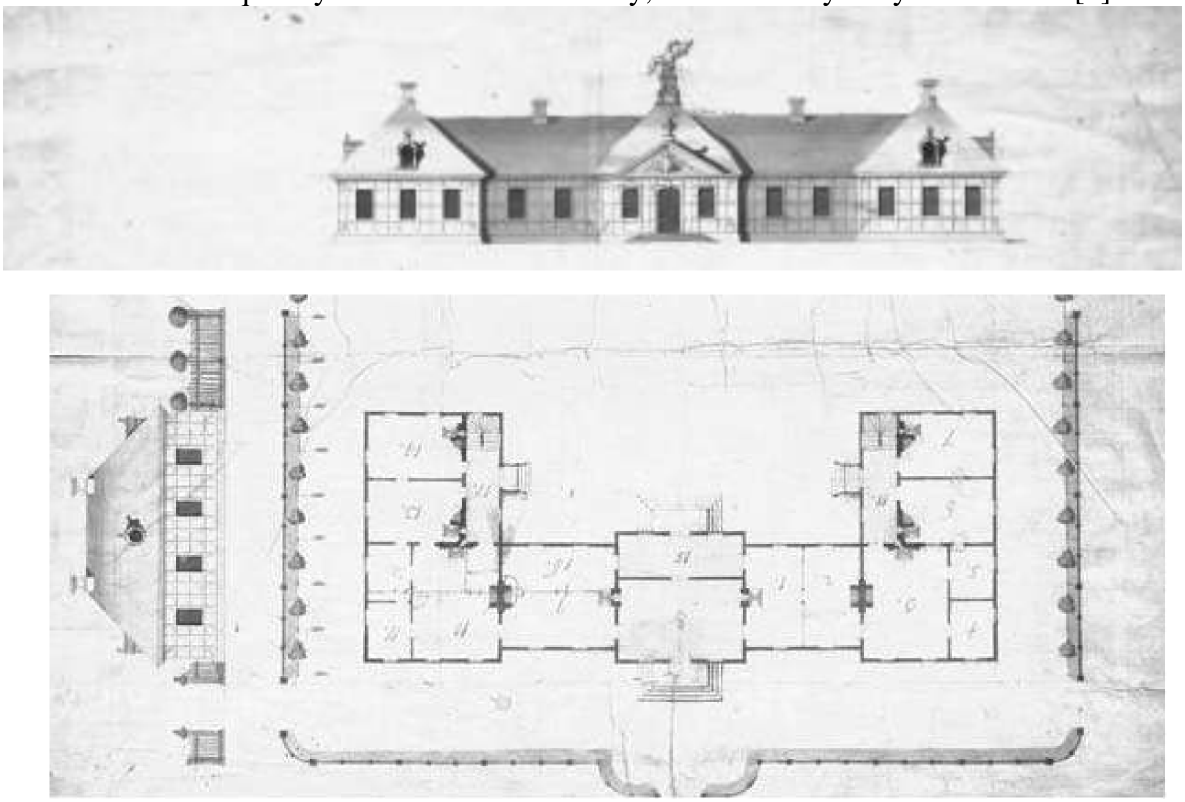

Fig. 1. Kutno, postal palace, design by J. M. Walter, 1750 [after: a copy from the archive of the Regional Museum in Kutno, original: Sächsisches Hauptstaatsarchiv Dresden, Kartensammlung, Baurisse von Polen, 18. Jh., Sig. Schr. VII, F. 91, Nr. 9 r]

The palace, despite the progressive degradation, has become an object of interest for historians and art historians since the second half of the twentieth century [1, footnote 1]. Since the 1990s, it has also been the subject of architectural [2], [3], [4] and archaeological studies, preservation analysis and conservation projects [5]. After the fire in 2003, the above- mentioned studies were updated [6]. In 2014 a separate publication was dedicated to the all current research of the palace [7].

The results of the architectural studies carried out in 1991 and in 2003 are contradictory in many points. These contradictions relate, among other things, to two essential issues in the context of the original architecture and the original structure of the construction: Firstly, whether the spatial arrangement applied in the middle of the eighteenth century was consistent with the view of Walter, as Siuder concludes [3], [4], or was it different from the very beginning as Filipowicz reckons [6], [8]? Secondly, was the middle avant-corps of the façade a half-timbered frame construction as it is Walter's design [3], [4] or was it made of brick as it is now [6], [8]? In addition, not much attention was paid to a building technique applied in the palace or the facts were incorrectly interpreted. For example, the analysis of the carpentry assembly signs of the walls and roof structure was not taken into consideration as a source of knowledge about the authentic substance of the monument. Also, the technique used to construct the trusses was not properly studied. Moreover, it was mistakenly stated that for instance the internal walls had no ties with the external ones [6], [8].

The abovementioned contradictions, deficiencies and errors have become a motivation for the resumption of architectural research in 2015, this time as a part of the student internship at Conservatory Department of Nicolaus Copernicus University under the guidance of the author of this article, supported by the Regional Museum in Kutno. As a part of the internship, the studies had to be limited to the wooden constructions (without brick foundations, materials used to fill the frames of the construction and the fitting elements). The most significant were the following aspects: the arrangement of supporting structure of the walls and truss, the method of connecting individual structural elements, the way of processing the structural elements and the systems of carpentry assembly signs applied in the palace. However, because of restricted access to the half-timbered frame construction, it was impossible to carry out comprehensive analyzes of wooden structures. Therefore, the results are only preliminary [9], [10], [11], [12], [13]. In addition to architectural ones [15], several dendrochronological studies [16], luminescence [17] and stratigraphic studies [18] were done in 2015-2016. Also, the conservation and functional programs of the palace were formulated [19], [20] as the current owner, the Regional Museum in Kutno, intends to restore the building and transform it into the "Museum - Saxon Palace in Kutno" in the next years. The results of all these studies shed some light on the original architecture and wooden construction of the building.

2 CHARACTERISTICS OF THE ORIGINAL ARCHITECTURE AND HALF-TIMBERED FRAME CONSTRUCTION OF THE PALACE IN KUTNO

2.1 The Architecture

Architectural research conducted during the student internship suggests that the wooden construction of the postal palace, with a few exceptions mentioned below, was erected in the mid-eighteenth century according to Walter’s design (Fig. 2). The cheeks in the truss beams in the central part of the main body prove for example that there was once a dividing wall in this place (Fig. 3) [9]. Its location coincides with the location in Walter’s plan (see Fig. 2, D5-D6). And the cheeks in the truss beams in the north part of the west wing show that there existed a narrow corridor, separated by a dividing wall [10, Fig. 1], which is exactly what Walter had planned (Fig. 2, 3E-3H).

In the preserved fragments of the external walls of the west wing it can be observed that not only the main posts have bigger cross-sections than the other ones, but also some additional posts between them. In addition, on the inner side of these posts there are some tenons or mortices as the remains of the rails in the dividing walls [10, Fig. 1]. Their location coincides with Walter's spatial design [see Fig. 2, 2A-2E, G1-G3]. The same discoveries were made in 1991 in the east wing [3, pp. 6-7, Fig. 25, 33, 39]. In turn, the original location of the stairs according to Walter’s design is proved by the trimmer between the tie beams in the north-west corner of the east wing [12, p. 5, Fig. 13]. Implementation of the spatial layout and the stairs leading from the ground floor to the attic in accordance with Walter's plan is also confirmed by the fragments of foundations discovered during the archaeological research [14].

The original existence of a half-timbered frame construction in the middle avant-corps of the façade is also proved by the tenons or mortices and the holes for wooden pegs in the posts in front of the brick avant-corps, which have been partially preserved until today [9, p. 5, Fig. 1] as well as several layers of plaster and paint at the joint of a half-timbered frame construction which has been uncovered during the internship (Fig. 4). The secondary origin of the avant-corps has also been verified by some archaeological [14] and thermoluminescence research, according to which its foundations come from the 1880s / 1890s [17].

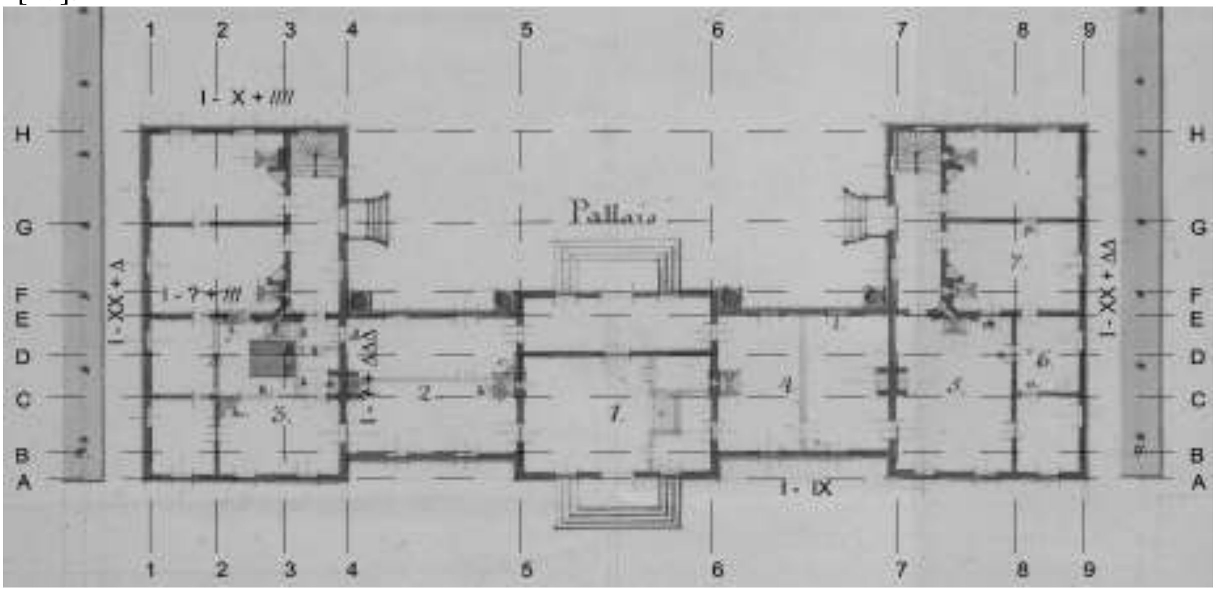

Fig. 2. Kutno, postal palace, the alteration design by J. M. Walter, 1752, a projection [after: a copy from the Regional Museum in Kutno, original: Sächsisches Hauptstaatsarchiv Dresden, 1006 OHMA, Akten, 1564-1920, Sig. I, Nr, 162a, Bl. 126], system of the axis and carpentry assembly signs overlaid by the author of this article

Fig. 3. Kutno, postal palace, middle part of the main body, tie beams with a cogged joint which proves the existence of a dividing wall here
(photo by U. Schaaf)

Fig. 4. Kutno, postal palace, middle part of the main body, south elevation, joint of the half-timbered frame construction and brick avant-corps, a few layers of plaster and paint on the construction prove the secondary origin of the avant corps
(photo by U. Schaaf)

The results of architectural, archaeological, thermoluminescence and dendrochronological studies in recent years have confirmed the conclusions drawn by Siuder's in his early architectural studies from the 1990s. At the same time, the hypothesis saying that Walter’s plan was merely a conceptual one has been rejected.

Therefore, the palace originally consisted of the main body and two side wings, which formed the whole C-shaped complex with an inner courtyard (see Fig. 1). The one-storey block was covered with the hipped roofs over the central part and side wings whereas the connectors were covered with the gable roofs. The facade was emphasized by three avant-corpses, one in its central part and two in the extension of the side wings. Looking from the courtyard, the central part of the elevate on was also accentuated with an avant-corps,

2.2 The Construction

The main supporting structure first of all consists of the external walls in the central part of the main body, the external walls of the side wings and, each time, the outer south and north walls of the connectors between the central part and the side wings (see Fig. 2). Several dividing walls support the tier of beams in the central part and in the side wings.

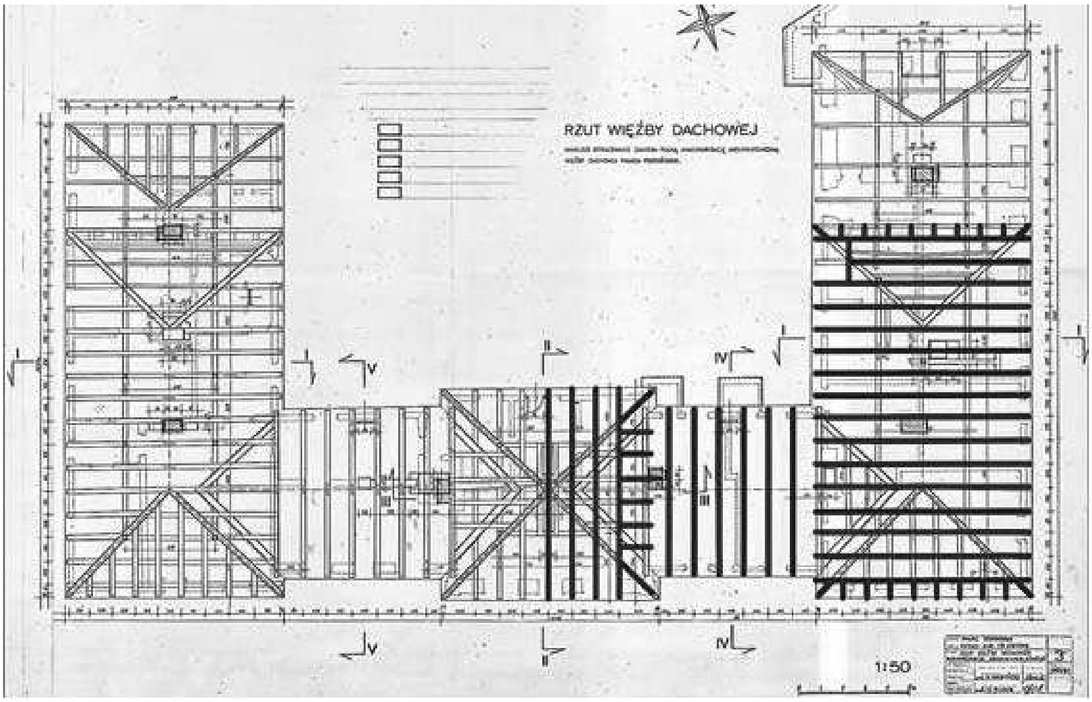

Fig. 5. Kutno, postal palace, plan of the roof truss [after: Inwentaryzacja …, drawing no 3], the tier of beams on the right marked by the author of this article

In the central part having a square plan the tier of beams itself is disposed on the north-south axis with some trimmed joists on the east and west sides (Fig. 5). In the side wings based on a rectangular plan, the tier of beams is arranged in the transverse direction, i.e. east-west, with trimmed joists on the short sides – from north and south. Over the rectangular connectors between the central part and the side wings there is only a transverse (north-south) tier of beams without any trimmed joists. The existence or absence of trimmed joists results directly from the shape of the roofs - hipped roofs or only gable ones. The central part and the side wings are covered with a queen posts and collar beam roof whereas the connectors are covered with a collar beam roof.

The layout of the doors, windows and, consequently, the layout of the structural elements in the external frame walls is symmetrical (Fig. 6). The distance between the posts is clearly rhythmical: wide spaces between the posts in case of the doors and windows, less wide spaces if there are some passing braces, and even smaller spaces in other segments.

The ground plate, three rows of rails and the top plate divide the outer walls of the construction in height into four fields. The rails under and over the windows are at the same height as the lower and upper rows of the rails. The location of the rails over the doors is adjusted to the height of the doors and slightly lower than the upper row of the rails.

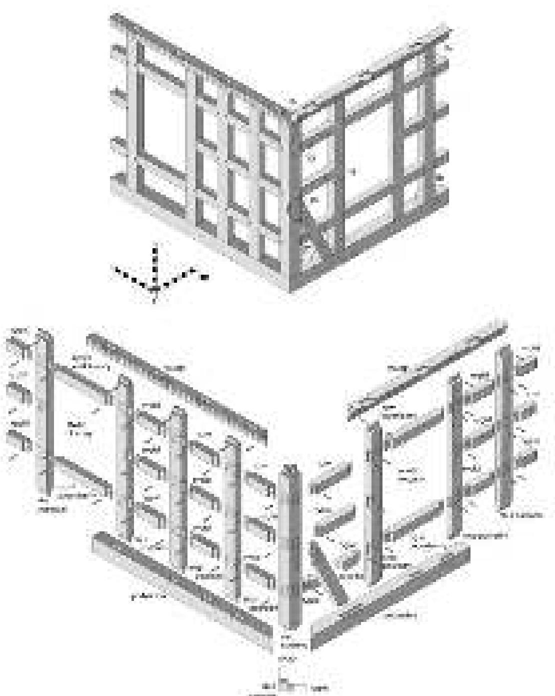

Fig. 6. Kutno, postal palace, south-west corner of the west wing, layout of the constructional elements and carpentry joints (elaborated by U. Schaaf, M. Prarat)

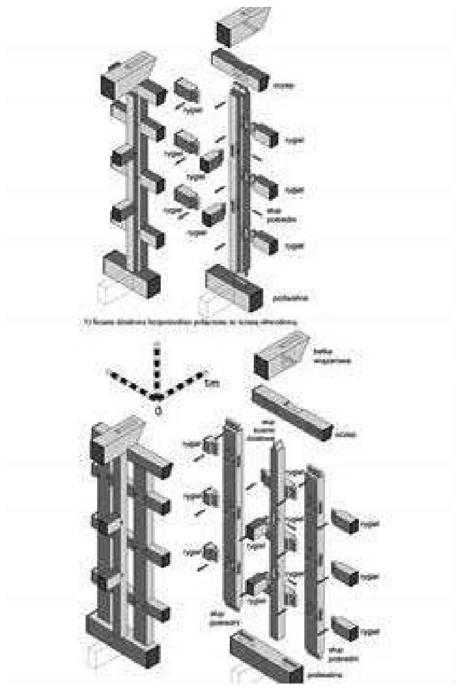

Fig. 7. Kutno, postal palace, layouts of the construction elements and carpentry joints, two ways of joining the dividing walls with the external ones, a) the rails of the dividing wall joint with a tenon to the main post of the external wall, b) dividing wall with a separated post in front of the external wall
(elaborated by U. Schaaf, M. Prarat).

The external walls are stiffened by the passing braces reaching from the ground plate to the middle rail across two fields. On the basis of the architectural studies it may be concluded that they occurred in the extreme sections, between the corner posts and the first intermediate ones. Firstly, in the three avant-corpses of the façade (from the south side) [10, Fig. 2]. Secondly, in the north part in the avant-corps of the main body [9, Fig. 1] and in the gables of the side wings [10, Fig. 1]. Thirdly, in the longitudinal walls of the side wings from the courtyard [11, Fig. 1]. The same stiffening technique was presented by Siuder in his conceptual design from 1991 [4, Fig. 7, 9].

This way of stiffening the frame walls is distinctly different from the design by Walter, which provided the lower and upper braces at the corner posts (see Fig. 1). Replacing short braces with the long passing braces also resulted in giving up the first intermediate post after the corner posts in the stiffened segments.

The preserved remains of the original dividing walls of the palace allow us to conclude that they were either connected directly with the posts of the external walls [10] or terminated with a separate post, attached from the inside to the external walls (Fig. 7) [9], [10]. The spacing between the posts is adapted to the location and size of the doors. Unlike the external walls, the internal frame walls are divided each time by the ground plate, two rows of the rails and the top plate into three fields in height. Yet, it has not been explained how these walls were stiffened with the passing braces.

Walter's design itself does not contain any information about construction techniques apart from the layout of the structural elements shown only in the drawings of the elevations. However, a detailed analysis of the existing material substance allowed to reveal many issues concerning the carpentry joints, timber and its processing as well as the systems of carpentry assembly signs applied in the construction of the palace.

Four **types of carpentry joints** were used in the half-timbered frame construction of the palace: mortise and tenon joints, cogged joints, lap joints and but joints (see Fig. 6, 7). Due to the lack of access it has not been explained yet how the main corner posts were joined to the ground plate. However, it has been recognized that they were joined to the top plate with the one-sidedly embedded and pegged tenon joints, arranged at right angles to each other. Both the main and intermediate posts were joined to the ground plate with a non-pegged tenon and to the top plate and with a pegged tenon.

The connections of particular sections of the ground plate in the external and internal corners, on the joints of the external walls and internal dividing walls as well as within the particular walls have not been recognized yet, either. The rails are connected with a tenon which is always protected with a wooden peg, both with the main and the intermediate posts. Random tests have shown that the mortise passes if there are rails at the same height on both sides of the post. The rails over the doors have an additional cut. The individual sections of the top plate at the external corners are connected to each other with a but joint, at an angle of 45°. The pegged embedded tenons, with which these sections of a top plate are connected to the corner posts, protect them from slipping. Within particular walls, if their lengths exceed the natural length of the timber, the top plates are made out of two parts and joint to each other with a lap joint.

Passing braces stiffening the timbered walls in the corners are connected to the ground plate with a non-pegged tenon at the bottom end, whereas at the top end they are connected with the main posts with a pegged tenon. In both cases the mortises extend the mortises of the intermediate post or the middle rail. Only the bottom rails are connected with the passing braces with a but joint [10], [11].

Both the tie beams and trimmed joists of the roof trusses are connected to the top plate with a single cogged side joint. The same connection was used in case of a diagonal trimmed joist in the external corners, except that the notch and the cheeks are L-shaped, including two sections of the top plate connecting each other at the right angle. The trimmed joists are bunged in the tie beams and the joint itself is protected with a wooden peg. The trimmers in the tier of beams are connected with the tie beams either with a lap joint or a half dovetail lap joint.

The rafters in the common ties and main ties are bungled at their lower ends in the beams, whereas in the ridge they are connected to each other with a pegged bridle joint (Fig. 8). The collars are spread between the rafters and are joined to them by a pegged tenon. Corner and jack and valley rafters are bungled in the tie beams or the trimmed joists and are connected to the rafters or the hip rafters with a but joint.

The crown posts are bungled in the tie beams (with no pegs) and in the collar purlins (with a wooden wheel). The braces that stiffen the timber truss in a longitudinal direction are in tur connected to the posts and the collar purlins with a pegged tenon. The single cogged joint connects the collar purlin to the collar beams.

The **processing method of the wood** used in the construction of the palace is a traditional one. According to some dendrochronological studies the main type of the wood applied here is pine [16]. Mainly the whole trees were used. Individual wooden elements were first processed with an axe and then smoothed with a broadaxe (Fig. 9). In case of the fields of the half-timbered construction, the sides of the posts were slightly carved (Fig. 10), which was to guarantee its better condition. The fillings themselves were made of brick on clay mortar.

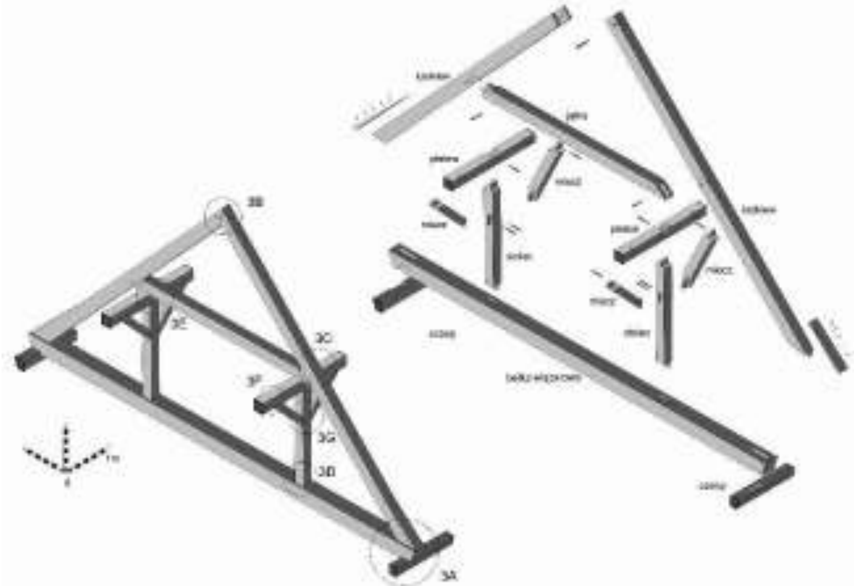

Fig. 8. Kutno, postal palace, roof truss over the east wing, full tie, layout of constructional elements and carpentry assembly signs (U. Schaaf, M. Prarat)

Fig. 9. Kutno, postal palace, central part of the main body, a tie beam with some traces of processing with a broadaxe – long, slightly rounded or angular cuts (photo by U. Schaaf)

Fig. 10. Kutno, postal palace, central part of the main body, a post with its side carved to guarantee the filling that it is better held in the field of the frame (photo by U. Schaaf)

The applied timber has four different cross-sections depending on the function of the elements in the structural system: very thick material (approx. 30 x 25 cm) in the ground plate; thick (approx. 26 x 26 cm) for main posts, tie beams and trimmed joists; medium thick (approx. 25 x 16 cm) for intermediate posts and crown posts; small timber (approx. 21 x 16 cm) for the rails, top plates, passing braces, rafters, purlins, collars and braces.

Limited access to the construction did not allow a complete grasp of the carpentry assembly system. Preliminary recognition shows that the system was essentially based on Roman signs (see Fig. 2). Individual walls were marked from the south to the north or from the north to south and from the west to the east by the Roman signs. In addition, the distinction was made through applying some slanted notches on particular longitudinal walls and the triangular notches on particular transverse walls. The structural elements of the preserved roof truss over the east wing were marked with Roman signs from south to north and from west to east. Here also, a triangular notation was used, amongst others, to subordinate the tie beams to the east wing [12, pp. 11-12, Fig. 1].

3 CONCLUSIONS

Architectural research conducted during the internship, supplemented with several dendrochronological and thermoluminescent studies and finally confronted with the results of previous research of particular authors proved a half-timbered construction of the postal palace of August III according to the design of Walter. There were only some minor inaccuracies in terms of stiffening the walls.

The analysis of the building techniques made it possible to complement the previous knowledge about the carpentry joints, processing of the timber and the system of carpentry assembly signs applied in the construction of the palace in the mid-eighteenth century. Not only was that knowledge useful at the chronological stratification, but it was also crucial due to planned conservation works [19], [20]. In order to obtain a comprehensive view of the half- timbered architecture it would be advisable to continue the research while the conservation is carrying out.

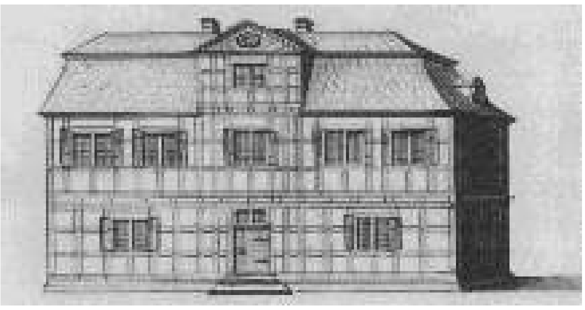

Fig. 11. Toruń, residential pavillion in the suburban garden of Albert Borowski, mid-eighteenth century [after: 21, Fig. 63].

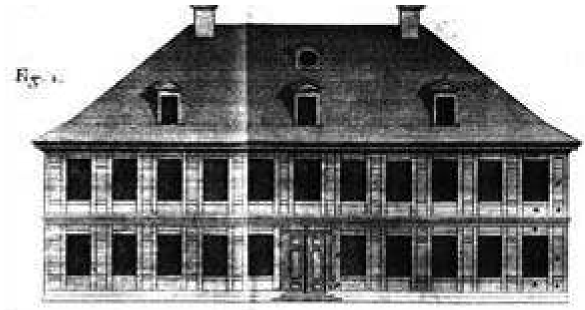

Fig. 12. A house, elevation [after: 22, Tab. 1, Fig. 1-2.]

However, with what has been reckoned until now, it can be concluded that the half-timbered architecture and the building technique applied in the palace were not distinctive in any way from the general trends popular in the mid-eighteenth century. Such conclusion suggests that there should be done some preliminary comparisons of the palace and other buildings as well as the content of the eighteenth-century textbooks for the carpenters and woodworkers. The rule of symmetry and rhythmization of the constructional elements in the elevation are for instance typical of a summer residential pavillion in the suburban garden of Albert Borowski in Toruń (Fig. 11) [21] or a house designed by Johann Friedrich Penther (Fig. 12) [22], both from the mid-eighteenth century. The construction of a hipped roof is elaborated in details by Johann Jacob Schübler in a coursebook from 1731 [23]. In 1736 the same author showed the ways of connecting the elements of the roof trusses (Fig. 14) [24], which are similar to those applied in Kutno. In turn, Jost Heimbergr describes and shows the construction of a pavillion roof with the centrings imposed on the rafters [25], whose shape resembles the one over the central part of the main body designed by Walter. Considering the palace solely in the context of a half-timbered construction it is hard to conclude it is a kind of a unique building. More likely, it represents a particular tendency. However, taking into account its original function or, most significantly, the historical context, partial re-enactment of its initial half-timbered architecture seems to be entirely justified.

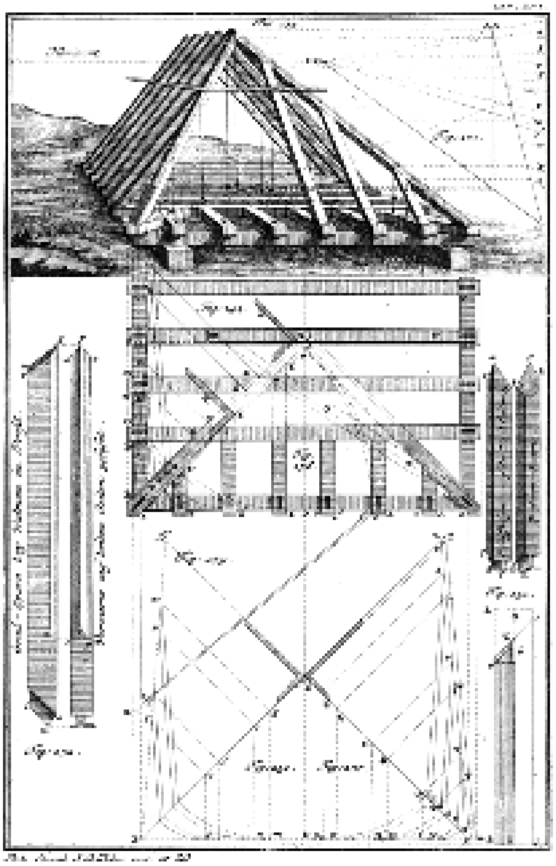

Fig. 13. A hipped roof [after: 23, Tab. XVI]

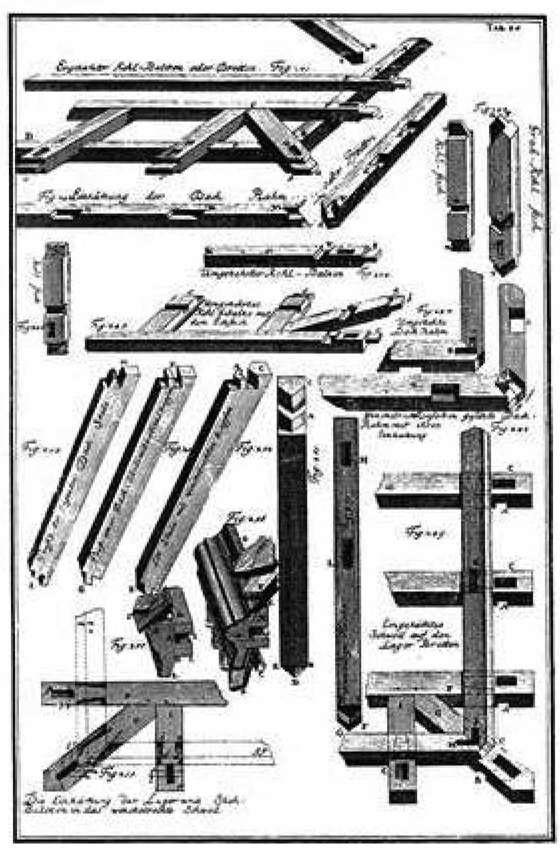

Fig. 14. Different carpentry joints to combine the elements of the roof construction [after: 24, Tab. 20]

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EVALUATION OF THE CONSTRUCTIVE MATERIAL FEATURES OF A MEDIEVAL TIMBER FRAMING IN ARQUATA SCRIVIA "GOTHIC HOUSE"

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Keywords: Timber framed building, constructive connections, on site investigation, dating

Abstract

The "Gothic House" of Arquata Scrivia is located in "via Interiore", that is what remains of the "decumano" of the old town centre of Arquata, in the North-West of Italy in Piedmont Region. This house represents what remain of a more significant presence of these buildings, today more than ever a rare case of timber framed constructions with a stone and brick infill for the area of Southern Piedmont.

The conservation of this almost unique example through a careful restoration, left us a testimony of the craftsman's culture and an unusual construction's technique for this area that has not been followed later on, replaced with most common techniques such as loadbearing structures in masonry with wooden horizontal elements and wooden roofs. This represents also a rare case in which knowledge has followed and not preceded the restoration. In recent years the building is a matter of studies at the University of Genoa and a research carried out on a structural model of the "gothic house" has been presented at the recent International Conference H.Ea.R.T.H. 2015 in Lisbon.

For this work researchers from other Universities has been involved. The present study pays attention to the constructive technologies, the definition of the constructive elements, the characteristics of the materials, bear in mind the restoration intervention, with a detailed study of the connections between the different elements and their condition through a cognitive survey developed mainly with non-destructive tests,

1 AN INTRODUCTION TO THE “GOTHIC HOUSE” OF ARQUATA

Arquata Scrivia is located on the Ligurian-Piedmontese Apennines, where the torrent Scrivia makes some large meanders in the flat territory.

The present hamlet of Arquata is the result of the refounding of the small old town as “new village” on the will of Tortona between 1244 and 1286. This was the beginning of medieval centre of Arquata, mainly represented by the “via Interiore” usually consider the original core of this town. In this peculiar street are still visible some traces of the old medieval walled village.

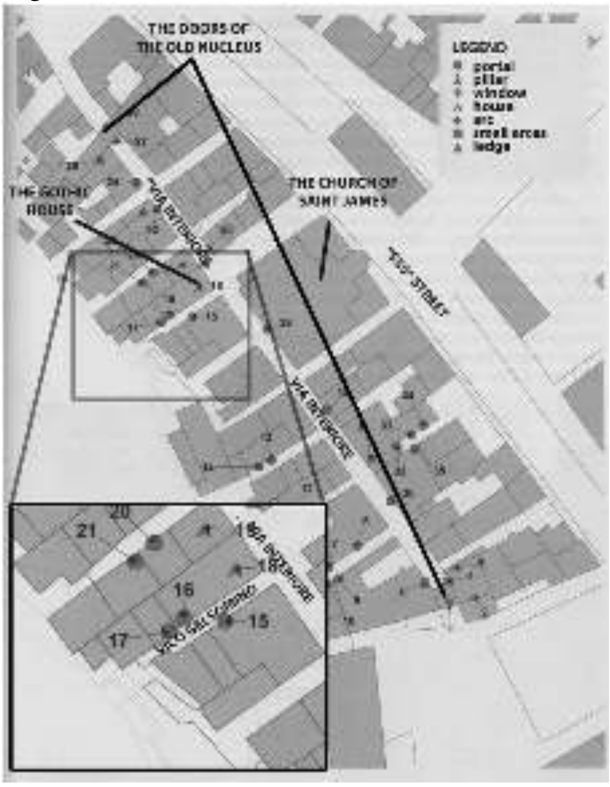

Figure 1: General plan of via Interiore with the indication of the architectonic elements RISALENTI AL medieval period (between XIII and XV sec)[1].

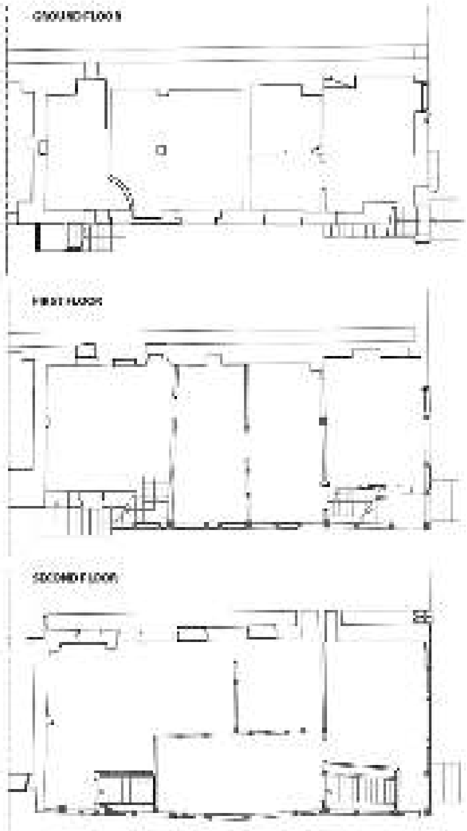

Figure 2: Plan view of the three flors.

The original organization of the old centre remains mainly the same, with a main road crossed perpendicularly with narrow secondary street forming blocks [1]. In every block there were two parallel row of buildings divided just with a narrow space called “ambitus”. For almost every building on “via Interiore”, the facade on the main street was a gabled facade, due of the primary importance of this front, and so the water rain was collected on one side in the “ambitus” and on the other in the alley.

The Gothic House of Arquata Scrivia is situated at the 31th of “via Interiore”, the heart of the city centre. On the extremities of “via Interiore” there are the two doors of the old nucleus and the number 31 occurs in front of a small square near the church of Saint James, on the opposite side of the street (Figure 1).

During the drafting of a work for a master degree at the faculty of Building Engineer and Architecture, every possible source was investigated, both direct and indirect.

1 The image is developted from a figure from the volume [**1**].

The aim of the work was the realization of a model to evaluate the anti-seismic characteristics to confront them with the behavior of others typology of timber framed construction across Europe. The opportunity to investigate deeply and directly the structure allowed to carry on a study on the sensitivity of the model to the different detected materials. The results of this study were presented at the recent International Conference H.E.A.R.T.H. 2015 hosted in Lisbon.

The documentation consulted indicates that the building dates back to XV-XVII century [**1**], period confirmed by the dating carried out on several elements of the structure that will be explained during this paper.

The building is a timber framed construction with a masonry ground floor and two upper floors with timber framed walls (Figure 2). The North-west wall is built in masonry in every level probably to contain the flue pipe, but maybe also to take advantage of a better way to use the space allocated to each building at the establishment of the new hamlet for the will of Tortona. That explains also the ledge on the lateral street, that permits to enlarge the upper floors.

Figure 3: View of the gothic house from the S35 road; the tower of the castle of Arquata is visible on the background.

Figure 4: The gothic house facade vico Gelsomino. The ledges of the upper floors are visible.

Admittedly, on the facade on vico Gelsomino every floor protrude besides the underlying just the width of a beam, in particular the beam that represent the lower border of the framed panel. In the rear portion of the building there are also some ledges deeper due to the stairs sticking out (Figure 4).

In 1999 and 2005 the building was involved in two conservative interventions, developed by the Technical Study Fera who also followed the execution of the intervention.

As it will be explained later, all the works carried out aimed to preserve the structure, maintaining its legibility. This aspect, with the rich and detailed project's and building site's documentation, lead to a deep knowledge of the structure, its materials and details, as of the operations realized.

The importance of this building is due to it's distinctiveness in this region but also for its behavior proved during the years.

2 A BRIEF NOTE ON THE RESTORATION OF THE "GOTHIC HOUSE"

The restoration of the so-called Gothic House located in the Via Interiore of the village Arquata Scrivia takes a long diatribe at the conclusion that animated movements of citizens debates on what fate should have the building. At the time the building was not under the constraint of the Architectural heritage, and being part of private property for the current legislation could be demolished to give rise to a same cubic capacity building but of an entirely different consistency, which fortunately did not happen. The same fate has not befallen other buildings that existed in the Via Interiore, evidence of a constructive knowledge of wood, which were then demolished in recent times with a great loss to the entire scientific community and of everybody else. There are very few still existing in Italy examples of this building tradition. As far as we know in the Genoese territory are no longer existing row houses with these construction characters and as is known Arquata Scrivia, although today is located in Piedmont, is historically in the Genoese cultural interest area. The architectural characteristics of the old village center reflect the Genoese influence and are therefore a clear example of this belonging to the Republic of Genoa.

Figure 5: The situation of the roof after the removal. of the tiles.

Figure 6: The situation of the roof after the restoration and before the application of the tiles.

The row house in question has suffered many years of neglect, the will of a few for its demolition was followed by a more or less conscious abandonment perhaps in the hope that degradation could operate what some hoped, that its collapse. Instead, a series of intelligent municipalities have worked so that the building could be restored for the enjoyment of all, because they understand the importance of the existing building in their village. The building was in fact abandoned for many years and the cladding tiles, laid on a sparse plank in chestnut with a thickness of about 2 centimeters, not constantly maintained formed in a short time deficiencies leaving water the ability to penetrate inside. During the winter months the country is subject to sometimes heavy snowfall followed by rains that make particularly heavy snowpack. With the frosts are formed of icy snow cover plates, sliding toward the slope of the roof. For several years, certainly more than a dozen, many parts of the horizontal and vertical structures have been exposed to the elements, creating different situations of deteriorations both of the woods as well for the masonry. Despite these widespread situation of deterioration the building has remained miraculously intact almost in its entirety. The particular brick mixed structure, stone and wood has been able to adapt to various failures without ever collapsing completely. The construction has been shown to have an extraordinary adaptability to the geometric changes due to localized structural failure like if it was formed by deformable plastic material. The restoration project was preceded by an accurate architectural and structural survey in order to clarify the different situations.

Figure 7: The facade on "vico Gelsomino" on the left and on "via Interiore" on the right.

The building before the work was with a portion of the roof smashed due to the collapse of a truss occurred for the rotting of the support of the chain and partially of the strut in correspondence with the masonry on the side of the "tregenda" (gap between the next row house). This situation had leave open to the water for several years a large portion of the building below, creating further deterioration of the wooden structural material of the horizontal structures. The different seepage occurring in most parts of the roof had caused widespread degradation throughout the entire building.

In the intervention of restoration it has, however, tried to preserve as much as possible all existing artifact existing on site both they were made of wood or brick and stone.

The load-bearing wooden structures, where necessary have been consolidated with the insertion of rods made of fiberglass or stainless steel. The heads of the beams have been consolidated with the prosthetic reconstructions in resin and steel bars. Similarly in some particular danger situations were made seams of the walls with metal bars. The floors were consolidated with the laying of phenolic plywood made integral to the joists, previously consolidated, with connectors. The sandstone staircase, because of an old modification had been charged by an infill masonry, had a longitudinal tear of most of the steps. In this has intervened going to insert stainless steel bars in the length of the tread in order to allow to reconstruct the original lift.

The woods at the end of work, have been treated with natural oils made with bergamot essence for their protection. The plasters, realized as a result of the analyzes carried out, were redone with the use of natural limes.

3 CONSTRUCTIVE FEATURES AND A QUALITATIVE ANALYSIS OF THE SEISMIC RESPONSE

Timber, generally speaking, is a material with an elasto-fragile behavior. However, it could experience plastic deformations, dependent by the stress typology and application with respect the grain orientation, reaching significant seismic energy value dissipation. Moreover, the joints of ancient timber structure represent a further contribution in the dynamic actions response [7] which, by means of frictions and with the aid of metallic elements, permit high value displacements to which correspond limited ruptures and deformations. [8]

The Medieval building in Arquata Scriva (Italy), with three floors, has an in-plan regular development, characterized by the main facade of about 7 meters and the two lateral fronts approximately length 15 meters. A shape close to a rectangle, not particularly elongated[2] hence avoiding torsions triggering in case of seismic induced actions. In elevation the regularity characterizes the masses as well as the resistant elements distribution.

The vertical structure is composed by masonry with timber frames both relatively the external and the partition walls, excluding the wall localized at north (fronting the so-called trexenda) devoid of wooden elements. The latter wall is characterized by a certain stones organization variability, in general of chaotic type, with components of different sizes. Instead, the frame masonry infill is arranged with regular courses of bricks laid on the long narrow side and mortar bed about two centimetres thick.

The timber frames, with a not particular complex configuration, present a dimensional hierarchy in which the base horizontal members have a dimension approximately of 15x15cm, whilst the posts and the top horizontal elements is about 10x10cm.

The corner post is characterized by a resistant section nearly double if it is compared with the other vertical members constituting the framing. The infill frame, with a width constant dimension, includes the entire inter-floor height. The stiffness under in plane frame actions is given prevalently by the masonry infill. The framed wall lies on the floor through the main beams. In fact, the latter projects outward for about 30 cm respect with the lower floor, on which, orthogonally disposed, a wooden element rests: the base of the frame.

All the junctions are carpentry joints, for example the node post to base member is classifiable as mortise and tenon, a geometry typical of the roman opus craticium. At the opposed side the vertical element is bonded to the horizontal member by means of a half lap joints stiffened by a pyramidal metallic nail. A shape that is not completely effective in case of repeated oscillations due to the earthquake. Anyway, an effective ductility can be recognized for the described nodes, with the possibility of developing perpendicular to the grain compressions and consequently, topical plastic deformations.

Furthermore, it is worth highlighting that the typical craftsmanship of the carpentry joints, so with inaccuracies and the no perfect co-planarity among the faces of the concurrent members, could generate frictions entailing high value of seismic energy dissipation.[8]

2

The building is in plan regular, according to Eurocode 8, if the slenderness $\lambda = Lmax/Lmin$ is not higher than 4, where Lmax and Lmin are respectively the larger and smaller in plan dimension of the building, measured in orthogonal directions. Eurocode 8, Seismic Design of Buildings.

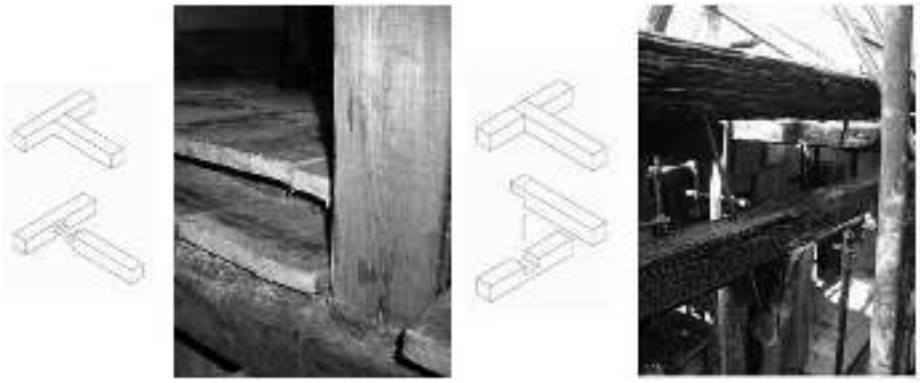

Figure 8: An example of mortise-tenon joint, an example of half lap joint and a pictures of the structure's details.

Looking to the timber floors, they are constituted by a main order with beams (about 27x27cm) on which is constrained a joists row of 15x15 cm dimensions. The beams are arranged with a limited inter-axial distance that ensures a contribution to the floor stiffness, useful in the seismic actions distribution to the shear walls. At this purpose a second boarding is present that, although is arranged parallel[3] to that constrained to the beam, cooperates in the floor deformability reduction. The floor members rest on a perimetral beam that beside distributes the concentrated load on a largest surface, reduces the wall overturning tendency. Even if such a ring beam is not totally effective in the restraining action given the lack of bonding elements to the masonry panel. Vaults and iron beams characterize the ground floor, may be as a result of recent intervention.

The roof carpentry is constituted by king post trusses. The latter, in addition to entailing a limited mass at the building top, do not transfer horizontal actions onto the supporting walls. Such trusses, complete in all its functional elements, present the strut constrained to the tie-beam through a notch, a precise will of reducing the truss in-plane deformability. The timber tie, crossing the external wall, by means of a wooden peg is linked to the summit of the panel, constituting an effective constrain in counteracting wall rotation due to possible seismic action. Moreover, a metallic tie-rod, crossing transversally the building and providing a further contribution in the bond among the walls, is present.

The timber frames has limited seismic mass with respect to a entire masonry building, with a discrete deformability that ensures, by means mainly of frictions generated among the infill components and at the interface timber-masonry, besides the joints contribution, high value of dissipated energy. Recent experimental studies [8] have proved that cyclic horizontal loads, applied to masonry specimen reinforced with timber frames entail ruptures and deformations, under moderate displacements, of the masonry infill, leaving the timber structure almost undamaged and in elastic field.

3 The boarding would present a module of elasticity value even of 20 times higher, dependant of the wooden species, if placed with the grain orthogonal to the below boarding.

4 KNOWLEDGE SURVEYS

The applied methodology was based on Non Destructive Test because the case study is Monumental restricted.
The masonry, the wooden elements of slabs and of the timber frame combined with masonry was analyzed with accuracy.
The target of the analysis was to define the conservation, as a diagnosis, referred to a degradation even if the building was recently restored.
We used different instrument to increment the knowledge of the building: Thermographic-survey allow to locate the hidden relief as tie-rods or beams (Figure 9); endoscopy to define the masonry and slabs stratigraphy (Figure 10); the combination sonic-sclerometric test help to verify the strenght value.

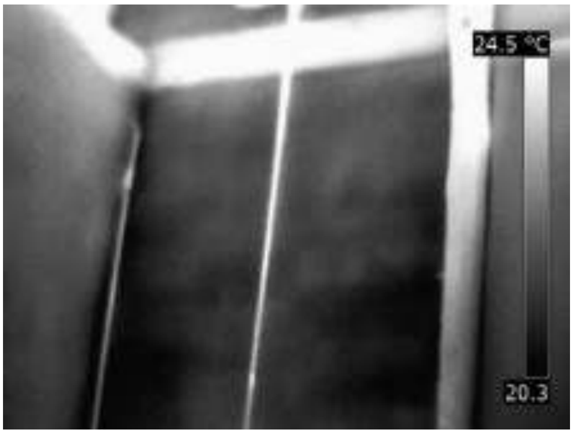

Figure 9: Thermographic analysis

Figure 10: Endoscopyc survey

First step is the analysis of wooden elements was based on first sight survey of lack and alteration applying the Technological Survey Protocol combined with an experimental sheet. Second step is verifying the moisture percentage with electrical-resistivity; last survey is recognize the wooden species using macro or microscopic method in lab.
The sclerometer Pylodin examin the density of small section whereas for bigger beams is better to use an electronic hammer that create a dendrogram. (Figure 11).
This instrument help us to locate some metallic reinforcements in the beams. All this surveys help us to reach an high knowledge level about the building, the materials and the whole structure; each detail was analyzed mechanically due to determine a seismic behavioral model. Most of all the Non Distructive Test is the best way to assess a building because of its mobile ad light-weight instruments. However N.D.T. needs qualified technicians to locate application points and an high experience in situ. Basilar is the developing of the survey project that help the planning of the survey and the correct use of the instruments.

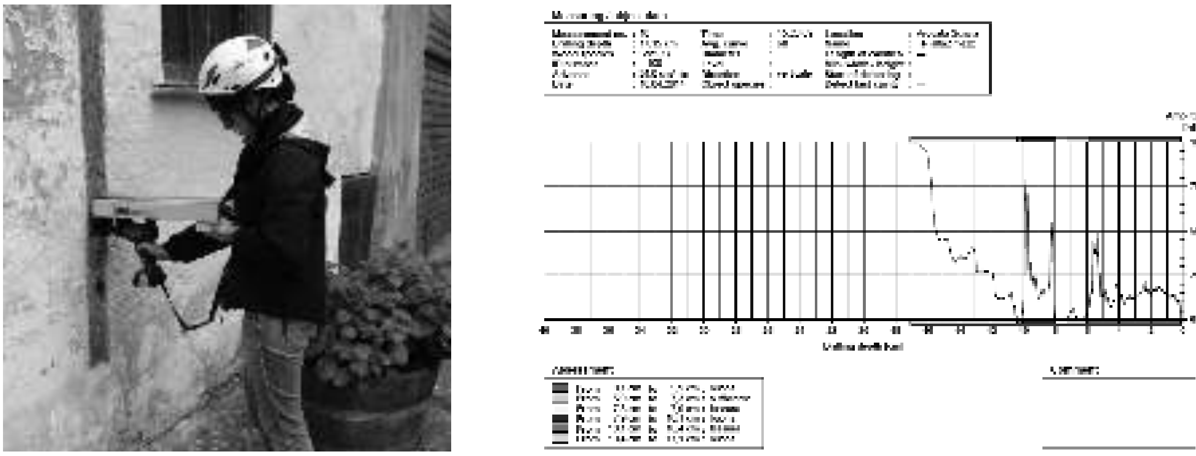

Figure 11: Resi 400 Electronic Hammer and the dendrogram obtained

5 CONCLUSIONS

At this moment in history, while many may assume cultural heritage is a resource able to unite populations and move the economy forward, restoration seems to be in a period of crisis, being unable to provide satisfactory design solutions. The content and methods of restoration as a discipline are being questioned, and an open and continuous dialogue with the various branches within the discipline could prove useful, as was seen in a recent debate during the First National Conference of Italian Society for Architecture Preservation [**1**].

Italian restorers are discussing one's ability to recognise the values of cultural heritage and the necessity to open up to the world – including the cognitive as well as the physical world - by understanding and studying the different kinds of relationships with pre-existing artefacts. Cultural heritage can be evaluated from many points of view: form and design, materials and substance, use and function, traditions and techniques, location and setting, but also spirit and feeling, as stressed by the Nara Document of Authenticity in 1994. It becomes obvious that the concept of authenticity differs in cultures around the world, as explained in the aforesaid document: "Conservation of cultural heritage in all its forms and historical periods is rooted in the values attributed to the heritage. Our ability to understand these values depends, in part, on the degree to which information sources about these values may be understood as credible or truthful. Knowledge and understanding of these sources of information, in relation to original and subsequent characteristics of the cultural heritage, and their meaning, is a requisite basis for assessing all aspects of authenticity".

Restoration, as a product of Western thought, is intrinsically connected to the physicality of artefacts, placing the preservation of the material object at the centre of the project. Restoration in Italy is undoubtedly a motivated and responsible cultural act, based on the rigorous study of an existing artefact in its physicality, which is also historically stratified. The study is not just a preliminary act, but it is a fundamental stage of the project. It is a part of the entire process of governing the existing artefact (project, construction site and subsequent management of the artefact).

The importance of conservation of the material object is an unavoidable premise in preserving and passing it down to future generations together with the immaterial values it carries or conveys. Moreover, preserving matter means it will be possible to study it in the years to follow, as cultural artefacts are an inexhaustible source of knowledge.

The example of the "Gothic house" in Arquata Scrivia is emblematic of how the choice of preservation guarantees the fruition of an artefact as well the possibility to study it in further depth even years after the preservation intervention. Both the restorations of the Gothic house in 1999 and in 2005, for which the decision was made to preserve the structure and the possibility to interpret it, allowed us to study the building in the years to follow, applying new methods of analysis and therefore finding new interesting data.

During the new study in 2014, it was possible to apply two methods of absolute dating to different sections of the house, that, at the time of the conservation project, were not considered or were not feasible: dendrochronology and mensiochronology of the bricks. This was important to chronologically collocate the building. The name “Gothic house”, with which the building is known in the area, associates it with ancient medieval structures, even if there is no documented evidence regarding its origin. Alfredo D’Andrade, painter, archaeologist and restorer from the 19th century, believed that the house of Arquata should be examined in further detail and he listed it among the significant examples of Italian medieval architecture [2]. However, in his writings he did not provide useful elements to date it.

The possibility to use methods of absolute dating in the restored sections of the building (walls and slabs) allowed us to place the edification of the “Gothic house” in a precisely defined period (Figure 8). The analyses showed that the structure of the “Gothic house” was realised during the late Middle Ages, when the town of Arquata [**1**] was quickly developing also thanks to its location along a road of great merchant trading.

Thanks to the method of dendrochronology (dating executed by Dr. Severino Fossati), a series of main beams in chestnut wood belonging to the first and second slab were dated as well as some rafters. The dendrochronological curve used for the comparison was that of the chestnut tree from the region of Canton Ticino [3], currently the most complete and the closest to the Ligurian curve which can be considered accurate only from 1600 onwards [5]. The wood analysed provided homogeneous dating between the end of 100 and the start of 1600.

With the mensiochronology method of the bricks (dating executed by architect Rita Vecchiattini), some significant wall portions were dated where bricks had been left in view and they had also not been altered in their visibility with plastering of joints and flowing beds of mortar. The mensiochronological curve used for the comparison was that of Genoa [6] because, even if today the town of Arquata is located in the province of Alessandria in the region of Piedmont, Arquata actually belonged to the Republic of Genoa from the Middle Ages until the Unification of Italy. The bearing walls were dated between the fifteenth and the seventeenth century, which is coherent with the wooden beams. While the two internal partition walls date back to the 1800 and therefore are probably the result of subsequent modifications.

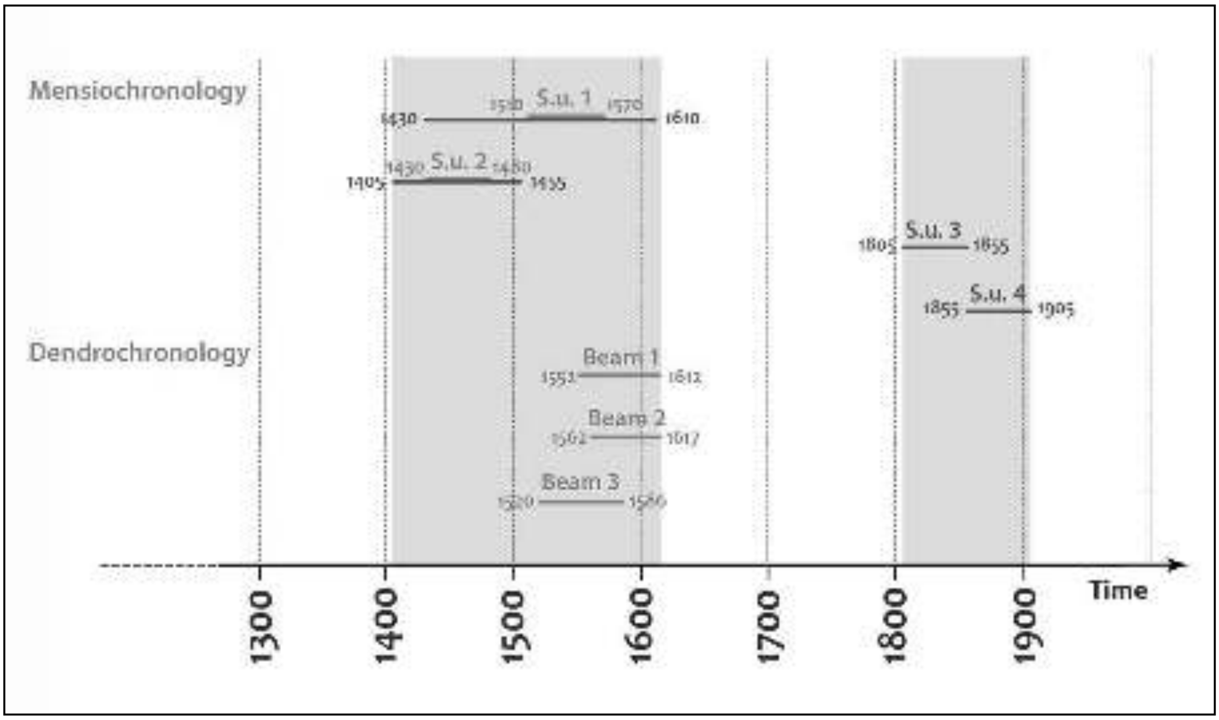

Figure 11: Results of the various datings performed on brick walls and beams in chestnut wood.

This would not have been possible had the conservation architect not studied the building and taken the responsibility to declare that the structure, in its historic context, could be preserved. However, preserving a building's structure is not enough, in order to continue to appreciate and study it, it is also necessary that it is not hidden from sight, for example compartmented or covered with intumescent products.

The issue of safety in the last few years has become extremely important, thus highlighting the often contradictory relationship between safety and conservation instances. The equilibrium between the two depends upon the architect and the project supervisor taking on responsibilities, but it can also be imposed if we genuinely want to pass on to future generations what we inherited.

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CHAPTER IV

EXPERIMENTAL METHODS AND TESTS

SHATIS'17
4th International Conference on
Structural Health Assessment of Timber Structures
20-22 September 2017 Istanbul, Turkey

ASSESSMENT OF TIMBER STRUCTURES. LOOKING AT THE PAST TO PLAN THE FUTURE

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Keywords: Timber structures, information, assessment, research

Abstract.

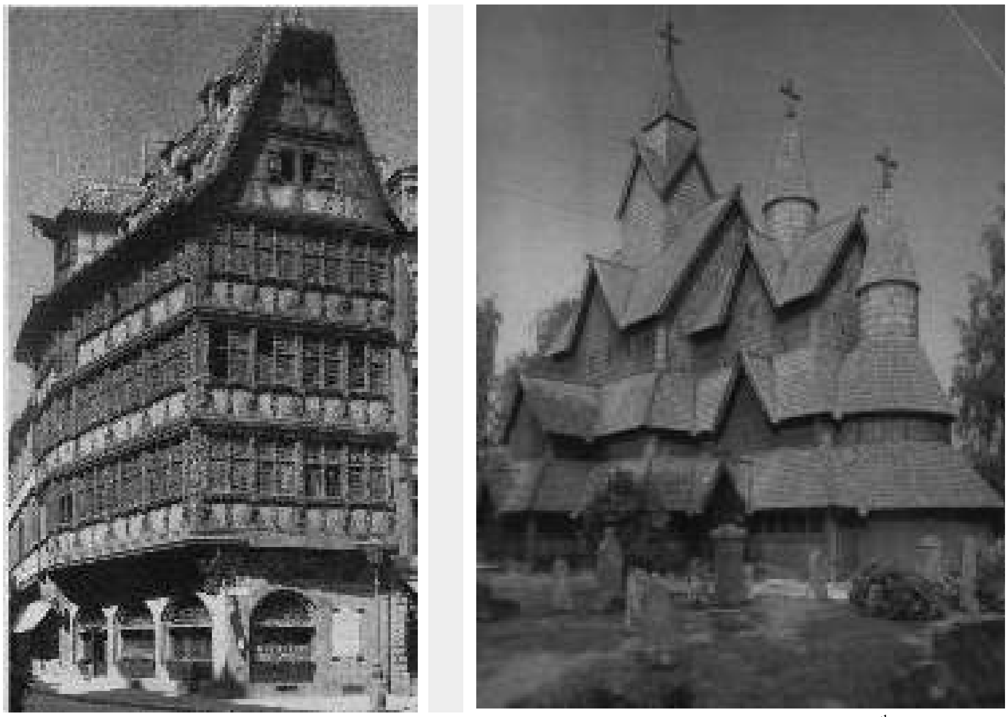
The capacity of using any material for construction is dependent upon the ability not only to understand its behaviour when applied at the construction site but also to assess its performance onsite in different stages of its working life. This capacity enables a reasonable, safe and economical maintenance of existing timber structures within an acceptable level of uncertainty.

Onsite assessment even though based on existing documental information, often needs also the use of research outputs (visual grading rules, non-destructive testing methods, models) that should be disseminate properly by the stake-holders involved in the rehabilitation of existing constructions. Also a strong research effort should be addressed to those areas that can answer the frequent questions that still came out during the rehabilitation of a timber structure.

These steps are im-portant to overcome the fact that although timber is one of the first materials to be used by man for construction and its mechanical behaviour studied since the seventieth century the capacity of pre-dicting its properties onsite is perceived still as limited, the impact of some factors not still very acquired and the apprehension of the information by construction technical staff limited.

1 INTRODUCTION

Timber structures clearly reach the safety conditions provided by concrete or steel structures when proper timber selection and construction design, execution and maintenance operations are ensured. This is confirmed by the many cases of old existing timber structures still in service all around the world, figure 1.



Maison Kammerzell, Strasbourg, 1589 Stave church at Heddal, Telemark, mid-13th century

Figure 1: Old existing timber structures

Old methods based on trial and error (some empirical rules are included in ancient carpenter's manuals) and following methods (based on visual grading from the beginning of the twentieth century) are reliable in the sense that old timber structures are mostly over designed and show a high level of robustness. New challenges are brought by modern timber constructions based on more optimise timber grading methods (e.g. machine strength grading from the middle of XX century) and optimized design rules which raises a new discussion about robustness. This last topic was within the scope of COST E55 [1] and is being handling during the ongoing discussion for the new generation of Eurocodes.

Recent studies showed that poor design or execution is the main causes for catastrophic failures observed in timber structures. Also decay problems, or termite attack in some geographic locations, are the cause of failures, but these are mainly local affecting frequently a few members or connections. These type of failures are dealt without problems by old timber structures since they are able to provide alternative load paths and redistributed stresses by the remained members and thus avoiding the collapse of the structure.

Indications of structural malfunctioning (e.g. deformations or biodeterioration) imply the existence of tools to assess the timber performance onsite at different stages of its working life. As for other types of structures, also in timber structures different factors can be involved and only a proper assessment can support the efficient measures (repair or strengthening) to restore its safety and serviceability, figure 2.

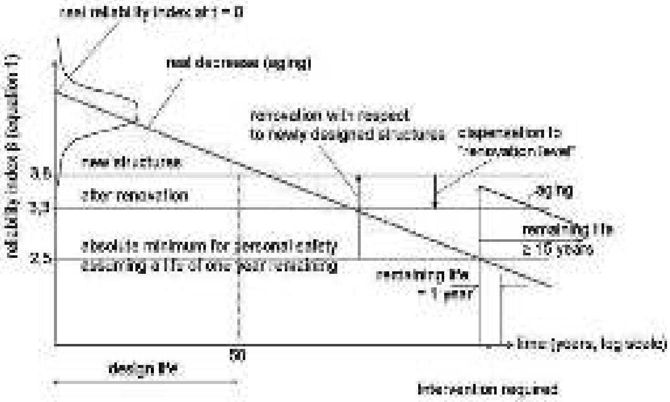


Figure 2. Different criterion to the assessment of structures according to NEN 8700 (Jorissen 2012).

At this stage assessment plays a crucial rule and basic knowledge about timber structures required. With few exceptions the quality of this process is threatened by the fact that most engineers are more aware of the new technological construction methods or materials and thus neglect the structural capacity of “old” materials and “vernacular” constructions. This is stressed by the fact that building codes are mainly focus on new construction, the lack of documents describing original design rules makes the capacity to assign design values to members restricted to experts. Also there is still a need for more research regarding connections since the capacity of predicting the resistance capacity of wood connections is scarce. All these raises difficulties to ensure a reliable structural analysis of existing timber structures. Finally, sometimes to understand defects and abnormalities between expected and real performance, more visible when numerical modelling is included, it is crucial the ability to “spend time” in rescuing and analysing the information available and establish a timeline of events that could have affected the building while in service and possibly explain these abnormalities.

The new concerns about sustainability and building heritage makes conservation, repair and strengthening of existing structures a large share of the building activity today and probably in the medium-term. This reality can lead to a more open mind able to integrate in rehabilitation works experts in the type of construction and materials and in the case of timber to overcome the vision of timber as a non-durable or complex material. Unfortunately, this same process can be pernicious leading to traditional “concrete” construction companies to maintain the facades, financially support the documentation of the building history, and then a new concrete interior is built following modern design codes. This problem of communication is even enlarged when many structural engineers have insufficient or any basic understanding of timber as a construction material. To overcome this problem timber as other types of traditional construction materials needs to communicate efficiently to technical staff involved in the field of rehabilitation. Useful information should be turn available in an efficient way (guidelines, rules, standards) and gaps of information addressed by researchers.

The present paper is driven by two questions regarding assessment of existing timber structures: *What do we know?* and; *What do we need to know?* In the opinion of the author a large amount of research has yet to be collated, analysed and synthesised to assist construction works and to guide future research projects. Lack of proper channels to deliver available information to the market is a gross obstacle making the fact that timber as one of first materials to be used by Man for construction and its mechanical behaviour studied since the seventieth century still the rehabilitation of timber structures is in most countries a fight against demolish and rebuild current trend.

2 RESEARCH – KNOWLEDGE AND GAPS

The increasing acknowledgement of the necessity to preserve and protect our building cultural heritage sites and the necessity of renovation of many European city centres lead to the relevance to discuss various topics related to timber structures, including new and existing structures, and justified the working plan of many recent committees, table 1.

Table 1: Recent committees and major outputs for assessment.

COMMITTEE	OUTPUT
COST IE0601	Guidelines for assessment of historic timber structures [2]
COST E55	Report on Assessment of Timber Structures [1]
	Assessment of Failures and Malfunctions: Guidelines for Quality Control [3]
	Report on Design for Robustness of Timber Structures [4]
	Modelling the Performance of Timber Structures- Special Issue of Engineering Structures journal http://www.sciencedirect.com/science/journal/01410296/33/11?sdc=1
COST F1101	State of the Art Report on the combined use of NDT/SDT for the assessment of structural timber members on site [5]
	Assessment of structural timber members by non- and semi-destructive methods – Special issue of Construction and Building Materials journal http://www.sciencedirect.com/science/journal/09500618/101/part/P2?sdc=1
RILEM TC 215-AST	Report on In situ Assessment of Structural Timber [6]

These committees working plan include(d) not only aspects of survey, allocation of timber member's properties and behaviour of timber connections but also reinforcement and durability issues (including mechanical and biological deterioration processes). Also as consequence of the guidelines produced by COST IE0601 [2] a standardization work started inside the Technical Committee 346 (Working Group 10) of the European Committee for Standardization. As a result of the work done so far a draft European Standard is now at Public Inquiry (prEN 17121:2017). The activity of all these committees are also closely linked with different events (conferences) that took place between 2005 and 2017, figure 3. To these events it should be added more broad events as the International Conference on Structural Analysis of Historical Constructions that is now in its 11th edition.

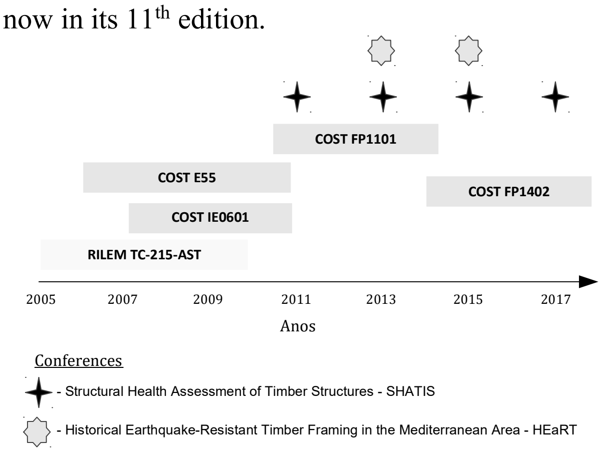


Figure 3: Activity of different committees and events that took place between 2005 and 2017

Assessment of timber structures involves the consideration of multiple aspects including: 1) the gathering of information to understand the structural system– documental, expeditious survey and analysis of wood samples at the laboratory; 2) prediction of structural behaviour of members and connections, and; 3) the need to take into account time-variant effects. Other factors as the required level of structural performance and the type of structural analysis models to apply need also to be taken into account, namely when a more detailed structural analysis is required or the possibility of limiting imposed loads or limiting public access is an option, figure 4.

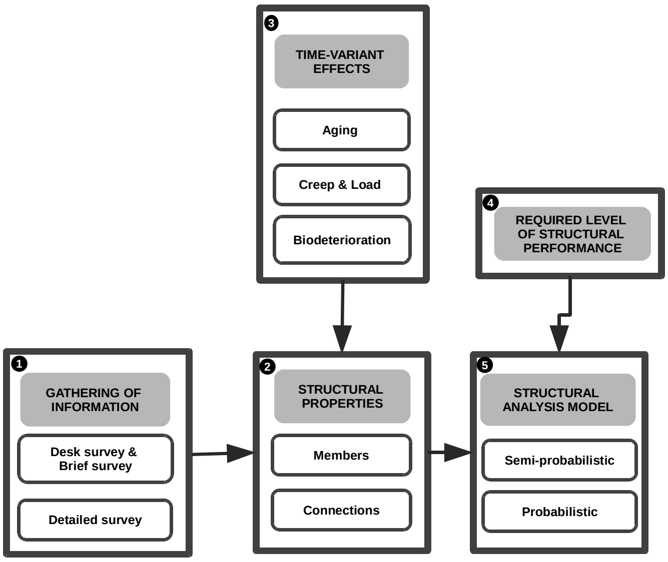


Figure 4: Figure example

Along the following sections an attempt of sum up and discuss the information collected so far is made.

2.1 Gathering of information

The potential cost of an in-depth structural assessment of an existing building leads that a multilevel approach is recommended by the majority of current guidelines [1, 7, 8]. Thus a first step (named preliminary) includes collecting all the information related to the structure: historical information about the structure and their uses; design information; drawings. This stage is complemented by a visual inspection that allows the recognition of the structural system and preliminary assess of the structural condition (quality of the construction and execution and conservation condition).

The usefulness of a sort of template that inspectors could use to guide their work and promote a better communication between architects, wood experts, biologists and design engineers is recognized by everyone. As an example, in the field of design of new timber structures the arrangement of wood species and strength grades into what is called strength classes (EN 338) turned easier the work of design engineers and promoted the construction of timber structures (e.g. buildings and bridges).

A first daft of a template to guide during a preliminary survey was attempted by COST FP1101 and aimed to assist inspectors to collect information about the construction, structural systems and damage types [9]. This work was mostly based on the previous experience gathered from the assessment of traditional timber structures in Italy. Unfortunately the template is not yet public.

The majority of inspectors recognizes that the amount of initial information and their quality is extremely important to define an efficient assessment procedure (e.g. using NDT and SDT methods) and, consequently, to the quality and reliability of the results obtained. Therefore although all phases of assessment are equally important the first always will have an significant impact on the others. The lack of proper information affects the results of structural analyses and can lead to the adoption of erroneous conservation, repair or strengthening solutions or even modification of the structure, figure 5.

Figure 5: Voided slabs used to replace a roof timber structure in a classified historic building

2.2 Structural properties

Existing guidelines provide general information on procedures to be followed to allocate structural properties to timber members and in a less extended way to timber connections. Normally this information, as for other materials, dependents upon the information collected at the preliminary assessment, testing on site or at the laboratory from samples collected on site.

Design information used when the structure was built is always the preferable starting point. But often, unfortunately still in recent works, that information was lost during the construction process. However some information always exist and that is the wood species information. So, the first principle is to know what kind of material was applied. Timber as concrete or steel is a generic brand that includes materials as light as 350kg/m3 to up to 1080kg/m3 and a characteristic bending strength as 14 up to 70MPa. Thus guidelines [2, 7, 8, 10] mention the need to identify the wood species used in construction, being especially important in the case of historic construction in order to maintain the same species when replacing parts of a structure [11]. The identification of wood species process has some limitations related to the difficulties in provide ascertain answers at the species level [12], for instance the differentiation of different *Pinus* species is difficult and often impossible. However, these limitations can be overcome for existing structures if documentation, namely from historical sources or oral sources, is available.

Once identified the wood species then visual strength grading can be used, either based on allowable stresses or on characteristic values. The amount of uncertainty that the application of a certain grading process not knowing exactly which wood were used, i.e. knowing only that belongs to Genus *Pinus*, is not known and is more relevant in case of deterministic or semi-probabilistic analyses. This issue and the question of the applicability of recent visual grading standards to old timber should be considered by the wood expert at the time of a final decision upon timber's member quality.

However a very cautious approach taken during this first preliminary assessment can result on the demolition of the entire structure even if that structure performed satisfactorily for decades or centuries. The need to update our initial information and be able to refine the first mechanical properties allocated to the different timber's members have accelerated the level of research in the field of non or semi-destructive testing methods (NDT/SDT). And thus a variety of methods were developed for assessing the mechanical performance of timber members [1, 6, 13], figure 6.

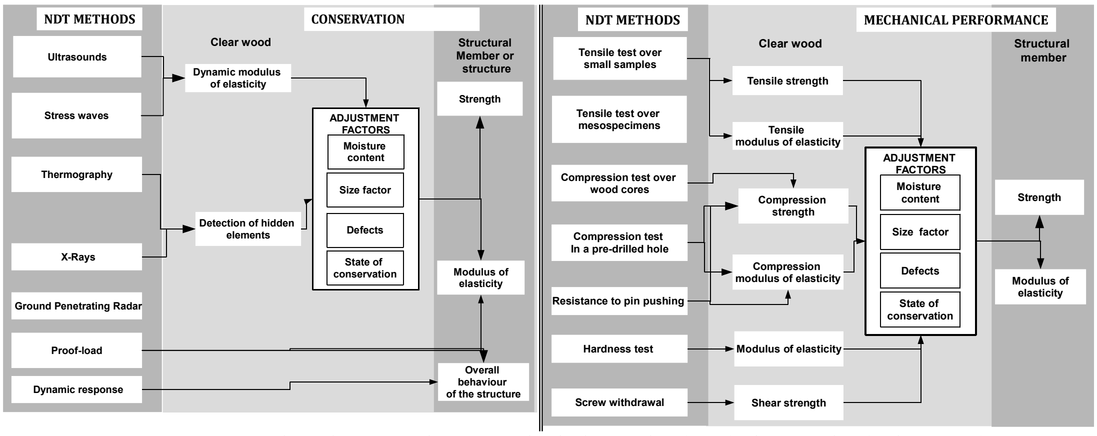


Figure 6: Examples of NDT and SDT methods for evaluation timber mechanical properties

But also NDT and SDT techniques were developed for assessing density and other factors affecting the mechanical performance of timber members, as its conservation condition (e.g. drilling resistance, R-rays, Ground Penetrating Radar).

But the question is how these methods should be applied. The existing guidelines merely mention the possibility of using them for updating visual strength grading information but not explaining how. RILEM Technical Committee 215-AST published a series of three papers which can be considered as a first step in that direction, explaining not only the principles, equipments and limitations of different non-destructive techniques but also some possible procedures to be follow to obtain information from timber members [14-16].

COST FP1101 worked in the next issue – *What to do with all results obtained from NDT/SDT values*? The application of multivariate statistical models, Bayesian models and other type of models allows to use uncertainty as part of the prediction process [17-20].

Also since most of NDT and SDT methods are based on correlation between an indicative property (i.e. time-of-flight) and a property of wood (i.e. modulus of elasticity) adjustment factors related to environmental issues (affecting wood's moisture content), volume effects and method procedure were considered [21].

Wood properties are studied already since the end of the ninetieth century. Furthermore, during the twentieth century a large amount of work was produced regarding the correlation between wood properties and also the effect of wood's features has on strength and stiffness. An attempt to validate these regression models should be done on species or group of species basis. As an example, many regression models were produced between bending strength and modulus of elasticity for the same species and it should be asked why past information (existing regression curves) are seldom used to validate a new curve. Moreover it should be important that models could be presented in a way that allows other researchers to use them (reporting at least the regression equation, sample size, r-square and mean-squared error).

The issue of combining new information with those obtained from more traditional methods (e.g. visual appraisal) is important, namely the weight (confidence) that new NDT/SDT methods results should have over the weight of basic information. Another crucial aspect deals with the fact that information obtained from NDT/SDT methods is often related to clear wood properties. The need to include information about defects is evident since a good approximation to the real resistance of timber members is required for a reliable but optimize structural analysis, figure 7.

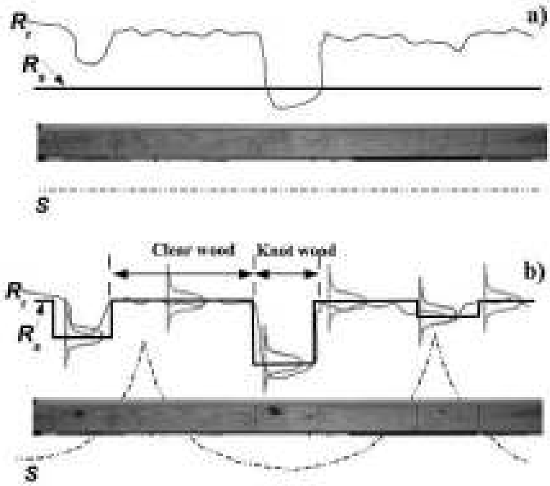


Figure 7. Changed of the view of a timber member from: a) homogeneous to b) heterogeneous. R_r – Real resistance profile; R_S – Simulated resistance profile; S – Load profile

2.3 Time-variant effects

Time-variant effects are here understood as the irreversible modification of wood's physical, chemical and mechanical properties while in-service. These included aging related to the level of stress in timber members (load duration and creep) and accidental effects as biological or physical deterioration. A thoroughly examination of the effects of aging in wood can be found in [22].

These time-variant effects result in a shift to the left of the probability density function of resistance and increasing the probability of failure (and consequent decreasing of the reliability index β), figure 2. Different factors affect the decreasing performance of timber members being more relevant the magnitude and type of stress acting upon the member and the moisture content variation of that member. These are function of load-history and moisture-history which is not known exactly and difficult to predict. Most studies show that there is no significant difference between the mechanical properties of old and new timber elements [23]. This finding can questioned the sense to apply to old timber structures with more than one century back and showing no significant damage factors as k_{mod} or k_{def} from design rules applied to new structures.

In most situations the level of stress on timber members is well below 40% of the short-term ultimate load when it seems that only above 53% the load-duration effect takes place [22]. The maximum stress level from the preliminary structural analysis can provide relevant information about the possibility of overstressed (above 35%) members and where mechanical aging damage can have occurred. This or other methods should be the target of research to seek ways to detect with more confidence the presence of irreversible mechanical damage (e.g. compression failures).

Other factor is fissures. Fissures are often the cause of many concerns about the safety of existing timber structures. Their strength-reduction effect is today based on a visual appraisal of its location and characteristics (length and depth). Areas subject to significant perpendicular to the grain or shear stress are critical and should be carefully checked. There is a lack of knowledge about the way to verify these fissures, namely 1) how affected is the integrity of the member and connection by the presence of cracks; 2) is that crack stable or is prone of evolution with time (non-stationary process).

Also biodeterioration is a challenge during inspection and more important when dealing with historic structures where the principle of maintain as much of possible the original material is established. The combination of NDT/SDT (e.g. drill resistance, X-rays) and stochastic decay models can be a solution to get information about the short and long term safety and serviceability of a timber structure [24]. However such a models needs to be complemented by the calibration of different parameters before being able to be considered reliable.

A consistent solution is also central if the repair/strengthening works can not be done immediately and it is necessary to define a safe and reasonable timeframe for these interventions.

3 KNOWLEDGE TRANSFER

The advance of science and technology and the natural development of our societies lead to a shift from an empirical knowledge learning process to a sophisticated scientific knowledge learning process supported in research about new materials, constructions systems and design rules. This shift implies the necessity to a continuous and quick transfer of knowledge from research into the construction sector.

Special courses like Advanced Masters in Structural Analysis of Monuments and Historical Constructions (http://www.msc-sahc.org/index.asp) are essential tools for disseminating new knowledge but also some of the trial and error knowledge gathered in centuries and that can be use to understand the behaviour of old timber structures.

4 FINAL REMARKS

Designers will be more and more challenged to assess the safety and serviceability of existing constructions. One of the ways to protect timber structures is to deliver guidelines or codes in a format similar to those accepted for other materials being the Swiss code SIA269/5 [10] an example.

However assessment should not be only based in general information that few experts can use but also to include practical procedures that can be followed by common engineers involved in assessment works. For that it is necessary to collect, identify gaps of information and knowledge, and, finally, harmonize all this information to support the rehabilitation of existing timber constructions or structures. Without reliable and accepted assessment procedures the survival of historic timber structures will be based on modifications and replacements that rapidly or one step at the time will replace the entire timber structures by a new one.

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YENİKAPI BYZANTINE SHIPWRECKS, ISTANBUL-TURKEY

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Keywords: Theodosian harbour, Yenikapı shipwrecks, Constantinople, waterlogged wood conservation

Abstract

Salvage excavations conducted in the Yenikapı district of the Istanbul 2004–2013 have brought to light the Byzantine Era Theodosian Harbour on the Sea of Marmara. In addition to thousands of archaeological artefacts, a total of 37 shipwrecks dating from the 5th to the 11th centuries AD were uncovered, constituting the biggest collection of medieval ships uncovered at a single excavation site.

The present article deals with 31 wrecks' conservation procedure in the charge of the team from Istanbul University's Department of Conservation of Marine Archaeological Objects. Maximum water content measurements, ESEM, XRF, XRD and ICP-MS analyses were used to determine the amount of damage to the pieces, and the conditions that had caused this damage. Moreover, determining the genus and type of the ship timbers plays a key role in the identification of the geographical locations where the ships were built, of the types of trees and methods used in different sections of the ship, and the various repairs that the ships had undergone. Analyses of nearly 3.000 samples belonging to 26 shipwrecks have been completed as part of the project. Following this phase, chemical and mechanical methods were used to remove iron composites from the wood.

Providing mechanical strength to the wood cells is possible by substituting the water they contain with certain chemical substances. For their conservation, PEG (polyethylene glycol) and melamine-formaldehyde resin are being employed. Melamine formaldehyde preferred for elements of highly deteriorated non-durable materials such as plane wood. Wrecks in a relatively better condition, and those likely to be displayed in the future, were first soaked in a % 45solution of PEG 2000 and then freeze dried. Istanbul University has established two laboratories for the conservation and restoration of the waterlogged timbers from the ships. The new Department of Conservation of Marine Archaeological Objects has enabled the launch of the first academically organized training in this field in Turkey.

1 INTRODUCTION

In 2004 Istanbul Archaeological Museums initiated salvage excavations at the Yenikapı stations site of the Marmaray and Metro Projects, which unites Asia and Europe at Istanbul by rail for the first time in history via a crossing under the Bosporus. The excavations lasted for nine years without interruption and were completed in 2013. Construction of the main hub station at Yenikapı led to the biggest ever archaeological harbour excavation.

Salvage excavations brought to light Constantinople's on of the main harbour the Theodosian Harbour. The excavations were conducted in an area of 58,000 sqm by a team of about 50 archaeologists and 600–1000 workmen. The excavations were carried out in a deposit 12 m deep, representing a time span ranging from the late Ottoman period back to the Neolithic period (Fig. 1).

Figure 1: Aerial view showing west part of Yenikapı excavation area

Excavations in the deep deposit brought to light hundreds of thousands of waterlogged organic and inorganic items and architectural remains [1, 2, 3]. The remains of the 37 ships uncovered constitute the largest assemblage of ship-finds dating to the Byzantine period and are of great importance for studies of boat and ship archaeology. Istanbul Archaeological Museums (IAM) delegated the scientific work on the wrecks to Istanbul University Department of Conservation of Marine Archaeological Objects (IU, 27 wrecks) and the Institute of Nautical Archaeology (INA, 8 wrecks). The present author undertook the direction of the project on behalf of IU, together with other IU academicians, full-time experts, IU undergraduate and graduate students: a team which has been working on the documentation, lifting, and conservation of the wrecks since 2005 [4, 5, 6].

2 FIELD AND POST EXCAVATION DOCUMENTATION

Timbers were labelled sequentially starting from the keel. In situ field documentation was made using geodesic equipment (a Total Station) within a reference system. Points were measured on each wreck and combined digitally to provide 3D images. They were combined in AutoCAD with the help of sketches drawn while the wrecks were being measured in situ. In addition, the interior surface of the planking was drawn in detail on clear acetate at full scale. Every timber of the wrecks was photographed both overall and in detail. In addition, each wreck documented as photo-mosaic (Fig. 2).

Figure 2: On site documentation

Cataloguing the timbers continues at the lab. Timbers, whose joining details were not visible during in situ recording, were documented in detail at the lab after disassembling. Each timber was photographed in detail with wide-angle and macro lenses (Fig. 3).

Figure 3: Detail photographing ship timbers for catalogue

The most important stage of lab work has been the digitization of each piece using a FaroArm coordinate-measuring machine (CMM), which was used for the first time in archaeology in Turkey by our team [7] (Fig. 4).

Figure 4: Full scale drawing of ship timbers by 3D digitizer

3 ANALYSIS

3.1 Timber Identification

Identification of timbers used in the Yenikapı wrecks is of utmost importance for the identification of the regions where individual vessels were built, what kinds of timber were chosen for specific construction elements, and the repairs they underwent. These analyses are being conducted by Prof. Dr. Ünal Akkemik, IU Faculty of Forestry Department of Forest who has already identified 2800 samples from 27 wrecks [8].

3.2 Chemical Analysis

During periods when the wrecks were under water or covered with sediments, chemicals accumulated in their fabric. Sulphur, in particular, from the harbour floor that is stored in the timber has a direct effect on the future conservation process and long-term stability of the wood. Therefore, the sulphur content of timber samples taken from each wreck is being analysed within the framework by Dr.Gökçe Kılıç [9].

3.3 Dating

Out of 37 wrecks uncovered at Yenikapı only four still had their cargoes in them. Therefore, it was not possible to employ the comparative or typological dating methods widely used on archaeological wreck sites as, for most, only the timbers survived. It was, therefore, decided to take a minimum of three samples from each wreck for radiocarbon analysis. The average of three dates obtained from keel, frames, and planking is used to date each wreck. These analyses are being carried out by the Oxford Radiocarbon Accelerator Unit (ORAU) at Oxford University [10].

3.4 Determining the Level of Deterioration

Waterlogged timbers were recovered at various levels of deterioration, which is graded by the amount of water the timber has absorbed. The level of deterioration has to be determined individually for each object since it guides the quantity of chemicals used for conservation. The analyses of samples taken from hull timbers of Yenikapı wrecks indicate that the maximum moisture content (Umax) ratios of the wood samples fall in the range % 280–900. Based on this, classification of the woods from Yenikapı Shipwrecks was considered as Class I - II and conservation procedure was carried out based on this parameter. Fourier Transform Infrared Spectroscopy (FTIR) is used to determine the chemical deterioration of Yenikapı Shipwreks. According to this analysis, chemical structures of cellulose, hemicellulose and lignin altered. These analyses have been carried out by Dr. Namık Kılıç of IU Department of Conservation of Marine Archaeological Objects [11].

4 CONSERVATION OF WATERLOGGED TIMBERS

First stage was preventive conservation. As the timbers were waterlogged, conservation started, as a matter of course, before recording on site commenced. The wrecks were protected under temporary tents to shield them from sunlight and other external elements, within which recording and lifting were conducted. In order to prevent the timbers from shrinking and cracking the tent interiors were sprayed with atomized water to maintain 100% relative humidity.

4.1 Lifting the Wreck Timbers

Following the completion of on-site recording, the timbers were lifted using a procedure developed on site. It was decided to disassemble wreck timbers to reveal the joining details, to identify building techniques and construction details, and to facilitate absorption of the necessary chemicals during the conservation process. Dissassemblage started usually with the ceiling and stringers, as well as any other internal timbers, then progressed with frames, and concluded with planking. Iron nails and wooden fasteners holding each member in place were removed with methods chosen to cause the least damage to the surrounding timber; this often meant cutting the nails with chisels or otherwise breaking them (Fig. 5, 6).

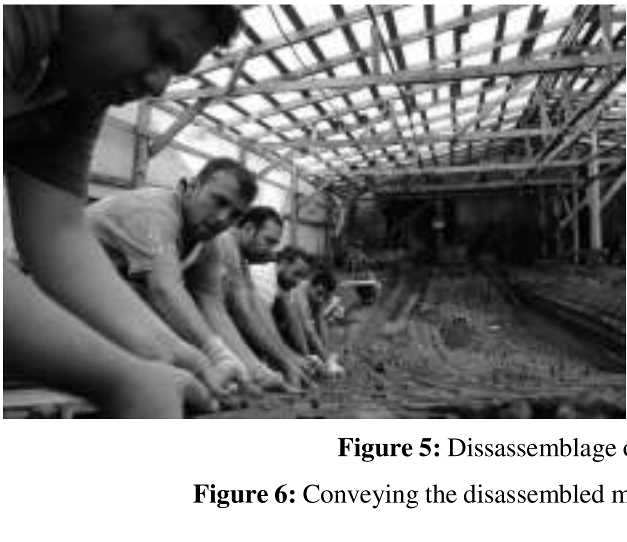
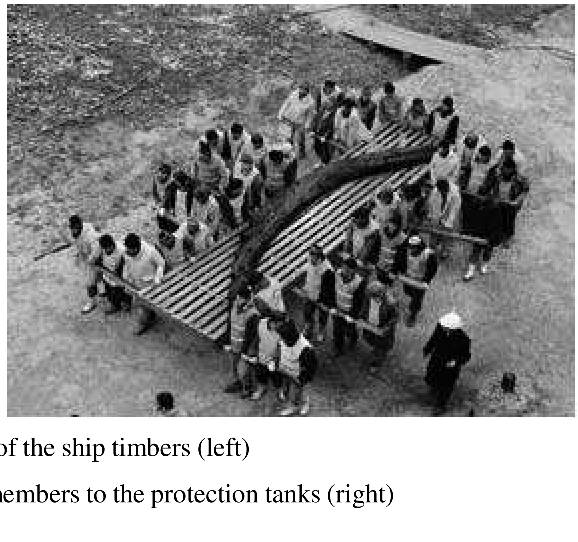

Figure 5: Dissassemblage of the ship timbers (left)

Figure 6: Conveying the disassembled members to the protection tanks (right)

Planking was removed using convenient methods, such as L-shaped timber carriers or negative timber moulds, while extremely frail timber elements were lifted using an epoxy mould method. Some planking of YK 6 was block lifted. All of these methods had one common goal: to preserve the original curvature and twist of the timbers. Bespoke timber cradles not only prevent deformation during the conservation process but also proved useful for transportation, and during documentation of the original shape. Disassembled and supported ship members were placed in wooden chests, custom-built to their dimensions, taken into the IU Yenikapı Shipwrecks Research Centre and stored in tanks. The tanks, built of concrete with stainless-steel liners, have an average size of 4 x 10 x 1.2 m and were roofed to prevent the unwanted effects of direct sunlight [12].

4.2 Desalinization and Cleaning

In order to desalinate the timbers once they were placed in the storage tanks, the water in the tanks is circulated and renewed with fresh water. The incoming water line is at the bottom of the tank while the outlet is at the top; thus contamination from bacteria, fungi and algae caused by still water is prevented. The initial salinity of the timbers was found to be lower than expected, probably as a result of fresh water brought into the harbour by the Lycus. The salinity levels are dropped to 280 ppm using fresh water in the tanks, but then further lowered to 40 ppm using distilled water before the chemical impregnation procedure starts. After desalinization, to prevent biological activity, bacterial and fungal growth, the biocide EXOCIDE 1012 solution is added into the pools at a concentration of 1:1000 (Fig. 7).

The iron traces formed by the corrosion of the iron nails were cleaned with a % 5 EDTA and mixture of disodium EDTA and oxalic acid. The solution, applied to the timbers with swabs, was left on the surface for approximately four hours before being washed under flowing water for at least 15 minutes and up to two days. This process was repeated until the iron traces were totally removed (Fig. 8)

Figure 7: Salinity level control (left)
Figure 8: Cleaning of iron corrosion stains (right)

4.3 Conservation Treatment

The Yenikapı shipwrecks have survived in better condition than many underwater wrecks in the Mediterranean region because they were buried relatively quickly under a thick layer of muddy sediment. Biological activity over the centuries has still caused degradation at different levels in the cellular structure of the timber. For their conservation, PEG (polyethylene glycol) and melamine formaldehyde resin are being employed. Melamine formaldehyde preferred for elements of highly deteriorated non-durable materials such as plane wood [13]. Wrecks in a relatively better condition, and those likely to be displayed in the future, were first soaked in a % 45 solution of PEG 2000 and then freeze dried. Istanbul University has established two laboratories for the conservation and restoration of the waterlogged timbers from the ships. The new Department of Conservation of Marine Archaeological Objects has enabled the launch of the first academically organized training in this field in Turkey

4.4 PEG Impregnation-Vacuum Freeze Drying Method

Ship timbers are first impregnated with PEG 2000 dissolved in tap water. Impregnation begin % 5 concentration to minimized the occurrence of osmotic collapse. Also biocide is added at a ratio of 1:1000 to eliminate any bacterial formation in the solution. The procedure was completed when the PEG concentration of the solution reach % 45 [14, 15].

Following the PEG impregnation procedure, the process of dehydration is performed by a freeze-drier used for the first time in Turkish scientist by the Istanbul University the Ship Conservation and Reconstruction Laboratory (Fig. 9)

Figure 9: Loading of PEG pre-treated objects to freeze dryer

Expansion of the water in the wood cells due to freeze is avoided by pre-impregnation with PEG before freeze-drying. By the use of freeze-drying, water is removed without incurring either the force of its surface tension on delicate structures or the drag of consolidant from the core to the surface of the wood. Also known as sublimation, freeze-drier primarily reaches to the freezing point of PEG solution and then eliminates the solidified water in the timber through vaporization [16] (Fig. 10a-b).

Figure 10a-b: Before (above) and after (below) photos of a freeze dried timbers

4.5 Melamine Formaldehyde Method

In Yenikapı project, melamine-formaldehyde resin (Kauramin® 800) are being used for conservation of highly deteriorated waterlogged timbers. Low molecular wait melamine- formaldehyde prepolymer dissolved % 25 concentration in deionized water. % 10 butanediol or triethylene glycol based on melamine-formaldehyde added to solution for flexible structure for the wood. % 0.5 triethanolamine based on melamine-formaldehyde added to the solution to extend the curing time of the solution (12-14 months). % 5 urea based on melamine- formaldehyde added to the solution to reduce the viscosity of the solution. Urea also binds free formaldehyde and reduce the health hazard.

The woods are placed in the solution. The boxes are covered and sealed with plastic foils. Impregnation time differ the thickness and species of the woods. Monitoring and pH measurements of the solution is very important in this stage. If the pH drops to values 7-6, polymerization start soon [17].

When the melamine resin cured, impregnated samples were removed from the solution, wrapped in wet paper, then plastic foils and placed in oven set to 35-50 °C. The drying period of samples differed between ten to twelve days [18], (Fig. 11-12-13).

Figure 11: Placing the waterlogged woods into melamine-formaldehyde solution
Figure 12: Loading of melamine-formaldehyde pre-treated objects to oven

Figure 13: A group of waterlogged wood from YK 26 shipwreck, after melamine formaldehyde conservation

5 STATUS OF THE YENIKAPI SHIPWRECKS

The 37 shipwrecks uncovered in the course of salvage excavations at Yenikapı and dated from the 5th to the 11th centuries AD constitute the biggest medieval shipwreck assemblage uncovered at a single site to date. The wide date range of the wrecks provides us with a unique opportunity to comprehend the development of shipbuilding technologies in the Mediterranean region.

The "PEG Impregnation-Vacuum Freeze Drying Method" which was used for waterlogged wood of the Yenikapı shipwrecks, has been applied by İstanbul University specialists for the first time in conservation in Turkey. As a result, there is no surface-tension, shrinkage, cracks, and dimensional deformity observed on dried wood. The timbers seems natural after the conservation procedure.

The melamine-formaldehyde method (Kauramin® 800) provided dimensional stabilization for highly deteriorated waterlogged wood of Yenikapı. Surface details of the woods preserved very well. After conservation the weight is low but the surface collar is unnaturally light.

The continuing study of the Yenikapı shipwrecks will certainly make great contributions to our understanding of medieval shipbuilding technologies and improving skills about conservation of waterlogged archaeological objects (Table 1).

Table 1: Conservation status of the Yenikapı shipwrecks

Shipwreck	Desalinization	Cleaning of Iron Products	Conservation Method	Status
YK 1	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG 2000 impregnation-vacuum freeze drying	Completed in 2016
YK 2	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG impregnation-vacuum freeze drying	Concentration of PEG % 25 (continuing)
YK 3	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG 2000 impregnation-vacuum freeze drying + Melamine formaldehyde method	Concentration of PEG % 35 (continuing) Melamine formaldehyde method used for conservation of highly deteriorated timbers and keel
YK 6	Completed	Cleaned % 5 mixture of disodium EDTA	PEG 2000 impregnation-vacuum freeze drying	Concentration of PEG % 25 (continuing)
YK 7	Completed	Cleaned % 5 mixture of disodium EDTA	PEG 2000 impregnation-vacuum freeze drying	Concentration of PEG % 10 (continuing)
YK 8	Completed	Cleaned % 5 mixture of disodium EDTA	PEG 2000 impregnation-vacuum freeze drying + Melamine formaldehyde method	Melamine formaldehyde method used for conservation of highly deteriorated planks
YK 9	Completed	Cleaned % 5 mixture of disodium EDTA	PEG 2000 impregnation-vacuum freeze drying + Melamine formaldehyde method	Ready for PEG impregnation. Melamine formaldehyde method used for conservation of wooden sticks inside the boat
YK 10	Completed	Not started	Not started	
YK 12	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG 2000 impregnation-vacuum freeze drying	Concentration of PEG % 45 (continuing)
YK 13	Completed	Not started	Not started	
YK 15	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG 2000 impregnation-vacuum freeze drying	Ready for PEG impregnation.
YK 16	Completed	Not started	PEG 2000 impregnation-vacuum freeze drying + Melamine formaldehyde method	Melamine formaldehyde method used for conservation of highly deteriorated floor timbers
YK 17	Completed	Not started	Not started	
YK 18	Completed	Not started	Not started	
YK 19	Completed	Cleaned % 5 mixture of disodium EDTA	PEG 2000 impregnation-vacuum freeze drying	Ready for PEG impregnation.

YK 20	Completed	Cleaned % 5 mixture of disodium EDTA	PEG 2000 impregnation-vacuum freeze drying	
YK 21	Completed	Not started	Not started	
YK 22	Completed	Not started	Not started	
YK 25	Completed	Not started	Not started	
YK 26	Completed	Not started	Melamine formaldehyde method	Ready for Melamine formaldehyde method
YK 27	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG 2000 impregnation-vacuum freeze drying	Concentration of PEG % 15 (continuing)
YK 28	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG 2000 impregnation-vacuum freeze drying	Ready for PEG impregnation.
YK 29	Completed	Not started	Not started	
YK 30	Completed	Cleaned % 5 mixture of disodium EDTA and oxalic acid	PEG 2000 impregnation-vacuum freeze drying	Concentration of PEG % 10 (continuing)
YK 31	Completed	Cleaned % 5 mixture of disodium EDTA	PEG 2000 impregnation-vacuum freeze drying	Ready for PEG impregnation.
YK 32	Completed	Not started	Not started	
YK 34	Completed	Not started	Not started	
YK 35	Completed	Not started	Not started	
YK 36	Completed	-	Melamine formaldehyde method	Completed in 2015
YK 37	Completed	Not started	Not started	

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ESTIMATION OF TIMBER MEMBERS' PROPERTIES COMBINING DIRECT AND INDIRECT INFORMATION: TWO APPLICATIONS

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Abstract. *In conservation or rehabilitation works of ancient buildings serious doubts are often raised about the real condition of its structural timber members. Improving the practical knowledge upon visual and local recognition of the members in service and their connections is therefore of huge importance. This subject is nowadays under discussion and international committees (RILEM, COST E55, COST IE0601, COST FP1101) have recently promoted some guidance documents to support actions to assess the structural condition of timber members.*

This research presents two practical situations. The assessment of two pine wood structural elements included in historic buildings located in the center of Lisbon city: a timber floor and a timber roof truss. These structural elements, composed for several timber members were submitted to: i) visual strength grading according to national and international standards; ii) semi-destructive testing; iii) safety assessment. Methodologies for property estimation of timber members in service are applied and their results discussed.

Through the development of this study it became clear the importance of visual inspection for this kind of cultural heritage buildings. The use of a visual strength grade standard specific for timber members in service is also very important. In this case, the standard used accepted four beams that would have been replaced if only other methods had been used. On the other hand, the closer estimation of the density of the elements leads to a more real safety assessment. This study gains great relevance in the current reality in which the industry of building rehabilitation is on the rise but is not always accompanied by the necessary technical knowledge of the technicians involved and often leads to the removal of timber structural members in perfect physical and mechanical conditions..

1 INTRODUCTION

Evaluation of ancient timber structures in service has become a subject of great interest in the last decades. In fact, this initial evaluation deeply influences the crucial decisions in the process of rehabilitation. But the problem faced by the technicians remains as twenty years ago “the lack of data about the real load bearing capacity of ancient wooden members, due to a diffuse bad knowledge of wood mechanical properties by architects and engineers and to a lack of appropriated standards about the mechanical grading of old wood. From this situation derives the need of an in situ diagnostic technique, based on reliable and quick non-destructive technologies, from which is obtained some data about the structural characteristics of each single wooden member” [1].

I this sense, research on the knowledge of the resistant properties of timber structures in service in old buildings and their evaluation processes have been intensely developed by the scientific community in recent years. Several international committees (RILEM, COST E55, COST IE0601, COST FP1101) have recently promoted some documents to support actions to assess the structural condition of timber elements [2, 3, 4, 5].

The physical-mechanical assessment of timber on site by non-semi-destructive methods (NDT / SDT) has been the object of important studies involving laboratorial research for over three decades [6, 7, 8, 9, 10, 11, among others]. A state-of-the-art report on “Combined use of NDT / SDT methods for assessment of structural timber members”, promoted by COST FP1101, has recently been published, gathering opinions from the most eminent European researchers on the subject [5]. While stress waves, spectrography, tomography and other methods have provided promising results, drill resistance and penetration resistance tests deserve to be in the limelight for their simplicity of use and the relative adequacy of the response to the objectives. The mechanical strength, modulus of elasticity and density, have been correlated with drill resistance and with penetration resistance results and shown as useful methods for assessing wood [10, 11, 12, 13, 14, 15].

Core drilling is a multifunctional semi-destructive method (SDT) of great utility, since it makes direct readings, providing information on wood species, water content, resistance and density [16]. It can also be used as a method of calibrating the data obtained indirectly by the correlation with the tests of resistance to drilling and penetration. The authors of [17, 18] have developed several techniques to obtain other resistance parameters from the carts extracted from structural elements, such as tensile strength and compression, which have been shown to be more accurate than indirect readings.

Density knowledge is critical since it is an important property due to its direct positive impact on the strength and stiffness of the wood. This parameter is also used to predict the modulus of elasticity. Density can be estimated using SDT methods as core drilling, drill resistance and penetration resistance [14, 16, 19].

Regarding the attribution of parameters of strength and stiffness to timber members in service, several authors indicate that visual strength grading (VSG) will remain as the basic method for assessing the mechanical performance of timber members in situ [11, 20, 21]. Quantification of the original mechanical resistance is related to the presence of natural defects such as knots, slop of grain, cracks and others, through the definition of quality classes by the application of national standards. However, VSG standards were developed having in mind the grading of sawn timber at sawmill yards. So, the full application on site of the rules applied at the sawmill yard is not possible or logical and leads to gross underestimation of the real mechanical performance of timber members [21]. Several studies agree on the fact that the sole use of the VSG method can lead to the demolishing of safe structures and that other NDT/SDT methods should be used to ensure a proper assessment procedure. Combined with a visual grading survey, these evaluation methods are an excellent complement to achieve a proficient level of reliability in the structural analysis, diagnosis and inspection of existing constructions [22]. But an important characteristic of several ancient timber structures is that they can effectively bear higher loads than expected [20], which stresses the need of adequate procedures for diagnostic and assessment of the real bearing capacity [22]. UNI 11119:2004 is a standard that establishes criteria for the diagnosis of old timber elements and strength grading and can be performed using both on-site inspections and NDT techniques. This standard and the authors of the document [4] propose conditioning the VSG upon the stress condition and the position of defects in relation to applied stress [21].

2 SURVEY DEVELOPMENT

2.1 Timber structures

This work presents two practical situations located in the center of Lisbon city in which evaluation conditioned the future use of the structure. The first case was performed by [23] in which a wooden floor with a structure of 14 joists of a XIXth century palace was assessed. The second case is a timber truss representative of all roof in an early XXth century wooden building.

2.2 Materials and methods for diagnosis

First step for the in-situ assessment of old timber beams is usually the visual inspection. It can be done thoroughly, but generally the main information on the history and actual conditions of the material is achievable through this kind of approach [24]. In this sense, preliminary stage of survey consisted in a general assessment of timber structure, acknowledgement of its main defects and risks as well as a first approach of identification of the wood species. There was also carried an historic and technical survey about the target building. The structural frames of the chosen areas were also analyzed, regarding the construction processes, wood species and moisture content of the wood. There were made five to seven moisture content readings per element, with the purpose of allowing a proper evaluation of the average moisture content in each area. Two different moisture meters were used: one using the resistance method and another using the capacitance method (see [25]).

In this stage, it is also important to ensure, whenever possible, conditions of accessibility, lighting and cleaning, for the next survey stage [4].

The second and more detailed stage of survey consisted in a thorough visual inspection and application of non-destructive inspection techniques [4]. Wood was tested initially using a cutting object (knife or chisel) and a scale, both to check fissure deep and width, as well as surface integrity. This kind of sharp tools can also be used to evaluate the progress of biological degradation, by detecting soft or disintegrating material. For this purpose, a series of micro-drilling tests with the Resistograph® IML Resi-B-1280 regulated to a penetration speed of the needle of 20 cm/min were carried out. The residual cross section could be estimated on this basis. All measuring obtained data was later registered using 3D design software. Based on the service situations of each timber structure, the use classes from EN 335-2:2011, related with the hazard of biological attack were defined.

Then a series of penetration resistance tests was carried out with the Pilodyn® 6J equipment in each timber member taking the caution of performing the test in the radial direction of the growth rings, whenever possible. This test gives direct information on the surface hardness of the wood and indirectly on its density [26].

To predict the mechanical strength of each timber member by Visual Strength Grading (VSG), there were followed two different standards: Portuguese NP 4305:1995 and Italian UNI 11119:2004. Visual grading assigns quality grades to wood elements, which are related with strength grades and corresponding mechanical properties, such as: characteristic value, modulus of elasticity, shear modulus, material density, bending strength, and compression strength both parallel and perpendicular to the grain.

Portuguese visual strength grading standard applies to maritime pine wood (*Pinus pinaster*, Ait.) for structural purposes graded in sawmill. Its objective is to estimate the mechanical strength and stiffness through visual evaluation, covering aspects as: wood density; presence of pitch; occurrence of defects such as knots, slop of grain and resin pockets; wane; damaged material due to biological degradation, shakes and distortion. EE (special structural timber) and E (regular structural timber).

Italian visual standard establishes objectives, procedures and criteria in the evaluation of the state of conservation and assessment of load-bearing timber members in service in cultural heritage buildings. This standard considers the evaluation of a critical zone, which corresponds to the most stressed area of the element, considering visible surface alterations and/or defects that can influence strength and stiffness characteristics, thus influencing mechanical performance of the timber element. The analysis of these defects should be made over a minimum length of 150 mm, which can be extended if the critical section is close to the end of this range. For diagnosis purposes, only the residual resisting cross sections should be considered. This standard includes three quality grades (I, II, III) which are related with verified aspects on site such as wane, single knots or group of knots, slope of grain, shrinkage checks, frost cracks and ring shakes.

2.3 Safety assessment

According to the Eurocode 5 (EN 1995:2004) safety checks are related with ultimate limit states (bending, shear and torsion) and serviceability limit states (deflection and vibration). This study presents the safety assessment deal with the ULS of bending and the SLS of deflection for the timber floor joists. In fact, this limit states are usually the most penalizing. The defined load combinations were based on Eurocode 0 (EN 1990:2009). For the timber truss were checked the safety measures related to all limit states.

2.4 Background

Estimation of the wood density was based on studies by [19] who correlated the resistance to penetration of the wood with its density applying the dynamic impact method. The method consists of introducing a metal pin in the timber with a given energy. The penetration depth obtained is inversely proportional to the timber hardness in the cross-section and can be used as a measure of the timber's density [26]. This research was carried out with different pine timbers (*Pinus pinaster* and *Pinus sylvestris*) new and collected in two buildings that were 80 and 150 years old. The 65 samples used were free of defects, with both sapwood and heartwood and with the fibres parallel to the piece axis. It was obtained a very good correlation with a determination coefficient r^2=0,80 using the Pilodyn 6J Forest equipment.

The equation (1) of the correlation model [19] is shown below.

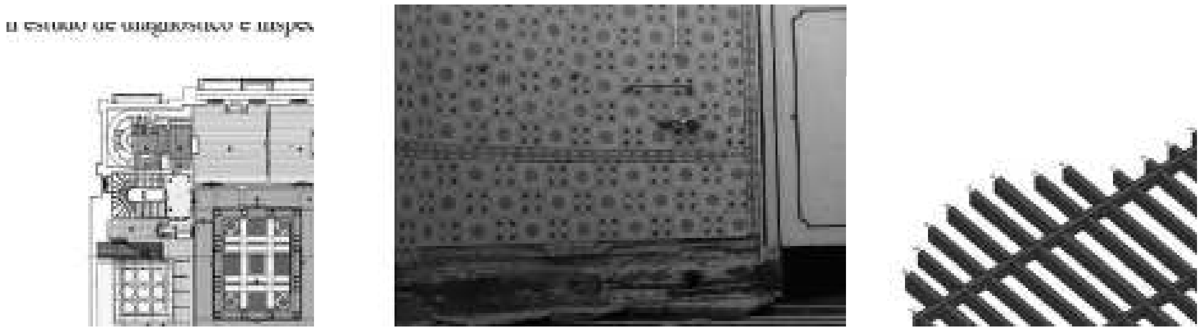
$$Pd=-0.0312\rho+33.043 \tag{1}$$

where: Pd = penetration depth (mm), ρ = density of the pine wood

3 TIMBER FLOOR

3.1 Presentation

The timber floor was composed by fourteen joists and two sets of blockings (Figure 1). It makes up the floor of a division on the top level of a Lisbon´s palace dated from 1877. This floor is a valuable component of ancient interiors due not only to its historical value but also to the rich stucco-decorated ceiling that it supports.



a) **b)** **c)**

Figure 1: Timber floor: a) location in the building; b) view; c) structure draw

3.2 Survey results

3.2.1 Visual characteristics

Based on NP EN 335-2:2011 studied area was classified as use class 1. At inspection, the room had a temperature medium of 23ºC and Relative Humidity medium of 58%. The timber elements don’t shown signs of high moisture content nor in the present situation nor in the past. The moisture content medium measured was within the expected values (Table 1). The area was well lighted and wide thus easing handling of inspection equipment. In four joists, the n.º 3, 8, 11 and 13, was verified biological damage caused by boring beetles attack, identified for a rough surface and the presence of sawdust. The wood was identified as pine, probably maritime pine. The VSG according to the standard NP 4305 (related to maritime pine in sawmill) was done to each joist covering aspects as: presence of pith; growth rate; occurrence of defects such as knots, slop of grain and resin pockets, wane, cracks and distortion. The class assignment was done not considering biological degradation. The VSG f according to the standard UNI 11119 (related to all timber species in load-bearing structures’ members) was done to each beam considering the position of defects in relation to stress condition who is for all joists in the mid span. The critical sections were defined there.

Data of visual characteristics is presented in Table 1.

Table 1: Characteristics of timber floor members obtained on-site.

Joist n. º	Length [m]	Section* [cm]	M.C. [%]	Grade NP 4305	Grade UNI 11119
1, 3	1.98	22x9	10.9±0.5	E	II
2, 4, 5, 14	1.73 to 2.20	22x9	10.1±0.2	E	I
6, 12, 13	2.82 to 3.20	22x8.5	9.4±0.6	E	I
7	3.33	22x9	9.7±0.6	EE	I
8	3.89	21x11	10.0±0.3	E	III
9, 10	3.88 to 3.95	22x9	9.8±0.8	E	I
11	3.69	21x8.5	10.4±0.3	E	I

* Section of sound wood

3.2.2 Reference properties

Two reference properties for the studied floor are presented in table 2. Density was estimated through the penetration resistance method. Average modulus of elasticity on bending was estimated by VSG using two different standards: NP 4305 and UNI 11119 for pine wood.

Table 2: Properties obtained from SDT and Grading Standards.

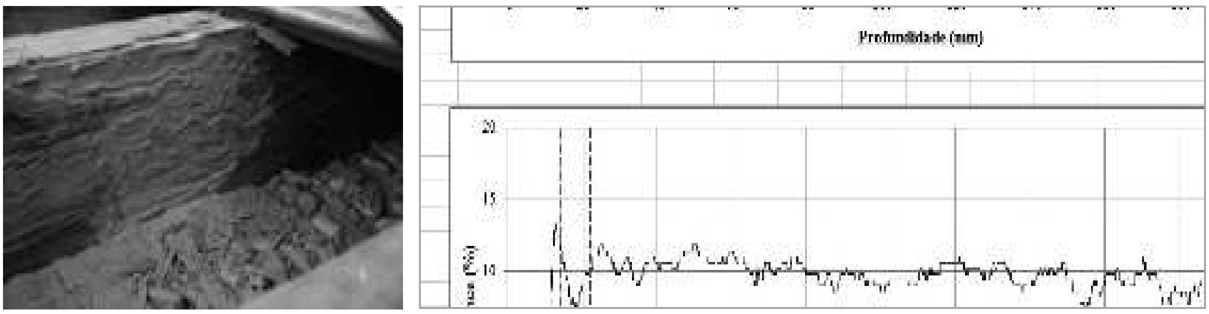
Joist n.º	Pilodyn®		NP 4305				UNI 11119			
	Penetration depth [mm]	$\rho_{m,SDT}$ [kg/m^3]	Grade	Strength class EN 338	$E_{0,mean}$ [GPa]	$f_{m,k}$ [MPa]	$\rho_{m,VSG}$ [kg/m^3]	Grade	$E_{0,mean}$ [GPa]	$f_{m,k}$ [MPa]
1, 3	18,1±1,5	479	E	C18	9	18	380	II	12	10
2, 4, 5, 14	18,4±1,8	469	E	C18	9	18	380	I	13	12
6, 12, 13	17,2±1,8	508	E	C18	9	18	380	I	13	12
7	17,8±0,4	489	EE	C35	13	35	480	I	13	12
8	18,5±1,9	465	E	C18	9	18	380	III	13	8
9, 10	18,0±1,1	483	E	C18	9	18	380	I	11	12
11	17,5±1,7	496	E	C18	9	18	380	I	13	12

As shown in table 2 the estimated values of density by SDT are much higher than those corresponding to the strength classes of EN 338. This can succeed due to particular characteristics of the wood species under assessment.

Considering the modulus of elasticity, it is also clear in table 2 that the values estimated through the grade attributed by UNI 11119 are much higher than those by NP 4305.

3.2.3 Biological degradation

Biological degradation caused by boring beetles attack was detected in the joists n.º 3, 8, 11 and 13, identified for a rough surface and the presence of sawdust. With the help of a knife the powdered wood was removed and sound wood was found at a depth of 0.5 to 2 cm from the face (Figure 3 a)). This fact was confirmed with the use of the *resistograph®* by means of several profiles performed in the degraded zones. Figure 3b) shows a drill resistance profile that detected two weaker spots (A and B) located 1 or 2 cm away from the faces. These weaker areas can be related with biological degradation by house longhorn beetle.



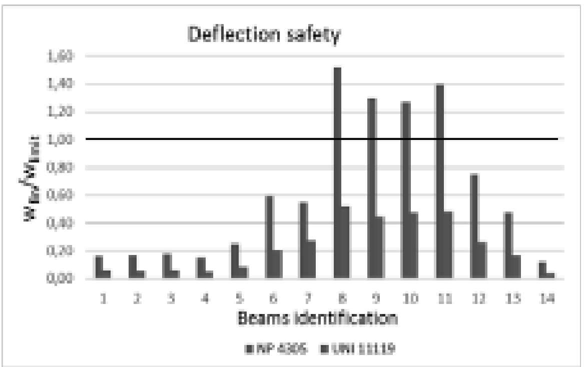
a) b)

Figure 3: Beetle attack in a joist of the timber floor: a) view; b) drill profile

3.3 Safety assessment

Beams were defined as being simply supported with a uniformly distributed load along the length of the element. According to Eurocode 1 (NP 1991:2009), regarding type of occupancy and use of the room, the studied area belongs to category A. For the safety calculation were used higher density values for safety reasons and the effective section for the degraded joists n.º 3, 8, 11 and 13.

For service class 1, the deflection modification factor (k_{def}) for solid timber is 0.6. The final deflection, w_{fin}, was calculated for the quasi-permanent combination of actions. The deflection which results from the effects of actions (such as axial and shear forces, bending moments and joint slip) and from moisture shall remain above the limiting value *l/250* (*l* – span of a beam on two supports) (EN 1995:2004 -National Annex). A ratio above one is not acceptable. The ratio between final deflection and limiting deflection was calculated using the following values of $E_{0,m}$: i) according to NP 4305; ii) according to UNI 11119 (Figure 4). Regarding bending situation of the joists, the safety condition is that the ratio between bending stress and bending strength is less than or equal to 1. The partial factor for material properties (γ_M) for solid timber is equal to 1,3. For the modification factor for duration of load and moisture content (k_{mod}), it was admitted that this case fits in service class 1, giving values of 0,6 for permanent loads and 0,8 for variable loads.



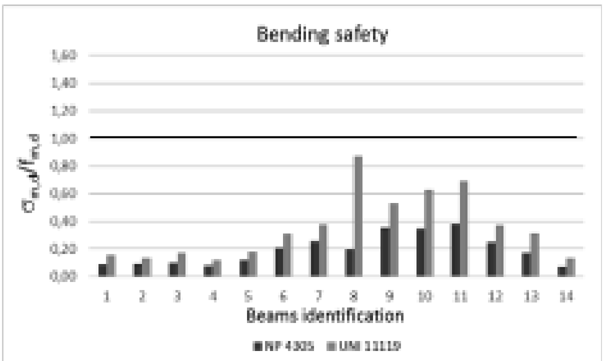


Figure 4: Safety assessment of deflection and bending for values of $E_{0,m}$ and $f_{m,k}$ according to two standards

As shown in the Figure 4 a), for the values of $E_{0,m}$ according to NP 4305 the beams of larger spans (3.70 to 3.95 m), numbers 8, 9, 10 and 11 present a deflection above the limiting value. But, for the values of $E_{0,m}$ according to UNI 11119, all joists are under the limit of deflection, even joist 11 that has a cross section reduction due to by biological attack. In fact, using the values of $E_{0,m}$ provided by UNI 11119, even for the beams degraded by beetles, the effective cross sections remaining are still enough for safety. With regard to bending safety, as shown in Figure 4 b), the joists are safe for values of $f_{m,k}$ according to both standards, even for the beams degraded by beetles. However, to avoid the progress of the biological attack, the application of a preservative treatment is recommended.

4 TIMBER TRUSS

4.1 Presentation

The present truss belongs to a timber roof truss of a wooden building dated from 1902 and located in the center of Lisbon city. Trusses occur at regular intervals, linked by longitudinal timbers such as purlins (Figure 4). It is like a common king-post timber truss. Consists of two diagonal principal rafters that meet at the top of the truss, one horizontal tie beam, one central vertical king-post, two diagonal struts and two more small struts near the ends of the tie beam. Tie beam is connected vertically through pairs of wooden planks nailed with three members: the king post and the two rafters.

King post is working in tension to support the tie beam below from a truss top above. Tie beam is under tension because it ties the bottom end of the two principal rafters and is also under bending due to the king post and the wooden planks nailed to the rafters, and even due to the small struts who overload the beam. Diagonal members of the truss (principal rafters and struts) are working in compression, but principal rafters are also under bending due to the purlins (Figure 5).

a) **b)** **c)**

Figure 5: Timber roof truss: a) location on the roof; b) view; c) structure draw

4.2 Survey results

4.2.1 Visual characteristics

All timber members were visually inspected and all measurements done. At inspection, the space had a temperature average of 19ºC and Relative Humidity average of 67%. The moisture content of each member was measured with both the moisture meters, one using the resistance method and another using the capacitance method (Figure 6a)). The data were calibrated with each other.

Based on NP EN 335-2:2011, studied area was classified as use class 2 due the risk of wetting through the roof. At inspection, the timber elements don't show signs of high moisture content nor in the present situation nor in the past. None of the members revealed signs of biological degradation. The wood was identified as pine, probably maritime pine.

The VSG according to the standard NP 4305 was done to each timber member covering aspects as: presence of pith; growth rate; occurrence of defects such as knots, slop of grain and resin pockets, wane, cracks and distortion. The VSG f according to the standard UNI 11119 was done to each member considering the position of defects in relation to stress condition of each member. The critical sections were defined there.

Data of visual characteristics are presented in Table 3.

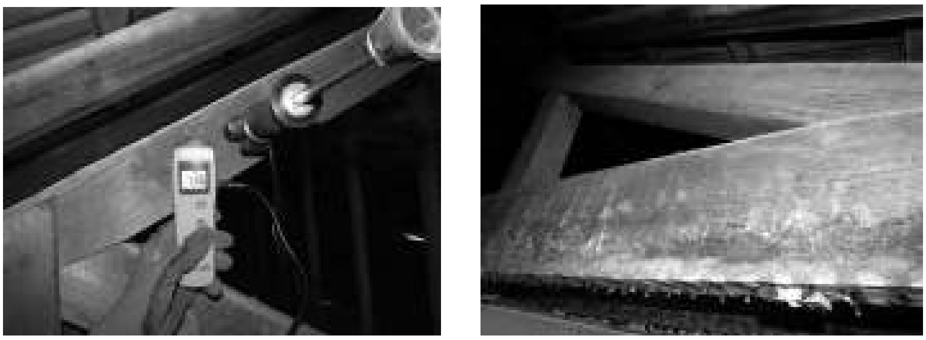
Table 3: Characteristics of timber truss members obtained on-site.

Truss member	Length [m]	Section [cm]	M.C. [%]	Grade NP 4305	Grade UNI 11119
Tie beam	5.80	17.0x7.5	14.9±0.6	EE	I
Principal rafter SW	3.20	10.0x7.5	12.1±0.2	E	I
Strut SW	1.20	9.5x7.5	13.4±0.3	E	II
King-post	1.55	9.5x9.5	13.7±0.6	E	II
Strut NE	1.20	10.0x7.5	12.8±0.8	E	III
Principal rafter NE	3.20	13.0x8.5	13.4±0.3	EE	I

All members were graded as structural timber according to NP 4305.

The grain was straight in all the structural elements and the knots relatively small and concentrated in the middle area of the timber members. Only the NW struct had medium-sized but still acceptable knots. The growth rate was relatively easy to measure due to the radial cut of almost all the elements (Figure 6 b)). The king post was cut with the pith inside.

The joints are in good condition, but show some small gaps in the contact areas. These gaps can drastically reduce the stiffness and strength of the joints by causing a high concentration of stresses in the areas that are in contact [4]. It is therefore recommended that when necessary, the gaps be sealed with metal or hardwood wedges, properly nailed or screwed in order to improve the adjustment of the surfaces in contact.



a) **b)**

Figure 6: a) Moisture meters; b) view of characteristics of tie beam and principal rafter NE

4.2.2 Reference properties

The estimated values for the average density get with the penetration resistance method were 600 to 670 kg/m^3 for all members except the strut NE. These values were in general higher than the average value measured for clear maritime pine wood to 12% moisture content (530 to 600 kg/m^3) [27] and much higher than the mean values attributed by EN338 for the resistance classes C18 or C35.

Table 4: Properties obtained from SDT and Grading Standards.

Truss member	Pilodyn®		NP 4305				UNI 11119			
	Penetration depth [mm]	$\rho_{m,SDT}$ [kg/m^3]	Grade	Strength class EN 338	$E_{0,mean}$ [GPa]	$f_{m,k}$ [MPa]	$\rho_{m,VSG}$ [kg/m^3]	Grade	$E_{0,mean}$ [GPa]	$f_{m,k}$ [MPa]
Tie beam	12.1±1.8	672	EE	C35	13	35	480	I	13	12
Principal rafter SW	14.5±0.8	594	E	C18	9	18	380	I	13	12
Strut SW	13.0±1.4	642	E	C18	9	18	380	II	12	10
King-post	11.8±0.4	681	E	C18	9	18	380	I	13	12
Strut NE	20.2±0.8	412	E	C18	9	18	380	III	11	8
Principal rafter NE	13.0±0.9	642	EE	C35	13	35	480	I	13	12

4.3 Safety assessment

With the application of a structural analysis software were made the calculus of safety assessment for the higher values of density and of modulus of elasticity.

It was concluded that the timber truss is in good structural condition for grades assigned by both standards.

5 CONCLUSIONS

This evaluation and survey process highlighted the need to obtain data from diverse sources, even with limited technical means. However, in the two cases under study the data obtained were quite different, which leads us to the conclusion that the work done would have to be complemented with other inspection SDT/NDT techniques, laboratory tests and/or destructive testing on wood samples.

In respect to the condition of the timber elements studied, is clear that the materials used in the construction of these buildings had an above average quality. This is particularly relevant considering the buildings' age, which is of more than a century, and that was not verified the occurrence of severe degradation that could compromise the structural safety. The verified degradation was very localized, and the biological agents were all verified. This way it is possible to obtain the effective cross section on structural strength.

Through the development of this study it became clear the importance of visual inspection for this kind of cultural heritage buildings. The use of a visual strength grade standard specific for timber members in service is also very important. In this case, the standard used accepted four beams that would have been replaced if only other methods had been used. On the other hand, the closer estimation of the density of the elements leads to a more real safety assessment.

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CHARACTERISTICS AND EVALUATION OF TERMITE DAMAGE TO HISTORICAL WOODEN BUILDINGS IN SOUTH KOREA

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Keywords: Termites, Wooden buildings, Damage characteristics, Evaluation

Abstract.

Introduction:
Wood has long been used as a building material, and there are many wooden cultural heritages in South Korea. Termite damage to wooden buildings has been increasing as a result of global warming and forest fertilization. Therefore, in this report, the characteristics and current state of termite damage to South Korean wooden cultural heritages will be discussed.

Developments:
In South Korea, two species of subterranean termites are currently damaging wooden buildings throughout the country. They damage the inside of wooden members where it is moist, such as the lower part of wooden pillars; this consequently reduces the structural integrity of wooden cultural heritages. The National Research Institute of Cultural Heritage (NRICH) conducts a comprehensive inspection of all wooden cultural heritages scattered throughout the country every five years to preserve their original state. The methods and tools employed during these inspections include visual and surrounding forest inspections, termite detection dogs, microwave detectors, resistographs, ultrasound, and termite colony eliminators.

Remarks and Conclusion:
As a result of climate change, termite damage is increasing in South Korea, and is expected to continue to increase in the future. In order to preserve the original states of wooden cultural heritages, precautionary measures are required. In addition, the roles given to each field working to preserve these structures must be diligently performed. In the future, diagnostic tools such as X-ray tomosynthesis, suitable for wooden cultural heritages located outdoors, and control measures such as physical barriers, must be actively implemented.

1 INTRODUCTION

Wood has been used as a building material since prehistoric times because it is abundantly available, possesses high strength relative to its weight, and offers various other advantages as a building material. In South Korea, particularly, numerous wooden buildings were built in accordance with the social and cultural characteristics of society that emphasized harmony with nature; many of these buildings still exist as cultural heritages. As of late 2016, 180 out of 751 national architectural cultural heritages were wooden cultural heritages, and 184 out of 294 important folk heritages were wooden buildings(Table 1) [1].

Table 1: National-designated architectural cultural heritages in Korea

	National Treasure	Treasure	Important Folk Heritage
Wooden buildings	24	156	184
Other structures	71	500	110
Total	95	656	294

Wooden buildings are exposed to the open air and are consequently damaged by various environmental and human factors. Among these factors, damage caused by insects and microorganisms is the most significant; among the insects and microorganisms causing damage, in South Korea, termite is the worst offenders. Termites can damage the original state of wooden cultural heritages and reduce their structural integrity(Fig. 1). As a result, various diagnostic and control efforts are being made to prevent termite damage to wooden cultural heritages at an early stage. In this report, the characteristics of termite damage and the current diagnostic methods for Korean wooden buildings will be discussed.

Figure 1: Termite-induced damage to wooden pillars (Nogang Seowon Academy).

2 CHARACTERISTICS OF KOREAN TERMITES

The taxonomic infraorder of termites is *Isoptera*, and they are distinctively classified as social insects. They are further categorized as subterranean, drywood, or dampwood termites according to the ecological environment in which they live. In South Korea, *Reticulitermes speratus kyushuensis*, one of the subterranean termite species, is spread across the country; additionally, according to recent reports, *Reticulitermes speratus kanmonensis* has also been found to inhabit certain regions throughout the country [2]. Thus, two species are found in South Korea. Both species lack the ability to hold moisture, are vulnerable to dry climate, and live in dark places with high humidity. Consequently, they mainly live underground or in wood, where a certain number of workers dig galleries in every direction in search of food. Termite-induced damage to wooden buildings originates at the base of pillars, or baseboards, with high moisture content. In this case, it is difficult to visually detect the damage because it is contained within the wood in an area devoid of light. This damage yields hollowed-out wood, which consequently reduces its strength and structural integrity.

Termite damage to wooden cultural heritages has been reported since the 1970s, but did not receive much consideration until the late 1990s when termite damage was reported near Jongmyo Shrine and Haeinsa Janggyeong Panjeon Temple, two UNESCO World Heritage sites [3]. The inspection conducted by the Korea National University of Cultural Heritage in 2009 showed that 78 out of 231 buildings had been damaged by termites, with the inside of wooden members of 18 buildings having been damaged by termites at the time of the inspection [4]. The termite distribution inspection by the National Institute of Forest Science confirmed the presence of termite colonies throughout all regions of South Korea. In addition, the inspection by the NRICH in 2014 and 2015 confirmed that 138 out of 156 buildings had termite damage, and that termite colonies were present near 30 of the 138 damaged buildings at the time of inspection [5], indicating that the extent of termite damage was very severe and increasing.

The reason for this increase in termite damage is complex. Most wooden cultural heritages in South Korea are Buddhist temple buildings, most of which are located in the mountains because of the anti-Buddhist sentiments of the Joseon Dynasty. Since termites are xylophagous insects, and thus have a high population density in forests, it is common for them to migrate toward cultural heritages from nearby forests [6].

Additionally, recent climate change is also directly impacting termite activity and subsequent damage. The ranges and periods of termite activity are most affected by the atmospheric temperature. With the average annual temperature exhibiting a recent rise because of climate change, existing termites are active for longer periods and thus consuming more food. Furthermore, because of climate change, there is concern that foreign termite species, such as *Coptotermes Formosanus*, which could not survive the climatic conditions of South Korea to date, will be introduced [7].

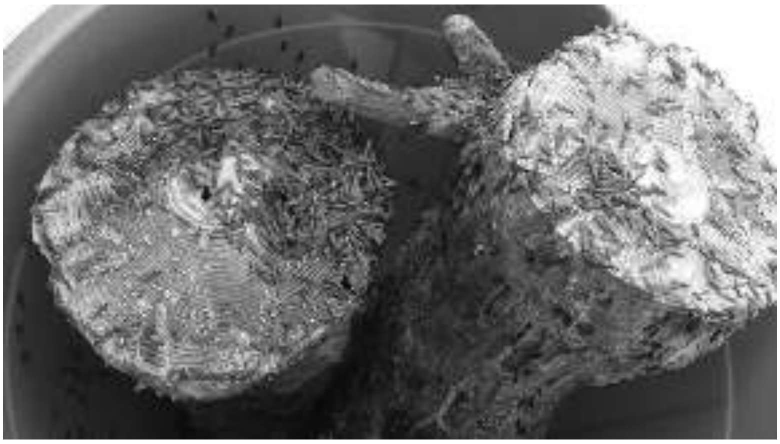

Figure 2: Swarm of termite alates.

3 EVALUATION OF TERMITE DAMAGE IN SOUTH KOREA

In order to prevent termite damage to wooden cultural heritages, the NRICH has catalogued wooden cultural heritages throughout the country according to region and performs an inspection of each heritage approximately every five years. In addition, an emergency inspection can be conducted upon special request by the cultural heritage managers or owners.

The termite inspection performed by the NRICH implements visual and sound inspections, termite detection dogs, and microwave detectors. First, a visual inspection and sound inspection are conducted to approximately demarcate the area of damage in main structural members such as pillars; then, detection dogs and microwave detectors are used to determine whether the inside of wooden members is currently being inhabited by termites, and whether the wooden members are currently being damaged.

Detection dogs detect volatile organic compounds (VOCs) secreted by termites(Fig. 3). Even if the termite colony is not active at the time of inspection, the dogs may detect the recent presence of termites from residual VOCs. Therefore, although detection dogs can accurately detect recent termite damage, they cannot detect termite damage that occurred long enough in the past for the residual VOCs to completely desorb. Furthermore, they cannot detect whether termites are active within the wood members at the time of inspection. To compensate for the limitations of this method, a microwave detector is used to detect the activity of termite colonies. The microwave detector detects microwave signal speed changes upon passing through termite bodies. Previous studies have reported that the microwave detector can be used on pine and zelkova trees, which are the main species used to construct wooden buildings in South Korea, with diameters up to 17 cm [4]; because of this, this tool is widely applied.

Figure 3: Termite detection dog sniffing VOCs* [8]
*Image source: http://www.samsunglifeblogs.com/1222

The above-mentioned detection dogs and microwave detectors are used to determine whether the wood has been damaged by termites, and whether the termites are active at the time of inspection. In addition to these methods, drilling resistographs and ultrasounds are used to estimate the residual strength of termite-damaged wooden members and to determine whether there exists a necessity for replacement. A drilling resistograph is useful because it can identify physical defects and biological damage within wooden members. However, small drill holes remain after inspection, thereby preventing retesting of the same area. This method yields limited application to cultural heritages, as preserving the original state of the structure is particularly important.

The ultrasound technique is mainly used to obtain a non-destructive diagnosis of stone cultural heritages, but is also applied to various wooden cultural heritages. Ultrasound is a completely non-destructive diagnostic method, and is thus suitable for monitoring since repeated measurements at the same area are possible. However, this method yields limitations for structural wooden members with large diameters (e.g., pillars) since physical defects and biological damage within the wood cannot be distinguished. Recent studies on pillars of cultural heritages have reported that this method also yields limitations with respect to identifying the physical properties within deep sections in wooden building pillars [9]. Therefore, this method must be cross-validated with other inspection methods.

In addition to the aforementioned on-site methods, the inhabitation of termite colonies in surrounding wooden structures and forests is inspected, and control measures such as soil treatment are established based on the inspection results. Termite bait is also installed around wooden buildings to monitor the activity of termite colonies and to remove them by exchanging the bait when they are identified. In 2013 and 2014, termite colony eliminators were installed throughout the entire temple of Haeinsa Janggyeong Panjeon, which is a UNESCO World Heritage site, and were monitored to exterminate termite colonies. As a result, termite colonies were identified in three places, and they were found to be only 30 m away from the Janggyeong Panjeon temple(Fig. 4). The termites were subsequently treated with a colony eliminator [10].

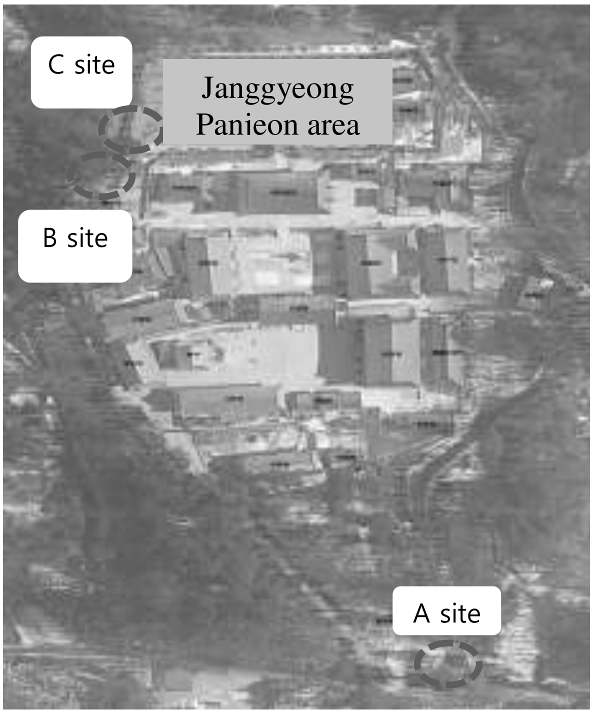

Figure 4: Termite monitoring and bait within the Haein-sa Buddhist temple.

In Korea, studies for diagnosing pillar interiors of outdoor wooden buildings using portable X-ray tomosynthesis equipment are being conducted, foregoing the use of X-ray CT equipment because it is difficult to directly apply to outdoor wooden buildings(Fig. 5). It was recently confirmed that synthesis of 19 X-ray images could diagnose the inside of wood up to 30 cm in diameter [11]. A corresponding field device has been developed and is currently being tested. However, since the device that was recently developed is limited in the height that it can measure, additional research and development will be required in the future.

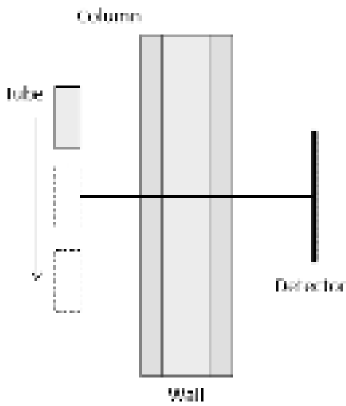

Figure 5: X-ray tomosynthesis geometry (parallel motion-type)

4 CONCLUSION

Termite damage is exhibiting an increase in South Korea that is concurrent with climate change, and this trend will continue. Moreover, it is expected that the activity period, activity range, and food intake of termites will increase because of climate change. Furthermore, there are concerns that other termite species will be introduced.

In order to preserve the original state, a matter important when considering wooden cultural heritages, introduction of termites must be prevented before damage can be done. R. speratus kyushuensis, a termite species found in South Korea, has a small population, and does not have a wide food-search range (references). Furthermore, they use wooden buildings as a food source rather than their habitat (references). Therefore, only a certain number of termites eat wooden buildings, while the rest of the colony resides elsewhere. Therefore, it is important to ensure that external termite colonies cannot approach wooden buildings.

The owners and managers of cultural heritages of concern must remove wooden structures near wooden cultural heritages to prevent the introduction of termites from nearby mountains; additionally, they must reduce the moisture content in wooden members via regular ventilation. The government and experts must receive training for daily management and perform regular monitoring to maintain these important wooden cultural heritages.

Furthermore, technologies that have recently been developed or are currently being applied in the field of forest preservation must be actively implemented in accordance with the characteristics of cultural heritages. Lastly, the application of termite colony eliminators that incorporate recent IOT(Internet of Things) technology to automatically transmit the state of termite damage, physical barriers to prevent termite invasion that were not previously used in the construction of cultural heritages, and wood preservatives that do not affect traditional paintwork, should be actively considered.

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