A complex assessment methodology for historic roof structures

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A complex assessment methodology for historic roof structures

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Keywords: Historic timber roof structures; multidisciplinary assessment methodology; interdisciplinarity; linear analysis; seismic vunerability; climate change vulnerability

Summary,
Timber structures are a complex research topic, offering research opportunities in a wide array of domains, reaching from heritage timber structures to contemporary ones.
However, existing/heritage timber structures, bring forward a series of challenges. The complexity of this topic arises from the fact that their structural behaviour is also strongly connected to their state of conservation, the type of the traditionally crafted joints used and their link to other structural parts made of different materials.
Considering this, an extensive research was performed on roof structures from Timisoara, a city from the western part of Romania. The study highlights that besides understanding their structural behaviour, there are a series of factors, which influenced the shape of the roof and the used structural typology and that the aesthetic features and the relation with the surrounding urban area has to also be taken into consideration. Noteworthy is also to consider environmental factors which can affect their state of conservation and the link to the building and acknowledge how these structures can influence the behaviour of the buildings during seismic events. Only after such a comprehensive analysis, the roof structure can be structurally evaluated and decisions and strengthening measures, if necessary, taken.
Therefore, by respecting the ICOMOS/ISCARSAH principles which are encouraging a multidisciplinary assessment of heritage structures, a comprehensive assessment methodology suitable for historic timber roof structures was developed, which can be used to determine their value and vulnerability and be used as a decision-making tool for the planning and hierarchisation of future interventions.

Keller Alexandra-Iasmina

O metodologie complexa de evaluare a sarpantelor istorice

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Keywords: șarpante istorice; metodologie de evaluare multidisciplinară; interdisciplinaritate; analiză lineară; vulnerabilitate seismică; vulnerabilitate climatică

Rezumat,
Structurile din lemn sunt un subiect complex de cercetare, oferind oportunități de studiu în multiple direcții, de la analiza unor structuri și materiale contemporane până la analiza și protejarea patrimoniului istoric de lemn.
Cu toate acestea, structurile de lemn istorice prezintă o serie de provocări. Complexitatea acestui subiect rezultă din legătura dintre comportamentul lor structural și starea lor de conservare, tipul îmbinărilor tradiționale utilizate cât și de modul in care sunt conectate cu alte structuri din diferite materiale.
Având în vedere acest lucru, a fost efectuată o cercetare amplă asupra șarpantelor din Timișoara. Studiul subliniază că, pe lângă înțelegerea comportamentului lor structural, există o serie de factori, care au influențat forma acoperișului și tipologia structurală utilizată și că trebuie luate în considerare caracteristicile estetice ale acestora cât și relația cu zona urbană înconjurătoare. De asemenea, studiul arată cu ajutorul unor simulări lineare că factorii de mediu și evenimentele extreme cauzate de schimbările climatice pot afecta starea de conservare a șarpantelor istorice, dar și că acestea pot influența comportarea seismică a unei clădiri istorice reducându-i astfel vulnerabilitatea seismică.
Prin urmare, prin respectarea principiilor ICOMOS/ISCARSAH care încurajează o evaluare multidisciplinară a structurilor patrimoniului, a fost dezvoltată o metodologie de evaluare cuprinzătoare, adecvată pentru șarpante istorice din lemn, care poate fi utilizată pentru a determina valoarea și vulnerabilitatea acestora cât și ca instrument în procesul decizional, pentru planificarea și ierarhizarea intervențiilor viitoare.

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SYMBOLS AND ABBREVIATIONS

Symbols

M_{eq}	equilibrium moisture content of timber elements
h_r	the relative humidity of the surrounding air (fractional)
T	air temperature (Fahrenheit)
F_c	impact force of the hailstone [N]
m	mass of a hailstone [g]
v_0	hail-stone velocity [m/s]
COR	Newton coefficient of restitution [m/s]
k_n	stiffness coefficient of a hailstone [m/s]
p	exponent of the spring behaviour [m/s]
k_{ax}	axial stiffness of the joint [kN/mm]
E_α	elastic modulus of the timber at an α angle with the fibre
A_c	the compressed area of the joint
l	notch length
E_0	elastic modulus of the timber along its fibre
E_{90}	elastic modulus of the timber perpendicular to its fibre
A_{insert}	cross-sectional area of the inserted element of the joint
h	the height of the base timber element of the joint
α	angle between the timber elements forming the joint
A_{vert}	horizontal contact area, where the vertical load is transferred
A_{horiz}	vertical contact area, where the horizontal load is transferred
b	width of the base element [cm]
$b_{inserted\ element}$	width of the inserted element [cm]
b_{tennon}	width of the tenon [cm]
t_{notch}	depth of the notch [cm]
t_{notch}^*	resulting depth of the notch if a tenon is also used [cm]

Abbreviations

S1.1.	Scenario involving rigid supports and rigid joints
S1.2.	Scenario involving sliding supports and rigid joints

S1.3.	Scenario involving hinged-sliding supports and rigid joints
S2.1.	Scenario involving rigid supports and hinged joints
S2.2.	Scenario involving sliding supports and hinged joints
S2.3.	Scenario involving hinged-sliding supports and hinged rigid joints
S3.1.	Scenario involving rigid supports and semi-rigid joints determined using the Hölzer method
S3.2.	Scenario involving sliding supports and semi-rigid joints determined using the Hölzer method
S3.3.	Scenario involving hinged-sliding supports and semi-rigid joints determined using the Hölzer method
S4.1.	Scenario involving rigid supports and semi-rigid joints determined using the component method
S4.2.	Scenario involving sliding supports and semi-rigid joints determined using the component method
S4.3.	Scenario involving hinged-sliding supports and semi-rigid joints determined using the component method
S5.1.	Scenario involving rigid supports and semi-rigid joints determined using the Heimeshoff & Köhler method
S5.2.	Scenario involving sliding supports and semi-rigid joints determined using the Heimeshoff & Köhler method
S5.3.	Scenario involving hinged-sliding supports and semi-rigid joints determined using the Heimeshoff & Köhler method
D.l.	Damage limitation
S.d.	Significant damage

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1 INTRODUCTION

Timber structures are a complex research topic, offering research opportunities in a wide array of domains. On the one hand, there is a significant amount of studies concerning contemporary timber structures, connected to the understanding of their structural behaviour and load-bearing capacity. At the same time, they are also focused on the development of new timber-based materials with improved mechanical properties and of aesthetically appealing joining materials and technologies.

On the other hand, there are also studies concerning existing/heritage timber structures, which bring forward a series of other problems. The complexity of this topic arises from the fact that the structural behaviour of heritage structures is also strongly connected to their state of conservation and the type of the traditionally crafted joints used to connect the structural elements.

If an assessment of a historic timber roof is made, it cannot be analysed as an independent structure but part of a coherent system, together with the building it belongs. Therefore, considering the numerous factors which are influencing the architecture of a building, it becomes clear that a roof structure cannot be assessed without considering the link to its surroundings, the connection to the architectural style of the building and ultimately acknowledge its effect on the structural behaviour of the building.

Moreover, the proper understanding and assessment of historic timber roof structures are also necessary since attics are valuable spaces usually placed in central positions in the cities, which can be reused, becoming highly attractive for investors. All these factors lead to a diversity of research topics and a high attractiveness of the study.

Respecting the ICOMOS principles [1,2] and the Venice Charter [3] which are encouraging a multidisciplinary assessment of heritage structures, different assessment methodologies suitable for historic timber roof structures have been developed in recent years, by various task groups of COST Actions or different researcher groups [4–6]. These methodologies were developed in order to offer clear guidelines for assessors but also ensure the safety of the built cultural heritage.

However, all these methodologies and procedures mainly focus on the structural features of roof structures, their on-site assessment [7,8], the analysis of the mechanical properties of the timber elements and their static behaviour, without looking at the roof structure as a part of the building and understanding its connection with everything that shapes and surrounds it and how its environment is affecting it.

Therefore, in order to be able to develop proper conservation strategies, a complex evaluation is needed, which is considering taking all the factors, which influenced the shape of the roof and the used structural typology. Special attention has to be given to aesthetic features and the relation with the surrounding urban area. At the same time, all the environmental factors which can affect their state of conservation have to be also taken into consideration, since they can affect the visual appearance of the structure and can ultimately lead to their failure. Noteworthy is also to consider the link to the building and acknowledge how these structures can influence the behaviour of the buildings during seismic events. Only after such a comprehensive analysis, the roof structure can be structurally evaluated and decisions and strengthening measures, if necessary, taken.

The study was performed on roof structures from Timisoara, a city from the western part of Romania. Compared to other European cities, where the interest for assessing and reusing timber roof structures is high, in Timisoara, the roof structures are still in their original shape, presenting only little alterations (Fig. 1.1). The primary reason is the low interest of the investors in using the additional space available in the attics, but also the more complicated owning rights of these spaces. They offer, therefore, an extensive database for the assessment of roof structures of West-European influence, starting with the 18th century up until the beginning of the 20th century, presenting the changes concerning structural design and joinery detailing. At the same time, due to the diversity of contexts in which the buildings were built, this database can help develop existing assessment methodologies and procedures, adding interdisciplinary and transdisciplinary features and ultimately offering a better insight on the value of historic timber roof structures.

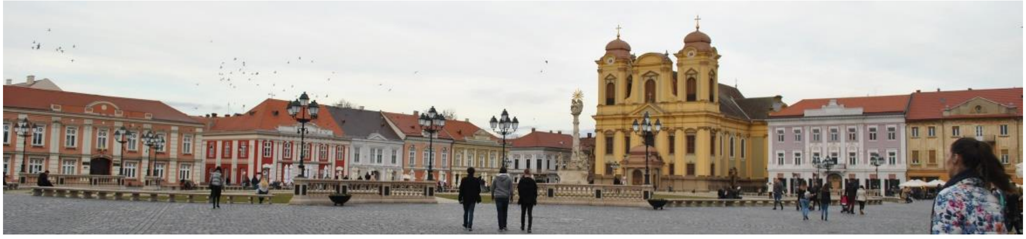


Fig. 1.1 Timisoara roofscape – completing the aesthetics of the Union square

1.1 Objectives

Considering the complexity of historic timber roof structures and the diversity of factors which ultimately influence their value, the main objective of the thesis is to increase the knowledge concerning historic timber roof structures. This is done, not only by looking at them as structural systems but as part of a more complex system where all the surrounding and composing elements are interlinked and are influencing each other from an aesthetic, symbolic but also a structural point of view.

Therefore, the scopes of the thesis are:

1. The analysis of current, international assessment methodologies and procedures in order to identify the main differences between them;
2. The identification of additional features which have to be taken into consideration when assessing the value and vulnerability of historic timber roof structures starting from the already developed methodologies and respecting the ICOMOS recommendations and principles;
3. Performing a desk and on-site survey of selected roof structures from Timisoara, from different periods and contexts;
4. Highlighting the importance of addressing the value of roof structures from different points of view:
    - 4.1. Identification of the role urban planning principles have in defining the roof shape;
    - 4.2. Identification of the link between architectural styles and roof shape while highlighting the importance of the roof in defining the aesthetics of a building;

    4.3. Highlighting the presence of various geometric ratios in defining both the exterior appearance of the roof, the position of various structural elements and the evolution of the used ratios in time;
    4.4. Identification of the way urban planning principles, architectural styles, symbolic and geometric ratios and sophisticated structural features are ultimately influencing the value and importance of a roof structure;
5. Understanding of the way current meteorological conditions and future climatic changes can affect the state of conservation of the roof structure elements and roof structures as a whole and identification of features which are influencing the response of the roof structure when subjected to extreme meteorological events like high wind velocities;
6. Calibration based on analysed experimental tests from the literature of a roof structure, in order to identify parameters which must be considered during linear finite element simulations:
    6.1. Analysis of full-scale laboratory tests and numerical simulations performed on historic timber roof structures;
    6.2. Analysis of various semi-rigid modelling methods suitable for traditionally crafted joints while highlighting the main differences between the analysed axial stiffness determination methods;
    6.3. Analysis of the effect of the material mechanical properties, the cross-section of the timber elements or cross-sections loss and joint axial stiffness;
7. Understanding of the role of timber roof structure types from Timisoara on the seismic behaviour of historic masonry structures, by comparing the different effects of the selected roof structures in terms of horizontal displacement, inter-story drift, damage level of the historic masonry structure, deformed shape of the building and internal forces
8. Understanding of the effect of the decay of the timber elements on the influence of the selected roof structures on the seismic behaviour of the considered historic masonry building by comparing the same parameters
9. Highlighting the importance of considering a multi-, inter- and transdisciplinary assessment when addressing historic timber roof structures and development of an improved assessment procedure which can be used to determine the general value and vulnerability of historic timber roof structures, considering the value of the roof structure from an urbanistic, architectural, symbolic and structural point of view and vulnerability caused by climate change and its decay which is also highlighting the effect of the assessed roof structure type on the seismic behaviour of the building:
    9.1. Development of a score assigned to each answer of the assessment methodology and formulas for the calculation of the value and vulnerability indexes;
    9.2. Development of an easy-to-use Excel form and mobile application which can be used to evaluate a roof structure on-site and automatically obtain all the results;
    9.3. Calibration of the developed assessment procedure based on a selection of analysed roof structures with significant urban, architectural, symbolic and structural value;
    9.4. Application of the developed assessment procedure on all the surveyed roof structures and analysis of the obtained results;
    9.5. Identification of the way the influence of each considered feature (urban planning, architecture, symbolism and structure) is changing over time in Timisoara.

1.2 Outline of the thesis

The thesis is organised in 6 chapters, considering its main objectives and approaching historic timber roof structures from two points of view which are later on interlinked at the end of the thesis. The study can, therefore, be divided into three main parts.

The first part is connected to an analysis of existing assessment methodologies suitable for historic timber structures meant to highlight the common approached topics, the main differences between them but also all the features which are not taken into consideration during the first phases of the assessment. The second part is focusing on the structural assessment of historic timber roofs. First, they are approached as an independent system with the main scope to identify parameters which are important to be considered during numerical simulations. Secondly, the roof structure is considered as part of the building, and the influence of the roof structure on the structural behaviour of the building during seismic events is analysed. Ultimately, the last part considers all the observations and conclusions drawn from the first two parts and represents an assessment methodology proposal which can be used to determine the value of a roof structure from multiple points of view and its vulnerability.

The first chapter presents a general overview of the thesis and highlights its main scope and topics.

In chapter two, a review of all relevant literature is presented, concerning norms, principles and studies performed in the domain of historic timber structures and roof structures. The need for a multidisciplinary approach is once again highlighted, and a transition towards the key features of roof structures in Timisoara is made. Subsequently, the chapter highlights all the features which are influencing the shape of roofs and ultimately affect the chosen roof structure type. It is presenting how urban planning principles, main architectural styles from the 18th to the 20th century, and the philosophy of the craftsman are influencing the shape of roofs and the roof structure. All these features are explained based on various analysed examples from Timisoara. Additionally, characteristic features of roof structures are also brought forward, like used joint types and main threats which can affect their state of conservation, highlighting the causes of decay and the risks of current and future climatic conditions.

The third chapter is focusing on the structural behaviour of historic roof structures and the parameters which have to be taken into consideration when performing finite element numerical simulations. This chapter starts with a review of studies performed concerning the structural behaviour of historic timber roof structures, analysing the conditions and results obtained during full-scale laboratory tests performed in Italy and Portugal.

At the same time, it is also highlighting all the parameters which have to be taken into consideration during the numerical simulations performed on these structures, by analysing the challenges of other numerical simulations and bringing forward the need to consider the axial stiffness of historic timber joints. Considering this, three different determination methods for the axial stiffness of the timber joints are presented in this chapter, by reviewing each calculation method and bringing forward the main differences between them.

Ultimately, based on a selected tested full-scale roof structure, a calibrated finite element model in the numerical simulation software SCIA Engineer is made, and calibrated input parameters proposed which can ensure a similar structure behaviour of the modelled roof structure as during the laboratory test.

Based on the conclusions from the third chapter, the fourth chapter is focusing on the effect of three particular roof structures types from Timisoara on the seismic behaviour of an 18th century historic masonry building. Considering the difficulties of performing full-scale laboratory tests encountered by other researchers due to the dimensions of historic roof structures and considering their significant width and height in Timisoara, the chapter is first highlighting why only numerical simulations were performed during the study.

Subsequently, the parameters used to define the numerical models (materials, loads and axial stiffness of the joint) are presented as well as the main features of local earthquakes. The results of the numerical simulations are presented for the hypothetical building without a roof and the same building with each chosen roof structure. The results are subsequently compared in order to highlight their different effects. The results which are analysed in this chapter are the out-of-plane horizontal displacement and the inter-story drift of each floor of the building, the deformed shape of the building, the damage level of the historic masonry walls and the out-of-plane internal forces (axial force, shear force and bending moment). Due to the rounded edges and a possibly decayed outer layer of the timber elements subsequently, a cross-section reduction of all the timber elements was considered, the numerical simulations once again performed, and the results analysed.

The fifth chapter is based on the observations from the previous chapters which highlight that roof structures, their context and the building they belong to are interlinked and that their value, can be influenced by their immediate context, the surrounding urban planning principles, architectural features or even symbolic factors. At the same time, it is also based on the presented threats which affect the state of conservation of historic timber roof structures and how their presence is influencing the seismic behaviour of a building.

The chapter is, therefore, presenting the proposed holistic roof structure assessment procedure, which is taking all the factors surrounding roof structures into account respecting in this way the ICOMOS principles. First, all the levels of the assessment are presented, each considered criterion brought forward, and the selection of possible choices and corresponding scores explained. Subsequently, the results and calculation methods are presented, the methodology offering information about the predominant value of the roof structure, its ideal and real value, decay index, climate change vulnerability, the effect on the seismic behaviour of the building based on the structural type and the general vulnerability of the roof structure.

In the last part of the chapter, the proposed methodology is validated based on selected roof structures which are representative for the four main categories of the assessment (urban, architectural, symbolic and structural value). Subsequently, it is applied to all the analysed roof structures from Timisoara and the obtained results compared. Ultimately, they are classified based on their value and vulnerability to observe patterns based on the period in which they were built and their location.

Finally, the sixth chapter is presenting a summary of the results, the conclusions of the thesis, main personal contributions and a complete presentation of the research dissemination in conference and journal papers and their citations. At the same time, the chapter also presents an outline of possible future studies related to the assessment of historic timber roof structures and their effect on the seismic behaviour of historic masonry buildings.

2 ASSESSMENT OF HISTORIC TIMBER ROOF STRUCTURES

The assessment of historic timber roof structures is a complex topic which needs to be approached from multiple angles in order to be able to gather all the necessary information about all the features which affected the shape and choice of roof structure type. Besides the fact that they have an essential structural value, representing the knowledge of the timber craftsmen, some structures have a significant aesthetic, archaeological and cultural value, due to their appearance and craftsmanship. Since the construction techniques are based on the traditional skills of the craftsmen, each structure becomes unique and special. At the same time, this type of multicriterial assessment is of great importance to properly preserve timber structures with all their valuable elements, both structural and aesthetic and find suitable intervention strategies.

2.1 State-of-the-art

Studies concerning the protection of historic timber structures acknowledge that in order to be able to develop comprehensive assessment guidelines, a holistic approach has to be taken into consideration which can combine data from various fields and therefore help in the decision-making process [9]. At the same time, they also acknowledge that the value of a timber structure is related to several factors like history, aesthetics, science, technology, anthropology or symbolism, features which are also important in correctly understanding the structure [10].

The need to use an interdisciplinary approach when addressing historic timber roof structure is highlighted by various other studies [4,11,12] which are also bringing forward the importance of first understanding the architecture of a timber structure and the principles which led to the choice of type and shape [13]. It is also highlighted that the assessment of historic timber structures is a rather complex problem, since timber structures evolved differently in various regions, influenced by the knowledge of the craftsman, but also due to the mechanical properties of the timber and its current state of conservation [8].

The assessment of these structures needs, therefore, to involve experts from various fields like engineers, architects or restorers. Still, different experience in working with historic timber structures can lead to various opinions about the current state of the assessed structure and ultimately to diverse intervention strategies. This is also bringing forward the lack of objectivity of current assessment methods.

Today, research topics in the domain of timber structures are diverse, from studies performed on historic structures to new ones and the development of new timber-based materials. Still, the studies and papers concerning historic timber roof structures are rather few when comparing them with other research areas [14]. Roof structures are, in current practice, only seldom thoroughly inspected, which is leading to a lack of complete information about their history and their structural behaviour. Ultimately this is causing a misunderstanding of their structural behaviour [15], hindering the development of suitable intervention strategies and ultimately to inadequate strengthening interventions or even complete replacement of the structures [16,17].

2.1.1 Standards, norms and principles

In order to increase the awareness of the complexity of heritage timber structures, various committees within the ICOMOS organization have been focusing on their assessment and the development of intervention principles suitable for structures with significant historical value.

In 1999, the "Principles for the preservation of historic timber structures" were adopted during the 12th General Assembly in Mexico by ICOMOS [1]. The main aim of the principles was to define the primary purpose of the preservation of historic timber structures with significant cultural value and propose recommendations for their inspection, monitoring, maintenance, repair and replacement which would lead to the preservation of the structures without altering their historical value. The document is highlighting the importance of respecting the 16th article of the Venice Charter [3], which encourages a complex assessment of the existing structure, from multiple points of view, comprehensive diagnosis and a critical analysis of the observations. Subsequently, the ICOMOS/ISCARSAH Charter and Guidelines were developed in 2003 [2], presenting principles and recommendations for the analysis and development of intervention strategies suitable for heritage structures. The charter is also highlighting the need to analyse these structures from a multidisciplinary point of view in order to be able to develop conservation and intervention strategies properly.

In this scope, various standards have been developed in recent years, meant to guide professionals in a preliminary evaluation of historic load-bearing structures and define principles for the interventions in these structures.

Therefore, in Italy, for example, two different standards were developed, offering for the first time clear principles and procedures for various professionals in assessing existing timber structures. The first was UNI 11119 "Cultural heritage. Wooden artefacts. Load-bearing structures. On-site inspections for the diagnosis of timber members" [18], focusing on the state of conservation of the timber bearing structures, their structural behaviour and defines methods and assessment procedures for the strength grading of the historic timber. UNI 111138 "Cultural heritage. Wooden artefacts. Load-bearing structures. Criteria for the preliminary evaluation, design and execution of works" [19], on the other hand, is instead focusing on the conservation and restoration of these structures, without offering precise intervention details. Both standards highlight the importance of a multidisciplinary assessment of historic timber structures, focusing on their structural features as well as their historical aspects and cultural and aesthetical characteristics.

Starting from the principles and guidelines adopted by ICOMOS, the ISO 13822:2010 standard [20] was developed in 2010 highlighting that the proper assessment of an existing structure is highly important and a complex task for professionals. Still, due to economic considerations and respect for the existing built environment, the international standard is stating that it is necessary to assess existing structures when a change of use is planned. In that situation, the reliability of the structure has to be checked in order to see if it can still bear de applied loads, if significant decay is visible, or damages appeared caused by accidental events or loads.

Subsequently, similar standards, although not only related to the assessment of existing timber structures were developed in the United States of America (ASCE

41-13) [21] or Switzerland (SIA 269) [22], including procedures suitable for the assessment of historic timber structures and decision-making guides.

Despite the significant amount of timber heritage buildings in Romania, reaching from historic timber churches and vernacular timber residential architecture to sophisticated timber roof structures, there is currently no specific standard developed for their protection, which could guide the preliminary evaluation of these structures and define intervention principles.

2.1.2 Assessment of historic timber roof structures

A general overview of papers written in this research area is showing that most papers were written by architects focusing on the history, evolution and typology of roofs and roof structures. Structural engineers, on the other hand, are instead focusing on their structural behaviour as a whole, the assessment of the timber mechanical properties and the behaviour of the timber joints.

At the same time, it could be observed that the first studies are mainly related to the typological assessment of the evolution of timber roof structures and the knowledge of the craftsman used to develop these structures. These studies also focus on the geometric principles used to define timber roof structures and the philosophy behind them. More recent studies analyse performed laboratory tests and developed numerical simulations of timber joints and rarely of whole timber trusses.

2.1.2.1 Uni-criterial assessment - History of roof structures related studies

The first studies concerning the evolution of roof structures were performed by architect Ostendorf in 1908 [23], who assessed more than 1000 structures from different regions in Europe, trying to identify how they evolved and their main structural features. His studies started with ancient roof structures, from the first rafter or purlin roofs and followed their changes in western, central and northern Europe. At the same time, he was trying to intuitively understand the struggles which craftsman went through while developing suitable structural solutions for shed or hip roofs. Ultimately, he was addressing tower roof structures and their various solutions related to different shapes. This research is one of the most comprehensive researches concerning common roof structure types and their evolution in Europe and is the base for many contemporary researches, being cited by studies concerning roof structures or timber joints even today.

Starting from the studies of Ostendorf and various other studies performed in the 19th century, concerning the development of different timber structures in Germany [24,25], the evolution of roof structures in Europe based on the geometry of the roof, used structural type, main used structural elements was also analysed [26]. At the same time, their structural behaviour and the load transfer through the timber elements towards the walls is also highlighted. Subsequently, main threats are also brought forward, and a comprehensive decay assessment methodology meant to help identify the safety of the timber structure developed.

Also, at European level, a study has been performed on mansard roofs in the 18th century [27]. This study is also somewhat focused on the evolution and expansion of a specific roof structure type during the Baroque period while also highlighting the changes of the techniques used by the carpenters connected to the political and religious context of the period. At the same time, studies have also been performed concerning roof structure types in central Europe, built between 1500 and 1700, highlighting their main structural features [28].

Besides studies performed on a European level, studies concerning the evolution of roof structures from a particular country, region or city have also been performed. Therefore, an extensive research has been performed in Romania concerning historic timber roof structures from the Transylvania (central) region [29–31]. Due to the diversity of roof structure types in this region, from Romanesque to Eclectic, the study is mainly aiming to create an inventory of roof structures, highlighting their geometry, characteristic features, while also describing the decays and prior interventions made. Subsequently, the study was extended, including also the structural behaviour of some selected roof structures, meant to bring forward if current safety standards can be used to evaluate historic roof structures [32].

Studies have also been performed in Scotland, where the roof structures of more than 1500 buildings were assessed [16,33]. In this study not only the evolution of the structural types was taken into consideration, but also the shape and geometry of the roof, the most common structural arrangements, the connections with the wall, the type of timber dressing, used timber joints, timber species, roofing material and signs of the carpenters. This survey is therefore complex since it addresses construction techniques but also architectural and symbolic features like roof shape, carpenter marks and roofing materials.

In Germany, for example, three different approaches concerning the assessment of historic timber roof structures were identified. The Institute of Mathematics and Construction Informatics in Munich conducted a study which was mainly focused on understanding the structural behaviour of historic timber roof structure from the 17th and 18th century [34]. After the geometric survey, the assessment of the material properties and the state of conservation of the structures, the axial stiffness of the timber joints was calculated, and complex numerical simulations were made using the Abaqus software, in order to understand the load transfer between the composing structural elements. Although the topic of the study is complex and advances have been made regarding the structural behaviour of historic timber roof structures, the research is addressing only the structure without looking at it in a multidisciplinary way. Remarkable are also the studies performed in the Bavarian region, focusing on Baroque roof structures, their evolution and constructive principles related to the function of the building and span of the roof structure, but also on the used load-bearing elements, clearly explaining the structural role of each element [35,36].

The study performed at the University of Stuttgart is approaching the topic in a completely different way [37]. Twenty-two different roof structures from the southern part of Germany, in a good state of conservation and presenting different structural types, were assessed. The main scope of the research was to highlight the diversity of structural types from this region and understand their main structural features and construction stages by building a scale model. Although the study was performed together with architecture students, it is only focusing on the structural features of the roofs. Still, by building the scale models, the team brought the aesthetics of historic roof structures in the attention of the residents and the models are now exhibited in local museums.

The study, performed at the University of Bamberg, on the other hand, is approaching the topic more subtly by highlighting the beauty of the “hidden treasure” of heritage buildings, their roof structure. It presents the evolution of timber structures from the 12th to the 19th century, by analysing the used construction techniques and used roof structures types. Ultimately, an exhibition of scale models was made, and an informative brochure was written containing information about their main features. Despite starting from the idea of highlighting the hidden structures of heritage buildings, the link between roof and building is not addressed in the brochure.

All these studies highlight the evolution and main features of roof structures from various regions, bringing forward their shape and structural characteristics. Still, no study was identified, which is acknowledging the role of the roof structure in its broader context and looking further than the structure itself.

2.1.2.2 Multicriterial assessment

2.1.2.2.1 Multidisciplinary assessment

Respecting the ICOMOS principles encouraging a multidisciplinary assessment of heritage structures, different assessment methodologies suitable for historic timber roof structures have been developed in recent years, by various task groups of COST Actions or different researcher groups, in order to ensure the safety of the built timber heritage. Nevertheless, all these methodologies and procedures mainly focus on the roof structure, its on-site assessment [7,8], the analysis of the mechanical properties of the timber elements [9,38] and its static behaviour [39–41], without looking at the roof structure as a part of the building and understanding its connection with everything that shapes and surrounds it.

2.1.2.2.1.1 Assessment methodologies

During the COST action IE0601 [4,7] concerning the Conservation of cultural heritage, the “Assessment of timber structures” task group developed a set of principles and approaches which would lead to an elaborate assessment of historic timber structures with significant value. The group acknowledges that a complete survey and evaluation of structures with historical importance is necessary in order to develop proper repair methods suitable for them. Therefore, a more precise and clear methodology must be used in order to justify future interventions.

The methodology was developed from micro to macro assessment, providing a set of criteria which should be considered during the assessment. Therefore, the survey starts with a preliminary analysis of the structure (Fig. 2.1), first with a desk survey, leading to a visual survey and ultimately to a measured one. Only after this, the structure is analysed taking the internal forces and the overall load transfer into consideration. The first part of the assessment ends in a preliminary report which states what else should be more thoroughly assessed and specifying what must be done in the future.

The second part of the assessment goes further into detail, assessing the state of conservation of the timber elements, identifying their mechanical properties mainly by non-destructive methods and offering a complete diagnostic report highlighting what measures should be taken. A detailed presentation of possible repairs ends this part of the survey.

The last part addresses the timber joints trying to understand how they work and how loads are transferred in the roof structure. The geometry, state of conservation, and used materials have to be also determined.

Throughout the guidelines, the task group is offering extensive information concerning how to assess these types of historical timber structures, presenting how to conduct a correct geomatic survey, how to use non-destructive methods to determine the mechanical properties of the timber, how to identify the decays of the structural elements, but also what problems should be evaluated during the assessment.

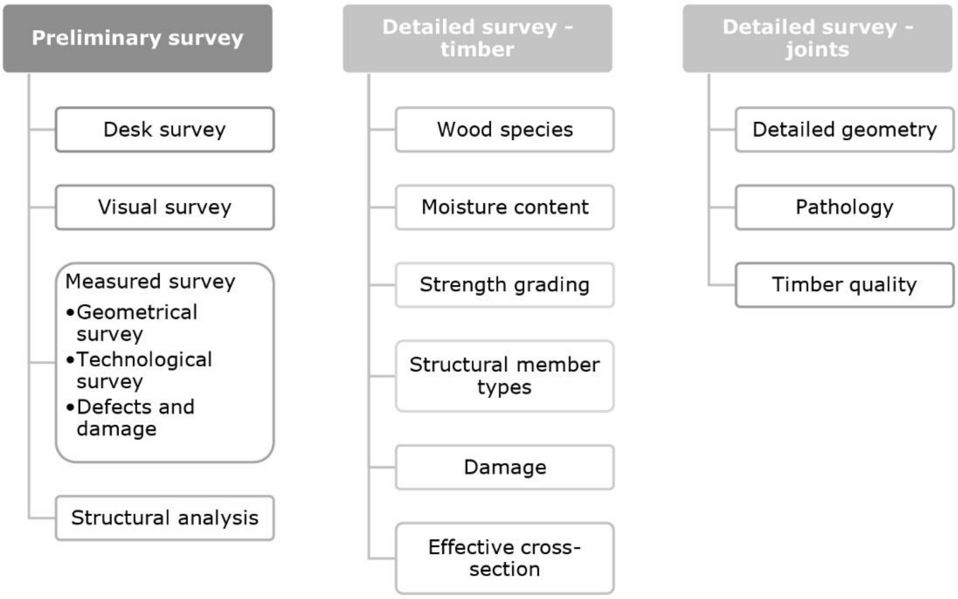


Fig. 2.1 Assessment methodology criteria developed during the COST Action IE0601

Within the COST action E55 [5,42], a task group for the assessment of timber structures was made, with the scope to develop a comprehensive assessment methodology which could evaluate historic timber structures from every point of view. The task group highlights the fact that existing timber structures need complex guidelines for a complete inspection in order to maintain them and use them in the future.

The assessment guideline is also organized after a series of criteria into three phases (Fig. 2.2). The first phase represents a preliminary inspection of the timber structure through a site visit. It addresses a detailed photographic inspection of the structure, the determination of the moisture content and the preliminary mapping of the damages. The second phase represents a more comprehensive inspection of the structure by using non-destructive and destructive tests in order to determine the mechanical properties of the timber element and their state of conservation. Ultimately the third phase represents laboratory testing, in order to determine the limit states of the timber element, by using load test and microscopic /macroscopic tests on specimens from the structures.

Every criterion is organized into four chapters, which are meant to guide the person interested in assessing the structure: principles, application, evaluation end literature. In this way, precise specifications are offered on how to assess the structure without leaving any doubts and offering clear examples and pictures in order to make everything more transparent. In the end, the guideline also offers information (criteria and thresholds) to help make decisions and develop conservation strategies.

Compared to the first guideline, which was mainly focused on the preliminary visual survey of historic timber structures, this guideline offers a complete collection of information about the timber structure but is also more time-consuming.

As the authors also highlight, despite its complexity, the assessment procedure is not ensuring a reliable tool in the evaluation of existing timber structures, and it becomes clear, that in order to accurately assess them, it is necessary to combine different methods and obtain a general overview [42] of the structure and its behaviour.

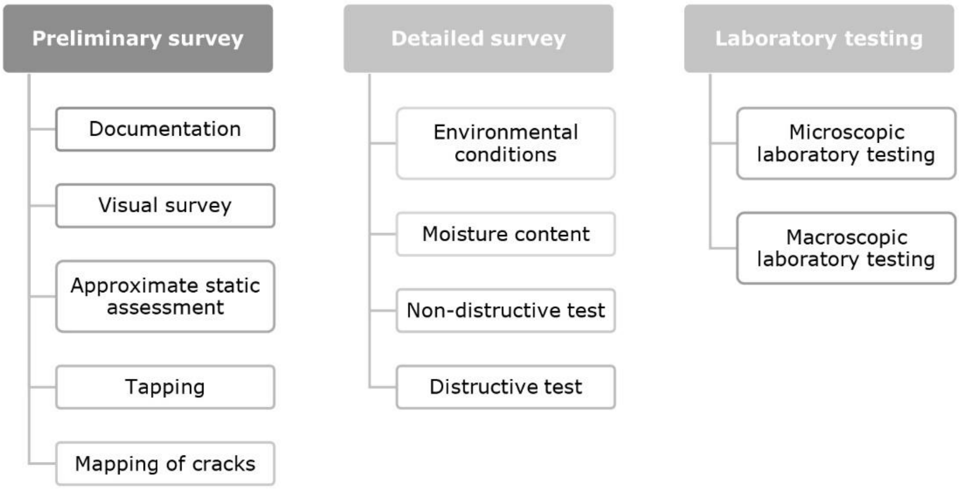


Fig. 2.2 Assessment methodology criteria developed during the COST Action E55

The working group of the COST Action FP1101 [6,43,44] "Assessment of timber Structures", has developed an inspection procedure, organized in a tree-like structure meant to offer information about the structure, its materials and state of conservation.

The procedure was organized on different scales and can also be divided into three sections (Fig. 2.3). For each section, the assessor has to respond to a list of questions, either selecting an answer from a provided list or only by filling out the answer.

The first section of the form includes information about the building and its context, highlighting the age of the building, its importance, height, number of floors and used structure. At the same time, it also considers environmental factors, highlighting their influence on the state of conservation of the timber structure. The second section starts assessing the timber roof as a whole, addressing its shape, span, structural type and past interventions. Subsequently, the third section of the procedure, focuses on the components of the timber structure, first on the timber elements and then on the connections and other details, evaluating their damages and possible failures.

Therefore, this approach is offering a structured and hierarchical way to objectively assess historic timber roof structures, only by a visual survey.

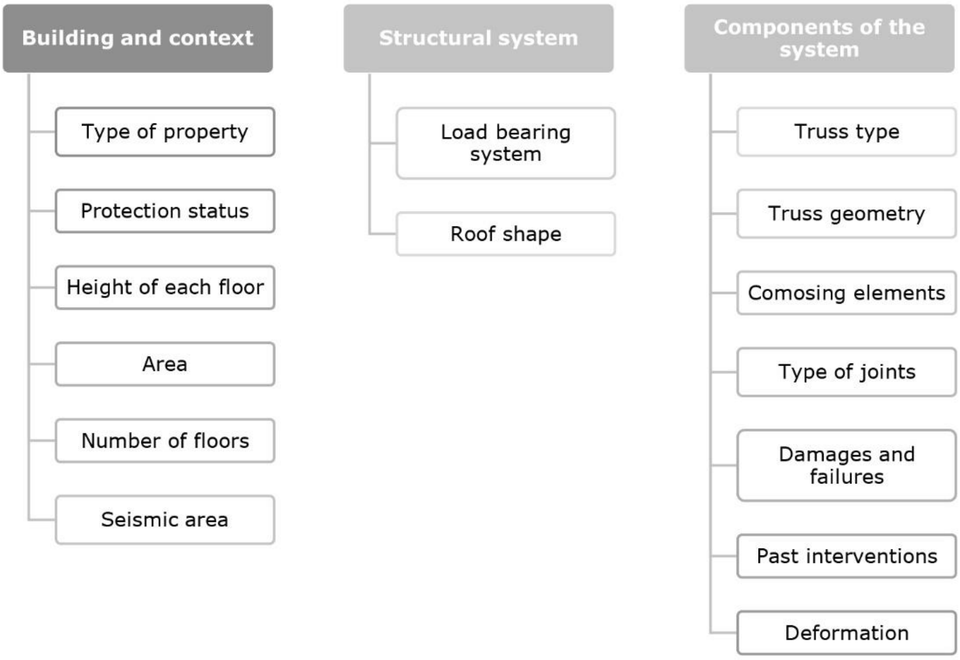


Fig. 2.3 Assessment methodology criteria developed during the COST Action FP1101

Subsequently starting from the outcomes of the COST action PF1101 [8] it was concluded that in order to supervise the assessed information better, an assessment methodology should be organized into four assessment levels:

1. the first one related to the micro-climatic and macro-climatic conditions which could affect the mechanical properties of the timber on the long term.
2. the second level is related to the structural system and its composing elements aiming to identify the main structural type, understand the behaviour of the load-bearing structure and the importance of each structural element. At this level also the type of carpentry joints is taken into consideration since their type and state of conservation can significantly influence the behaviour of the structure.
3. the third assessment level is connected to the various changes which the structure has undergone. For this level, human-made changes are taken into consideration since they can alter the behaviour of the roof structure and compromise its safety but also the decay caused by climatic conditions or exceptional loads like earthquakes or high wind velocities.
4. the last assessment level is focusing on the mechanical properties of the timber elements based on visual grading or destructive and non-destructive measurement.

2.1.2.2.1.2 Assessment templates

Due to the complexity of the assessment methodologies, in order to simplify the evaluation of historic timber structures, different assessment procedures and templates have been developed.

The observations made during the COST action FP1101 regarding the assessment of historic timber structures led to the development of an assessment template (Fig. 2.4) which could be used to assess the structure as a whole, down to its every detail from multiple points of view [45,46]. Therefore, by using this tool, the geometry of the roof and its configuration is taken into consideration, the roof structure type, its errors from a structural point of view and missing elements, the mechanical properties of the used timber and its decay and the state of conservation of the timber joints can be assessed. Besides these features strongly related to the roof structure, the template is also considering additional factors like the environment of the structure or alterations made.

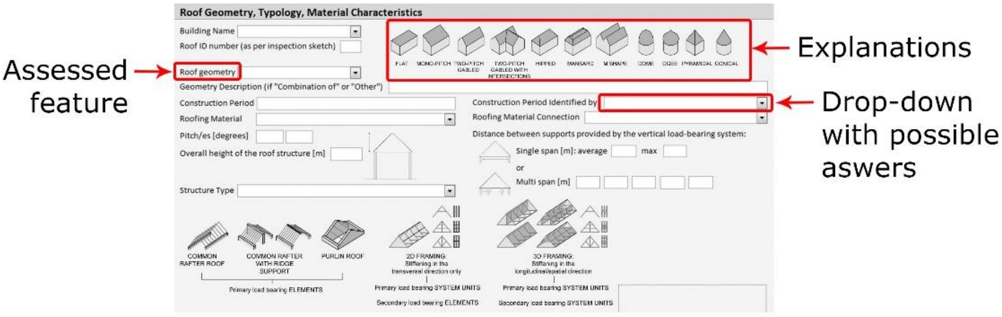
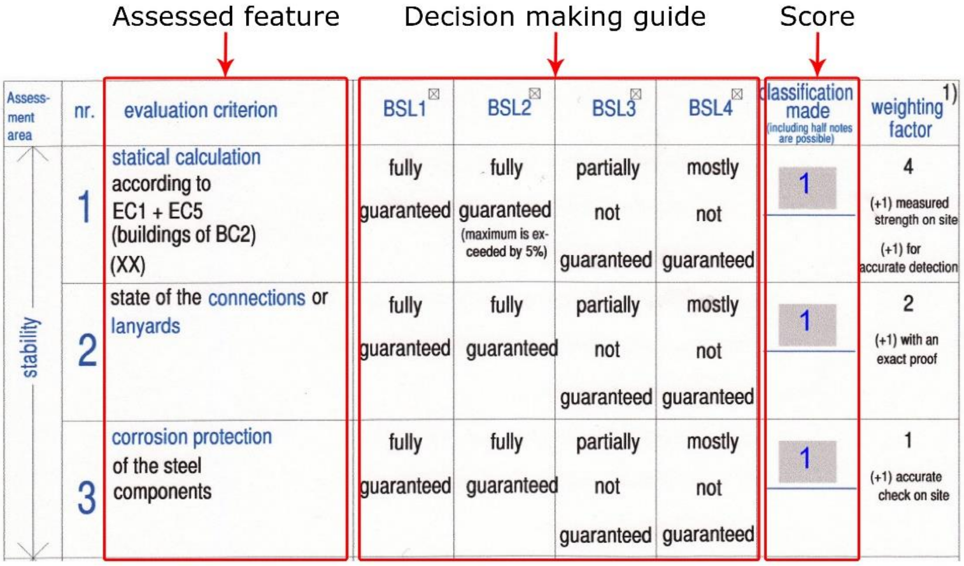


Fig. 2.4 Fragment of the assessment template developed during the COST Action FP1101 [46]

A similar procedure was also developed at TU Eindhoven in 2011 [47,48], which can be used for all types of timber structures, not only roof structures (Fig. 2.5). Due to their complexity and the need to approach timber structures from different perspectives, the procedure is involving architects, structural engineers, physicists, biologists, chemists, historians and various other experts for the investigations. The procedure is a checklist, divided into six chapters:

1. General information about the building (location, age, style) and scope of the assessment;
2. Detail survey of the structure - geometric survey, structural components, structural behaviour;
3. State of conservation of the timber elements and strength classes according to EN 335;
4. Historic assessment and characteristic structural elements and details
5. Load bearing structure analysis – assessment of vertical and horizontal structural elements of the roof structure, joints and applied loads.
6. Condition of the timber elements – visual assessment and NDT test results

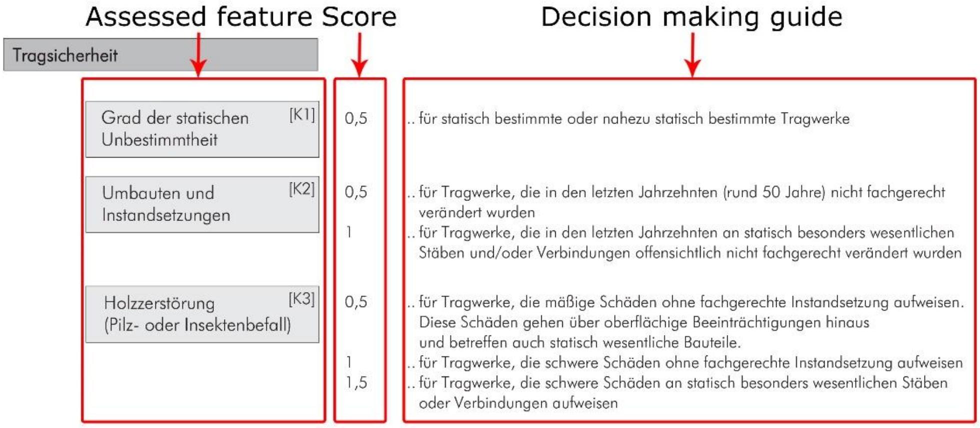
The final score is determined based on the answers provided at the 6th level of the assessment and presents the current state of the timber structure. The final score states if the structure is in a very good/good (with damages covering up to 5% of the structure), fair (with damages covering up to 25% of the structure), poor (with damages covering up to 50% of the structure) or deficient (with significant damages) state of conservation. This assessment procedure is complex, combining expertise from different professionals, but is still not considering any of this information provided in the final grading of the vulnerability of the structure.



Assess-ment area	nr.	evaluation criterion	BSL1☒	BSL2☒	BSL3☒	BSL4☒	classification made (including half notes are possible)	weighting factor 1)
stability	1	statical calculation according to EC1 + EC5 (buildings of BC2) (XX)	fully guaranteed	fully guaranteed (maximum is exceeded by 5%)	partially not guaranteed	mostly not guaranteed	1	4 (+1) measured strength on site (+1) for accurate detection
	2	state of the connections or lanyards	fully guaranteed	fully guaranteed	partially not guaranteed	mostly not guaranteed	1	2 (+1) with an exact proof
	3	corrosion protection of the steel components	fully guaranteed	fully guaranteed	partially not guaranteed	mostly not guaranteed	1	1 (+1) accurate check on site

Fig. 2.5 Fragment of the assessment template developed by Michael Abels [47]

Meisel [14] is also proposing an assessment template suitable for assessing the state of conservation of a historic timber roof structure, focusing on the visual assessment of the layout of the roof structure, its decay and the causes of the decay (Fig. 2.6). The form is organised in a tree-like structure, offering a list of criteria and possible answers organised by the severity of the decay. A score is assigned to each answer from 0.5 representing insignificant decay up to 1 or 1.5 representing significant damages. 0 points mean no visible decay.



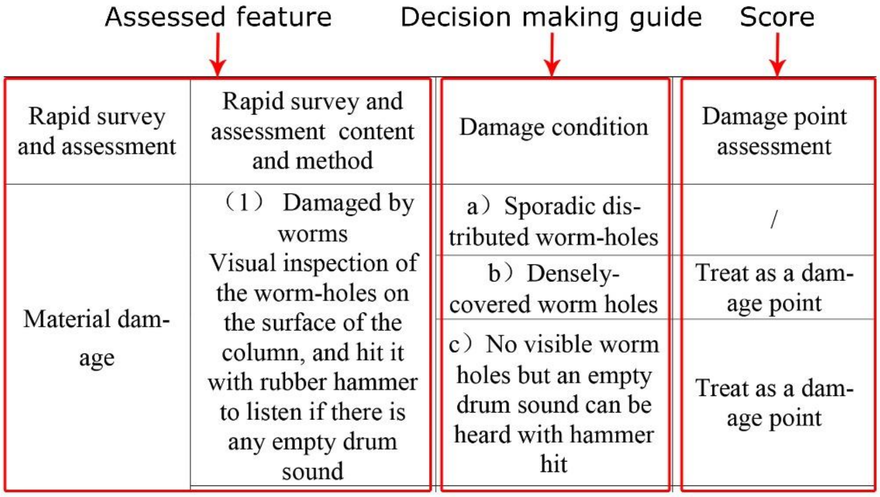
Tragsicherheit

Grad der statischen Unbestimmtheit [K1]	0,5	.. für statisch bestimmte oder nahezu statisch bestimmte Tragwerke
Umbauten und Instandsetzungen [K2]	0,5	.. für Tragwerke, die in den letzten Jahrzehnten (rund 50 Jahre) nicht fachgerecht verändert wurden
	1	.. für Tragwerke, die in den letzten Jahrzehnten an statisch besonders wesentlichen Stäben und/oder Verbindungen offensichtlich nicht fachgerecht verändert wurden
Holzzerstörung (Pilz- oder Insektenbefall) [K3]	0,5	.. für Tragwerke, die mäßige Schäden ohne fachgerechte Instandsetzung aufweisen. Diese Schäden gehen über oberflächige Beeinträchtigungen hinaus und betreffen auch statisch wesentliche Bauteile.
	1	.. für Tragwerke, die schwere Schäden ohne fachgerechte Instandsetzung aufweisen
	1,5	.. für Tragwerke, die schwere Schäden an statisch besonders wesentlichen Stäben oder Verbindungen aufweisen

Fig. 2.6 Fragment of the assessment template developed by Andreas Meisel [14]

Based on the obtained sum, the form is offering an intervention recommendation, stating if the roof structure is in a good state of conservation and no immediate strengthening interventions are necessary or if they are necessary in the next five year, following months or immediately. The form was applied on 100 roof structures from Graz from the 13th up to the 19th century and calibrated based on the observations made.

An equally complex assessment procedure has also been developed at the Beijing Jiaotong tie University [49], developed for Chinese heritage timber structures (Fig. 2.7). The procedure is based on the on-site assessment of the structure, focusing on its state of conservation. Therefore, it includes a geometric survey, the identification of damages and a series of non-destructive tests performed on damaged elements. A list of elements and features which have to be assessed is provided together with a list of possible answers to choose from, depending on the severity of the damage or cause of the decay. Based on the answers the procedure is grading the structure from A to D, providing a result stating if the structure is safe (grade A) or dangerous and measures should be taken locally (grade B), globally (grade C) or immediately (Grade D).



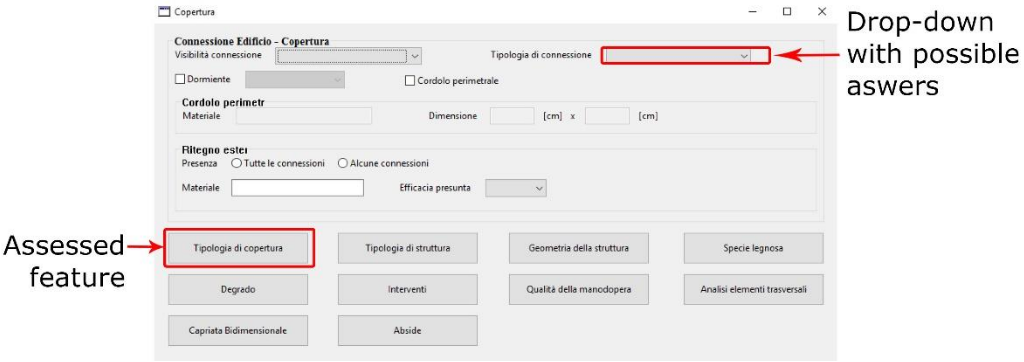
Rapid survey and assessment	Rapid survey and assessment content and method	Damage condition	Damage point assessment
Material damage	（1） Damaged by worms Visual inspection of the worm-holes on the surface of the column, and hit it with rubber hammer to listen if there is any empty drum sound	a）Sporadic distributed worm-holes	/
		b）Densely-covered worm holes	Treat as a damage point
		c）No visible worm holes but an empty drum sound can be heard with hammer hit	Treat as a damage point

Fig. 2.7 Fragment of the assessment template developed at the Beijing Jiaotong University [49]

Despite their complexity, both these procedures are only focusing on the safety of the structure and the decay of the structural elements and are not considering the value of the heritage building/roof structure as a whole and in relation to its context, which could affect the resulted vulnerability.

At the same time, an assessment template has been developed, which can be used to determine the seismic vulnerability of a historic roof structure [50–52]. The procedure is starting from the fact that roof structures are significantly influencing the seismic behaviour of historic buildings [53–57] and a set of criteria has to be defined, based on a preliminary inspection of the roof structure. The procedure is similar to the ones developed to determine the seismic vulnerability of heritage structures, like the one developed by Benedetti and Petrini in 1984 [58] or more recent ones [59–66].

It is defining a set of features which influence the seismic behaviour of the building like truss type, connection to the walls, state of conservation of the carpentry joints, the cross-section of the timber elements and state of conservation of the roof structure (Fig. 2.8). Each feature is subsequently divided into a set of assessed criteria, and a list of possible answers and corresponding vulnerability level of each answer is provided. The level of vulnerability was defined, in this case from A to D, A representing low vulnerability level and D high. Additionally, in order to be able to evaluate the vulnerability of each feature objectively, the procedure is also offering a guideline. Ultimately, the global vulnerability is determined based on the vulnerability of each assessed feature.



Drop-down with possible aswers

Assessed feature

Fig. 2.8 Fragment of the seismic vulnerability assessment template [67]

These assessment procedures (Table 2-1) are highlighting that due to their complexity, historic timber structures need an organised framework which is clearly defining the steps which need to be followed and is bringing forward features which need to be addressed during the assessment. By using forms with and without scores or checklists, the assessors can objectively evaluate the main features of the structures, their detail and state of conservation.

Still, as previously observed, these methodologies, despite being multi-criterial are focusing on the structure as an individual system without looking at it as part of the building and acknowledging the connection, both visually/aesthetically and structurally between all the composing elements of the building.

Table 2-1 Assessment procedures analysis

Developer	Structure type	Assessed feature	Type of form	Scope
FP1101 working-group	Roof structure	Multi-criterial assessment	Inspection form	General assessment of the roof structure
TU Eindhoven	Timber structures	Multi-criterial assessment	Checklist	Vulnerability of the structure
Andreas Meisel	Roof structure	State of conservation	Inspection form with scores	Strengthening intervention planning
Parisi et. al	Roof structure	Multi-criterial assessment	Inspection form with scores	Seismic vulnerability
Beijing Jiaotong University	Chinese timber structures	State of conservation	Inspection form with scores	Vulnerability and strengthening intervention planning

2.1.2.2.2 Transdisciplinary assessment

Besides trying to address the topic of historic timber structures from various professionals involved in the assessment of heritage structures, there are also studies which try to look further and include features which are from other domains, like philosophy or cosmogony. According to Basarab Nicolaescu [68–71], a proper analysis of a topic can only be made by considering the main features of the system but also its link to others. In this way, the transdisciplinary assessment is looking further than the multidisciplinary and interdisciplinary methodological approaches and is considering the assessed topic as part of a complex interlinked system.

Matila Ghyka [72] is highlighting in his studies that geometric principles and their philosophical meanings are defining art and architecture and their aesthetics since the antiquity. According to Lawlor [73], the use of geometric principles is the essential tool of philosophical language, a link between the metaphysical and the physical dimension. By using ratios and geometric principles, people were trying to recreate the cosmic order and create a symbolic link between the microcosm, comprising the building and its surrounding, and the macrocosm (Fig. 2.9a,b) [74], therefore becoming a symbol of cosmic order.

These principles can also be observed in traditional Chinese architecture where a link between architecture, nature, philosophy and cosmic order [75] is present. At the same time, the importance of the principles of the architectural style, ratios, and sacred pattern was highlighted in the 12th century Chinese building standards handbook (Yingzao fashi), presenting architectural and structural principles inspired from nature [76], like flowers or branches (Fig. 2.9c). The handbook is describing the relation between the human factor and the technical principles used to shape the defined style [77]. Recent studies performed for the 2014 International Architecture Exhibition in Venice [78] also brought forward that the used bracketing system is presenting a high symbolic value, being linked to the societal status of the person inhabiting the building.

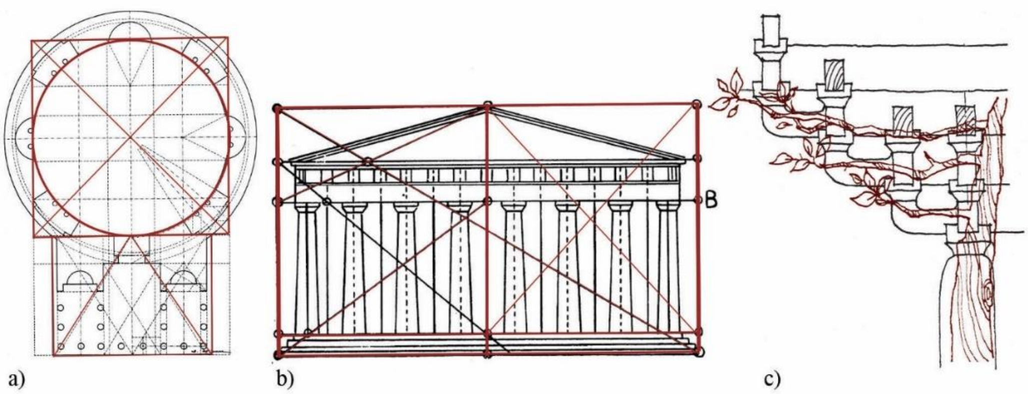


Fig. 2.9 Ratios defining architecture [79]

In the European context, studies performed on buildings from different periods brought forward that complex geometric principles were used to define architecture. Notable, are the golden ratios used to define the main elements of the Parthenon [80], the perfect geometric shapes used in the Pantheon, like circles, squares and triangles, the complex dynamic ratios used to define the patterns of

Islamic architecture [81], or Baroque architecture [82] up until the complexity of ratios used for the dome of Milano [72] or Le Corbusiers architecture [83,84].

Similar studies have also been performed concerning roof structures. The first acknowledgement of the role of geometric principles in defining roof structures is mentioned by architect Schübler in 1782 [85] while trying to define the geometry and ratios used by the craftsman in Italian, French and German roofs and roof structures. The study is highlighting the link between the height of the building and the height of the roof and the position of key structural elements and timber joints based on geometric principles, being the first complex assessment of historic timber structures (Fig. 2.10).

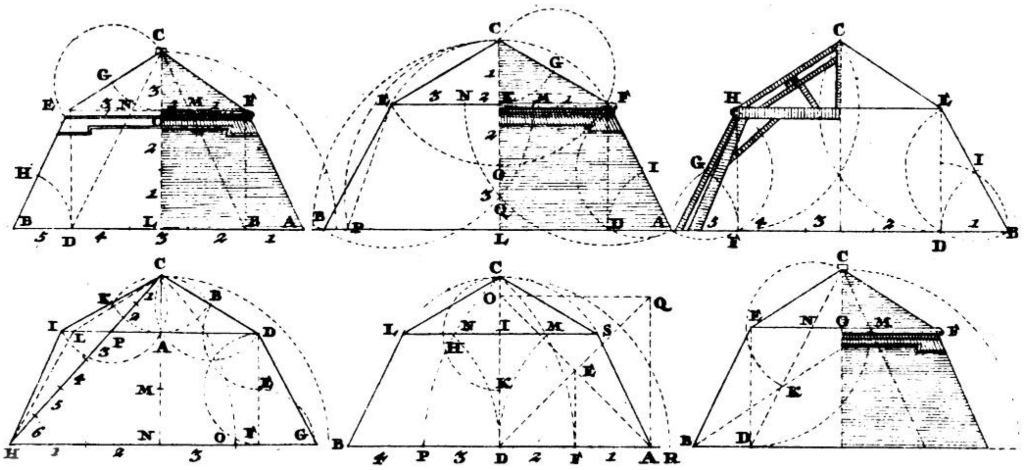


Fig. 2.10 Geometric analysis of the roof shape [85]

In recent years the interest for the symbolic analysis of historic timber roof structures is growing since researcher are trying to understand the way of thinking of the craftsman. Extensive studies have therefore been performed on gothic roof structures from the Czech Republic [39–41,86–88] addressing both the geometric analysis of the roof structures and the used proportions but also performing a static analysis of the assessed roof structure. The multidisciplinary research team composed of engineers and architectural engineers is proposing for the first time a transdisciplinary assessment of roof structures which is not only focusing on their structural behaviour but is also trying to understand the geometry and philosophy behind them.

Similar studies have also been performed in Romania, this time focusing on the evolution of the used ratios connected to the history of architecture, the changes of urban planning principles and evolution of timber roof structures [79,89,90]. In this case, roof structures are only addressed based on their type and used structural elements without actually analysing their structural behaviour and mainly focusing on their role in defining the aesthetics of the building and surrounding urban space.

2.2 New assessment criteria for historic roof structures

Compared to many European towns where the importance of historic roof structures is not fully appreciated, and they are substituted by contemporary structures, the city of Timisoara is still offering a great variety of roof structure types, in a good state of conservation, that can bring forward new information regarding their evolution and improve assessment methods.

After the Austrian Empire conquered the city in the 18th century, a new Vauban like fortress was built and a new city was shaped. The buildings were placed inside the fortress along a rectangular grid of streets, having a continuous façade.

Throughout the historic area of the city, the same type of roof structure was used with only insignificant differences, mainly caused by the position of the building, at the intersection between two streets or in the frontage. Only gable roofs were used in this area, with clear connections, for corner buildings.

According to the classifications of continental roof structures [30,31], the ones built in this period are Baroque, presenting all the characteristic structural features: visible differences between main and secondary frames and a sophisticated Baroque strutting device composed of a straining beam and inner rafters, also connected by a counterbrace. The structure is a purlin roof, with two different layers of structural elements, connected between each other: the exterior one, composed of rafters and collar beam, comprising the outer layer and the interior one containing the strutting device. Both main and secondary trusses have a tie-beam in the inferior part and are connected by the purlins but also by passing-braces placed in the plane of the rafters.

Buildings built at the beginning of the 19th century, start to present different architectural features and the roof structures begin to change. The roofs were no longer exclusively gable roofs, but also hip roofs were used for some buildings. This is why these roof structures are presenting more simplified solutions for the central part of the roof but complex systems to solve the hip ends.

Regarding the used structural type, the central part of the roofs often presents transitional types between the angle braced rafter roofs and the queen-post roof structures, the angle-braces being replaced by inclined posts. The passing-braces are no longer placed near the rafters and are connected to the posts and rafters by an additional horizontal beam. In this case, also a clear difference between the main and secondary trusses can be observed, the secondary ones being only composed of rafters. Still, all the frames are connected by purlins.

The 19th century marks the appearance of the queen-post roof structure in Timisoara, with all its characteristic features. Roof structures in this period used to be simple purlin roofs, with principal and secondary trusses with a clear difference between each other. Main trusses have an interior hanging device, composed of posts and straining beam, additionally connected between each other by a passing-brace and exterior rafters connected to the tie-beam, while the secondary frames only use rafters. Additionally, in the top part of the rafters, collar ties were used to connect them better.

Later on, at the end of the 19th century, due to the significant development of the technology, but also due to new architectural styles, roof structures evolve and present a wide array of solutions, strongly influenced by the architectural requirement. This results in adaptations of the queen-post roof structure, which are no longer symmetrical but use different solutions towards the street, where walls had to be higher than towards the inner courtyard, where the appearance of the roof is not so important. At the same time, a first attempt to combine queen and king-post roof structures was also observed, due to the increasing height of the roofs.

The 20th century in Timisoara marked a period in which the city suffered significant changes, and new build areas appeared. In the new squares and along the new main streets of the city, buildings had to be imposing and mark the contour of these public spaces. This led to monumental buildings but also to a clear interest in highlighting the importance of the roofs. Therefore, in this period, roofs are rather high, completing the building and additionally marking the importance of the building in the city.

In order to satisfy the required shape of the roof, roof structures had to be adapted. First, the great height of the roof had to be solved, leading to a mix of historic roof structure types, placed one over the other. Queen post structures were most commonly used in the inferior part of the roof, sometimes even with an intermediate collar beam, while the upmost part was most of the time a king post structure.

Secondly, buildings were most of the time L or U shaped, which led to the need to find suitable solutions to connect hip roofs to gable roofs, shed roofs to hip roofs or even sometimes connect two hip roofs by a semi-circular structure. Therefore, besides the already complex solutions used to solve the hip roof, additional timber elements were necessary to properly connect all the areas of the roof and make the structure work as a whole. Semi-circular connections, on the other hand, proved out to be an interpretation of the solutions developed for roof structures placed over towers or church altars.

These roof structures do not bring any real improvement or development of the used structural types. Still, they present a definite improvement of the used technology, the cross-section of the timber elements being more reduced. At the same time, they present the most significant advances concerning connections between various roof structure types and shapes, which are sophisticated, creative and have an essential aesthetical value, going far beyond a simple structural solution.

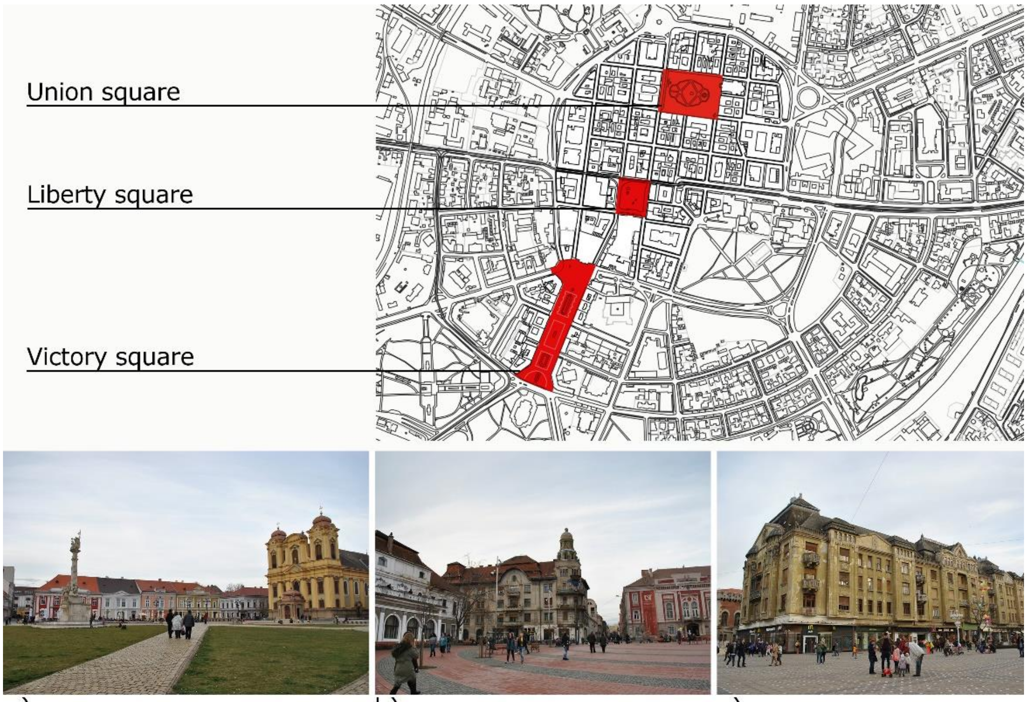
2.2.1 Value of historic timber roof structures

Starting from the assessment procedures and guidelines developed in these COST actions, it got clear that there is still place for improvement, and that specific criteria related to the aesthetical and architectural value of the roof structures are not adequately taken into consideration.

Therefore, in order to be able to conserve historic roof structures properly, a complex evaluation is needed. The assessment should consider all the factors that influenced the shape of the roof and used structural typology while also considering its aesthetics and the relation with the surrounding urban area. Only after such an analysis, the roof structure can be structurally evaluated and strengthening measures, if necessary, can be taken.

2.2.1.1 Urban value of historic roof structures

Analysing roof structures, from different periods of the history of Timisoara, urban planning principles and urban context are proving out to have a significant influence on how the roof was shaped and how important it in the surrounding environment is (Fig. 2.11).



a) b) c)

Fig. 2.11 Roof structures defining urban space in the main squares of the city a) Union square, b) Liberty square, c) Victory square

When the new city was developed in the 18th century inside the Vauban fortress, a rectangular grid of streets was planned, connecting the three main squares and the entrances in the fortress. Despite presenting similar shapes and aesthetics, two different perceptions of roofs in this period were observed. The roofs of the buildings placed along the narrow streets could not be perceived from the street level and had a purely functional purpose, protecting the building from environmental factors (Fig. 2.12). The roofs placed on the other hand, in the historic squares of the city, despite not being any different from the ones facing the street, shape together with the building the limits of the square and help create the aesthetics of the space.

The end of the 19th century and the beginning of the 20th century, at the beginning of modern architecture, urban planning principles suffer significant changes leading to bold and imposing architectural styles which acknowledge and highlight the relationship between urban space, building and roof.

In Timisoara, after the fortress was torn down, along the proposed new streets but mainly in the square, the new urban planning principles and their effect on the way the roof was shaped is visible. Monumental buildings placed on the edge of the square, imposing buildings placed along the radial streets, all show roofs which have more than a strictly functional purpose. They have a significant height increasing the monumentality of the building and are completing the importance of the square and street.

Besides the historical evolution of the urban planning principles, another criterion which is significantly influencing the value of a building and roof structure is its position related to protected urban areas or in the protected area of an architectural monument. Since in Timisoara 3 major protected urban areas can be identified, different approaches have to be taken into consideration in these areas, in order to preserve the original ambience and only little changes can be made to the exterior appearance of the buildings and ultimately to the roof. At the same degree, buildings placed in the vicinity or in the protected area of one heritage building, of local or national importance, have to preserve the authenticity of the environment of the heritage building.

Fig. 2.12 Roof structures with no urban value placed along narrow streets

2.2.1.1.1 Urban analysis

Another essential feature which is influencing the way the building and the roof are perceived from the street level is the relationship between the buildings, their distance from the street and their height.

Buildings placed in a continuous frontage (Fig. 2.13a), create a homogeneous impression, leading to a global perception of the whole aggregate of buildings. Due to their closeness, the aesthetics of each building and roof is not as important, and only peculiar features catch the eye of the pedestrian. This is the case of many buildings from the historic part of the city. Corner towers, specific for the beginning of the 20th century, chamfered corners or roofs with imposing appearance and decorative elements may emphasize the building.

On the contrary, buildings placed in a discontinuous frontage (Fig. 2.13b), are highlighted by their surrounding free space and do not need specific urban contexts or architectural features in order to be observed by the passerby.

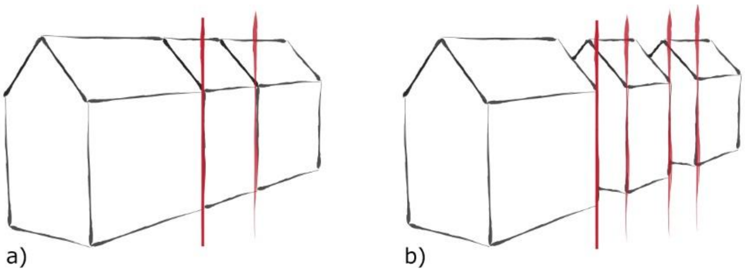


Fig. 2.13 Used frontage (a) continuous frontage; b) discontinuous frontage)

The same principle is also applicable when assessing the influence of the height of the buildings on the way a roof structure can be perceived. A continuous frontage (Fig. 2.14a) of buildings with constant height at the cornice also offer a uniform appearance, where unique elements can highlight a specific building. Variable cornice heights (Fig. 2.14b), automatically create a hierarchy of the buildings, highlighting the higher ones and putting the lower ones into the background.

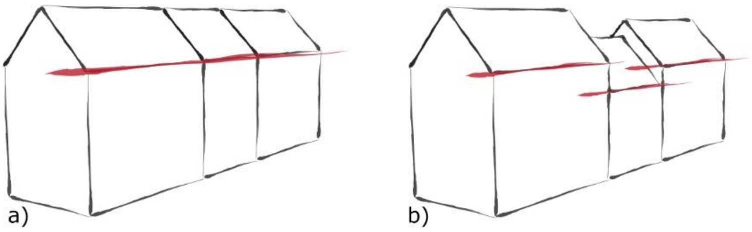


Fig. 2.14 Variations of the height of the buildings (a) constant height at the cornice; b) variable height at the cornice)

One of the most significant urban planning features defining how the passerby perceives roofs is the alignment of the buildings and their distance to the street. An alignment that has not been clearly defined (Fig. 2.15a) indicates a period in which the urban planning principles were not clear, and the area evolved randomly. This is mostly the case of residential areas where roofs do not have significant value, and only insignificantly influence the way the street and the building is perceived.

If the buildings are aligned, the distance towards the street is essential. Aligned buildings placed on the street limit (Fig. 2.15b), create narrow streets which make the perception of the roofs almost impossible. The aesthetics of the building is, in this case, independent and is not completed by the roof. Only corner buildings become important in these areas and are treated in a more careful way, most of the times having a corner tower or spectacular corner decoration, meant to highlight the building. If, however, the alignment is withdrawn from the road (Fig. 2.15c), the street profile gets wider and depending on the height of the building, the roof can be perceived from the ground level. In this case, roofs also help shape the ambience of the street and complete the aesthetics of the buildings. At the same time, corner buildings are far more imposing and highlighted.

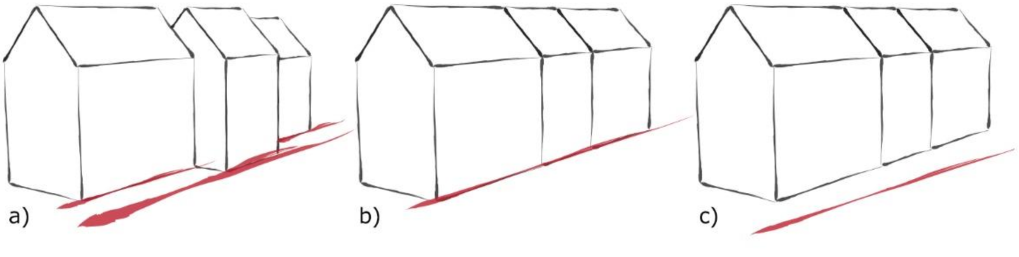


Fig. 2.15 Alignment (a) No clearly defined alignment; b) street alignment; c) alignment withdrawn from the road)

2.2.1.1.2 The geometry of the roof structure related to the urban context

Despite being related to the aesthetics of the building, the shape of the roof is also influencing how the building can be perceived from the street level.

1. Shed roofs (Fig. 2.16a) were almost exclusively used for annexe buildings facing interior courtyards and cannot be perceived by the pedestrian. They are mainly roofs with a functional purpose and do not add up to the aesthetics of the building.
2. Gable (Fig. 2.16b) and hip roofs (Fig. 2.16c) were used in the 18th and 19th century. They were used in the main squares of the city, helping shape the ambience of the space and highlighting the buildings. If however they were placed along the narrow streets of the old city, the roofs become almost invisible from the street level and do not have an essential role in defining the urban space.
3. Gambrel (Fig. 2.16d) and jerkinhead roofs (Fig. 2.16e) were commonly used at the beginning of the 20th century since their shape is appropriate for imposing roofs and monumental buildings. Due to their steep slope in the inferior part, these roof structures seem higher from the street level and increase the height of the building, ultimately shaping the limits of squares and streets.

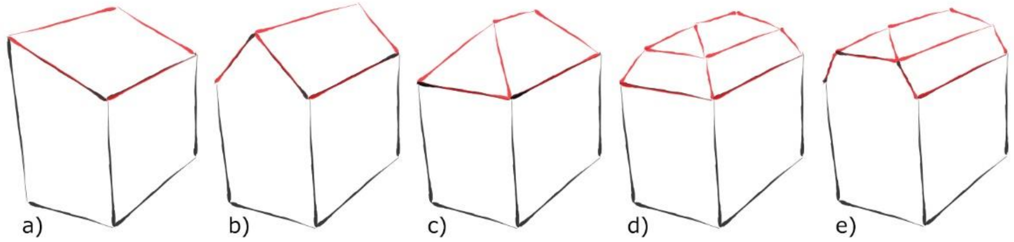


Fig. 2.16 Roof shapes (a) shed or lean-to roof; b) gable roof; c) hipped roof; d) gambrel roof; e) jerkinhead roof)

2.2.1.2 The architectural value of the roof structure

Architectural styles shape roofs in the same way as urban planning principles [91]. Considering the evolution periods of the city of Timisoara, it was observed that the role of the roof structure is changing in time. First, in the 18th century, roof structures proved out to have mainly a functional purpose, or in exceptional situation an urban space-defining role. Due to the strict urban rules, roofs are mostly invisible in the historic part of the city. Only in the main squares, their importance is brought forward, completing the aesthetics of the building, but still keeping a simple appearance, without having a decorative purpose.

The late 19th century styles, have the most significant influence on the way roofs relate to the buildings. The Secession style uses corner roof structures to highlight the corner of buildings and increase their monumentality (Fig. 2.17). Later, at the beginning of the 20th century, the emergence of new urban planning principles offers the building the possibilities to be perceived for from a more significant distance which affects the scale and detailing of the buildings. Buildings with simple functions, like residential buildings, are treated like imposing ones being extremely expressive and present a significant number of decorative elements also in the area of the roof structure. Roofs become in this period one of the leading aesthetic and expressive features of the architecture.

Fig. 2.17 Secession style corner roof structures

Considering this, in order to properly understand how the roof is connected to the building, it is essential to analyse the history and the original functional purpose of the building. Residential buildings from the 18th century are differently decorated and have a completely different scale compared to the ones from the 20th century, which ultimately affects also the shape and dimension of the roof. At the same time, the aesthetic of 20th century roofs is far more expressive than the one of the previous centuries.

The study of the historic part of Timisoara brings forward how important the connection between urban planning principles and architectural styles is. They are strongly influencing each other. Therefore, it was observed that architectural aesthetics is highlighted by the urban layout and the urban space defined by the aesthetics of the building. Ultimately, the roof is only emphasizing this connection.

2.2.1.3 The symbolic value of the roof structure

Besides being influenced by urban planning principles and architectural styles, roof structures from the 18th, 19th and 20th century in Timisoara show that there are also more sophisticated features which are defining their shape and complete the aesthetics and technical aspects of the structure. These features can be identified by performing a transdisciplinary assessment which exceeds the typical assessment boundaries.

Geometry has a significant influence in shaping architecture, urban space and art all around the globe [72,74,92,93]. Plato’s ratios were used since the Middle Ages to express order and define patterns which would create a close connection between spaces, buildings and their environment and different parts of the building. These principles where used by the craft guilds starting with the Middle Ages until their fall in the 20th century. They used different symbols and symbolic ratios to define architecture.

During the study, complex geometries and ratios were identified all around the city. These ratios are highly symbolic in the 18th century, in the active periods of the guilds, but become more straightforward towards the 20th century when the traditional knowledge is starting to be replaced by technology and efficiency. This analysis highlights the fact that historic timber roof structures are marked by the history of the craft guilds and their traditional knowledge. The skills of the craftsmen had an essential effect on the complexity of the roof structures and the used typology.

In addition to the high symbolic value of the used ratios, the historical value of timber roof structures is also enhanced by the presence of the carpenters mark or numbering signs, used to mark matching joints, placed on the timber elements, certifying that there is something far more complex behind the shape and details of the roof (Fig. 2.18).

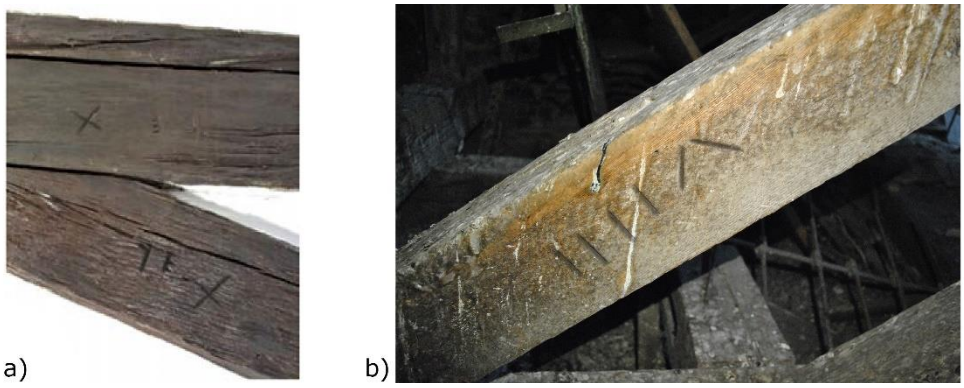

a) b)

Fig. 2.18 Markings on timber elements a) Carpenters mark, b) numbering of timber elements

2.2.1.3.1 The ratio between the roof and the building

Since the squares of the city where planed with different functions and in different periods of its development, each square presented a different approach concerning used ratios as a connection between the exterior appearance of the building and the roof [90]. At the same time, due to urban planning principles, in the 18th century, only the proportions of important buildings can be perceived from the street level or of the buildings placed in the main squares. At the beginning of the 20th century, on the other hand, due to larger surrounding spaces around the buildings, the used ratios, which are more simplified, can be easily perceived by the passerby.

The Saint George square was reshaped in the 20th century, and the buildings are mostly Secession style, despite being one of the oldest ones in the city. In this area a wide array of ratios was observed reaching from highly symbolic Golden ones (Φ) towards the dynamic √2, √3, so no clear pattern could be observed since the buildings were developed in the period of the decline of the craft guilds.

The Liberty Square is rectangular shaped, which used to be placed in connection with one of the entrances of the fortress and had the primary military function. It presents a mix of original Baroque buildings but also insertions of 20th-century buildings. This is why, in this square, a clear difference between the 18th century building ratios (√2, √3 and Golden ratio – Φ) and the 20th century buildings defined only by dynamic and static ratios can be observed (Fig. 2.19).

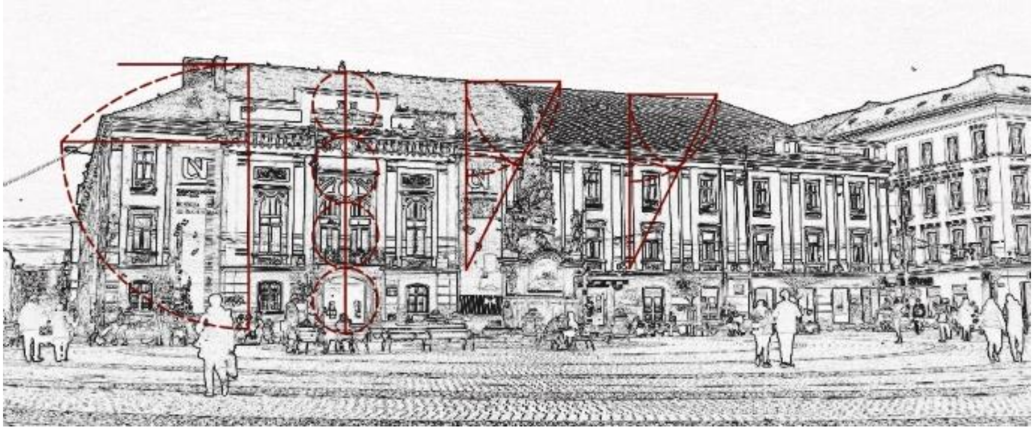


Fig. 2.19 Geometric analysis of the Liberty square

The main square of the city, Union Square, is the largest one from the city and used to be an administrative square. Most of the buildings from this square are Baroque, revealing almost the exclusive use of √2 ratios, meant to define the ratio between the roof and building. The newer buildings respect the original ratios and define the height of the roof compared to the building in the same way (Fig. 2.20).

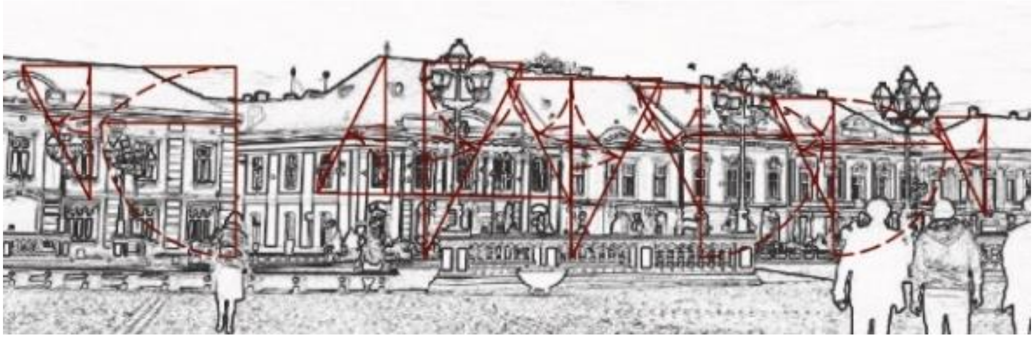


Fig. 2.20 Geometric analysis of the Union square

Finally, the Victory square developed in the 20th century presents the period of the fall of the craft guilds. In this area, the ratios are static, presenting an evident lack of interest towards proportions. Only 1/3 or 1/6 ratios are being used to define the main decorative elements of the building, while the roof is linked to the building by √3 ratios (Fig. 2.21).

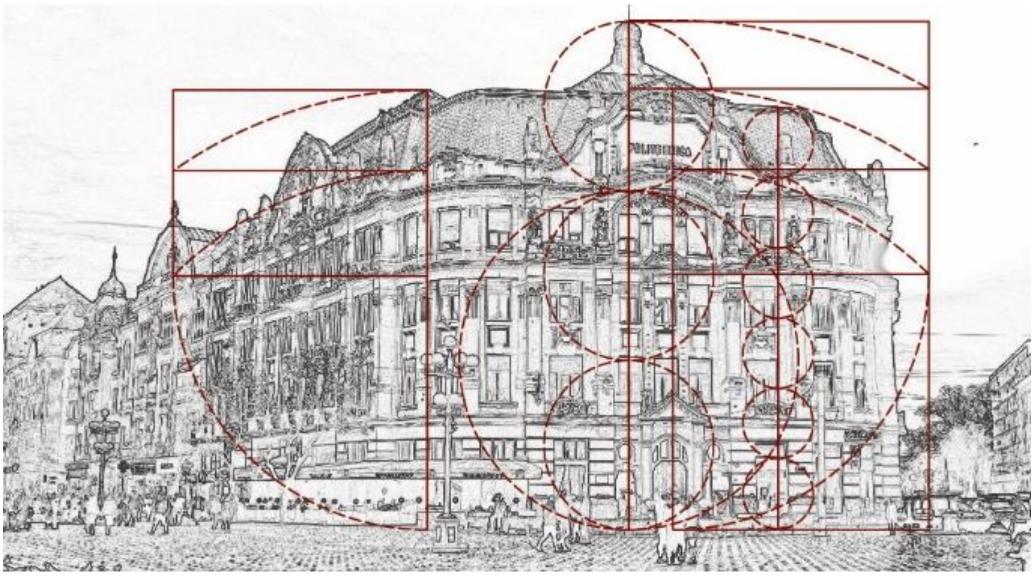


Fig. 2.21 Geometric analysis of the Victory square

2.2.1.3.2 The ratio between structural elements

Roof structures in Timisoara are marked, depending on the period in which they were built, by very steep slopes, ranging from 45° in the 18th and 19th century up to about 70° at the beginning of the 20th century. The slope of the roofs combined with the fact that the width of the buildings seldom exceeds 12-13 m, is significantly influencing the geometry of the roof trusses and ultimately the ratio between their height and width and the position of important structural elements.

The geometric analysis of the trusses revealed that 18th century roof structures were governed by a complex series of Golden ratios (Φ) (Fig. 2.22), also by dynamic geometric proportions like √2. These ratios define the proportion between the rafter length and the roof height, the position of the collar or straining beam but also the rafter slopes.

Later on, at the beginning of the 19th century, the geometric analysis of the structures built in that period showed that mainly Golden ratios (Φ) were used to define the position of the joints but also local static symmetries like 1:1 were used to define the rafters.

The beginning of the 19th century continues the 18th century principles using mainly Golden ratios (Φ) to define the position of the carpentry joint but also marks the use of static symmetries like ½ used for specific divisions. Still, the analysis revealed a high interest in the symbolic and sacred geometry.

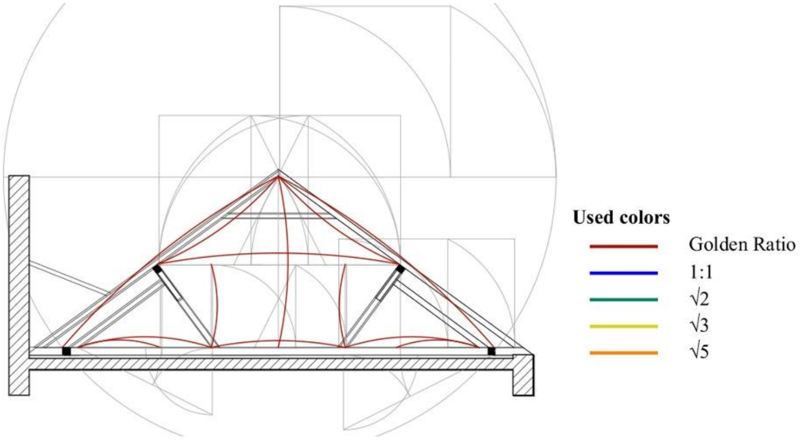


Fig. 2.22 Symbolic analysis of an 18th century roof structure

Like in the case of the ratio between the roof and the building perceived from the street level, starting with the end of the 19th century, used ratios start to be mixed up and to get more simplified. Buildings from the end of the 19th century still present the presence of local Golden ratios (Φ), but the mainly used proportions in this period are the dynamic ones √2, √3 and √5 marking the main joins and intersections of the roof (Fig. 2.23).

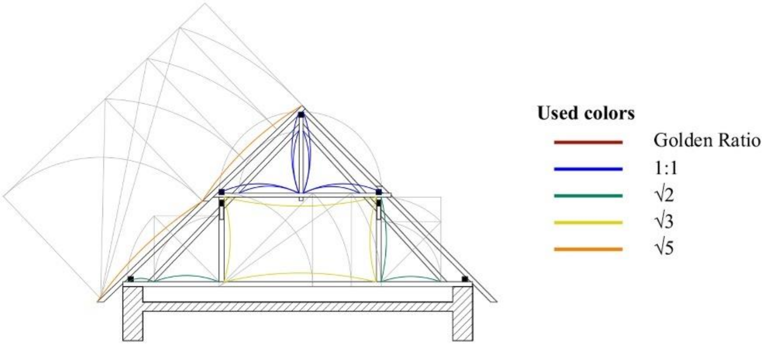


Fig. 2.23 Symbolic analysis of a 19th century roof structure

The roof structures from the beginning of the 20th century present a predominant use of static ratios 1/3, 2/3 and ½. As in previous periods, the ratios are used to define the position of the purlins, the distance between the posts related to the height of the roof and their position related to the ridge purlins. The Golden ratio disappears almost completely while dynamic ratios are used mainly locally to define the position of more important structural elements or joints (Fig. 2.24).

Later on, as roof structures become more and more efficient, the use of complex principles in defining them and the position of their structural elements disappears, and static ratios are used seldom.

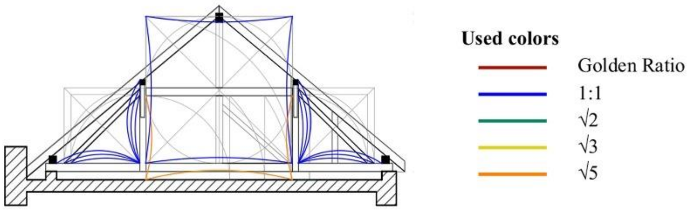


Fig. 2.24 Symbolic analysis of a 20th century roof structure

The study of symbolic ratios used to define historic timber roof structures performed in the historic part of Timisoara highlighted that sacred ratios, and dynamic and static ones were used for all roof structures until the beginning of the 20th century [79,89]. In the same time, it brought forward that not only were symbolic ratios used by the craftsman, but their use is also connected to the philosophy and the active periods of the guilds. Therefore, in their active period, used ratios are sacred and dynamic and used to define the position of main structural elements and the geometry of the roof structure, while toward the decline of the craft guilds, the used ratios get more simplified and disappearing completely after their fall (Fig. 2.25).

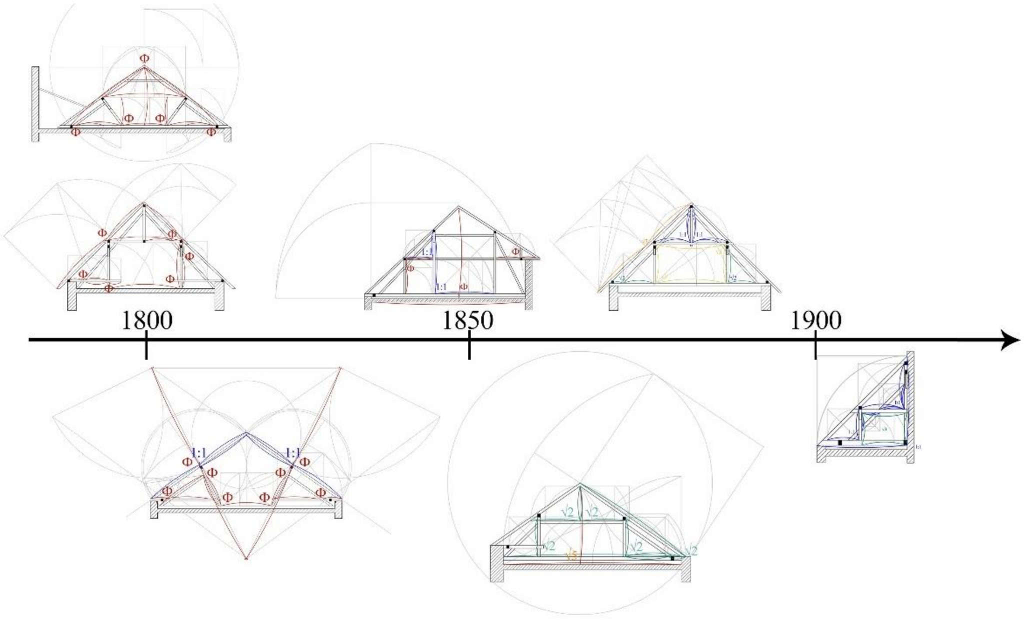


Fig. 2.25 Evolution of the used ratios in historic timber roof structures in Timisoara

2.2.1.4 The structural value of the roof structure

The appearance of the first roof structures is related to the first human settlements from the Stone age. After settling down, humans started to need protection from environmental factors and had to find various solutions in order to protect themselves, their animals and food [14].

Since the only available materials at that time were earth, stone, timber and different plants, the first version of the roof structure is supposed to have been made of timber elements tied together, with plant stalks placed in overlapping layers as a covering. These first structures represent the cornerstone in the evolution of roof structures.

Subsequently, due to changing climatic conditions in the Neolithic period, the evolution of new technologies and tools, useful for woodworking, led to the development of various structural typologies.

However, the most important factor which led to the evolution of roof structures until de Middle Ages were the local climatic conditions and the available timber species. This ultimately led to the definition of different roof shapes and used structural types. Considering this, roof structures can be divided into two different categories, considering their composing structural elements: German and Roman roof structures [23].

The Roman roof structure was initially developed to cover spaces with a large span and used to have only a low height. Although no roof structures of the original Roman type were preserved until today, these types could be distinguished by the presence of only main frames, placed at a small distance between each other, of about 3-3.5m. All the frames were connected in the transversal direction by tie-beams and in the longitudinal direction by eaves, intermediate and ridge purlins. Another feature of these roofs was that the roof structure used not to be connected with the historic masonry wall and was just placed on the top of it.

Due to the high diversity of used timber species and dimensions, for these roof structures, the craftsman used the same structural type and only changed the used timber and adapted the cross-section of the elements, which led to only little evolution of the roof structures in this region. Today, a version of this roof structure can be still found in the Alpine region where roofs with a low slope and similar structural features are still typical [34].

Since the North-European area did not offer the same advantages concerning available timber, in this area, the roof structures had to evolve continuously. They had to be adapted for every building in order to be economically efficient, which led to a high diversity of structural solutions. At the same time, due to the climatic conditions from the western and north-western part of Europe, these roof structures had to have a rather steep slope, in order to protect the building from rain better.

The structure, in this case, presented a clear difference between the main and secondary frames, the main one being composed of two rafters, connected by a tie-beam in the inferior part and the secondary ones only by rafters. At the same time, the frames were connected between each other by complex systems of longitudinal rigidity enhancing elements, placed between the rafters or between the king posts, if used. Additionally, the roof structure used to be connected with the walls by a wall-plate and is not just placed on the wall.

After the Middle Ages the two types started to merge, the new emerged roof structures combining features and structural elements from both regions. Depending on the area of Europe where they evolved, each new roof structure type presenting more German or more Roman features (Fig. 2.26).

Therefore, according to the sketches of Ostendorf, roof structures from southern France, present typical German features but replace the complex longitudinal rigidity enhancing systems with the purlins of the Roman roof. At the border between Germany and France, the distance between two main frames reaches 4 m, and the region marks the appearance of the hanging device meant to transfer the gravitational loads by using angle-braces, passing-braces and posts towards the exterior structural elements and finally the wall plate and walls. In the north of France, on the other hand, the Roman influences are rather predominant, the roof structures presenting similar shapes as the German ones, and the presence of the wall-plate but the tie-beam is still used for every truss, both main and secondary. This type will be the base for later Baroque style roof structures. Purlins in all these cases are placed between the rafters and compound rafters.

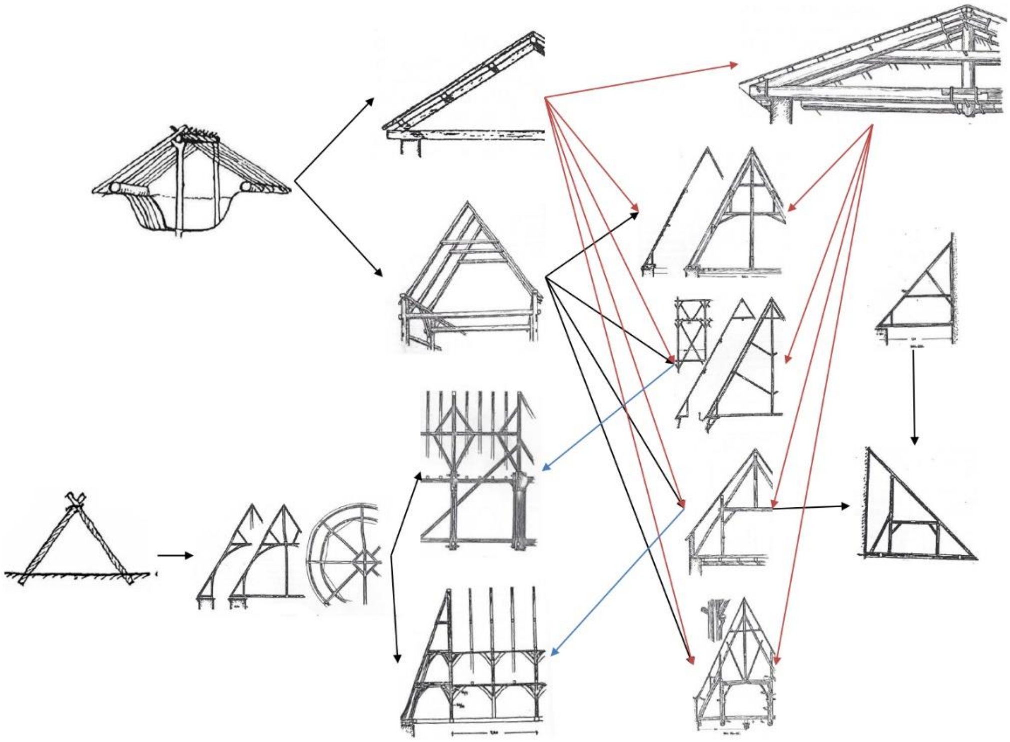


Fig. 2.26 Evolution of roof structures (after [23])

The areas of German influence present a completely different adaptation of the two structural typologies. There, roof structures are evolving towards purlin roofs with a queen post hanging device. Purlins, in this case, are no longer placed between the rafters and compound rafters but are connected with the hanging device and its angle-braces, passing-braces and posts.

Architectural requirements lead, in time, to the emerging of roofs with different shapes and more complicated roof structures, adapted to the shed roof and the hip roof. The shed roof appeared as a covering of the buildings connected to the exterior walls of medieval fortified churches. Considering that these buildings often had a wide span, and the roofs could not have a significant height, new structures had to be developed and old ones adapted. While the intermediate support of the rafters could be solved by adapting already used roof structure types, the main problem arises from the connection of the structure to the fortress wall without directly transferring the loads towards its top. If the spans were small, the top part of the rafters could be supported by ridge purlins placed directly on timber elements inserted in the walls. For larger spans, on the other hand, the solutions had to be more evolved, since the out-of-plane loads caused by the structures had to be transferred through other elements of the roof structure and not through the walls. This leads to the development of the extensive use of angle-braces and passing-braces which connect the tie-beam with the rafters, therefore transferring the loads directly from the exterior envelope towards the tie-beam.

Hip roofs, on the other hand, present two different problems. First, the structural typology used for the two ends of the roof has to be adapted since the slope is more pronounced than on the main sides of the roof. Secondly, the connection between the structure of the main roof and that of the two additional ends had to be solved in order to transfer the loads from the steep slope towards the main structure. Two different solutions emerged in this case. The first one represents an adaptation of the roof structure with longitudinal rigidity enhancing system, used to increase the rigidity of the main structure and be able to transfer the loads. The second solution supposed the use of a complex system of purlins and collar beams to connect the steep slope with the last frame of the main structure and transferring the loads towards the hanging device of this frame. In order to be able to transfer the loads, the cross-section of the timber elements used in the last version had to be increased.

2.2.1.4.1 Structural types

A comprising classification of historic roof structures is a rather tricky task since adaptations of common roof structure types, local influences and geographic evolutions, led to a great variety of structures. Still, considering the evolution of roof structures, they can be divided into purlin and rafter-roof structures.

In Timisoara two main types of roof structures, with various adaptations mainly at the end of the 19th, the beginning of the 20th century can be identified.

In the 18th century, the most common truss type was the German-inspired “liegender Stuhl”. This structural type, most of the times a purlin roof structure, first appeared in the 14th century, developed by the need to directly transfer the loads from the rafters to the main load-bearing structure [14,34,94]. Therefore, inclined posts were used (inner rafters), placed parallel with the rafters, forming together with the straining beam, the straining device of the structure. In order to increase the rigidity of the roof structure also in the longitudinal direction, ridge, intermediate and eaves purlins were used, and diagonal compound rafters placed in the plane of the rafters. At the same time, an additional lower plate was used to connect the straining beams and collar beams of the principal and secondary trusses (Fig. 2.27).

In this case, the only difference between the main and secondary trusses of the roof structure is the missing straining device (inner rafters, straining beam and counterbraces) for the secondary frames. Tie beams and collar beams are used for all the trusses.

Structures from this period, usually have a 45° slope, a width of about 12m and a height of around 6m. At the same time, all the timber elements also used have a significant cross-section.

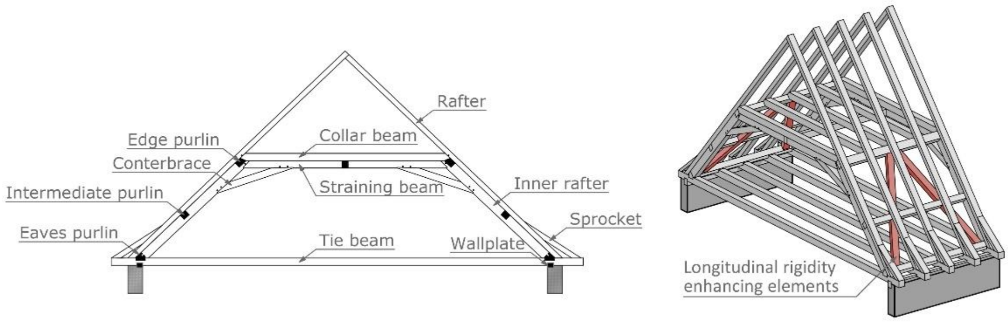


Fig. 2.27 18th century roof structure type

The main advantage is the utterly free attic space and the possibility to cover a wide span, making this roof structure useful for both private and religious functions. In Timisoara, this truss type can be found mainly in the area of the old fortress. For residential buildings, the structure uses just the inner rafters for the straining device while for buildings with a more significant width, like churches, an additional hanging post is placed in the central part of the truss [14,23].

In the 19th and 20th century, mainly queen-post roof structures were used, adapted to the shape and position of the building. Despite being developed at the beginning of the 14th century, queen-post roof structures were most commonly used in the 19th century since they are efficient roof structures, able to cover wide spans and combine already know elements. In this case, the hanging device is composed of two hanging posts, connected in the upper part by a collar beam.

In Timisoara, three different types of queen-post roof structures were identified. The first one was mainly observed over buildings from the beginning of the 19th century (Fig. 2.28). This represents a more peculiar type of queen-post roof since the posts are placed almost perpendicular to the rafters, and they are not connected in the upper part by a collar beam. Still, an additional angle brace is placed in the upper part of the roof, connecting the rafters. A clear difference between the main and secondary frame of the structure can be observed, the secondary ones being only composed of rafters.

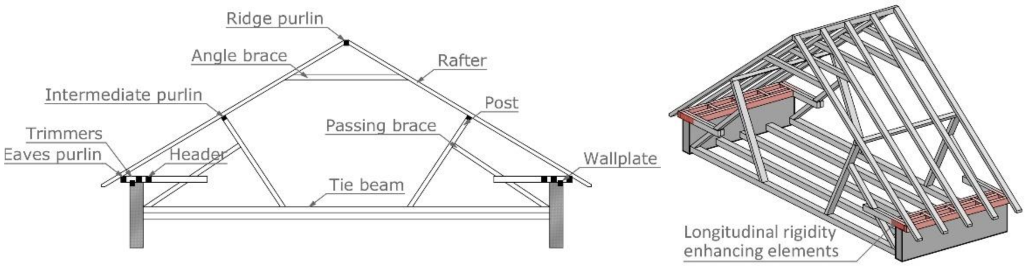


Fig. 2.28 Beginning of the 19th-century roof structure type

In this case, in order to increase the rigidity of the structure also in the longitudinal direction, all the trusses are connected by eaves and intermediate purlins and an additional system of timber elements, composed of headers and trimmers, placed over the walls.

Towards the end of the 19th century, a more common type of queen-post roof structure was used, with hanging post placed perpendicular to the tie-beam. This type was also one of the most frequently used types all around Middle-Europe in the 19th century.

The structure is mainly composed of two posts and one collar beam for forming the hanging device, additionally connected by two passing braces. If the roof was higher, additional queen posts or even king post structures (Fig. 2.29) were stacked one over the other, also connected to the passing braces. In the longitudinal direction, only purlins were used to connect the main and secondary trusses of the roof, without the use of any additional complex systems. Like at the beginning of the 19th century, a clear difference between main and secondary frames can be observed, the secondary ones being also composed only of rafters.

Some of the main advantages are that this type of structures can be easily adapted for various shapes of buildings and can be used as multilayer structures, suitable for roofs with significant height. At the same time, compared to the 18th century structures, they present a more efficient use of timber, the cross-section of the elements being significantly reduced.

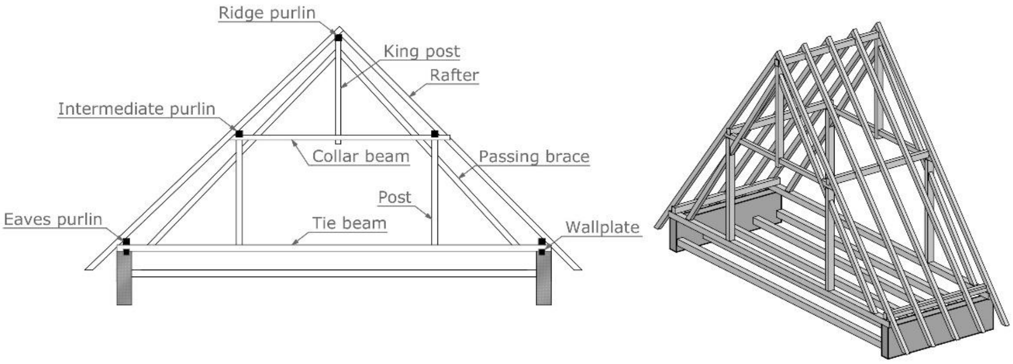


Fig. 2.29 End of the 19th-century roof structure type

2.2.1.4.2 *Joint typology*

Starting from the first timber structures, the connections between the linear timber elements was possible through a great variety of joints. Their shape and dimensions were not made using mathematical equations, but by using the knowledge of the craftsman and adapted for each structure. From a technical point of view, the forces were transferred mainly through compression and friction between the timber elements. Subsequently, at the end of the 19th century, as roof structures started to become more and more efficient from both economic and structural point of view, the use of codes and equations became the primary way to design timber joints. Still, traditionally crafted timber joints can be divided into four main categories based on their main features.

Tenon and mortise joints have significant importance in the development of timber roof structures since they permit the connection of two timber elements which are placed perpendicular one to the other creating T shapes or sometimes L shaped joints (Fig. 2.30). Since the tenon of one of the elements is inserted in the mortise hole of the other, they can additionally be connected by using wooden pegs or later in the 20th century by steel pegs, increasing their axial stiffness. The transfer of loads, in this case, happens at the contact between the two elements, mainly perpendicular to the timber element fibres. From a geometric point of view, tenons can be placed on the edge or in the middle of the inserted element and can have a length of about 1/6 up to 1/2 of the base element in order to ensure a proper transfer of loads.

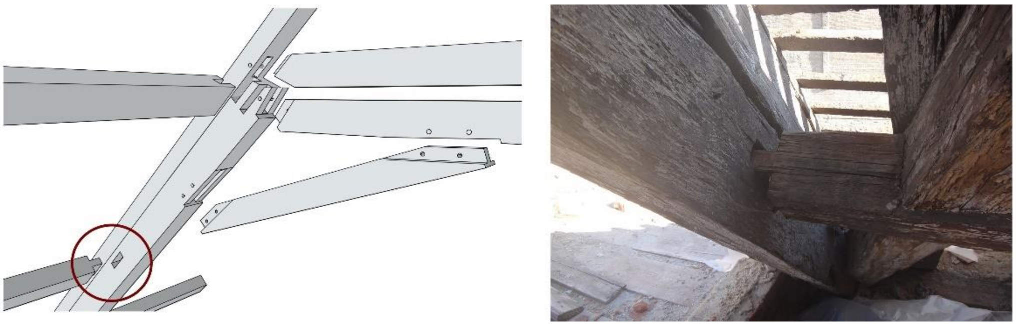


Fig. 2.30 Tenon and mortise joint

Notch joints are most of the time used to transfer inclined forces (30 up to 60 degrees) to another timber element, most of the time used as a connection between counter braces and rafters or in the case of inner rafters at their connection to the tie-beam. The groove of the notch can reach around 1/3 to 1/6 of the base element. Compared to the tenon and mortise joints, which can also take over tension loads, due to the presence of the pegs, in the case of notch joints only compressive axial forces can be transferred through the contact area of the element, at a certain angle to the fibres of the timber.

In order to increase the stiffness of the notched joint, a tenon and mortise hole can be added. In this case, the mortise can have a depth of around 1/3 of the element in which the tenon is inserted. The main advantage of this type of joint is that besides transferring compressive axial forces, it can also take over tensile axial forces, which is possible due to the presence of additional fasting elements like wooden or steel pegs (Fig. 2.31).

Fig. 2.31 Notch joint

Lap joints were usually used for timber elements which have to cross each other or had to join without having to make a mortise hole (Fig. 2.32). If the timber elements had to cross, most of the time half of each timber element was subtracted, leading to a joint which has the same height as the two structural elements. One of the most common lap joints is the dovetail joint, which permits the connection between two timber elements and also ensures the possibility to transfer small tension loads without any wooden or steel fasteners. Still, most of these joints use wooden pegs in order to increase the stiffness of the joint.

Fig. 2.32 Lap joint

Scarf joints are joints which ensure the connection between two timber elements in order to obtain a longer one if the available timber does not have the necessary length. Developed starting with the 10th century, this joint suffered continuous changes, from the plain scarf joint, which proved its problems even from the beginning up to more complex under-squinted ends or keyed scarfs. The combination with principles from the lap joints was necessary in order to increase the compressive, tensile and shear stiffness of this joint. Therefore, most of these joints also use wooden or steel fasteners (Fig. 2.33).

Fig. 2.33 Scarf joint with steel fastener

2.2.2 Vulnerability of historic timber roof structures

2.2.2.1 Damages and decays

In order to be able to develop intervention strategies and choose proper rehabilitation techniques, a comprehensive analysis of the state of the structure is necessary, with clear documentation of all damages and their causes [5].

The assessment guidelines developed during the various COST actions define a clear list of defects and damages that have to be evaluated [7]: biological damages, defects of the timber, mechanical damages, structural damages (cracks and fissures) and the state of conservation of supporting structures and other not load-bearing elements. Considering this, the decay of historic roof structures can be classified in two categories, decays which affect the active cross-section of the timber elements (moisture-induced decay, fungal or insect attacks, or mechanical and chemical damages) and decays caused by structural errors or wrong interventions [2]. At the same time, the analysis of roof structures shows that their damage can also occur due to various other reasons like fire, which can cause the partial or complete loss of a roof structure or due to unauthorised human intervention and partial or total replacement of otherwise valuable elements.

In the case of roof structures from Timisoara, humidity is the main cause of the decay of the structural elements. The damages are mainly visible in the area close to the roof envelope, affecting rafters and purlins but also in the area of the roof to wall connecting load-bearing elements, affecting wall plates, ridge purlins and the ends of tie-beams (Fig. 2.34). At the same time, the lack of maintenance is also leading to a significant deposit of pigeon droppings on the timber elements and the attic floor, which is significantly affecting the state of conservation of the timber and is representing an additional combustible load (Fig. 2.35).

Fig. 2.34 Decay of the timber elements caused by water infiltration

Fig. 2.35 Pigeon droppings deposit

These damages of the timber elements are mainly related to the state of conservation of the roof envelope, missing envelope elements (tiles or metal sheets) and it is therefore also relevant to address the state of conservation of the roof envelope in the area of the ridge, cornice and chimneys.

2.2.2.2 Climatic vulnerability

In recent years the effect of climate change on the state of conservation of heritage structures is getting more and more in the attention of the scientific community. International climate change organization like the IPCC (Intergovernmental Panel on Climate Change) [95] and even the Romanian National Meteorological Authority [96] acknowledge that different meteorological parameters, like temperatures, precipitation quantities have suffered a notable change in recent years. Also, they highlight the need to take extreme events more seriously (heavy precipitations causing flooding, hailstorms or high wind speeds), since they will take place more often in the future.

Recent extreme events in the western part of Romania, brought forward that heritage structures in this region can be severely affected by climate change too. In the middle of September 2017, a windstorm with maximum wind velocities of 145 km/hour heavily affected their roof coverings.

In order to accurately assess the influence of climate change on timber roof structures and roof coverings, future climate change scenarios were assessed. To be able to develop future climate scenarios an imposed off-set method was used, which only considers the change of different meteorological parameters with specific coefficients determined by the Intergovernmental Panel of Climate Change, without taking the way they influence each other into consideration [97].

Using current Energy Plus Weather File (EPW) available for Timisoara and a future climate weather file generator "CCWorldWeatherGen" [98], meteorological parameters most likely to affect the state of conservation of timber roof structures were processed and analysed. The weather file generator, together with the IPCC offset parameters, makes it possible to evaluate meteorological parameters until 2080 quickly, and identify which one of them will be a real threat for historic timber roof structures. In this case, humidity and temperature values for current and future weather scenarios were taken into consideration. At the same time, extreme events which were more likely to affect the state of conservation of roof coverings directly, were also taken into account and hailstone was chosen as it is causing the most damage to roof tiles.

2.2.2.2.1 Moisture

Timber is a structural material significantly influenced by its environment, quickly absorbing humidity from the surrounding and drying out when temperatures are high and humidity low [99]. This causes swelling and shrinkage of the wood and ultimately, additional stress in the structural members and changes of the mechanical properties of wood [100].

Moisture loads are influenced by the local climate, the cross-section of the timber elements and the presence of a protective coating [101]. If, however, the timber elements are protected from direct contact with liquid water and sun, the moisture content is influenced by the local microclimate, the relative humidity and temperature of the surrounding air [99], which is the case of historic roof structures.

Timber elements absorb and lose moisture, according to the changes of the surrounding air, both daily and seasonal, until they reach a moisture equilibrium. Various analytical expressions have been developed in time, in order to be able to determine the equilibrium moisture content of the wood. One of the most reliable formulas used to predict the equilibrium moisture content of timber elements (M_{eq}) [102], is the Hailwood and Horrobin sorption theory [103] which is based on the relative humidity and temperature of the surrounding air (Eq. (1)). This analytical approach is used even today in various timber element moisture-related studies [104].

$$M_{eq} = \frac{1800}{W} \times \left(\frac{kh_r}{1 - kh_r} + \frac{k_1kh_r + 2k_1k_2k^2{h_r}^2}{1 + k_1kh_r + k_1k_2k^2{h_r}^2} \right) \tag{1}$$

Where h_r, is the relative humidity of the surrounding air (fractional), T is the surrounding air temperature (Fahrenheit) and W, k1 and k2, are different coefficients determined by considering the air temperature [99,102]:

$$W = 339 + 1.29T + 0.0135T^2 \tag{2}$$

$$k = 0.805 + 7.36 \times 10^{-4}T - 2.73 \times 10^{-6}T^2 \tag{3}$$

$$k_1 = 6.27 + 9.38 \times 10^{-3}T - 3.03 \times 10^{-4}T^2 \tag{4}$$

$$k_2 = 1.91 + 4.07 \times 10^{-2}T - 2.93 \times 10^{-4}T^2 \tag{5}$$

Using the available EPW file for Timisoara, the current and future relative humidity was determined. Scenarios show that the relative humidity is likely to remain quite similar for winter months while dropping (up to 20%) for summer months. Using the obtained monthly data, the equilibrium moisture content of timber in local climatic conditions was determined. Fig. 2.36, presents the equilibrium moisture content for current and future relative humidity scenarios, determined using Eq. (1). The results show minor changes of M_{eq}, until 2080 compared with the obtained results for current climatic conditions.

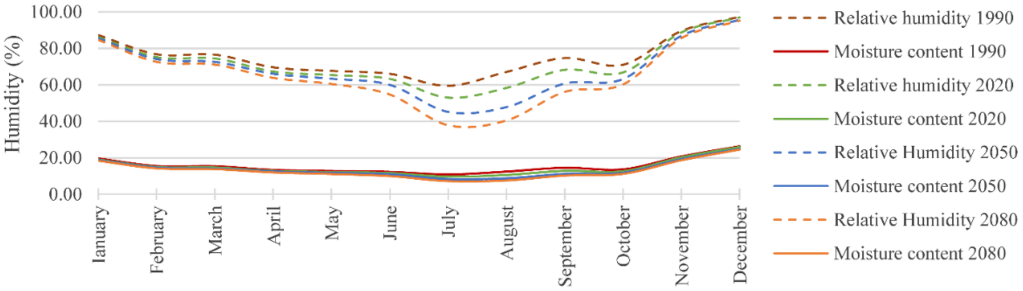


Fig. 2.36 Moisture content analysis, monthly mean humidity (current and future scenarios)

The analysis shows that moisture load will remain a constant threat for timber structures, without suffering a severe change in the future. Still, extreme events, like heavy precipitation and floods, can change the data, raising the relative humidity and ultimately raising the moisture loads in timber structures and affecting their mechanical properties.

2.2.2.2.2 Hail

Hail does not have a significant impact on the structural characteristics of timber structures but influences the overall aesthetic value of heritage buildings damaging the envelope of the roof and valuable features of the facades [105].

Almost like in the case study of Oravita [106], Timisoara is influenced mainly by moist air coming from the Atlantic Ocean and warm air from the Mediterranean Sea, which can favour the formation of hailstones. Taking the climatic conditions into consideration, the diameter of hailstones can get up to 80 mm in this region [107].

In order to determine the impact force of hailstone, of various diameters, on roof coverings of heritage buildings in Timisoara, a deterministic equation was used (Eq. (6)) [105].

$$F_c = k_n \left[1 + (0.2p + 1.3)\left(\frac{1 - COR}{COR}\right)\left(\frac{-\mathrm{c} + \sqrt{c^2 + 4\mathrm{d}}}{2}\right)\right]\left(\frac{p+1}{2k_n} m_1 v_0^2 COR\right)^{\frac{p}{p+1}} \times$$
$$\times \left[1 - \left(\frac{-c + \sqrt{c^2 + 4d}}{2}\right)^2\right]^{\frac{p}{2}} \tag{6}$$

Where F_c is the impact force of the hailstone (N) and $c = \frac{p \times COR}{(p+2)(0.2p+1.3)(1-COR)^2}$ and $d = \frac{2}{p+2}$

The equation permits the estimation of the impact force of hailstones, considering only five parameters: m, the mass of the hailstone (g); v_0, the hail-stone velocity (m/s); COR, the Newton coefficient of restitution (m/s); k_n, the stiffness coefficient of the hail-stone (m/s) and p, the exponent of the spring behaviour (m/s). While m and v_0 are influenced by the hailstone density (870 kg/m3) and diameter (mm), the other three parameters (COR, k_n and p) can be determined using following equations, based on the velocity of the hailstones:

$$k_n = 2.200 v_0 + 170.269 \tag{7}$$

$$p = 0.010 v_0 + 1.263 \tag{8}$$

$$COR = -0.001 v_0 + 0.049 \tag{9}$$

According to the national norm concerning the design of roof coverings (NP 069 – 2014) [108], roof tiles can be divided into four classes, according to their shape and break resistance. Still, one of the most accurate classifications of roof tiles was identified in the Swiss Hail register (Hagelregister) developed by the Association of cantonal fire insurance (Vereinigung Kantonaler Feuerversicherungen) [109]. According to the register, roof tiles can be divided into five categories, according to their hailstone breaking resistance from HW 1 (roof tile resistant to the impact of a 10 mm hailstone) to HW 5 (roof tile resistant to the impact of a 50 mm hailstone). The resistance was experimentally determined by dropping hailstones perpendicular to the roof tiles with a velocity of 86 km/h [110]. The corresponding hail impact force of the break resistance classes HW3 to HW5 is presented in Table 2-2.

Table 2-2. Hail impact force corresponding to HW3 to HW5 resistance class

Break resistance	Hail diameter (mm)	Mass (g)	Hail velocity (m/s)	Roof impact angle (grade)	Hail impact force (N)
HW3	30	12.29	30.00	90.00	446.2
HW4	40	29.14	30.00	90.00	815.1
HW5	50	56.91	30.00	90.00	1387.0

Since experimental testing was made by constant hail velocity and perpendicular to the roof pitch, additional calculus was made considering three different scenarios. Wind velocity was also taken into consideration for all assessed scenarios. Therefore, taking the vertical velocity of the hailstone, determined by its mass and gravitational acceleration, and the deviation caused by the wind, various impact angles towards the roof were determined.

1. The first scenario supposed hailstone falling on a roof with a 30° pitch considering a mean wind velocity of 4.6 m/s (Table 2-3). Since impact angles are lower than 90°, hailstone impact forces are lower than the tested values for the same diameter hailstones.
2. The second scenario supposed hailstones falling on a 45° roof, also with a 4.6 m/s wind velocity (Table 2-4). Considering the significant angle of the roof pitch, impact angles and therefore also hailstone impact forces are significantly lower than in the 30° roof pitch scenario.
3. The third scenario supposed the same 45° roof pitch, but with a maximum wind velocity of 40 m/s [111] (Table 2-5). Despite the high wind velocity, hailstone impact velocities slightly exceed the ones obtained considering the first scenario.

Table 2-3 Hailstone roof impact force (Hailstone 30-80 mm; Roof angle 30°; Wind velocity 4.6 m/s)

Hail diameter (mm)	Mass (g)	Vertical velocity (m/s)	Wind velocity (m/s)	Hail velocity (m/s)	Impact angle (grade)	Roof angle (grade)	Roof impact angle (grade)	Roof impact force (N)
30	12.29	22.09	4.60	22.56	11.5	30.00	71.53	423.2
40	29.14	25.14	4.60	25.56	10.2	30.00	70.21	767.0
50	56.91	28.20	4.60	28.57	9.2	30.00	69.15	1296.2
60	98.34	31.26	4.60	31.59	8.3	30.00	68.29	2106.0
70	156.17	34.31	4.60	34.62	7.6	30.00	67.57	3348.6
80	233.11	37.37	4.60	37.65	7.0	30.00	66.97	5283.1

Table 2-4 Hailstone roof impact force (Hailstone 30-80 mm; Roof angle 45°; Wind velocity 4.6 m/s)

Hail diameter (mm)	Mass (g)	Vertical velocity (m/s)	Wind velocity (m/s)	Hail velocity (m/s)	Impact angle (grade)	Roof angle (grade)	Roof impact angle (grade)	Roof impact force (N)
30	12.29	22.09	4.60	22.56	11.5	45.00	56.53	372.2
40	29.14	25.14	4.60	25.56	10.2	45.00	55.21	669.5
50	56.91	28.20	4.60	28.57	9.2	45.00	54.15	1124
60	98.34	31.26	4.60	31.59	8.3	45.00	53.29	1817
70	156.1	34.31	4.60	34.62	7.6	45.00	52.57	2877
80	233.1	37.37	4.60	37.65	7.0	45.00	51.97	4522

Table 2-5 Hailstone roof impact force (Hailstone 30-80 mm; Roof angle 45°; Wind velocity 40 m/s)

Hail diameter (mm)	Mass (g)	Vertical velocity (m/s)	Wind velocity (m/s)	Hail velocity (m/s)	Impact angle (grade)	Roof angle (grade)	Roof impact angle (grade)	Roof impact force (N)
30	12.29	22.09	40.00	45.69	41.2	45.00	86.22	445.2
40	29.14	25.14	40.00	47.25	40.3	45.00	85.27	812.3
50	56.91	28.20	40.00	48.94	39.3	45.00	84.28	1380
60	98.34	31.26	40.00	50.76	38.3	45.00	83.26	2250
70	156.1	34.31	40.00	52.70	37.2	45.00	82.22	3589
80	233.1	37.37	40.00	54.74	36.2	45.00	81.18	5672

Comparing the obtained results, 30° roof pitches prove out to be slightly more vulnerable during hailstorms compared to 45° pitches, considering a mean wind velocity. The analysis shows that roofs with 30° pitches which use ceramic tiles are vulnerable for hailstones with diameters of about 50 mm even for mean wind velocities. 45° pitches, on the other hand, are slightly less vulnerable for mean wind velocities, the impact force of hailstones with a diameter of about 55 mm already exceeding the admitted hail impact force of the best resistance class. If however, the hailstone diameter is reaching the maximum for this area, of 80 mm, the impact forces are about four times higher for 30° pitches, and three times higher for 45° pitches than the defined hail force impact resistance of the maximum resistance class (Fig. 2.37).

At the same time, considering 45° roof pitches, wind velocity proved out to have a considerable influence on the hailstone impact force, high wind velocities raising the impact angle of the hailstone, even for bigger hailstone diameters, increasing, therefore, their impact force. Therefore it was observed that for high wind velocities the damages start to appear even for 50 mm hailstones and the impact forces for 80 mm hailstones are even higher than the ones recorded for 45° pitches under mean wind velocities (Fig. 2.38).

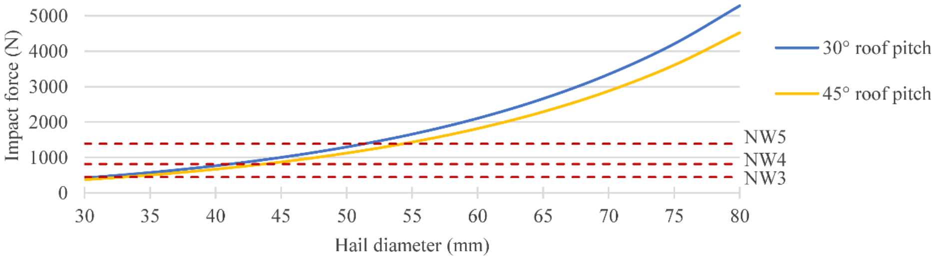


Fig. 2.37 Hailstone impact force analysis for 30° and 45° roof pitch at a constant 4.6 m/s wind velocity compared with characteristic hailstone break resistance values

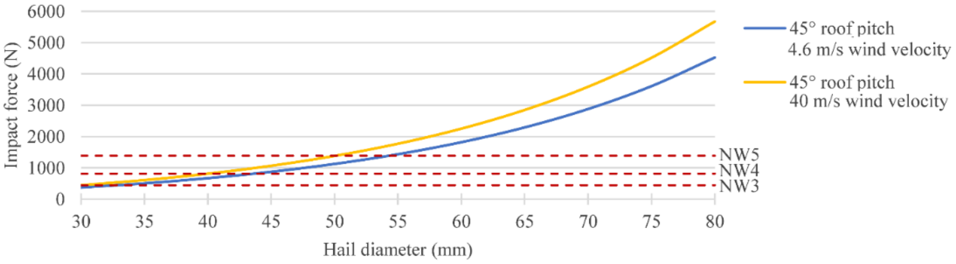


Fig. 2.38 Hailstone impact force analysis for 45° roof pitch at 4.6 and 40 m/s wind velocity compared with characteristic hailstone break resistance values

Since the size of hailstones is most likely to suffer a significant increase in the future, it can be concluded that they will heavily affect the integrity of historic roof tiles, the aesthetical value of the heritage building and in the long term the state of conservation of the timber structure.

2.2.2.2.3 Wind

High wind velocities can also significantly affect the state of conservation of roofs and roof structures. After the wind-storm of the 17th of September 2017, a significant number of roofs were affected (Fig. 2.39), bell tower roof structures even suffering complete or local failure (Fig. 2.40).

Fig. 2.39. Damage to heritage buildings - wind storm from the 17th of September 2017 in Timisoara (wind velocities of 145 m/s)

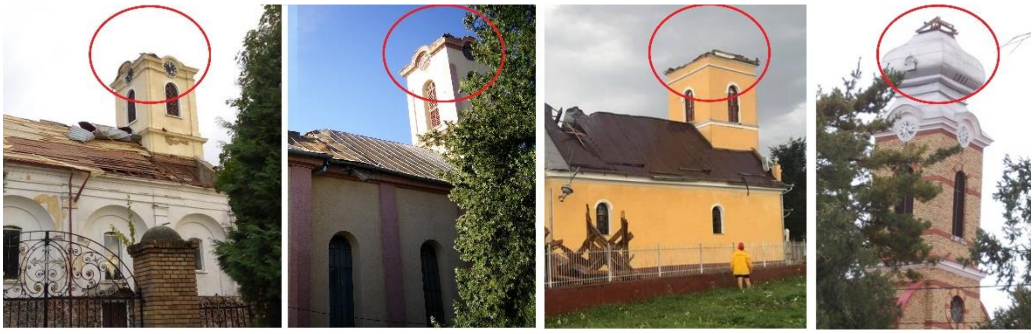


Fig. 2.40 Failure of bell tower roof structures - wind storm from the 17th of September 2017 in Timisoara (wind velocities of 145 m/s)

Considering the observed failures, a study using finite element models was performed which took the geometry of a bell tower roof structure from a church characteristic for the Banat region (Fig. 2.41), three different support scenarios and wind velocities up to hurricane force, with a wind velocity of over 32.7 m/s [112] into consideration. The scenarios were based on observations made during the analysis of bell tower roof structures and their supports in the Banat region but also considering various strengthening interventions made in order to increase the rigidity of the supports.

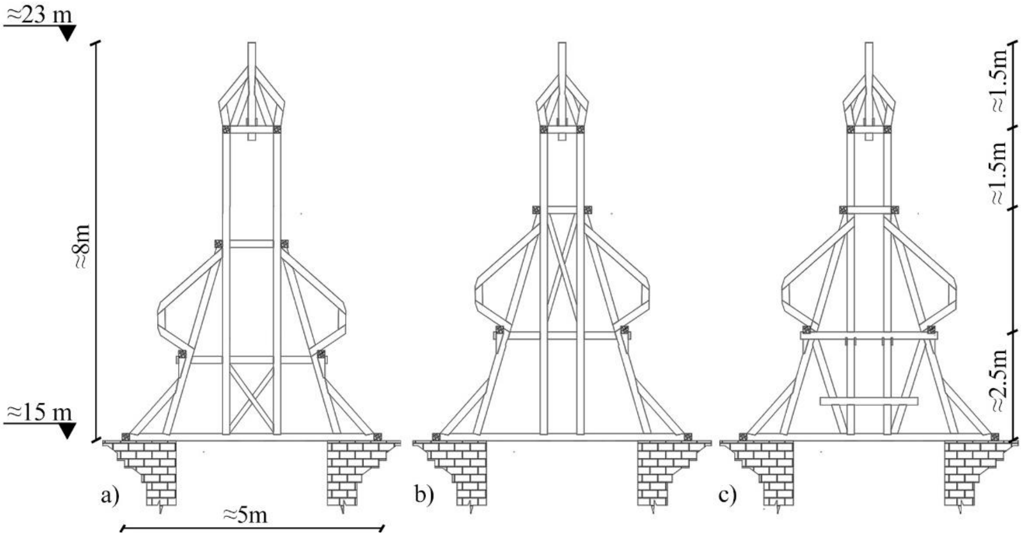


Fig. 2.41 Bell tower roof structure types and characteristic dimensions

Therefore, the first scenario considered sliding supports and had the purpose of helping determine the wind velocity, which would lead to the collapse of the roof structure. The second scenario supposed the use of rigid supports, in the case of strengthening interventions made in the area of the supports of the roof structure.

The scope was to determine how the failure of the roof would change and identify once again the wind velocity, which would lead to its failure. Since in the case of the first scenario, the vertical displacement of the support was blocked, another series of numerical simulation were performed, for a third scenario, which also supposed sliding supports but with free vertical displacement. The scope was to observe if, compared to the first scenario, the failure of the roof structure would chance (Fig. 2.42).

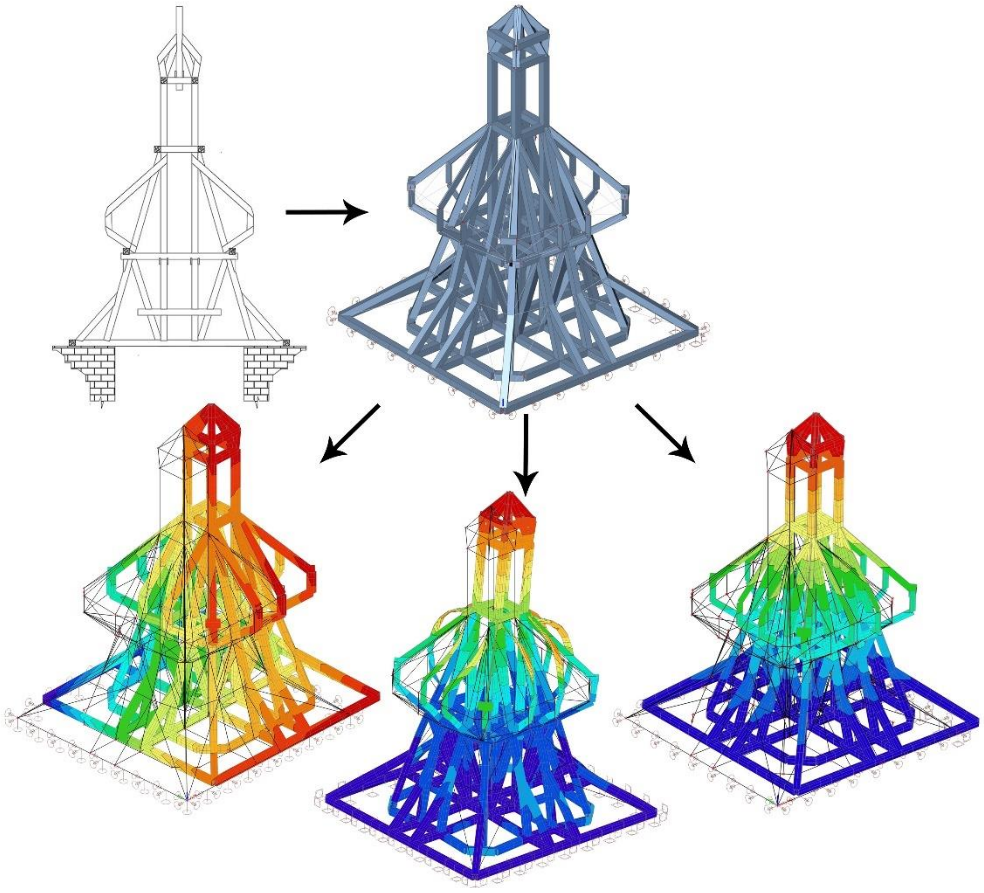


Fig. 2.42 From the geometry of the roof structure to failure analysis

The numerical analysis, performed for the same geometry of the roof structure, mechanical properties and wind velocities highlighted the importance of properly considering the support of the roof structure but also understanding the effect of different strengthening interventions performed on the supports. They showed a completely different behaviour of the roof structure under high wind velocities when comparing the horizontal displacement of its base and top (Table 2-6). The sliding supports from the first scenario cause a sliding failure of the roof structure, with only 1 mm difference between the recorded horizontal displacement at the base and the top of the roof. If the horizontal displacement of the roof structure supports is blocked, the top of the roof structure is more affected by the wind, presenting local failure in the top part despite presenting only little displacement. This is mainly caused by the rigidity differences between the base and top of the roof structure. If, however, the vertical displacement of the sliding supports is also left free, the roof structure is also presenting a clear overturning and sliding failure, and significant differences between the displacement at the base and top of the roof structure (Fig. 2.43).

If comparing the behaviour of the roof structure in the case of the first and third scenario, it can be observed that the overturning and sliding failure is happening for slightly lower wind velocities than the sliding failure. Still, both failures happen for storm-wind velocities of around 32.6 m/s.

Table 2-6 Recorded horizontal displacements of the base and top of the roof structure for the considered scenarios [112]

Scenario	Displacement [mm]		
	Horizontal base displacement	Horizontal top displacement	Difference
1st scenario	401	402	1
2nd scenario	0	1.3	1.3
3rd scenario	480	399	-81

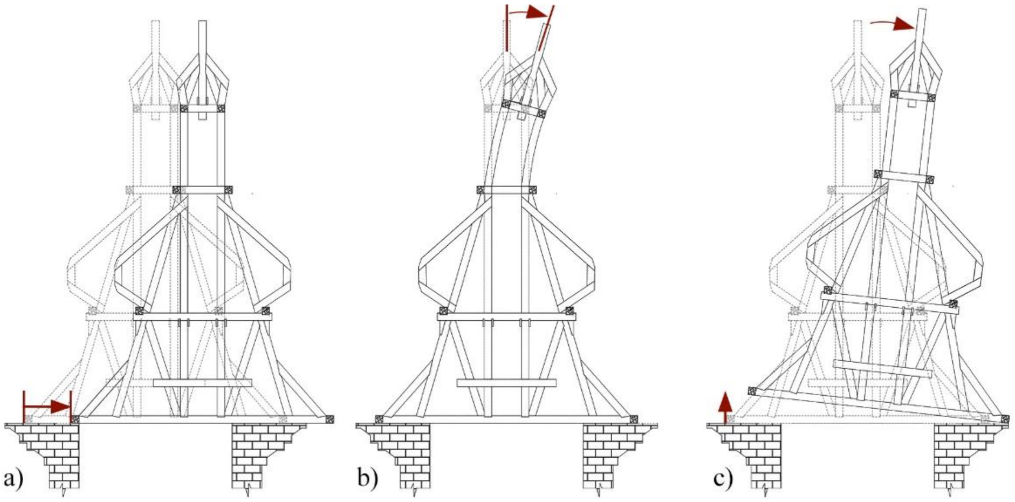


Fig. 2.43 Failure of bell tower roof structures influenced by support type (a) sliding support - sliding failure; b) rigid support – local failure at the top of the structure; c) sliding support with free vertical displacement - overturning–sliding failure) [112]

Despite being only performed on bell tower roof structures, the study highlights that high wind velocities can seriously affect the integrity of heritage roof structures and that their failure depends on the connection between the roof structure and historic masonry wall, the used structural solution, the rigidity of the joints and the state of conservation of the whole timber structure.

2.3 Conclusion

Based on the factors which are taken into consideration in other assessment methodologies and what has been observed to be relevant in defining roof and roof structures in Timisoara a list of features has been identified which should be included in a future proposed assessment methodology and procedure.

In the case of urban planning principles, it was observed that there are certain situations in which a roof is highlighted in its urban context or on the contrary, invisible and not playing any role in defining urban space. Therefore, seven new criteria have been identified:

1. Value of the urban area - which is related to the heritage value of the urban area where the assessed roof structure is placed and determines if the roof has an important role in creating an authentic context;
2. Position of the building – which influences how the building and its roof structure can be perceived from the pedestrian area, determining if the roof structure can stand out in its context or helps create a coherent urban ensemble;
3. Frontage, height of the building and urban alignment – these three features highlight the coherence of the urban context in which the roof is placed;
4. Shape of the roof – this feature is highly related to the development of urban planning principles, architectural styles and ultimately to the evolution of roofs and roof structure. The shape of the roof determines how imposing, and visible the roof structure is from the pedestrian area but also the complexity of the roof structure and its uniqueness, being one of the most important features which influence the value of the roof in defining the cityscape;
5. Pitch of the roof – the roof pitch is also determining the visibility of the roof structure and therefore its importance in defining urban space.

Architectural styles and aesthetical principles prove out to also influence the choice of roof, and also seven new criteria have been identified:

1. Period in which the building was built – which defines the value of the building at the local or national level and highlights the main aesthetical features of the period;
2. Heritage value of the building – determining the value and uniqueness of the building at an international, national or local level and therefore bringing forward their valuable architectural features and the need to preserve the building as a whole;
3. Height, main function of the building and architectural style – features which are related to the period in which the building was built. This feature also influences the role of the roof in defining the aesthetics of the building and its monumentality;
4. Roof shape – besides defining urban space, this feature is also related to the architecture of the building, the shape of the roof complementing its appearance;
5. Roof envelope – which is also related to the appearance of the roof and authentic appearance of the building.

Besides the connection to the surrounding urban space and the building, roofs and roof structures also proved out to have a high symbolic value which is defined by:

1. Ratio between the roof and the building– which highlights the symbolic link between the roof and the building and how the roof is perceived from the pedestrian area;
2. Ratio between the height and width of the roof structure and the position of important structural elements and joints – which highlight the influence of the

philosophy of the craft-guilds in defining roof structures and can help identify missing elements or altered structures;

3. Inscriptions of the craft-guilds – craftsman signs or numbering of the structural elements which mark the authenticity and historical value of the roof structure;
4. Elements with great symbolic value – which represent different symbolic or ornamental elements placed on the roof, highlighting particular functions or the importance of the building.

In the case of structural principles, starting from existing roof structure assessment methodologies, it was observed that a series of features define the structural value of a roof which are related to the general appearance of the structure, the used structural elements and its details. Therefore, seven features were considered to be relevant for a future structural preliminary visual assessment:

1. Structural type and style – which defines the complexity of the roof structure and its authenticity, while also highlighting the used structural elements and composition of the structure;
2. Construction system – which is related to the origin and main structural characteristics of the roof structure highlighting if the roof is a rafter or a purlin roof structure;
3. Truss typology – which is partially related to the previous feature, highlighting if the roof structure is using only main frames or main and secondary frames;
4. Existence of certain structural elements like tie beam, hanging device, rigidity enhancing systems or other unique structures;
5. Joint characteristics (material and type) – which also define the heritage value of the roof structure and its authenticity.

As already highlighted in previously developed assessment methodologies, it is important to determine the state of conservation of roof structures. In the case of a preliminary visual inspection, two categories of decay have been identified:

1. Decay of the exterior side of the roof (ridge, cornice, chimney and roof envelope material);
2. Decay of the timber elements which can be affected by humidity (tie-beams, compound rafters, rafters, purlins, straining beams, collar beams, counterbraces, wall-plates).

The analysis also highlights that meteorological factors like wind, hail and rain/humidity and above all climate change are a real threat for both roof structures in a good state of conservation but mainly for those already presenting signs of decay. It is, therefore, necessary to understand their effect and include them in future assessment methodologies and procedures in order to acknowledge the threat and find solutions to mitigate their effects.

At the same time, since historic buildings are a complex system where all the composing elements are interlinked and influence each other, the acknowledgement of the effect of the roof structure on the seismic behaviour of historic masonry buildings is also of great interest and should also be included in future assessment methodologies and procedures. It is therefore important to further identify how various roof structure types influence the seismic behaviour of historic masonry structures and understand how all the composing elements of the structure (timber elements, material, support and joints) and their state of conservation are influencing this effect.

2.4 Published research outcomes

The research outcomes presented in this chapter have been published in the following journals and conference proceedings:

1. I. Andreescu, A. Keller and M. Mosoarca, "Complex Assessment of Roof Structures", Procedia Engineering, ISSN: 1877-7058, vol. 161, pp. 1204-1210, 2016, DOI: 10.1016/j.proeng.2016.08.542, WOS:000387566500185. (Web of Science indexed paper)
2. M. Mosoarca, A. Keller, C. Petrus and A. Racolta, "Failure analysis of historical buildings due to climate change", Engineering Failure Analysis, ISSN: 1350-6307, vol. 82, pp. 666-680, 2017, DOI: 10.1016/j.engfailanal.2017.06.013, WOS:000413323400056. (Web of Science indexed paper, Impact factor - 2.897)
3. A. Keller, N. Chieffo, E. Opritescu, M. Mosoarca and A. Formisano, "Resilience of historic cities and adaptation to climate change", Urbanism Architecture Constructions, ISSN: 2069-0509, vol. 8(1), pp. 15-26, 2017, WOS:000388684600002. (Web of Science indexed paper)
4. I. Andreescu I., A. Keller, "Complex features in assessing historic roof structures" 3rd International Conference on Protection of Historical Constructions (PROHITECH'17), Mazzolani, F. Lamas, A. Calado, L. Proenca, J. and Faggiano, B. eds., 2017.
5. A. Keller, M. Mosoarca "A complex assessment of historic roof structures" In 4th International Conference on Structural Health Assessment of Timber Structures (SHATIS'17), Arun, G. ed., 157–168, 2017.
6. M. Mosoarca, A. Keller, C. Bocan, "Failure analysis of church towers and roof structures due to high wind velocities", Engineering Failure Analysis, ISSN: 1350-6307, vol 100, pp. 76-87, 2019, DOI: 10.1016/j.engfailanal.2019.02.046, WOS:000463165000007 (Web of Science indexed paper, Impact factor - 2.897)
7. I. Andreescu, A. Keller "Architecture as "Gesamtkunstwerk" – The Role of the Roof in Defining Architecture in the 19th and 20th Century in Timisoara", IOP Conference Series: Materials Science and Engineering, ISSN: 1757-8981, Proceedings of the 3rd World Multidisciplinary Civil Engineering, Architecture, Urban Planning Symposium (WMCAUS 2018), 18–22 June 2018, Prague, Czech Republic, vol. 471, DOI: 10.1088/1757-899X/471/7/072034, WOS:000465811803030. (Web of Science indexed proceedings paper)

3 STRUCTURAL ANALYSIS OF HISTORIC ROOF STRUCTURES – PRINCIPLES AND METHODS

Timber roof structures are highly neglected when performing assessments of historic buildings, due to their complexity, the inhomogeneous properties of the material and high uncertainties regarding the state of conservation and mechanical properties of the timber. Still, they are proof of the skills of the craftsmen and should be understood in order to ensure their protection for future generations.

3.1 Structural analysis of historic timber roof structures – state of the art

The misunderstanding of historic timber roof structures can lead to a partial or complete unnecessary replacement of the structure [113]. This is why, in order to ensure their protection, they have to be appropriately analysed from various points of view. Their behaviour is related to the mechanical properties of timber [113–116], the behaviour and stiffness of timber joints but also the geometry of the structure and composing structural elements.

3.1.1 Full-scale laboratory tests and numerical modelling

Therefore, in recent years numerous on-site assessments of historic timber elements using destructive and non-destructive tests [117–123] have been performed, as well as laboratory tests on full-scale timber roof structures and timber joints [124–129]. Due to the dimensions of roofs, only a few tests were performed on complete trusses (Fig. 3.1) while most of the tests are focusing on understanding the behaviours of several types of timber joints.

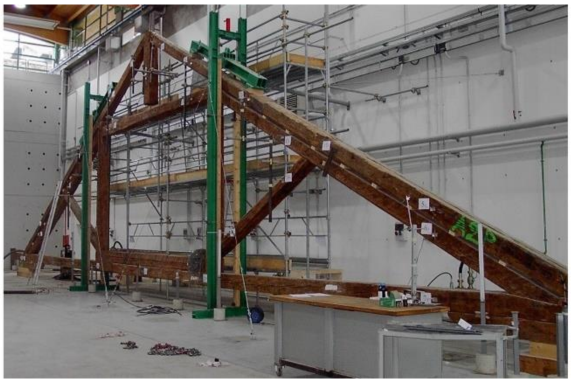


Fig. 3.1 Full-scale laboratory test setup - University of Trento [130]

Seven full-scale laboratory tests were identified which were performed on different types of trusses from Italy and Portugal:

1. Trento roof structure – Trento, Italy – one queen-post truss [131];

2. The roof structure of the "Teatro Sociale" – Trento, Italy - one queen-post truss [132];
3. The Caldonazzo Lake roof structure - Pergine Valsugana village, Italy – two king-post trusses [124,133–135];
4. Old factory roof structure - Avanca, Portugal – one queen-post truss [136];
5. Adico warehouse roof structure - Avanca, Portugal – one truss [124,137];
6. Chimico Laboratory roof structure - Coimbra, Portugal – four trusses [12,138–141];
7. General roof structure types from Italy – four king-post trusses [142].

All these tests were performed on historic timber trusses which were rebuilt in the laboratories using the same traditional techniques and subjected to vertical, symmetric or asymmetric loads, in order to understand the load transfer and their structural behaviour. At the same time, since non-destructive tests can offer vital information concerning the structural integrity and bearing capacity of the timber elements [120], all the analysed laboratory tests were preceded by preliminary non-destructive analysis of the historic timber.

At the University of Trento, laboratory tests were performed on four roof trusses, mainly focused on understanding the behaviour of the considered roof structure before and after strengthening, related to the behaviour of the timber joints under static and cyclic loads. The first tested truss [131] belonged to a historic building from the end of the 19th century. During the laboratory tests, static and cyclic loads were applied on the rafters, and the displacement of key points was recorded (Fig. 3.2). Subsequently, numerical simulations were performed in the ABAQUS 5.8 finite element software and the obtained displacements compared with the results from the laboratory test. Due to high discretisation of the non-linear model and complex modelling of the contact forces between the timber elements, the results obtained during the numerical simulations were within ±7% of the displacements recorded during the laboratory tests. Despite the conclusive results, the approach used during the numerical simulation is very time consuming and cannot be used by professionals for various projects.

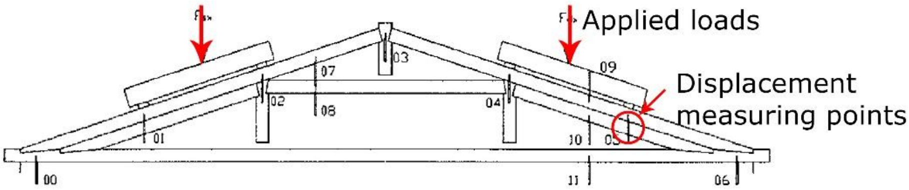


Fig. 3.2 Testing layout of the 19th century roof structure tested at the University of Trento (after [131])

The second laboratory test performed at the University of Trento [132] was performed on a truss from the 19th century Trento theatre. In this case, ten monotonic tests with symmetric and asymmetric loads were performed, and the displacement of thirty points placed in various locations of the truss recorded, as presented in Fig. 3.3. Like in the previous case, starting from the observations from the laboratory tests, numerical simulations were performed, but this time using the structural analysis program SAP 2000. For the numerical simulations, semi-rigid joints were considered with both rotational and axial stiffness using the component method. Despite being less time-consuming than the previous numerical simulations performed in ABAQUS 5.8 and considering a semi-rigid behaviour of the joints, the use of SAP 2000 led to a maximal error of about 20%.

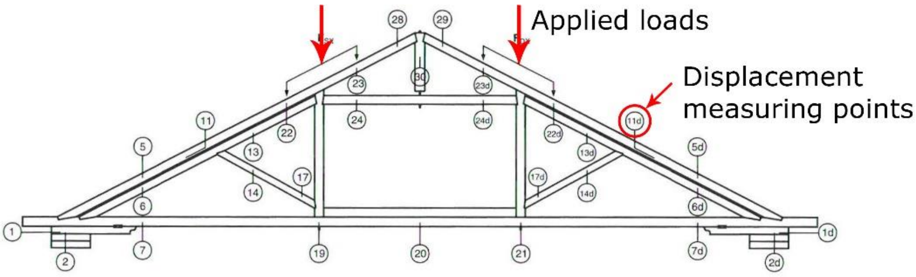


Fig. 3.3 Testing layout of the 19th century Trento theatre roof truss (after [132])

The third roof structure tested at the University of Trento belonged to a building from the Pergine Valsugana village in Italy [124,134,135]. In this case, two king-post trusses were assessed. The study also comprised a complex preliminary assessment of the trusses, a complete evaluation of the timber joints, their full-scale load bearing analysis and numerical simulations considering the obtained mechanical properties and the recorded structural behaviour.

During the load-bearing test, the roof trusses were subjected to cyclic tests with symmetric and asymmetric loads, and the relative displacement of selected points was monitored (Fig. 3.4). It was observed that both trusses presented asymmetric behaviour even when they were loaded symmetrically [135].

The subsequent numerical simulations were performed in SAP 2000, using the obtained geometric and mechanical properties of the surveyed trusses and applying the same loads as in the laboratory test. Like in the previous case, for the numerical simulations, semi-rigid joints were considered with both rotational and axial stiffness determined using the component method. Both values were later on calibrated based on the results of the laboratory test, highlighting the importance of properly considering the semi-rigid behaviour of timber joints. In this case, the displacements obtained during the numerical simulations also present a maximal error of about 20%.

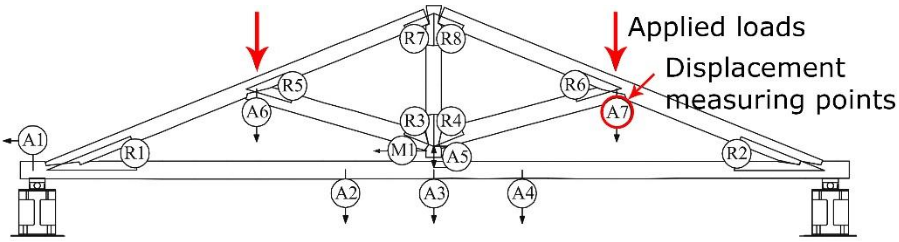


Fig. 3.4 Testing layout of the Caldonazzo Lake roof truss (after [135])

Subsequently, a series of Portuguese roof structures have been tested in different laboratories from the University of Aveiro and the University of Minho.

The roof structures tested at the University of Aveiro came from an old factory and warehouse from the Adico industry in Avanca, Portugal [136].

The first truss was a long span Howe timber roof truss with a span of 12.7 m from the beginning of the 20th century. Due to its bad state of conservation, the truss had to be brought into the laboratory without being disassembled. First, extensive non-destructive tests have been performed on the truss in order to identify the damages, their causes, the moisture content and the mechanical properties of the timber. Subsequently, the truss was subjected to a quasi-static cyclic test by applying five point-loads on the rafter above each of the five posts, where initially purlins were placed. The relative displacement and rotation of 13 points were recorded during the load-bearing tests, placed at the main joints of the truss (Fig. 3.5). As in the previous case, despite the symmetric loading, the roof structure presented an asymmetrical deformation higher at the right side of the truss. At the same time, the behaviour of the joints was assessed. The assessment highlighted the influence of joints on the behaviour of the truss and development of the recorded failure mechanism.

Numerical simulations were performed in SAP 2000, starting from the observations from the laboratory tests, modelling the variability of the cross-section, using characteristic mechanical properties for Portuguese maritime pine and applying the same loads as in the laboratory tests. Since the importance of the semi-rigid behaviour was highlighted during the tests, the axial stiffness of the joints was determined using the component method, while the rotational stiffness was ignored. Still, to obtain a similar behaviour as in the laboratory test, a complex calibration had to be performed. This was done by reducing the axial stiffness of the joints with 20%, except for the right rafter to tie-beam joint, where a reduction of only 5% was considered. The calibration led to a mean error of 4% of the displacement compared to the laboratory tests.

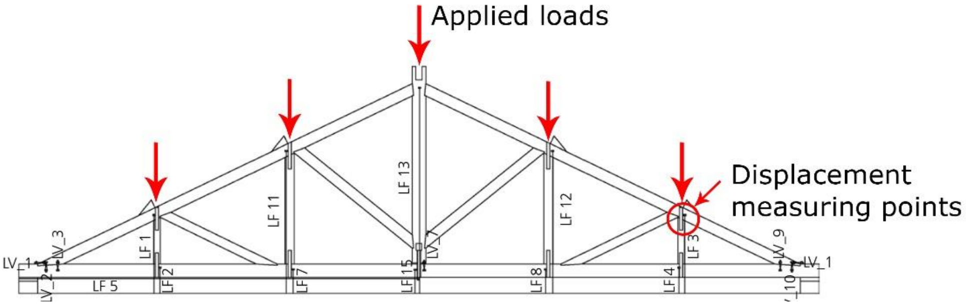


Fig. 3.5 Testing layout of the Avanca factory roof truss (after [136])

The same approach was also used for the king and queen-post roof structure from the Adico warehouse from the beginning of the 20th century [137]. Compared to the truss from the old Avanca factory, this truss presented a good state of conservation with only insignificant decay caused by insect attacks. For the laboratory tests, the same principles were used. Symmetric and asymmetric loads were applied to the rafters above the queen and king-posts, in order to assess the structural behaviour of the truss (Fig. 3.6). In this case, a second series of tests were also performed by applying loads in the areas of the purlins, which were not placed above the posts (Fig. 3.7).

Like in the previous two cases, the displacement of 8 points, representing the main joints of the truss was recorded, highlighting once again the asymmetric behaviour of the truss under symmetric loads. It was observed that the asymmetric behaviour of the truss is mainly caused by the geometry of the timber elements and the cross-section variation [137].

Numerical simulations were also performed in SAP 2000, considering the observations from the laboratory tests, using similar conditions as in the Avanca factory truss: variable cross-section of the timber elements, mechanical properties for the timber according to LNEC [143] and applying the same loads as in the laboratory tests. At the same time, the joints were considered semi-rigid, with the axial stiffness determined using the component method and the rotational stiffness, was ignored. The comparison of the displacements recorded during the laboratory tests and the numerical simulations revealed an error of about 33%. This led to a calibration of the used axial stiffness of the joints. After the calibration of the model, the error was reduced to 10% and the asymmetric behaviour of the truss obtained.

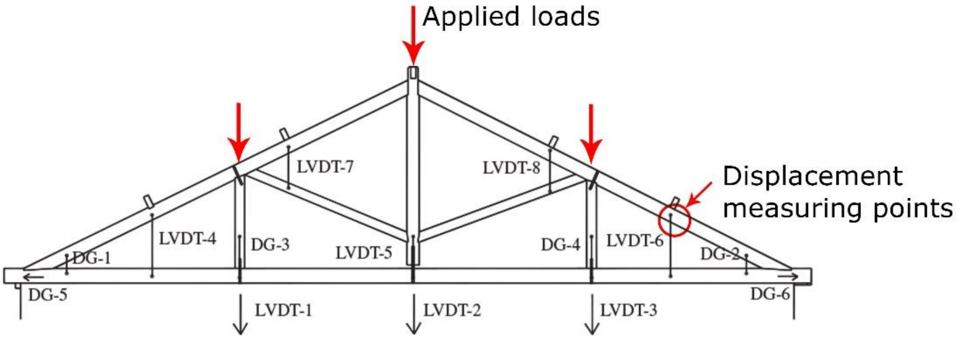


Fig. 3.6 Testing layout of the Adico warehouse roof truss – first series of tests (after [137])

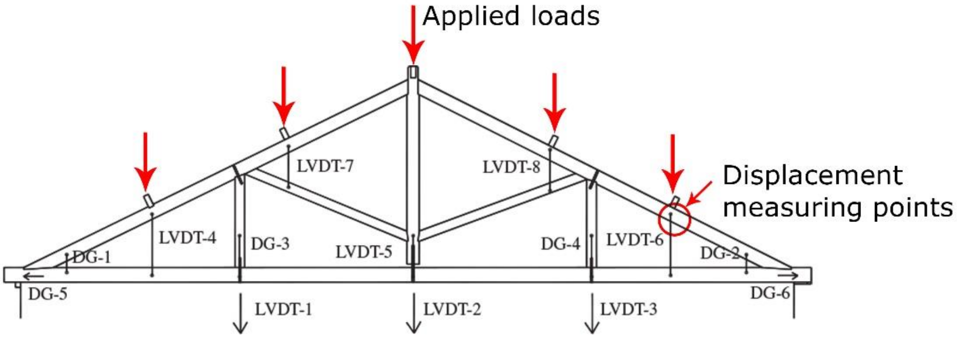


Fig. 3.7 Testing layout of the Adico warehouse roof truss – second series of tests (after [137])

The studies performed on the roof of the Chimico Laboratory present a different approach than the previously described trusses, mainly focusing on the visual inspection of the timber, the structural behaviour of the trusses and the changes of the load-bearing capacity after strengthening interventions [140]. During the first test, the loads were applied in order to lead to the failure of the timber elements of the trusses, while recording the vertical and horizontal displacement of nine key points (Fig. 3.8). In order to better understand the role of the support type on the structural behaviour of the truss, the supports were considered with free or fixed horizontal displacement. Depending on the developed failure mechanism and the area in which the failure appeared, strengthening interventions were applied, and the test resumed.

In this case, no numerical simulations were performed. Still, during the laboratory tests, the importance of the considered support typology and the connection between the timber elements on the structural behaviour of the truss was highlighted.

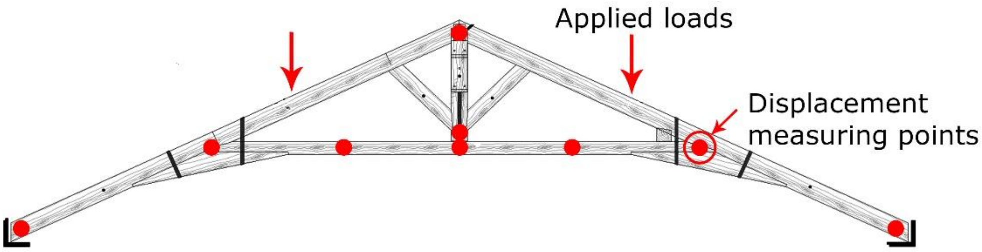


Fig. 3.8 Testing layout of the Chimico Laboratory roof truss (after [12])

At the same time, four full-scale tests were also performed at the Polytechnic University of Marche, on characteristic roof structure types from Italy, respecting the geometric features of historic trusses, but using new timber. Four different scenarios were taken into consideration, performing laboratory test on small and medium span trusses where the king post was connected to the tie beam or raised. The main scope of the study was to observe the failure of the trusses under symmetric loads, applied in the area of the former purlins, and analyse the effect of different strengthening techniques [142]. Like in the case of the Chimico Laboratory, after the first series of tests, various strengthening solutions have been applied, and the tests resumed (Fig. 3.9).

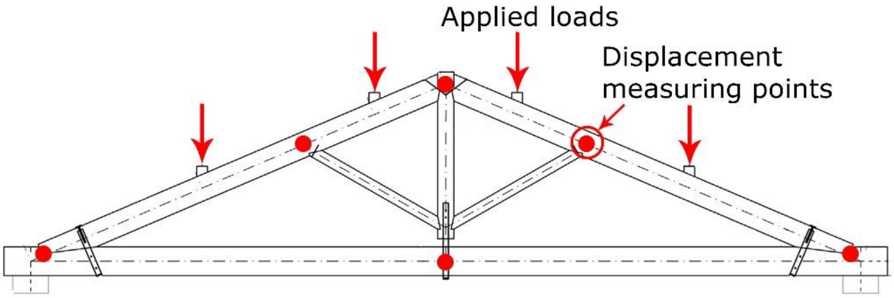


Fig. 3.9 Testing layout of the Chimico Laboratory roof truss (after [12])

These seven laboratory tests with corresponding numerical simulations brought the complex structural behaviour of historic timber roof structures forward. They highlight that despite symmetric loads, the displacement of the roof structure can be asymmetrical and difficult to predict due to variabilities of the cross-section of the timber elements (Table 3-1).

At the same time, despite common practices, where all the traditionally crafted timber joints are modelled as hinged [131], the laboratory tests and subsequent numerical simulations have highlighted that these joints have a rather semi-rigid behaviour and special attention has to be given to their proper calculation (Table 3-2). According to Koch [144], by considering a semi-rigid behaviour of the timber joints, the load-bearing capacity of the whole roof structure can be changed.

Table 3-1 Laboratory tests

Roof structure	Country	Laboratory	Loads	Timber	Recorded parameters
Trento roof	Italy	University of Trento	Symmetric	Old	Vertical and horizontal displacement
Trento theatre	Italy	University of Trento	Symmetric and asymmetric	Old	Vertical and horizontal displacement
Caldonazzo Lake	Italy	University of Trento	Symmetric and asymmetric	Old	Displacement and rotation
Avanca factory	Portugal	University of Aveiro	Symmetric and asymmetric	Old	Relative displacement and rotation
Adico warehouse	Portugal	University of Aveiro	Symmetric and asymmetric	Old	Relative displacement and rotation
Chimico Laboratory	Portugal	University of Minho	Symmetric and asymmetric	Old	Vertical and horizontal displacement
Italian roof structures	Italy	University of Marche	Symmetric	New	Relative displacement

Table 3-2 Numerical simulations performed based on laboratory test

Roof structure	Numerical simulation software	Material	Rotational stiffness	Axial stiffness	Calibration based on laboratory test	Maximal difference
Trento roof	ABAQUS 5.8	NDT	Yes	Yes	No	7%
Trento theatre	SAP 2000	NDT	Yes	Yes	No	20%
Caldonazzo Lake	SAP 2000	NDT	Yes	Yes	Yes	8%
Avanca factory	SAP 2000	Maritime pine	No	Yes	Yes	4%
Avanca warehouse	SAP 2000	According to LNEC	No	Yes	Yes	10%
Chimico Laboratory	No numerical simulation					
Italian roof structures	No numerical simulation					

3.1.2 Numerical modelling based on the features of the roof structures

Besides the multiple studies which involved the geometric survey of the timber, determination of the mechanical properties of the timber, full scale or detail laboratory tests which ultimately led to calibrated numerical simulations, a series of numerical simulations have also been performed without previous laboratory tests (Table 3-3). In the case of the Valentino Castle in Turin [11], numerical simulations have been performed on a truss based on the discretised point cloud obtained during the 3D laser scanning of the roof structure. The study is highlighting the need to use real mechanical properties for the used materials and model all the composing structural elements, including the planking system which is proving out to influence the structural behaviour of the roof structure significantly.

Numerical simulations have also been performed based on roof structures from the Czech Republic [39–41,86–88]. Despite the complexity of the assessment, the performed structural analysis using SCIA Engineer is rather conservative, using Eurocode based mechanical properties for the timber and considering the joints as hinged with rigid axial connection. The same approach has also been observed for the numerical simulations performed on roof structures from the Transylvania region [32].

Table 3-3 Performed numerical modelling of historic roof structures

Roof structure	Numerical simulation software	Material	Joints	Scope
Valentino Castle in Turin [11]	Nòlian®	NDT	No data	Role of the planking system
Holiest Christ's Body [39,88]	SCIA Engineer	C24 - STN EN 338	hinged	Static analysis – safety
Church of St. Peter of Alcantara [86,87]	SCIA Engineer	C24 - STN EN 338	hinged	Static analysis – safety
Roman-Catholic Church [40]	SCIA Engineer	C24 - STN EN 338	hinged	Static analysis – safety
Drăușeni church	PowerFrame	D35	hinged	Load-bearing capacity
Dwelling, Sibiu	PowerFrame	C22	hinged	Load-bearing capacity

3.1.3 Laboratory tests performed on timber joints and numerical modelling

Besides laboratory tests performed on full-scale timber roof structures, numerous tests have also been performed on various timber joint types, trying to understand their structural behaviour and the load transfer between the two timber elements composing the joint.

One of the most extensive experimental campaigns was conducted in 1989, by Heimeshoff and Köhler, at the University of Munich on behalf of the German Society of timber research. The study was meant to help better understand the structural behaviour of various timber joints types, commonly used in German roof structures [126]. During this campaign, 50 scarf joints, 43 lap joints and 50 notch joints were tested. The test setup supposed the use of timber elements with different cross-section. Tensile and compressive axial forces were applied on the joints, depending on their type and the recorded deformation. Ultimately, the main scope was to simplify the numerical modelling of historic timber joints and develop an equation which would help determine their axial stiffness. Subsequently, the study was extended at the Graz University of Technology on tenon and mortise joints, adapting the previously developed equations [145]. Also in this case, extensive laboratory tests have been performed.

Similar laboratory tests, but focused on a single joint type, have also been performed in recent years focusing on highlighting the semi-rigid behaviour of traditionally crafted timber joints under monotonic loads [14,146,147], cyclic loads

[148,149], or by comparing their behaviour with and without strengthening interventions [150–152].

3.1.4 Theoretical modelling of historic timber joints

Ultimately, all the laboratory tests highlight the importance of adequately addressing the semi-rigid behaviour of historic timber joints and their effect on the behaviour of the structure. The used joinery types are complex and are influenced by the knowledge and experience of the craftsman and are therefore difficult to analyse using contemporary methods.

According to the Eurocode 5 during numerical simulations of timber structures, the rigidity of the joints should also be taken into consideration. Despite this, as could be observed in the analysed performed numerical simulations, in order to simplify the analysis, they are usually modelled as perfectly hinged or sometimes rigid. However, traditionally crafted joints are neither rigid nor hinged but are presenting, as highlighted during laboratory tests, a semi-rigid behaviour.

Therefore, in order to understand the importance of the semi-rigid joints on the structural behaviour of the analysed timber truss, three different methods were identified and analysed, which are commonly used and presented in literature:

1. The component method, a method which is based on the geometric features of the roof structure joints and mechanical properties of the timber [153,154]
2. The equations developed by Heimeshoff and Köhler [14,126] which only consider the geometric features of the roof structure
3. The associated axial stiffness values determined by Hölzer [155,156], which are based on load-bearing capacity laboratory test performed on historic timber joints.

The main difference between the methods is what they consider to be important when determining the axial stiffness of a joint. The component method takes the geometric and mechanical properties of the joint into consideration, the method developed by Heimeshoff and Köhler only the joint type and its geometric properties while the method proposed by Hölzer is only considering the joint type. These principles make the main difference between the methods and ultimately the obtained results.

3.1.4.1 Component method

Despite being initially developed to determine the stiffness of steel joints, the component method was adapted and also applied on traditionally crafted joints from timber structures, considering the full-scale tests performed on historic timber structures in 1999 [148].

The main advantage of this method is that the stiffness of the joints is determined considering their geometric features but also the mechanical properties of the timber [153,154]. By using this method, the force applied on the joint can be divided into two components, one causing deformation of the base element parallel with the fibres while the second one is causing deformation of the base element loaded perpendicular to the fibres.

Therefore, the axial stiffness (k_x) of a joint can be determined using the following equation:

$$k_{ax} = \frac{E_\alpha \times A_c}{l} \tag{10}$$

Where E_α represents the elastic modulus of the timber at an α angle with the fibre, using the Hankinson Equation - Eq. (11); A_c, the compressed area of the joint which has to be determined according to the joint type, taking into consideration where the stress is assumed to be transmitted -Eq. (12); and l the notch length, where deformation due to compression is assumed to occur - Eq. (13):

$$E\alpha = \frac{E_0}{\cos^2 \alpha + \frac{E_0}{E_{90}} \sin^2 \alpha} \tag{11}$$

$$A_c = \frac{A_{insert}}{\sin \alpha} \tag{12}$$

$$l = \frac{h}{2 \sin \alpha} \tag{13}$$

Where E_0 is the elastic modulus of the timber along its fibre, E_{90} is the elastic modulus of the timber perpendicular to its fibre, A_{insert} is the cross-sectional area of the inserted timber element of the joint and h the height of the base timber element of the joint (Fig. 3.10).

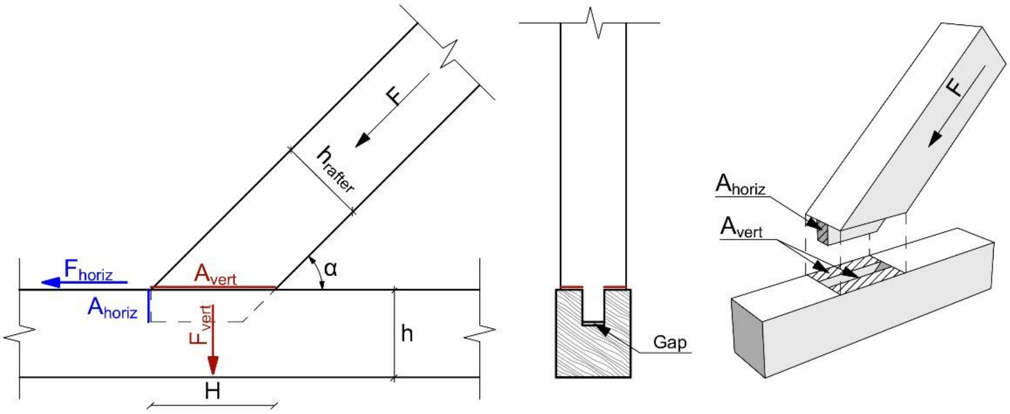


Fig. 3.10 Parameters used to determine the axial stiffness of a tenon and mortise joint

In the case of tenon and mortise joints, the vertical load is transferred through the upper contact area (A_{vert}) loaded at an α angle to the grain. A gap between tenon and mortise can be assumed so only the upper contact surface should be considered [153]. On the other hand, the horizontal load is transferred through the head of the tenon (A_{horiz}).

$$k_{horiz} = \frac{E_\alpha \times A_{horiz}}{l_{horiz}} \tag{14}$$

Where $l_{horiz} = \frac{H}{2}$

$$k_{vert} = \frac{E_\alpha \times A_{vert}}{l_{vert}} \tag{15}$$

Where $l_{vert} = \frac{h}{2}$

3.1.4.2 Heimeshoff and Köhler (1989)

In 1989 Heimeshoff and Köhler conducted an extensive experimental campaign meant to help better understand the structural behaviour of various timber joints types [126]. During the campaign 50 scarf joints, 43 lap joints and 50 notch joints were tested, with different geometric properties but using the same timber, by applying tensile axial forces on the scarf joint and compressive axial forces on the other two typologies. For each joint, the maximal applied loads and the recorded deformation of the joints were recorded with the scope to determine their axial stiffness.

Subsequently, after the tests, based on the obtained and analysed results, the authors developed an equation, which could be used to determine the axial stiffness of a historic timber joint, taking only its geometric properties into consideration:

$$k_{ax} = (45.2 - 42.1 \times sin^2\,\alpha) \times \frac{b}{12} \times (1 + \frac{t_{notch} - 2.34}{2.34} \times 0.1) \qquad (16)$$

Where k_{ax} is the axial stiffness of the joint [kN/mm]; α is the angle between the elements of the joint, b the width of the base element in cm and t_{notch} the depth of the notch in cm.

Subsequently, a research team from the Graz University of Technology [145] extended the study. They applied the same principles also on tenon and mortise joints and adapted the previously developed equation, considering the width of the compressed surface and the compressed area of the tenon:

$$b = b_{inserted\ element} - b_{tennon} \qquad (17)$$

$$t_{notch}{}^{*} = \frac{A_{horiz}}{b_{inserted\ element}} \qquad (18)$$

Where,

$$A_{horiz} = b_{inserted\ element} \times t_{notch} + b_{tennon} \times t_{tennon} \qquad (19)$$

Where $b_{inserted\ element}$ is the width of the inserted element in cm, b_{tennon} is the width of the tenon, $t_{notch}{}^{*}$ the relative depth of the notch if a tenon is also used and A_{horiz} the vertical contact area between the timber elements.

3.1.4.3 Holzer (2005)

Hölzer, on the other hand, considers that it is not possible to determine the stiffness of individual joints accurately and that the process to use the methods mentioned above, and various others which have been developed in recent years, to be rather cumbersome [156].

He therefore proposes, based on load-bearing capacity laboratory test performed on historic timber joints [14], associated axial stiffness values for each type of joint, which would be useful in an initial determination of the general structural behaviour of historic timber roof structures (Table 3-4). Only after this first check, the vulnerable joints can be identified, and further studies can be made. In this way, the effort to determine the exact axial stiffness of every joint is significantly reduced.

Table 3-4 Axial stiffness assumptions for various joint types [155,156]

Joint type	Tensile	Compression [N/mm]	
		90° angle between elements	30° angle between elements
Notch joint	0	$45*10^3$	
Tenon-Mortice joint	5/peg	$60*10^3$	$20*10^3$
Lap joint	5/peg	$60*10^3$	$20*10^3$

3.2 Analysis of historic roof structures - full-scale laboratory test and calibration

Until today no laboratory tests have been performed on full-scale historic timber roof structures in Timisoara due to their significant dimensions. Therefore, in order to obtain reliable results during numerical simulations, calibrations were performed starting from laboratory tests performed at the University of Trento. The roof structure from a building from Pergine Valsugana village near the Caldonazzo Lake in Italy was chosen for the calibration.

As described, it is one of the few full-scale timber frames of a historic roof structure which were studied during an extensive experimental load-bearing campaign [124,134,135]. The study comprised an accurate geometric, and mechanical assessment of the timber and the timber elements, a complete evaluation of the joins and a full-scale load bearing analysis of two selected trusses reconstructed in a laboratory. Ultimately a numerical simulation of the truss, considering the obtained mechanical properties and the recorded structural behaviour was made.

The roof structure is a Mediterranean king-post type from the beginning of the 20th century, presenting all the typical features from that area (Fig. 3.11). Therefore, the roof is having low height, while the timber elements are presenting significant cross-sections (Table 3-5). From a structural point of view, the frames were composed of a tie-beam, a king post, two rafters and two struts connecting the inferior part of the post with the middle of the rafters.

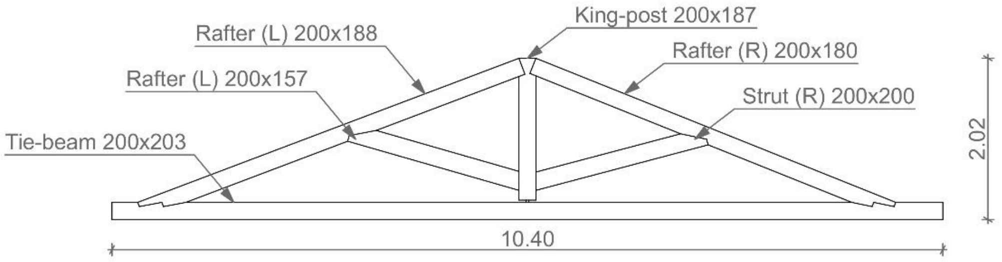


Fig. 3.11 Roof structure used for the numerical simulations

Table 3-5 Assumed cross-section of the timber elements

	Height (mm)	Width (mm)
King post	200	187
Strut (left)	200	157
Strut (right)	200	200
Rafter (left)	200	188
Rafter (right)	200	180
Tie beam	200	203

After the truss was reassembled at the "Materials and Structural Testing" laboratory of the University of Trento, in the first phase of the study, the trusses were assessed using non-destructive techniques.

First, the geometry of the timber elements was evaluated. The analysis highlighted that the proper geometric survey of historic timber structures is a sensitive matter since the edges of the timber elements are most of the time deteriorated and the cross-section of the bearing elements reduced. The trusses revealed an up to 15% reduction of their cross-section.

Subsequently, the moisture content, the material density and the mechanical properties of the used timber were determined by visual grading according to European standard EN518 [157] and Italian standard UNI 11035 [158] and by using various non-destructive test.

After determining the exact condition of the timber, the roof trusses were subjected to cyclic tests (symmetric and non-symmetric) in order to determine their structural behaviour. Two point-loads were applied on the strut to rafter joint, considering the self-weight of the original roof and snow load from the region of the Caldonazzo lake region. In the first phase, the self-weight of the roof was applied, while the additional load was later applied in 4 steps. Twelve different scenarios were evaluated during the tests, considering symmetric and asymmetric loading corresponding to the service and ultimate limit state of the roof structure (Table 3-6).

Table 3-6 Performed load-bearing test in the laboratory

Performed test	Abrv.	F_S	F_D
service limit state–unstrengthened-symmetric	S-U-S	40.5	46.2
service limit state–unstrengthened–asymmetric (higher load right)	S-U-D_x	34.2	53.7
service limit state–unstrengthened-asymmetric (higher load left)	S-U-S_x	61.9	34.9
ultimate limit state–unstrengthened-symmetric	U-U-S	82.2	84.6
ultimate limit state– unstrengthened–asymmetric (higher load right)	U-U-D_x	44.4	82.9
ultimate limit state–unstrengthened–asymmetric (higher load left)	U-U-S_x	82.8	44.9
service limit state–strengthened-symmetric	S-S-S	61.0	63.7
service limit state–strengthened–asymmetric (higher load right)	S-S-D_x	31.7	59.1
service limit state–strengthened-asymmetric (higher load left)	S-S-S_x	61.9	30.9
ultimate limit state–strengthened-symmetric	U-S-S	85.8	85.7
ultimate limit state– strengthened–asymmetric (higher load right)	U-S-D_x	41.8	83.6
ultimate limit state–strengthened–asymmetric (higher load left)	U-S-S_x	84.2	44.2

Subsequently, the displacement between the king-post and tie-beam, the connections between the struts and rafters and two additional points on the tie-beam (Fig. 3.12) was analysed. After the tests, it was concluded that both trusses are presenting an asymmetric behaviour [135], even when loaded symmetrically. This is additionally highlighting the need to assess the structural behaviour of historic timber structures accurately.

For future analysis, the tests performed for the ultimate limit state of the unstrengthened truss under symmetric loading was chosen, and the displacement of the considered points analysed (Table 3-7). For this scenario, a load of 82.2 kN was applied on the left rafter and 84.6 kN on the right.

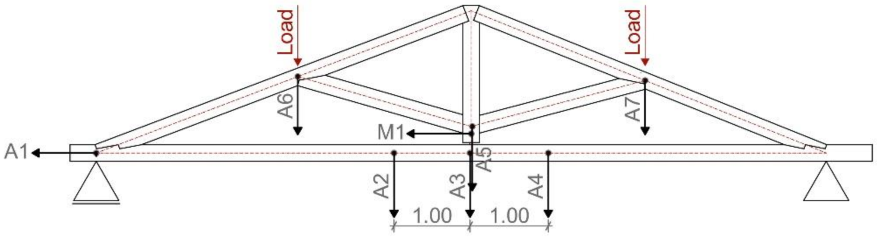


Fig. 3.12 Points where the displacement was measured

Table 3-7 Recorded displacements during the performed load-bearing test (U-U-S - Ultimate limit state-Unstrengthened-Symmetric scenario)

Node	Displacement (mm)
M1	8.1
A1	3.3
A2	13.3
A3	13.1
A4	12.1
A5	13.1
A6	21.2
A7	20

A finite element numerical simulation was made, starting from the results obtained during the experimental campaign, in order to observe if the same results could be obtained and to calibrate the laboratory test results.

3.2.1 Numerical analysis – SAP2000 – original calibration

First, the numerical simulation initially calibrated was analysed [124,135]. The structural analysis was made using the SAP2000 program, using the obtained geometric and mechanical properties of the surveyed trusses and applying the same loads as in the laboratory test. Therefore, a distributed load representing the self-weight of the timber elements and point-forces for the loads used in the performed tests were applied on the roof truss. In order to create a more accurate model, a frame of linear elements connected by nodes was modelled, and the joints of the truss were considered semi-rigid (Fig. 3.13), considering their geometric and mechanical properties (Table 3-8).

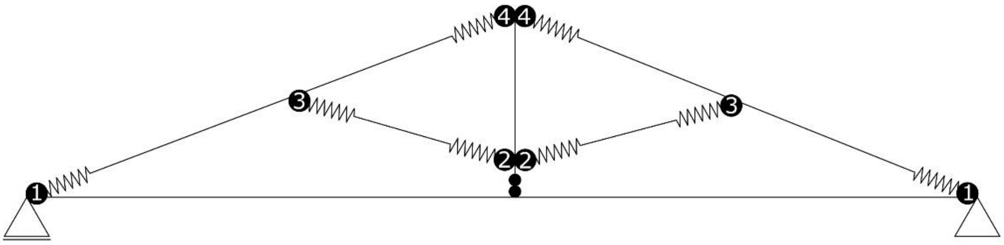


Fig. 3.13 Spring model of the assessed truss

Table 3-8 Calibrated axial and rotational stiffness of the joints of the roof truss [124]

Joint	Axial stiffness [N/mm]	Rotational stiffness [Nm/mm]
1 (L)	846,051	1500
1 (R)	788,889	1500
2 (L)	196,793	700
2 (R)	121,043	700
3 (L)	20,940	1500
3 (R)	15,952	1500
4 (L)	177,485	700
4 (R)	403,618	700

The model with calibrated semi-rigid joints proved out to present a similar behaviour as the full-scale truss, the differences between the recorded displacements being under 10% for every node (Table 3-9).

Table 3-9 Comparison of the obtained results during the laboratory tests and numerical simulation (U-U-S scenario) [124]

Node	Displacement - load-bearing test	Numerical simulation	
		Displacement [mm]	Compared to test
M1	8.1	7.6	-6.17%
A1	3.3	3.5	6.06%
A2	13.3	13.2	-0.75%
A3	13.1	12.7	-3.05%
A4	12.1	13.1	8.26%
A5	13.1	12.3	-6.11%
A6	21.2	20.7	-2.36%
A7	20	19.9	-0.50%

Considering the laboratory tests and the linear numerical simulations performed, despite being highly relevant to assess the geometric properties of the timber elements accurately, it is not necessary to also model the longitudinal cross-section variations of the timber elements [124]. At the same time, it is also highlighted that it is recommended to determine the geometry and axial stiffness of each timber joint since this property is significantly influencing the structural behaviour of the roof structure.

3.2.2 Calibrating the model with numerical analysis

Subsequently, starting from the conclusions of the calibration and the results obtained during the laboratory tests, new finite element models were created using the structural assessment program SCIA Engineer [159].

Two series of simulations were performed:

1. the first series considered the elastic modulus and density of the material as in the original calibration, determined during the non-destructive tests performed during the laboratory assessment of the truss (Table 3-10). The G modulus and the elastic modulus perpendicular to the grain were determined similar to the initially performed calibrations [124]:

$$\mathrm{G} = \frac{E_0}{16} \tag{20}$$

$$E_{90} = \frac{E_0}{30} \tag{21}$$

Table 3-10 Experimentally determined mechanical properties of the timber elements [135]

	$E_{0,mean}$ [N/mm^2]	Density [kN/m^3]	G [N/mm^2]	E_{90} [N/mm^2]
King-post	12316	4.57	769	410
Tie-beam	12183	4.32	761	406
Strut (L)	10312	3.84	644	343
Strut (R)	12156	4.50	759	405
Rafter (L)	13443	4.79	840	448
Rafter (R)	12640	4.47	790	421

2. The second series of simulations assumed a simplified approach, replacing the mechanical properties of the timber determined during the non-destructive tests with characteristic material properties for spruce (Table 3-11) according to the Eurocode - EN 338:2016 [160].

Table 3-11 Mechanical properties of spruce

Self-weight		4.9 kN/m^3
Tensile strength	$f_{t,0,k}$	104 N/mm^2
Compressive strength	$f_{c,0,k}$	47 N/mm^2
	$f_{c,90,k}$	7.7 N/mm^2
Bending-strength	$f_{m,k}$	87 N/mm^2
Shear-strength	$f_{v,k}$	10 N/mm^2
Modulus of elasticity	$E_{0.05}$	9000 N/mm^2
Mean modulus of elasticity	$E_{0,mean}$	12000 N/mm^2
	$E_{90,mean}$	400 N/mm^2

In order to also understand the effect of joints, three different scenarios were considered for each series of models (Table 3-12):

3. Model 1 – respecting the axial and rotational stiffness of the joints used to calibrate the model (Table 3-8);
4. Model 2 – using the same axial stiffness of the joints used to calibrate the model but considering no rotational stiffness;
5. Model 3 – considering various analysed joint axial stiffness determination methods. Since the proper calculation of the joint axial stiffness proved out to have a significant [influence on the structural behaviour of the roof truss, three different

methods were used and compared. The axial stiffness was therefore determined using:

5.1. the component method [154];
5.2. the equations developed by Heimeshoff and Köhler [126];
5.3. the assumed stiffness values proposed by Hölzer [155,161]
5.4. two simplified methods considering hinged or rigid joints.

Table 3-12 Used mechanical properties and joint stiffness

Model	Axial stiffness	Rotational stiffness
1	Numerical simulation [124]	Numerical simulation [124]
First series of numerical simulations – material according to non-destructive tests		
2	Numerical simulation–calibrated axial stiffness [124]	Free rotation
3.1.	Rigid	No rotation
3.2.	Rigid	Free rotation
3.3.	Theoretically determined values [124]	Free rotation
3.4.	Heimeshoff & Köhler method	Free rotation
3.5.	Hölzer method	Free rotation
Second series of numerical simulations – material according to Eurocode 5		
4	Numerical simulation–calibrated axial stiffness [124]	Free rotation
5.1.	Rigid	No rotation
5.2.	Rigid	Free rotation
5.3.	Component method	Free rotation
5.4.	Heimeshoff & Köhler method	Free rotation
5.5.	Hölzer method	Free rotation

3.2.2.1 Joint axial stiffness

First, the equations of the component method were applied to the assessed roof structure. The axial stiffness of every joint was determined and later on compared with the ones used in the original numerical simulations (Table 3-13). Subsequently, the resulted values were used in the performed numerical simulations.

The results show that the highest axial stiffness is recorded at the rafter to tie-beam joints. These joints are presenting the lowest angle between the composing elements and the highest cross-section of the timber members. All the other joints either form a wider angle between the timber elements or have a lower cross-section and are therefore presenting significantly lower axial stiffnesses.

Compared to the initially theoretically determined values, the lowest differences were recorded in the case of the rafter to tie-beam joints (1L and 1R) and the left rafter to the king-post joint (4L). For these joints, the theoretically determined values are about 10% lower than in the case of the values determined using the component method. Significant differences were observed for the strut to rafter joint axial stiffnesses which are about 3 times higher when determined using the component method.

Since the component method is considering the cross-section of the timber elements but also their mechanical properties, the observed differences are caused by two factors. First, in the case of the original theoretical determined values of the axial stiffness, the mechanical properties of the timber elements were based on the non-destructive tests performed in the laboratory. At the same time, the cross-section of the timber elements corresponded with the geometric survey of the roof truss. In the case of the calibrations performed in SCIA Engineer, the cross-section of the timber elements was assumed based on the cross-section area offered in the literature

[124,134,135] while the mechanical properties were considered according to the Eurocode.

Table 3-13 Axial stiffness determined with the component method

Joint	Joint properties					Non-destructive test material mechanical properties	Eurocode based material mechanical properties		Comparison (%)
	Base element		Inserted element		Angle [°]				
	Width [mm]	Depth [mm]	Width [mm]	Depth [mm]		Theoretical determined values k_{ax} [N/mm] [124]	E_α [N/mm^2]	Component method k_{ax} [N/mm]	
1(L)	200	203	200	188	21	846051	2367	890016	5.20
1(R)	200	203	200	180	21	788889	2367	852143	8.02
2(L)	200	203	200	157	72	196793	463	145312	-26.16
2(R)	200	203	200	200	72	121043	463	185111	52.93
3(L)	200	188	200	157	37	20940	463	145312	593.94
3(R)	200	180	200	200	37	15952	463	185111	1060.42
4(L)	200	187	200	188	68	177485	463	174004	-1.96
4(R)	200	187	200	180	68	403618	463	166600	-58.72

Subsequently, using the equations of Heimeshoff and Köhler, the axial stiffness of every joint of the assessed roof structure was also determined (Table 3-14). In this case, the rafter to tie-beam joints (joints 1L and 1R) and the strut to rafter joints (joints 3L and 1R) present the highest axial stiffnesses. The other four joints are presenting approximately similar values but lower than the ones determined for joints 1 and 3. Since this method is only considering the cross-section of the timber elements, there are no differences between the axial stiffnesses for the two considered series of simulations.

Compared to the axial stiffnesses determined with the component method, a high variation of the results was observed, the differences between the calculated values using this method are significantly lower. However, the obtained axial stiffnesses determined using the Heimeshoff and Köhler method are under 10% of the originally calibrated values.

Table 3-14 Axial stiffness determined with the Heimeshoff & Köhler method

Joint	Base element		Inserted element		Tenon/Mortise			Angle [°]	K_b	K_t	k_{ax} [N/mm]
	Width [mm]	Depth [mm]	Width [mm]	Depth [mm]	Width [mm]	Depth (step) [mm]	Depth (tenon) [mm]				
1(L)	200	203	200	188	0	50	0	21	1.67	1.11	62846
1(R)	200	203	200	180	0	50	0	21	1.67	1.11	62846
2(L)	200	203	200	157	0	50	0	72	1.67	1.11	11245
2(R)	200	203	200	200	0	50	0	72	1.67	1.11	11245
3(L)	200	188	200	157	0	50	0	37	1.67	1.11	47304
3(R)	200	180	200	200	0	50	0	37	1.67	1.11	47304
4(L)	200	187	200	188	0	50	0	68	1.67	1.11	14226
4(R)	200	187	200	180	0	50	0	68	1.67	1.11	14226

Since all the joints used for the assessed roof structure are notch joints, according to Hölzer, their axial stiffness would be 45000 N/mm. In this case, no differences between the joints are considered, and all of them are treated equally. At the same time, compared with the previously determined axial stiffness, in this case, they are significantly lower, mainly the rafter to tie-beam joint (Table 3-15).

Table 3-15 Axial stiffness considering the values determined by Hölzer

Joint	k_{ax} [N/mm]
1(L)	45000
1(R)	45000
2(L)	45000
2(R)	45000
3(L)	45000
3(R)	45000
4(L)	45000
4(R)	45000

Subsequently, the results were compared (Fig. 3.14), in order to highlight the differences between the way each method is addressing the axial stiffness of historic timber joints and compare the result with the ones used in the original numerical simulation (Table 3-16).

When analysing the axial stiffness values, it was observed that the values obtained with the component method are the closest to the ones used in the initial model of the historic roof structure. Since during the original calibration the same method was used, the values were expected to be quite similar for the rafter to tie-beam and the rafter to post joint. The main differences appear at the strut to rafter joint where, during the calibration of the original model, the values were adapted [124], and 20 times reduced, in order to obtain the same displacements as in the laboratory tests. At the same time, another factor which is influencing the results, are the used mechanical properties of the timber, since in the original model the results obtained during the non-destructive laboratory tests were used while Eurocode values were considered for the subsequent simulations.

Despite presenting lower values than the ones used in the original model, it can be observed that all the obtained values using the Heimeshoff and Köhler method are approximately 95% lower. Exceptions can be observed once again for the strut to rafter joint where the obtained axial stiffnesses are about two times higher.

Ultimately, the axial stiffnesses obtained using the Hölzer method are about 80% lower than the ones used in the model, presenting the same significant difference at the strut to rafter joint.

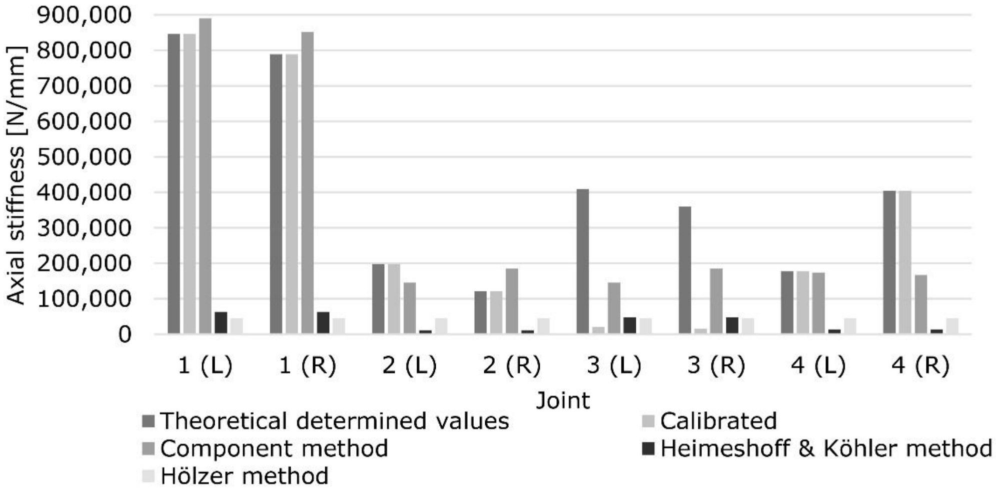


Fig. 3.14 Comparative axial stiffness values based on the used calculation method

Table 3-16 Comparison of the axial stiffness of the roof structure joints determined with the three considered methods [N/mm]

Joint	Model [124]	Component method		Heimeshoff & Köhler		Hölzer	
	k_{ax} [N/mm]	k_{ax} [N/mm]	Compared to model	k_{ax} [N/mm]	Compared to model	k_{ax} [N/mm]	Compared to model
1(L)	846,051	890,016	5.2%	62,846	-92.6%	45000	-94.7%
1(R)	788,889	852,143	8.0%	62,846	-92.0%	45000	-94.3%
2(L)	196,793	145,312	-26.2%	11,245	-94.3%	45000	-77.1%
2(R)	121,043	185,111	52.9%	11,245	-90.7%	45000	-62.8%
3(L)	20,940	145,312	593.9%	47,304	125.9%	45000	114.9%
3(R)	15,952	185,111	1060.4%	47,304	196.5%	45000	182.1%
4(L)	177,485	174,004	-2.0%	14,226	-92.0%	45000	-74.6%
4(R)	403,618	166,600	-58.7%	14,226	-96.5%	45000	-88.9%

3.2.2.2 Analysing the influence of material properties and joint axial stiffness

Starting from the mechanical properties of the timber, according to the non-destructive tests but also according to Eurocode 5 and by considering the various axial stiffness determination methods, the 13 resulting numerical models were analysed and compared to the results obtained during the laboratory tests.

The primary purpose of the different models was to identify and highlight what to take into consideration when performing numerical simulations on historic timber roof structures.

3.2.2.2.1 Non-destructive test material

The original numerical simulation performed after the laboratory test and after calibrating the axial stiffness of the timber joints, presents only little differences concerning the displacement of the chosen points. By comparing the results, it was observed that the differences reach up to 10%, recorded at the A4 point (see Fig. 3.12), located on the tie-beam. All the other joint present an increase or reduction of the displacement of about 5%.

First, a similar numerical simulation was performed in SCIA Engineer, considering the calibrated axial stiffnesses and the mechanical properties determined during the experimental campaign in order to understand if this program could be used for further numerical simulations concerning historic timber roof structures. The obtained results were mostly in the same range as the ones obtained in the initial numerical simulation. The main differences appear at the joints connected to the steel rod, where the original behaviour was not properly obtained. The obtained horizontal displacement at the lower end of the king-post presents an 85% decrease, while its vertical displacement a decrease of about 20%. All the other joins present an increase of a maximum of 5% (Table 3-17).

Table 3-17 Comparative displacement analysis of the truss with the same mechanical properties

Node	Displacement - laboratory test [124]	Numerical simulation [124]		Numerical simulation SCIA	
		Displ. [mm]	Compared to test	Displ. [mm]	Compared to test
M1	8.1	7.6	-6.17%	1.2	-85.19%
A1	3.3	3.5	6.06%	3.4	3.03%
A2	13.3	13.2	-0.75%	12.6	-5.26%
A3	13.1	12.7	-3.05%	12.9	-1.53%
A4	12.1	13.1	8.26%	12.7	4.96%
A5	13.1	12.3	-6.11%	10.2	-22.14%
A6	21.2	20.7	-2.36%	19.8	-6.60%
A7	20	19.9	-0.50%	20.9	4.50%

In order to simplify future numerical simulations, an additional model was made, which considered the same conditions as the previous one but is ignoring the rotational stiffnesses calibrated during the study, as observed in other studies (Table 3-2). The results show a slight increase in the differences compared to the original calibrated model if comparing the obtained displacements with the ones obtained during the laboratory test. In this case, the displacements are 20-25% lower for all the joints, except for the lower part of the king-post, where a decrease of 80% was recorded. The lowest difference was recorded for the vertical displacement of the left strut and rafter joint which is presenting the same displacement as in the laboratory test and the right strut and rafter joint where an increase of 15% was recorded (Table 3-18).

In order to understand if it is worth to determine the axial stiffness of the joint, two more conservative models were made, respecting the geometry of the truss and using the mechanical properties of the timber elements according to the performed non-destructive tests (Table 3-18).

First, the joints were considered rigid, except for the joint involving the steel rod which was modelled as hinged. The differences in this case, compared to the laboratory tests are higher than in the previous case, the truss suffering significantly less displacement for all the joints. The horizontal displacement of the lower part of the king-post is still presenting the highest reduction, of around 80%, while the horizontal displacement in the area of the sliding support is suffering a slight increase of 15%. The measuring points placed on the tie-beam are showing around 30% less vertical displacement, and the strut to rafter joints about 55% less. This model shows that the use of rigid joints significantly increases the stiffness of the truss and the behaviour is entirely different compared to the real one.

When considering all the joints as being hinged there is still a significant decrease of the displacements. The decrease reaches 30%, for the points on the tie-beam and around 50 up to 55% in the strut to rafter joints. The lower part of the king-post is still suffering the maximum decrease of the horizontal displacement of about 80% while the horizontal displacement of the A1 point is suffering an increase of 15%. It can be observed that the free rotation of the joints is enabling a better distribution of the applied loads in the truss. Still, since no axial displacement is possible, the results are close to the ones obtained in the model with rigid joints (Table 3-18).

Table 3-18 Comparative displacement analysis of the truss

Node	Displacement - laboratory test [124]	Numerical simulation (no rotation)		Rigid joints		Hinged joints	
		Displ. [mm]	Compared to test	Displ. [mm]	Compared to test	Displ. [mm]	Compared to test
M1	8.1	1.3	-83.95%	1.7	-79.01%	1.6	-80.25%
A1	3.3	3.7	12.12%	3.8	15.15%	3.8	15.15%
A2	13.3	9.9	-25.56%	9.0	-32.33%	8.7	-34.59%
A3	13.1	10.4	-20.61%	9.1	-30.53%	9.1	-30.53%
A4	12.1	9.9	-18.18%	8.7	-28.10%	8.7	-28.10%
A5	13.1	10.4	-20.61%	9.1	-30.53%	9.1	-30.53%
A6	21.2	21.2	0.00%	9.8	-53.77%	10.3	-51.42%
A7	20	22.5	12.50%	9.1	-54.50%	8.9	-55.50%

Subsequently, the models with axial stiffness for each joint were made and also compared with the laboratory test results, in order to understand how various axial stiffness calculation methods are influencing the behaviour of the roof truss (Table 3-19).

Two different behaviours of the truss were observed considering the three axial stiffness calculation methods. On the one hand, by using the theoretically determined values, calculated with the component method, the numerical simulation offered a similar behaviour to the calibrated model, since only the strut to rafter joint axial stiffness were different. Therefore, it was observed that the horizontal displacement of the lower end of the king-post is suffering the most significant reduction of 80% while the other joints are suffering a decrease of 15 up to 20%. An exception was observed for the strut to rafter joints which are presenting a decrease of up to 40%. Only the A1 joint is suffering an increase of horizontal displacement of about 15%.

The other two methods, on the other hand, present a clear increase of the displacement of the measuring points. A decrease of the horizontal displacement of the lower part of the king-post was still observed, of 60% in the case of the Heimeshoff and Köhler method model and 70% if using the Hölzer method to determine the axial stiffness of the joints.

The Heimeshoff and Köhler method presents only a slight increase in the horizontal displacement of the A1 joint of about 5%. The points placed on the tie-beam are suffering a significant increase of the vertical displacement of 150% while the strut to rafter joints are presenting an increase of 90% on the left and 120% on the right.

The joints with an axial stiffness determined using the Hölzer method, cause a similar behaviour of the truss, but the increases of the displacements are lower than the previous ones. Therefore, the tie beam points suffer an increase of the vertical displacement up to 100% while the strut to rafter joint displacements are 45% higher on the left and 60% higher on the right.

It can be observed (Fig. 3.15) that the low values of the axial stiffness of the joints determined using the Heimeshoff and Köhler method and the Hölzer method are leading to a significant increase of the vertical displacement of the selected points of the roof structure. The component method, on the other hand, is offering, in this case, the best results, close to the recorded displacements during the laboratory tests.

Table 3-19 Comparative displacement analysis of the truss with joints with axial stiffness

Node	Displacement - laboratory test [124]	Theoretical determined values		Heimeshoff & Köhler		Hölzer	
		Displ. [mm]	Compared to test	Displ. [mm]	Compared to test	Displ. [mm]	Compared to test
M1	8.1	1.5	-81.48%	3.3	-59.26%	2.5	-69.14%
A1	3.3	3.8	15.15%	3.5	6.06%	3.6	9.09%
A2	13.3	10.1	-24.06%	30.5	129.32%	24.4	83.46%
A3	13.1	10.7	-18.32%	32.6	148.85%	26.0	98.47%
A4	12.1	10.1	-16.53%	30.5	152.07%	24.4	101.65%
A5	13.1	10.7	-18.32%	32.6	148.85%	26.0	98.47%
A6	21.2	12.9	-39.15%	40.2	89.62%	30.6	44.34%
A7	20	12.0	-40.00%	44.1	120.50%	32.5	62.50%

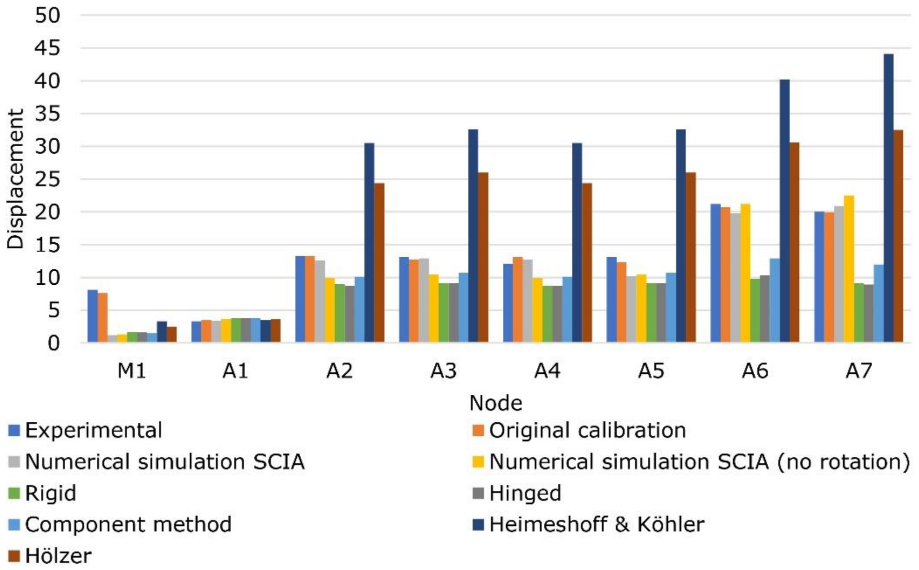


Fig. 3.15 Comparative displacement analysis of the obtained displacement of each node (non-destructive test material)

3.2.2.2.2 Eurocode 5 material

Subsequently, the second series of simplified numerical simulations were performed which considered standardised mechanical properties of spruce for the timber elements.

The first model, considered, as in the case of the first series of the numerical simulations, the axial stiffnesses of the original calibration but ignored the calibrated rotational stiffnesses from the study. In this case, the differences reach about 15% for all the joint except for the horizontal displacement of the lower part of the king-post, where a decrease of 90% was recorded. The lowest difference was recorded for the vertical displacement of the left strut and rafter joint, which is only suffering an increase of 5% (Table 3-20).

In order to understand if it is worth to determine the axial stiffness of the joint, once again two more conservative models were made, respecting the geometry of the truss but considering the joints rigid or hinged (Table 3-20).

First, the joints were considered rigid. The displacements, in this case, compared to the laboratory tests, are lower. The horizontal displacement of the lower part of the king-post is still suffering the highest reduction of the horizontal displacement, of around 80%, while the horizontal displacement in the area of the sliding support is suffering a slight increase of 5%. The measuring points placed on the tie-beam are presenting around 30 up to 40% less vertical displacement, and the strut to rafter joints about 55% less. Even in this case, the model is highlighting that the use of rigid joints significantly increases the stiffness of the truss and the behaviour is entirely different compared to the real one.

When considering all the joints as being hinged, a significant decrease of the displacement can still be observed, of 25 up to 30% for the tie-beam points and 50 up to 55% for the strut to rafter joints. The lower part of the king-post is still suffering the maximum decrease of the horizontal displacement of about 85% while the horizontal displacement of the A1 point is suffering an increase of 20% (Table 3-20).

Table 3-20 Comparative displacement analysis of the truss compared to the laboratory test results

Node	Displacement - laboratory test [124]	Numerical simulation (no rotation)		Rigid joints		Hinged joints	
		Displ. [mm]	Compared to test	Displ. [mm]	Compared to test	Displ. [mm]	Compared to test
M1	8.1	0.8	-90.12%	1.8	-77.78%	1.4	-82.72%
A1	3.3	3.7	12.12%	3.5	6.06%	3.9	18.18%
A2	13.3	10.3	-22.56%	7.8	-41.35%	9.1	-31.58%
A3	13.1	10.9	-16.79%	8.8	-32.82%	9.6	-26.72%
A4	12.1	10.3	-14.88%	7.8	-35.54%	9.1	-24.79%
A5	13.1	10.9	-16.79%	8.8	-32.82%	9.6	-26.72%
A6	21.2	22.2	4.72%	9.0	-57.55%	10.8	-49.06%
A7	20	22.4	12.00%	8.9	-55.50%	8.9	-55.50%

Subsequently, numerical simulations with joint axial stiffness were made and also compared with the laboratory test results, in order to understand how various axial stiffness calculation methods are influencing the behaviour of the roof truss (Table 3-21).

Once again, two different behaviours of the truss were observed, considering the three axial stiffness calculation methods. On the one hand, by applying the component method for the considered mechanical properties of spruce, the model offered a similar behaviour to the original calibrated model. Therefore, the horizontal displacement of the king-post lower end is suffering a reduction of 80% while the other joints are suffering a decrease of 10%. An exception was observed for the strut to rafter joints which are presenting a decrease of 35%. Only the A1 joint is suffering an increase of horizontal displacement of about 20%.

The other two methods, on the other hand, also present a clear increase of the displacement of the measuring points. The lower part of the king-post is the only point presenting a decrease of 70% for both methods.

The Heimeshoff and Köhler method presents only a slight increase of the horizontal displacement of the A1 joint of 5%, while all the other values are about 100-150% higher than the once obtained during the laboratory test. The tie-beam is presenting an apparent increase of the vertical displacement of 150% and the strut to rafter joints 95% on the left and 120% on the right.

The joints with the values determined by Hölzer, cause a similar behaviour of the truss, but the increases of the displacements are lower than the previous ones. Therefore, the tie beam points suffer an increase of the vertical displacement of 90 up to 105% while the strut to rafter joints displacements are with 50% higher on the left and 60% higher on the right.

Besides the apparent differences between the structural behaviour of the truss with the different joint axial stiffness, the most peculiar observation is how the used method is changing the asymmetrical behaviour of the truss. During the laboratory test, a higher displacement of the left strut to rafter joint was observed compared to the right one. The same behaviour was identified for most of the simulations, except for the models using the Heimeshoff & Köhler and the Hölzer method, where the asymmetry is reversed, and the right strut to rafter joint is suffering more displacement. This is mainly caused by the way the axial stiffness is approached within these two methods since they are mainly focused on the geometrical features of the joint and its type (Table 3-21).

Similar to the first series of numerical simulation, it can be observed (Fig. 3.16) that the low values of the axial stiffness of the joints determined using the Heimeshoff and Köhler method and the Hölzer method are also leading to a significant increase of the vertical displacement of the selected points of the roof structure. The component method, on the other hand, is offering, also in this case, the best results, close to the recorded displacements during the laboratory tests.

Table 3-21 Comparative displacement analysis of the truss with joints with axial stiffness compared to the laboratory test results

Node	Displacement - laboratory test [124]	Component method		Heimeshoff & Köhler		Hölzer	
		Displ. [mm]	Compared to test	Displ. [mm]	Compared to test	Displ. [mm]	Compared to test
M1	8.1	1.7	-79.01%	2.6	-67.90%	2.2	-72.84%
A1	3.3	3.9	18.18%	3.5	6.06%	3.7	12.12%
A2	13.3	10.8	-18.80%	31.1	133.83%	25.0	87.97%
A3	13.1	11.5	-12.21%	33.1	152.67%	26.6	103.05%
A4	12.1	10.8	-10.74%	31.0	156.20%	25.0	106.61%
A5	13.1	11.5	-12.21%	33.1	152.67%	26.6	103.05%
A6	21.2	14.0	-33.96%	41.7	96.70%	31.7	49.53%
A7	20	12.6	-37.00%	43.9	119.50%	32.3	61.50%

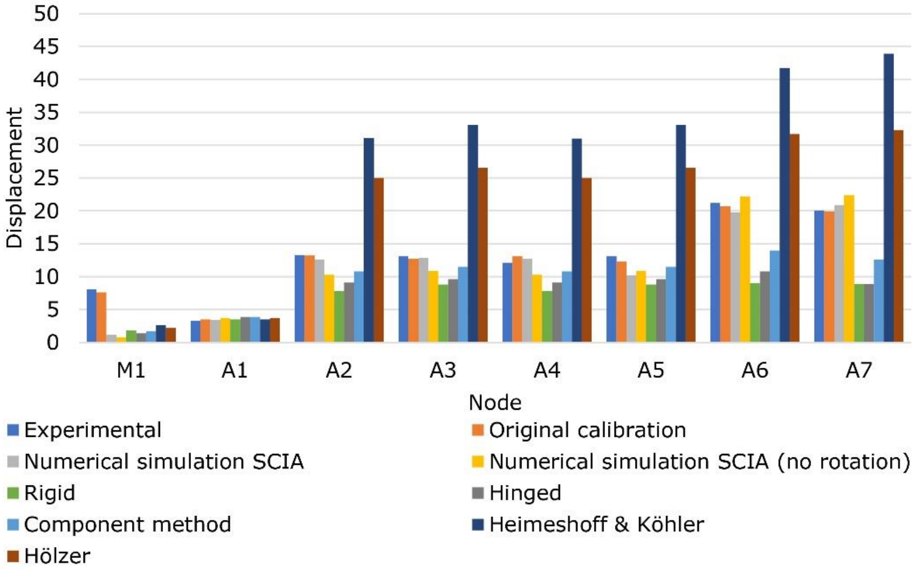


Fig. 3.16 Comparative displacement analysis of the obtained displacement of each node (Eurocode 5 material)

3.2.2.2.3 Comparative analysis based on material properties

Subsequently, after observing the importance and effect of the different considered axial stiffness calculation methods, the impact of the mechanical properties of the timber elements on the behaviour of the roof truss was analysed. The comparison was performed considering the previously performed numerical simulations, with the same geometric features of the timber elements, same axial stiffness but different mechanical properties of the timber according to the non-destructive tests and the standardised values for spruce (Table 3-22, Fig. 3.17).

In the case of the calibrated axial stiffness values, it was observed that by using Eurocode based material mechanical properties, the vertical displacement of the measuring points is increasing with about 5%, except for the right strut to rafter joint where the displacements are approximately similar. The base of the king-post, on the other hand, is presenting 40% less horizontal displacement, compared to the simulation performed with non-destructive test determined mechanical properties.

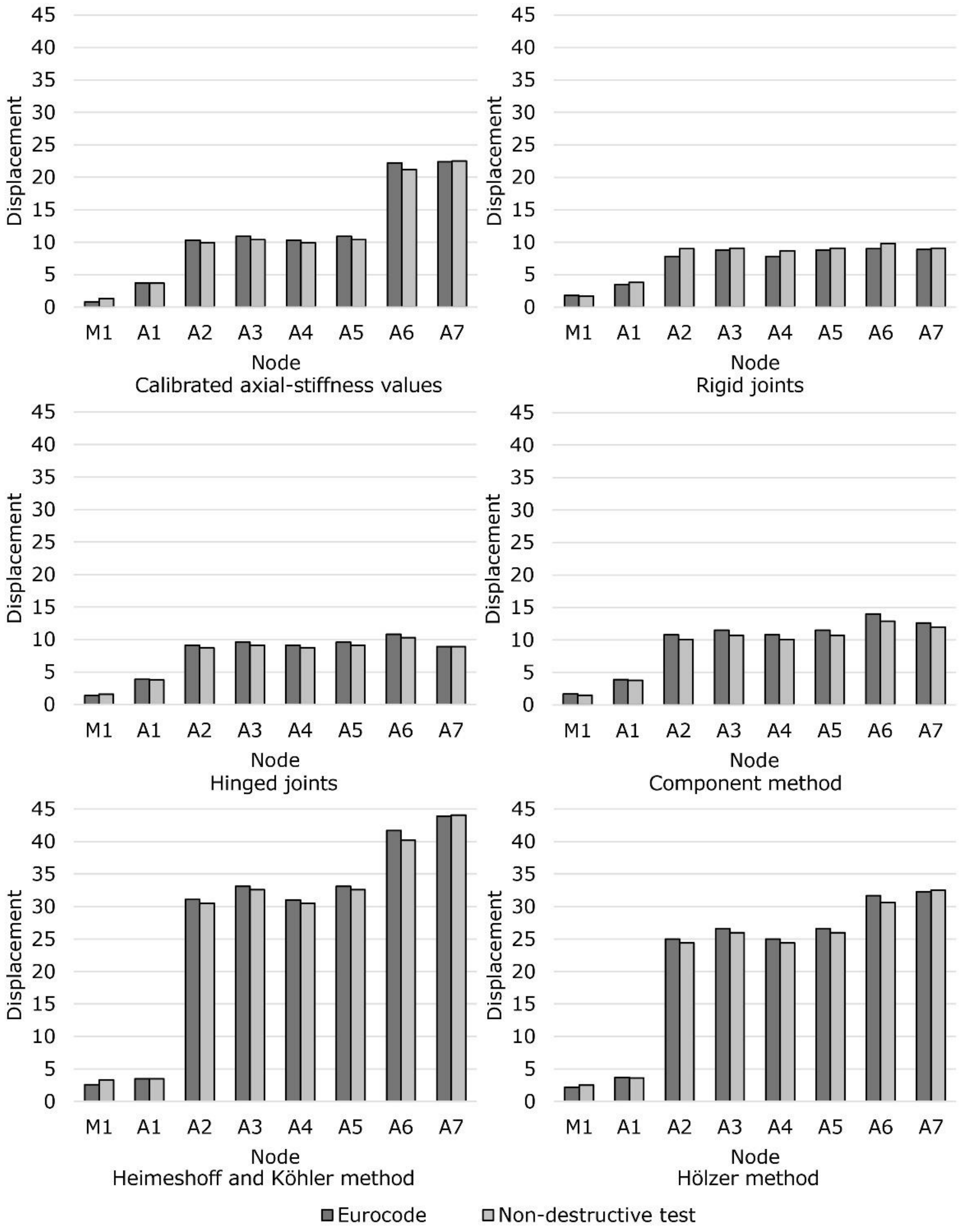
If considering the joints as being rigid, the differences between the two cases vary significantly. The decreases, in this case, range from 5%, at the base of the king-post, to 15% on the tie-beam. The strut to rafter joints present an up to 10% reduction of the displacement if standardised material properties are used. The only increase of 5% was recorded in the case of the horizontal displacement of the base of the king-post. In the case of hinged joints, the mechanical properties prove out to have a limited impact, the use of standardised materials leading to a maximum increase of the vertical displacement of the measuring points of 5%. An exception is the horizontal displacement of the base of the king-post, where a reduction of 15% was recorded.

Subsequently, the same analysis was performed for the trusses with semi-rigid joints. The differences between the three considered methods are once again apparent. In the case of the semi-rigid joints determined using the component method, the use of characteristic spruce mechanical properties is leading to a 5 up to 10% increase of the vertical displacement of the measuring points. Even in the case of the horizontal displacement of the base of the king-post, an increase of 15% was observed. The Heimeshoff and Köhler method and Hölzer method determined axial stiffness, on the other hand, present the lowest differences between the two considered cases, with an up to 5% increase of the vertical displacement if using standardised mechanical properties. At the same time, in both cases, the right strut to rafter joint is presenting a slight decrease in the vertical displacement of 0.5%. At the base of the king-post, the changes vary depending on the used calculation method, with a reduction of 20% in the case of the Heimeshoff and Köhler method and a 10% decrease if using the Hölzer method.

It can be concluded, based on the performed comparison, that in this case, the effect of the joint axial stiffness is having a more significant impact on the structural behaviour of the analysed truss than the mechanical properties of the timber elements. The minor differences are mainly caused by the good state of conservation of the analysed roof truss, and the low variation of the mechanical properties of the timber observed during the performed laboratory tests.

Table 3-22 Comparative displacement analysis of the obtained displacement of each node depending on the used material mechanical properties

Node	Displacement [mm]		Difference	Displacement [mm]		Difference
	Non-destructive test material	Eurocode material		Non-destructive test material	Eurocode material	
	Calibrated axial stiffness (no rotation)			Rigid joints		
M1	1.3	0.8	-38.46%	1.7	1.8	5.88%
A1	3.7	3.7	0.00%	3.8	3.5	-7.89%
A2	9.9	10.3	4.04%	9.0	7.8	-13.33%
A3	10.4	10.9	4.81%	9.1	8.8	-3.30%
A4	9.9	10.3	4.04%	8.7	7.8	-10.34%
A5	10.4	10.9	4.81%	9.1	8.8	-3.30%
A6	21.2	22.2	4.72%	9.8	9.0	-8.16%
A7	22.5	22.4	-0.44%	9.1	8.9	-2.20%
	Hinged joints			Component method		
M1	1.6	1.4	-12.50%	1.5	1.7	13.33%
A1	3.8	3.9	2.63%	3.8	3.9	2.63%
A2	8.7	9.1	4.60%	10.1	10.8	6.93%
A3	9.1	9.6	5.49%	10.7	11.5	7.48%
A4	8.7	9.1	4.60%	10.1	10.8	6.93%
A5	9.1	9.6	5.49%	10.7	11.5	7.48%
A6	10.3	10.8	4.85%	12.9	14.0	8.53%
A7	8.9	8.9	0.00%	12.0	12.6	5.00%
	Heimeshoff and Köhler method			Hölzer method		
M1	3.3	2.6	-21.21%	2.5	2.2	-12.00%
A1	3.5	3.5	0.00%	3.6	3.7	2.78%
A2	30.5	31.1	1.97%	24.4	25.0	2.46%
A3	32.6	33.1	1.53%	26.0	26.6	2.31%
A4	30.5	31.0	1.64%	24.4	25.0	2.46%
A5	32.6	33.1	1.53%	26.0	26.6	2.31%
A6	40.2	41.7	3.73%	30.6	31.7	3.59%
A7	44.1	43.9	-0.45%	32.5	32.3	-0.62%



Calibrated axial-stiffness values

Rigid joints

Hinged joints

Component method

Heimeshoff and Köhler method

Hölzer method

Fig. 3.17 Comparative displacement analysis of the obtained displacement of each node depending on the used material mechanical properties

3.2.2.3 Calibration

Two different behaviours were observed starting from the comparative analysis of the obtained displacement during the numerical simulations.

First, the models involving rigid, hinged or component method determined semi-rigid joints caused less displacement of the considered points. The high axial-stiffness of the joints mainly causes this in all three cases. The models involving the Heimeshoff & Köhler or the Hölzer method determined semi-rigid joints, on the other hand, are presenting significantly (even up to 150%) higher vertical displacement, highlighting that the low axial stiffness determined using these methods is leading to a more elastic roof truss and therefore to higher deformability of the timber elements.

Therefore, in order to be able to calibrate the considered models and be able to perform numerical simulations on local types of roof structures, two different approaches had to be considered. During the calibration process, the horizontal displacement of the joint between the king-post and the tie-beam (M1), connected through a steel rod, and the horizontal displacement of the support (A1) were not taken into consideration, since their displacement is highly influenced by the properties of the used steel and the timber to steel connection. Only the vertical displacement of timber to timber joints was further considered.

First, in the case of the models which presented less vertical displacement, a solution had to be found in order to decrease the rigidity of the truss and obtain similar displacements as during the laboratory tests. A reduction of the cross-section of the timber elements was therefore taken into consideration.

The cross-section of the timber elements was subsequently reduced with 10, 15 and 20% and numerical simulations performed meant to highlight how the behaviour of the roof truss is changing in each case. In the case of the models involving rigid or hinged joints, only the cross-section of the timber elements had to be changed, the joint properties remaining the same. In the case of the models involving semi-rigid joints determined using the component method, on the other hand, new axial stiffnesses had to be calculated for each new model, based on the new cross-sectional properties of the timber elements (Table 3-23).

Table 3-23 Axial stiffness determined with the component method before and after calibration

Joint	Axial stiffness, according to Eq. (10) k_{ax}[N/mm]	Axial stiffness after calibration k_{ax}[N/mm]
1(L)	890016	756513
1(R)	852143	724321
2(L)	145312	123515
2(R)	185111	157344
3(L)	145312	123515
3(R)	185111	157344
4(L)	174004	147903
4(R)	166600	141610

Ultimately, after analysing the displacement of each point, it was observed that by considering a 15% reduction of the cross-section of all the timber elements similar vertical displacements can be obtained, with the difference reaching up to 20% for most of the considered points (Table 3-24). In this case, if the joints are considered rigid, the vertical displacement of all the tie-beam measuring points is presenting an up to 5% increase, compared to the laboratory test. The two points placed on the strut to rafter joint, on the other hand, are presenting a decrease of the vertical displacement of 30%, on the left and about 35% on the right. This is highlighting that the use of rigid joints is only partially ensuring a structural behaviour close to the one obtained during the laboratory test, with close results at the bottom part of the truss, on the tie beam, but is not able to ensure similar deformation at the top part of the roof structure. The same observation can be made in the case of the hinged joints model, which is also presenting a 5% increase of the vertical displacements of the tie beam and up to 35% decrease of the strut to rafter joints displacements.

If, however, a cross-section reduction of 15% is considered and the joints are considered semi-rigid and determined using the component method, the obtained results are quite similar to the ones obtained during the laboratory test. In this case, the vertical displacements of the tie-beam measuring points, a 10 up to 20% increase were observed while the strut to rafter joint are presenting a 20% decrease. The 20% difference compared to the laboratory tests, which were obtained during the calibration process, are in the range of other performed numerical simulations and calibration, as presented in Table 3-2.

Table 3-24 Comparative displacement analysis of the calibrated model to the laboratory test results

Node	Displacement - laboratory test [124]	Rigid joints		Hinged joints		Component method	
		Displ. [mm]	Compared to test	Displ. [mm]	Displ. [mm]	Displ. [mm]	Compared to test
A2	13.3	13.2	-0.75%	12.6	-5.26%	14.7	10.53%
A3	13.1	13.3	1.53%	13.3	1.53%	15.6	19.08%
A4	12.1	12.6	4.13%	12.6	4.13%	14.7	21.49%
A5	13.1	13.3	1.53%	13.3	1.53%	15.6	19.08%
A6	21.2	14.6	-31.13%	14.8	-30.19%	19.8	-6.60%
A7	20	12.5	-37.50%	12.6	-37.00%	16.1	-19.50%

Subsequently, a second series of calibrations have been performed in order to identify parameters which have to be adjusted in the finite element models, which are using semi-rigid joints determined using the Heimeshoff and Köhler or the Hölzer method. Considering the significant differences of the obtained vertical displacements, compared to the ones recorded during the laboratory tests it was concluded that this is caused by the low axial stiffness of the joints, which is leading to a more deformable truss and therefore to a higher displacement of the measuring points.

Therefore, adjustments of the axial stiffness determine using the Heimeshoff and Köhler and the Hölzer methods was taken into consideration. In both cases the axial stiffness, originally calculated, was increased, first two times, then three times and ultimately four times reaching with each increase closer displacement to the original laboratory test. Finally, it was observed that by considering a four times (Table 3-25, Table 3-26) increase of the axial stiffness the vertical displacement of the truss are in the 20% range of the laboratory test for all the considered points (Table 3-27).

Table 3-25 Axial stiffness determined with the Heimeshoff & Köhler method before and after calibration

Joint	Axial stiffness, according to Eq. (6) k_{ax}[N/mm]	Axial stiffness after calibration k_{ax}[N/mm]
1(L)	62846	251385
1(R)	62846	251385
2(L)	11245	44980
2(R)	11245	44980
3(L)	47304	189216
3(R)	47304	189216
4(L)	14226	56905
4(R)	14226	56905

Table 3-26 Axial stiffness determined with the Hölzer method before and after calibration

Joint	Axial stiffness, according to Eq. (6) k_{ax}[N/mm]	Axial stiffness after calibration k_{ax}[N/mm]
All joints	45000	180000

Therefore, in the calibrated model using the four times increased Heimeshoff & Köhler method calculated joints it was observed that the points placed on the tie beam, are presenting a 10 up to 20% increase of the vertical displacements compared to the laboratory test. The strut to rafter joints, on the other hand, are presenting vertical displacements reaching an up to 10% decrease

In the case of the Hölzer method calibrated model, the vertical displacements on the tie beam are closer to the ones from the laboratory test, reaching an increase of 5 up to 10%. On the contrary, the displacements are suffering a reduction of 20 up to 25% on the strut to rafter joints. This is highlighting the fact that considering the same axial stiffness for all the joints and ignoring their geometrical features or mechanical properties, the global behaviour of the truss cannot be fully captured.

Table 3-27 Comparative displacement analysis of the calibrate, d model to the laboratory test results

Node	Displacement - laboratory test [124]	Heimeshoff & Köhler		Hölzer	
		Displ. [mm]	Compared to test	Displ. [mm]	Compared to test
M1	8.1	1.7	-79.01%	1.6	-80.25%
A1	3.3	3.8	15.15%	3.8	15.15%
A2	13.3	14.8	11.28%	13.2	-0.75%
A3	13.1	15.7	19.85%	14.0	6.87%
A4	12.1	14.8	22.31%	13.2	9.09%
A5	13.1	15.7	19.85%	14.0	6.87%
A6	21.2	19.2	-9.43%	16.3	-23.11%
A7	20	18.5	-7.50%	15.1	-24.50%

By considering a 20% reduction of the cross-section of the timber elements in the case of the models involving rigid, hinged or component method semi-rigid joints or by considering a four times increase of the calculated axial stiffness of the joints calculated using the Heimeshoff and Köhler or the Hölzer method, it was possible to obtain similar results to the laboratory tests, with differenced reaching up to 20% for all the considered measuring points (Fig. 3.18).

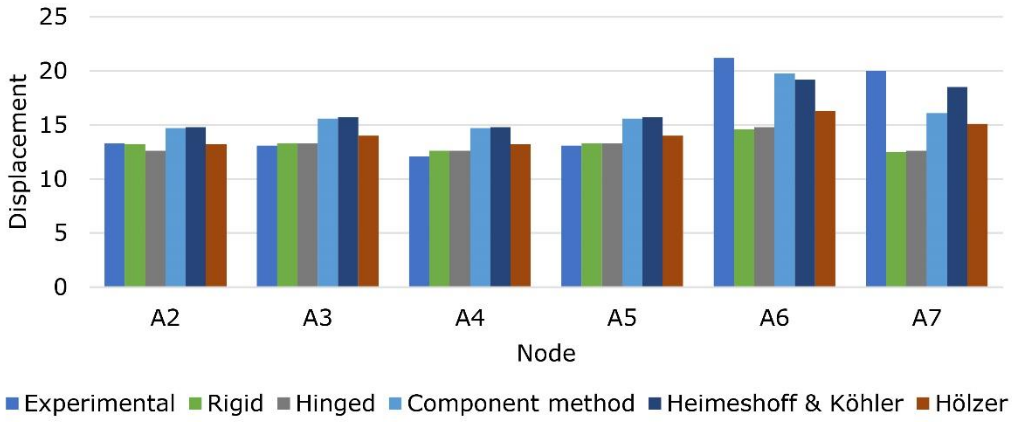


Fig. 3.18 Comparative displacement analysis of the obtained displacement of each node after calibration

3.3 Conclusions

The analysis of a laboratory test performed on a historic timber roof structure and subsequent numerical simulations have brought forward a series of approaches but also parameters which have to be taken into consideration when assessing the behaviour of this type of structure. Two important parameters were identified. On the one hand, material and cross-section related properties of the timber elements are significantly influencing the behaviour of the roof structures. It is therefore essential to perform a preliminary visual inspection of the roof structure to identify its decay and possible cross-section loss, and, if it is possible, non-destructive tests in order to determine the mechanical properties of the timber.

Timber joints, on the other hand, prove out to also significantly influence the structural behaviour of historic timber roof structures. The used joinery types are complex and are influenced by the knowledge and experience of the craftsman and are therefore difficult to analyse using contemporary methods. By analysing the numerical simulations presented in the literature, it was observed that the joints could be modelled as rigid, hinged or semi-rigid, with or without considering their rotational stiffness. In either way, they ultimately affect the structural behaviour and deformation of the roof structure.

At the same time, multiple methods used to determine the axial stiffness of traditionally crafted joints were identified. Three of these methods were further analysed and considered in future numerical simulations. The main difference between the analysed methods is the parameters which are taken into consideration. Therefore it was observed that component method is taking the geometric and mechanical properties of the joint into consideration, the method developed by Heimeshoff and Köhler only the joint type and its geometric properties while the method proposed by Hölzer is only considering the joint type.

In order to understand the importance of these factors, a series of numerical simulations were performed based on a truss tested at the University of Trento. First, the axial stiffness of the joints was determined using the three considered methods and subsequently compared with the axial stiffness values calibrated after the laboratory tests. It was observed that:

1. the axial stiffnesses obtained with the component method are the closest to the calibrated values;
2. the values obtained with the Heimeshoff and Köhler method are approximately 95% lower;
3. the axial stiffnesses obtained using the Hölzer method are about 80% lower.

Subsequently, thirteen different numerical simulations were performed on the roof truss, considering the same geometric properties, but:

4. two different mechanical properties for the timber elements:
    - 4.1. according to the non-destructive tests performed during the experimental campaign;
    - 4.2. for spruce according to the Eurocode.
5. various joint characteristics:
    - 5.1. respecting the axial and rotational stiffness of the original calibration;
    - 5.2. respecting the axial of the original calibration but ignoring the rotational stiffness;
    - 5.3. using rigid or hinged joints ;
    - 5.4. using semi-rigid joint determined using the component method, the method of Heimeshoff and Köhler and the method proposed by Hölzer.

The numerical simulations highlight the differences between the structural behaviour of the truss with the different joint axial stiffnesses. Therefore, if comparing the obtained results with the displacements recorded during the laboratory tests, it was observed that:

1. if the mechanical properties of the timber are considered according to the non-destructive tests:
    - 1.1. the use of the same conditions as in the initially performed calibration is leading to a decrease of 5% of the displacement for most of the measuring points;
    - 1.2. by ignoring the rotational stiffness, the resulted displacements are 20-25% lower in all the points;
    - 1.3. if using rigid or hinged joints, the displacements are between 30%, recorded on the tie-beam, and 50% lower;
    - 1.4. if using the component method to determine the axial stiffness of the joints the displacements decrease with up to 25%, on the tie-beam, and 40% for the other joints;
    - 1.5. if using the Heimeshoff and Köhler method to determine the axial stiffness of the joints, the displacements increase with up to 150%;
    - 1.6. if using the Hölzer method to determine the axial stiffness of the joints the displacements increase with up to 100%.
2. if the mechanical properties of the timber are considered according to the Eurocode:
    - 2.1. by ignoring the rotational stiffness, the results present a decrease of 15% of the displacement of the tie-beam and an increase of up to 10% of the strut to rafter joint displacement;
    - 2.2. if using rigid or hinged joints, the displacements are between 35%, recorded on the tie-beam, and 55% lower;
    - 2.3. if using the component method to determine the axial stiffness of the joints the displacements decrease with up to 10%, on the tie-beam, and 35% for the other joints;

  - 2.4. if using the Heimeshoff and Köhler method to determine the axial stiffness of the joints the displacements increase with up to 155%, on the tie-beam, and up to 120% for the other joints;
  - 2.5. if using the Hölzer method to determine the axial stiffness of the joints the displacements increase with up to 105%, on the tie-beam, and up to 60% for the other joints.

Still, the most peculiar observation is that depending on the used joint stiffness, the asymmetrical behaviour of the truss is changing. During the laboratory test, a higher displacement of the left strut to rafter joint was observed compared to the right one. The same behaviour was identified for most of the models, except for the semi-rigid joint determined using the methods of Heimeshoff & Köhler and Hölzer, where the asymmetry is reversed. At the same time, if comparing the results obtained, it can be concluded that the effect of the joint axial stiffness is having a more significant impact on the structural behaviour of the analysed truss than the mechanical properties of the timber elements. The minor differences are mainly caused by the good state of conservation of the analysed roof truss, and the low variation of the mechanical properties of the timber observed during the performed laboratory tests.

Subsequent calibrations of the finite element numerical models were performed based on the previous observations. It can, therefore, be concluded that:

1. if the joints are modelled as hinged or rigid:
   - 1.1. it is necessary to reduce the cross-section of the timber elements with 15% in order to a similar deformation as observed during the laboratory test;
   - 1.2. the recorded displacements during the numerical simulation are about 5% higher on the tie-beam;
   - 1.3. due to the high axial stiffness of the joints, the deformation of the rafter is up to 40% lower than the one obtained during the laboratory test.
2. if the joints are considered semi-rigid and determined using the component method:
   - 2.1. it is necessary to reduce the cross-section of the timber elements with 15% in order to obtain a similar deformation as observed during the laboratory test;
   - 2.2. the recorded displacements during the numerical simulation are about 20% higher on the tie-beam and 20% lower on the rafter.
3. if the joints are considered semi-rigid and determined using the Heimeshoff and Köhler method:
   - 3.1. it is necessary to increase the calculated axial stiffness of the joints four times in order to a similar deformation as observed during the laboratory test;
   - 3.2. the recorded displacements during the numerical simulation are about 20% higher on the tie-beam and 10% lower on the rafter.
4. if the joints are considered semi-rigid and determined using the Hölzer method:
   - 4.1. it is necessary to increase the calculated axial stiffness of the joints four times in order to obtain a similar deformation as observed during the laboratory test;
   - 4.2. the recorded displacements during the numerical simulation are about 20% higher on the tie-beam and up to 25% lower on the rafter.

Since the obtained results are in the 20% range of the displacements obtained during the laboratory test, the observations can be further used for other numerical simulations using the finite element software SCIA Engineer.

4 INFLUENCE OF THE ROOF STRUCTURE ON THE SEISMIC BEHAVIOUR OF HISTORIC BUILDINGS

Besides being connected with their urban context, the building is composed of elements which are closely linked, each one of them influencing the aesthetics but also the structural behaviour of the other. Despite this, when assessing heritage buildings, their structural behaviour and seismic vulnerability, professionals usually check the integrity of the main load-bearing structure without looking and considering the effect of the roof structure on its global behaviour.

Studies performed on the role of historic roof structures in the structural behaviour of heritage buildings highlight the fact that their presence can, depending on the type, enhance the structural behaviour of a building but can also lead to significant damage if the connection to the wall is decayed or was poorly designed [50,53,54,57,162] (Fig. 4.1). At the same time, the beneficial effect of the mechanical properties of timber inserted in historic masonry structures is highlighted in numerous studies, mainly performed in seismic areas [163–165].

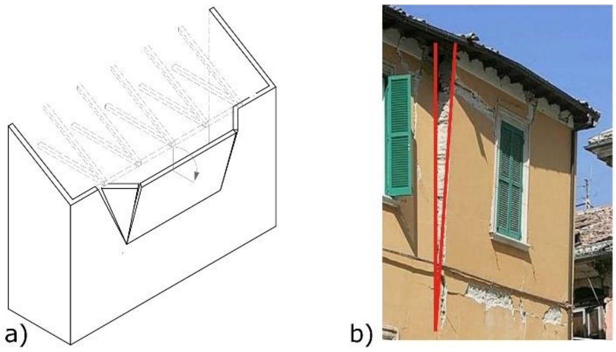


Fig. 4.1 a) Out-of-plane failure mechanism; b) Out-of-plane failure after the L'Aquila Earthquake (2009) (after [166])

Therefore, in order to also understand the link between the roof structure and the building it belongs to, an 18th-century building from the city centre of Timisoara was considered, on top of which three different types of roof structures were placed. The main scope was to understand and be able to compare the effect of these types of common roof structures from Timisoara on the seismic behaviour of the chosen historic masonry building [167,168].

4.1 Timisoara seismic area

Romania is mainly marked by shallow, crustal type earthquakes which are originating within 15-20km of the Earths exterior surface. The Banat region is considered to be the second seismic region of the country [169], with earthquakes with a low focal depth of up to 20km [65,66,170], after the Vrancea seismic zone, which is mainly affected by earthquakes with intermediate focal depth [171].

The main characteristic of the Banat seismic area is that the intensity of the earthquakes is getting lower with the distance to the epicentre, producing significant damage only around 7 up to 8 km around it and that each earthquake is followed by a high number of after-events [172]. Still, 94 seismic events with intensities between V and VIII on the Mercalli scale were recorded in this region in the last 300 years, the latest being documented in 1991, in the area of the Banloc village, about 50 km from Timisoara (Table 4-1). Since 1991 mainly II up to III on the Mercalli scale intensity earthquakes were recorded in this region [173,174].

Table 4-1 Maximum seismic activity in the Banat area and intensity on the Mercalli scale starting with the 19th century (after [172])

Seismic intensity	V	VI	VII	VII-VIII
Year	1889	1973	1879	1879
	1896		1859	1915
	1902		1900	1991
	1907		1941	
	1950		1959	

4.2 Numerical analysis of historic roof structures from Timisoara

4.2.1 Description of the 18th-century building

Built in the continuous front of an aggregate of buildings, placed close to the St. George square, the chosen historic building presents all the specific features of 18th-century buildings in Timisoara (Fig. 4.2).

It was built around an inner courtyard, with a clear hierarchy between the composing wings from both geometrical and functional point of view. Therefore, the wing facing the street, having a commercial or residential purpose, was having a ground floor and two upper floors while the wing facing the courtyard was having just one upper floor and is comprising functions which are complementary to the ones from the main wing.

From a structural point of view, the main load-bearing structure of the building was made using historic brick masonry with lime-based mortar. The thickness of the walls is decreasing with the height of the building, on the exterior from 90 cm on the underground and ground-floor, down to 60 cm at the first floor and 45cm at the second floor and the interior, from 75 cm down to 30 cm.

The floors also change with the height of the building, from historic masonry cross-vaults used on the underground and ground floor to a timber-beam flooring on the first and second floor.

Despite its historical and architectural value, the building was significantly damaged due to the lack of maintenance. Due to its position, in the historical part of Timisoara and close to the main squares of the city, the building was partially demolished, and a hotel is planned to be built in its place.

Since it was desired to evaluate the influence of the roof structures on the seismic behaviour of a historic masonry building, only the main wing was considered for the study (marked area Fig. 4.2) and the torsional effect caused by the interaction between the wings ignored.

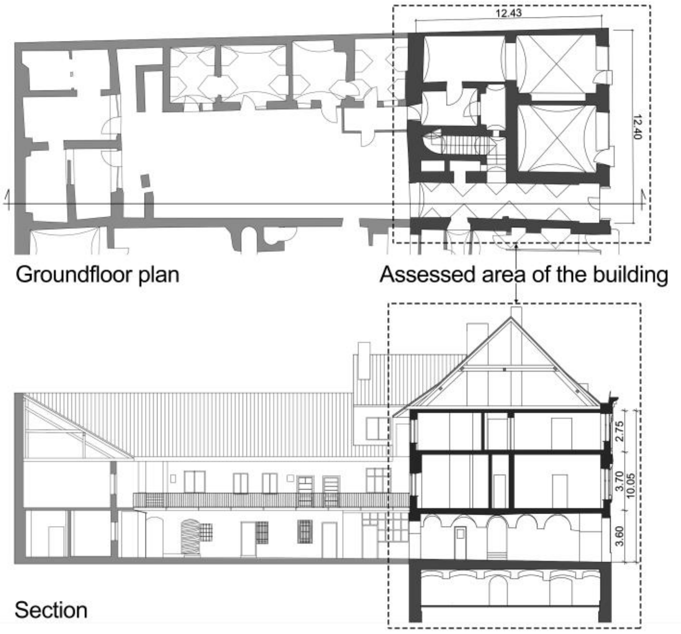


Fig. 4.2 Groundfloor plan and section of the 18th-century building [167]

4.2.2 Geometric survey of the roof structures

As presented in the historical evolution of the roof structures in Timisoara until the beginning of the 19th century, three main structural types were identified. These three types present almost entirely different approaches, being built in different phases of the history of the craft guilds, but also in different contexts.

These three roof structures were placed, in the numerical simulations, on top of the 18th-century building in order to understand how each type is influencing the seismic behaviour of the structure and if there are specific differences between the effects. It must be mentioned that from the three chosen roofs only the 18th century one belongs to the building, the other two belonging to other buildings from the city with a similar width and their layout and geometrical features were used for the numerical simulations.

4.2.2.1 Roof structure 1 - St. George square building

The first roof structure, which was considered for the numerical simulation is an 18th-century structure which is comprising all the specific elements of roof structures from that period in Timisoara.

The structure is composed of main and secondary frames which present apparent differences between each other. They were placed at an almost equal distance between each other of up to 85 centimetres. All the trusses are connected in the longitudinal direction by intermediate and ridge purlins but also by passing-braces placed in the plane of the rafters.

The main frames (Fig. 4.3a) present the specific Baroque strutting device composed of a straining beam (190x200 mm) and compound rafters (190x290 mm). They are additionally connected by two counterbraces placed on each side (175x190 mm). The exterior layer of structural elements is composed of rafters (175x150mm) connected in the central part by a collar beam (190x220 mm). The secondary frames (Fig. 4.3b) on the other hand, are only composed by the outer layer of timber elements. A specific element for Baroque roof structure is the presence of the tie beam (190x200 mm) which is placed in the inferior part of all the trusses.

This roof structure is presenting a significant cross-section of all the timber elements and a very rigid layout in both longitudinal and transversal direction.

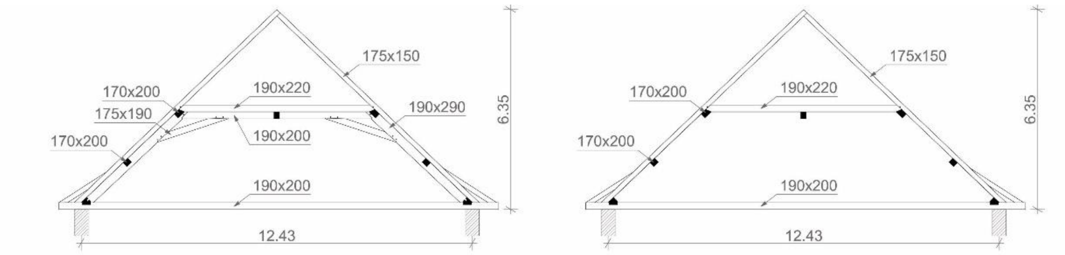


Fig. 4.3 First roof structure – main and secondary truss

4.2.2.2 Roof structure 2 – Archdukes house

The second roof structure belongs to a building which was built at the end of the 19th century, along the new street connecting the old fortress with the Fabric district. The building is the oldest one on that street and is a corner building with a hip roof. Architectural requirements from that period imposed the use of higher exterior walls, increasing the monumentality of the building and leading to adaptations of the typical roof structure with inclined posts.

The structure is also composed of main and secondary trusses which present a clear difference between each other. Therefore, the main truss (Fig. 4.4a) is presenting a high complexity of timber elements.

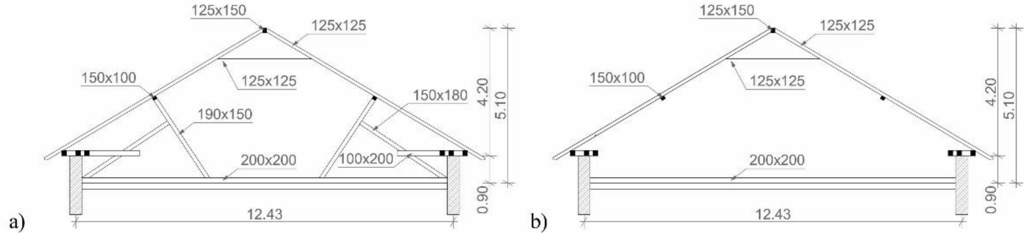


Fig. 4.4 Second roof structure – main and secondary truss

It is composed of an inclined post (190x150 mm) placed almost perpendicular to the rafters (125x125 mm). Passing braces (190x150 mm) were used to additionally connect the tie-beam (200x200 mm) to the rafters. The posts are not connected in the upper part by any collar beam, only an angle brace (125x125 mm) was used to connect the rafters at about 2/3 from their height. The secondary trusses (Fig. 4.4b) are only composed of rafters, connected by the angle brace and purlins.

In order to increase the rigidity of the structure in the longitudinal direction, all the trusses are connected by eaves and intermediate purlins (150x100 mm) but also by a more complex system, composed of headers and trimmers, placed over the walls.

The evolution of the used timber cross-section can be clearly observed, compared to the 18th-century roof. At the same time, this type is presenting a completely different approach, being composed of a multitude of triangles placed in various parts of the main truss, therefore leading to a structural type with high rigidity.

Since the chosen 18th-century building is a frontage building, for the numerical simulations, only the geometric properties of the roof structure in the gable roof part were taken into consideration.

4.2.2.3 Roof structure 3 – I.C. Bratianu high-school – wing B

The 3rd roof structure belongs to a building placed in one of the blocks from the exterior part of the former fortress which significantly changed its appearance at the beginning of the 20th century when the fortress was torn down, and the importance of that block changed. The building has a gable roof with a clear queen-post roof structure. Since the roof had to be imposing and highly visible from the street level, an additional king-post (170x170 mm) was placed above the queen post structure.

The structure is also presenting a clear difference between main and secondary trusses. The main ones (Fig. 4.5a) are presenting a hanging device, composed of two posts (180x180 mm) connected in the upper part by a collar beam (110x170 mm). A tie beam (180x180 mm) was placed at the inferior part of the queen post being also connected to exterior walls, wall plate and ridge purlin (190x185 mm). A passing brace is additionally connecting the tie-beam to the straining beam and king post. The rafters (110x170 mm) are forming the outer layer of the structure being supported by the ridge, eaves and intermediate purlins. This other layer also represents the layout of the secondary trusses (Fig. 4.5b).

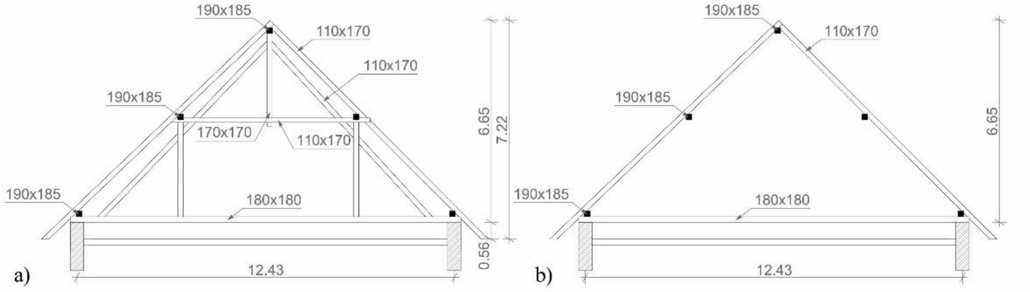


Fig. 4.5 Third roof structure – main and secondary truss

For the 19th and 20th-century roof structures, similar roofs were modelled, and placed on the chosen 18th-century building model, respecting their layout and the cross-section of the timber elements. In order to be able to compare the results, the same number of main and secondary trusses with the same distance between them was considered in the numerical simulations.

4.2.3 Material

The exact mechanical properties of historic timber are most of the times unknown since destructive tests on heritage buildings are challenging to make. At the same time, only a reduced number of laboratory tests on materials from historic buildings were performed in Timisoara.

Considering this, a more conservative approach was thought to be suitable, and a minimum strength class was used for the timber elements in the numerical simulations, chosen according to EN338 [160] (Table 4-2). The mechanical properties for the historic masonry walls were chosen for historic brick masonry with lime-based mortar, according to the national standard (Table 4-3).

Table 4-2 Mechanical properties of the used timber

Self-weight		5.7 kN/m³
Tensile strength	$F_{t,0,k}$	11.00 N/mm²
Compressive strength	$f_{c,0,k}$	18.00 N/mm²
	$f_{c,90,k}$	4.80 N/mm²
Bending-strength	$f_{m,k}$	18.00 N/mm²
Shear-strength	$f_{v,k}$	3.50 N/mm²
Modulus of elasticity	$E_{0.05}$	9500 N/mm²
Mean modulus of elasticity	$E_{0,mean}$	8000 N/mm²
	$E_{90,mean}$	630 N/mm²
Shear-modulus	G_{mean}	590 N/mm²

Table 4-3 Mechanical properties of the used brick and limestone mortar historic masonry

Self-weight		18 kN/m³
Modulus of elasticity	$E_{0.05}$	750 N/mm²
Compressive strength	$f_{c,0,k}$	1.5 N/mm²
Partial safety factor	γ_M	1.00
Shear-strength	$f_{v,k}$	0.2 N/mm²
Flexular-strength	$f_{x,k1}$	0.18 N/mm²
	$f_{x,k1}$	0.36 N/mm²

4.2.4 Loads

In order to better understand the effect of the seismic action on the historic masonry building and the influence of the three chosen roof structures, a seismic load combination was considered in the numerical simulations. This combination was meant to simulate a severe condition, combining the seismic action with the self-weight of the structure, the dead and live load but also with the specific snow load of the region.

$$Comb. = Self\ weight + Dead\ loads + 0.4 \times Live\ loads + 0.4 \times Snow + Earthquake \qquad (22)$$

The seismic response spectrum used for the performed simulations was determined according to the Romanian Seismic Design Code [175] (Fig. 4.6) using the characteristic seismic spectrum parameters for the Banat region (Table 4-4).

Since the structure was built using unreinforced brick historic masonry with lime-based mortar, the behaviour factor 1.50. Despite this, since the building has two upper levels and has a slightly irregular shape, both in-plane and in elevation, the Romanian Seismic Design Code [175] is recommending to multiply the specific value with 1.10. Therefore, a behaviour factor of 1.65 was considered in the numerical simulations.

Table 4-4 Characteristic seismic spectrum parameters in the Banat region

Peak ground acceleration	a_g	0.20g
Dynamic amplification factor of horizontal acceleration	β_0	2.5
Lower limit of the period of the constant spectral acceleration branch	T_B (s)	0.14
Upper limit of the period of the constant spectral acceleration branch	T_C (s)	0.70
Beginning of the constant displacement response range of the spectrum	T_D (s)	3.00
Behaviour factor	q	1.65

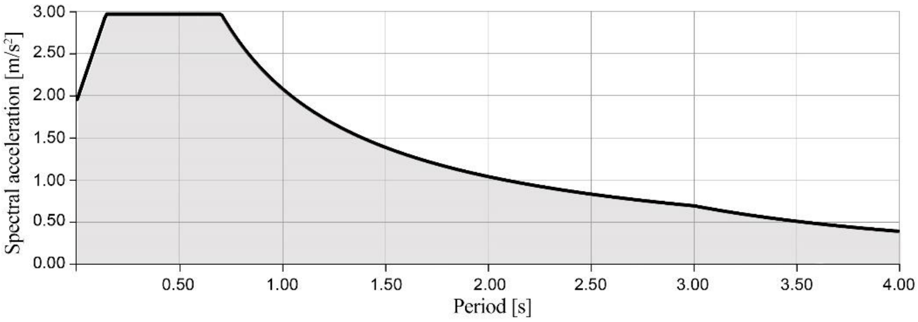


Fig. 4.6 Seismic response spectrum for Timisoara

The self-weight of the historic masonry structure, the cross-vaults, timber slabs and the roof was determined automatically by the software, based on the density of the material. Additional dead-load was applied on all the slabs, representing the finishing layers. This load was calculated by hand, based on the thickness of each layer and the density of the material.

Considering the primary functional purpose of the building, for the ground floor, the two upper floors and the attic floor the live-loads were considered 1.50 kN/sqm, while on the exterior layer of the roof a live-load of 0.5 kN/sqm was applied.

The snow-load was determined according to the national code CR 1-1-3/2012 [176], considering the characteristic values of the ground snow load, geometric properties and shape of the gable roof, its slope and the exposure of the roof considering the position of the building in the urban area. Since the three roofs have different slopes, different values of the applied snow-loads were obtained (Table 4-5).

Table 4-5 Applied loads on the roof structure

	Dead load	Live load	Roof slope	Snow load
Roof structure 1	0.57 kN/sqm	0.5 kN/sqm	43°	0.68 kN/sqm
Roof structure 2			30°	1.17 kN/sqm
Roof structure 3			46°	0.55 kN/sqm

4.2.5 Joint axial stiffness

Considering the results obtained during the first numerical simulations of the roof truss assessed in the laboratory of the University of Trento, it was considered of utmost importance also to understand the effect of the three axial stiffness calculation methods on the structural behaviour of roof structures from Timisoara. Compared to the one assessed during the experimental campaign, which was of Mediterranean influence, the ones which were used for this study are of western influence and present an entirely different structural typology.

For all the three roof structures, the axial stiffness of every joint was determined by using each method, defined in chapter 3, before the numerical simulations were performed. In order to be able to apply the calculation methods, a comprehensive study of the used timber joints, their geometry and the used joint type was made.

For the 18th-century roof structure, the traditional carpentry joints are mainly tenon and mortise joints, presenting no use of any steel fasteners. The stub mortise and tenon joints were reinforced with wooden dowel pins [168].

For the spring model, seven different joints were identified in this roof structure type. Due to the good state of conservation and insignificant cross-sectional differences between the timber elements, the roof structure was considered to be symmetrical, and no differences were made between left and right joints (Fig. 4.7).

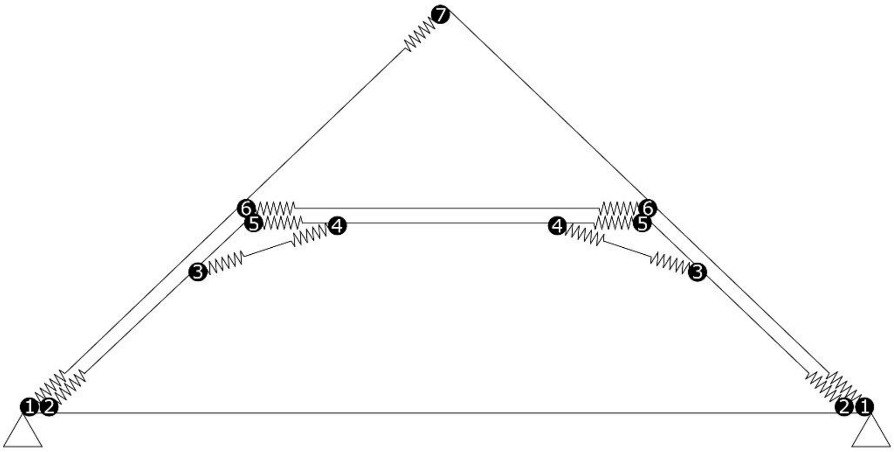


Fig. 4.7 Spring model - first roof structure

First, the component method was used to determine the axial stiffness of the joints (Table 4-6).

Table 4-6 First roof structure – axial stiffness determined with the component method

Joint	Base element		Inserted element		Angle [°]	E_α (N/mm2)	Horizontal stiffness k_h [N/mm]	Vertical stiffness k_v [N/mm]	k_{ax} [N/mm]
	Width (mm)	Height (mm)	Width (mm)	Height (mm)					
1	190	200	175	150	43	1,223	43,231	313,693	316,658
2	190	200	190	290	43	1,223	24,278	658,457	658,904
3	190	290	175	190	25	2,752	69,031	995,435	997,826
4	190	200	175	190	18	4,319	54,640	3,098,434	3,098,916
5	190	290	190	200	43	1,223	51,044	313,178	317,311
6	190	220	175	150	43	1,223	47,554	285,176	289,113
7	175	150	175	150	90	600	23,333	140,000	141,931

The influence of the geometry of the joint and the angle between the elements composing the joint was observed. Therefore, the highest axial stiffness was obtained for both joints of the counterbrace, the maximum being recorded in the upper part of the counterbrace forming only an 18-degree angle with the straining beam. Except for the inner rafter to tie beam joint, which is having a significant axial stiffness due to the increased cross-section of the inner rafter, all the other joint have an approximately similar axial stiffness.

Subsequently, the Heimeshoff and Köhler method was used to determine the axial stiffness of the joints (Table 4-7). In this case, more uniform values were obtained, mainly since only the geometry of the joint is taken into consideration. The joints connected to the tie beam are presenting the highest values, due to the increased cross-section of the timber elements while the rafter to rafter joined is presenting the lowest axial stiffness.

Table 4-7 First roof structure – axial stiffness determined with the Heimeshoff & Köhler method

	Base element		Inserted element		Tenon/Mortise						
Joint	Width [mm]	Depth [mm]	Width [mm]	Depth [mm]	Width [mm]	Depth (step) [mm]	Depth (tenon) [mm]	Angle [°]	K_b	K_t	k_{ax} [N/mm]
1	190	200	175	150	50	0	60	43	2.08	1.05	50,421
2	190	200	190	290	50	0	60	43	2.33	1.03	56,442
3	190	290	175	190	60	20	40	25	0.96	1.04	34,111
4	190	200	175	190	60	20	40	18	0.96	1.04	37,278
5	190	290	190	200	60	40	70	43	1.08	1.17	26,360
6	190	220	175	150	50	0	50	43	1.04	0.96	25,113
7	175	150	175	150	50	0	50	90	1.04	0.96	3,039

When comparing the results obtained with the three methods, the differences concerning the two approaches become highly visible (Table 4-8). On the one hand, high axial stiffnesses were obtained using the component method while approximately similar, but lower results were obtained using the other two methods.

On the other hand, it was observed that the maximum and the minimum axial stiffness was not determined for the same joints. The component method is considering the maximum axial stiffness to be at the counterbrace joints, while the Heimeshoff and Köhler method is considering the joints connected to the tie-beam to be rather stiff. The minimum, on the other hand, was recorded at the rafter to rafter joint by the first two methods while the Hölzer method is considering the counterbrace joints to be less stiff, due to the low angle between the elements composing the joint.

Table 4-8 First roof structure – comparison of the obtained joint axial stiffnesses [N/mm]

Joint	k_{ax} (Component method)	k_{ax} (Heimeshoff & Köhler)	k_{ax} (Hölzer)
1	316,658	50,421	60,000
2	658,904	56,442	60,000
3	997,826	34,111	20,000
4	3,098,916	37,278	20,000
5	317,311	26,360	60,000
6	289,113	25,113	60,000
7	141,931	3,039	60,000

The same principles were subsequently applied on the second roof structure, and a spring model developed (Fig. 4.8). The second truss is presenting eight different joints which are partially tenon and mortise joints, but also lap joins.

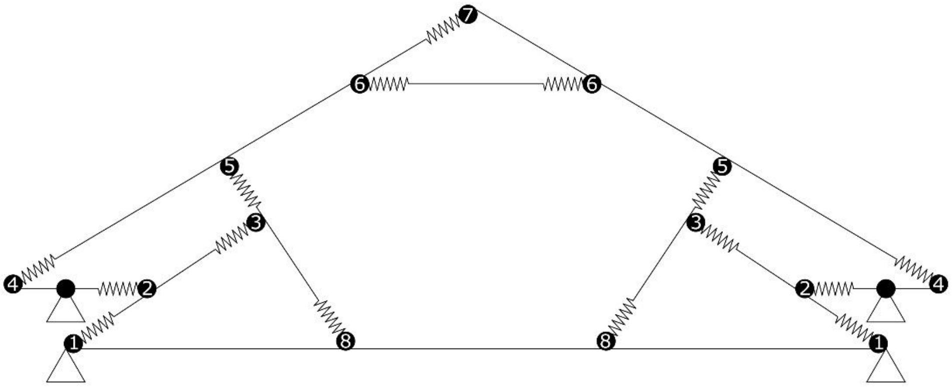


Fig. 4.8 Spring model – second roof structure

Compared to the 18th-century roof, where the timber elements had a high diversity of cross-section, which significantly influenced the axial stiffness of the joint, the cross-sections are approximately the same, with only little differences. This is also influencing the axial stiffness of the joint, the differences between the obtained results being much lower than for the first roof.

In this case, the main differences between the axial stiffnesses of the joints are caused by the angle between the composing timber elements of the joint and ultimately the angle of the transferred loads to the grain of the base timber element (Table 4-9). Therefore, it can be observed that the maximum axial stiffness was obtained for the joint made by the passing brace and tie-beam (joint 1), by the rafter and the trimmer (joint 4) and by the angle brace and rafter (joint 6). The minimum axial stiffness was recorded for the post to rafter joint, which is presenting an almost 90-degree angle between the two elements (joint 5).

Table 4-9 Second roof structure – axial stiffness determined with the component method

Joint	Base element		Inserted element		Angle [°]	E_α (N/mm2)	Horizontal stiffness k_h (kN/m)	Vertical stiffness k_v (kN/m)	k_{ax}
	Width (mm)	Height (mm)	Width (mm)	Height (mm)					
1	200	200	150	180	33.6	1,767	36,218	574,774	575,914
2	150	180	100	200	33.6	1,767	159,037	354,799	388,812
3	150	180	180	150	90	600	28,800	120,000	123,408
4	180	180	125	125	30.7	2,026	41,375	459,308	461,168
5	180	150	125	125	87	602	20,025	83,666	86,029
6	125	125	125	125	30.7	2,026	28,733	661,404	662,028
7	180	150	125	125	59.3	798	22,873	128,902	130,916
8	200	200	180	150	56.4	847	37,622	183,023	186,850

Since the Heimeshoff and Köhler method is only considering the cross-section of the timber elements, the obtained axial stiffnesses using this method are significantly lower than the ones obtained with the component method (Table 4-10).

In this case, the maximum axial stiffness was obtained for the passing brace to tie-beam joint (joint 1) since the angle between the elements is low and the cross-section of the timber elements significant. The lowest axial stiffness, on the other hand, was also obtained for the post to rafter joint (joint 5) due to the reduced cross-section of the rafter, closely followed by the passing-brace to post joint (joint 3).

Table 4-10 Second roof structure – axial stiffness determined with the Heimeshoff & Köhler method

	Base element		Inserted element		Tenon/Mortise						
Joint	Width [mm]	Depth [mm]	Width [mm]	Depth [mm]	Width [mm]	Depth (step) [mm]	Depth (tenon) [mm]	Angle [°]	K_b	K_t	k_{ax}
1	200	200	150	180	50	0	60	33.6	1.67	1.07	50,925
2	150	180	100	200	50	180	0	33.6	0.83	2.44	27,036
3	150	180	180	150	50	0	60	90	1.08	0.97	3,162
4	180	180	125	125	40	20	60	30.7	0.71	1.07	22,925
5	180	150	125	125	40	0	60	87	0.71	0.98	2,145
6	125	125	125	125	40	0	40	30.7	0.71	0.95	22,808
7	180	150	125	125	40	0	60	59.3	0.71	0.98	9,390
8	200	200	180	150	60	0	60	56.4	1.00	0.99	15,067

When comparing the results obtained with the three methods, the differences concerning the two approaches are highlighted once again (Table 4-11). Like for the first roof structure, high axial stiffnesses were obtained using the component method while approximately similar, but lower results were obtained using the other two.

At the same time, it was observed that despite the two different approaches, the maximum and the minimum axial stiffness was recorded at the same joints for both the component method and the Heimeshoff and Köhler method. Still, the values obtained with the Heimeshoff and Köhler method are at least ten times lower than the ones obtained with the component method. Since with the Hölzer method, the axial stiffness is only based on the joint type and angle between the elements forming the joint, the axial stiffnesses are entirely different with low values for low angles and high values for angles closer to 90 degrees.

Table 4-11 Second roof structure – comparison of the obtained joint axial stiffnesses

Joint	k_{ax} (Component method)	k_{ax} (Heimeshoff & Köhler)	k_{ax} (Hölzer)
1	575,914	50,925	20,000
2	388,812	27,036	20,000
3	123,408	3,162	60,000
4	461,168	22,925	20,000
5	86,029	2,145	60,000
6	662,028	22,808	20,000
7	130,916	9,390	60,000
8	186,850	15,067	60,000

The third roof structure, being built at the beginning of the 20th century, is similar to the second one, presenting nine different joints which are either tenon and mortise joints or lap joints (Fig. 4.9). Since this roof structure is already presenting a more economically efficient way of building, the differences between the cross-section of the timber elements are low.

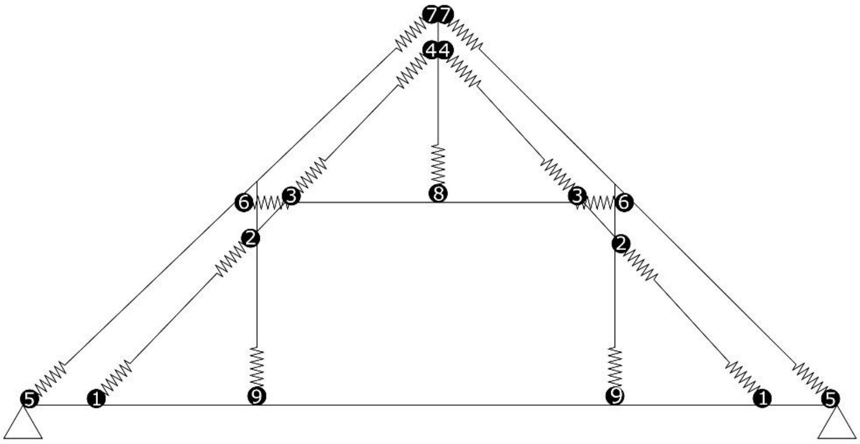


Fig. 4.9 Spring model – third roof structure

Compared to the first two roof structures, in this case, by using the component method, approximately similar results were obtained for all the angles, and no significant discrepancies were observed (Table 4-12).

The maximum, in this case, was recorded for the passing brace to queen-post joint (joint 2) since it is a joint connecting the elements at a low angle. As in the previous two roof structures, the lowest axial stiffnesses were obtained for the elements joining at a 90-degree angle, the king post to collar beam joint (joint 8) and the queen-post to tie-beam joint (joint 9).

Table 4-12 Third roof structure – axial stiffness determined with the component method

Joint	Base element		Inserted element		Angle [°]	E_α (N/mm2)	Horizontal stiffness k_h (kN/m)	Vertical stiffness k_v (kN/m)	k_{ax}
	Width (mm)	Height (mm)	Width (mm)	Height (mm)					
1	180	180	110	170	22	1,110	20,663	213,717	214,714
2	160	160	110	170	22	1,183	18,899	265,277	265,950
3	110	170	110	170	77	1,110	122,084	169,716	209,065
4	170	170	110	170	77	1,183	130,078	187,255	228,001
5	180	180	110	170	35	1,183	21,261	235,802	236,759
6	110	170	110	170	35	1,183	130,078	187,255	228,001
7	170	170	110	170	68	1,110	19,516	226,289	227,129
8	110	170	170	170	90	600	22,667	136,000	137,876
9	180	180	160	160	90	600	24,000	113,778	116,281

The use of the Heimeshoff and Köhler for this roof is presenting significant differences between the obtained results, with half of the joints presenting high axial stiffnesses while the others are rather low (Table 4-13).

Therefore, a maximum axial stiffness was also recorded for the passing brace to queen-post joint (joint 2), but also for the passing brace to tie-beam joint (joint 1) since the contact surface is similar. The lowest axial stiffness was obtained, like previously, for the queen-post to tie-beam joint (joint 9).

Table 4-13 Third roof structure – axial stiffness determined with the Heimeshoff & Köhler method

	Base element		Inserted element		Tenon/Mortise						
Joint	Width [mm]	Depth [mm]	Width [mm]	Depth [mm]	Width [mm]	Depth (step) [mm]	Depth (tenon) [mm]	Angle [°]	K_b	K_t	k_{ax}
1	180	180	110	170	40	0	60	22	1.17	1.09	43,385
2	160	160	110	170	40	0	60	22	1.17	1.09	43,385
3	110	170	110	170	40	0	60	77	0.58	0.99	2,875
4	170	170	110	170	40	0	60	77	0.58	0.99	2,875
5	180	180	110	170	40	0	60	35	0.58	0.99	17,235
6	110	170	110	170	50	0	40	35	0.50	0.98	14,762
7	170	170	110	170	40	0	50	68	0.58	0.98	4,949
8	110	170	170	170	50	0	50	90	1.00	0.96	2,918
9	180	180	160	160	50	0	50	90	0.92	0.97	2,675

When comparing the results obtained with the three methods, the same observations can be made, as in the previous two cases (Table 4-14). Similar to the first two roof structures, high axial stiffnesses were obtained using the component method and approximately similar, lower results were obtained using the other two methods.

Like for the second roof structure, it was observed that despite the different approaches, the maximum and the minimum axial stiffness was recorded at the same joints for both the component method and the Heimeshoff and Köhler method. The Hölzer method, on the other hand, is once again presenting completely different values, with low axial stiffnesses for low angles between the timber elements composing the joints and high values for angles closer to 90 degrees.

Table 4-14 Third roof structure – comparison of the obtained joint axial stiffnesses

Joint	k_{ax} (Component method)	k_{ax} (Heimeshoff & Köhler)	k_{ax} (Hölzer)
1	214,714	43,385	20,000
2	265,950	43,385	20,000
3	209,065	2,875	60,000
4	228,001	2,875	60,000
5	236,759	17,235	20,000
6	228,001	14,762	20,000
7	227,129	4,949	60,000
8	137,876	2,918	60,000
9	116,281	2,675	60,000

The obtained results for all the three roof structures highlight the differences between the approaches. At the same time, they show how the layout of the structure, the mechanical properties of the timber, the used joint type and the cross-section of the timber elements are influencing the axial stiffness of a joint.

4.3 Seismic behaviour of historic buildings with and without roof structures

In order to understand the influence of common roof structures from Timisoara on the seismic behaviour of the chosen 18th-century building, four three-dimensional finite element models were made:

1. The 18th-century building without roof structure – as a reference case to understand the effect of the roof structures;
2. The 18th-century building with the 18th century (first) roof structure (Fig. 4.10a);
3. The 18th-century building with the 19th century (second) roof structure (Fig. 4.10b);
4. The 18th-century building with the 20th century (third) roof structure (Fig. 4.10c).

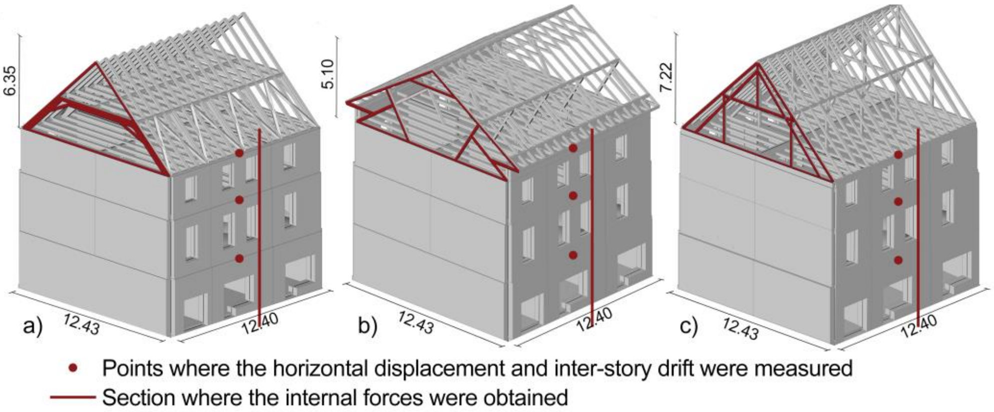


Fig. 4.10 The finite element models - a) first; b) second; c) third roof structure [167]

The finite element analysis software SCIA engineer [159] was used to perform the numerical simulations. Despite being more conservative, the seismic Equivalent Lateral Forces (ELF) method was used to perform the seismic analysis of the building with and without roof structures. This method supposes a static analysis of the structure, considering the mass of the load-bearing elements for each story of the building and the seismic response spectrum of the area in which the building is placed. The horizontal seismic force is subsequently determined by the software and distributed on each floor.

Since the distance between consecutive vertical load-bearing elements is low, having a maximum of 6 meters and the thicknesses of the walls are rather high (reaching from 90 cm down to 45), introducing eccentricities in the numerical models would have scarcely influenced the seismic behaviour of the structure. Consequently, despite being aligned on the exterior, in the model, the eccentricities were ignored [167].

Due to the complex nature of the connection between the roof structure and the historic masonry wall and since this connection is relevant in the seismic behaviour of the building three different support typologies were considered for the numerical simulation, applied on each model containing a roof structure:

1. First, a rigid support was considered. Despite representing a rather conservative approach, an ideal connection between the roof and the building was considered relevant as a reference for the research;
2. Secondly, a sliding support was considered for which the axial stiffness in the longitudinal and transversal direction of the roof was modelled according to

performed calibrations on Baroque roof structures [145]. According to the study, a horizontal axial stiffness of about 50 kN/m can be assumed in the direction of the trusses and 10 kN/m along the wall;
3. Third, the supports where considered hinged on one side of the building and sliding on the other side.

By considering these three support scenarios, it was possible to understand how a very rigid connection or a connection which only considers the friction between the roof and the wall may influence the behaviour of the historic building during a seismic event.

Subsequently, the traditionally crafted timber joints were considered for each of the support scenarios as:

1. rigid joints;
2. hinged joints;
3. semi-rigid joints considering the three methods previously described (component, Heimeshoff and Köhler and Hölzer methods).

In this way, both simple analysis using rigid or hinged joints were taken into consideration but also a more time-consuming way of determining the axial stiffness of the joint was used. The main scope was to understand how each type of joint is influencing the relationship between the roof and the building. Considering this, 15 scenarios were created, influenced by the support type and joint typology. In order to simplify the identification of each scenario, a string of two numbers was associated which each scenario, the first one representing the support type and the second one the joint type (Table 4-15).

Table 4-15 Assessed scenarios [167]

Scenarios	1. Rigid support	2. Sliding support	3. Hinged-sliding support
1. Rigid joints	S1.1.	S1.2.	S1.3.
2. Hinged joints	S2.1.	S2.2.	S2.3.
3. Hölzer method	S3.1.	S3.2.	S3.3.
4. Component method	S4.1.	S4.2.	S4.3.
5. Heimeshoff & Köhler method	S5.1.	S5.2.	S5.3.

For each model and each scenario, the following parameters were assessed and compared. The main scope was to understand which roof structure is or is not improving the seismic behaviour of the building and how the results are connected with the used support and joint type:

1. The out-of-plane horizontal displacement of each floor – meant to highlight how each roof is influencing the horizontal displacement of each floor of the building compared to the no roof case;
2. The inter-story drift of each floor – meant to highlight the relative horizontal displacement between two consecutive floors and help determine the deformed shape of the building and its damage level;
3. Deformed shape;
4. Damage level;
5. Internal forces:
    - 5.1. Vertical axial forces;
    - 5.2. Out-of-plane shear forces;
    - 5.3. Out-of-plane bending moments.

4.3.1 Out-of-plane displacement analysis

4.3.1.1 Building with no roof structure

In the first phase of the study in order to understand the seismic behaviour of the historic masonry building, the out-of-plane horizontal displacement of every floor of the building without any roof structure was evaluated (Fig. 4.11). The analysis showed that the horizontal displacement is continuously increasing, starting with the first-floor slab, up to the last floor where a horizontal displacement of 19.86mm was obtained. At the same time, the influence of the floor typology was observed. Due to the presence of timber beam floors starting with the second floor, the horizontal displacement is suffering a significant increase. Therefore, while the first floor displacement is just 2.18mm, on the second floor, a 5.10mm increase was observed, reaching a horizontal displacement of 7.28mm. Ultimately, on the top floor, the horizontal displacement is reaching the maximum of 19.86mm.

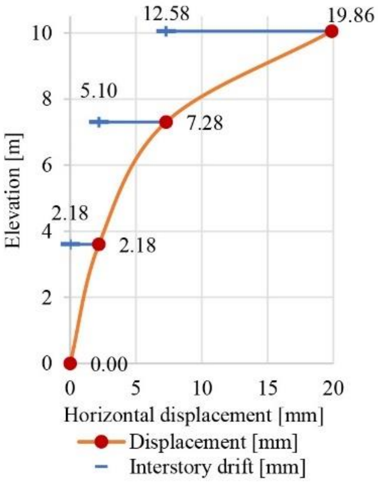


Fig. 4.11 Out-of-plane horizontal displacement analysis of the historic masonry building without a roof structure

4.3.1.2 Roof structures with complete cross-section timber elements

Subsequently, each of the three chosen roof structures was placed on the historic masonry building. The presence of the first roof structure showed that this type has the most diverse influence on the seismic behaviour of the building, depending on the considered scenario (Table 4-16, Fig. 4.12). At the first floor, the presence of the roof structure is reducing the horizontal displacement with around 5%. The minimum horizontal displacement on the first floor was recorded at the S4.3 scenario with a reduction of 25%, while the maximum was recorded at the S3.1 scenario, with an increase of about 5%. Scenarios S3.1, S4.1, S5.1, S1.2, S2.2 and S3.2 present a slight increase in horizontal displacement, of up to 5% while the other scenarios are presenting a reduction. Considering the support scenarios, the minimum displacement was recorded for all the scenarios, at the hinged sliding support, except the rigid joint scenario, where the minimum appeared at the rigid support. The maximum displacement, on the other hand, was recorded for the sliding support at the rigid and hinged joints and the rigid support for the other three semi-rigid joint scenarios. At the second floor, the horizontal displacement was reduced due to the presence of the roof with a mean of 10%, the minimum being recorded at the S4.3 scenario, with a reduction of 35%, while the maximum was recorded at the S3.1 scenario with an insignificant increase compared to the no roof structure scenario. The S3.1 is at this floor the only one which is presenting an increase of the horizontal displacement. Considering the support scenarios, the minimum horizontal displacement was also recorded for the hinged, sliding support ones with an only exception, at rigid joints where the minimum appears at the rigid support. The maximum appears like at the first floor at the sliding support for rigid and hinged joints and rigid supports for the other semi-rigid joint scenarios.

On the top floor, the displacement of the historic masonry wall was reduced with a mean of 50%, the minimum displacement being recorded at the S1.1 scenario, with a reduction of the top horizontal displacement of 55%, while the maximum displacement was recorded at the S4.2 scenario, with a reduction of 40%. At this floor, the minimum appears for all the scenarios at the rigid support while the maximum was recorded at the sliding support scenarios.

Table 4-16 Out-of-plane horizontal displacement of the scenarios - building with the first roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Story	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]
	S1.1		S1.2		S1.3	
3.6	1.84	-15.71%	2.22	1.66%	1.94	-11.03%
7.3	5.43	-25.42%	7.02	-3.60%	5.87	-19.42%
10.05	8.75	-55.96%	10.57	-46.76%	10.28	-48.22%
	S2.1		S2.2		S2.3	
3.6	2.17	-0.68%	2.24	2.34%	2.01	-7.85%
7.3	6.90	-5.30%	7.10	-2.47%	6.42	-11.89%
10.05	8.97	-54.83%	11.12	-43.99%	9.98	-49.75%
	S3.1		S3.2		S3.3	
3.60	2.26	3.40%	2.22	1.74%	2.06	-5.59%
7.30	7.33	0.59%	7.09	-2.61%	6.69	-8.11%
10.05	9.17	-53.85%	11.13	-43.99%	10.04	-49.44%
	S4.1		S4.2		S4.3	
3.60	2.20	0.68%	2.17	-0.53%	1.63	-25.23%
7.30	7.01	-3.81%	6.84	-6.09%	4.66	-36.04%
10.05	9.08	-54.29%	12.36	-37.76%	9.91	-50.12%
	S5.1		S5.2		S5.3	
3.60	2.23	1.96%	2.13	-2.57%	1.89	-13.44%
7.30	7.25	-0.48%	6.71	-7.86%	5.90	-19.01%
10.05	9.17	-53.85%	12.19	-38.65%	10.56	-46.86%

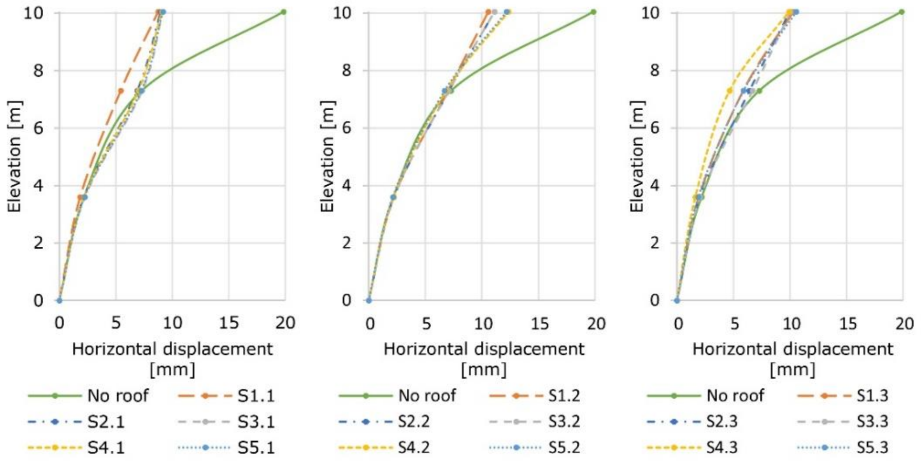


Fig. 4.12 Out-of-plane horizontal displacement analysis of the scenarios - building with the first roof structure with complete cross-section timber elements (comparison to the no roof structure case)

The presence of the second roof structure shows that this type is influencing the building in a different way than the first (Table 4-17, Fig. 4.13). On the contrary to the first roof, the presence of the second roof structure is increasing the horizontal displacement with around 5% at the first and second floor. The minimum horizontal displacement at the first floor, of 2.17 mm, was recorded at the S1.2 scenario with a reduction of just 0.6%, while the maximum, 2.38 mm, was recorded at the S5.3 scenario, with an increase of 10% compared to the no roof case. In this case, only the S1.1 and S1.2 scenarios are presenting a decrease of the horizontal displacement while the other scenarios are clearly showing an increase.

At the second floor the minimum horizontal displacement, 7.05 mm, was still recorded at the S1.2 scenario, with a reduction of 5%, while the maximum, 8.30 mm was recorded at the S5.2 scenario with an increase compared to the no roof structure scenario of 15%. At this floor, only the S1.1, S1.2 and S1.3 scenarios are presenting a decrease of the horizontal displacement.

Although most of the scenarios present an increase of the horizontal displacement of the historic masonry building up to the second floor, on the top floor, an apparent reduction of the displacement was observed for all the assessed scenarios, with a mean of 40%. The minimum displacement, 11.03 mm, was recorded at this floor at the S5.3 scenario, with a reduction of the top horizontal displacement of 45%, while the maximum displacement, 12.37 mm, was recorded at the S1.2 scenario, with a reduction of 40%.

When analysing the displacement of the historic masonry building with the second roof structure, it was observed that the minimal displacement was recorded for the first and second floor at the rigid support scenario, except for the rigid joints scenarios where the minimum was recorded for the sliding support scenario. In contrast, on the top floor, the minimum was recorded for the hinged-sliding support, the exception being the rigid joints scenarios where the minimum was recorded at the rigid support. The maximum displacement at the first and second floor was mainly recorded for the sliding and the hinged-sliding supports, which present little differences between each other. The top floor, on the other hand, presents the maximum displacement for rigid support scenarios for the hinged, Hölzer and component method determined joints while for the sliding support, the maximum was recorded for the rigid and Heimeshoff & Köhler method calculated joint scenarios.

Table 4-17 Out-of-plane horizontal displacement of the scenarios - building with the second roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Story	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]
	S1.1		S1.2		S1.3	
3.6	2.18	-0.15%	2.17	-0.60%	2.19	0.23%
7.3	7.09	-2.67%	7.05	-3.22%	7.12	-2.27%
10.05	11.81	-40.55%	12.37	-37.74%	12.25	-38.33%
	S2.1		S2.2		S2.3	
3.6	2.25	3.02%	2.31	5.66%	2.31	5.82%
7.3	7.64	4.92%	7.91	8.63%	7.90	8.52%
10.05	11.52	-42.02%	11.32	-42.99%	11.16	-43.81%
	S3.1		S3.2		S3.3	
3.60	2.26	3.32%	2.31	5.89%	2.32	6.04%
7.30	7.67	5.37%	7.93	8.95%	7.92	8.81%
10.05	11.48	-42.22%	11.30	-43.09%	11.14	-43.92%
	S4.1		S4.2		S4.3	
3.60	2.25	3.17%	2.31	5.82%	2.31	5.97%
7.30	7.67	5.28%	7.93	8.90%	7.92	8.79%
10.05	11.51	-42.07%	11.33	-42.97%	11.16	-43.79%
	S5.1		S5.2		S5.3	
3.60	2.34	7.33%	2.37	8.69%	2.38	8.84%
7.30	8.18	12.37%	8.30	13.91%	8.29	13.82%
10.05	11.07	-44.24%	11.18	-43.71%	11.03	-44.47%

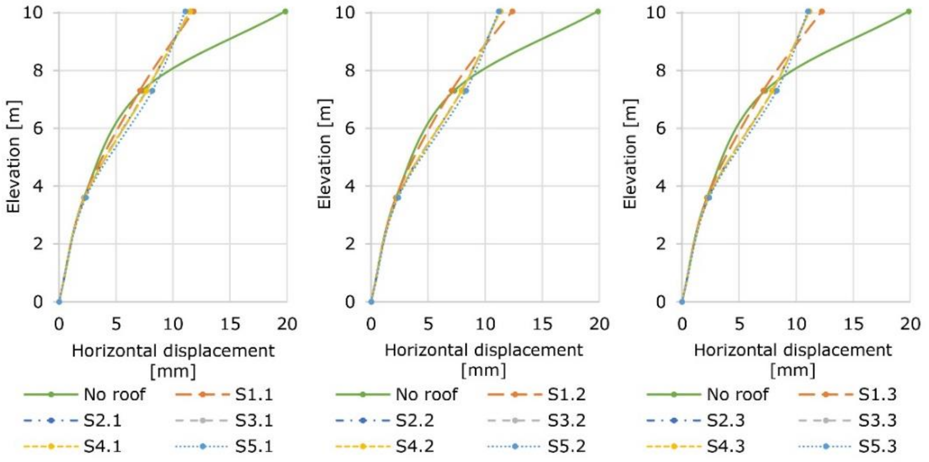


Fig. 4.13 Out-of-plane horizontal displacement analysis of the scenarios - building with the second roof structure with complete cross-section timber elements (comparison to the no roof structure case)

The presence of the third roof structure on the historic masonry building is presenting once again, a completely different influence on the seismic behaviour of the structure (Table 4-18, Fig. 4.14). Compared to the other two assessed behaviours, this roof is reducing the horizontal displacement of the building for all the floors, without presenting any increase.

Only slight differences between the horizontal displacement of the building, considering the chosen support scenarios and joint axial stiffnesses could be observed. At the first and second floor, due to the presence of the roof structure, the horizontal displacement is decreasing with a mean of 10%. The minimum horizontal displacement on the first floor, of 1.77 mm, was recorded at the S1.1 scenario with a decrease of 20% compared to the no roof case, while the maximum, of 2.04 mm, was recorded at the S5.1 scenario, with a decrease of about 5%.

On the second floor, on the other hand, the minimum horizontal displacement, of 5.9 mm, was recorded at the S1.3 scenario, with a reduction of 20%, while the maximum, of 6.72 mm was observed at the S5.1 scenario, with a reduction of about 10%. At this floor, almost all the assessed scenarios, except for the ones involving rigid joints, the horizontal displacement reduction ranges between 10 and 15%. On the top floor, the displacement of the historic masonry wall is presenting the most visible reduction with a mean of only 20%, the minimum displacement being recorded at the S5.3 scenario, with a reduction of the top horizontal displacement of 30%, while the maximum displacement was recorded at the S1.2 scenario, with a reduction of 10%. At this floor, the minimum appears for all the scenarios at the rigid support while the maximum was recorded at the sliding support scenarios.

Table 4-18 Out-of-plane horizontal displacement of the scenarios - building with the third roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Story	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]
	S1.1		S1.2		S1.3	
3.6	1.77	-19.03%	1.88	-14.05%	1.78	-18.50%
7.3	5.92	-18.74%	6.32	-13.23%	5.90	-19.05%
10.05	17.57	-11.56%	18.06	-9.09%	17.99	-9.42%
	S2.1		S2.2		S2.3	
3.6	1.94	-11.25%	1.98	-9.21%	1.99	-8.76%
7.3	6.38	-12.46%	6.30	-13.55%	6.30	-13.48%
10.05	15.06	-24.17%	15.46	-22.16%	15.39	-22.52%
	S3.1		S3.2		S3.3	
3.60	2.01	-8.16%	1.98	-9.37%	1.98	-9.21%
7.30	6.43	-11.67%	6.27	-13.89%	6.27	-13.86%
10.05	15.22	-23.39%	15.36	-22.67%	15.30	-22.99%
	S4.1		S4.2		S4.3	
3.60	1.90	-13.07%	1.99	-8.84%	1.99	-8.76%
7.30	6.34	-13.00%	6.29	-13.68%	6.28	-13.71%
10.05	15.40	-22.47%	15.83	-20.31%	15.76	-20.66%
	S5.1		S5.2		S5.3	
3.60	2.04	-6.50%	2.01	-7.78%	2.02	-7.70%
7.30	6.72	-7.70%	6.54	-10.22%	6.53	-10.35%
10.05	14.40	-27.50%	14.53	-26.83%	14.28	-28.11%

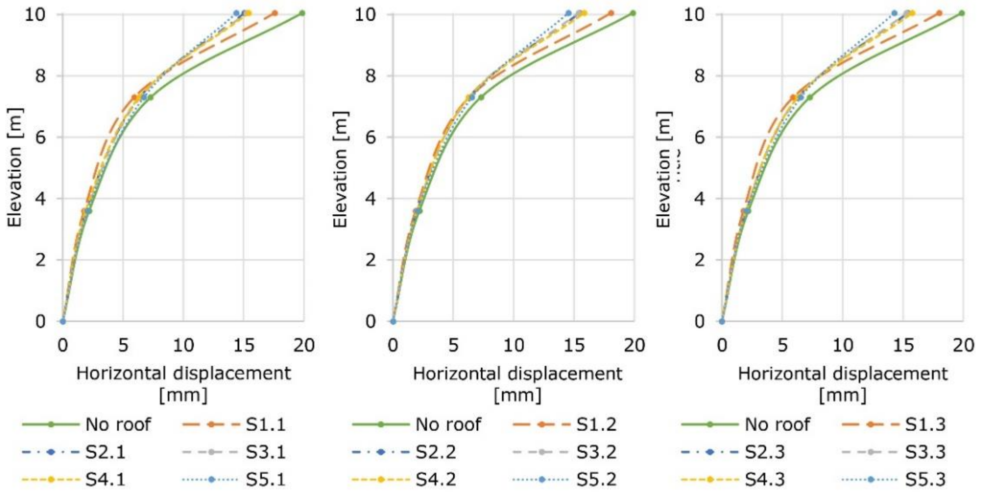


Fig. 4.14 Out-of-plane horizontal displacement analysis of the scenarios - building with the third roof structure with complete cross-section timber elements (comparison to the no roof structure case)

While comparing the influence on the seismic behaviour of the three chosen roof structures (Table 4-19, Table 4-20, Table 4-21, Table 4-22, Table 4-23, Table 4-24, Fig. 4.15), on the historic masonry building, the following was observed:

1. The out-of-plane horizontal displacement of all the models is approximately similar at the first floor of the building, and the differences start to be observed above;
2. The presence of the third roof structure is causing the lowest horizontal displacement for the first and second floor of the building for almost all the assessed scenarios. Exceptions were observed at the S4.3 and S5.3., where the first roof structure is presenting the lowest horizontal displacement at the first and second floor;
3. The presence of the third roof structure is presenting the highest horizontal displacement for the first and second floor of the building for all the assessed scenarios;
4. At the top floor of the building, it was observed that:
    - 4.1. The first roof structure is presenting the lowest horizontal out-of-plane displacement;
    - 4.2. The third roof structure is presenting the highest horizontal out-of-plane displacement.

Table 4-19 Mean out-of-plane horizontal displacement analysis of the building without a roof and with the three roof with complete cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Displacement [mm]	2.18	2.08	2.29	1.95
	Compared to no roof		-5%	+5%	-10%
2nd floor	Displacement [mm]	7.28	6.55	7.77	6.32
	Compared to no roof		-10%	+5%	-15%
top floor	Displacement [mm]	19.86	10.22	11.44	15.71
	Compared to no roof		-50%	-40%	-20%

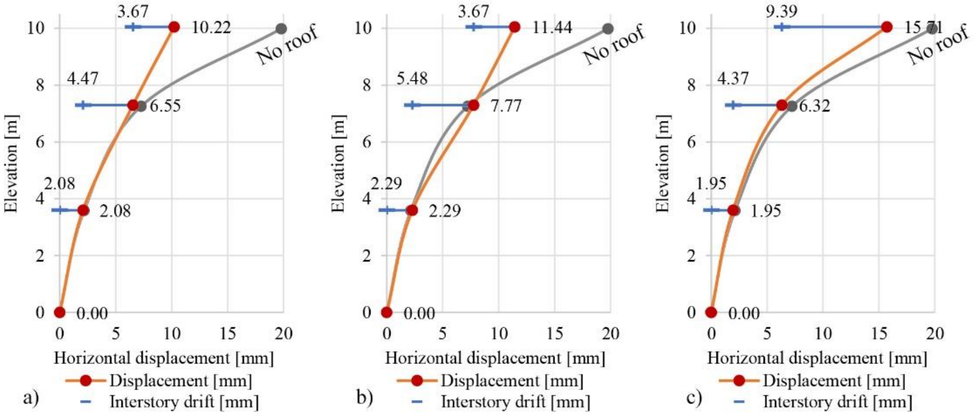


Fig. 4.15 Mean out-of-plane horizontal displacement of the historic masonry building with complete cross-section roof structure (a) first roof structure; b) second roof structure; c) third roof structure)

Table 4-20 Minimum out-of-plane horizontal displacement analysis of the building without a roof and with the three roof structures with complete cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Displacement [mm]	2.18	1.63	2.17	1.77
	Compared to no roof		-25%	-1%	-20%
	Scenario		S4.3.	S1.2.	S1.1.
2nd floor	Displacement [mm]	7.28	4.66	7.05	5.90
	Compared to no roof		-35%	-5%	-20%
	Scenario		S4.3.	S1.2.	S1.3.
top floor	Displacement [mm]	19.86	8.75	11.03	14.28
	Compared to no roof		-55%	-45%	-30%
	Scenario		S1.1.	S5.3.	S5.3.

Table 4-21 Maximum out-of-plane horizontal displacement analysis of the building without a roof and with the three roof structures with complete cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Displacement [mm]	2.18	2.26	2.38	2.04
	Compared to no roof		+5%	+10%	-5%
	Scenario		S3.1.	S5.3.	S5.1.
2nd floor	Displacement [mm]	7.28	7.33	8.30	6.72
	Compared to no roof		0%	+15%	-10%
	Scenario		S3.1.	S5.2.	S5.1.
top floor	Displacement [mm]	19.86	12.36	12.37	18.06
	Compared to no roof		-40%	-40%	-10%
	Scenario		S4.2.	S1.2.	S1.2.

Table 4-22 Minimum and maximum out-of-plane horizontal displacement analysis of the building with the with the three roof structures with complete cross-section timber elements for rigid support scenarios

		No roof	1st roof structure		2nd roof structure		3rd roof structure	
			Min	Max	Min	Max	Min	Max
1st floor	Displacement [mm]	2.18	1.84	2.26	2.18	2.34	1.77	2.04
	Compared to no roof		-15%	+5%	0	+5%	-20%	-5%
	Scenario		S1.1.	S3.1.	S1.1.	S5.1.	S1.1.	S5.1.
2nd floor	Displacement [mm]	7.28	5.43	7.33	7.09	8.18	5.92	6.72
	Compared to no roof		-25%	0%	-5%	+10%	-20%	-10%
	Scenario		S1.1.	S3.1.	S1.1.	S5.1.	S1.1.	S5.1.
top floor	Displacement [mm]	19.86	8.75	9.17	11.07	11.81	14.40	17.57
	Compared to no roof		-55%	-55%	-45%	-40%	-30%	-10%
	Scenario		S1.1.	S3.1.	S5.1.	S1.1.	S5.1.	S1.1.

Table 4-23 Minimum and maximum out-of-plane horizontal displacement analysis of the building with the with the three roof structures with complete cross-section timber elements for sliding support scenarios

		No roof	1st roof structure		2nd roof structure		3rd roof structure	
			Min	Max	Min	Max	Min	Max
1st floor	Displacement [mm]	2.18	2.13	2.24	2.17	2.37	1.88	2.01
	Compared to no roof		0%	+2.5%	0%	+10%	-15%	-10%
	Scenario		S5.2.	S2.2.	S1.2.	S5.2.	S1.2.	S5.2.
2nd floor	Displacement [mm]	7.28	6.71	7.10	7.05	8.30	6.27	6.54
	Compared to no roof		-10%	-2.5%	-5%	+15%	-15%	-10%
	Scenario		S5.2.	S2.2.	S1.2.	S5.2.	S3.2.	S5.2.
top floor	Displacement [mm]	19.86	10.57	12.36	11.18	12.37	14.53	18.06
	Compared to no roof		-50%	-40%	-45%	-40%	-30%	-10%
	Scenario		S1.2.	S4.2.	S5.2.	S1.2.	S5.2.	S1.2.

Table 4-24 Minimum and maximum out-of-plane horizontal displacement analysis of the building with the with the three roof structures with complete cross-section timber elements for hinged-sliding support scenarios

		No roof	1st roof structure		2nd roof structure		3rd roof structure	
			Min	Max	Min	Max	Min	Max
1st floor	Displacement [mm]	2.18	1.63	2.06	2.19	2.38	1.78	2.02
	Compared to no roof		-25%	-5%	0%	+10%	-20%	-10%
	Scenario		S4.3.	S3.3.	S1.3.	S5.3.	S1.3.	S5.3.
2nd floor	Displacement [mm]	7.28	4.66	6.69	7.12	8.29	5.90	6.53
	Compared to no roof		-35%	-10%	0%	+15%	-20%	-10%
	Scenario		S4.3.	S3.3.	S1.3.	S5.3.	S1.3.	S5.3.
top floor	Displacement [mm]	19.86	9.91	10.56	11.03	12.25	14.29	17.99
	Compared to no roof		-50%	-45%	-45%	-40%	-30%	-10%
	Scenario		S4.3.	S5.3.	S5.3.	S1.3.	S5.3.	S1.3.

4.3.1.3 Roof structures with 20% reduced cross-section timber elements

Since the roof structures were built in the 18th, 19th respectively in the 20th century, although they were in a good state of conservation without any major decays, it was considered that the study should have also have a second step where the roof structures are considered decayed. According to other studies, when performing numerical simulations concerning historic roof structures, a reduction of the cross-section of the timber elements should be taken into consideration, due to the rounded edges and possibly decayed outer layer of the timber elements. The studies state that a reduction of 15 up to 20% can be expected [138]. This is also consistent with the observations made during the calibration process presented in chapter 3, where a cross-sectional reduction of about 20% was considered in order to obtain similar results as during the analysed laboratory tests.

Therefore, in this study, a reduction of 20% of all timber elements was considered, in order to be able to observe if the decay of the roof structure would influence the seismic behaviour of the historic masonry building in a different way.

First, as for the complete cross-section analysis, the first roof structure with decayed timber elements was placed on the historic masonry building (Table 4-25, Fig. 4.16). At the first floor of the building, a mean horizontal displacement of 2.15mm was observed, which means only a slight reduction of its value compared to the no roof scenario. The minimum displacement was recorded as at the S5.2 scenario with a reduction of the horizontal displacement of 10% while the maximum displacement was recorded at the S3.1 scenario with an increase of 10%.

Table 4-25 Out-of-plane horizontal displacement of the scenarios - building with the first roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Story	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]
	S1.1		S1.2		S1.3	
3.6	2.23	2.27%	2.11	-3.63%	2.00	-8.38%
7.3	7.23	-0.75%	6.77	-7.07%	6.34	-12.89%
10.05	9.95	-49.88%	10.41	-47.59%	10.75	-45.85%
	S2.1		S2.2		S2.3	
3.6	2.35	7.55%	2.14	-2.11%	2.12	-3.10%
7.3	7.77	6.66%	6.83	-6.23%	7.02	-3.60%
10.05	10.46	-47.35%	12.75	-35.81%	11.64	-41.39%
	S3.1		S3.2		S3.3	
3.60	2.35	7.70%	2.11	-3.47%	2.11	-3.55%
7.30	7.88	8.20%	6.87	-5.69%	7.11	-2.36%
10.05	10.56	-46.83%	12.73	-35.92%	11.56	-41.79%
	S4.1		S4.2		S4.3	
3.60	2.33	6.87%	2.03	-6.87%	2.05	-5.97%
7.30	7.79	6.96%	6.49	-10.90%	6.84	-6.03%
10.05	10.72	-46.05%	14.65	-26.24%	13.27	-33.19%
	S5.1		S5.2		S5.3	
3.60	2.28	4.46%	2.00	-8.61%	2.04	-6.65%
7.30	7.74	6.28%	6.33	-13.05%	6.81	-6.52%
10.05	10.59	-46.69%	14.10	-29.01%	13.00	-34.57%

On the second floor, the mean horizontal displacement is suffering a slight decrease, reaching 5%. The minimum displacement is also suffering a slight decrease reaching a reduction of 15% of the horizontal displacement at the S5.2 scenario while the maximum displacement is remaining at an increase of 10% recorded at the S3.1 scenario.

Ultimately on the top floor, the influence of the roof structure on the seismic behaviour of the building can be better observed, the mean horizontal displacement presenting a total reduction of 40%. In this case both the minimum and the maximum displacement present a reduction of the horizontal displacement, the minimum representing a reduction of 50%, recorded at the S1.1 scenario while the maximum is representing a reduction of 25% recorded at the S4.2 scenario.

The displacement analysis shows that the presence of the decayed roof structure is increasing the horizontal displacement at the 1st and 2nd floor only for the scenarios involving rigid supports, compared to the no roof scenario. All the other scenarios present a decrease in the horizontal displacement, starting from approximately 5% on the first floor up to 25-45% on the top floor.

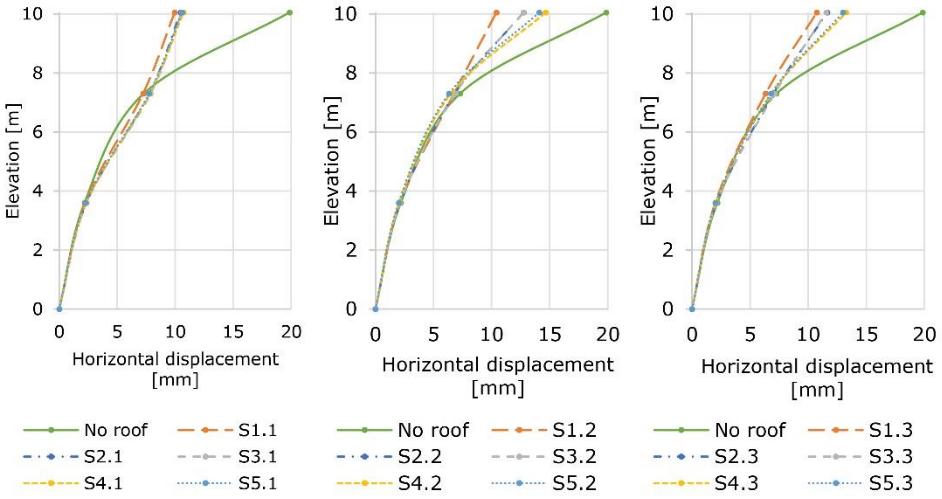


Fig. 4.16 Out-of-plane horizontal displacement analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

When placing the second roof structure on the historic masonry building, the numerical simulations presented an utterly different behaviour, compared to the first roof (Table 4-26, Fig. 4.17). The presence of this type of roof structure on the first floor is increasing the horizontal displacement with a mean of 10% compared to the no roof scenario. In this case, the minimum horizontal displacement is presenting an decrease of 10% recorded at the S1.2 scenario while the maximum was recorded at the S5.3 scenario, presenting an increase of 15%.

Even at the 2nd floor, the horizontal displacement tends to increase, presenting a 20% higher mean horizontal displacement, with the minimum increase of 10% for the S1.2 scenario and the maximum increase of 20% recorded at the S5.2 scenario.

Despite the increase of the horizontal displacement from the 1st and 2nd floor compared to the no roof scenario, the effect of the roof structure can be observed on the top floor. Here a 40% decrease of the mean horizontal displacement was observed.

The minimum horizontal displacement represents a reduction of 45% compared to the no roof scenario, recorded at the S5.3 scenario while the maximum was representing a reduction of only 35% recorded at the S1.2 scenario. At this floor, all the assessed scenarios present a clear reduction of the horizontal displacement without any exception.

At the same time, when comparing the joint typologies, it was observed that for all the three support types the displacement of the historic masonry building is somewhat similar except for the rigid joint scenarios which prove out to have a higher displacement at the second floor. Still, the most peculiar observation about this roof structure is that when comparing the support typologies for each of the joint types, only slight differences between the obtained horizontal displacements can be observed at any of the floors of the building reaching up to 10%.

Table 4-26 Out-of-plane horizontal displacement of the scenarios - building with the second roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Story	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]
	S1.1		S1.2		S1.3	
3.6	2.37	8.69%	2.38	8.91%	2.38	8.99%
7.3	8.25	13.23%	8.27	13.59%	8.27	13.55%
10.05	12.13	-38.95%	12.53	-36.92%	12.37	-37.73%
	S2.1		S2.2		S2.3	
3.6	2.41	10.42%	2.44	11.78%	2.45	11.93%
7.3	8.69	19.37%	8.82	21.14%	8.81	20.98%
10.05	11.02	-44.52%	11.23	-43.48%	11.07	-44.26%
	S3.1		S3.2		S3.3	
3.60	2.41	10.42%	2.41	10.35%	2.45	11.93%
7.30	8.70	19.44%	8.70	19.46%	8.81	21.00%
10.05	11.02	-44.51%	11.13	-43.97%	11.07	-44.24%
	S4.1		S4.2		S4.3	
3.60	2.41	10.42%	2.44	11.78%	2.45	11.93%
7.30	8.70	19.51%	8.83	21.23%	8.82	21.09%
10.05	11.03	-44.47%	11.24	-43.43%	11.08	-44.20%
	S5.1		S5.2		S5.3	
3.60	2.44	11.56%	2.46	12.69%	2.46	12.61%
7.30	8.85	21.48%	8.91	22.29%	8.90	22.25%
10.05	11.01	-44.58%	10.93	-44.97%	11.10	-44.13%

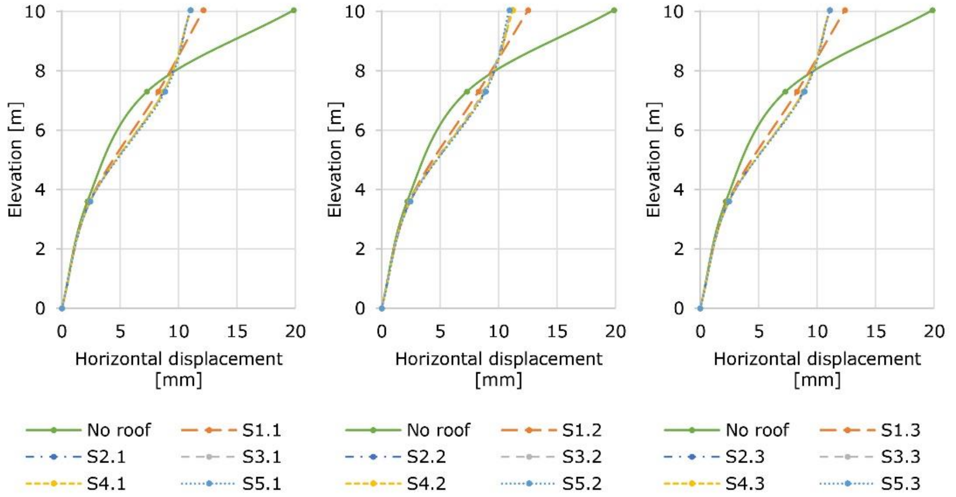


Fig. 4.17 Out-of-plane horizontal displacement analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Ultimately the decayed third roof structure was placed on the historic masonry building and numerical simulations performed for all the 15 scenarios (Table 4-27, Fig. 4.18). In this case, on the 1st floor, the mean horizontal displacement is presenting almost no change compared to the no roof structure case. Still, the minimum horizontal displacement was recorded at the S1.2 scenario while the maximum displacement is presenting an increase of 5% recorded at the S2.1 scenario.

On the 2nd floor the mean displacement is still presenting a slight increase, of 5% but this time the minimum is closer to the no roof structure case presenting no difference, at the S1.2 scenario, while the maximum displacement is presenting a 10% increase recorded at the S2.1 scenario.

Like for the second roof structure, the effect of this type can mostly be observed at the top floor, in this case, the mean displacement is suffering a reduction of 25% compared to the no roof scenario, the minimum displacement presenting a reduction of 30% recorded at the S5.1 scenario while the maximum displacement is presenting a reduction of 25% recorded at the S2.3 scenario.

In this case, depending on the scenario, the roof structure is partially increasing the horizontal displacement of the historic masonry building at the 1st and 2nd floor. Still, on the top floor, all the scenarios are presenting a clear decrease of the horizontal displacement.

Considering the support typologies, it was observed that at the 1st and 2nd floor mainly scenarios involving sliding supports are presenting a minimum and rigid support scenarios a maximum horizontal displacement at the 1st and 2nd story of the building. On the top floor on the other hand, the maximum displacement was recorded for sliding support scenarios and the minimum at the rigid support scenarios.

Table 4-27 Out-of-plane horizontal displacement of the scenarios - building with the third roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Story	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]	Displacement [mm]	Compared to no roof [%]
	S1.1		S1.2		S1.3	
3.6	2.17	-0.60%	2.13	-2.34%	2.14	-1.96%
7.3	7.29	0.16%	7.24	-0.63%	7.26	-0.27%
10.05	14.04	-29.32%	14.29	-28.04%	14.23	-28.37%
	S2.1		S2.2		S2.3	
3.6	2.25	2.95%	2.23	2.11%	2.23	2.27%
7.3	7.84	7.61%	7.72	5.96%	7.73	6.12%
10.05	14.41	-27.44%	14.97	-24.64%	15.00	-24.48%
	S3.1		S3.2		S3.3	
3.60	2.23	1.96%	2.20	0.83%	2.21	1.06%
7.30	7.78	6.77%	7.62	4.69%	7.64	4.92%
10.05	14.10	-29.01%	14.66	-26.19%	14.67	-26.16%
	S4.1		S4.2		S4.3	
3.60	2.24	2.49%	2.22	1.74%	2.23	1.96%
7.30	7.79	6.91%	7.67	5.37%	7.69	5.57%
10.05	13.94	-29.84%	14.78	-25.60%	14.79	-25.53%
	S5.1		S5.2		S5.3	
3.60	2.20	0.91%	2.18	-0.38%	2.18	-0.30%
7.30	7.80	7.16%	7.61	4.46%	7.62	4.58%
10.05	13.88	-30.10%	14.39	-27.56%	14.38	-27.60%

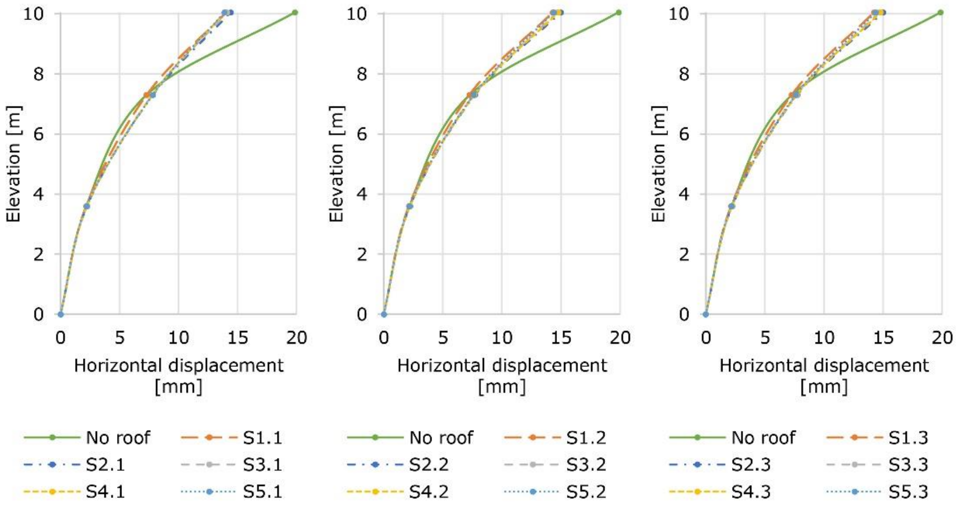


Fig. 4.18 Out-of-plane horizontal displacement analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

While comparing the influence on the seismic behaviour of the three chosen roof structures with decayed timber elements, on the historic masonry building, the following were observed (Table 4-28, Table 4-29, Table 4-30, Table 4-31, Table 4-32, Table 4-33, Fig. 4.19):

1. The second roof structure is presenting the maximum horizontal displacement at the first and second floor of the building;
2. The third roof structure, on the other hand, his presenting the maximum horizontal displacement on the top floor;
3. At the top floor of the building, the minimum horizontal displacement was recorded for the second roof structure for all the assessed scenarios, except the ones involving rigid joins or rigid supports, where the minimum was recorded for the first roof structure;
4. Even though the second roof is presenting the highest horizontal displacement for the first two floors of the building, still due to the high rigidity of the roof structure, it has in all the cases the lowest horizontal displacement at the upper floor of the building.
5. The first roof on the other hand, despite having a minimum displacement at the 1st and 2nd floor, does not present the best effect on the top horizontal displacement of the building

Table 4-28 Mean out-of-plane drift and inter-story drift analysis of the building without a roof and with the three roof structures with reduced cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Displacement [mm]	2.18	2.15	2.42	2.20
	Compared to no roof		0%	+10%	0%
2nd floor	Displacement [mm]	7.28	7.05	8.69	7.62
	Compared to no roof		-5%	+20%	+5%
top floor	Displacement [mm]	19.86	11.81	11.33	14.44
	Compared to no roof		-40%	-40%	-25%

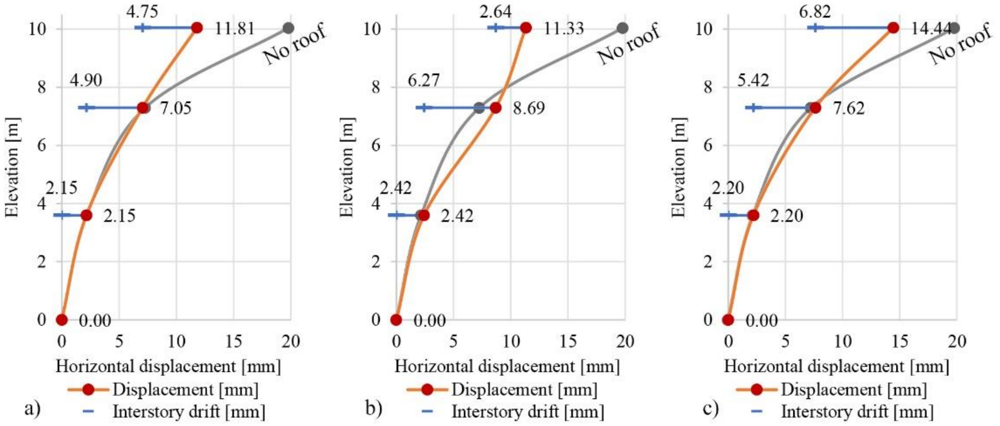


Fig. 4.19 Mean out-of-plane horizontal displacement of the historic masonry building with reduced cross-section roof structure (a) first roof structure; b) second roof structure; c) third roof structure)

Table 4-29 Minimum out-of-plane horizontal displacement analysis of the building without a roof and with the three roof structures with reduced cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Displacement [mm]	2.18	2.00	2.23	2.13
	Compared to no roof		-10%	-10%	0%
	Scenario		S5.2.	S1.2.	S1.2.
2nd floor	Displacement [mm]	7.28	6.33	8.25	7.24
	Compared to no roof		-15%	+10%	0%
	Scenario		S5.2.	S1.2.	S1.2.
top floor	Displacement [mm]	19.86	9.95	10.93	13.88
	Compared to no roof		-50%	-45%	-30%
	Scenario		S1.1.	S5.3.	S5.1.

Table 4-30 Maximum out-of-plane horizontal displacement analysis of the building without a roof and with the three roof structures with reduced cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Displacement [mm]	2.18	2.35	2.46	2.25
	Compared to no roof		+10%	+15%	5%
	Scenario		S3.1.	S5.3.	S2.1.
2nd floor	Displacement [mm]	7.28	7.88	5.87	7.84
	Compared to no roof		+10%	+20%	+10%
	Scenario		S3.1.	S5.2.	S2.1.
top floor	Displacement [mm]	19.86	14.65	12.53	15.00
	Compared to no roof		-25%	-35%	-25%
	Scenario		S4.2.	S1.2.	S2.3.

Table 4-31 Minimum and maximum out-of-plane horizontal displacement analysis of the building with the with the three roof structures with reduced cross-section timber elements for rigid support scenarios

		No roof	1st roof structure		2nd roof structure		3rd roof structure	
			Min	Max	Min	Max	Min	Max
1st floor	Displacement [mm]	2.18	2.23	2.35	2.37	2.44	2.17	2.25
	Compared to no roof		0%	10%	10%	10%	0%	5%
	Scenario		S1.1.	S3.1.	S1.1.	S5.1.	S1.1.	S2.1.
2nd floor	Displacement [mm]	7.28	7.23	7.88	8.25	8.85	7.29	7.84
	Compared to no roof		0%	10%	15%	20%	0%	10%
	Scenario		S1.1.	S3.1.	S1.1.	S5.1.	S1.1.	S2.1.
top floor	Displacement [mm]	19.86	9.95	10.72	11.01	12.13	13.88	14.41
	Compared to no roof		-50%	-45%	-45%	-40%	-30%	-25%
	Scenario		S1.1.	S4.1.	S5.1.	S1.1.	S5.1.	S2.1.

Table 4-32 Minimum and maximum out-of-plane horizontal displacement analysis of the building with the with the three roof structures with reduced cross-section timber elements for sliding support scenarios

		No roof	1st roof structure		2nd roof structure		3rd roof structure	
			Min	Max	Min	Max	Min	Max
1st floor	Displacement [mm]	2.18	2.00	2.14	2.38	2.46	2.13	2.23
	Compared to no roof		-10%	0%	10%	15%	0%	0%
	Scenario		S5.2.	S2.2.	S1.2.	S5.2.	S1.2.	S2.2.
2nd floor	Displacement [mm]	7.28	6.33	6.87	8.27	8.91	7.24	7.72
	Compared to no roof		-15%	-5%	15%	20%	0%	5%
	Scenario		S5.2.	S3.2.	S1.2.	S5.2.	S1.2.	S2.2.
top floor	Displacement [mm]	19.86	10.41	14.65	10.93	12.53	14.29	14.97
	Compared to no roof		-50%	-25%	-45%	-35%	-30%	-25%
	Scenario		S1.2.	S4.2.	S5.2.	S1.2.	S1.2.	S2.2.

Table 4-33 Minimum and maximum out-of-plane horizontal displacement analysis of the building with the with the three roof structures with reduced cross-section timber elements for hinged-sliding support scenarios

		No roof	1st roof structure		2nd roof structure		3rd roof structure	
			Min	Max	Min	Max	Min	Max
1st floor	Displacement [mm]	2.18	2.00	2.12	2.38	2.46	2.14	2.23
	Compared to no roof		-10%	-5%	10%	15%	0%	0%
	Scenario		S1.3.	S2.3.	S1.3.	S5.3.	S1.3.	S2.3.
2nd floor	Displacement [mm]	7.28	6.34	7.11	8.27	8.90	7.26	7.73
	Compared to no roof		-15%	0%	15%	20%	0%	5%
	Scenario		S1.3.	S3.3.		S5.3.	S1.3.	S2.3.
top floor	Displacement [mm]	19.86	10.75	13.27	11.07	12.37	14.23	15.00
	Compared to no roof		-45%	-35%	-45%	-40%	-30%	-25%
	Scenario		S1.3.	S4.3.	S2.3.	S1.3.	S1.3.	S2.3.

4.3.1.4 Comparison

In order to better understand the effect of the decay of the timber elements on the seismic behaviour of the historic masonry structure, the obtained results for the complete cross-section roof structure and reduced cross-section were subsequently compared.

For the first roof structure (Table 4-34, Table 4-35), the displacement of the first floor is quite similar for all the assessed scenarios, varying with about 5% between the complete section roof structure and the reduced section one. The exception is the S1.1 scenario where the displacement on the first floor for the reduce cross-section is 20% higher than the one for complete cross-section.

At the second floor, all the scenarios present a significant increase in the horizontal displacement of the historic masonry building when the cross-section of the timber element is reduced. The increase, in this case, is around 10 up to 30%. Still, an exception was found in this case for all the scenarios involving sliding supports where the horizontal displacement at the second floor is suffering a slight (maximum 5%) decrease in the reduced cross-section case.

Finally, the top floor is presenting the most evident increase of the top horizontal displacement of the building, increasing with up to 30%. Even the sliding support scenarios present a significant increase in the displacement, in this case, of up to 15-20%.

Table 4-34 Out-of-plane horizontal displacement comparison - building with the first roof structure with complete and reduced cross-section timber elements [mm]

	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison
	S1.1			S1.2			S1.3		
3.6	1.84	2.23	21.33%	2.22	2.11	-5.20%	1.94	2.00	2.97%
7.3	5.43	7.23	33.08%	7.02	6.77	-3.60%	5.87	6.34	8.10%
10.05	8.75	9.95	13.81%	10.57	10.41	-1.56%	10.28	10.75	4.57%
	S2.1			S2.2			S2.3		
3.6	2.17	2.35	8.29%	2.24	2.14	-4.35%	2.01	2.12	5.16%
7.3	6.90	7.77	12.63%	7.10	6.83	-3.86%	6.42	7.02	9.41%
10.05	8.97	10.46	16.55%	11.12	12.75	14.61%	9.98	11.64	16.65%
	S3.1			S3.2			S3.3		
3.6	2.26	2.35	4.16%	2.22	2.11	-5.12%	2.06	2.11	2.16%
7.3	7.33	7.88	7.57%	7.09	6.87	-3.16%	6.69	7.11	6.26%
10.05	9.17	10.56	15.23%	11.13	12.73	14.40%	10.04	11.56	15.11%
	S4.1			S4.2			S4.3		
3.6	2.20	2.33	6.15%	2.17	2.03	-6.38%	1.63	2.05	25.76%
7.3	7.01	7.79	11.19%	6.84	6.49	-5.11%	4.66	6.84	46.94%
10.05	9.08	10.72	18.05%	12.36	14.65	18.51%	9.91	13.27	33.94%
	S5.1			S5.2			S5.3		
3.6	2.23	2.28	2.44%	2.13	2.00	-6.20%	1.89	2.04	7.85%
7.3	7.25	7.74	6.78%	6.71	6.33	-5.63%	5.90	6.81	15.41%
10.05	9.17	10.59	15.54%	12.19	14.10	15.72%	10.56	13.00	23.14%

Table 4-35 Mean out-of-plane horizontal displacement comparison - building with the first roof structure with complete and reduced cross-section timber elements [mm]

	Complete cross-section	Reduced cross-section	Comparison
1st floor	2.08	2.15	+5%
2nd floor	6.55	7.05	+10%
top floor	10.22	11.81	+15%

When placing the second roof structure on the historic masonry building (Table 4-36, Table 4-37), the displacement analysis shows that due to the decrease of the cross-section of the timber elements, the most significant increase can be observed at the second floor of the building (with up to 20%), while the displacement of the first and top floor are rather insignificant.

The most peculiar observation, in this case, is that on the top floor, the decayed roof structure is causing 5% less horizontal displacement than in the complete cross-section case. Due to the roof structure typology, it was observed that the decayed timber elements are suffering more vertical displacement and not transferring the horizontal loads towards the historic masonry walls.

Table 4-36 Out-of-plane horizontal displacement comparison - building with the second roof structure with complete and reduced cross-section timber elements [mm]

	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison
		S1.1			S1.2			S1.3	
3.6	2.18	2.37	8.85%	2.17	2.38	9.57%	2.19	2.38	8.74%
7.3	7.09	8.25	16.34%	7.05	8.27	17.37%	7.12	8.27	16.18%
10.05	11.81	12.13	2.68%	12.37	12.53	1.32%	12.25	12.37	0.97%
		S2.1			S2.2			S2.3	
3.6	2.25	2.41	7.18%	2.31	2.44	5.79%	2.31	2.45	5.78%
7.3	7.64	8.69	13.78%	7.91	8.82	11.51%	7.90	8.81	11.48%
10.05	11.52	11.02	-4.31%	11.32	11.23	-0.86%	11.16	11.07	-0.80%
		S3.1			S3.2			S3.3	
3.6	2.26	2.41	6.87%	2.31	2.41	4.21%	2.32	2.45	5.56%
7.3	7.67	8.70	13.35%	7.93	8.70	9.65%	7.92	8.81	11.20%
10.05	11.48	11.02	-3.97%	11.30	11.13	-1.55%	11.14	11.07	-0.58%
		S4.1			S4.2			S4.3	
3.6	2.25	2.41	7.03%	2.31	2.44	5.64%	2.31	2.45	5.63%
7.3	7.67	8.70	13.51%	7.93	8.83	11.32%	7.92	8.82	11.31%
10.05	11.51	11.03	-4.14%	11.33	11.24	-0.80%	11.16	11.08	-0.72%
		S5.1			S5.2			S5.3	
3.6	2.34	2.44	3.94%	2.37	2.46	3.68%	2.38	2.46	3.47%
7.3	8.18	8.85	8.10%	8.30	8.91	7.36%	8.29	8.90	7.40%
10.05	11.07	11.01	-0.60%	11.18	10.93	-2.24%	11.03	11.10	0.61%

Table 4-37 Mean out-of-plane horizontal displacement comparison - building with the second roof structure with complete and reduced cross-section timber elements [mm]

	Complete cross-section	Reduced cross-section	Comparison
1st floor	2.29	2.42	+5%
2nd floor	7.77	8.69	+10%
top floor	11.44	11.33	0%

The displacement comparison, between the two assessed cases, for the third roof structure (Table 4-38, Table 4-39), shows that on the first floor the differences between the displacements are rather low, despite presenting a 20% increase in the reduced cross-section case.

At the second floor, when reducing the cross-section of the timber elements, the horizontal displacement of the historic masonry building is increasing with 20 up to 25%, while the top floor is presenting two different behaviours of the historic masonry structure. On the one hand, rigid joint scenarios present a up to 20% decrease of the displacement of the historic masonry building when reducing the cross-section of the timber elements with 20%. The other scenarios, on the other hand, show almost the same top horizontal displacement of the structure for the two assessed cases, the differences reaching up to 5%.

Table 4-38 Out-of-plane horizontal displacement comparison - building with the third roof structure with complete and reduced cross-section timber elements [mm]

	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison	Displacement complete cross-section [mm]	Displacement reduced cross-section [mm]	Comparison
		S1.1			S1.2			S1.3	
3.6	1.77	2.17	22.76%	1.88	2.13	13.62%	1.78	2.14	20.30%
7.3	5.92	7.29	23.25%	6.32	7.24	14.52%	5.90	7.26	23.20%
10.05	17.57	14.04	-20.08%	18.06	14.29	-20.84%	17.99	14.23	-20.92%
		S2.1			S2.2			S2.3	
3.6	1.94	2.25	16.00%	1.98	2.23	12.48%	1.99	2.23	12.09%
7.3	6.38	7.84	22.93%	6.30	7.72	22.56%	6.30	7.73	22.65%
10.05	15.06	14.41	-4.32%	15.46	14.97	-3.18%	15.39	15.00	-2.53%
		S3.1			S3.2			S3.3	
3.6	2.01	2.23	11.02%	1.98	2.20	11.25%	1.98	2.21	11.31%
7.3	6.43	7.78	20.88%	6.27	7.62	21.57%	6.27	7.64	21.80%
10.05	15.22	14.10	-7.33%	15.36	14.66	-4.55%	15.30	14.67	-4.12%
		S4.1			S4.2			S4.3	
3.6	1.90	2.24	17.90%	1.99	2.22	11.60%	1.99	2.23	11.75%
7.3	6.34	7.79	22.89%	6.29	7.67	22.07%	6.28	7.69	22.34%
10.05	15.40	13.94	-9.50%	15.83	14.78	-6.64%	15.76	14.79	-6.14%
		S5.1			S5.2			S5.3	
3.6	2.04	2.20	7.92%	2.01	2.18	8.03%	2.02	2.18	8.02%
7.3	6.72	7.80	16.10%	6.54	7.61	16.35%	6.53	7.62	16.65%
10.05	14.40	13.88	-3.59%	14.53	14.39	-1.00%	14.28	14.38	0.72%

Table 4-39 Mean out-of-plane horizontal displacement comparison - building with the third roof structure with complete and reduced cross-section timber elements [mm]

	Complete cross-section	Reduced cross-section	Comparison
1st floor	1.95	2.20	+15%
2nd floor	6.32	7.62	+20%
top floor	15.71	14.44	-10%

4.3.2 Inter-story drift analysis

4.3.2.1 Building with no roof structure

Considering the inter-story drift of the building without roof structure, it was observed that it is increasing with every floor of the building from 0.06% on the first floor up to 0.46% at the last. At the same time, it was observed that at the last floor the drift is more than twice as the one recorded at the floor below highlighting the fact that the building is highly vulnerable in the top part and that the seismic action is leading to a pronounced out-of-plane behaviour of the exterior historic masonry wall.

4.3.2.2 Roof structures with complete cross-section timber elements

The presence of the first roof structure as already presented in the displacement analysis is significantly changing the behaviour of the historic masonry building. Concerning the inter-story drift, the analysis showed a significant decrease in the values, mainly on the top floor, due to the presence of the 18th-century roof structure (Table 4-40, Fig. 4.20).

Table 4-40 Out-of-plane drift and inter-story drift of the scenarios - building with the first roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Story	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]
	S1.1			S1.2			S1.3		
3.6	1.84	0.05	-15.71	2.22	0.06	1.66	1.94	0.05	-11.03
7.3	3.59	0.10	-29.58	4.80	0.13	-5.86	3.93	0.11	-23.01
10.05	3.31	0.12	-73.65	3.55	0.13	-71.75	4.42	0.16	-64.90
	S2.1			S2.2			S2.3		
3.6	2.17	0.06	-0.68	2.24	0.06	2.34	2.01	0.06	-7.85
7.3	4.73	0.13	-7.28	4.87	0.13	-4.53	4.40	0.12	-13.62
10.05	2.08	0.08	-83.50	4.02	0.15	-68.04	3.56	0.13	-71.67
	S3.1			S3.2			S3.3		
3.60	2.26	0.06	3.40	2.22	0.06	1.74%	2.06	0.06	-5.59
7.30	5.07	0.14	-0.61	4.87	0.13	-4.47%	4.63	0.12	-9.19
10.05	1.84	0.07	-85.38	4.03	0.15	-67.94%	3.35	0.12	-73.36
	S4.1			S4.2			S4.3		
3.60	2.20	0.06	0.68%	2.17	0.06	-0.53%	1.63	0.05	-25.23
7.30	4.81	0.13	-5.73%	4.67	0.13	-8.48%	3.02	0.08	-40.68
10.05	2.07	0.08	-83.53%	5.52	0.20	-56.10%	5.25	0.19	-58.28
	S5.1			S5.2			S5.3		
3.60	2.23	0.06	1.96	2.13	0.06	-2.57	1.89	0.05	-13.44
7.30	5.02	0.14	-1.52	4.58	0.12	-10.13	4.01	0.11	-21.39
10.05	1.92	0.07	-84.76	5.47	0.20	-56.48	4.66	0.17	-62.99

At the first floor of the building, the analysis showed a mean inter-story drift of 0.06%, which represents a reduction of approximately 5% compared to the no roof structure case. At this floor, the minimum inter-story drift was recorded for the S4.3 scenario leading to a reduction of the inter-story drift of 25% while the maximum was recorded at the S3.1 scenario with an increase of the value with 5%. At this floor, more than half of the scenarios present a decrease of the inter-story drift, mainly at the rigid support semi-rigid joints scenarios (S3.1, S4.1 and S5.1) and sliding support scenarios (S1.2, S2.2 and S3.2).

At the 2nd floor, a mean reduction of 10% of the inter-story drift was recorded with a minimum drift of 0.08% observed at the S4.3 scenario, representing a reduction of 40% of the value compared to the no roof structure case and a maximum of 0.14% at the S3.1 scenario, representing an almost insignificant decrease of the value. Already at this floor, all the scenarios are highlighting the effect of the presence of the roof structure, all the recorded inter-story drifts showing a decrease compared to the no roof case.

Finally, on the top floor, the effect of the roof structure can be observed more clearly, the inter-story drift presenting a mean decrease of 70% (Table 4-43). The minimum inter-story drift was recorded for the S3.1 scenario presenting a decrease of 85% while the maximum was recorded at the S4.2 and S4.3 scenario, with a decrease of 55% (Table 4-44, Table 4-45).

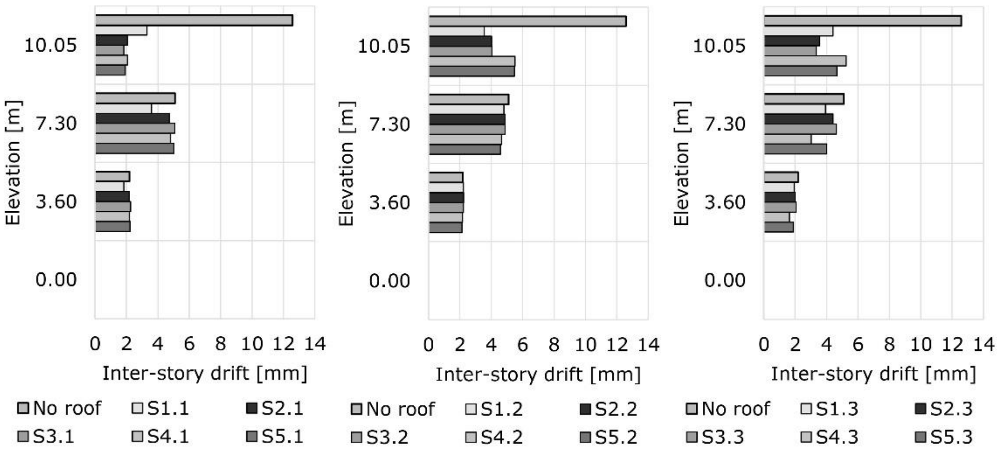


Fig. 4.20 Out-of-plane drift analysis of the scenarios - building with the first roof structure with complete cross-section timber elements (comparison to the no roof structure case)

As already observed in the horizontal displacement analysis the presence of the second roof structure is causing a completely different behaviour of the historic masonry building, showing mainly an increase of the inter-story drift for the first two floors while significantly reducing its values only on the top floor (Table 4-41, Fig. 4.21).

Therefore, at the first-floor a mean increase of the inter-story drift of 5% was observed with a minimum of 0.06%, recorded at the S1.2 scenario representing an almost insignificant decrease compared to the no roof structure case. The maximum inter-story drift at this floor was recorded for the S5.3 scenario, causing an increase of 10%. At this floor, only the S1.1 and S1.2 scenarios, are presenting a slight decrease of the inter-story drift.

On the 2nd floor, a mean increase of the inter-story drift of 10% was observed, with a minimum of 0.14%, representing a 5% decrease of its value recorded at the S1.2 scenario and a maximum of 0.16% recorded at the S5.3 scenario representing an increase of 15%. On this floor, all the scenarios except the ones involving rigid joints are presenting an increase of the inter-story drift.

Finally, on the top floor, all the scenarios are presenting a significant decrease of the inter-story drift of about 70% (Table 4-43). On this floor, the minimum was recorded at the S5.3 scenario presenting a decrease of 80% while the maximum was recorded at the S1.2 scenario, presenting a decrease of only 60% (Table 4-44, Table 4-45).

While comparing the support types, it was observed that for all three support scenarios the maximum inter-story drift for the first and second floor was recorded for the Heimeshoff and Köhler determined joints (S5.1, S5.2 and S5.3) while the minimum was noted at the rigid joints (S1.1, S1.2, S1.3). On the top floor, on the other hand, the maximum drift was recorded for rigid joints (S1.1, S1.2, S1.3), and the minimum for Heimeshoff and Köhler determined joints (S5.1, S5.2 and S5.3). Therefore, it was observed that for each support, the maximum for the top floor represents the minimum for the first and second while the minimum from the top floor represents the maximum from the two floors beneath.

Table 4-41 Out-of-plane drift and inter-story drift of the scenarios - building with the second roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Story	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]
	S1.1			S1.2			S1.3		
3.6	2.18	0.06	-0.15	2.17	0.06	-0.60	2.19	0.06	0.23
7.3	4.91	0.13	-3.75	4.88	0.13	-4.34	4.93	0.13	-3.33
10.05	4.72	0.17	-62.47	5.32	0.19	-57.73	5.13	0.19	-59.21
	S2.1			S2.2			S2.3		
3.6	2.25	0.06	3.02	2.31	0.06	5.66	2.31	0.06	5.82
7.3	5.39	0.15	5.73	5.60	0.15	9.90	5.59	0.15	9.68
10.05	3.88	0.14	-69.19	3.41	0.12	-72.88	3.26	0.12	-74.11
	S3.1			S3.2			S3.3		
3.60	2.26	0.06	3.32	2.31	0.06	5.89	2.32	0.06	6.04
7.30	5.42	0.15	6.25	5.62	0.15	10.26	5.61	0.15	10.00
10.05	3.80	0.14	-69.77	3.37	0.12	-73.22	3.21	0.12	-74.45
	S4.1			S4.2			S4.3		
3.60	2.25	0.06	3.17	2.31	0.06	5.82	2.31	0.06	5.97
7.30	5.41	0.15	6.18	5.62	0.15	10.23	5.61	0.15	10.00
10.05	3.84	0.14	-69.48	3.40	0.12	-73.01	3.24	0.12	-74.24
	S5.1			S5.2			S5.3		
3.60	2.34	0.07	7.33	2.37	0.07	8.69	2.38	0.07	8.84
7.30	5.84	0.16	14.53	5.92	0.16	16.15	5.91	0.16	15.95
10.05	2.89	0.11	-77.02	2.88	0.10	-77.07	2.74	0.10	-78.21

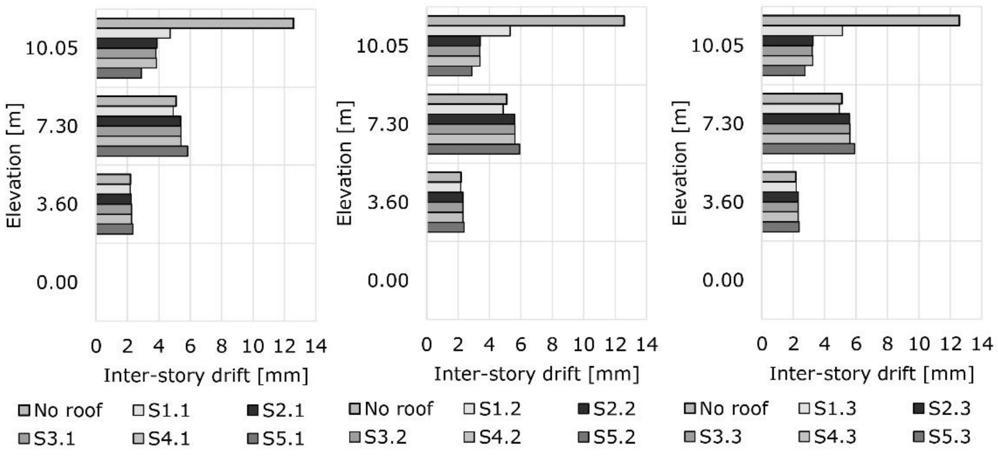


Fig. 4.21 Out-of-plane drift analysis of the scenarios - building with the second roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Ultimately the presence of the 3rd roof structure is also presenting a completely different behaviour of the historic masonry building causing a decrease of the inter-story drifts for all the floors, for all the assessed scenarios (Table 4-42, Fig. 4.22).

On the 1st floor, the presence of the roof structure is causing a mean reduction of the inter-story drift of 10% with a minimum recorded for the S1.3 scenario representing a reduction of 20% and a maximum recorded at the S5.1 scenario representing a reduction of 5%. The differences between the support scenarios reach up to 5%.

On the 2nd floor, the analysis showed a mean reduction of 15%. The minimum, in this case, was recorded for the S1.3 scenario, presenting a reduction of 20% of the inter-story drift, while the maximum was also recorded for the S5.1 scenario, with a reduction of 10%. Even at the second floor, the inter-story drift presents certain similarities, the scenarios involving hinged joints, Hölzer, component method and Heimeshoff and Köhler determined joints presenting similar values for sliding supports and hinged-sliding supports while the rigid joints are presenting similar inter-story drifts for rigid supports and hinged-sliding supports.

At the top floor, a mean reduction of the inter-story drift of 25% was recorded (Table 4-43). On this floor, the minimum was observed at the S5.1 scenario causing a reduction of 40%, and the maximum was recorded for the S1.3 scenario causing a reduction of only 5%.

While comparing the scenarios, considering the axial stiffness of the joints, it was observed that for the first and second floor the minimum inter-story drift was recorded for rigid joints (S1.1, S1.2 and S1.3) while the maximum was recorded for Heimeshoff and Köhler determined joints (S5.1, S5.2 and S5.3). On the top floor, on the other hand, the maximum was recorded for rigid joints while the Heimeshoff and Köhler determined joints present the minimum inter-story drift (Table 4-44, Table 4-45). Like for the second roof structure, in this case, the maximum inter-story drift from the top floor represents the minimum from the first and second while the minimum from the third represents the maximum from the two floors beneath.

Table 4-42 Out-of-plane drift and inter-story drift of the scenarios - building with the third roof structure with complete cross-section timber elements (comparison to the no roof case)

Story	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]
		S1.1			S1.2			S1.3	
3.6	1.77	0.05	-19.03	1.88	0.05	-14.05	1.78	0.05	-18.50
7.3	4.15	0.11	-18.61	4.44	0.12	-12.88	4.12	0.11	-19.29
10.05	11.65	0.42	-7.41	11.74	0.43	-6.69	12.10	0.44	-3.84
		S2.1			S2.2			S2.3	
3.6	1.94	0.05	-11.25	1.98	0.06	-9.21	1.99	0.06	-8.76
7.3	4.44	0.12	-12.98	4.31	0.12	-15.40	4.31	0.12	-15.50
10.05	8.69	0.32	-30.94	9.16	0.33	-27.15	9.09	0.33	-27.75
		S3.1			S3.2			S3.3	
3.60	2.01	0.06	-8.16	1.98	0.06	-9.37	1.98	0.06	-9.21
7.30	4.43	0.12	-13.17	4.29	0.12	-15.83	4.29	0.12	-15.86
10.05	8.78	0.32	-30.18	9.09	0.33	-27.75	9.02	0.33	-28.27
		S4.1			S4.2			S4.3	
3.60	1.90	0.05	-13.07	1.99	0.06	-8.84	1.99	0.06	-8.76
7.30	4.44	0.12	-12.98	4.29	0.12	-15.76	4.29	0.12	-15.83
10.05	9.06	0.33	-27.95	9.54	0.35	-24.15	9.47	0.34	-24.69
		S5.1			S5.2			S5.3	
3.60	2.04	0.06	-6.50	2.01	0.06	-7.78	2.02	0.06	-7.70
7.30	4.68	0.13	-8.22	4.52	0.12	-11.26	4.51	0.12	-11.49
10.05	7.68	0.28	-38.96	7.99	0.29	-36.45	7.75	0.28	-38.39

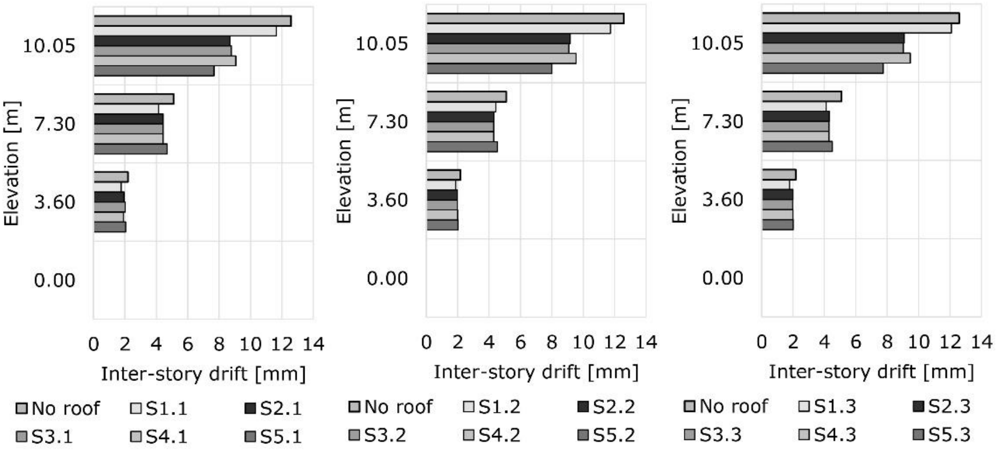


Fig. 4.22 Out-of-plane drift analysis of the scenarios - building with the third roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Table 4-43 Mean out-of-plane drift and inter-story drift analysis of the building without a roof and with the three roof structures with complete cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Drift [mm]	2.18	2.08	2.29	1.95
	Inter-story drift [%]	0.06	0.06	0.06	0.05
	Compared to no roof		-5%	5%	-10%
2nd floor	Drift [mm]	5.10	4.47	5.48	4.37
	Inter-story drift [%]	0.14	0.12	0.15	0.12
	Compared to no roof		-10%	10%	-15%
top floor	Drift [mm]	12.58	3.67	3.67	9.39
	Inter-story drift [%]	0.46	0.10	0.10	0.26
	Compared to no roof		-70%	-70%	-25%

Table 4-44 Minimum out-of-plane drift and inter-story drift analysis of the building without a roof and with the three roof structures with complete cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Drift [mm]	2.18	1.63	2.17	1.78
	Inter-story drift [%]	0.06	0.04	0.06	0.05
	Compared to no roof		-25%	0%	-20%
	Scenario		S4.3	S1.2	S1.3
2nd floor	Drift [mm]	5.10	3.02	4.88	4.12
	Inter-story drift [%]	0.14	0.08	0.14	0.11
	Compared to no roof		-40%	-5%	-20%
	Scenario		S4.3	S1.2	S1.3
top floor	Drift [mm]	12.58	1.84	2.74	7.68
	Inter-story drift [%]	0.46	0.05	0.07	0.21
	Compared to no roof		-85%	-80%	-40%
	Scenario		S3.1	S5.3	S5.1

Table 4-45 Maximum out-of-plane drift and inter-story drift analysis of the building without a roof and with the three roof structures with complete cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Drift [mm]	2.18	2.26	2.38	2.04
	Inter-story drift [%]	0.06	0.06	0.06	0.06
	Compared to no roof		+5%	+10%	-5%
	Scenario		S3.1	S5.3	S5.1
2nd floor	Drift [mm]	5.10	5.07	5.91	4.68
	Inter-story drift [%]	0.14	0.14	0.16	0.13
	Compared to no roof		0%	15%	-10%
	Scenario		S3.1	S5.3	S5.1
top floor	Drift [mm]	12.58	5.52	5.32	12.10
	Inter-story drift [%]	0.46	0.15	0.15	0.34
	Compared to no roof		-55%	-60%	-5%
	Scenario		S4.2	S1.2	S1.3

4.3.2.3 Roof structures with 20% reduced cross-section timber elements

Starting from the displacements recorded for the historic masonry building with the decayed roof structures, the inter-story drift was also analysed.

The presence of the decayed roof structure is still leading to a slight reduction of the inter-story drift for the first two floors while significantly influencing its values at the top floor (Table 4-46, Fig. 4.23). Therefore, on the first floor, a mean reduction of 1.5% was observed with a minimum recorded for the S5.2 scenario, which represents a 10% reduction of the inter-story drift while the maximum was recorded at the S3.1 scenario presenting an increase of 10%. At this floor, all the scenarios involving rigid support are presenting an increase of 5 up to 10% of the inter-story drift. All the other scenarios show that despite the decay, the presence of the roof structure is still reducing the horizontal displacement of the building. On the second floor, a mean reduction of the inter-story drift of 5% was observed. At this floor the minimum was also recorded for the S5.2 scenario, with a reduction of 15% of the inter-story drift, while the maximum was recorded for the S3.1 scenario, still presenting an increase of 10%. Even on this floor, all the scenarios involving a rigid support except for the S1.1 scenario are presenting an increase of the inter-story drift of about 10%. Ultimately on the top floor, a mean reduction of 60% was recorded (Table 4-49), with a minimum observed at the S3.1 scenario, presenting a reduction of 80%. The maximum, on the other hand, was recorded for the S4.2 scenario, presenting a reduction of 35% (Table 4-50, Table 4-51).

Table 4-46 Out-of-plane drift and inter-story drift of the scenarios - building with the first roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Story	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]
	S1.1			S1.2			S1.3		
3.6	2.23	0.06	2.27	2.11	0.06	-3.63	2.00	0.06	-8.38
7.3	4.99	0.13	-2.04	4.66	0.13	-8.54	4.34	0.12	-14.82
10.05	2.73	0.10	-78.33	3.64	0.13	-71.05	4.41	0.16	-64.94
	S2.1			S2.2			S2.3		
3.6	2.35	0.07	7.55	2.14	0.06	-2.11	2.12	0.06	-3.10
7.3	5.42	0.15	6.28	4.69	0.13	-7.99	4.90	0.13	-3.82
10.05	2.69	0.10	-78.62	5.92	0.22	-52.94	4.62	0.17	-63.26
	S3.1			S3.2			S3.3		
3.60	2.35	0.07	7.70	2.11	0.06	-3.47	2.11	0.06	-3.55
7.30	5.53	0.15	8.41	4.76	0.13	-6.63	5.00	0.14	-1.84
10.05	2.68	0.10	-78.69	5.86	0.21	-53.42	4.45	0.16	-64.62
	S4.1			S4.2			S4.3		
3.60	2.33	0.06	6.87	2.03	0.06	-1.99	2.05	0.06	-5.97
7.30	5.45	0.15	6.99	4.46	0.12	-3.32	4.79	0.13	-6.05
10.05	2.93	0.11	-76.73	8.16	0.30	-35.16	6.43	0.23	-48.92
	S5.1			S5.2			S5.3		
3.60	2.28	0.06	4.46	2.00	0.06	-8.61	2.04	0.06	-6.65
7.30	5.46	0.15	7.06	4.34	0.12	-14.95	4.77	0.13	-6.47
10.05	2.85	0.10	-77.35	7.77	0.28	-38.25	6.19	0.23	-50.80

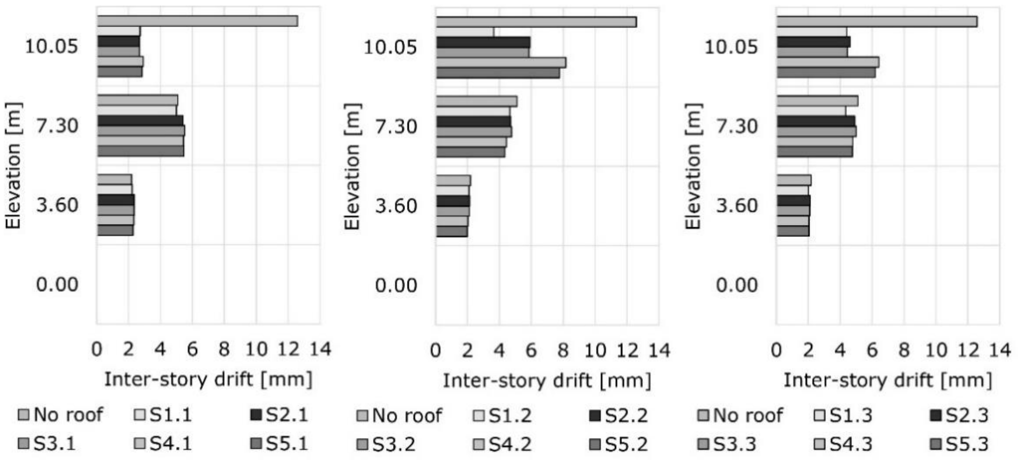


Fig. 4.23 Out-of-plane drift analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

The inter-story drift analysis of the building with the decayed second roof structure is once again highlighting the effect of the roof on the behaviour of a historical building during seismic events (Table 4-47, Fig. 4.24). Its effect can be observed on all the floors of the building, the presence of the decayed roof increasing the inter-story drift on the first two floors, while significantly decreasing it on the 3rd. Therefore, on the first floor, a mean inter-story drift of 0.07% was observed, representing an increase of about 10% compared to the no roof structure case. On this floor, the minimum inter-story drift was recorded for the S1.1 scenario presenting an increase of 10% while the maximum was recorded for the S5.2 scenario, increasing the drift with 15%. All the considered scenarios present on this floor a clear increase of the inter-story drift of 10 up to 15%. On the second floor, the inter-story drift is even higher, presenting a mean of 0.17%, representing an increase of 25%. The minimum on this floor was recorded for the S1.1 scenario presenting an increase of 15% while the maximum was also recorded for the S5.2 scenario, with an increase of 25%. As for the first floor, all the scenarios present a clear increase of the inter-story drift of 15 up to 25%. The top floor is clearly presenting an improvement of the seismic behaviour of the building due to the presence of the roof, presenting a reduction of the inter-story drift of 80% (Table 4-49). On this floor the minimum inter-story drift of 0.05% was obtained, representing a reduction of 85%, recorded for the S5.2 scenario, while the maximum was recorded for the S1.2 scenario meaning a reduction of 65%. All the scenarios present a decrease of the inter-story drift of 65 up to 80% (Table 4-50, Table 4-51).

Like in the complete cross-section case, while comparing the scenarios, it was observed that for all three support types the maximum inter-story drift for the first and second floor was recorded for the Heimeshoff and Köhler method determined joints (S5.1, S5.2 and S5.3) while the minimum was recorded for rigid joints (S1.1, S1.2, S1.3). On the top floor, on the other hand, the maximum drift was recorded for rigid joints (S1.1, S1.2, S1.3), while the minimum for Heimeshoff and Köhler determined joints (S5.1, S5.2 and S5.3). Therefore, it was also observed that for each joint type the maximum for the top floor represents the minimum for the first and second while the minimum from the top floor represents the maximum from the two floors beneath.

Table 4-47 Out-of-plane drift and inter-story drift of the scenarios - building with the second roof structure with reduced cross-section timber elements (comparison to the no roof case)

Story	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]
	S1.1			S1.2			S1.3		
3.6	2.37	0.07	8.69	2.38	0.07	8.91	2.38	0.07	8.99
7.3	5.87	0.16	15.18	5.89	0.16	15.60	5.89	0.16	15.50
10.05	3.88	0.14	-69.16	4.26	0.15	-66.16	4.10	0.15	-67.42
	S2.1			S2.2			S2.3		
3.6	2.41	0.07	10.42	2.44	0.07	11.78	2.45	0.07	11.93
7.3	6.28	0.17	23.20	6.38	0.17	25.15	6.37	0.17	24.85
10.05	2.33	0.08	-81.51	2.40	0.09	-80.89	2.26	0.08	-82.03
	S3.1			S3.2			S3.3		
3.60	2.41	0.07	10.42	2.41	0.07	10.35	2.45	0.07	11.93
7.30	6.29	0.17	23.30	6.29	0.17	23.37	6.37	0.17	24.89
10.05	2.32	0.08	-81.53	2.43	0.09	-80.69	2.26	0.08	-82.02
	S4.1			S4.2			S4.3		
3.60	2.41	0.07	10.42	2.44	0.07	11.78	2.45	0.07	11.93
7.30	6.29	0.17	23.40	6.39	0.17	25.28	6.37	0.17	25.02
10.05	2.33	0.08	-81.51	2.41	0.09	-80.86	2.26	0.08	-82.00
	S5.1			S5.2			S5.3		
3.60	2.44	0.07	11.56	2.46	0.07	12.69	2.46	0.07	12.61
7.30	6.41	0.17	25.73	6.44	0.17	26.41	6.44	0.17	26.38
10.05	2.16	0.08	-82.82	2.02	0.07	-83.92	2.19	0.08	-82.56

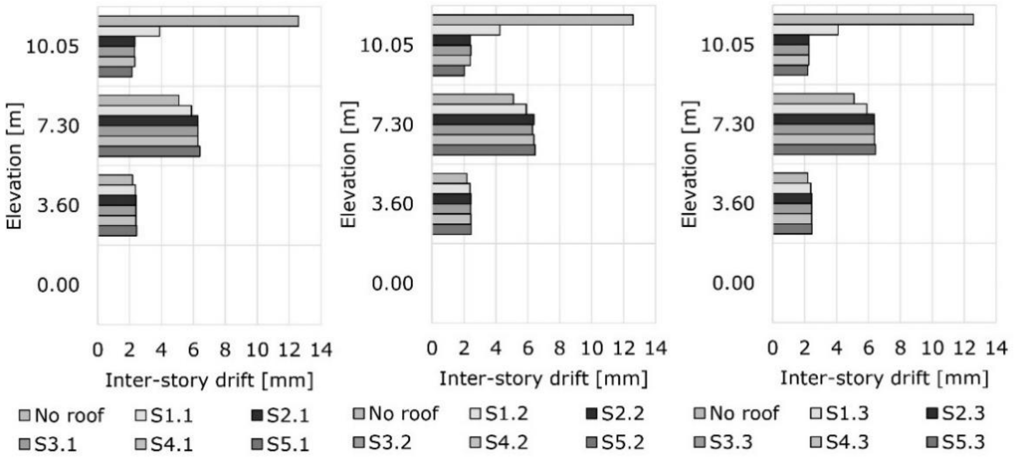


Fig. 4.24 Out-of-plane drift analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

The presence of the third decayed roof structure is once again presenting a different effect on the seismic behaviour of the building, changing the inter-story drift differently on each floor and highlighting the differences between the assessed scenarios (Table 4-48, Fig. 4.25).

Table 4-48 Out-of-plane drift and inter-story drift of the scenarios - building with the third roof structure with reduced cross-section timber elements (comparison to the no roof case)

Story	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]	Drift [mm]	Inter-story drift [%]	Compared to no roof [%]
	S1.1			S1.2			S1.3		
3.6	2.17	0.06	-0.60	2.13	0.06	-2.34	2.14	0.06	-1.96
7.3	5.12	0.14	0.49	5.10	0.14	0.10	5.12	0.14	0.45
10.05	6.74	0.25	-46.39	7.06	0.26	-43.90	6.96	0.25	-44.64
	S2.1			S2.2			S2.3		
3.6	2.25	0.06	2.95	2.23	0.06	2.11	2.23	0.06	2.27
7.3	5.59	0.15	9.61	5.49	0.15	7.61	5.49	0.15	7.77
10.05	6.58	0.24	-47.73	7.25	0.26	-42.35	7.27	0.26	-42.20
	S3.1			S3.2			S3.3		
3.60	2.23	0.06	1.96	2.20	0.06	0.83	2.21	0.06	1.06
7.30	5.55	0.15	8.83	5.42	0.15	6.34	5.43	0.15	6.57
10.05	6.32	0.23	-49.72	7.04	0.26	-44.07	7.03	0.26	-44.15
	S4.1			S4.2			S4.3		
3.60	2.24	0.06	2.49	2.22	0.06	1.74	2.23	0.06	1.96
7.30	5.55	0.15	8.80	5.45	0.15	6.93	5.46	0.15	7.12
10.05	6.15	0.22	-51.11	7.10	0.26	-43.53	7.10	0.26	-43.53
	S5.1			S5.2			S5.3		
3.60	2.20	0.06	0.91	2.18	0.06	-0.38	2.18	0.06	-0.30
7.30	5.60	0.15	9.84	5.43	0.15	6.54	5.44	0.15	6.67
10.05	6.08	0.22	-51.67	6.78	0.25	-46.10	6.77	0.25	-46.22

On the first floor of the building, the inter-story drift is suffering only little change compared to the no roof structure case, the minimum inter-story drift presenting a reduction of only 2.5%, for the S1.2 scenario, while the maximum is showing a slight increase of about 2% recorded for the S2.1 scenario.

On this floor, only the scenarios involving rigid joints are presenting a decrease of the inter-story drift, the other ones showing a slight increase of a maximum of 2%. The second floor is starting to present specific changes in the inter-story drift showing a mean increase of 5%. The minimum on this floor was recorded for the S1.2 scenario, showing only a slight decrease of the inter-story drift of about 2% while the maximum was recorded for the S5.1 scenario representing an increase of 10%. Even on this floor, the scenarios present different behaviours of the building for the S5.2 and S5.3 scenarios being the only ones displaying a decrease of the inter-story drift, while all the others are showing an increase of up to 10%.

On the top floor, the roof structure is starting to influence the inter-story drift, leading to a reduction of 45% (Table 4-49). The minimum on this floor was recorded for the S5.1 scenario leading to a reduction of the inter-story drift of 50% while the maximum was recorded for the S2.3 scenario leading to a reduction of only 40%. Still, on this floor, all the scenarios are presenting a clear reduction of the inter-story drift of 40 up to 50% (Table 4-50, Table 4-51).

When comparing the support types, in this case, no clear pattern of the changes between the minimum and maximum inter-story drifts could be observed.

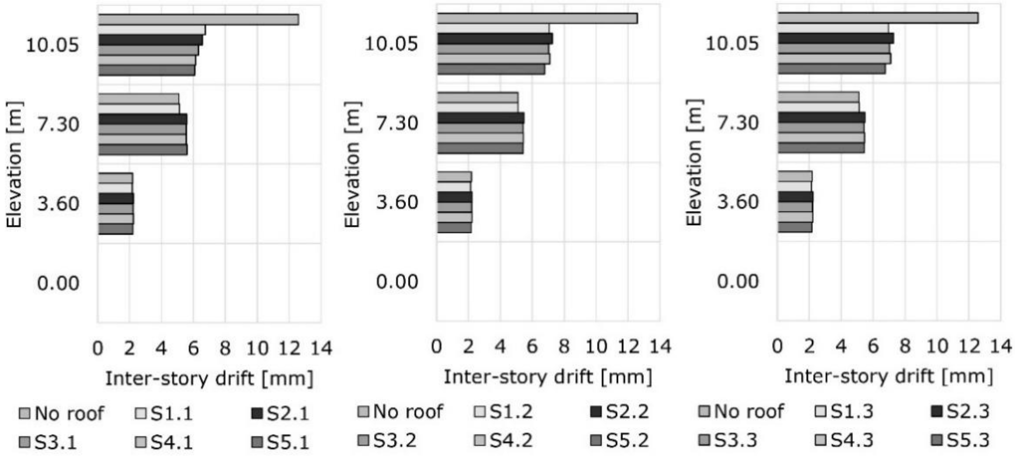


Fig. 4.25 Out-of-plane drift analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Table 4-49 Mean out-of-plane drift and inter-story drift analysis of the building without a roof and with the three roof structures with reduced cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Drift [mm]	2.18	2.15	2.42	2.20
	Inter-story drift [%]	0.06	0.06	0.07	0.06
	Compared to no roof		0%	10%	0%
2nd floor	Drift [mm]	5.10	4.90	6.27	5.42
	Inter-story drift [%]	0.14	0.14	0.17	0.15
	Compared to no roof		-5%	25%	5%
top floor	Drift [mm]	12.58	4.75	2.64	6.82
	Inter-story drift [%]	0.46	0.13	0.07	0.19
	Compared to no roof		-60%	-80%	-45%

Table 4-50 Minimum out-of-plane drift and inter-story drift analysis of the building without a roof and with the three roof structures with reduced cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Drift [mm]	2.18	2.00	2.37	2.13
	Inter-story drift [%]	0.06	0.05	0.06	0.06
	Compared to no roof		-10%	10%	0%
	Scenario		S5.2	S1.1	S1.2
2nd floor	Drift [mm]	5.10	4.34	5.87	5.10
	Inter-story drift [%]	0.14	0.12	0.16	0.14
	Compared to no roof		-15%	15%	0%
	Scenario		S5.2	S1.1	S1.2
top floor	Drift [mm]	12.58	2.68	2.02	6.08
	Inter-story drift [%]	0.46	0.07	0.05	0.17
	Compared to no roof		-80%	-85%	-50%
	Scenario		S3.1	S5.2	S5.1

Table 4-51 Maximum out-of-plane drift and inter-story drift analysis of the building without a roof and with the three roof structures with reduced cross-section timber elements

		No roof	1st roof structure	2nd roof structure	3rd roof structure
1st floor	Drift [mm]	2.18	2.35	2.46	2.25
	Inter-story drift [%]	0.06	0.06	0.07	0.06
	Compared to no roof		10%	15%	0%
	Scenario		S3.1	S5.2	S2.1
2nd floor	Drift [mm]	5.10	5.53	6.44	5.60
	Inter-story drift [%]	0.14	0.15	0.18	0.15
	Compared to no roof		10%	25%	10%
	Scenario		S3.1	S5.2	S5.1
top floor	Drift [mm]	12.58	8.16	4.26	7.27
	Inter-story drift [%]	0.46	0.22	0.12	0.20
	Compared to no roof		-35%	-65%	-40%
	Scenario		S4.2	S1.2	S2.3

4.3.2.4 Comparison

In order to better understand the effect of the decay of the timber elements on the seismic behaviour of the historic masonry structure, the obtained results for the complete and reduced cross-section timber elements were subsequently compared considering the drift of the building.

For the first roof structure (Table 4-52, Table 4-53), the inter-story drift on the first floor is slightly varying according to the scenarios, presenting both an increase and decrease of the values. Sliding support scenarios are the only ones presenting a decrease of the drift when the roof structure is decayed, of around 5% compared to the complete cross-section case. All the other scenarios present a clear increase of 5 up to 25%, recorded in the S4.3 scenario.

On the second floor, the inter-story drift is presenting an increase when the cross-section of the timber elements is reduced, for all the assessed scenarios except for the sliding support scenarios where the drift of the two cases is approximately equal.

On the top floor, all the assessed scenarios present a significant increase of the drift for the reduced cross-section roof structure, even for the sliding support scenarios where the drift increase is rather significant, of up to 40-45%.

Table 4-52 Out-of-plane horizontal drift comparison - building with the first roof structure with complete and reduced cross-section timber elements [mm]

	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison
	S1.1			S1.2			S1.3		
3.6	1.84	2.23	21.33%	2.22	2.11	-5.20%	1.94	2.00	2.97%
7.3	3.59	4.99	39.11%	4.80	4.66	-2.85%	3.93	4.34	10.63%
10.05	3.31	2.73	-17.77%	3.55	3.64	2.46%	4.42	4.41	-0.11%
	S2.1			S2.2			S2.3		
3.6	2.17	2.35	8.29%	2.24	2.14	-4.35%	2.01	2.12	5.16%
7.3	4.73	5.42	14.62%	4.87	4.69	-3.63%	4.40	4.90	11.35%
10.05	2.08	2.69	29.57%	4.02	5.92	47.23%	3.56	4.62	29.68%
	S3.1			S3.2			S3.3		
3.6	2.26	2.35	4.16%	2.22	2.11	-5.12%	2.06	2.11	2.16%
7.3	5.07	5.53	9.08%	4.87	4.76	-2.27%	4.63	5.00	8.09%
10.05	1.84	2.68	45.74%	4.03	5.86	45.29%	3.35	4.45	32.79%
	S4.1			S4.2			S4.3		
3.6	2.20	2.33	6.15%	2.17	2.03	-6.38%	1.63	2.05	25.76%
7.3	4.81	5.45	13.49%	4.67	4.46	-4.53%	3.02	4.79	58.37%
10.05	2.07	2.93	41.24%	5.52	8.16	47.77%	5.25	6.43	22.41%
	S5.1			S5.2			S5.3		
3.6	2.23	2.28	2.44%	2.13	2.00	-6.20%	1.89	2.04	7.85%
7.3	5.02	5.46	8.71%	4.58	4.34	-5.37%	4.01	4.77	18.98%
10.05	1.92	2.85	48.62%	5.47	7.77	41.89%	4.66	6.19	32.92%

Table 4-53 Mean out-of-plane horizontal drift comparison - building with the first roof structure with complete and reduced cross-section timber elements [mm] [mm]

	Complete cross-section	Reduced cross-section	Comparison
1st floor	2.08	2.15	+5%
2nd floor	4.47	4.90	+10%
top floor	3.67	4.75	+30%

When placing the second roof structure on the historic masonry building, the structure is suffering an increase of the drift on the first and second floor while presenting an apparent decrease at the top floor (Table 4-54, Table 4-55).

On the first floor, the inter-story drift is approximately equal for the 2 assessed cases for all the assessed scenarios, presenting an increase of up to 10%. On the second floor, on the other hand, the reduced section case is presenting a slight increase of the drift for all the scenarios of up to 20%.

Ultimately, on the top floor, the decay of the timber elements is causing a decrease of the drift compared to the complete cross-section case, of up to 25-40%. The most significant difference between the two cases on this floor was recorded for the S2.1, S3.1 and S4.1 scenario.

Table 4-54 Out-of-plane horizontal drift comparison - building with the second roof structure with complete and reduced cross-section timber elements [mm]

	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison
		S1.1			S1.2			S1.3	
3.6	2.18	2.37	8.85%	2.17	2.38	9.57%	2.19	2.38	8.74%
7.3	4.91	5.87	19.67%	4.88	5.89	20.84%	4.93	5.89	19.48%
10.05	4.72	3.88	-17.83%	5.32	4.26	-19.95%	5.13	4.10	-20.13%
		S2.1			S2.2			S2.3	
3.6	2.25	2.41	7.18%	2.31	2.44	5.79%	2.31	2.45	5.78%
7.3	5.39	6.28	16.53%	5.60	6.38	13.87%	5.59	6.37	13.84%
10.05	3.88	2.33	-39.97%	3.41	2.40	-29.55%	3.26	2.26	-30.60%
		S3.1			S3.2			S3.3	
3.6	2.26	2.41	6.87%	2.31	2.41	4.21%	2.32	2.45	5.56%
7.3	5.42	6.29	16.05%	5.62	6.29	11.89%	5.61	6.37	13.53%
10.05	3.80	2.32	-38.92%	3.37	2.43	-27.91%	3.21	2.26	-29.62%
		S4.1			S4.2			S4.3	
3.6	2.25	2.41	7.03%	2.31	2.44	5.64%	2.31	2.45	5.63%
7.3	5.41	6.29	16.21%	5.62	6.39	13.65%	5.61	6.37	13.65%
10.05	3.84	2.33	-39.41%	3.40	2.41	-29.11%	3.24	2.26	-30.14%
		S5.1			S5.2			S5.3	
3.6	2.34	2.44	3.94%	2.37	2.46	3.68%	2.38	2.46	3.47%
7.3	5.84	6.41	9.78%	5.92	6.44	8.83%	5.91	6.44	8.99%
10.05	2.89	2.16	-25.23%	2.88	2.02	-29.86%	2.74	2.19	-19.93%

Table 4-55 Mean out-of-plane horizontal drift comparison - building with the second roof structure with complete and reduced cross-section timber elements [mm]

	Complete cross-section	Reduced cross-section	Comparison
1st floor	2.29	2.42	+5%
2nd floor	5.48	6.27	+15%
top floor	3.67	2.64	-30%

The drift comparison, between the two assessed cases, for the third roof structure (Table 4-56, Table 4-57), presents almost the same pattern as the second roof structure case.

Therefore, the inter-story drift analysis shows a more significant increase of the drift for this first floor for all the assessed scenarios, of 10 up to 25%. On the second floor the differences are approximately in the same range, presenting an increase of 20 up to 25% in the decayed roof structure case.

Despite the fact that the displacement of the two cases is approximately similar on the top floor, the inter-story drift for the reduced cross-section case is around 20-30% lower than in the complete cross-section case. For rigid joint scenarios the inter-story drift decreases with up to 40% when comparing the two cases.

Table 4-56 Out-of-plane horizontal drift comparison - building with the third roof structure with complete and reduced cross-section timber elements [mm]

	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison	Drift complete cross-section [mm]	Drift reduced cross-section [mm]	Comparison
	S1.1			S1.2			S1.3		
3.6	1.77	2.17	22.76%	1.88	2.13	13.62%	1.78	2.14	20.30%
7.3	4.15	5.12	23.46%	4.44	5.10	14.90%	4.12	5.12	24.46%
10.05	11.65	6.74	-42.10%	11.74	7.06	-39.88%	12.10	6.96	-42.42%
	S2.1			S2.2			S2.3		
3.6	1.94	2.25	16.00%	1.98	2.23	12.48%	1.99	2.23	12.09%
7.3	4.44	5.59	25.96%	4.31	5.49	27.20%	4.31	5.49	27.54%
10.05	8.69	6.58	-24.31%	9.16	7.25	-20.87%	9.09	7.27	-19.99%
	S3.1			S3.2			S3.3		
3.6	2.01	2.23	11.02%	1.98	2.20	11.25%	1.98	2.21	11.31%
7.3	4.43	5.55	25.34%	4.29	5.42	26.34%	4.29	5.43	26.65%
10.05	8.78	6.32	-27.99%	9.09	7.04	-22.59%	9.02	7.03	-22.14%
	S4.1			S4.2			S4.3		
3.6	1.90	2.24	17.90%	1.99	2.22	11.60%	1.99	2.23	11.75%
7.3	4.44	5.55	25.03%	4.29	5.45	26.93%	4.29	5.46	27.26%
10.05	9.06	6.15	-32.15%	9.54	7.10	-25.56%	9.47	7.10	-25.03%
	S5.1			S5.2			S5.3		
3.6	2.04	2.20	7.92%	2.01	2.18	8.03%	2.02	2.18	8.02%
7.3	4.68	5.60	19.68%	4.52	5.43	20.06%	4.51	5.44	20.51%
10.05	7.68	6.08	-20.82%	7.99	6.78	-15.19%	7.75	6.77	-12.71%

Table 4-57 Mean out-of-plane horizontal drift comparison - building with the third roof structure with complete and reduced cross-section timber elements [mm]

	Complete cross-section	Reduced cross-section	Comparison
1st floor	1.95	2.20	+15%
2nd floor	4.37	5.42	+25%
top floor	9.39	6.82	-30%

4.3.3 Deformed shape

Like for the other assessed parameters, in order to understand the effect of the roof structures on the structural behaviour of the historic masonry building, the deformed shape of the building was evaluated.

4.3.3.1 Building with no roof structure

First the deformed shape of the building without roof structure was assessed. Since the horizontal displacement and inter-story drift are continuously rising towards the top of the building, the structure is presenting a flexural deformation.

4.3.3.2 Roof structures with complete cross-section timber elements

While analysing the deformed shape of the building with the first roof structure, it was observed that while most of the scenarios are presenting shear deformation, there are still cases where the historic masonry structure is presenting flexural deformation (S1.3, S4.2, S4.3, S5.2, S5.3), similar to the building with no roof structure. Still, its presence is changing the general behaviour of the building, significantly reducing the inter-story drift of the last floor for most of the considered scenarios (Table 4-58).

Table 4-58 Deformed shape analysis of the scenarios - building with the first roof structure with complete cross-section timber elements

	1. Rigid support	2. Sliding support	3. Hinged-sliding support
1. Rigid joints	shear	shear	flexural
2. Hinged joints	shear	shear	shear
3. Hölzer	shear	shear	shear
4. Component method	shear	flexural	flexural
5. Heimeshoff & Köhler	shear	flexural	flexural

The presence of the second roof structure has the most visible effect on the behaviour of the historic masonry building, changing the deformed shape almost completely to shear. The inter-story drift is continuously rising until the 2nd floor but is suffering a change on the third where the presence of the roof structure is reducing the horizontal displacement and ultimately the inter-story drift. Still, the S1.2 and S1.3 scenarios are presenting the same behaviour as the no roof structure case, leading to a flexural deformation of the building (Table 4-59).

Table 4-59 Deformed shape analysis of the scenarios - building with the second roof structure with complete cross-section timber elements

	1. Rigid support	2. Sliding support	3. Hinged-sliding support
1. Rigid joints	shear	flexural	flexural
2. Hinged joints	shear	shear	shear
3. Hölzer	shear	shear	shear
4. Component method	shear	shear	shear
5. Heimeshoff & Köhler	shear	shear	shear

The third roof structure has only little influence on the deformation of the building. Since the inter-story drift is continuously rising towards the top of the building, the structure is clearly presenting a flexural deformation for all the considered scenarios (Table 4-60).

Table 4-60 Deformed shape analysis of the scenarios - building with the third roof structure with complete cross-section timber elements

	1. Rigid support	2. Sliding support	3. Hinged-sliding support
1. Rigid joints	flexural	flexural	flexural
2. Hinged joints	flexural	flexural	flexural
3. Hölzer	flexural	flexural	flexural
4. Component method	flexural	flexural	flexural
5. Heimeshoff & Köhler	flexural	flexural	flexural

When comparing the deformed shape of the building with the three roof structures, the different effect of each one of them can be clearly identified (Table 4-61). The first and second roof structures have a significant influence on the deformation of the building, causing for most of the scenarios a change of the deformation from flexural to shear. However, both still present scenarios where the original flexural deformation is still visible. On the contrary, the third roof structure is clearly presenting the same behaviour of the building, showing flexural deformation for all the scenarios.

Table 4-61 Deformed shape comparison of the building with the three roof structures with complete cross-section timber elements

	Roof 1	Roof 2	Roof 3
S1.1	shear	shear	flexural
S1.2	shear	flexural	flexural
S1.3	flexural	flexural	flexural
S2.1	shear	shear	flexural
S2.2	shear	shear	flexural
S2.3	shear	shear	flexural
S3.1	shear	shear	flexural
S3.2	shear	shear	flexural
S3.3	shear	shear	flexural
S4.1	shear	shear	flexural
S4.2	flexural	shear	flexural
S4.3	flexural	shear	flexural
S5.1	shear	shear	flexural
S5.2	flexural	shear	flexural
S5.3	flexural	shear	flexural

4.3.3.3 Roof structures with 20% reduced cross-section timber elements

After the cross-section of the timber elements of the first roof structure was reduced with 20%, it was observed that it no longer has the same effect on the behaviour of the historic masonry structure. The deformed shape of the building remains mostly shear deformation, while there are still scenarios which present a flexural deformation of the building, like the S1.3, S2.2, S3.2, S4.2, S4.3, S5.2 and S5.3 scenarios. Therefore, almost half of the scenarios present a shear deformation while the other half is presenting a flexural deformation of the building (Table 4-62).

Table 4-62 Deformed shape analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements

	1. Rigid support	2. Sliding support	3. Hinged-sliding support
1. Rigid joints	shear	shear	flexural
2. Hinged joints	shear	flexural	shear
SEMI-RIGID JOINTS			
3. Hölzer	shear	flexural	shear
4. Component method	shear	flexural	flexural
5. Heimeshoff & Köhler	shear	flexural	flexural

Since the presence of the decayed roof structure is improving the seismic behaviour of the building, according to the horizontal displacement and inter-story drift analysis, the deformed shape of the building is also significantly influenced. Therefore, the decay of the second roof structure is causing a complete change of the deformation of the historic masonry building, presenting only shear deformation for all the assessed scenarios (Table 4-63).

Table 4-63 Deformed shape analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements

	1. Rigid support	2. Sliding support	3. Hinged-sliding support
1. Rigid joints	shear	shear	shear
2. Hinged joints	shear	shear	shear
SEMI-RIGID JOINTS			
3. Hölzer	shear	shear	shear
4. Component method	shear	shear	shear
5. Heimeshoff & Köhler	shear	shear	shear

Despite presenting less top horizontal displacement and better behaviour than the full cross-section case, the third roof structure is still leading to flexural deformation of the building, the inter-story drift raising continuously with the height of the building (Table 4-64).

Table 4-64 Deformed shape analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements

	1. Rigid support	2. Sliding support	3. Hinged-sliding support
1. Rigid joints	flexural	flexural	flexural
2. Hinged joints	flexural	flexural	flexural
SEMI-RIGID JOINTS			
3. Hölzer	flexural	flexural	flexural
4. Component method	flexural	flexural	flexural
5. Heimeshoff & Köhler	flexural	flexural	flexural

The decay of the roof structures has a different effect on the behaviour of the historic masonry building during the seismic event (Table 4-65).

On the one hand, the first roof structure is clearly showing that its decay is increasing the number of scenarios which are leading to a flexural deformation of the building, similar to the original no roof structure case. The second roof structure, on the other hand, despite being decayed, is improving the seismic behaviour of the building and is significantly influencing its deformation leading to a complete change of the deformed shape to shear. Ultimately the third roof structure is preserving the same behaviour of the building clearly presenting for all the considered scenarios flexural deformation.

This comparison shows the differences between the effect of the three roof structures, highlighting that their type and state of conservation is affecting the seismic behaviour of a historic masonry building.

Table 4-65 Deformed shape comparison of the building with the three roof structures with reduced cross-section timber elements

	Roof 1	Roof 2	Roof 3
S1.1	shear	shear	flexural
S1.2	shear	shear	flexural
S1.3	flexural	shear	flexural
S2.1	shear	shear	flexural
S2.2	flexural	shear	flexural
S2.3	shear	shear	flexural
S3.1	shear	shear	flexural
S3.2	flexural	shear	flexural
S3.3	shear	shear	flexural
S4.1	shear	shear	flexural
S4.2	flexural	shear	flexural
S4.3	flexural	shear	flexural
S5.1	shear	shear	flexural
S5.2	flexural	shear	flexural
S5.3	flexural	shear	flexural

4.3.3.4 Comparison

The deformation comparison of the two assessed cases, with complete and reduced cross-section (Table 4-66), show that:

1. For the first roof structure, the decrease of the cross-section of the timber elements is not influencing the deformation type of the historic masonry building, for almost all scenarios. Still like for the displacement and inter-story drift analysis, the sliding support scenarios present an exception, the decayed timber elements causing a change of the deformation from shear to flexural, for the S2.2 and S3.2 scenarios.
2. For the second roof structure, the deformation also remains the same for almost all scenarios, except for the S1.2 and S1.3 scenarios, where the reduction of the cross-section of the timber elements is causing a shear deformation of the building, compared to the initial flexural deformation. Still, the decay of the roof structure is improving the behaviour of the building, presenting only shear deformation for all the other scenarios.
3. The third roof structure is the only one where the reduction of the cross-section of the timber elements does not influence the deformation of the historic masonry building, presenting a flexural deformation for all the assessed scenarios in both cases.

Table 4-66 Deformed shape comparison - building with the three roof structures with complete and reduced cross-section timber elements

	1st roof structure		2nd roof structure		3rd roof structure	
	Complete cross-section	Reduced cross-section	Complete cross-section	Reduced cross-section	Complete cross-section	Reduced cross-section
S1.1	shear	shear	shear	shear	flexural	flexural
S1.2	shear	shear	flexural	shear	flexural	flexural
S1.3	flexural	flexural	flexural	shear	flexural	flexural
S2.1	shear	shear	shear	shear	flexural	flexural
S2.2	shear	flexural	shear	shear	flexural	flexural
S2.3	shear	shear	shear	shear	flexural	flexural
S3.1	shear	shear	shear	shear	flexural	flexural
S3.2	shear	flexural	shear	shear	flexural	flexural
S3.3	shear	shear	shear	shear	flexural	flexural
S4.1	shear	shear	shear	shear	flexural	flexural
S4.2	flexural	flexural	shear	shear	flexural	flexural
S4.3	flexural	flexural	shear	shear	flexural	flexural
S5.1	shear	shear	shear	shear	flexural	flexural
S5.2	flexural	flexural	shear	shear	flexural	flexural
S5.3	flexural	flexural	shear	shear	flexural	flexural

4.3.4 Damage level of the historic masonry building

Considering the ranges of the inter-story drift limit-states [59] (Table 4-67), according to the Eurocode 8, Part 3, the FaMIVE procedure (Failure Mechanism Identification and Vulnerability Evaluation), which is an analytical seismic vulnerability approach, based on on-site observations of the damage level of historic masonry buildings [177,178], and based on experimental tests performed on historic masonry structures, the damage level of the assessed structure was subsequently determined. Compared to the proposed inter-story drift values according to the FaMIVE procedure and the experimental tests, the Eurocode also considers the slenderness of the wall when evaluating its out-of-plane behaviour.

Table 4-67 Inter-story drift values for damage limit states

Limit state	Damage limitation	Significant damage	Near collapse	Collapse
		In-plane behaviour		
Eurocode 8 Part 3		0.40–0.60	0.53–0.80	
Experimental	0.18–0.23	0.65–0.90	1.23–1.92	2.10–2.80
FaMIVE	0.026-0.132	0.069-0.679	0.990-1.579	1.801-2.547
		Out-of-plane behaviour		
Eurocode 8 Part 3		0.8–1.2 (H_0/D)	1.06–1.60 (H_0/D)	
Experimental	0.33	0.88	2.30	4.80
FaMIVE	0.263-0.691	0.841-1.580	1.266-1.961	2.167-5.562
		Combined behaviour		
FaMIVE	0.030-0.168	0.181-0.582	0.724-1.401	1.114-3.307

4.3.4.1 Building with no roof structure

First, the damage level of the building without roof structure was analysed. Considering the in-plane behaviour of the structure, it was observed that the historic masonry walls would suffer significant damage according to the Eurocode at the top floor and at the second and top floor according to the FaMIVE limit state.

The out-of-plane behaviour shows, on the other hand, only limited damage. Ultimately, considering the combined behaviour, the structure without a roof is suffering significant damage only at the top according to the FaMIVE limit state (Table 4-68).

Considering these damage levels, it can be observed that the building is highly vulnerable at the top floor due to the significant recorded displacement and inter-story drift. On the second floor, the building is suffering significant damage only in-plane, according to the FaMIVE limit state.

Table 4-68 Damage state of the historic masonry building for all relevant floors without roof structure (D.l.–Damage limitation; S.d.–Significant damage)

	In-plane behaviour			Out-of-plane behaviour			Combined behaviour
Elevation	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	S.d.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.

4.3.4.2 Roof structures with complete cross-section timber elements

As already observed in the horizontal displacement and inter-story drift analysis, the 18th century roof structure is significantly improving the seismic behaviour of the historic masonry building which can also be observed in the damage level analysis of all the assessed scenarios (Table 4-69).

The in-plane behaviour shows that for rigid support scenarios, all the significant damages, identified in the case without roof structure, are disappearing on the top floor. Still, at the S3.1 and S5.1 scenarios, significant damage is still present on the second floor according to the FaMIVE limit state. All the other scenarios present significant damage at the top floor according to the FaMIVE limit state. Exceptions are the S1.2, S2.3 and S3.3 scenarios which present limited damage on all the floors according to all three limit states.

The out-of-plane damage level is limited for all the assessed scenarios due to the presence of the 18th century roof structure.

The combined behaviour, on the other hand, still presents significant damage on the top floor of the building, for the S4.2, S5.2, S4.3 and S5.3 scenarios. All the other scenarios present the disappearance of significant damage on the top floor. At the same time, no additional damage was observed in the walls below.

Table 4-69 Damage state of the historic masonry building for all relevant floors with the first roof structure with complete cross-section timber elements (D.l. – Damage limitation; S.d. – Significant damage)

Elevation	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour
	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE
	S1.1							S1.2							S1.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
	S2.1							S2.2							S2.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S3.1							S3.2							S3.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S4.1							S4.2							S4.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S5.1							S5.2							S5.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.

The presence of the second roof structure is mainly visible on the top floor of the building, significantly influencing its damage level (Table 4-70).

The in-plane behaviour shows that the damage level is still present only at the FaMIVE limit state but suffers particular variation according to the scenarios. Therefore, it was observed that rigid support scenarios present significant damage at the second and top floor except for the S1.1 scenario where the damage is only present at the top of the building and the S5.1 scenario where the significant damage was recorded at the second floor. Sliding support scenarios, on the other hand, present the significant damage at the top floor only for the S1.2 scenario while all the other scenarios are presenting significant damage on the second floor. Ultimately, the hinge sliding support present significant damage for the top floor for the rigid joint scenario (S1.3), while the other scenarios are presenting it on the second floor only.

The out-of-plane behaviour shows the complete lack of the significant damage.

Finally, the combined behaviour of the historic masonry building with the second roof structure, shows the disappearance of the significant damage from the top floor, except for the rigid joint scenarios (S1.1, S1.2 and S1.3) where the significant damage is still present at the top floor.

In this case, it can be clearly observed that the presence of the roof structure is reducing the damage level of the historic masonry building on the top floor and shifting the damage, for most of the scenarios, towards the second floor.

Table 4-70 Damage state of the historic masonry building for all relevant floors with the second roof structure with complete cross-section timber elements (D.l. – Damage limitation; S.d. – Significant damage)

Elevation	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour
	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE
	S1.1							S1.2							S1.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S2.1							S2.2							S2.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S3.1							S3.2							S3.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S4.1							S4.2							S4.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S5.1							S5.2							S5.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.

The presence of the third roof structure is presenting only little changes in the damage level of the historic masonry structure (Table 4-71).

The in-plane behaviour of this roof structure compared with the no roof structure case presents still specific differences. First, the significant damage initially recorded on the second floor of the building, according to the FaMIVE limit state, disappears for all the assessed scenarios and is being replaced by limited damage. At the same time, the significant damage from the top floor, according to EC 8 Part 3 limit state, is also replaced by limited damage for all the scenarios, except for the rigid joint ones (S1.1, S1.2 and S1.3). The out-of-plane behaviour, due to the reduction of the inter-story drift ratio at the top floor, also presents limited damage for all the scenarios. The combined behaviour remains in this case, as in the no roof structure case, with a significant damage level recorded on the top floor of the building for all the assessed scenarios. Since the presence of this roof structure is only slightly changing the behaviour of the historic masonry building, the recorded damage level is also approximately similar to the no roof case.

Table 4-71 Damage state of the historic masonry building for all relevant floors with the third roof structure with complete cross-section timber elements (D.l. – Damage limitation; S.d. – Significant damage)

Elevation	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour
	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE
	S1.1							S1.2							S1.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	S.d.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	S.d.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	S.d.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S2.1							S2.2							S2.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S3.1							S3.2							S3.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S4.1							S4.2							S4.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S5.1							S5.2							S5.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.

In order to better understand the effect of each roof structure on the damage level of the historic masonry building, the results obtained for the three cases were subsequently compared (Table 4-72).

In-plane it was observed that the first roof structure is significantly reducing the damage level of the historic masonry walls, presenting significantly more limited damage, on all the floors according to all limit states. The second roof structure, on the other hand, is presenting significant damage on both the 2nd and the top floor of the building, while the third roof structure is mainly displaying damage on its top floor.

Considering the out-of-plane behaviour, the recorded inter-story drifts are rather low and limited damage was recorded in all the cases.

The combined behaviour also presents a significant reduction in the damage level of the building. While the first two roof structures are clearly highlighting the importance of the presence of the roof structure on the seismic behaviour of the building, presenting mainly limited damage on the top floor, it was observed that the third roof structure is only having a limited effect, still presenting significant damage on the top floor for all the assessed scenarios.

Despite the recorded damages, all three roof structures are influencing the behaviour of the building and are reducing the damage of the historic masonry walls.

Table 4-72 Comparison of the damage state of the historic masonry building for all relevant floors without and with the three roof structures with complete cross-section timber elements

Limit state	No roof	Roof 1	Roof 2	Roof 3
		In-plane behaviour		
EC8 Part 3	Significant damage top floor	Damage limitation	Damage limitation	Significant damage top floor (S1.1, S1.2, S1.3)
Experimental	Damage limitation	Damage limitation	Damage limitation	Damage limitation
FaMIVE	Significant damage 2nd and top floor	Significant damage 2nd floor (S3.1, S5.1) top floor (S2.2, S3.2, S4.2, S5.2, S1.3, S4.3, S5.3)	Significant damage 2nd floor (S5.1 S2.2, S3.2, S4.2, S5.2. S2.3, S3.3, S4.3, S5.3) top floor (S1.1, S1.2, S1.3) 2nd and top floor (S2.1, S3.1, S4.1)	Significant damage top floor
		Out-of-plane behaviour		
EC8 Part 3	Damage limitation	Damage limitation	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation	Damage limitation	Damage limitation
FaMIVE	Damage limitation	Damage limitation	Damage limitation	Damage limitation
Combined behaviour	Significant damage top floor	Significant damage top floor (S4.2, S5.2. S4.3, S5.3)	Significant damage top floor (S1.1, S1.2, S1.3.)	Significant damage top floor

4.3.4.3 Roof structures with 20% reduced cross-section timber elements

When reducing the cross-section of the elements of the roof structure, the influence on the seismic behaviour of the historic masonry building with the first roof structure is rather peculiar (Table 4-73).

Table 4-73 Damage state of the historic masonry building for all relevant floors with the first roof structure with reduced cross-section timber elements (D.l. – Damage limitation; S.d. – Significant damage)

Elevation	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour
	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE
	S1.1							S1.2							S1.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
	S2.1							S2.2							S2.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
	S3.1							S3.2							S3.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
	S4.1							S4.2							S4.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S5.1							S5.2							S5.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.

The in-plane behaviour shows, for scenarios S1.1 and S1.2 only limited damage. For rigid support scenarios, the appearance of a significant damage level on the second floor of the building is visible, according to the FaMIVE limit state, while the top floor is only presenting damage limitation. The sliding support scenarios, on the other hand, present significant damage only on the top floor of the building, for all the scenarios at the FaMIVE limit state.

Ultimately, the hinged-sliding support presents two different distributions of the damage level. The S3.3 scenario is showing significant damage on the second and top floor of the building only for the FaMIVE limit state; while all the other scenarios (S1.3, S2.3, S4.3 and S5.3) are only presenting significant damage on the top floor for the FaMIVE limit state.

The out-of-plane behaviour still presents only limited damage for all the considered scenarios.

Finally, the combined behaviour shows for all rigid support scenarios limited damage on all the levels of the building. The same was observed for the S1.2 scenario, which also presents limited damage on all the floors of the building. All the other scenarios are showing significant damage on the top floor.

When reducing the cross-section of the timber elements, of the second roof structure, various changes of the damage level of the historic masonry building at the second and top floor were observed (Table 4-74).

Table 4-74 Damage state of the historic masonry building for all relevant floors with the second roof structure with reduced cross-section timber elements (D.l. – Damage limitation; S.d. – Significant damage)

Elevation	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour
	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE
	S1.1							S1.2							S1.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
	S2.1							S2.2							S2.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S3.1							S3.2							S3.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S4.1							S4.2							S4.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
	S5.1							S5.2							S5.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
10.1	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.

The in-plane behaviour shows that the significant damage remains present only according to the FAMIVE limit state, mainly on the second floor of the building. Still, scenarios involving rigid joints, present the appearance of additional significant damage also on the second floor of the building. Considering the out-of-plane behaviour, it was observed that the structure would suffer no damage on all the floors according to all three limit states.

Ultimately, the combined behaviour shows that the top floor of the building would not suffer significant damage anymore. Still, the damage is appearing on the second floor, for almost all the considered scenarios, except the the S1.1, S1.2, S1.3, S2.1, S3.1 and S3.2, where limited damage was observed.

The reduction of the cross-section of the timber element of the third roof structure has a rather peculiar influence on the damage level recorded on the historic masonry walls of the building (Table 4-75).

Table 4-75 Damage state of the historic masonry building for all relevant floors with the third roof structure with reduced cross-section timber elements (D.l. – Damage limitation; S.d. – Significant damage)

Elevation	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour	In-plane behaviour			Out-of-plane behaviour			Combined behaviour
	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE	EC8 Part 3	Experimental	FaMIVE	EC8 Part 3	Experimental	FaMIVE	FaMIVE
	S1.1							S1.2							S1.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S2.1							S2.2							S2.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S3.1							S3.2							S3.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S4.1							S4.2							S4.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.
	S5.1							S5.2							S5.3						
3.6	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.
7.3	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	D.l.
10.1	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.	D.l.	D.l.	S.d.	D.l.	D.l.	D.l.	S.d.

The in-plane behaviour analysis of the building presents a replacement of the significant damage level recorded on the top floor, for the no roof case, according to the EC8 Part 3, with limited damage for all the assessed scenarios. The FaMIVE limit state, on the other hand, still presents the same significant damage level on the second and top floor just like in the case of the building without roof structure for all the assessed scenarios. Despite the roof structure being decayed, the out-of-plane behaviour presents no damage on all the floors and all the assessed scenarios.

Finally, the combined behaviour remains as in the case of the historic masonry building without roof structure, presenting significant damage on the top floor.

Like in the complete cross-section cases, in order to better understand the effect of each decayed roof structure on the damage level of the historic masonry building, the results obtained for the 3 cases where compared (Table 4-76).

Table 4-76 Comparison of the damage state of the historic masonry building for all relevant floors without and with the three roof structures with reduced cross-section timber elements

Limit state	No roof	Roof 1	Roof 2	Roof 3
		In-plane behaviour		
EC8 Part 3	Significant damage top floor	Damage limitation	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation	Damage limitation	Damage limitation
FaMIVE	Significant damage 2nd and top floor	Significant damage 2nd floor (S2.1, S3.1, S4.1, S5.1) top floor (S2.2, S3.2, S4.2, S5.2 S1.3, S2.3, S4.3, S5.3) 2nd and top floor (S3.3)	Significant damage 2nd floor (S2.1, S3.1, S4.1, S5.1, S2.2, S3.2, S4.2, S5.2, S2.3, S3.3, S4.3, S5.3) 2nd and top floor (S1.1, S1.2, S1.3)	Significant damage 2nd and top floor
		Out-of-plane behaviour		
EC8 Part 3	Damage limitation	Damage limitation	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation	Damage limitation	Damage limitation
FaMIVE	Damage limitation	Damage limitation	Damage limitation	Damage limitation
Combined behaviour	Significant damage top floor	Significant damage top floor (S2.2, S3.2, S4.2, S5.2, S4.3, S5.3)	Significant damage 2nd floor (S4.1, S5.1, S2.2, S4.2, S5.2, S2.3, S3.3, S4.3, S5.3)	Significant damage top floor

In-plane, two different changes of the damage level of the historic masonry building were observed for all three roof structures. On the one hand, it was observed that according to the FaMIVE limit state, for some scenarios the damage is appearing on the 2nd floor of the building while the top floor is displaying limited damage. This can be observed mainly for the building with the first and second roof structure. On the other hand, it was observed that also for the FaMIVE limit state, all scenarios are presenting significant damage on both the 2nd and top floor of the building for the 3rd roof structure but also for some scenarios of the 1st and 2nd roof. Only a few scenarios still present significant damage only on the top floor.

Out-of-plane the presence of the roof structures is, is still presenting limited damage.

Ultimately the combined behaviour is also presenting two different distributions of the damage of the historic masonry structure. On the one hand, the first and third roof structures are still presenting significant damage on the top floor of the building while the other 2 floors below are only presenting limited damage. The seismic load applied to the building with the second roof structure, on the other hand, is causing significant damage only on the 2nd floor, the third one suffering only limited damage. Only a few scenarios, for the first roof structure and the second one, present no damage at all on all the floors.

4.3.4.4 Comparison

In order to fully understand the effect of the decay of the first roof structure on the seismic behaviour of the historic masonry building, the two cases were subsequently compared (Table 4-77):

1. Considering the in-plane behaviour, it was observed that:
    1.1. For rigid support scenarios, the second floor, is presenting significant damage according to the FaMIVE limit state for more scenarios in the decayed case;
    1.2. The sliding support scenarios present for the same scenarios significant damage at the top floor according to the EC8 Part 3 and Experimental limit state;
    1.3. The hinged-sliding support, on the other hand, presents the appearance of additional significant damage on the second floor of the S3.3 scenario, according to the FaMIVE limit state, in the reduced cross-section case and significant damage for all the other scenarios.
2. The out-of-plane damage level comparison shows that the decay of the roof structure is not changing the damage of the building;
3. The combined behaviour also presents more damage of the historic masonry building with the decayed roof structure, on the top floor of the building.

Table 4-77 Damage state of the historic masonry building for all relevant floors comparison - building with the first roof structure with complete and reduced cross-section timber elements

Limit state	Complete cross-section	Reduced cross-section
In-plane behaviour		
EC8 Part 3	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation
FaMIVE	Significant damage 2nd floor (S3.1, S5.1) top floor (S2.2, S3.2, S4.2, S5.2, S1.3, S4.3, S5.3)	Significant damage 2nd floor (S2.1, S3.1, S4.1, S5.1) top floor (S2.2, S3.2, S4.2, S5.2, S1.3, S2.3, S4.3, S5.3) 2nd and top floor (S3.3)
Out-of-plane behaviour		
EC8 Part 3	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation
FaMIVE	Damage limitation	Damage limitation
Combined behaviour	Significant damage top floor (S4.2, S5.2. S4.3, S5.3)	Significant damage top floor (S2.2, S3.2, S4.2, S5.2, S4.3, S5.3)

While comparing the damage level of the building with the second roof with a complete cross-section and 20% reduced cross-section, changes were also observed (Table 4-78). The comparison shows that the decay of the roof structure is shifting the damage of the historic masonry building mainly towards the second floor, limiting the damage on the top floor.

The in-plane behaviour for the reduced cross-section roof structure presents the disappearance of the significant damage from the top floor of the building according to the FaMIVE limit state, for the rigid support (S2.1, S3.1 and S4.1) scenarios. While reducing the cross-section of the timber elements, it was observed that the significant damage remains mainly present at the second floor, only for scenarios involving rigid joints the significant damage appearing at both the second and top floor.

The out-of-plane behaviour shows no difference between the two assessed cases, both displaying no damage for all the floors of the building for all scenarios.

The combined behaviour presents that for the reduced cross-section case, damage is appearing exclusively on the second floor of the building, At the same time, the significant damage initially placed at the top floor, for rigid joint scenarios is disappearing.

Table 4-78 Damage state of the historic masonry building for all relevant floors comparison - building with the second roof structure with complete and reduced cross-section timber elements

Limit state	Complete cross-section	Reduced cross-section
	In-plane behaviour	
EC8 Part 3	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation
FaMIVE	Significant damage 2nd floor (S5.1 S2.2, S3.2, S4.2, S5.2. S2.3, S3.3, S4.3, S5.3) top floor (S1.1, S1.2, S1.3) 2nd and top floor (S2.1, S3.1, S4.1)	Significant damage 2nd floor (S2.1, S3.1, S4.1, S5.1, S2.2, S3.2, S4.2, S5.2, S2.3, S3.3, S4.3, S5.3) 2nd and top floor (S1.1, S1.2, S1.3)
	Out-of-plane behaviour	
EC8 Part 3	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation
FaMIVE	Damage limitation	Damage limitation
Combined behaviour	Significant damage top floor (S1.1, S1.2, S1.3)	Significant damage 2nd floor (S4.1, S5.1, S2.2, S4.2, S5.2, S2.3, S3.3, S4.3, S5.3)

Ultimately, the damage level of the building with the third roof structure was compared, for the two cases. The decay of the timber elements has a completely different effect on the behaviour and damage of the building (Table 4-79).

Considering the in-plane behaviour, the appearance of significant damage on the second floor of the building according to the FaMIVE limit state was observed, for all the assessed scenarios. Still, the damage previously recorded on the top floor according to the FaMIVE limit state still remains present. In can be observed that the decay of this roof structure is no longer shifting the damage of the historic masonry towards the second floor, like in the case of the second roof structure, but is instead extending it over the top two floors. However, according to the EC8 Part 3 limit state, less damage of the historic masonry building is expected when the roof structure is decayed, showing limited damage on the top floor.

The out-of-plane behaviour shows no difference between the two assessed cases, both displaying no damage for all the floors of the building for all scenarios.

Ultimately, no difference was also observed for the combined behaviour of the structure, both cases presenting significant damage of the historic masonry building on the top floor for all considered scenarios.

The reduction of the damage level of the building, in this case, is mainly caused by the behaviour of the decayed roof structure. If reducing the cross-section of the timber elements, the roof structure is presenting significantly more deformation, and it no longer helps transmit the seismic forces from one side of the building to the other. Therefore, the horizontal displacement of the historic masonry walls and thus, the inter-story drift are lower on the superior part of the building, causing, less damage to the historic masonry walls [167].

Table 4-79 Damage state of the historic masonry building for all relevant floors comparison - building with the third roof structure with complete and reduced cross-section timber elements

Limit state	Complete cross-section	Reduced cross-section
	In-plane behaviour	
EC8 Part 3	Significant damage top floor (S1.1, S1.2, S1.3)	Damage limitation
Experimental	Damage limitation	Damage limitation
FaMIVE	Significant damage top floor	Significant damage 2nd and top floor
	Out-of-plane behaviour	
EC8 Part 3	Damage limitation	Damage limitation
Experimental	Damage limitation	Damage limitation
FaMIVE	Damage limitation	Damage limitation
Combined behaviour	Significant damage top floor	Significant damage top floor

4.3.5 Internal forces

After these parameters were analysed, the focus was shifted towards the analysis of the internal forces recorded on the historic masonry walls. In order to understand the behaviour of the historic building under seismic loads the vertical axial forces, out-of-plane shear forces and out-of-plane bending moments were analysed (Fig. 4.26) an the bottom and top of each floor for each model (Fig. 4.27).

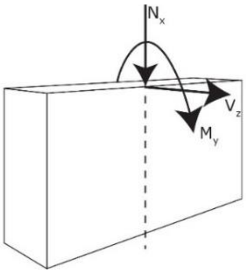


Fig. 4.26 Assessed internal forces on the historic masonry wall

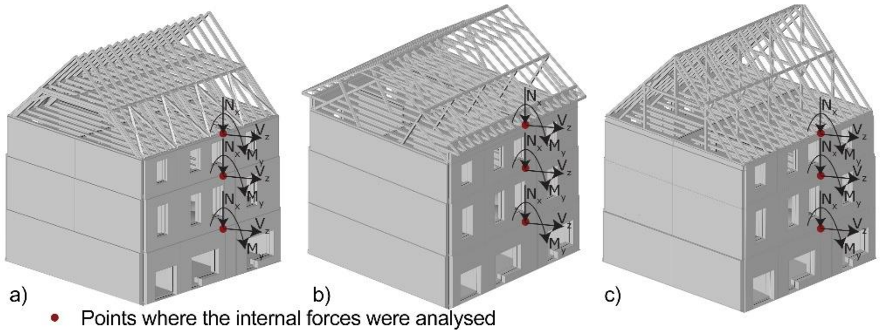


Fig. 4.27 The finite element models with marked points where the internal forces where analysed

4.3.5.1 Building with no roof structure

In order to determine a baseline for future comparisons with the building with the three roof structures, first, the internal forces for the building with no roof structure were assessed (Table 4-80).

Concerning the vertical axial forces (N_x), a decrease of the values was observed starting with the base of the historic masonry building towards its top. Despite the seismic lateral forces, all along the walls, clear compressive axial forces were observed. The most significant decrease of the axial forces was observed on the 1st floor above the cross-vault.

The out-of-plane shear forces (V_z), perpendicular to the historic masonry walls, also present high values on the base of the structure, decreasing towards the top of the building. In the area of the cross-vault, the shear forces are again presenting a peculiar behaviour of the structure, values in the opposite direction.

Ultimately, the recorded bending moments (M_y), which can lead to an out-of-plane failure of the historic masonry walls, show the highest value in the area of the first timber beam floor. This corresponds with the area where the applied lateral forces are beginning to influence the out-of-plane behaviour of the building, and the out-of-plane inter-story drift was presenting the most significant rise.

Table 4-80 Internal forces on the historic masonry walls of the building with no roof structure

Elevation [m]	N_x [kN]	V_z [kN]	M_y [kNm]
0	-105.31	3.40	2.99
3.45	-57.63	3.40	4.68
3.75	-56.70	-1.47	3.22
7.2	-24.74	2.11	5.97
7.4	-23.20	1.69	3.66
10.05	-1.35	1.69	2.53

4.3.5.2 Roof structures with complete cross-section timber elements

4.3.5.2.1 Vertical axial forces (N_x)

Subsequently, the three roof structures were placed on the historic masonry building, and the internal forces were assessed by comparing them do the no roof structure case.

First, the recorded vertical axial forces on the masonry walls were analysed. For the first roof structure, it was observed that considering the assessed scenarios, the chosen support typology is having a more considerable influence on the changes of the axial force than the different joint axial stiffness calculation methods. Therefore, the differences between the obtained values, range up to 10% for every floor of the building when comparing the scenarios with the same support type. Still, scenarios involving rigid joints, present up to 20% higher values than the other joint typologies, on all the floors of the building (Table 4-81, Fig. 4.28).

At the same time, for all the assessed scenarios, higher compressive axial forces were identified at the base of the structure while at the top, most of them are presenting tensile axial forces. An exception was identified for the S1.2 scenario, which is still presenting compressive force, with values close to the no roof structure case. Despite this, the obtained tensile axial forces for the scenarios involving sliding and hinged-sliding supports are significantly lower than the ones obtained for the rigid support scenarios.

The distribution of tensile and compressive axial forces on the historic masonry wall is consistent with the results obtained for the displacement and out-of-plane inter-story drift analysis, the scenarios involving rigid joints presenting a better out-of-plane behaviour than the scenarios with the other two support types.

Table 4-81 Vertical axial forces analysis of the scenarios - building with the first roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With first roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	-105.31	-118.19	-109.21	-104.78	-108.32	-106.21
3.45	-57.63	-70.52	-61.54	-57.1	-60.65	-58.53
3.75	-56.70	-68.35	-60.31	-56.79	-59.56	-58.15
7.2	-24.74	-32.02	-26.97	-24.92	-26.47	-26.73
7.4	-23.20	-8.86	-1.35	-1.57	0.52	-3.09
10.05	-1.35	12.99	20.5	20.28	22.37	18.76
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	-105.31	-104.81	-104.06	-103.85	-105.28	-106.19
3.45	-57.63	-57.13	-56.38	-56.18	-57.61	-58.51
3.75	-56.7	-58.00	-57.34	-57.03	-58.09	-58.43
7.2	-24.74	-29.30	-28.67	-28.06	-28.09	-27.45
7.4	-23.2	-22.75	-21.65	-19.60	-14.45	-11.26
10.05	-1.35	-0.90	0.20	2.25	7.40	10.59
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	-105.31	-114.95	-113.7	-111.26	-122.84	-117.39
3.45	-57.63	-67.27	-66.02	-63.58	-75.16	-69.72
3.75	-56.7	-65.17	-63.33	-61.49	-71.92	-65.60
7.2	-24.74	-31.85	-29.72	-29.03	-35.11	-29.53
7.4	-23.2	-19.64	-19.26	-19.39	-19.01	-14.51
10.05	-1.35	2.21	2.59	2.47	2.85	7.34

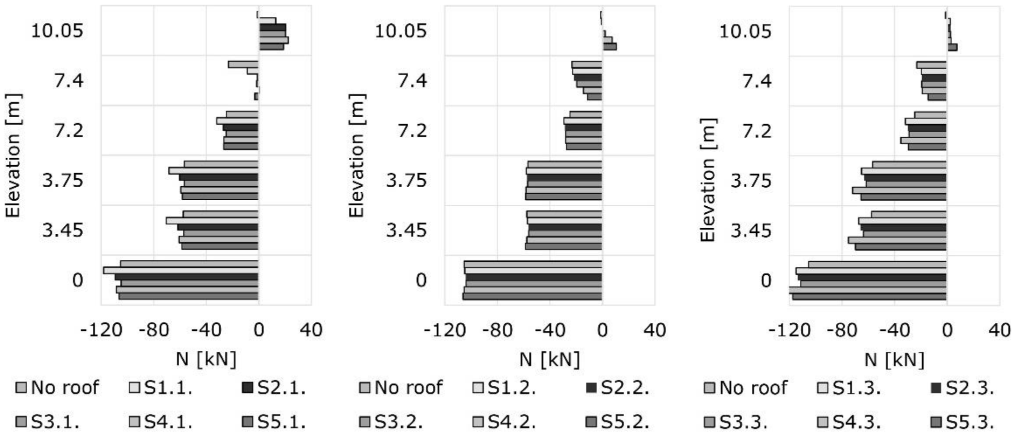


Fig. 4.28 Vertical axial forces diagrams analysis of the scenarios - building with the first roof structure with complete cross-section timber elements (comparison to the no roof case)

The presence of the second roof structure is showing a similar distribution of the axial forces as in the no roof structure case, presenting only compressive axial forces on all the floors of the historic masonry building (Table 4-82, Fig. 4.29). Due to the presence of the roof structure, at the top of the building, the obtained compressive axial forces are significantly higher compared to the no roof structure case, presenting ten times higher values. In this case, no significant differences could be observed between the assessed scenarios, the values presenting up to 5% variation depending on the chosen support and joint axial stiffness. The influence of the support type is no longer highly visible as in the first roof structure case. Still, the scenarios involving rigid joints, present once again an exception, presenting lower values than the other considered scenarios.

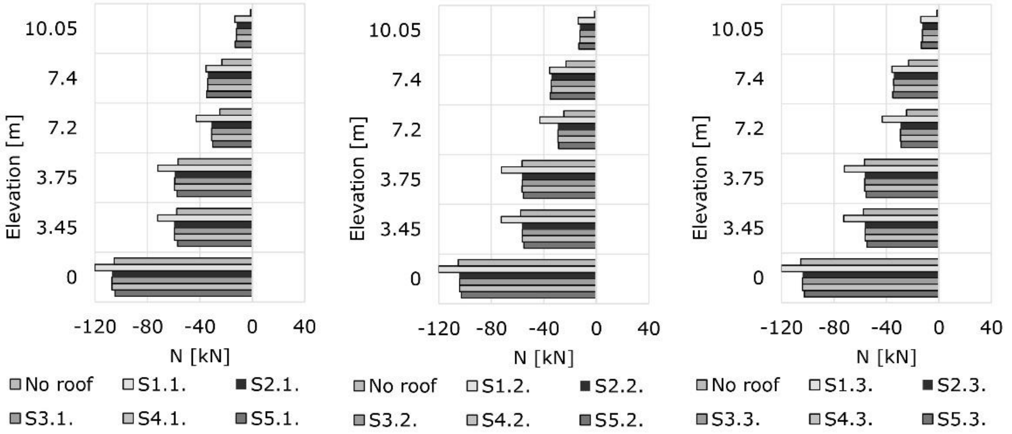


Fig. 4.29 Vertical axial forces diagrams analysis of the scenarios - building with the second roof structure with complete cross-section timber elements

Table 4-82 Vertical axial forces analysis of the scenarios - building with the second roof structure with complete cross-section timber elements (comparison to the no roof case)

Elevation [m]	Without roof	With second roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	-105.31	-119.96	-107.17	-106.97	-107.08	-104.71
3.45	-57.63	-72.29	-59.49	-59.29	-59.40	-57.03
3.75	-56.7	-71.93	-59.39	-59.22	-59.31	-57.36
7.2	-24.74	-42.79	-31.18	-31.08	-31.13	-30.30
7.4	-23.2	-35.28	-33.95	-34	-33.97	-34.86
10.05	-1.35	-13.42	-12.09	-12.15	-12.12	-13.01
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	-105.31	-120.27	-104.28	-104.10	-104.23	-102.78
3.45	-57.63	-72.59	-56.60	-56.43	-56.55	-55.11
3.75	-56.7	-72.25	-56.74	-56.57	-56.69	-55.54
7.2	-24.74	-43.24	-29.29	-29.17	-29.26	-28.89
7.4	-23.2	-35.63	-34.40	-34.45	-34.42	-35.18
10.05	-1.35	-13.78	-12.55	-12.60	-12.57	-13.33
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	-105.31	-120.24	-104.12	-103.94	-104.07	-102.60
3.45	-57.63	-72.57	-56.44	-56.26	-56.39	-54.92
3.75	-56.7	-72.21	-56.65	-56.49	-56.60	-55.45
7.2	-24.74	-43.29	-29.31	-29.2	-29.28	-28.92
7.4	-23.2	-35.8	-34.41	-34.47	-34.44	-35.21
10.05	-1.35	-13.94	-12.56	-12.61	-12.58	-13.36

Ultimately, for the third roof structure, it is once again highlighted that the chosen support type and joint axial stiffness are having limited influence on the changes of the axial force along the exterior historic masonry wall. Therefore, the differences between the obtained values, range up to 10% for every floor of the building when comparing the scenarios with the same support type (Table 4-83, Fig. 4.30).

At the same time, for all the assessed scenarios, higher compressive axial forces were identified at the base of the structure while at the top, most of them are presenting tensile axial forces. Still, the scenarios involving rigid joints, are presenting 10% higher compressive axial forces on these floors.

While rigid joint scenarios are presenting compressive axial forces until the base of the top floor and low tensile axial forces only at the top of the building, all other scenarios are presenting tensile axial forces starting with the base of the top floor. Despite this, they are up to ten times higher than the ones obtained for the rigid joint scenarios.

The distribution of tensile and compressive axial forces on the historic masonry wall is once again consistent with the results obtained for the displacement and inter-story drift analysis, the scenarios involving rigid joints presenting a better out-of-plane behaviour than the scenarios with the other two support types.

Table 4-83 Vertical axial forces analysis of the scenarios - building with the third roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With third roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	-105.31	-115.58	-111.69	-111.86	-112.15	-109.88
3.45	-57.63	-67.91	-64.02	-64.18	-64.47	-62.21
3.75	-56.70	-65.11	-60.84	-60.61	-61.01	-59.14
7.2	-24.74	-28.16	-26.85	-25.39	-26.73	-24.85
7.4	-23.20	-19.37	2.4	3.23	0.49	-0.04
10.05	-1.35	2.48	24.25	25.08	22.34	21.82
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	-105.31	-115.84	-112.66	-112.87	-112.73	-110.92
3.45	-57.63	-68.16	-64.99	-65.19	-65.05	-63.24
3.75	-56.7	-63.46	-61.14	-61.51	-61.32	-59.98
7.2	-24.74	-27.41	-25.53	-26.07	-25.81	-25.32
7.4	-23.2	-16.40	7.80	3.91	5.73	1.88
10.05	-1.35	5.45	29.66	25.76	27.58	23.73
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	-105.31	-115.07	-112.90	-113.00	-112.91	-111.18
3.45	-57.63	-67.39	-65.22	-65.33	-65.23	-63.51
3.75	-56.7	-63.37	-61.38	-61.60	-61.54	-60.17
7.2	-24.74	-28.28	-25.73	-26.14	-26.18	-25.45
7.4	-23.2	-20.85	8.33	4.17	5.56	2.20
10.05	-1.35	1.01	30.18	26.02	27.41	24.05

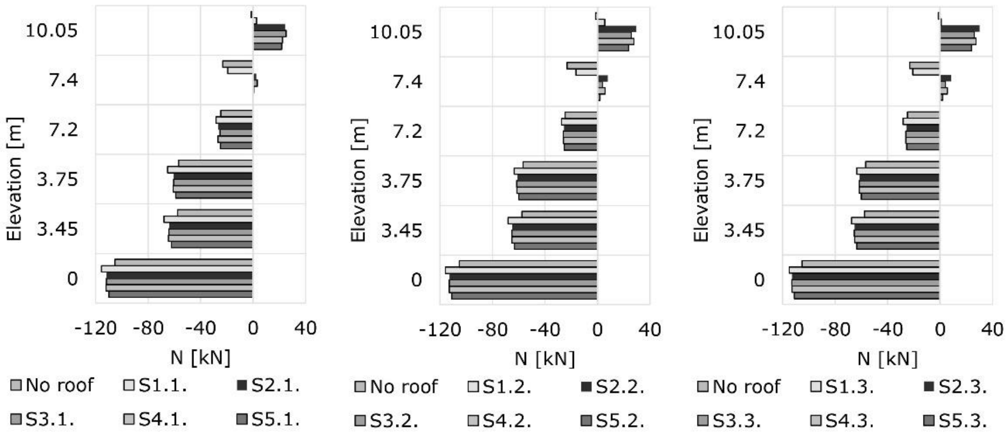


Fig. 4.30 Vertical axial forces diagrams analysis of the scenarios - building with the third roof structure with complete cross-section timber elements (comparison to the no roof structure case)

While comparing the axial forces on the historic masonry building without and with the three chosen roof structures, the following was observed (Table 4-84):

1. The axial forces on the wall for the building with all the three roof structures are approximately similar on the first and second floor of the building, and the differences start to be observed above this threshold
2. On the top floor:
    2.1. The first roof structure is causing tensile axial forces at the top of the wall;
    2.2. The second roof structure is presenting only compressive axial forces both at the bottom and top of the wall
    2.3. The third roof structure is causing tensile axial forces at the base and top of the wall; exceptions are the rigid joint scenarios which are presenting tensile axial forces only on the top of the building
3. Rigid support scenarios are presenting a different distribution of the axial forces compared to the other scenario (for all the roof structures):
    3.1. Higher compressive axial forces at the base of the structure
    3.2. Lower compressive axial forces on the top
4. Only little differences (under 10%) were observed while comparing the results obtained for the same support type and different joint axial stiffnesses.

Table 4-84 Comparison of the vertical axial forces on the historic masonry walls of the building without and with the three roof structures with complete cross-section timber elements

		Roof structures				Roof structures				Roof structures		
		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3
0	S1.1	-118.19	-119.96	-115.58	S1.2	-104.81	-120.27	-115.84	S1.3	-114.95	-120.24	-115.07
3.45		-70.52	-72.29	-67.91		-57.13	-72.59	-68.16		-67.27	-72.57	-67.39
3.75		-68.35	-71.93	-65.11		-58.00	-72.25	-63.46		-65.17	-72.21	-63.37
7.2		-32.02	-42.79	-28.16		-29.30	-43.24	-27.41		-31.85	-43.29	-28.28
7.4		-8.86	-35.28	-19.37		-22.75	-35.63	-16.40		-19.64	-35.80	-20.85
10.05		12.99	-13.42	2.48		-0.90	-13.78	5.45		2.21	-13.94	1.01
0	S2.1	-109.21	-107.17	-111.69	S2.3	-104.06	-104.28	-112.66	S2.3	-113.70	-104.12	-112.90
3.45		-61.54	-59.49	-64.02		-56.38	-56.60	-64.99		-66.02	-56.44	-65.22
3.75		-60.31	-59.39	-60.84		-57.34	-56.74	-61.14		-63.33	-56.65	-61.38
7.2		-26.97	-31.18	-26.85		-28.67	-29.29	-25.53		-29.72	-29.31	-25.73
7.4		-1.35	-33.95	2.40		-21.65	-34.40	7.80		-19.26	-34.41	8.33
10.05		20.50	-12.09	24.25		0.20	-12.55	29.66		2.59	-12.56	30.18
0	S3.1	-104.78	-106.97	-111.86	S3.2	-103.85	-104.10	-112.87	S3.3	-111.26	-103.94	-113.00
3.45		-57.10	-59.29	-64.18		-56.18	-56.43	-65.19		-63.58	-56.26	-65.33
3.75		-56.79	-59.22	-60.61		-57.03	-56.57	-61.51		-61.49	-56.49	-61.60
7.2		-24.92	-31.08	-25.39		-28.06	-29.17	-26.07		-29.03	-29.20	-26.14
7.4		-1.57	-34.00	3.23		-19.60	-34.45	3.91		-19.39	-34.47	4.17
10.05		20.28	-12.15	25.08		2.25	-12.60	25.76		2.47	-12.61	26.02
0	S4.1	-108.32	-107.08	-112.15	S4.2	-105.28	-104.23	-112.73	S4.3	-122.84	-104.07	-112.91
3.45		-60.65	-59.40	-64.47		-57.61	-56.55	-65.05		-75.16	-56.39	-65.23
3.75		-59.56	-59.31	-61.01		-58.09	-56.69	-61.32		-71.92	-56.60	-61.54
7.2		-26.47	-31.13	-26.73		-28.09	-29.26	-25.81		-35.11	-29.28	-26.18
7.4		0.52	-33.97	0.49		-14.45	-34.42	5.73		-19.01	-34.44	5.56
10.05		22.37	-12.12	22.34		7.40	-12.57	27.58		2.85	-12.58	27.41
0	S5.1	-106.21	-104.71	-109.88	S5.2	-106.19	-102.78	-110.92	S5.3	-117.39	-102.60	-111.18
3.45		-58.53	-57.03	-62.21		-58.51	-55.11	-63.24		-69.72	-54.92	-63.51
3.75		-58.15	-57.36	-59.14		-58.43	-55.54	-59.98		-65.60	-55.45	-60.17
7.2		-26.73	-30.30	-24.85		-27.45	-28.89	-25.32		-29.53	-28.92	-25.45
7.4		-3.09	-34.86	-0.04		-11.26	-35.18	1.88		-14.51	-35.21	2.20
10.05		18.76	-13.01	21.82		10.59	-13.33	23.73		7.34	-13.36	24.05

4.3.5.2.2 Out-of-plane shear forces (V_z)

Subsequently, the shear forces were analysed while considering the influence of the three chosen roof structures. The effect of the presence of the first roof structure is mostly visible, above the top of the second floor (Table 4-85, Fig. 4.31). Up until that point, the values only vary with up to 5% for the rigid support and sliding support scenarios and up to 10% for the hinged-sliding support scenarios which are presenting lower values at the base of the structure.

The shear forces of the models with the first roof structure are presenting, starting with the top of the second floor, an increase, which is getting higher, up until the top of the building. Therefore, at the top of the second floor, the most significant increase was recorded for the sliding support scenarios, of up to 40% for the S1.2 case. The other scenarios present an increase of about 15-20%. On the top floor, on the other hand, the rigid support scenarios are presenting the most significant increase of the shear forces, presenting forces twice as high as recorded in the no roof structure case. Sliding support scenarios present an up to 45% increase of the values and hinged-sliding support scenarios up to 25%.

When comparing the variation of the recorded shear forces, only around 10 up to 15 % differences were observed. Higher differences were recorded only for the S4.3 scenario, which is only presenting a 5% increase of the shear forces at the top of the building compared to the no roof case.

Table 4-85 Out-of-plane shear forces analysis of the scenarios - building with the first roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With first roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	3.40	3.07	3.38	3.45	3.41	3.38
3.45	3.40	3.07	3.38	3.45	3.41	3.38
3.75	-1.47	-1.70	-1.44	-1.40	-1.42	-1.38
7.2	2.11	2.17	2.52	2.54	2.54	2.58
7.4	1.69	3.54	3.64	3.66	3.44	3.31
10.05	1.69	3.54	3.64	3.66	3.44	3.31
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	3.4	3.33	3.35	3.33	3.25	3.20
3.45	3.4	3.33	3.35	3.33	3.25	3.20
3.75	-1.47	-1.36	-1.35	-1.38	-1.43	-1.49
7.2	2.11	2.91	2.69	2.64	2.54	2.51
7.4	1.69	2.48	2.40	2.32	2.26	2.21
10.05	1.69	2.48	2.40	2.32	2.26	2.21
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	3.4	3.08	3.03	3.06	2.77	2.91
3.45	3.4	3.08	3.03	3.06	2.77	2.91
3.75	-1.47	-1.57	-1.49	-1.48	-1.87	-1.57
7.2	2.11	2.26	2.4	2.51	1.84	2.19
7.4	1.69	2.02	2.09	2.10	1.75	1.93
10.05	1.69	2.02	2.09	2.10	1.75	1.93

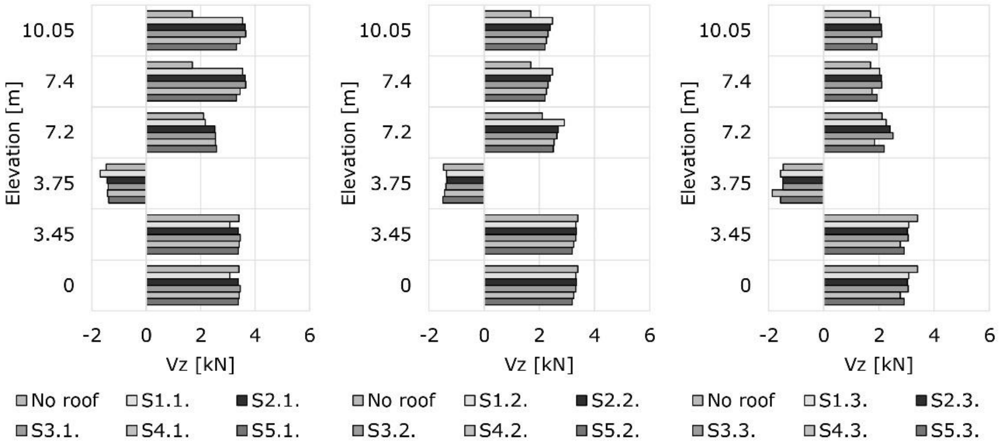


Fig. 4.31 Out-of-plane shear forces diagrams analysis of the scenarios - building with the first roof structure with complete cross-section timber elements (comparison to the no roof case)

Like in the previous case, the effect of the presence of the second roof structure is visible on the top of the second floor and the top floor (Table 4-86, Fig. 4.32).

On the first floor and the base of the second floor, the values vary with up to 5%. The scenarios involving rigid joints (S1.1, S1.2 and S1.3) present a decrease of the shear forces of up to 25%. These are also the only scenarios which are presenting a higher shear force in the area of the cross-vault.

The recorded shear forces are presenting, starting with the top of the second floor, a significant increase compared to the no roof structure case. Therefore, it was observed that at the top of the second floor, all the scenarios are presenting an increase of the shear forces of 50%. On the top floor, two different changes were observed. On the one hand, almost all the scenarios are presenting a four times increase of the shear forces, both at the bottom and top part of the wall. On the other hand, scenarios which are considering rigid joints are presenting a decrease of the shear forces of 20% for the S1.1 scenario and 50% for the S1.2 and S1.3 scenarios.

When comparing the variation of the recorded shear forces, excluding the rigid support scenarios, less than 5% differences were observed.

Table 4-86 Out-of-plane shear forces analysis of the scenarios - building with the second roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With second roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	3.4	3.25	3.36	3.36	3.36	3.39
3.45	3.4	3.25	3.36	3.36	3.36	3.39
3.75	-1.47	-1.61	-1.43	-1.43	-1.43	-1.36
7.2	2.11	3.31	3.13	3.14	3.13	3.27
7.4	1.69	1.37	5.21	5.17	5.21	5.23
10.05	1.69	1.37	5.21	5.17	5.21	5.23
	No roof	S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	3.4	3.24	3.37	3.37	3.37	3.39
3.45	3.4	3.24	3.37	3.37	3.37	3.39
3.75	-1.47	-1.63	-1.38	-1.37	-1.38	-1.34
7.2	2.11	3.34	3.37	3.39	3.37	3.45
7.4	1.69	0.86	4.17	4.08	4.17	4.22
10.05	1.69	0.86	4.17	4.08	4.17	4.22
	No roof	S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	3.4	3.25	3.38	3.38	3.38	3.40
3.45	3.4	3.25	3.38	3.38	3.38	3.40
3.75	-1.47	-1.61	-1.38	-1.37	-1.37	-1.33
7.2	2.11	3.36	3.37	3.39	3.38	3.45
7.4	1.69	0.86	4.17	4.09	4.17	4.22
10.05	1.69	0.86	4.17	4.09	4.17	4.22

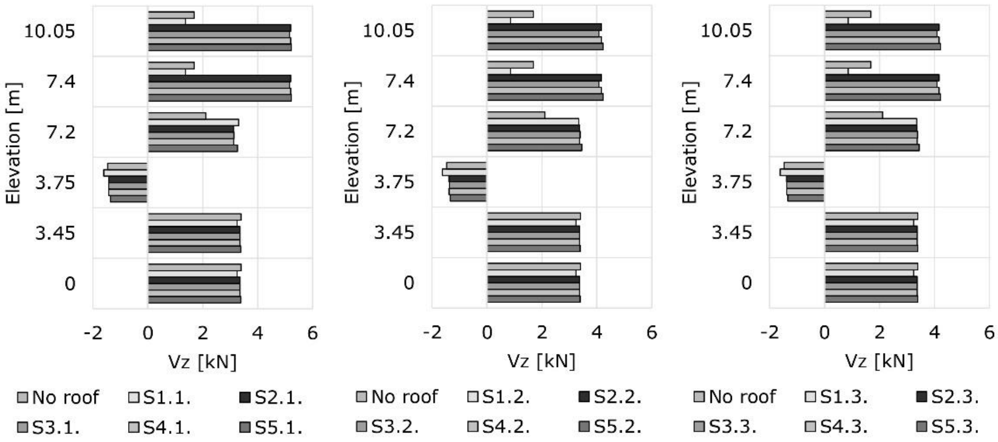


Fig. 4.32 Out-of-plane shear forces diagrams analysis of the scenarios - building with the second roof structure with complete cross-section timber elements (comparison to the no roof structure case)

On the contrary to the previously analysed models with the first and second roof structure, the presence of the third is also influencing the recorded shear forces on the first floor of the building (Table 4-87, Fig. 4.33).

Table 4-87 Out-of-plane shear forces analysis of the scenarios - building with the third roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With third roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	3.40	2.77	3.00	3.19	2.85	3.13
3.45	3.40	2.77	3.00	3.19	2.85	3.13
3.75	-1.47	-1.75	-1.61	-1.56	-1.71	-1.53
7.2	2.11	2.31	3.22	3.26	3.19	3.48
7.4	1.69	2.34	2.54	2.40	2.51	2.47
10.05	1.69	2.34	2.54	2.40	2.51	2.47
	No roof	S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	3.40	2.83	3.15	3.16	3.20	3.11
3.45	3.40	2.83	3.15	3.16	3.20	3.11
3.75	-1.47	-1.75	-1.61	-1.58	-1.57	-1.56
7.2	2.11	2.55	3.26	3.26	3.24	3.46
7.4	1.69	2.36	2.35	2.22	2.32	2.27
10.05	1.69	2.36	2.35	2.22	2.32	2.27
	No roof	S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	3.40	2.81	3.19	3.17	3.21	3.12
3.45	3.40	2.81	3.19	3.17	3.21	3.12
3.75	-1.47	-1.87	-1.58	-1.57	-1.55	-1.55
7.2	2.11	2.40	3.26	3.25	3.24	3.45
7.4	1.69	2.23	2.37	2.23	2.34	2.28
10.05	1.69	2.23	2.37	2.23	2.34	2.28

Therefore, it was observed that even at the base of the building, the shear forces are 5 up to 10% lower due to the presence of the roof structure. The differences get even higher at the top of the first floor where they are 15% lower, up to 20% in the case of the rigid support scenarios. Finally, in the area of the cross-vault, all the scenarios present a decrease of the shear forces with up to 10% except for the rigid support scenarios which present an up to 30% decrease.

The recorded shear forces are presenting, starting with the top of the second floor, an increase compared to the no roof structure case, of up to 50%, for all the considered scenarios. Exceptions were recorded for the rigid joint scenarios, which present an increase of the shear forces of 10% for the S1.1, 20% for the S1.2 and 15% for the S1.3 scenario.

On the top floor, the obtained values vary only with 10%. On this floor, the rigid support scenarios are no longer causing a peculiar change of the shear forces. Both at the bottom of this floor and its top an increase of 30 up to 40% was observed. Only the S2.1 and S4.1 scenarios are presenting an increase of 50%.

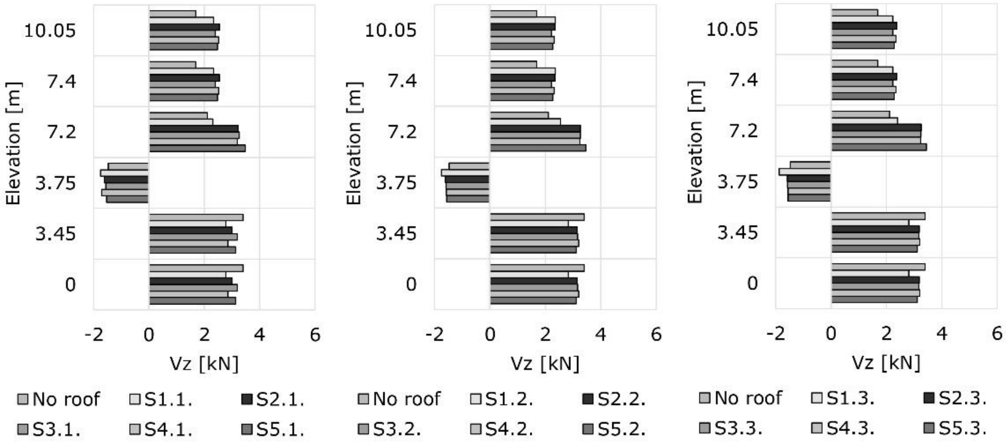


Fig. 4.33 Out-of-plane shear forces diagrams analysis of the scenarios - building with the third roof structure with complete cross-section timber elements (comparison to the no roof case)

While comparing the shear forces on the historic masonry building without and with the three chosen roof structures, the following was observed (Table 4-88):

1. Even with the roof structures, the building is still presenting shear forces in the opposite direction in the area of the cross-vault, compared to all the other floors;
2. The shear forces on the first floor and the base of the second floor of the building when comparing the chosen joint types:
   - 2.1. Are approximately similar for the first and second roof structure;
   - 2.2. Present variation of up to 30% in the case of the third roof structure.
3. On the top floor:
   - 3.1. The first roof structure is causing a two-times increase of the shear forces for rigid support scenarios and up to 45% for the other scenarios;
   - 3.2. The second roof structure is causing a 20% decrease of the shear forces for rigid support scenarios and a four times increase for all the other scenarios;
   - 3.3. The third roof structure is causing an increase of 30 up to 40% of the shear forces.

Table 4-88 Comparison of the out-of-plane shear forces on the historic masonry walls of the building without and with the three roof structures with complete cross-section timber elements

		Roof structures				Roof structures				Roof structures		
		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3
0	S1.1	3.07	3.25	2.77	S1.2	3.33	3.24	2.83	S1.3	3.08	3.25	2.81
3.45		3.07	3.25	2.77		3.33	3.24	2.83		3.08	3.25	2.81
3.75		-1.70	-1.61	-1.75		-1.36	-1.63	-1.75		-1.57	-1.61	-1.87
7.2		2.17	3.31	2.31		2.91	3.34	2.55		2.26	3.36	2.40
7.4		3.54	1.37	2.34		2.48	0.86	2.36		2.02	0.86	2.23
10.05		3.54	1.37	2.34		2.48	0.86	2.36		2.02	0.86	2.23
0	S2.1	3.38	3.36	3.00	S2.3	3.35	3.37	3.15	S2.3	3.03	3.38	3.19
3.45		3.38	3.36	3.00		3.35	3.37	3.15		3.03	3.38	3.19
3.75		-1.44	-1.43	-1.61		-1.35	-1.38	-1.61		-1.49	-1.38	-1.58
7.2		2.52	3.13	3.22		2.69	3.37	3.26		2.40	3.37	3.26
7.4		3.64	5.21	2.54		2.40	4.17	2.35		2.09	4.17	2.37
10.05		3.64	5.21	2.54		2.40	4.17	2.35		2.09	4.17	2.37
0	S3.1	3.45	3.36	3.19	S3.2	3.33	3.37	3.16	S3.3	3.06	3.38	3.17
3.45		3.45	3.36	3.19		3.33	3.37	3.16		3.06	3.38	3.17
3.75		-1.40	-1.43	-1.56		-1.38	-1.37	-1.58		-1.48	-1.37	-1.57
7.2		2.54	3.14	3.26		2.64	3.39	3.26		2.51	3.39	3.25
7.4		3.66	5.17	2.40		2.32	4.08	2.22		2.10	4.09	2.23
10.05		3.66	5.17	2.40		2.32	4.08	2.22		2.10	4.09	2.23
0	S4.1	3.41	3.36	2.85	S4.2	3.25	3.37	3.20	S4.3	2.77	3.38	3.21
3.45		3.41	3.36	2.85		3.25	3.37	3.20		2.77	3.38	3.21
3.75		-1.42	-1.43	-1.71		-1.43	-1.38	-1.57		-1.87	-1.37	-1.55
7.2		2.54	3.13	3.19		2.54	3.37	3.24		1.84	3.38	3.24
7.4		3.44	5.21	2.51		2.26	4.17	2.32		1.75	4.17	2.34
10.05		3.44	5.21	2.51		2.26	4.17	2.32		1.75	4.17	2.34
0	S5.1	3.38	3.39	3.13	S5.2	3.20	3.39	3.11	S5.3	2.91	3.40	3.12
3.45		3.38	3.39	3.13		3.20	3.39	3.11		2.91	3.40	3.12
3.75		-1.38	-1.36	-1.53		-1.49	-1.34	-1.56		-1.57	-1.33	-1.55
7.2		2.58	3.27	3.48		2.51	3.45	3.46		2.19	3.45	3.45
7.4		3.31	5.23	2.47		2.21	4.22	2.27		1.93	4.22	2.28
10.05		3.31	5.23	2.47		2.21	4.22	2.27		1.93	4.22	2.28

4.3.5.2.3 Out-of-plane bending moments (M_y)

Finally, the out-of-plane bending moments were analysed for the building with the three roof structures and subsequently compared to the no roof case.

For the first roof structure, a variation of the obtained values was observed for all the considered scenarios (Table 4-89, Fig. 4.34). On the contrary to the axial forces and sheer forces previously analysed, the bending moments are presenting up to 40% differences between the scenarios even from the base of the building. At the same time, no clear pattern was observed when comparing the obtained results with the no roof structure scenario.

Therefore, at the base of the first floor of the building, S1.1, S2.1, S5.2 and all the scenarios involving hinged and sliding supports are presenting a decrease of the bending moment, varying from 5 up to 60% (recorded for the S4.3 scenario), while all the other scenarios are presenting an increase of up to 10%. On the top of the same floor, on the other hand, the behaviour changes. All the scenarios are presenting a decrease of the bending forces of 15 up to 20%. At this point, the variations are no longer as high as they were at the base of the wall.

The second floor is presenting both at the bottom and the top a decrease of the bending moment, but with high variations between the considered scenarios. It was therefore observed that at the base of the 2nd floor scenarios involving rigid supports, except for the S1.1 scenario are presenting only a slight decrease of up to 5% of the bending forces while the other scenarios are presenting decreases ranging from 10 up to 25%. At the top of the floor, the scenarios involving rigid support present a decrease of the bending moment of 15 up to 20%, while the other scenarios are presenting higher variations, from 5%, recorded for the S1.2 scenario, to 40%, recorded for the S4.3 scenario.

The most peculiar behaviour was recorded at the base of the top floor, where the bending moments are close to 0kN, presenting a significant decrease compared to the no roof structure case. Despite being still close to 0, scenarios S3.1, S2.3, S3.3, S4.3 and S5.3 are presenting negative bending moment values.

Ultimately, at the top of the building, two different behaviours were observed. On the one hand, scenarios involving rigid supports are presenting a four times increase of the bending moments, for the S1.1, S2.1 and S3.1 scenarios and a three-times increase for S4.1 and S5.1. The sliding and hinged-sliding support scenarios, on the other hand, present a slight decrease of the bending forces of up to 10%. Scenarios involving component method (S4.2 and S4.3) and Heimeshoff and Köhler method determined joints (S5.2 and S5.3) are presenting a significant decrease of the out-of-plane bending moments of 80 up to 100%.

Table 4-89 Out-of-plane bending forces analysis of the scenarios - building with the first roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With first roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	2.99	1.74	2.95	3.30	3.07	3.17
3.45	4.68	3.99	4.31	4.33	4.34	4.21
3.75	3.22	2.48	3.00	3.13	3.04	3.03
7.2	5.97	4.80	5.03	4.84	5.03	4.96
7.4	3.66	0.31	0.08	-0.19	0.28	0.18
10.05	2.53	7.71	8.04	8.01	6.49	6.42
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	2.99	3.29	3.35	3.28	3.09	2.89
3.45	4.68	3.65	3.83	3.86	3.76	3.80
3.75	3.22	2.83	2.92	2.94	2.88	2.88
7.2	5.97	5.84	5.14	5.00	4.85	4.81
7.4	3.66	1.45	0.54	0.37	0.45	0.45
10.05	2.53	2.30	2.35	2.46	0.42	0.58
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	2.99	2.21	2.26	2.45	1.12	1.80
3.45	4.68	3.59	3.79	3.91	3.79	3.87
3.75	3.22	2.47	2.77	2.93	2.37	2.71
7.2	5.97	4.66	4.60	4.73	3.73	4.24
7.4	3.66	0.37	-0.09	-0.04	-0.46	-0.17
10.05	2.53	2.39	2.26	2.33	0.06	0.33

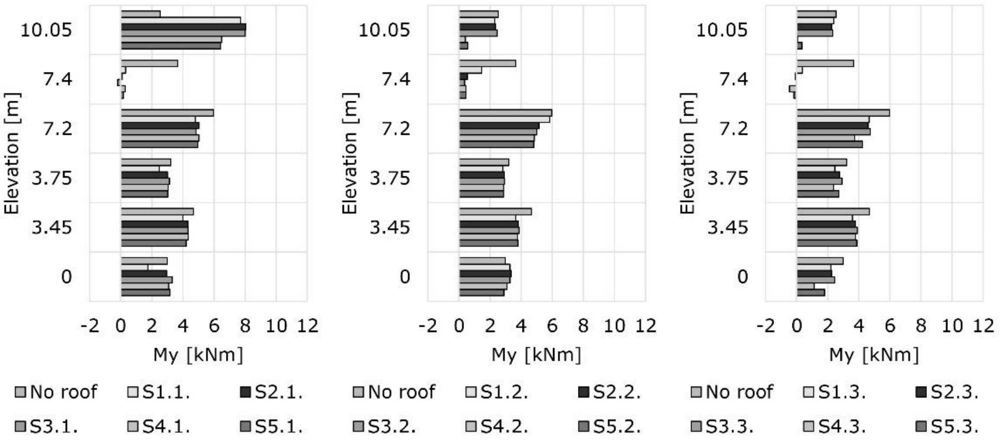


Fig. 4.34 Out-of-plane bending forces diagrams analysis of the scenarios - building with the first roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Even the presence of the second roof structure is leading to a variation of the obtained values when considering the different support and joint types (Table 4-90, Fig. 4.35). Still, the differences, in this case, are not as high as in the first roof structure case.

At the base of the first floor of the building, all the scenarios are presenting an increase of the bending moments, ranging up to 20%, for the S5.1, S5.2 and S5.3. Rigid joint scenarios, on the other hand, present approximately similar values to the ones recorded in the no roof structure case while all the other scenarios are presenting an increase of about 15%.

On the top of the same floor, on the other hand, the behaviour changes in this case also, all the scenarios presenting a slight decrease of the bending forces of 10 up to 20%. Still, variations of 10% can be still identified on this floor between scenarios.

The second floor is presenting both at the bottom and the top an increase of the bending moment, with variations of less than 5% between the considered scenarios. Only the rigid joints scenarios are presenting a decrease at the base of the second floor of 10%.

At the base of the top floor, the recoded bending moments present once again a decrease, of 10% in the case of the rigid joint scenarios up to 40% for all the others. Excluding the rigid joints scenarios, at this point, the differences between the obtained bending moments are under 5%.

Ultimately, at the top of the building, two different behaviours were observed. On the one hand, scenarios involving rigid joints are presenting a 40% increase of the bending moments, while all the other scenarios are showing an up to four-times increase. Despite the high difference between the two behaviours, the variations of the bending moments are still under 10%, excluding the rigid support scenarios.

Table 4-90 Out-of-plane bending forces analysis of the scenarios - building with the second roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With second roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	2.99	3.00	3.25	3.27	3.26	3.54
3.45	4.68	3.82	4.12	4.13	4.13	4.25
3.75	3.22	2.95	3.32	3.34	3.33	3.54
7.2	5.97	7.03	6.50	6.52	6.50	6.55
7.4	3.66	2.98	1.66	1.68	1.66	1.62
10.05	2.53	4.62	12.69	12.57	12.68	12.18
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	2.99	3.00	3.44	3.46	3.45	3.64
3.45	4.68	3.82	4.20	4.21	4.21	4.29
3.75	3.22	2.95	3.45	3.46	3.46	3.60
7.2	5.97	7.15	6.96	6.99	6.96	6.93
7.4	3.66	3.22	2.21	2.25	2.20	2.12
10.05	2.53	3.57	9.86	9.66	9.86	9.57
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	2.99	3.04	3.46	3.47	3.46	3.66
3.45	4.68	3.82	4.20	4.20	4.20	4.28
3.75	3.22	2.95	3.45	3.46	3.46	3.59
7.2	5.97	7.19	6.96	6.99	6.96	6.93
7.4	3.66	3.24	2.19	2.23	2.19	2.10
10.05	2.53	3.55	9.82	9.62	9.82	9.53

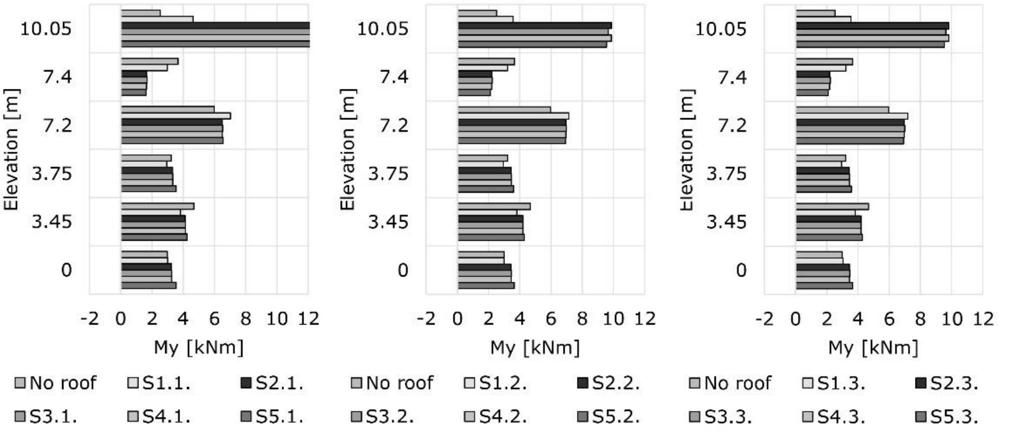


Fig. 4.35 Out-of-plane bending forces diagrams analysis of the scenarios - building with the second roof structure with complete cross-section timber elements (comparison to the no roof structure case)

Finally, considering the third roof structure also limited variation of the obtained values was observed for all the considered scenarios (Table 4-91, Fig. 4.36). Like in the case of the axial and shear forces, the bending moments are presenting up to 20-30% differences between the scenarios starting with the base of the building. The main differences appear in this case between the scenarios involving rigid joints and all the other scenarios.

At the base of the first floor of the building, all the considered scenarios are presenting a decrease of the bending moment of 20 up to 50%. The variations at this level are rather high for all the scenarios considering rigid joints or rigid supports, while only the S2.2, S3.2, S4.2, S5.2, S2.3, S3.3, S4.3 and S5.3 are presenting a similar decrease of the bending moment of 25%. At the top of the first floor, the recorded values also present a decrease of the bending moments reaching from 20% for rigid joints scenarios down to 5% for the others.

The second floor is presenting two different behaviours on the bottom and top of the floor. First, a slight decrease of the bending moments was observed at the bottom, of up to 35%, recorded for the S1.1 scenario. Scenarios S1.2 and S1.3 are also presenting a decrease of the bending moments of up to 15%. All the other scenarios are presenting a slight decrease of the forces of 5% (a decrease of 10% was only observed for the S4.1 scenario). At the top of the second floor, the bending moments are presenting an increase of around 15% for the rigid joint scenarios and 35-40% for all the other scenarios.

On the top floor also two different trends of the bending moments were identified, for both the bottom and the top of the floor. At the bottom, the S1.1 and S1.3 scenarios are presenting a slight decrease of the bending moments of 5% while all the other scenarios are presenting an increase of up to 20%. The exceptions are the S2.1 and S4.1 scenarios which are presenting a 15% increase and the S1.2 scenario, which is presenting almost no difference compared to the no roof structure case. At the top, on the other hand, all the scenarios involving rigid joints are showing a 35% increase of the bending moments, while the other scenarios are presenting a decrease of 10 up to 25%. In this point, the highest variations between the scenarios were identified, of up to 15%, except for the S2.1, S3.1, S4.1 and S5.1 scenarios which present insignificant differences between each other.

Table 4-91 Out-of-plane bending forces analysis of the scenarios - building with the third roof structure with complete cross-section timber elements

Elevation [m]	Without roof	With third roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	2.99	1.34	2.00	2.31	1.78	2.39
3.45	4.68	3.85	4.31	4.53	4.02	4.46
3.75	3.22	2.06	3.08	3.15	2.97	3.24
7.2	5.97	6.57	7.89	7.99	7.89	8.21
7.4	3.66	3.36	4.22	4.46	4.27	4.54
10.05	2.53	3.46	2.20	2.15	2.11	2.11
	No roof	S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	2.99	1.57	2.20	2.21	2.27	2.28
3.45	4.68	3.83	4.54	4.49	4.55	4.45
3.75	3.22	2.72	3.15	3.10	3.13	3.21
7.2	5.97	7.01	8.05	8.03	8.09	8.21
7.4	3.66	3.70	4.31	4.41	4.43	4.44
10.05	2.53	3.39	2.28	1.93	2.18	1.87
	No roof	S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	2.99	1.47	2.25	2.23	2.29	2.30
3.45	4.68	3.83	4.59	4.51	4.57	4.47
3.75	3.22	2.69	3.17	3.11	3.13	3.21
7.2	5.97	6.75	8.05	8.02	8.08	8.18
7.4	3.66	3.51	4.30	4.40	4.42	4.42
10.05	2.53	3.36	2.32	1.95	2.16	1.89

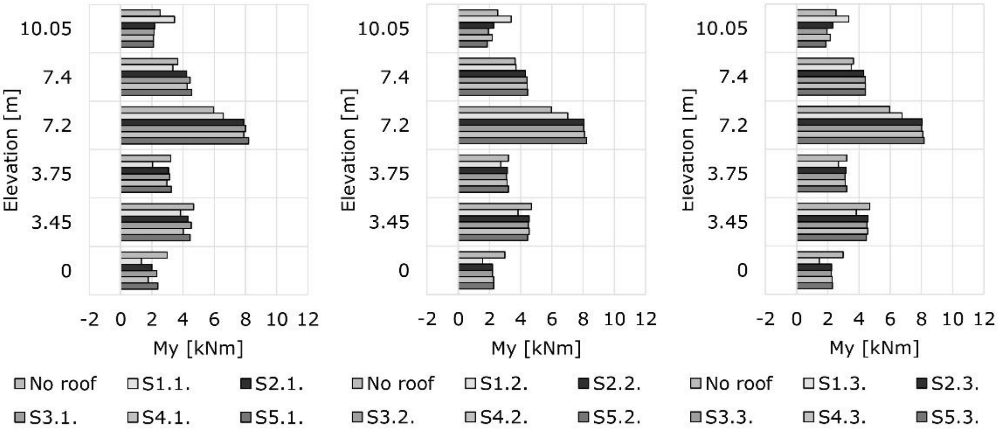


Fig. 4.36 Out-of-plane bending forces diagrams analysis of the scenarios - building with the third roof structure with complete cross-section timber elements (comparison to the no roof structure case)

While comparing the out-of-plane bending moments on the historic masonry building without and with the three chosen roof structures, the following was observed (Table 4-92):

1. The highest bending moments were identified:
    1.1. at the top of the second floor, in the area of the first timber beam flooring for sliding support and hinged-sliding support scenarios and at the top of the last floor for rigid support scenarios for the first roof structure;
    1.2. at the top of the second floor, in the area of the first timber beam flooring for the third roof structure for rigid and hinged and sliding support scenarios and the top of the building for sliding support scenarios (except S1.2) for the second and third roof structure;
2. Significant variations of the bending moment values between the assessed scenarios for all the considered measuring points.
3. Decrease of the bending moments at the bottom of the second floor.
4. Top of the second floor (compared to the no roof case):
    4.1. Decrease of the bending moment values for the first roof structure;
    4.2. Increase of the bending moment values for the second and third roof structure;
5. Bottom of the top floor (compared to the no roof case):
    5.1. Decrease of the bending moment values for the first roof structure, values close to 0kN;
    5.2. Decrease of the bending moment values for the second and third roof structure;
6. Top of the last floor (compared to the no roof case):
    6.1. Increase of the bending moment values for rigid support scenarios and a slight decrease for all the other scenarios in the case of the first roof structure;
    6.2. Significant increase of the bending moment values for the second roof structure;
    6.3. Increase of the bending moment values for rigid joints scenarios and a slight decrease for all the other scenarios in the case of the third roof structure.

Table 4-92 Comparison of the out-of-plane bending forces on the historic masonry walls of the building without and with the three roof structures with complete cross-section timber elements

		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3
0	S1.1	1.74	3.00	1.34	S1.2	3.29	3.00	1.57	S1.3	2.21	3.04	1.47
3.45		3.99	3.82	3.85		3.65	3.82	3.83		3.59	3.82	3.83
3.75		2.48	2.95	2.06		2.83	2.95	2.72		2.47	2.95	2.69
7.2		4.80	7.03	6.57		5.84	7.15	7.01		4.66	7.19	6.75
7.4		0.31	2.98	3.36		1.45	3.22	3.70		0.37	3.24	3.51
10.05		7.71	4.62	3.46		2.30	3.57	3.39		2.39	3.55	3.36
0	S2.1	2.95	3.25	2.00	S2.3	3.35	3.44	2.20	S2.3	2.26	3.46	2.25
3.45		4.31	4.12	4.31		3.83	4.20	4.54		3.79	4.20	4.59
3.75		3.00	3.32	3.08		2.92	3.45	3.15		2.77	3.45	3.17
7.2		5.03	6.50	7.89		5.14	6.96	8.05		4.60	6.96	8.05
7.4		0.08	1.66	4.22		0.54	2.21	4.31		-0.09	2.19	4.30
10.05		8.04	12.69	2.20		2.35	9.86	2.28		2.26	9.82	2.32
0	S3.1	3.30	3.27	2.31	S3.2	3.28	3.46	2.21	S3.3	2.45	3.47	2.23
3.45		4.33	4.13	4.53		3.86	4.21	4.49		3.91	4.20	4.51
3.75		3.13	3.34	3.15		2.94	3.46	3.10		2.93	3.46	3.11
7.2		4.84	6.52	7.99		5.00	6.99	8.03		4.73	6.99	8.02
7.4		-0.19	1.68	4.46		0.37	2.25	4.41		-0.04	2.23	4.40
10.05		8.01	12.57	2.15		2.46	9.66	1.93		2.33	9.62	1.95
0	S4.1	3.07	3.26	1.78	S4.2	3.09	3.45	2.27	S4.3	1.12	3.46	2.29
3.45		4.34	4.13	4.02		3.76	4.21	4.55		3.79	4.20	4.57
3.75		3.04	3.33	2.97		2.88	3.46	3.13		2.37	3.46	3.13
7.2		5.03	6.50	7.89		4.85	6.96	8.09		3.73	6.96	8.08
7.4		0.28	1.66	4.27		0.45	2.20	4.43		-0.46	2.19	4.42
10.05		6.49	12.68	2.11		0.42	9.86	2.18		0.06	9.82	2.16
0	S5.1	3.17	3.54	2.39	S5.2	2.89	3.64	2.28	S5.3	1.80	3.66	2.30
3.45		4.21	4.25	4.46		3.80	4.29	4.45		3.87	4.28	4.47
3.75		3.03	3.54	3.24		2.88	3.60	3.21		2.71	3.59	3.21
7.2		4.96	6.55	8.21		4.81	6.93	8.21		4.24	6.93	8.18
7.4		0.18	1.62	4.54		0.45	2.12	4.44		-0.17	2.10	4.42
10.05		6.42	12.18	2.11		0.58	9.57	1.87		0.33	9.53	1.89

4.3.5.3 Roof structures with 20% reduced cross-section timber elements

4.3.5.3.1 Vertical axial forces (N_x)

Subsequently, as in the case of the previously analysed parameters, the decayed three roof structures were placed on the historic masonry building, and the out-of-plane internal forces were assessed by comparing them to the no roof structure case.

First, the axial forces were analysed. For the first roof structure (Table 4-93, Fig. 4.37), considering all the assessed scenarios, it was once again observed that the chosen support typology is having a more considerable influence on the changes of the axial force recorded on the historic masonry walls, then the different joint axial stiffness calculation methods.

Therefore, the differences between the obtained values, range up to 5% until the top of the second floor, and up to 10% for the bottom of the third level. At the top of the building, the differences are significantly higher, ranging up to 100%, depending on the chosen joint axial stiffness. Scenarios involving rigid support present the highest increase of the axial forces on the top of the building, of about 15 times, while scenarios involving hinged-sliding supports are presenting an increase of about five times. At the same time, for all the assessed scenarios, higher compressive axial forces were identified at the base of the structure while at the top, all of them are presenting tensile axial forces. Scenarios involving rigid supports present tensile axial forces about twice as high as the other scenarios. Still, the floors below present only compression and no additional tensile axial forces were identified.

Table 4-93 Vertical axial forces analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With first roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	-105.31	-105.79	-100.36	-99.71	-100.42	-100.62
3.45	-57.63	-58.12	-52.68	-52.03	-52.74	-52.95
3.75	-56.70	-57.02	-52.63	-52.07	-52.63	-53.01
7.2	-24.74	-24.36	-21.72	-21.33	-21.70	-22.59
7.4	-23.20	-1.23	-0.86	-1.3	0.45	-2.28
10.05	-1.35	20.62	20.99	20.55	22.31	19.57
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	-105.31	-108.84	-104.10	-104.25	-107.26	-107.75
3.45	-57.63	-61.16	-56.43	-56.57	-59.58	-60.07
3.75	-56.7	-59.88	-56.05	-55.99	-58.44	-58.72
7.2	-24.74	-27.01	-25.17	-24.58	-25.70	-25.86
7.4	-23.2	-9.83	-14.36	-12.18	-7.58	-8.39
10.05	-1.35	12.02	7.49	9.68	14.27	13.46
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	-105.31	-111.11	-105.63	-105.56	-107.23	-107.07
3.45	-57.63	-63.44	-57.95	-57.88	-59.55	-59.40
3.75	-56.7	-61.00	-56.85	-56.82	-58.31	-58.03
7.2	-24.74	-27.23	-25.37	-25.38	-26.49	-25.97
7.4	-23.2	-14.57	-16.14	-16.15	-14.10	-13.09
10.05	-1.35	7.29	5.71	5.7	7.75	8.76

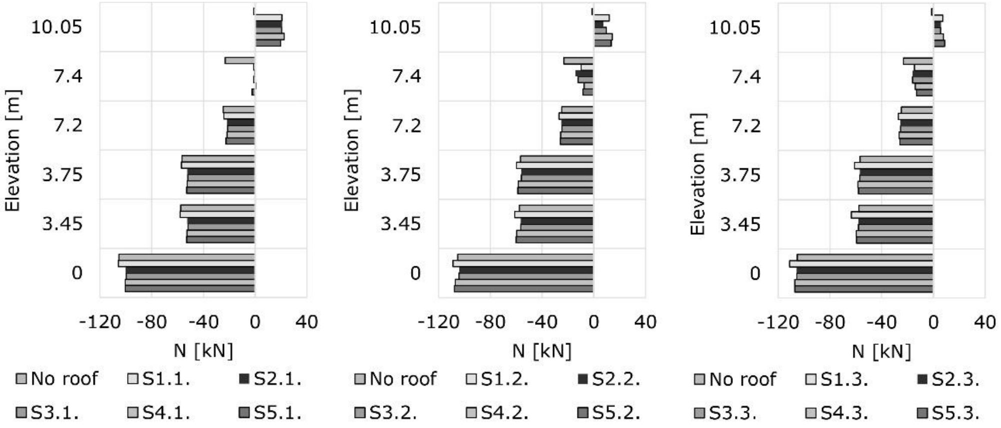


Fig. 4.37 Vertical axial forces diagrams analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements (comparison to the no roof case)

The presence of the second roof structure is no longer showing a similar distribution of the axial forces as in the no roof structure case (Table 4-94, Fig. 4.38), also presenting tensile axial forces on the top of the last floor, for the scenarios involving semi-rigid joints determined using the component method and the Heimeshoff and Köhler method (S4.1, S5.1, S4.2, S5.2, S4.3, S5.3). Scenario S4.1 is the only scenario which is also presenting tensile axial forces at the base of the top floor. Due to the presence of the roof structure, at the top of the building, all the other scenarios are presenting up to 10 times higher compressive axial forces, compared to the no roof structure case.

In this case, no significant differences could be observed between the assessed scenarios, the values presenting up to 5% variation depending on the chosen support and joint axial stiffness, until the base of the top floor. The variations are higher at the top of the building. Still, the scenarios involving rigid joints, present once again an exception, presenting with up to 20% higher compressive forces than the other considered scenarios.

Table 4-94 Vertical axial forces analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With second roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	-105.31	-112.55	-100.53	-100.48	-100.41	-100.73
3.45	-57.63	-64.87	-52.85	-52.80	-52.73	-53.05
3.75	-56.7	-65.25	-53.40	-53.35	-52.63	-53.22
7.2	-24.74	-38.82	-27.22	-27.19	-21.69	-23.21
7.4	-23.2	-37.39	-34.67	-34.69	0.44	-4.19
10.05	-1.35	-15.53	-12.82	-12.84	22.29	17.66
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	-105.31	-112.87	-98.47	-100.70	-107.26	-108.13
3.45	-57.63	-65.19	-50.79	-53.02	-59.58	-60.45
3.75	-56.7	-65.59	-51.41	-53.55	-58.44	-59.08
7.2	-24.74	-39.45	-25.64	-27.35	-25.70	-26.32
7.4	-23.2	-38.69	-35.17	-34.73	-7.56	-9.22
10.05	-1.35	-16.83	-13.32	-12.87	14.29	12.63
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	-105.31	-112.76	-98.35	-98.30	-107.22	-107.31
3.45	-57.63	-65.09	-50.67	-50.62	-59.54	-59.63
3.75	-56.7	-65.55	-51.38	-51.34	-58.30	-58.18
7.2	-24.74	-39.53	-25.73	-25.69	-26.48	-25.95
7.4	-23.2	-38.77	-35.20	-35.22	-14.10	-12.82
10.05	-1.35	-16.92	-13.35	-13.37	7.75	9.03

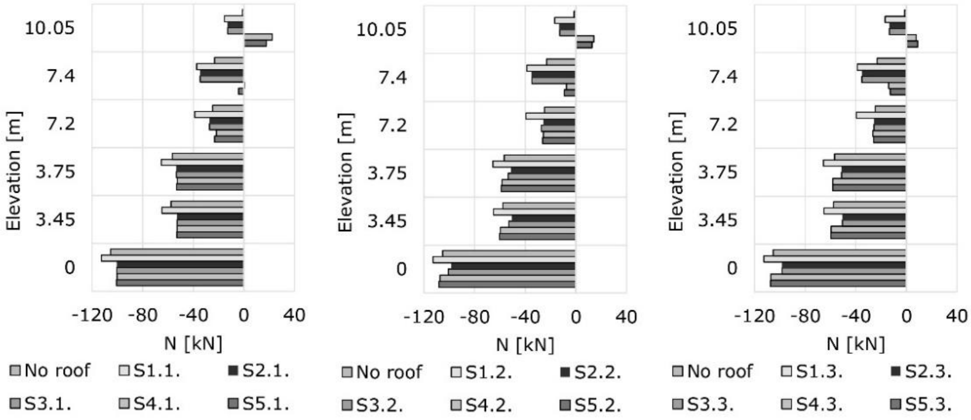


Fig. 4.38 Vertical axial forces diagrams analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements (comparison to the no roof case)

Ultimately, for the third roof structure (Table 4-95, Fig. 4.39), it is once again highlighted that the chosen support type and joint axial stiffness are having limited influence on the changes of the axial force along the exterior historic masonry wall, until the top of the building. Therefore, the differences between the obtained values, compared to the no roof scenario, range up to 10% for every floor of the building, when comparing the scenarios with the same support type, while at the top of the buildings, the differences vary with up to 300%. Still, the difference between the scenarios for all the floors is rather low.

Table 4-95 Vertical axial forces analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With third roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	-105.31	-105.44	-101.12	-101.39	-101.34	-101.56
3.45	-57.63	-57.77	-53.44	-53.71	-53.66	-53.88
3.75	-56.70	-55.44	-51.83	-51.99	-51.93	-52.13
7.2	-24.74	-22.26	-19.75	-19.90	-19.67	-20.27
7.4	-23.20	12.65	16.13	11.21	14.67	6.06
10.05	-1.35	34.50	37.98	33.06	36.52	27.91
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	-105.31	-106.05	-101.69	-102.21	-101.92	-102.58
3.45	-57.63	-58.38	-54.01	-54.53	-54.24	-54.90
3.75	-56.7	-55.93	-52.29	-52.68	-52.45	-52.96
7.2	-24.74	-22.87	-20.16	-20.45	-20.25	-20.74
7.4	-23.2	9.51	14.30	10.51	13.89	6.24
10.05	-1.35	31.36	36.15	32.36	35.75	28.09
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	-105.31	-106.08	-101.74	-102.20	-101.94	-102.63
3.45	-57.63	-58.40	-54.07	-54.52	-54.26	-54.96
3.75	-56.7	-55.98	-52.37	-52.69	-52.50	-53.01
7.2	-24.74	-22.90	-20.26	-20.48	-20.32	-20.78
7.4	-23.2	9.98	14.25	10.54	13.94	6.27
10.05	-1.35	31.84	36.10	32.39	35.79	28.12

At the same time, for all the assessed scenarios, compressive axial forces were identified at the base of the structure, lower than in the case of the no roof scenario, while at the top and bottom of the last floor, all of them are presenting tensile axial forces. The results are consistent with the significant out-of-plane behaviour of the historic masonry wall at the top of the building.

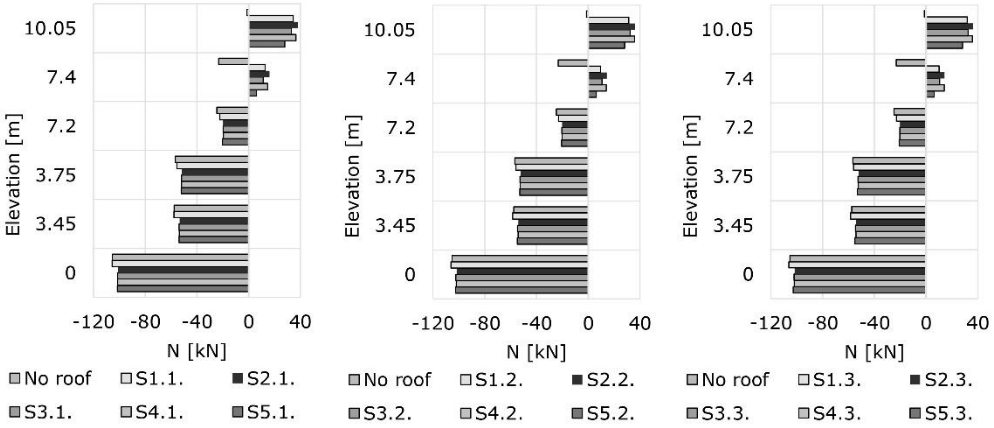


Fig. 4.39 Vertical axial forces diagrams analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

While comparing the axial forces on the historic masonry building without and with the three chosen decayed roof structures, the following was observed (Table 4-96):

1. The axial forces on the wall for the building with all the three roof structures are approximately similar on the first and second floor of the building, and the differences start to be observed above this threshold.
2. On the top floor:
   2.1. The first roof structure is causing tensile axial forces only at the top of the wall for all the assessed scenarios;
   2.2. The second roof structure is presenting tensile axial forces only at the top of the wall for scenarios involving the component method and the Heimeshoff and Köhler method (S5.1, S4.2, S5.2, S4.3, S5.3), while only scenario S4.1 is presenting tensile axial forces at the base and top of the last floor;
   2.3. The third roof structure is presenting tensile axial forces at the base and top of the last floor for all the considered scenarios.
3. Only little differences were observed while comparing the results obtained for the same support type and different joint axial stiffnesses.

Table 4-96 Comparison of the vertical axial forces on the historic masonry walls of the building without and with the three roof structures with reduced cross-section timber elements

		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3
0	S1.1	-105.79	-112.55	-105.44	S1.2	-108.84	-112.87	-106.05	S1.3	-111.11	-112.76	-106.08
3.45		-58.12	-64.87	-57.77		-61.16	-65.19	-58.38		-63.44	-65.09	-58.40
3.75		-57.02	-65.25	-55.44		-59.88	-65.59	-55.93		-61.00	-65.55	-55.98
7.2		-24.36	-38.82	-22.26		-27.01	-39.45	-22.87		-27.23	-39.53	-22.90
7.4		-1.23	-37.39	12.65		-9.83	-38.69	9.51		-14.57	-38.77	9.98
10.05		20.62	-15.53	34.50		12.02	-16.83	31.36		7.29	-16.92	31.84
0	S2.1	-100.36	-100.53	-101.12	S2.3	-104.10	-98.47	-101.69	S2.3	-105.63	-98.35	-101.74
3.45		-52.68	-52.85	-53.44		-56.43	-50.79	-54.01		-57.95	-50.67	-54.07
3.75		-52.63	-53.40	-51.83		-56.05	-51.41	-52.29		-56.85	-51.38	-52.37
7.2		-21.72	-27.22	-19.75		-25.17	-25.64	-20.16		-25.37	-25.73	-20.26
7.4		-0.86	-34.67	16.13		-14.36	-35.17	14.30		-16.14	-35.20	14.25
10.05		20.99	-12.82	37.98		7.49	-13.32	36.15		5.71	-13.35	36.10
0	S3.1	-99.71	-100.48	-101.39	S3.2	-104.25	-100.70	-102.21	S3.3	-105.56	-98.30	-102.20
3.45		-52.03	-52.80	-53.71		-56.57	-53.02	-54.53		-57.88	-50.62	-54.52
3.75		-52.07	-53.35	-51.99		-55.99	-53.55	-52.68		-56.82	-51.34	-52.69
7.2		-21.33	-27.19	-19.90		-24.58	-27.35	-20.45		-25.38	-25.69	-20.48
7.4		-1.30	-34.69	11.21		-12.18	-34.73	10.51		-16.15	-35.22	10.54
10.05		20.55	-12.84	33.06		9.68	-12.87	32.36		5.70	-13.37	32.39
0	S4.1	-100.42	-100.41	-101.34	S4.2	-107.26	-107.26	-101.92	S4.3	-107.23	-107.22	-101.94
3.45		-52.74	-52.73	-53.66		-59.58	-59.58	-54.24		-59.55	-59.54	-54.26
3.75		-52.63	-52.63	-51.93		-58.44	-58.44	-52.45		-58.31	-58.30	-52.50
7.2		-21.70	-21.69	-19.67		-25.70	-25.70	-20.25		-26.49	-26.48	-20.32
7.4		0.45	0.44	14.67		-7.58	-7.56	13.89		-14.10	-14.10	13.94
10.05		22.31	22.29	36.52		14.27	14.29	35.75		7.75	7.75	35.79
0	S5.1	-100.62	-100.73	-101.56	S5.2	-107.75	-108.13	-102.58	S5.3	-107.07	-107.31	-102.63
3.45		-52.95	-53.05	-53.88		-60.07	-60.45	-54.90		-59.40	-59.63	-54.96
3.75		-53.01	-53.22	-52.13		-58.72	-59.08	-52.96		-58.03	-58.18	-53.01
7.2		-22.59	-23.21	-20.27		-25.86	-26.32	-20.74		-25.97	-25.95	-20.78
7.4		-2.28	-4.19	6.06		-8.39	-9.22	6.24		-13.09	-12.82	6.27
10.05		19.57	17.66	27.91		13.46	12.63	28.09		8.76	9.03	28.12

4.3.5.3.2 Out-of-plane shear forces (V_z)

Subsequently, the shear forces were analysed while considering the influence of the three chosen roof structures. The effect of the presence of the first roof structure is mostly visible, starting with the base of the top floor (Table 4-97, Fig. 4.40). Up until that point, the values only vary with up to 5%, compared to the no roof scenario.

Starting with the base of the top floor, the analysis of the models with the first roof structure are presenting two different distributions of the shear forces, depending on the chosen support type. On the one hand, rigid support scenarios are presenting a significant increase of the shear forces, twice as high as in the no roof structure case. The differences between these scenarios are under 10%. The other scenarios, involving sliding and hinged-sliding supports present only a slight increase of the shear forces, of up to 30%, showing a variation of the values of 10%.

Table 4-97 Out-of-plane shear forces analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With first roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	3.40	3.42	3.53	3.53	3.50	3.45
3.45	3.40	3.42	3.53	3.53	3.50	3.45
3.75	-1.47	-1.37	-1.32	-1.32	-1.34	-1.34
7.2	2.11	2.53	2.46	2.45	2.45	2.46
7.4	1.69	3.69	3.74	3.75	3.46	3.37
10.05	1.69	3.69	3.74	3.75	3.46	3.37
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	3.4	3.13	3.16	3.13	3.06	3.03
3.45	3.4	3.13	3.16	3.13	3.06	3.03
3.75	-1.47	-1.54	-1.49	-1.52	-1.60	-1.62
7.2	2.11	2.41	2.35	2.33	2.09	2.04
7.4	1.69	2.24	2.22	2.18	2.07	2.00
10.05	1.69	2.24	2.22	2.18	2.07	2.00
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	3.4	3.02	3.07	3.06	3.02	3.01
3.45	3.4	3.02	3.07	3.06	3.02	3.01
3.75	-1.47	-1.55	-1.53	-1.54	-1.59	-1.61
7.2	2.11	2.35	2.57	2.58	2.34	2.39
7.4	1.69	2.08	2.24	2.23	2.03	2.05
10.05	1.69	2.08	2.24	2.23	2.03	2.05

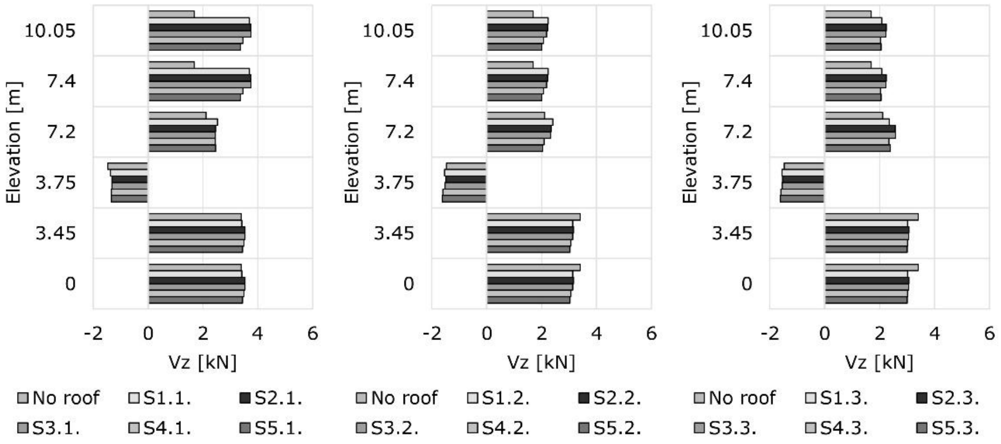


Fig. 4.40 Out-of-plane shear forces diagrams analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Like in the previous case, the effect of the presence of the second roof structure is visible on the top of the second floor and on the top floor (Table 4-98, Fig. 4.41).

On the first floor, the variation between the assessed scenarios is relatively low, under 5%. Only scenario S4.1 is presenting a slight increase of the shear forces compared to the no roof case; all the others showing a decrease of the forces. On the base of the second floor, the values start to vary with up to 20%, presenting an increase of the shear forces of up to 10%, for all the scenarios except for the S4.2, S5.2, S4.3 and S5.3 scenarios which present a decrease of 10%.

Starting with the top of the second floor, the recorded shear forces are starting to present a significant increase compared to the no roof structure case, of 60%. The exceptions are the S4.2, S5.2, S4.3 and S5.3 scenarios which are presenting only a 20% increase of the forces.

On the top floor, three different changes in the shear force were observed. On the one hand, scenarios involving rigid joints are presenting a decrease of the shear forces of 10%, for scenario S1.1 and up to 40% for S1.2 and S1.3. Scenarios involving hinged joints and semi-rigid joints determined using the Hölzer method, present an up to 2.5 times increase of the shear forces at the top of the building. Scenarios S4.1, S5.1, S4.2, S5.2, S4.3 and S5.3, on the other hand, present only a 20% increase of the shear forces.

For this roof structure, the importance of properly considering the roof to wall connection and joint axial stiffness becomes visible, each scenario influencing the shear forces in a different way.

Table 4-98 Out-of-plane shear forces analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With second roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	3.4	3.30	3.39	3.39	3.50	3.35
3.45	3.4	3.30	3.39	3.39	3.50	3.35
3.75	-1.47	-1.44	-1.32	-1.32	-1.34	-1.39
7.2	2.11	3.60	3.37	3.37	2.45	2.45
7.4	1.69	1.50	5.38	5.35	3.46	3.17
10.05	1.69	1.50	5.38	5.35	3.46	3.17
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	3.4	3.29	3.39	3.38	3.06	3.03
3.45	3.4	3.29	3.39	3.38	3.06	3.03
3.75	-1.47	-1.45	-1.29	-1.32	-1.60	-1.61
7.2	2.11	3.65	3.54	3.37	2.09	2.03
7.4	1.69	0.97	4.34	5.34	2.06	1.98
10.05	1.69	0.97	4.34	5.34	2.06	1.98
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	3.4	3.30	3.39	3.39	3.02	3.00
3.45	3.4	3.30	3.39	3.39	3.02	3.00
3.75	-1.47	-1.44	-1.29	-1.29	-1.59	-1.62
7.2	2.11	3.65	3.54	3.55	2.34	2.40
7.4	1.69	0.97	4.35	4.29	2.03	2.05
10.05	1.69	0.97	4.35	4.29	2.03	2.05

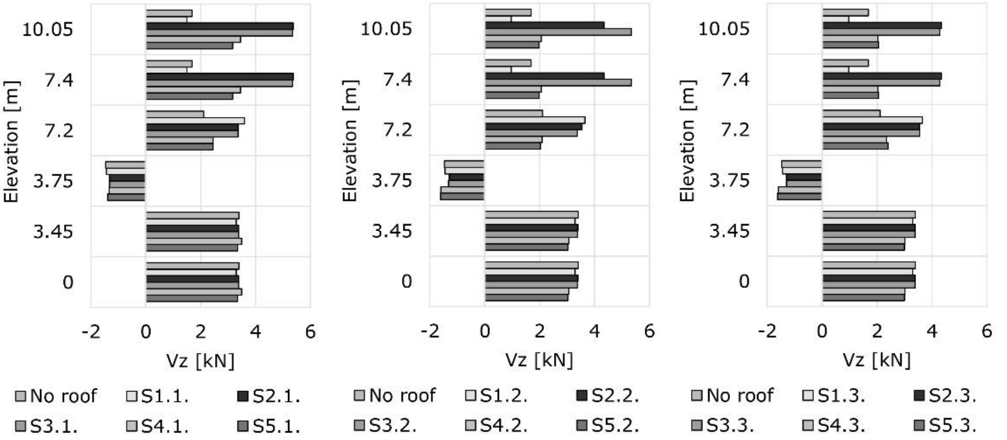


Fig. 4.41 Out-of-plane shear forces diagrams analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

On the contrary to the previously analysed models with the first and second roof structure, the presence of the third one is also influencing the recorded shear forces on the first floor of the building (Table 4-99, Fig. 4.42).

Table 4-99 Out-of-plane shear forces analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With third roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	3.40	3.28	3.33	3.27	3.31	3.15
3.45	3.40	3.28	3.33	3.27	3.31	3.15
3.75	-1.47	-1.43	-1.39	-1.41	-1.39	-1.46
7.2	2.11	3.39	3.24	3.29	3.25	3.36
7.4	1.69	3.04	2.44	2.37	2.43	2.33
10.05	1.69	3.04	2.44	2.37	2.43	2.33
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	3.4	3.20	3.28	3.22	3.27	3.12
3.45	3.4	3.20	3.28	3.22	3.27	3.12
3.75	-1.47	-1.48	-1.41	-1.44	-1.41	-1.49
7.2	2.11	3.44	3.28	3.31	3.34	3.33
7.4	1.69	3.00	2.34	2.25	2.39	2.15
10.05	1.69	3.00	2.34	2.25	2.39	2.15
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	3.4	3.23	3.29	3.23	3.28	3.13
3.45	3.4	3.23	3.29	3.23	3.28	3.13
3.75	-1.47	-1.47	-1.41	-1.43	-1.40	-1.48
7.2	2.11	3.42	3.26	3.3	3.32	3.32
7.4	1.69	2.97	2.31	2.24	2.37	2.13
10.05	1.69	2.97	2.31	2.24	2.37	2.13

Therefore, it was observed that even at the base of the building, the shear forces are 5 up to 10% lower due to the presence of the roof structure. Despite this, in the area of the cross-vault, the differences between the scenarios and the no roof case are insignificant, under 5%, with only little variations between the various considered scenarios.

Starting with the top of the second floor, the recorded shear forces are starting to show an increase compared to the no roof structure case of up to 60%. The recorded values vary with under 5%.

On the top floor, the rigid joint scenarios are causing the most significant increase of the shear forces of up to 80% while all the other scenarios are presenting an increase of 30 up to 40%.

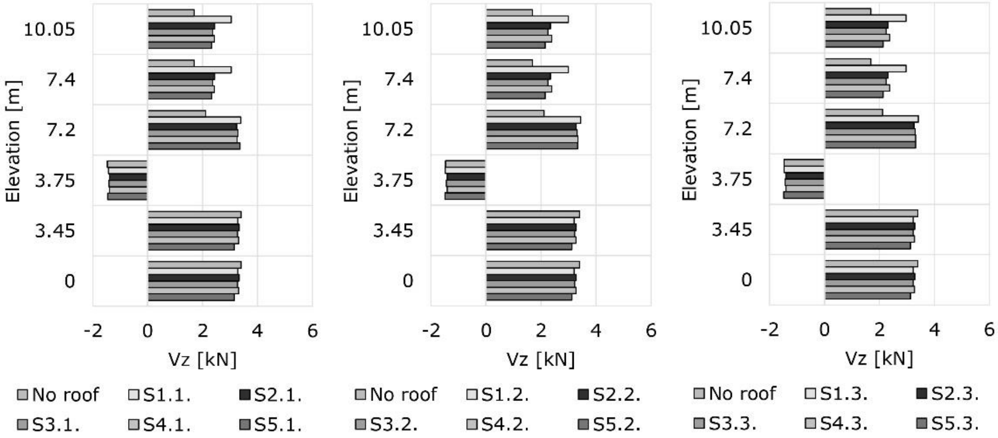


Fig. 4.42 Out-of-plane shear forces diagrams analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

While comparing the shear forces on the historic masonry building without and with the three chosen decayed roof structures, the following was observed (Table 4-100):

1. Even with the decayed roof structures, the building is still presenting shear forces in the opposite direction around the cross-vault.
2. The shear forces on the first floor and the base of the second floor of the building when comparing the chosen joint types are approximately similar for all the three roof structures.
3. On the top floor:
    - 3.1. The first roof structure is causing a two times increase of the shear forces for rigid support scenarios and up to 30% for the other scenarios;
    - 3.2. The second roof structure is causing a 10% decrease of the shear forces for rigid joints scenarios and an up to 2.5 times increase for all the other scenarios;
    - 3.3. The third roof structure is causing an 80% increase of the shear forces for rigid joints scenarios and an up to 40% increase for all the other scenarios.

Table 4-100 Comparison of the out-of-plane shear forces on the historic masonry walls of the building without and with the three roof structures with reduced cross-section timber elements

		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3
0	S1.1	3.42	3.30	3.28	S1.2	3.13	3.29	3.20	S1.3	3.02	3.30	3.23
3.45		3.42	3.30	3.28		3.13	3.29	3.20		3.02	3.30	3.23
3.75		-1.37	-1.44	-1.43		-1.54	-1.45	-1.48		-1.55	-1.44	-1.47
7.2		2.53	3.60	3.39		2.41	3.65	3.44		2.35	3.65	3.42
7.4		3.69	1.50	3.04		2.24	0.97	3.00		2.08	0.97	2.97
10.05		3.69	1.50	3.04		2.24	0.97	3.00		2.08	0.97	2.97
0	S2.1	3.53	3.39	3.33	S2.3	3.16	3.39	3.28	S2.3	3.07	3.39	3.29
3.45		3.53	3.39	3.33		3.16	3.39	3.28		3.07	3.39	3.29
3.75		-1.32	-1.32	-1.39		-1.49	-1.29	-1.41		-1.53	-1.29	-1.41
7.2		2.46	3.37	3.24		2.35	3.54	3.28		2.57	3.54	3.26
7.4		3.74	5.38	2.44		2.22	4.34	2.34		2.24	4.35	2.31
10.05		3.74	5.38	2.44		2.22	4.34	2.34		2.24	4.35	2.31
0	S3.1	3.53	3.39	3.27	S3.2	3.13	3.38	3.22	S3.3	3.06	3.39	3.23
3.45		3.53	3.39	3.27		3.13	3.38	3.22		3.06	3.39	3.23
3.75		-1.32	-1.32	-1.41		-1.52	-1.32	-1.44		-1.54	-1.29	-1.43
7.2		2.45	3.37	3.29		2.33	3.37	3.31		2.58	3.55	3.30
7.4		3.75	5.35	2.37		2.18	5.34	2.25		2.23	4.29	2.24
10.05		3.75	5.35	2.37		2.18	5.34	2.25		2.23	4.29	2.24
0	S4.1	3.50	3.50	3.31	S4.2	3.06	3.06	3.27	S4.3	3.02	3.02	3.28
3.45		3.50	3.50	3.31		3.06	3.06	3.27		3.02	3.02	3.28
3.75		-1.34	-1.34	-1.39		-1.60	-1.60	-1.41		-1.59	-1.59	-1.40
7.2		2.45	2.45	3.25		2.09	2.09	3.34		2.34	2.34	3.32
7.4		3.46	3.46	2.43		2.07	2.06	2.39		2.03	2.03	2.37
10.05		3.46	3.46	2.43		2.07	2.06	2.39		2.03	2.03	2.37
0	S5.1	3.45	3.35	3.15	S5.2	3.03	3.03	3.12	S5.3	3.01	3.00	3.13
3.45		3.45	3.35	3.15		3.03	3.03	3.12		3.01	3.00	3.13
3.75		-1.34	-1.39	-1.46		-1.62	-1.61	-1.49		-1.61	-1.62	-1.48
7.2		2.46	2.45	3.36		2.04	2.03	3.33		2.39	2.40	3.32
7.4		3.37	3.17	2.33		2.00	1.98	2.15		2.05	2.05	2.13
10.05		3.37	3.17	2.33		2.00	1.98	2.15		2.05	2.05	2.13

4.3.5.3.3 Out-of-plane bending moments (M_y)

Finally, the out-of-plane bending moments were analysed for the building with the three decayed roof structures and subsequently compared to the no roof structure case.

For the first roof structure, a variation of the obtained values was observed for all the considered scenarios (Table 4-101, Fig. 4.43).

On the contrary to the axial forces and shear forces previously analysed, the bending moments are presenting up to 15% differences between the scenarios even from the base of the building. At the same time, significant differences were observed at the top of the building, depending on the considered support type.Therefore, at the base of the building, two different behaviours were identified, scenarios S4.2, S5.2, S4.3 and S5.3 presenting a slight decrease of the bending moments while all the other scenarios are presenting an up to 25% increase.

The most peculiar behaviour was recorded at the base of the top floor, where the bending moments are presenting a significant decrease compared to the no roof structure case. In this case only almost all scenarios are presenting bending moments in the opposite direction, except for S1.1, S1.2. Ultimately, at the top of the building, two different behaviours were observed. On the one hand, scenarios involving rigid supports are presenting a three times increase of the bending moments, for the S1.1, S2.1 and S3.1 scenarios and a 2.5 times increase for the S4.1 and S5.1 scenarios. The sliding and hinged-sliding support scenarios, on the other hand, present a decrease of the bending forces of up to 25%, for the scenarios involving rigid joints, hinged joints or semi-rigid joints determined using the Hölzer method. Scenarios involving component method (S4.2 and S4.3) and Heimeshoff and Köhler method determined joints (S5.2 and S5.3) are presenting a significant decrease of the out-of-plane bending moments of up to 85%

Table 4-101 Out-of-plane bending forces analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With first roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	2.99	3.17	3.65	3.68	3.60	3.52
3.45	4.68	4.45	4.53	4.51	4.47	4.37
3.75	3.22	3.18	3.36	3.37	3.33	3.27
7.2	5.97	4.87	4.38	4.32	4.39	4.39
7.4	3.66	0.05	-0.54	-0.60	-0.33	-0.33
10.05	2.53	8.11	7.59	7.61	6.25	6.28
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	2.99	2.57	2.85	2.77	2.50	2.37
3.45	4.68	3.78	3.92	3.93	3.91	3.95
3.75	3.22	2.76	3.00	3.01	2.92	2.91
7.2	5.97	4.77	4.24	4.22	3.77	3.66
7.4	3.66	0.54	-0.37	-0.35	-0.51	-0.66
10.05	2.53	2.07	1.97	2.04	0.65	0.68
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	2.99	2.24	2.70	2.70	2.54	2.52
3.45	4.68	3.96	4.15	4.18	4.18	4.20
3.75	3.22	2.87	3.22	3.25	3.20	3.21
7.2	5.97	4.46	4.64	4.65	4.13	4.27
7.4	3.66	-0.06	-0.11	-0.09	-0.48	-0.30
10.05	2.53	1.81	1.76	1.81	0.36	0.47

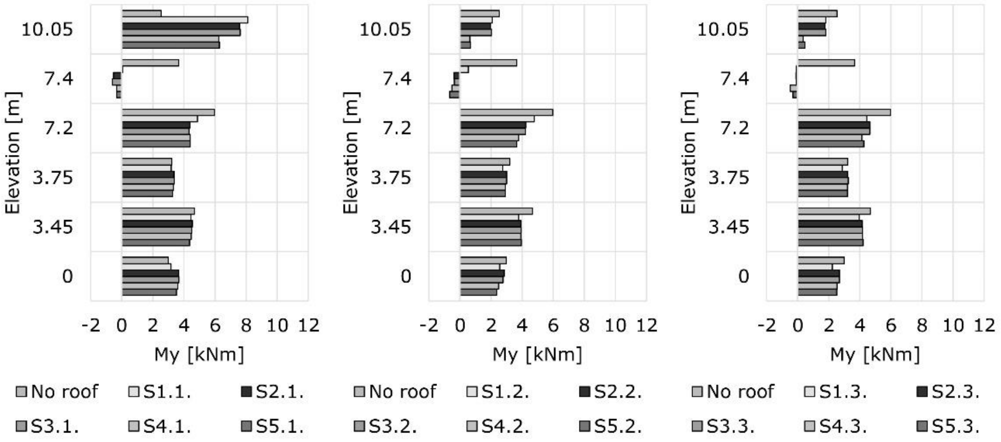


Fig. 4.43 Out-of-plane bending forces diagrams analysis of the scenarios - building with the first roof structure with reduced cross-section timber elements (comparison to the no roof case)

Even the presence of the second roof structure is leading to clear variation of the obtained values when considering the different support and joint types (Table 4-102, Fig. 4.44), presenting increases or decreases on each floor of the building compared to the no roof structure case.

At the base of the first floor of the building, most of the scenarios are presenting an increase of the bending moments, ranging up to 30%. Only scenarios S4.2, S5.2, S4.3 and S5.3, are presenting a decrease of the bending forces up to 20%. On the top of the same floor, on the other hand, the behaviour changes in this case also, all the scenarios presenting a slight decrease of the bending forces of 10%, (15% for the S4.2 and S5.2 scenarios). Still, variations of 10% can be still identified on this floor.

The second floor is presenting both at the bottom and the top two different effects of the presence of the roof structure. Scenarios involving rigid, hinged or semi-rigid joints determined using the Hölzer method, present an increase of the bending moment of up to 20%, with variations of less than 10% between the considered scenarios, while the others are presenting a decrease of up to 30%.

At the base of the top floor, the recorded bending forces present once again a decrease, of 15% (20% for S1.1) in the case of the rigid joint scenarios, up to 60% for all the hinged joints and Hölzer determined joints and up to 115% for the other scenarios. The variations of the obtained results on this floor are significant, highlighting the importance of the considered support and joint type.

Ultimately, at the top of the building, three different behaviours were observed. On the one hand, scenarios involving rigid joints are presenting only a slight increase of the bending forces of 15%, except for the S1.1 scenario, which is presenting a 65% increase of the values. Hinged joints and Hölzer determined joints scenarios present a significant increase of the bending forces, of up to 5 times. Ultimately, scenarios S4.1 and S5.1 are presenting a 2.5 times increase of the obtained values while scenarios S4.2, S5.2, S4.3 and S5.3 are presenting a significant decrease of the bending forces of 80%.

Table 4-102 Out-of-plane bending forces analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With second roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	2.99	3.54	3.72	3.73	3.60	3.36
3.45	4.68	4.16	4.47	4.47	4.47	4.18
3.75	3.22	3.46	3.82	3.82	3.33	3.20
7.2	5.97	7.19	6.5	6.51	4.39	4.31
7.4	3.66	2.93	1.48	1.50	-0.33	-0.49
10.05	2.53	4.21	12.13	12.04	6.25	6.26
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	2.99	3.55	3.82	3.72	2.50	2.35
3.45	4.68	4.18	4.52	4.47	3.91	3.96
3.75	3.22	3.48	3.89	3.82	2.92	2.90
7.2	5.97	7.30	6.85	6.49	3.77	3.62
7.4	3.66	3.14	1.94	1.49	-0.51	-0.72
10.05	2.53	2.86	9.35	12.04	0.65	0.67
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	2.99	3.56	3.84	3.84	2.54	2.48
3.45	4.68	4.17	4.51	4.51	4.18	4.19
3.75	3.22	3.47	3.89	3.89	3.20	3.21
7.2	5.97	7.31	6.85	6.87	4.13	4.30
7.4	3.66	3.14	1.92	1.95	-0.48	-0.25
10.05	2.53	2.81	9.31	9.17	0.37	0.50

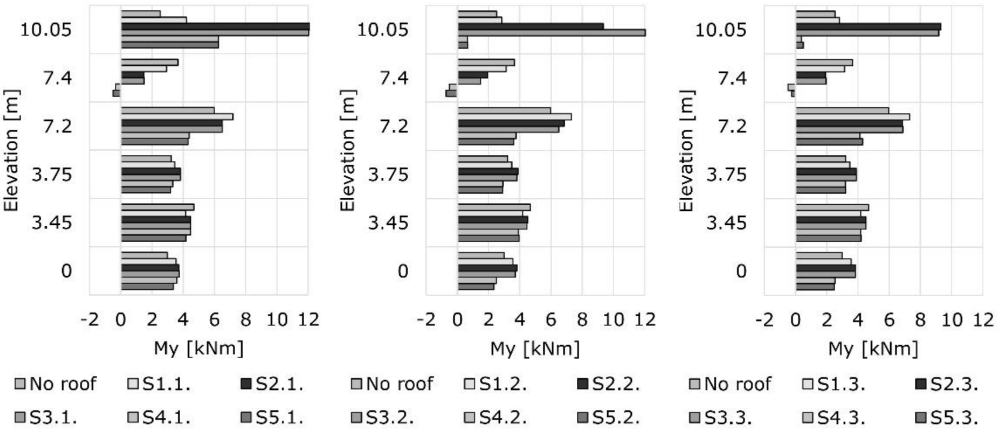


Fig. 4.44 Out-of-plane bending forces diagrams analysis of the scenarios - building with the second roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

Finally, considering the third roof structure also limited variation of the obtained values was observed for all the considered scenarios (Table 4-103, Fig. 4.45).

At the base of the first floor of the building, all the considered scenarios are presenting a slight decrease of the bending moment of up to 10%, only scenarios S2.1, S3.1, S4.1, S2.2, S2.3 and S4.3 are presenting an insignificant increase of the values of up to 3%. The variations at this level are rather low, reaching up to 10%. At the top of this floor, all the scenarios are presenting an increase of the values of 5% (10% in the case of the S2.1 scenario), with only little variations between the obtained bending forces.

At the second floor an increase of the bending moments was observed, both at its base, of 15 up to 20%, and top, of 15 up to 25%. The considered scenarios present only little differences.

The effect of the roof structure is visible on the top floor of the building. At the bottom of the floor, all the scenarios are presenting a slight decrease in the bending moments compared to the no roof structure case, of up to 30%. Scenarios S1.1, S1.2 and S1.3 are presenting the lowest decrease of the bending moments of 10%. At the top, on the other hand, all the scenarios involving rigid joints are showing a 50% increase (60% in the case of S1.1) of the bending moments, while the others are presenting an increase of up to 30%. Scenarios S5.2 and S5.3 are presenting the lowest increase of only 2.5%.

The significant variations between the bending forces recorded at the top of the building, highlight the importance of properly considering the support type and the joint axial stiffness, once again.

Table 4-103 Out-of-plane bending forces analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements

Elevation [m]	Without roof	With third roof structure				
		S1.1.	S2.1.	S3.1.	S4.1.	S5.1.
0	2.99	2.81	3.10	3.00	3.06	2.88
3.45	4.68	4.92	5.04	4.93	5.02	4.76
3.75	3.22	3.71	3.90	3.83	3.88	3.77
7.2	5.97	7.42	6.65	6.83	6.64	6.96
7.4	3.66	3.16	2.46	2.68	2.36	2.87
10.05	2.53	3.98	3.44	3.03	3.37	2.77
		S1.2.	S2.2.	S3.2.	S4.2.	S5.2.
0	2.99	2.67	3.02	2.91	2.99	2.77
3.45	4.68	4.82	4.97	4.86	4.95	4.72
3.75	3.22	3.64	3.84	3.76	3.82	3.69
7.2	5.97	7.58	6.82	6.96	7.00	6.97
7.4	3.66	3.26	2.53	2.75	2.71	2.82
10.05	2.53	3.81	3.26	2.86	3.16	2.59
		S1.3.	S2.3.	S3.3.	S4.3.	S5.3.
0	2.99	2.71	3.04	2.92	3.01	2.78
3.45	4.68	4.86	4.99	4.88	4.97	4.73
3.75	3.22	3.67	3.85	3.77	3.84	3.70
7.2	5.97	7.52	6.77	6.92	6.95	6.94
7.4	3.66	3.18	2.48	2.71	2.66	2.79
10.05	2.53	3.84	3.28	2.87	3.18	2.59

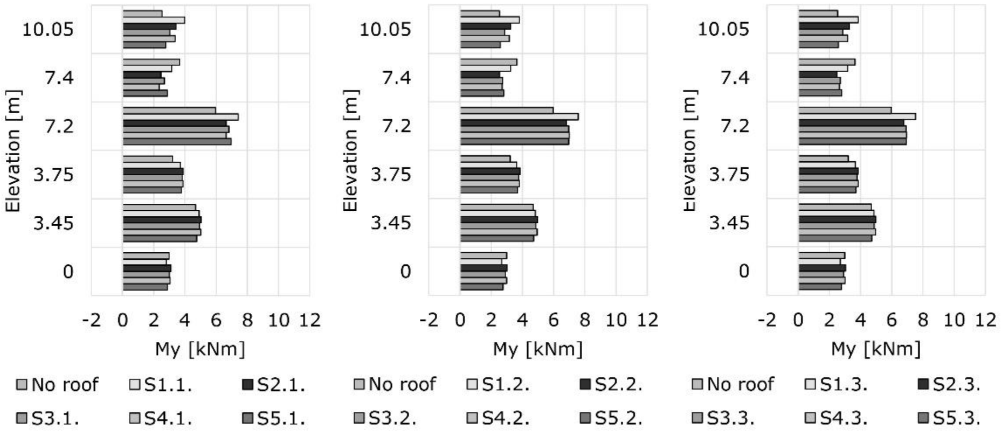


Fig. 4.45 Out-of-plane bending forces diagrams analysis of the scenarios - building with the third roof structure with reduced cross-section timber elements (comparison to the no roof structure case)

While comparing the out-of-plane bending moments on the historic masonry building without and with the three decayed chosen roof structures, the following was observed (Table 4-104):

1. The highest bending moments were identified:
    1.1. at the top of the second floor, in the area of the first timber beam flooring for scenarios involving sliding supports and hinged-sliding supports and at the top of the last floor for rigid support scenarios, for the first roof structure;
    1.2. at the top of the second floor, for rigid joint and component method and Heimeshoff and Köhler determined joint scenarios and at the top of the last floor for all the other scenarios in the case of the second roof structure;
    1.3. at the top of the second floor, in the area of the first timber beam flooring for the third roof structure.
2. Significant variations of the bending moment values between the assessed scenarios starting with the top of the second floor.
3. Top of the second floor:
    3.1. Decrease of the bending moment values for the first roof structure;
    3.2. Decrease/increase of the bending moment values for the second roof structure depending on the considered scenario;
    3.3. Increase of the bending moment values for the third roof structure.
4. Bottom of the top floor:
    4.1. Decrease of the bending moment values for the first structure, values close to 0kN;
    4.2. Decrease of the bending moment values for the second and third roof structure.
5. Top of the last floor:
    5.1. Increase of the bending moment values for rigid support scenarios and a slight decrease for all the other scenarios in the case of the first roof structure;
    5.2. Significant increase of the bending moment values for the second roof structure, slight decrease for scenarios S4.2, S5.2, S4.3 and S5.3;
    5.3. Slight increase of the bending moment values for the third roof structure.

Table 4-104 Comparison of the out-of-plane bending forces on the historic masonry walls of the building without and with the three roof structures with reduced cross-section timber elements

		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3		Roof 1	Roof 2	Roof 3
0	S1.1	3.17	3.54	2.81	S1.2	2.57	3.55	2.67	S1.3	2.24	3.56	2.71
3.45		4.45	4.16	4.92		3.78	4.18	4.82		3.96	4.17	4.86
3.75		3.18	3.46	3.71		2.76	3.48	3.64		2.87	3.47	3.67
7.2		4.87	7.19	7.42		4.77	7.30	7.58		4.46	7.31	7.52
7.4		0.05	2.93	3.16		0.54	3.14	3.26		-0.06	3.14	3.18
10.05		8.11	4.21	3.98		2.07	2.86	3.81		1.81	2.81	3.84
0	S2.1	3.65	3.72	3.10	S2.3	2.85	3.82	3.02	S2.3	2.70	3.84	3.04
3.45		4.53	4.47	5.04		3.92	4.52	4.97		4.15	4.51	4.99
3.75		3.36	3.82	3.90		3.00	3.89	3.84		3.22	3.89	3.85
7.2		4.38	6.50	6.65		4.24	6.85	6.82		4.64	6.85	6.77
7.4		-0.54	1.48	2.46		-0.37	1.94	2.53		-0.11	1.92	2.48
10.05		7.59	12.13	3.44		1.97	9.35	3.26		1.76	9.31	3.28
0	S3.1	3.68	3.73	3.00	S3.2	2.77	3.72	2.91	S3.3	2.70	3.84	2.92
3.45		4.51	4.47	4.93		3.93	4.47	4.86		4.18	4.51	4.88
3.75		3.37	3.82	3.83		3.01	3.82	3.76		3.25	3.89	3.77
7.2		4.32	6.51	6.83		4.22	6.49	6.96		4.65	6.87	6.92
7.4		-0.60	1.50	2.68		-0.35	1.49	2.75		-0.09	1.95	2.71
10.05		7.61	12.04	3.03		2.04	12.04	2.86		1.81	9.17	2.87
0	S4.1	3.60	3.60	3.06	S4.2	2.50	2.50	2.99	S4.3	2.54	2.54	3.01
3.45		4.47	4.47	5.02		3.91	3.91	4.95		4.18	4.18	4.97
3.75		3.33	3.33	3.88		2.92	2.92	3.82		3.20	3.20	3.84
7.2		4.39	4.39	6.64		3.77	3.77	7.00		4.13	4.13	6.95
7.4		-0.33	-0.33	2.36		-0.51	-0.51	2.71		-0.48	-0.48	2.66
10.05		6.25	6.25	3.37		0.65	0.65	3.16		0.36	0.37	3.18
0	S5.1	3.52	3.36	2.88	S5.2	2.37	2.35	2.77	S5.3	2.52	2.48	2.78
3.45		4.37	4.18	4.76		3.95	3.96	4.72		4.20	4.19	4.73
3.75		3.27	3.20	3.77		2.91	2.90	3.69		3.21	3.21	3.70
7.2		4.39	4.31	6.96		3.66	3.62	6.97		4.27	4.30	6.94
7.4		-0.33	-0.49	2.87		-0.66	-0.72	2.82		-0.30	-0.25	2.79
10.05		6.28	6.26	2.77		0.68	0.67	2.59		0.47	0.50	2.59

4.3.5.4 Comparison

Subsequently, in order to understand the influence of the decay of the roof structure on the behaviour of the building during seismic events and the effect on the internal forces recorded on the historic masonry walls, the results obtained for the building with the roof structures with complete cross-section timber elements were compared with the ones obtained for the decayed timber elements (Table 4-105, Table 4-106, Table 4-107, Table 4-108, Table 4-109).

For the first roof structure, the analysis showed that:

1. For the axial forces:
   1.1. The influence of the roof structure is visible even from the base of the building, its decay reducing the compressive axial forces up until the base of the top floor. The reductions vary between 10 and 25%. Still, scenario S1.2 is presenting a slight increase of the compression at the base of the structure of 5%, while scenarios S2.2, S3.2, S4.2 and S5.2 are presenting only an insignificant difference between the two considered cases;

   1.2. At the top of the second floor, all the scenarios are presenting a 10 up to 25% decrease of the compressive axial forces for all the considered scenarios;
   1.3. At the base of the top floor, both cases present a reduction of the compressive axial forces compared to the no roof structure case. Still, in the case of the decayed roof structure, the compressive axial forces are lower than in the complete cross-section case, the decay reducing the forces from 10%, at the S1.2 scenario up to 85% at the S1.1 scenario;
   1.4. At the top of the last floor, both cases are also presenting tensile axial forces. Still, the obtained tensile axial forces in the case of the decayed roof structure are up to 10 times higher. The scenarios S3.1 and S4.1 are presenting no differences between the two cases, while the scenario S1.2 is presenting the most significant increase.
2. For the shear forces:
   2.1. In both cases, the building is presenting shear forces in the opposite direction in the area of the cross-vault;
   2.2. In both cases the shear forces on the first floor and the base of the second floor of the building when comparing the chosen joint types, but with the same roof support, are approximately similar. Still, in the case of the decayed roof structure, the shear forces are about 10 up to 15% higher than in the case of the not decayed roof;
   2.3. On the top floor, both cases present an increase of the shear forces. At the same time, it was observed that the effect of the decay of the roof structure on the shear forces on this floor is variable and significantly influenced by the considered combination of support and joint type. Therefore, scenarios involving sliding joints are presenting a slight decrease of the shear forces of up to 10%, while all the other scenarios are presenting a slight increase of the forces of up to 5%. Scenario S3.1, S4.1 and S5.1 are the only ones which are showing no differences between the two assessed cases.
3. For the out-of-plane bending moments:
   3.1. Both cases present significant variations of the bending moment values between the assessed scenarios starting with the top of the second floor;
   3.2. Both cases present the highest bending moments at the top of the second floor, in the area of the first timber beam flooring for scenarios involving sliding supports and hinged-sliding supports and at the top of the last floor for rigid support scenarios;
   3.3. The effect of the decay of the roof structure is visible on all the floors of the building. Therefore, at the base of the second floor, the bending moments are up to 30% higher in the case of the decayed roof structure;
   3.4. Starting with the top of the second floor, the decayed roof structure is causing less bending forces, of up to 20%. Still, both cases present a decrease of the bending forces compared to the no roof structure case;
   3.5. At the bottom of the top floor, both cases present a decrease of the bending moment values, with values close to 0kN. Also, in this case, the decayed roof structure is causing for more scenarios, negative values of the bending forces, than the complete cross-section case;
   3.6. At the top of the last floor, in both cases, the rigid support scenarios are presenting an increase of the bending moment values, compared to the no roof structure case, while the other scenarios are presenting a decrease of the

bending forces. At this point, the decayed roof structure is causing up to 20% less bending forces than the complete roof structure case.

Table 4-105 Comparison of the internal forces on the historic masonry walls of the building with the first roof structure with complete (C.) and reduced (R.) cross-section timber elements for rigid joint scenarios

	Elev.	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
	[m]	C.	R.	%	C.	R.	%	C.	R.	%
S1.1	0	-118.2	-105.8	10	3.07	3.42	10	1.74	3.17	80
	3.45	-70.52	-58.12	20	3.07	3.42	10	3.99	4.45	10
	3.75	-68.35	-57.02	15	-1.7	-1.37	20	2.48	3.18	30
	7.2	-32.02	-24.36	25	2.17	2.53	15	4.8	4.87	0
	7.4	-8.86	-1.23	85	3.54	3.69	5	0.31	0.05	-85
	10.05	12.99	20.62	60	3.54	3.69	5	7.71	8.11	5
S1.2	0	-104.8	-108.8	-5	3.33	3.13	-5	3.29	2.57	-20
	3.45	-57.13	-61.16	-5	3.33	3.13	-5	3.65	3.78	5
	3.75	-58	-59.88	-5	-1.36	-1.54	-15	2.83	2.76	0
	7.2	-29.3	-27.01	10	2.91	2.41	-15	5.84	4.77	-20
	7.4	-22.75	-9.83	55	2.48	2.24	-10	1.45	0.54	-65
	10.05	-0.9	12.02	1435	2.48	2.24	-10	2.3	2.07	-10
S1.3	0	-114.9	-111.1	5	3.08	3.02	0	2.21	2.24	0
	3.45	-67.27	-63.44	5	3.08	3.02	0	3.59	3.96	10
	3.75	-65.17	-61	5	-1.57	-1.55	0	2.47	2.87	15
	7.2	-31.85	-27.23	15	2.26	2.35	5	4.66	4.46	-5
	7.4	-19.64	-14.57	25	2.02	2.08	5	0.37	-0.06	-115
	10.05	2.21	7.29	230	2.02	2.08	5	2.39	1.81	-25

Table 4-106 Comparison of the internal forces on the historic masonry walls of the building with the first roof structure with complete (C.) and reduced (R.) cross-section timber elements for hinged joint scenarios

	Elev.	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
	[m]	C.	R.	%	C.	R.	%	C.	R.	%
S2.1	0	-109.2	-100.3	10	3.38	3.53	5	2.95	3.65	25
	3.45	-61.54	-52.68	15	3.38	3.53	5	4.31	4.53	5
	3.75	-60.31	-52.63	15	-1.44	-1.32	10	3	3.36	10
	7.2	-26.97	-21.72	20	2.52	2.46	0	5.03	4.38	-15
	7.4	-1.35	-0.86	35	3.64	3.74	5	0.08	-0.54	-775
	10.05	20.5	20.99	0	3.64	3.74	5	8.04	7.59	-5
S2.2	0	-104.0	-104.1	0	3.35	3.16	-5	3.35	2.85	-15
	3.45	-56.38	-56.43	0	3.35	3.16	-5	3.83	3.92	0
	3.75	-57.34	-56.05	0	-1.35	-1.49	-10	2.92	3	5
	7.2	-28.67	-25.17	10	2.69	2.35	-15	5.14	4.24	-20
	7.4	-21.65	-14.36	35	2.4	2.22	-10	0.54	-0.37	-170
	10.05	0.2	7.49	3645	2.4	2.22	-10	2.35	1.97	-15
S2.3	0	-113.7	-105.6	5	3.03	3.07	0	2.26	2.7	20
	3.45	-66.02	-57.95	10	3.03	3.07	0	3.79	4.15	10
	3.75	-63.33	-56.85	10	-1.49	-1.53	-5	2.77	3.22	15
	7.2	-29.72	-25.37	15	2.4	2.57	5	4.6	4.64	0
	7.4	-19.26	-16.14	15	2.09	2.24	5	-0.09	-0.11	-20
	10.05	2.59	5.71	120	2.09	2.24	5	2.26	1.76	-20

Table 4-107 Comparison of the internal forces on the historic masonry walls of the building with the first roof structure with complete (C.) and reduced (R.) cross-section timber elements for Hölzer determined joint scenarios

	Elev. [m]	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
		C.	R.	%	C.	R.	%	C.	R.	%
S3.1	0	-104.78	-99.71	5	3.45	3.53	0	3.3	3.68	10
	3.45	-57.1	-52.03	10	3.45	3.53	0	4.33	4.51	5
	3.75	-56.79	-52.07	10	-1.4	-1.32	5	3.13	3.37	10
	7.2	-24.92	-21.33	15	2.54	2.45	-5	4.84	4.32	-10
	7.4	-1.57	-1.3	15	3.66	3.75	0	-0.19	-0.6	-215
	10.05	20.28	20.55	0	3.66	3.75	0	8.01	7.61	-5
S3.2	0	-103.85	-104.25	0	3.33	3.13	-5	3.28	2.77	-15
	3.45	-56.18	-56.57	0	3.33	3.13	-5	3.86	3.93	0
	3.75	-57.03	-55.99	0	-1.38	-1.52	-10	2.94	3.01	0
	7.2	-28.06	-24.58	10	2.64	2.33	-10	5	4.22	-15
	7.4	-19.6	-12.18	40	2.32	2.18	-5	0.37	-0.35	-195
	10.05	2.25	9.68	330	2.32	2.18	-5	2.46	2.04	-15
S3.3	0	-111.26	-105.56	5	3.06	3.06	0	2.45	2.7	10
	3.45	-63.58	-57.88	10	3.06	3.06	0	3.91	4.18	5
	3.75	-61.49	-56.82	10	-1.48	-1.54	-5	2.93	3.25	10
	7.2	-29.03	-25.38	15	2.51	2.58	5	4.73	4.65	0
	7.4	-19.39	-16.15	15	2.1	2.23	5	-0.04	-0.09	-125
	10.05	2.47	5.7	130	2.1	2.23	5	2.33	1.81	-20

Table 4-108 Comparison of the internal forces on the historic masonry walls of the building with the first roof structure with complete (C.) and reduced (R.) cross-section timber elements for component method determined joint scenarios

	Elev. [m]	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
		C.	R.	%	C.	R.	%	C.	R.	%
S4.1	0	-108.32	-100.42	5	3.41	3.5	5	3.07	3.6	15
	3.45	-60.65	-52.74	15	3.41	3.5	5	4.34	4.47	5
	3.75	-59.56	-52.63	10	-1.42	-1.34	5	3.04	3.33	10
	7.2	-26.47	-21.7	20	2.54	2.45	-5	5.03	4.39	-15
	7.4	0.52	0.45	-15	3.44	3.46	0	0.28	-0.33	-220
	10.05	22.37	22.31	0	3.44	3.46	0	6.49	6.25	-5
S4.2	0	-105.28	-107.26	0	3.25	3.06	-5	3.09	2.5	-20
	3.45	-57.61	-59.58	-5	3.25	3.06	-5	3.76	3.91	5
	3.75	-58.09	-58.44	0	-1.43	-1.6	-10	2.88	2.92	0
	7.2	-28.09	-25.7	10	2.54	2.09	-20	4.85	3.77	-20
	7.4	-14.45	-7.58	50	2.26	2.07	-10	0.45	-0.51	-215
	10.05	7.4	14.27	95	2.26	2.07	-10	0.42	0.65	55
S4.3	0	-122.84	-107.23	15	2.77	3.02	10	1.12	2.54	125
	3.45	-75.16	-59.55	20	2.77	3.02	10	3.79	4.18	10
	3.75	-71.92	-58.31	20	-1.87	-1.59	15	2.37	3.2	35
	7.2	-35.11	-26.49	25	1.84	2.34	25	3.73	4.13	10
	7.4	-19.01	-14.1	25	1.75	2.03	15	-0.46	-0.48	-5
	10.05	2.85	7.75	170	1.75	2.03	15	0.06	0.36	500

Table 4-109 Comparison of the internal forces on the historic masonry walls of the building with the first roof structure with complete (C.) and reduced (R.) cross-section timber elements for Heimeshoff & Köhler method determined joint scenarios

	Elev.	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
	[m]	C.	R.	%	C.	R.	%	C.	R.	%
S5.1	0	-106.21	-100.62	5	3.38	3.45	0	3.17	3.52	10
	3.45	-58.53	-52.95	10	3.38	3.45	0	4.21	4.37	5
	3.75	-58.15	-53.01	10	-1.38	-1.34	5	3.03	3.27	10
	7.2	-26.73	-22.59	15	2.58	2.46	-5	4.96	4.39	-10
	7.4	-3.09	-2.28	25	3.31	3.37	0	0.18	-0.33	-285
	10.05	18.76	19.57	5	3.31	3.37	0	6.42	6.28	0
S5.2	0	-106.19	-107.75	0	3.2	3.03	-5	2.89	2.37	-20
	3.45	-58.51	-60.07	-5	3.2	3.03	-5	3.8	3.95	5
	3.75	-58.43	-58.72	0	-1.49	-1.62	-10	2.88	2.91	0
	7.2	-27.45	-25.86	5	2.51	2.04	-20	4.81	3.66	-25
	7.4	-11.26	-8.39	25	2.21	2	-10	0.45	-0.66	-245
	10.05	10.59	13.46	25	2.21	2	-10	0.58	0.68	15
S5.3	0	-117.39	-107.07	10	2.91	3.01	5	1.8	2.52	40
	3.45	-69.72	-59.4	15	2.91	3.01	5	3.87	4.2	10
	3.75	-65.6	-58.03	10	-1.57	-1.61	-5	2.71	3.21	20
	7.2	-29.53	-25.97	10	2.19	2.39	10	4.24	4.27	0
	7.4	-14.51	-13.09	10	1.93	2.05	5	-0.17	-0.3	-75
	10.05	7.34	8.76	20	1.93	2.05	5	0.33	0.47	40

For the second roof structure, the analysis showed that (Table 4-110, Table 4-111, Table 4-112, Table 4-113, Table 4-114):

1. For the axial forces:
    1.1. The influence of the roof structure is visible even from the base of the building, its decay reducing the compressive axial forces up until the bottom of the second floor. The reductions vary between 5 and 15%. Only scenarios S5.2 and S5.3 are presenting a slight increase of the compression of 5% at the base and 10% at the top of the first floor;
    1.2. At the top of the second floor, all the scenarios are presenting an about 10% decrease of the compressive axial forces for all the considered scenarios, except for the S4.1 scenario which is presenting a 30% decrease and S5.1 which is presenting a 25% decrease;
    1.3. At the base of the top floor, both cases present an increase of the compressive axial forces compared to the no roof structure case, except for the scenarios involving semi-rigid joints determined using the component method and the Heimeshoff and Köhler method, which are presenting a decrease of the compressive axial forces in the decayed roof structure case. Still, if comparing the two cases, the compressive axial forces are approximately similar for the rigid joint, hinged joints and Hölzer determined joint scenarios, while all the other scenarios are presenting a decrease of the compressive forces of up to 80% in the case of the decayed roof structure;
    1.4. At the top of the last floor, both cases present compressive axial forces, except for the scenarios involving semi-rigid joints determined using the component method and the Heimeshoff and Köhler method for the decayed roof structure case, which are presenting tensile axial forces. Still, the obtained

compressive axial forces in the case of the decayed roof structure are up to 10% higher.

2. For the shear forces:
    2.1. In both cases, the building is presenting shear forces in the opposite direction in the area of the cross-vault;
    2.2. In both cases the shear forces on the first floor of the building when comparing the chosen joint types, but same roof support are approximately similar, presenting only a 5% increase for the S4.1 scenario and a 10% decrease in the case of the S4.2, S4.3, S5.2 and S5.3 scenarios;
    2.3. On the top floor, both cases present a decrease of the shear forces for rigid joint scenarios and an increase for the other scenarios. At the same time, it was observed that the effect of the decay of the roof structure on the shear forces on this floor is variable and significantly influenced by the considered combination of support and joint type. Therefore, if comparing the two cases, scenarios involving rigid joints, hinged joints and Hölzer determined joints are presenting a 5 up to 10% increase of the shear forces, while all the other scenarios are presenting a more significant decrease of the forces of up to 50%.
3. For the out-of-plane bending moments:
    3.1. Both cases present significant variations of the bending moment values between the assessed scenarios starting with the top of the second floor;
    3.2. Both cases present the highest bending moments at the top of the second floor, for rigid support scenarios and at the top of the last floor for all the other scenarios;
    3.3. The effect of the decay of the roof structure is visible on all the floors of the building. Therefore, until the top of the first floor, the bending moments are up to 15% higher in the case of the decayed roof structure;
    3.4. Starting with the top of the second floor, the decayed roof structure is presenting similar bending forces in the case of rigid joint, hinged joint and Hölzer determined joints and of up to 50% decrease in the case of the other scenarios;
    3.5. At the bottom of the top floor, both cases present a decrease of the bending moment values compared to the no roof model. Also, in this case, the decayed roof structure is causing for more scenarios (S4.1, S5.1, S4.2, S5.2, S4.3 and S5.3) negative values of the bending forces, than the complete cross-section case. At this point, the scenarios involving rigid joints, hinged joint and Hölzer determined joints are presenting an up to 10% decrease of the bending forces while the other scenarios are presenting an up to 110% decrease;
    3.6. At the top of the last floor, in both cases, all the scenarios are presenting an increase of the bending moment values, compared to the no roof structure case. At this point, the decayed roof structure is causing up to 20% less bending forces in the case of rigid joint scenarios, up to 5% less in the case of hinged joints and Hölzer determined joints and up to 95% in the case of the other scenarios.

Table 4-110 Comparison of the internal forces on the historic masonry walls of the building with the second roof structure with complete (C.) and reduced (R.) cross-section timber elements for rigid joint scenarios

Elev. [m]	N [kN] C.	Comp		Vz [kN]	Comp		My [kNm]	Comp		Comp
		R.	%	C.	R.	%	C.	R.	%	%
S1.1	0	-119.96	-112.55	5	3.25	3.3	0	3	3.54	20
	3.45	-72.29	-64.87	10	3.25	3.3	0	3.82	4.16	10
	3.75	-71.93	-65.25	10	-1.61	-1.44	10	2.95	3.46	15
	7.2	-42.79	-38.82	10	3.31	3.6	10	7.03	7.19	0
	7.4	-35.28	-37.39	-5	1.37	1.5	10	2.98	2.93	0
	10.05	-13.42	-15.53	-15	1.37	1.5	10	4.62	4.21	-10
S1.2	0	-120.27	-112.87	5	3.24	3.29	0	3	3.55	20
	3.45	-72.59	-65.19	10	3.24	3.29	0	3.82	4.18	10
	3.75	-72.25	-65.59	10	-1.63	-1.45	10	2.95	3.48	20
	7.2	-43.24	-39.45	10	3.34	3.65	10	7.15	7.3	0
	7.4	-35.63	-38.69	-10	0.86	0.97	15	3.22	3.14	0
	10.05	-13.78	-16.83	-20	0.86	0.97	15	3.57	2.86	-20
S1.3	0	-120.24	-112.76	5	3.25	3.3	0	3.04	3.56	15
	3.45	-72.57	-65.09	10	3.25	3.3	0	3.82	4.17	10
	3.75	-72.21	-65.55	10	-1.61	-1.44	10	2.95	3.47	20
	7.2	-43.29	-39.53	10	3.36	3.65	10	7.19	7.31	0
	7.4	-35.8	-38.77	-10	0.86	0.97	15	3.24	3.14	-5
	10.05	-13.94	-16.92	-20	0.86	0.97	15	3.55	2.81	-20

Table 4-111 Comparison of the internal forces on the historic masonry walls of the building with the second roof structure with complete (C.) and reduced (R.) cross-section timber elements for hinged joint scenarios

	Elev. [m]	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
		C.	R.	%	C.	R.	%	C.	R.	%
S2.1	0	-107.17	-100.53	5	3.36	3.39	0	3.25	3.72	15
	3.45	-59.49	-52.85	10	3.36	3.39	0	4.12	4.47	10
	3.75	-59.39	-53.4	10	-1.43	-1.32	10	3.32	3.82	15
	7.2	-31.18	-27.22	15	3.13	3.37	10	6.5	6.5	0
	7.4	-33.95	-34.67	0	5.21	5.38	5	1.66	1.48	-10
	10.05	-12.09	-12.82	-5	5.21	5.38	5	12.69	12.13	-5
S2.2	0	-104.28	-98.47	5	3.37	3.39	0	3.44	3.82	10
	3.45	-56.6	-50.79	10	3.37	3.39	0	4.2	4.52	10
	3.75	-56.74	-51.41	10	-1.38	-1.29	5	3.45	3.89	15
	7.2	-29.29	-25.64	10	3.37	3.54	5	6.96	6.85	0
	7.4	-34.4	-35.17	0	4.17	4.34	5	2.21	1.94	-10
	10.05	-12.55	-13.32	-5	4.17	4.34	5	9.86	9.35	-5
S2.3	0	-104.12	-98.35	5	3.38	3.39	0	3.46	3.84	10
	3.45	-56.44	-50.67	10	3.38	3.39	0	4.2	4.51	5
	3.75	-56.65	-51.38	10	-1.38	-1.29	5	3.45	3.89	15
	7.2	-29.31	-25.73	10	3.37	3.54	5	6.96	6.85	0
	7.4	-34.41	-35.2	0	4.17	4.35	5	2.19	1.92	-10
	10.05	-12.56	-13.35	-5	4.17	4.35	5	9.82	9.31	-5

Table 4-112 Comparison of the internal forces on the historic masonry walls of the building with the second roof structure with complete (C.) and reduced (R.) cross-section timber elements for Hölzer determined joint scenarios

	Elev.	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
	[m]	C.	R.	%	C.	R.	%	C.	R.	%
S3.1	0	-106.97	-100.48	5	3.36	3.39	0	3.27	3.73	15
	3.45	-59.29	-52.8	10	3.36	3.39	0	4.13	4.47	10
	3.75	-59.22	-53.35	10	-1.43	-1.32	10	3.34	3.82	15
	7.2	-31.08	-27.19	15	3.14	3.37	5	6.52	6.51	0
	7.4	-34	-34.69	0	5.17	5.35	5	1.68	1.5	-10
	10.05	-12.15	-12.84	-5	5.17	5.35	5	12.57	12.04	-5
S3.2	0	-104.1	-100.7	5	3.37	3.38	0	3.46	3.72	10
	3.45	-56.43	-53.02	5	3.37	3.38	0	4.21	4.47	5
	3.75	-56.57	-53.55	5	-1.37	-1.32	5	3.46	3.82	10
	7.2	-29.17	-27.35	5	3.39	3.37	0	6.99	6.49	-5
	7.4	-34.45	-34.73	0	4.08	5.34	30	2.25	1.49	-35
	10.05	-12.6	-12.87	0	4.08	5.34	30	9.66	12.04	25
S3.3	0	-103.94	-98.3	5	3.38	3.39	0	3.47	3.84	10
	3.45	-56.26	-50.62	10	3.38	3.39	0	4.2	4.51	5
	3.75	-56.49	-51.34	10	-1.37	-1.29	5	3.46	3.89	10
	7.2	-29.2	-25.69	10	3.39	3.55	5	6.99	6.87	0
	7.4	-34.47	-35.22	0	4.09	4.29	5	2.23	1.95	-15
	10.05	-12.61	-13.37	-5	4.09	4.29	5	9.62	9.17	-5

Table 4-113 Comparison of the internal forces on the historic masonry walls of the building with the second roof structure with complete (C.) and reduced (R.) cross-section timber elements for component method determined joint scenarios

	Elev.	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
	[m]	C.	R.	%	C.	R.	%	C.	R.	%
S4.1	0	-107.08	-100.41	5	3.36	3.5	5	3.26	3.6	10
	3.45	-59.4	-52.73	10	3.36	3.5	5	4.13	4.47	10
	3.75	-59.31	-52.63	10	-1.43	-1.34	5	3.33	3.33	0
	7.2	-31.13	-21.69	30	3.13	2.45	-20	6.5	4.39	-30
	7.4	-33.97	0.44	100	5.21	3.46	-35	1.66	-0.33	-120
	10.05	-12.12	22.29	285	5.21	3.46	-35	12.68	6.25	-50
S4.2	0	-104.23	-107.26	-5	3.37	3.06	-10	3.45	2.5	-30
	3.45	-56.55	-59.58	-5	3.37	3.06	-10	4.21	3.91	-5
	3.75	-56.69	-58.44	-5	-1.38	-1.6	-15	3.46	2.92	-15
	7.2	-29.26	-25.7	10	3.37	2.09	-40	6.96	3.77	-45
	7.4	-34.42	-7.56	80	4.17	2.06	-50	2.2	-0.51	-125
	10.05	-12.57	14.29	215	4.17	2.06	-50	9.86	0.65	-95
S4.3	0	-104.07	-107.22	-5	3.38	3.02	-10	3.46	2.54	-25
	3.45	-56.39	-59.54	-5	3.38	3.02	-10	4.2	4.18	0
	3.75	-56.6	-58.3	-5	-1.37	-1.59	-15	3.46	3.2	-10
	7.2	-29.28	-26.48	10	3.38	2.34	-30	6.96	4.13	-40
	7.4	-34.44	-14.1	60	4.17	2.03	-50	2.19	-0.48	-120
	10.05	-12.58	7.75	160	4.17	2.03	-50	9.82	0.37	-95

Table 4-114 Comparison of the internal forces on the historic masonry walls of the building with the second roof structure with complete (C.) and reduced (R.) cross-section timber elements for Heimeshoff & Köhler method determined joint scenarios

	Elev.	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
	[m]	C.	R.	%	C.	R.	%	C.	R.	%
S5.1	0	-104.71	-100.73	5	3.39	3.35	0	3.54	3.36	-5
	3.45	-57.03	-53.05	5	3.39	3.35	0	4.25	4.18	0
	3.75	-57.36	-53.22	5	-1.36	-1.39	0	3.54	3.2	-10
	7.2	-30.3	-23.21	25	3.27	2.45	-25	6.55	4.31	-35
	7.4	-34.86	-4.19	90	5.23	3.17	-40	1.62	-0.49	-130
	10.05	-13.01	17.66	235	5.23	3.17	-40	12.18	6.26	-50
S5.2	0	-102.78	-108.13	-5	3.39	3.03	-10	3.64	2.35	-35
	3.45	-55.11	-60.45	-10	3.39	3.03	-10	4.29	3.96	-10
	3.75	-55.54	-59.08	-5	-1.34	-1.61	-20	3.6	2.9	-20
	7.2	-28.89	-26.32	10	3.45	2.03	-40	6.93	3.62	-50
	7.4	-35.18	-9.22	75	4.22	1.98	-55	2.12	-0.72	-135
	10.05	-13.33	12.63	195	4.22	1.98	-55	9.57	0.67	-95
S5.3	0	-102.6	-107.31	-5	3.4	3	-10	3.66	2.48	-30
	3.45	-54.92	-59.63	-10	3.4	3	-10	4.28	4.19	0
	3.75	-55.45	-58.18	-5	-1.33	-1.62	-20	3.59	3.21	-10
	7.2	-28.92	-25.95	10	3.45	2.4	-30	6.93	4.3	-40
	7.4	-35.21	-12.82	65	4.22	2.05	-50	2.1	-0.25	-110
	10.05	-13.36	9.03	170	4.22	2.05	-50	9.53	0.5	-95

For the third roof structure, the analysis showed that (Table 4-115, Table 4-116, Table 4-117, Table 4-118, Table 4-119):

1. For the axial forces:
    1.1. The influence of the roof structure is visible even from the base of the building, the decay of the roof structure reducing the axial forces up until its top;
    1.2. Both cases present only little differences between the considered scenarios up until the top of the second floor;
    1.3. The differences between the two cases start to appear even from the base of the structure where the presence of the complete cross-section roof structure is increasing the compressive axial forces, and the decayed roof structure is reducing them. Therefore, at this point, the decay of the roof is causing a 10% reduction of the compression for all the considered scenarios compared to the complete cross-section case;
    1.4. The top of the first floor presents in both cases a slight reduction of the compressive axial forces, the decayed roof structure still causing a 15% reduction of the compression compared to the complete cross-section case;
    1.5. On the second floor, both at the bottom and top, the complete cross-section roof structure is causing a slight increase of the compressive axial forces compared to the no roof case, while the decayed one is causing a slight decrease. At these points, the difference between the two cases is of 10-15%;
    1.6. At the base of the top floor, all the scenarios except the ones involving rigid joints in the case of the complete cross-section roof structure are presenting tensile axial forces. In the case of the decayed roof structure, the tensile axial forces are significantly higher than in the complete cross-section case. Ultimately, at the top of the building, all the scenarios are presenting tensile axial forces, the decayed roof structure causing a 15 up to 30% increase, with significantly high differences at the rigid joint scenarios.

2. For the shear forces:
    2.1. In both cases, the building is presenting shear forces in the opposite direction in the area of the cross-vault;
    2.2. On the first floor and the base of the second floor of the building, the decayed roof structure is causing a 10 up to 20% increase of the shear forces compared to the no roof structure case. At the same time, it was observed that the differences between the obtained values, when considering the same support type, vary significantly less in the decayed roof structure case than in the full cross-section one;
    2.3. Starting with the top of the second floor, both cases present an increase of the shear forces. At this points, significant differences were only observed in the case of the rigid joint scenarios, where the decay of the roof structure is causing an increase of the shear forces compared to the complete cross-section roof structure of 35 up to 45% at the top of the second floor and 25 up to 30% at the last floor, both base and top. The other scenarios are presenting two different effects. On the one hand, scenarios S2.1, S2.3, S4.1, S5.1, S5.2 and S5.3 are presenting a slight decrease of 5% in the case of the decayed roof structure. The other scenarios, on the other hand, are presenting no differences between the two considered cases, highlighting the fact that this roof structure type is instead influencing the behaviour of the building on the second floor rather than at the top.
3. For the out-of-plane bending moments:
    3.1. Both cases present significant variations of the bending moment values between the assessed scenarios starting with the top of the second floor;
    3.2. The two cases present the highest bending moments in the same point, on the second floor;
    3.3. The effect of the decay of the roof structure is visible on all the floors of the building, increasing or decreasing the bending forces depending on the floor and considered scenario;
    3.4. Until the base of the second floor, the bending moments are significantly higher in the case of the decayed roof structure. The increases vary significantly, on each floor, from 20%, at the base of the structure, for Heimeshoff and Köhler determined joint scenarios, up to 110%, for the S1.1 scenario, from 5 to 35% at the top of the first floor and from 20% to 80% at the bottom of the second floor. It can be observed that the differences are higher at the base of these floors than at their top;
    3.5. Starting with the top of the second floor, the decayed roof structure is influencing the behaviour of the building in two different ways. Scenarios S1.1, S1.2 and S1.3 are the only scenarios presenting an increase of the bending forces of 10 up to 15% in the case of the decayed roof structure, while all the other scenarios are presenting a 15% decrease of the bending forces;
    3.6. At the bottom of the top floor, both cases present a decrease of the bending moment values, compared to the top of the second floor. Also, at this level, the decayed roof structure is causing for all the considered scenarios, a decrease of the bending moments of 35 up to 45%, except for the rigid joint scenarios which are presenting a decrease of only 5 to 10%;
    3.7. At the top of the last floor, the complete cross-section roof structure is causing an increase of the bending moment values for rigid support scenarios and a slight decrease for all the other scenarios while the decayed roof structure is

causing a slight increase of the bending moment values for all the considered scenarios if compared to the no roof case. Despite these different behaviours, the bending forces in the case of the decayed roof structure are about 15 up to 55% higher than the ones caused by the presence of the full-cross-section roof.

Table 4-115 Comparison of the internal forces on the historic masonry walls of the building with the third roof structure with complete (C.) and reduced (R.) cross-section timber elements for rigid joint scenarios

	Elev. [m]	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
		C.	R.	%	C.	R.	%	C.	R.	%
S1.1	0	-115.58	-105.44	10	2.77	3.28	20	1.34	2.81	110
	3.45	-67.91	-57.77	15	2.77	3.28	20	3.85	4.92	30
	3.75	-65.11	-55.44	15	-1.75	-1.43	20	2.06	3.71	80
	7.2	-28.16	-22.26	20	2.31	3.39	45	6.57	7.42	15
	7.4	-19.37	12.65	165	2.34	3.04	30	3.36	3.16	-5
	10.05	2.48	34.5	1290	2.34	3.04	30	3.46	3.98	15
S1.2	0	-115.84	-106.05	10	2.83	3.2	15	1.57	2.67	70
	3.45	-68.16	-58.38	15	2.83	3.2	15	3.83	4.82	25
	3.75	-63.46	-55.93	10	-1.75	-1.48	15	2.72	3.64	35
	7.2	-27.41	-22.87	15	2.55	3.44	35	7.01	7.58	10
	7.4	-16.4	9.51	160	2.36	3	25	3.7	3.26	-10
	10.05	5.45	31.36	475	2.36	3	25	3.39	3.81	10
S1.3	0	-115.07	-106.08	10	2.81	3.23	15	1.47	2.71	85
	3.45	-67.39	-58.4	15	2.81	3.23	15	3.83	4.86	25
	3.75	-63.37	-55.98	10	-1.87	-1.47	20	2.69	3.67	35
	7.2	-28.28	-22.9	20	2.4	3.42	45	6.75	7.52	10
	7.4	-20.85	9.98	150	2.23	2.97	35	3.51	3.18	-10
	10.05	1.01	31.84	3050	2.23	2.97	35	3.36	3.84	15

Table 4-116 Comparison of the internal forces on the historic masonry walls of the building with the third roof structure with complete (C.) and reduced (R.) cross-section timber elements for hinged joint scenarios

	Elev. [m]	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
		C.	R.	%	C.	R.	%	C.	R.	%
S2.1	0	-111.69	-101.12	10	3	3.33	10	2	3.1	55
	3.45	-64.02	-53.44	15	3	3.33	10	4.31	5.04	15
	3.75	-60.84	-51.83	15	-1.61	-1.39	15	3.08	3.9	25
	7.2	-26.85	-19.75	25	3.22	3.24	0	7.89	6.65	-15
	7.4	2.4	16.13	570	2.54	2.44	-5	4.22	2.46	-40
	10.05	24.25	37.98	55	2.54	2.44	-5	2.2	3.44	55
S2.2	0	-112.66	-101.69	10	3.15	3.28	5	2.2	3.02	35
	3.45	-64.99	-54.01	15	3.15	3.28	5	4.54	4.97	10
	3.75	-61.14	-52.29	15	-1.61	-1.41	10	3.15	3.84	20
	7.2	-25.53	-20.16	20	3.26	3.28	0	8.05	6.82	-15
	7.4	7.8	14.3	85	2.35	2.34	0	4.31	2.53	-40
	10.05	29.66	36.15	20	2.35	2.34	0	2.28	3.26	45
S2.3	0	-112.9	-101.74	10	3.19	3.29	5	2.25	3.04	35
	3.45	-65.22	-54.07	15	3.19	3.29	5	4.59	4.99	10
	3.75	-61.38	-52.37	15	-1.58	-1.41	10	3.17	3.85	20
	7.2	-25.73	-20.26	20	3.26	3.26	0	8.05	6.77	-15
	7.4	8.33	14.25	70	2.37	2.31	-5	4.3	2.48	-40
	10.05	30.18	36.1	20	2.37	2.31	-5	2.32	3.28	40

Table 4-117 Comparison of the internal forces on the historic masonry walls of the building with the third roof structure with complete (C.) and reduced (R.) cross-section timber elements for Hölzer determined joint scenarios

	Elev. [m]	N [kN]		Comp %	Vz [kN]		Comp %	My [kNm]		Comp %
		C.	R.		C.	R.		C.	R.	
S3.1	0	-111.86	-101.39	10	3.19	3.27	5	2.31	3	30
	3.45	-64.18	-53.71	15	3.19	3.27	5	4.53	4.93	10
	3.75	-60.61	-51.99	15	-1.56	-1.41	10	3.15	3.83	20
	7.2	-25.39	-19.9	20	3.26	3.29	0	7.99	6.83	-15
	7.4	3.23	11.21	245	2.4	2.37	0	4.46	2.68	-40
	10.05	25.08	33.06	30	2.4	2.37	0	2.15	3.03	40
S3.2	0	-112.87	-102.21	10	3.16	3.22	0	2.21	2.91	30
	3.45	-65.19	-54.53	15	3.16	3.22	0	4.49	4.86	10
	3.75	-61.51	-52.68	15	-1.58	-1.44	10	3.1	3.76	20
	7.2	-26.07	-20.45	20	3.26	3.31	0	8.03	6.96	-15
	7.4	3.91	10.51	170	2.22	2.25	0	4.41	2.75	-40
	10.05	25.76	32.36	25	2.22	2.25	0	1.93	2.86	50
S3.3	0	-113	-102.2	10	3.17	3.23	0	2.23	2.92	30
	3.45	-65.33	-54.52	15	3.17	3.23	0	4.51	4.88	10
	3.75	-61.6	-52.69	15	-1.57	-1.43	10	3.11	3.77	20
	7.2	-26.14	-20.48	20	3.25	3.3	0	8.02	6.92	-15
	7.4	4.17	10.54	155	2.23	2.24	0	4.4	2.71	-40
	10.05	26.02	32.39	25	2.23	2.24	0	1.95	2.87	45

Table 4-118 Comparison of the internal forces on the historic masonry walls of the building with the third roof structure with complete (C.) and reduced (R.) cross-section timber elements for component method determined joint scenarios

	Elev. [m]	N [kN]		Comp %	Vz [kN]		Comp %	My [kNm]		Comp %
		C.	R.		C.	R.		C.	R.	
S4.1	0	-112.15	-101.34	10	2.85	3.31	15	1.78	3.06	70
	3.45	-64.47	-53.66	15	2.85	3.31	15	4.02	5.02	25
	3.75	-61.01	-51.93	15	-1.71	-1.39	20	2.97	3.88	30
	7.2	-26.73	-19.67	25	3.19	3.25	0	7.89	6.64	-15
	7.4	0.49	14.67	2895	2.51	2.43	-5	4.27	2.36	-45
	10.05	22.34	36.52	65	2.51	2.43	-5	2.11	3.37	60
S4.2	0	-112.73	-101.92	10	3.2	3.27	0	2.27	2.99	30
	3.45	-65.05	-54.24	15	3.2	3.27	0	4.55	4.95	10
	3.75	-61.32	-52.45	15	-1.57	-1.41	10	3.13	3.82	20
	7.2	-25.81	-20.25	20	3.24	3.34	5	8.09	7	-15
	7.4	5.73	13.89	140	2.32	2.39	5	4.43	2.71	-40
	10.05	27.58	35.75	30	2.32	2.39	5	2.18	3.16	45
S4.3	0	-112.91	-101.94	10	3.21	3.28	0	2.29	3.01	30
	3.45	-65.23	-54.26	15	3.21	3.28	0	4.57	4.97	10
	3.75	-61.54	-52.5	15	-1.55	-1.4	10	3.13	3.84	25
	7.2	-26.18	-20.32	20	3.24	3.32	0	8.08	6.95	-15
	7.4	5.56	13.94	150	2.34	2.37	0	4.42	2.66	-40
	10.05	27.41	35.79	30	2.34	2.37	0	2.16	3.18	45

Table 4-119 Comparison of the internal forces on the historic masonry walls of the building with the third roof structure with complete (C.) and reduced (R.) cross-section timber elements for Heimeshoff & Köhler method determined joint scenarios

	Elev.	N [kN]		Comp	Vz [kN]		Comp	My [kNm]		Comp
	[m]	C.	R.	%	C.	R.	%	C.	R.	%
S5.1	0	-109.88	-101.56	10	3.13	3.15	0	2.39	2.88	20
	3.45	-62.21	-53.88	15	3.13	3.15	0	4.46	4.76	5
	3.75	-59.14	-52.13	10	-1.53	-1.46	5	3.24	3.77	15
	7.2	-24.85	-20.27	20	3.48	3.36	-5	8.21	6.96	-15
	7.4	-0.04	6.06	15250	2.47	2.33	-5	4.54	2.87	-35
	10.05	21.82	27.91	30	2.47	2.33	-5	2.11	2.77	30
S5.2	0	-110.92	-102.58	10	3.11	3.12	0	2.28	2.77	20
	3.45	-63.24	-54.9	15	3.11	3.12	0	4.45	4.72	5
	3.75	-59.98	-52.96	10	-1.56	-1.49	5	3.21	3.69	15
	7.2	-25.32	-20.74	20	3.46	3.33	-5	8.21	6.97	-15
	7.4	1.88	6.24	230	2.27	2.15	-5	4.44	2.82	-35
	10.05	23.73	28.09	20	2.27	2.15	-5	1.87	2.59	40
S5.3	0	-111.18	-102.63	10	3.12	3.13	0	2.3	2.78	20
	3.45	-63.51	-54.96	15	3.12	3.13	0	4.47	4.73	5
	3.75	-60.17	-53.01	10	-1.55	-1.48	5	3.21	3.7	15
	7.2	-25.45	-20.78	20	3.45	3.32	-5	8.18	6.94	-15
	7.4	2.2	6.27	185	2.28	2.13	-5	4.42	2.79	-35
	10.05	24.05	28.12	15	2.28	2.13	-5	1.89	2.59	35

4.3.6 The connection between the out-of-plane horizontal displacement and out-of-plane bending moment

After analysing all the parameters and observing the significant effect of the historic timber roof structures on the behaviour of the masonry building during the seismic event, the connection between the top out-of-plane displacement and obtained out-of-plane bending moments was analysed in order to observe if the two are connected, and specific patterns can be identified.

As previously, first, the building without roof structure was analysed (Fig. 4.46). The results show that at the top of the building due to the missing roof structure, the out-of-plane displacement of the masonry wall is significant. On the contrary to this, if comparing the out-of-plane bending moments from the base of the top floor and its upper part, it can be observed that the bending forces are getting lower towards the top.

As previously stated in the case of the building without roof structure, the minimum bending moment was recorded at the top of the building, which is also the place where the maximum displacement was obtained. The maximum bending moment in the case of the building, which is presenting a flexural deformation is located at the top of the second floor.

Subsequently, the same analysis was also performed for the building with the three roof structures with complete cross-section timber elements (Fig. 4.47). The analysis showed that the structures which are causing a shear deformation of the building (roof structure one and two) with lower inter-story drifts at the top floor, are presenting an increase of the bending forces between the base and top of the last floor. On the contrary, the 3rd roof structure, which is preserving the flexural deformation previously obtained for the building with no roof, is presenting the same pattern of the bending moment with a decrease of the forces from the base towards the top of the last floor.

At the same time, it was observed that the third roof structure is also causing a significant increase of the bending moment at the top of the second floor, significantly higher than in the case of the other two structures.

It can be concluded that there is an important connection between the effect of the roof structures on the out-of-plane deformation of the building and the obtained out-of-plane bending moments.

Ultimately the same analysis was also performed for the building with the decayed roof structures. The same connection and pattern was observed as in the previous cases (Fig. 4.48). The structures which are causing a shear deformation of the building (roof structure one and two), are still presenting a significant increase of the bending forces between the bottom and the top of the last floor. At the same time, it can also be observed that in the case of the first roof structure where the top inter-story drift is only slightly lower than the one obtained at the second floor, the bending forces at the top are lower than in the case of the second roof structure where the shear deformation is visible. For the third roof structure, which is causing a flexural deformation of the masonry building, but the out-of-plane displacement is lower than in the not decayed case, the bending forces are presenting a slight increase from the bottom towards the top of the last floor.

At the top of the second floor, on the other hand, the bending forces recorded for the first two roof structures, are lower than the ones recorded for the third roof structure.

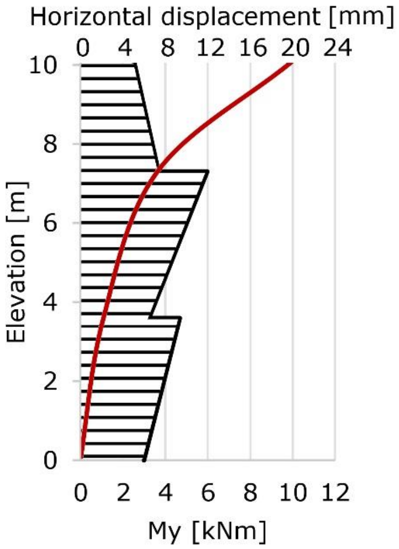


Fig. 4.46 Out-of-plane horizontal displacement and bending moment connection for the building with no roof structure

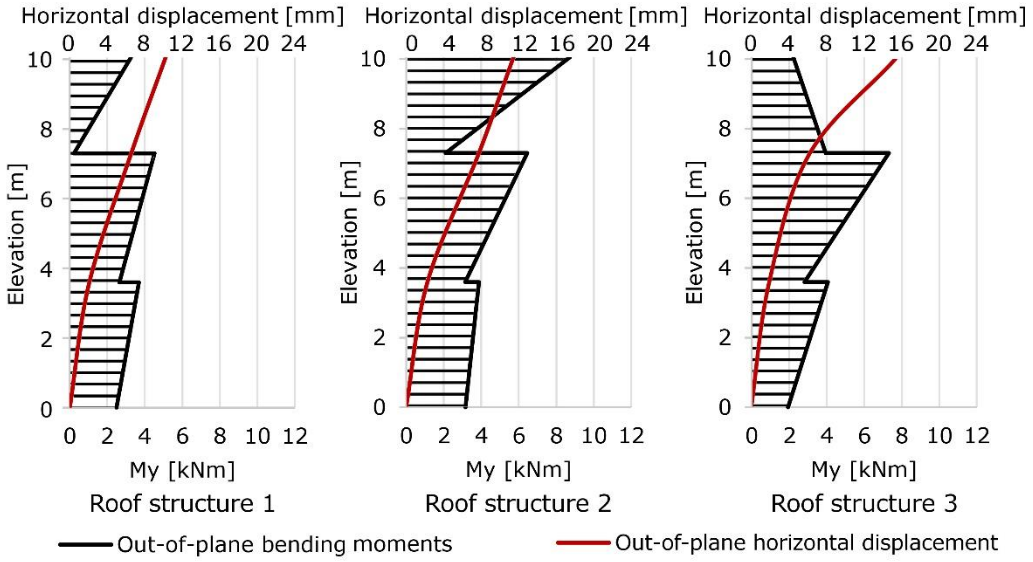


Fig. 4.47 Connection between the out-of-plane horizontal displacement and bending moment for the building with the three roof structures with a complete cross-section timber elements

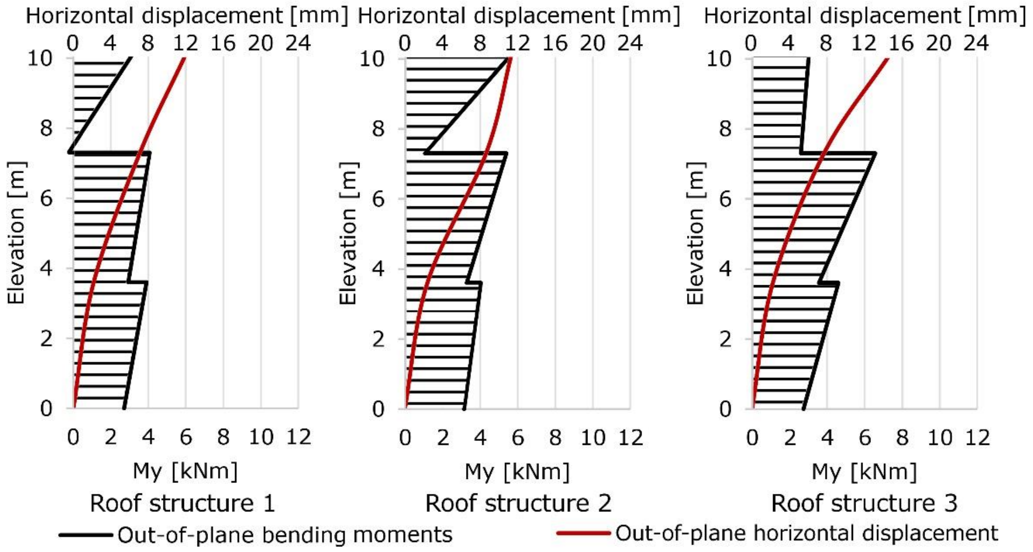


Fig. 4.48 Connection between the out-of-plane horizontal displacement and bending moment for the building with the three roof structures with a reduced cross-section timber elements

By analysing all the seven assessed cases, a clear connection between the out-of-plane horizontal displacement, inter-story drift, deformed shape and out-of-plane bending moments was observed, each roof structure causing a different behaviour of the building mainly at the upper part of the second floor and the top of the building.

4.4 Conclusion

Starting from previously performed calibrations, based on the laboratory tests performed at the University of Trento, the analysis of the influence of the three most common roof structure types from Timisoara, built in the 18th, 19th and beginning of the 20th century, on the seismic behaviour of an 18th century historic masonry building, brought forward that:

1. The presence of the roof structure in a good state of conservation is reducing the top horizontal displacement between 10 and 55% depending on the roof structure type, support of the roof structure and the joint axial stiffness:
    1.1. For the first roof structure with 40 up to 55%;
    1.2. For the second roof structure with 40 up to 45%;
    1.3. For the third roof structure with 10 up to 30%;
2. In the case of an up to 20% decayed roof structure, its effect on the seismic behaviour of the historic masonry building is slightly different, reducing the top horizontal displacement between:
    2.1. For the first roof structure with 25 up to 50%;
    2.2. For the second roof structure with 35 up to 45%;
    2.3. For the third roof structure with 25 up to 30%;
3. The decay of the roof structures is causing an increase of the horizontal displacements, compared to the no roof structure case:
    3.1. On the first and second floor of the building with up to 20% for the second roof structure;
    3.2. On the second floor of the building with up to 10% for the first and third roof structure.
4. Compared to the full cross-section case, the decay of the roof structures is causing:
    4.1. For the first roof structure, an increase of the horizontal displacement on all floors of up to 30%;
    4.2. For the second roof structure, an increase of the horizontal displacement of up to 15%, at the first and second floor and an up to 5% decrease on the top floor;
    4.3. For the third roof structure, an increase of the horizontal displacement of up to 25%, at the first and second floor and an up to 20% decrease on the top floor.
5. The presence of the complete cross-section roof structure is reducing the inter-story drift at the last floor between 5 and 85% depending on the roof structure type, support of the roof structure and the joint axial stiffness:
    5.1. For the first roof structure with 55 up to 85%;
    5.2. For the second roof structure with 60 up to 80%;
    5.3. For the third roof structure with 5 up to 40%.
6. In the case of a 20% decayed roof structure, its effect on the seismic behaviour of the historic masonry building is slightly different, reducing the inter-story drift at the last floor between:
    6.1. For the first roof structure with 35 up to 80%;
    6.2. For the second roof structure with 65 up to 85%;
    6.3. For the third complete cross-section roof structure with 40 up to 50%.
7. The decay of the roof structures is causing an increase of the inter-story drift, compared to the no roof structure case:

   7.1. On the first and second floor of the building with up to 25% for the second roof structure;
   7.2. On the second floor of the building with up to 10% for the first and third roof structure.
8. Compared to the full cross-section case, the decay of the roof structures is causing:
   8.1. For the first roof structure, an increase of the inter-story drift on all floors of up to 50%;
   8.2. For the second roof structure, an increase of the inter-story drift of up to 20%, at the first and second floor and an up to 40% decrease on the top floor;
   8.3. For the third roof structure, an increase of the horizontal displacement of up to 30%, at the first and second floor and an up to 40% decrease on the top floor.
9. Significantly reducing the damage level on all floors of the building.
10. Changing the deformed shape of the building from flexural to shear depending on the roof type and the support and joint stiffness.
11. The axial forces on the wall for the building with all the three roof structures are approximately similar on the first and second floor of the building, and the differences start to be observed above this threshold.
12. The bending moments present significant variations between the assessed scenarios for all the considered floors of the building, depending on the assessed scenario.
13. At the top floor of the building, depending on the roof structure type, with complete cross-section, the analysis of the internal forces showed that:
   13.1. Tensile or compressive axial forces can appear;
   13.2. Shear forces can suffer an increase, for 18th and 20th century roof structure, or decrease, for the 19th century roof, for certain scenarios;
   13.3. Bending moments can present and increase for the 19th century roof structure and increase or decrease based on the considered joint axial stiffness and support type for the 18th and 20th century roofs.
14. At the top floor of the building, depending on the roof structure type, with reduced cross-section, the analysis of the internal forces showed that:
   14.1. Tensile axial forces appear for all the considered roof structures;
   14.2. Shear forces suffer an increase for all the considered roof structures;
   14.3. Bending moments present an increase for 19th and 20th century roof structures and an increase or decrease based on the considered joint axial stiffness and support type for the 18th century roof.

Ultimately, the analysis highlights the importance of accurately determining the stiffness of the traditionally crafted joints, bringing forward that, depending on the used method to determine the axial stiffness or if they are considered rigid or hinged, the roof structure is influencing the behaviour of the historic masonry building during a seismic event in different ways.

4.5 Published research outcomes

The research outcomes presented in this chapter have been published in the following journals and conference proceedings:

1. A. Keller, N. Chieffo and M. Mosoarca, “Influence of roof structures on seismic behavior of historic buildings”, 3rd International Conference on Protection Of Historical Constructions, PROHITECH’17, Mazzolani, F., Lamas, A., Calado, L., Proenca, J., and Faggiano, B. eds., 2017
2. A. Keller, M. Mosoarca “Influence of roof structures on seismic vulnerability of historic buildings” in 5th International Conference on Structural Health Assessment of Timber Structures, SHATIS’19, Branco J., Sousa H. and Poletti E. eds, 171-178, 2019
3. N. Chieffo, I. Apostol, A. Keller, M. Mosoarca and A. Marzo, “Global behavior of historical historic masonry structures and timber roof framework”, 3rd International Conference on Protection Of Historical Constructions, PROHITECH’17, Mazzolani, F., Lamas, A., Calado, L., Proenca, J. and Faggiano, B. eds., 2017.
4. A. Keller, M.A. Parisi, E. Tsakanika, M. Mosoarca, “Influence of historic roof structures on the seismic behaviour of historic masonry structures”, Proceedings of the Institution of Civil Engineers - Structures and Buildings, https://doi.org/10.1680/jstbu.19.00098, 2019 (Impact factor – 0.877)

5 COMPLEX ASSESSMENT METHODOLOGY FOR HISTORIC ROOF STRUCTURES

The study of roofs, roof structures, their context and link to the building they belong to, has brought forward that they have a significant value, which is not always linked to their structural characteristics and the joint properties. The overall value and vulnerability of roof structures may be increased, or on the contrary decreased by their immediate context, the urban planning principles, architectural features or even symbolic factors. In addition to these features, the study brought forward that in order to accurately determine the structural vulnerability of historic roofs, their state of conservation and the effect of current and future climatic conditions should also be taken into account. Ultimately, as clearly highlighted in chapter 4, the studies also show that considering the roof structure type and its state of conservation, it may improve the seismic behaviour of the historic masonry building.

Consequently, based on the observations, a holistic procedure for a comprehensive assessment of historical roof structures was developed, based on a multi-, inter- and transdisciplinary assessment, taking all the factors surrounding roof structures into account and respecting in this way the ICOMOS and ISCARSAH principles.

5.1 Assessment levels

The assessment of features influencing the appearance of roofs and choice of roof structures brought forward that four main categories have to be taken into consideration when assessing their global value:

1. The urban value;
2. The architectural value;
3. The symbolic value;
4. The structural value.

At the same time, their vulnerability is highly influenced by environmental factors, leading to the decay of the timber elements, of the roof envelope material and the general appearance of the roof and building. Therefore, in order to determine the vulnerability of a roof structure, the following parameters should be taken into consideration:

1. The state of conservation of roof structures;
2. Climate change-induced damages.
3. The influence of the roof structure typology on the seismic behaviour of the building, based on the observations made during the numerical simulations.

The procedure was therefore classified into five assessment levels, each one of them organised in a tree-like structure. Each level contained criteria considered relevant for the assessment and a list of responses to choose from, for each criterion. Ultimately, to ensure the objectivity of the assessment, for each response of the procedure, a specific score was proposed. The score is one of the personal contributions of the research. It was determined based on the preliminary assessment of features influencing the value of a roof structure from all the point of views, considering the coherence of the context, value in the historic area or building, uniqueness of the structural elements or symbolic features.

Based on this proposed score, the procedure can automatically determine:

1. The urban value score of the roof structure;
2. The architectural value score of the roof structure;
3. The symbolic value score of the roof structure;
4. The structural value score of the roof structure;
5. The predominant value of the roof structure;
6. The ideal value of the roof structure;
7. The decay index of the roof structure;
8. The real value of the roof structure;
9. The climatic vulnerability of the roof structure;
10. The influence of the roof structure on the seismic behaviour of the building;
11. The vulnerability of the roof structure.

5.1.1 Urban value of historic roof structures

The literature analysis concerning the assessment of historic timber roof structures and roofs has brought forward that the influence of the surrounding urban area on the choice of roof shape is not taken into consideration. Still, the urban analysis of Timisoara highlighted that roof structures can also partially influence the way the public space is perceived, but also that the urban context defines their importance in the city (Table 5-1).

Therefore, based on the observations made during the analysis of the role of roofs in defining urban space, the assessment criteria were divided into three main categories:

C1. Value of the urban area
C2. Urban analysis
C3. Geometry of the roof

The value of the urban area criterion is mainly related to the heritage value of the context where the assessed roof structure is placed. If the surroundings are of no significant historical importance or the roof is placed in a position from which it cannot define the immediate urban space, the assessment procedure will consider that the roof does not have any value in its context. If however the building is placed near a heritage building, included in a protected urban area or has a significant role in defining the surrounding urban space, it would be considered of significant value from an urban point of view. The procedure is, therefore, offering four responses, with points based on the importance of the roof structure in its urban context:

- No valuable context / No defining role in the urban space;
- Protected area of an architectural monument;
- Protected urban area;
- Significant role in defining the urban space.

Therefore, if the roof structure is placed in a context without significant value or does not influence the surrounding urban space, the procedure will automatically assign 0 points for this first criterion. If the roof is placed in the protected area of a heritage building but has no significant influence on the urban context, the procedure will assign 1 point to the response, since the roof structure is still influencing the way the heritage building is perceived. If however, the building is placed in a protected urban area without actually being a building of significant importance or if the roof has no meaningful effect on the surrounding urban space, the procedure will assign 2 points to the chosen response. Ultimately, if the roof is of significant importance or is defining the urban space, the procedure will assign 4 points, the maximum amount which can be obtained for this criterion. In this case, no choice with 3 points was offered since there is a significant value difference from an urban planning point of view between a roof which is just place in a protected urban area and one which is defining urban space. Therefore, a clear differentiation between the choices was considered to be suitable in this situation.

The second category related to the urban analysis considers four different criteria, meant to define the position of the building and highlight how a pedestrian can perceive the roof:

- Position of the building;
- Frontage;
- Height;
- Alignment.

The first criterion which is taken into consideration is related to the position of the building in its urban context. The procedure is considering how the building and roof are perceived from the pedestrian area and if it is marking any particular feature of the surrounding urban space. Four possible responses are offered for this criterion:

- Independent building;
- Integrated into the urban alignment;
- Marks the urban silhouette;
- Marks an essential urban point.

Therefore, if the building is independent, without any connection, from a visual or aesthetical point of view to its context, or is not an important landmark in the city, then the procedure will assign only one point for the response since the building is not significantly influencing the urban context. If, however, the building is connected to its surrounding, being integrated into an urban alignment, the response will obtain 2 points. Ultimately, if the building and its roof are marking the urban silhouette or are marking an essential point in the urban space, the response will receive 3, respectively 4 points since the roof structure is defining the surrounding urban space.

Subsequently, the relationship between the assessed building and its immediate context is taken into consideration, highlighting if clear urban planning principles define the urban area or if any rules did not define its development. First, the frontage of the building is analysed, offering only two possible responses to choose from (discontinuous or continuous frontage). The lack of clear rules, leading to a discontinuous frontage, brings no additional points to the urban value assessment, while the clear identification of urban planning principles is rewarded with 2 points. The same rule also applies to the next criterion, "Height", where the identification of a constant height of the buildings and therefore a clearly defined aesthetics of the area is bringing 2 points to the urban value score while the variable height is bringing none.

Ultimately, the alignment criterion considers the withdrawal from the street alignment, and offers four possible responses to choose from:

- The building does not comply with the defined alignment;
- Alignment is withdrawn from the property limit;
- Alignment is withdrawn from the road;
- Street alignment.

The first possible response is once again related to the lack of clear urban planning rules and brings no additional points to the urban value. Subsequently, the other points are awarded based on observations made in the historic area of Timisoara. Since buildings placed close to the street are common in the city centre and part of a protected urban area, this response receives the maximum amount of 4 points. Alignments which are withdrawn from the street or the property limit are common for later urban developments and can obtain therefore three respectively 2 points. Due to the significant difference between no rules and a clearly defined urban pattern, no 1 point was offered for this criterion.

Table 5-1 Urban value assessment criteria and corresponding points for each response

C1. Value of the urban area		
1. Value of the urban area	No valuable context / No defining role in the urban space	0
	Protected area of an architectural monument	1
	Protected urban area	2
	Significant role in defining the urban space	4
C2. Urban analysis		
2. Position of the building	Independent building	1
	Integrated into urban alignment	2
	Marks the urban silhouette	3
	Marks an essential urban point	4
3. Frontage	Discontinuous front	0
	Continuous front	2
4. Height	Variable Height of the cornice	0
	Constant Height of the cornice	2
5. Alignment	The building does not comply with the defined alignment	0
	Alignment is withdrawn from the property limit	2
	Alignment is withdrawn from the road	3
	Street alignment	4
C3. Geometry		
6. Roof shape	Shed or lean-to roof	1
	Gable roof	2
	Hipped roof	2
	Jerkinhead roof	3
	Gambrel roof	3
	Pyramid Hip roof	3
	Mix of shapes	4
7. Roof pitch	<15°	1
	15°-30°	2
	30°-45°	3
	45°-65°	4

The last category of criteria is related to the geometry of the building and how its shape is influencing the way the pedestrian perceives the roof.

The first criterion is related to the used roof shape. The analysis of commonly used roof shapes in Timisoara (Fig. 2.16) brought forward that shed roofs were generally used as coverings for annexe buildings and placed towards the inner courtyard. Therefore, they are invisible from the street level and can obtain only one point. Gable and hipped roofs were mostly used in the city centre and are more challenging to see, due to the narrow profile of the streets. They represent the second group of roof shapes, bringing only two additional points to the urban value. Jerkinhead, gambrel and pyramid hip roofs were used at the beginning of the 20th century in close connection to the main new public square and are therefore highly visible. This is why they are awarded 3 points. Ultimately, the most complex roofs, composed of mixed shapes which are placed in prominent positions in the main squares of the city have significant urban value and receive therefore 4 points.

The last criterion which is assessed for this level is the roof pitch. This feature is highly related to the position of the building in the city, its construction date, but mainly to the visibility of the roof from the street level. Therefore, the points increase as the roof pitch gets higher, from 1 point for a roof pitch under 15° up to 4 points for 45 up to 65° roofs, since they are far more visible.

5.1.2 The architectural value of the roof structure

The second level of the procedure takes into consideration the architectural value of the roof and its influence on the way the building is perceived, while also addressing the architectural and functional features of the building (Table 5-2). The procedure is divided in this case into six categories of criteria which try to assess the architectural value from multiple points of view.

1. Historical analysis of the building;
2. Building analysis;
3. Functional analysis;
4. Aesthetic analysis;
5. Geometry of the roof structure;
6. Exterior appearance.

The first category is related to the historical analysis of the building, highlighting its age and the value of the building at the local or national level. The dating criterion is highly related to the city of Timisoara and marks the main periods in which the city evolved but also the periods in which certain architectural principles were characteristic. Therefore, due to the significant historical value of the buildings placed in the city centre, buildings built before the end of the 19th century were considered of significant architectural value and are receiving the maximum of 5 points. The points start to decrease, up until 1936, which is considered to be the beginning of the period in which the roof structures were designed to be economically efficient and technologically advanced and not traditionally crafted in Timisoara.

The heritage value of the building further determines the importance of the building from a historical, aesthetical and architectural point of view, profoundly influencing also the architectural value of the roof and roof structure since both are visually and structurally connected. Therefore, a class A heritage building, of national or international importance, receives the maximum of 3 points since this roof structure has to be preserved for future generations, while local or regional heritage buildings (class B) receive 2 points since they are valuable but at a smaller scale. Buildings which do not have a heritage value receive no point at this level. Due to the significant difference between a heritage building and a common historic one, no 1 point was offered for this criterion.

Subsequently, the next category is related to the building analysis, mainly focusing on the height of the building. Since in the historic part of the city, the height of the building is rather low, with a ground floor and two up to 3 upper floors, this type of buildings were considered of significant value and add 2 points to this assessment level while buildings which are higher and therefore built in the 20th century receive only one point.

The following category is connected to the functional analysis of the building, evaluating its original function and if this function changed over time, leading to possible changes in the shape of the buildings or its interior. The considered functions range from residential, which are the most characteristic buildings, and receive only one point, up to religious function which presents the most sophisticated typology of roofs which receive 3 points. Public functions or mixes of various functions in the same building, on the other hand, receive only 2 points since this type of buildings is generally from the 20th century.

Further on, the aesthetics of the building is analysed considering the architectural style, its corresponding aesthetical features and their influence on the way the roof is related to the building and how it is perceived. Main architectural styles from Timisoara are taken into consideration:

- Neoclassic
- Neo-Romanesque
- Neogothic
- Eclectic
- Secession
- Baroque

This criterion and the assigned points are also related to the construction period, meaning that Baroque-style buildings will receive 5 points while Neoclassic, Neogothic or Neo-Romanesque buildings will receive only 2 points. Due to interventions in time or neglect some of the buildings, even historic buildings, have lost their historical appearance. These buildings have, therefore, no architectural or aesthetical value receiving no point at this criterion. Due to the significant difference between a historic building with a clearly defined aesthetics and one with lost historic appearance, no 1 point was offered for this criterion.

The attention is shifting subsequently to the exterior appearance of the roof, its geometry and materials used for the roof envelope. Like in the case of the urban value, the shape of the roof is having a significant influence on how the building is perceived. Therefore, since shed roof are placed towards the inner courtyard, they do not contribute to the overall image of the building and receive only one point. Gable, hipped or jerkinhead roofs are, as previously stated, common in the historic part of the city and are only partially visible, influencing only little the way a building is perceived, bringing two points. Gambrel roofs are typical in the new square of the city, and significantly influence the aesthetics of the building leading therefore to 3 points. Ultimately pyramid roofs and a mix of different shapes were placed above corner buildings, meant to increase the monumentality of a building. In this case, the roof has one of the most important influences on the appearance of the building, receiving 4, respectively 5 points.

The last criterion considers the used material used for the roof envelope. The points are granted based on the period the different materials were used, from pressed ceramic tiles in the 18th century up to slates and metal roofing in the 20th century.

Table 5-2 Architectural value assessment criteria and corresponding points for each response

C4. Historical analysis of the building		
1. Dating	>1936	1
	1912-1936	2
	1900-1912	3
	1893-1900	4
	<1893	5
2. Monument	Not a monument	0
	Class B monument	2
	Class A monument	3
C5. Building analysis		
3. Height	>P+3	1
	<=P+3	2
C6. Functional analysis		
4. Original function	Habitation	1
	Public function	2
	Mixed function	2
	Religious	3
5. Changes in function	Yes	1
	No	2
6. Contemporary function	Habitation	1
	Public function	2
	Mixed function	2
	Religious	3
C7. Aesthetic analysis		
7. Architectural style	Lost historic appearance	0
	Neoclassic	2
	Neo-Romanesque	2
	Neogothic	2
	Eclectic style	3
	Secession	4
	Baroque	5
C8. Roof geometry		
8. Roof shape	Shed or lean-to roof	1
	Gable roof	2
	Hipped roof	2
	Jerkinhead roof	2
	Gambrel roof	3
	Pyramid Hip roof	4
	Mix of shapes	5
C9. Exterior appearance		
9. Roof envelope	Metal roofing	1
	Slates	2
	Pantile	3
	Scale roof tiles	4
	Ceramic tile	5
	Pressed Ceramic tile	6

5.1.3 The symbolic value of the roof structure

The third level of the procedure is related to the symbolic value of the roof and the roof structure, assessing the used ratios, first between the building and the roof and secondly between the main structural elements composing the roof structure (Table 5-3). The acknowledgement of the role of different types of ratios is also important since they were used to express order and define patterns which would create a close connection between spaces, buildings and their environment and different parts of the building. At the same time, they can offer additional information about the building since the used ratios evolved in time, from highly symbolic proportions in the 18th century, marked by the use of the golden ratio(Φ), to dynamic ones (√2, √3 or √5) and finally to more straightforward and static ratios (1/2, 1/3, 2/3 or 1/6) towards the 20th century when the traditional knowledge was slowly replaced by technology and efficiency.

The assessment level is divided into three main categories, shifting from the exterior appearance of the building towards the detail:

- The ratio between the roof and the building
- The ratio between structural elements
- Symbolic aesthetics

First, the ratio between the roof and the building is taken into consideration. Therefore, based on the complexity of the used ratios, the points granted range from 0, for buildings which present no symbolic link between the roof and the building as a whole, to 4 granted for buildings with a high symbolic value, using the golden ratio (Φ). Building presenting static or dynamic ratios, commonly used in the 19th and beginning of the 20th century, receive two respectively 3 points.

Subsequently, the attention is focused on the ratio between the structural elements and the general appearance of the roof, considering the same five possible responses:

- No ratio;
- Incoherent mix of ratios
- Static ratio (1/1; 1/2; 2/3 …);
- Dynamic ratio (√2, √3, √5);
- Golden ratio (Φ).

The analysis of the used ratios in roof structures brought forward they can be identified when considering the ratio between the height and width of the roof, the position of the main joints or structural elements. Therefore, these three areas have to be assessed, and their ratios identified. The points are granted based on their complexity, from 4 points for golden ratios, since they are rather rare, to 0 points for no ratio and one point for an incoherent mix of ratios. Static and dynamic ratios can obtain 2, respectively 3 points since they are common in the 19th and 20th century and can be identified in most of the structures.

Subsequently, the focus is shifted towards symbolic elements, like the marks of the craftsman, highlighting the authenticity and historical value of the roof structures and additional elements with high philosophical or symbolic value.

The inscriptions of the craftsmen are also classified based on their complexity and uniqueness:

- The numbering of structural elements;
- Dating;
- Messages;

- Craftsmen sign.

Since most of the roof structures until the 20th century have numbered structural elements, as a guide for the assembly of the structure on the building, this type of marking is receiving only 2 points. Dating markings of the roof structure or various massages on collar beams are also rather rare, receiving therefore 3 respectively 4 points. The study of the markings brought forward that, in Timisoara, the signs of the craftsman are rare, bringing, therefore, the maximum of 5 points to the assessment. Due to the high difference concerning the symbolic value between a roof structure with no inscriptions or those with any kind of marking from the craftsmen, no 1 point was assigned to the responses.

The last criterion which is taken at this level into consideration is related to the elements with high symbolic values. It is considering different symbolic elements placed on the roof, highlighting particular functions or specific importance of the building or any additional ornamental elements placed on the roof, which would increase its aesthetical value and importance. The points are granted based on the complexity of the symbolic element, from 0 points for elements with no symbolic value, up to 4, for additional decorations with high symbolic value.

Table 5-3 Symbolic value assessment criteria and corresponding points for each response

C10. 1. The ratio between the roof and the building		No ratio	0
		Incoherent mix of ratios	1
		Static ratio (1/1; 1/2; 2/3 …)	2
		Dynamic ratio (√2, √3, √5)	3
		Golden ratio (Φ)	4
C11. The ratio between structural elements	2. Height/width of the roof structure	No ratio	0
		Incoherent mix of ratios	1
		Static ratio (1/1; 1/2; 2/3 …)	2
		Dynamic ratio (√2, √3, √5)	3
		Golden ratio (Φ)	4
	3. Position of joints defined by	No ratio	0
		Incoherent mix of ratios	1
		Static ratio (1/1; 1/2; 2/3 …)	2
		Dynamic ratio (√2, √3, √5)	3
		Golden ratio (Φ)	4
	4. Position of purlins defined by	No ratio	0
		Incoherent mix of ratios	1
		Static ratio (1/1; 1/2; 2/3 …)	2
		Dynamic ratio (√2, √3, √5)	3
		Golden ratio (Φ)	4
C12. Symbolic aesthetics	5. Inscriptions	No inscriptions	0
		The numbering of structural elements	2
		Dating	3
		Messages	4
		Craftsmen sign	5
	6. Elements with great symbolic value	No symbolic elements	0
		Symbolic roof covering	2
		Symbolic structural elements	3
		Symbolic roof decorations	4

5.1.4 Structural value of the roof structure

After analysing the urban context of the roof, its link to the building and its symbolic value, the focus of the procedure is shifted to the structural assessment. This level of the assessment is divided into three main categories assessing the roof structure as a whole down to its structural elements, details and used joint typologies (Table 5-4).

The first category considers three different criteria meant to highlight the type and main features of the structure. Since 18th and 19th century roofs present a single type of structure and 20th century ones complex mixes of types, the first criterion is related to their complexity.

Simplified or standardised roof structures commonly used, starting with the middle of the 20th century, are considered of low complexity and receive only one point. More complex single typology roof structures, like the ones commonly used in the 18th and 19th century, which present a strong influence of the knowledge of the traditional craftsman, receive 2 points. The maximum of 3 points is granted to mixes of historical roof types since the connections between various areas of the roofs are of high complexity, and commonly used types had to be adapted in order to satisfy the requirements of the urban planning principles and the architectural needs.

Since roof structures can be generally divided into rafter or purlin roof structures, the next criteria are trying to identify to which category the assessed one belongs. Even though in Timisoara, only purlin roof structures can be identified, this criterion was introduced in the assessment procedure in order to be able to use it in other regions of Europe too.

Based on the classifications concerning continental roof structure types commonly used in Romania [30] the next criterion is trying to identify the used structural style, organised according to the main architectural styles:

- Eclectic Roof
- Baroque Roof
- Gothic Roof
- Romanesque roof

The points are once again granted based on the uniqueness of the roof structure and its complexity from one point for standardised roof structures up to 5 points for Romanesque roofs which are rather rare around Timisoara. The score of this criterion can be changed according to the commonly used roof structure types of each region.

The next category of criteria is connected to the structural elements used. First, the truss typology is taken into consideration, focusing on the use of only main trusses like in the case of historic Romanesque or Baroque roof structures or of both main and secondary trusses which is the most common case in this area, receiving therefore 2 points.

Based on the observations made during the assessment of historic roof structure types in Europe and mainly in Timisoara, the following five criteria are focusing on their distinctive features. For the first three criteria, the points are granted based on the observations made during the analysis of roof structures in Timisoara and their uniqueness. Therefore, the maximum amount of points is granted for tie-beams used for all the trusses, hanging devices used for every truss or collar ties used in the upper part of the hanging device. The last two criteria from this category, focus on unique structures or structural elements, like complex systems used for towers or longitudinal rigidity enhancing systems. Once again, unique solutions gain a maximum of 2 points while the lack of any unique structures is bringing no additional points.

The last category of criteria brings the main features of the timber joints into attention. First, the main joining materials are assessed. The maximum of 3 points is granted if the joints were traditionally crafted by using just wooden dowels while the use of other fastening materials is only bringing 1 point. The use of no additional fastening material is generally a sign of reduced joint stiffness and brings only 2 points despite the high historical value and prove of the traditional craftsmanship.

The last criterion addresses the type of traditional joints used in the roof structure, sorting them according to their stiffness. Since in Timisoara mainly, mortise and tenon joints, lap joints and butt joints were identified, only these three types were included in the procedure. Therefore, since most of the roof structures present tenon and mortise joints and they have high stiffness, according to studies, their presence is bringing 3 points to this assessment level, while butt joints are only bringing one. If however, multiple types of joints can be identified in one roof structure, the predominantly used type will be selected during the assessment.

Table 5-4 Structural value assessment criteria and corresponding points for each response

C13. Roof structure	1. Structural typology	Simplified and standardised roof structure	1
		Complex single typology roof structure	2
		Mix of structural typologies	3
	2. Construction system	Rafter roof structure	1
		Purlin roof structure	2
	3. Structural style	Standardised roof structure	1
		Eclectic Roof	2
		Baroque Roof	3
		Gothic Roof	4
		Romanesque roof	5
C14. Structural elements	4. Truss typology	Only main trusses	1
		Main and secondary trusses	2
	5. Tie beam	Only main trusses	1
		Every truss	2
	6. Hanging device	No hanging device	0
		Only main trusses	1
		Every truss	2
	7. Hanging device with	No hanging device	0
		Collar beam	1
		Collar ties	2
	8. Special structures	No special structures	0
		Complex, unique structures	1
		Towers	2
	9. Rigidity enhancing system	No rigidity enhancing system	0
		Central longitudinal system	1
		Longitudinal system in rafter plane	2
C15. Joint typology	10. Joining materials	Mechanical fasteners (nails, screws)	1
		Without additional materials	2
		Wood dowel	3
	11. Used traditional joints	Butt joint	1
		Lap joint	2
		Mortise and tenon	3

5.1.5 Value reduction factors

The next level of the assessment addresses the decay of the roof, taking both the decay of the roof envelope and other elements on the exterior part of the roof (Table 5-5) but also the decay of the structural elements into consideration (Table 5-6).

This level is divided into two categories, the first one considering only the decay visible from the outside while the second one is analysis the state of the decay of the composing timber elements.

In the first category of criteria, the state of conservation of the ridge of the roof, the cornice, chimney and envelope is taken into consideration. The decay is quantified based on the affected area of the respective zones. Therefore, no decay of that specific zone means 0 additional points to the value reduction, while a significantly affected surface, exceeding 30% of its total, means a high vulnerability of that area and increases the score with 4 points.

- Slight decay of the area – under 10%
- Moderate decay of the area – 10 up to 20%
- High decay of the area – 20 up to 30%

Concerning the decay of the envelope, a more thorough analysis of the type of decay is necessary, since depending on the type it may also lead to significant damage to the timber roof structure:

- Biological attacks – moss – affect only the aesthetics of the roof and represent no real threat to the load-bearing structure;
- Parts of the envelope are damaged – small cracks in the roof envelope mean local infiltrations of water which can lead to local damage of the timber elements;
- Distance between envelope elements – poor maintenance of the envelope, displacement of the composing elements of the envelope or missing pieces, can lead to more significant damage of the timber roof structure;
- If parts of the envelope are missing, it can significantly affect the state of conservation of the timber elements and measures must be taken immediately.

For this criterion, the points are granted based on the degree the decay of the roof envelope and how much it can affect the state of conservation of the timber roof structure. Therefore, if the envelope presents no visible decay, 0 points are added to this assessment level, while missing a of part of the roof envelope leads to additional 4 points.

The second category of this assessment level considers the state of conservation of the main timber elements composing the roof structure by evaluating the loss of the cross-section of each element. Considering the structural elements which are common in Timisoara, the considered elements are:

- Tie-beam;
- Passing brace;
- Rafter;
- Purlins;
- Straining beam;
- Collar beam;
- Counterbrace;

The procedure offers, therefore, the possibility to either state that the specific element was not used in the assessed roof structure or specify how much of the original cross-section was lost in time. For these criteria, the score is granted based on the general severity of the cross-section loss, no loss meaning no additional points are added to the level of the assessment, while a severe loss of over 30% brings 4 points. Low (under 10%), medium (10 up to 20%) and high (20 up to 30%) bring 1,2 respectively 3 points. The thresholds were defined according to performed geometric assessments of historic timber structures identified in literature [138].

The last criterion of the assessment level considers the type and state of conservation of the roof to wall connection since it proved out to have a significant influence of the seismic behaviour of the building. Therefore, a good connection between the roof and the wall brings no additional points to the assessment level while a decayed one is bringing additional 2 points.

Table 5-5 Structural value assessment criteria and corresponding points for each response – decay visible from the outside

C16. Decay visible from the outside	1. Decay of the ridge	No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	2. Decay of the cornice	No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	3. Decay at the chimney	No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	4. Decay of the envelope	No visible decay of the envelope	0
		Biological attics - moss	1
		Parts of the envelope are damaged	2
		Distance between envelope elements	3
		Parts of the envelope are missing	4

Table 5-6 Structural value assessment criteria and corresponding points for each response – decay of the roof structure

C17. Decay of the roof structure	5. Decay of the tie-beam	No tie-beam / No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	6. Decay of the compound rafter	No compound rafter / No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	7. Decay of the rafter	No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	8. Decay of the purlins	No purlins / No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	9. Decay of the straining beam	No straining beam / No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	10. Decay of the collar beam	No collar beam / No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	11. Decay of the counterbrace	No counterbrace / No decay	0
		<10%	1
		10-20%	2
		20-30%	3
		>30%	4
	12. Roof to wall connection	Rigid	0
		Semi-rigid	1
		Decayed	2

5.1.6 Climate change vulnerability

Climate change represents a high vulnerability for both roof structures in a good state of conservation but mainly for those already presenting signs of decay. The analysis of climatic factors and their effect on roofs and roof structures brought forward that there are three major threats:

- Rain and wind-driven rain
- Wind
- Hail

Like in the case of the other assessment levels, the climate change vulnerability assessment can also gain a maximum of 100 points and based on the obtained score can highlight how vulnerable the assessed roof structure to climate change is. To each of the three threats, a particular score was assigned, based on how much they can affect the state of conservation of the roof structure and can increase the existing damages (Table 5-7). Ultimately the climate change vulnerability is determined by making a sum of the scores of each risk:

$$S_{Climate} = Vul_{Rain} + Vul_{Wind} + Vul_{Hail} \tag{23}$$

Where $S_{Climate}$ is the climate change vulnerability index, Vul_{Rain} is the vulnerability of the roof and roof structure to rain and wind-driven rain, Vul_{Wind} their vulnerability to high wind velocities and Vul_{Hail} their vulnerability to hailstones.

Table 5-7 Climate change vulnerability – risks and scores

Risk	Score determined based on	Maximum score
Rain and wind-driven rain	C16. Decay visible from the outside	15
Wind	C17. Decay of the roof structure	50
Hail		35

It was observed that rain and wind-driven rain could affect the integrity of the timber roof structure only if the roof envelope or other exterior elements already present certain damages. Still, these damages on the roof structure only appear in time, due to repeated severe precipitation and prolonged lack of maintenance. This is why this tread can only obtain a maximum score of 15 points.

Considering these, this threat is linked to the score obtained for the "Decay visible from the outside" assessment category from the "Value reduction factors" assessment level (Table 5-5). If the ridge of the roof, the cornice, the chimney or the envelope are already damaged, high precipitation quantities can reach the interior of the roof, increase the moisture content of the timber elements and lead to further decay. The score is therefore assigned based on the state of conservation of the roof envelope and will be automatically determined using the following equation:

$$Vul_{Rain} = \frac{S_{C16} \times 15}{16} \tag{24}$$

Where Vul_{Rain} is the vulnerability of the roof and roof structure to rain and wind-driven rain and $S_{C16.}$ is the score obtained for the C16 category of the assessment procedure "Decay visible from the outside".

Wind damage, on the other hand, depends on the state of the conservation of the timber elements and the connection of the roof to the walls. Studies performed on the effect of high wind velocities on the damage of historic church bell-tower roof structures [112] brought forward that the type of connection between the roof and the wall is significantly influencing the structural behaviour of the roof structure during high-velocity winds and leading to different types of failure. Therefore, this threat is linked to the score obtained for the "Decay of the roof structure" assessment category from the "Value reduction factors" assessment level (Table 5-6). In this case, the threat is far more severe than in the rain and wind-driven rain, or even the hail case, since it can lead to the complete failure of the roof structure.

Therefore, this threat can obtain a maximum of 50 points, and the score is assigned based on the state of conservation of the timber elements of the roof structure and will be automatically determined using the following equation:

$$Vul_{Wind} = \frac{S_{C17} \times 50}{30} \qquad (25)$$

Where Vul_{Wind} is the vulnerability of the roof and roof structure to high wind velocities and $S_{C17.}$ is the score obtained for the C17 category of the assessment procedure "Decay of the roof structure".

The last main threat in this region is hail, significantly affecting the state of conservation of the roof envelope and ultimately leading to multiple additional damages to the timber roof structure. Since hail can affect both roof envelopes in a good or bad state of conservation, this threat is not linked to any previously assessed criterion and automatically adds 35 points to the climate change vulnerability.

The climate change vulnerability part of this procedure was calibrated based on the meteorological and climate change risk assessments performed for Timisoara and the western part of Romania. Therefore, if the procedure is used in another location, this level must be adapted in order to comprise all the local climatic threats and the score reassigned.

5.1.7 Seismic behaviour of the building and the role of historic roof structures

The effect of the roof structure on the seismic behaviour of historic masonry buildings is one of the original features of the proposed assessment procedure. The study proved that the seismic behaviour of a historic masonry building could change depending on the roof structure type and the state of conservation of the timber elements, being therefore linked to responses provided in the "Aesthetic analysis - Architectural style" criterion and the "Decay of the roof structure" criteria. The score is granted based on how much the roof structure reduces the top horizontal displacement, the top inter-story drift and the damage level of the historic masonry building and how the presence of the roof structure changes the internal forces on the historic masonry walls (Table 5-8), considering the observations made during the numerical analysis of an 18th century building with an 18th, 19th and beginning of the 20th century roof structure, presented in chapter 4.

For the influence on the seismic behaviour of the building the form does not offer any score but states, at the end of the results what the effect of the considered roof structure type would be. Therefore, for Baroque roof structures, since they proved out to have the most visible effect on the seismic behaviour of the building, the form will state "The effect of the roof structure on the seismic behaviour of the building is very high", for a roof structure with a good state of conservation and "The effect of the roof structure on the seismic behaviour of the building is high" for a decayed one. The queen post roof structure with inclined posts, commonly used at the end of the 19th century had a significant effect at the top of the building but transferred the loads and deformations towards the floor below, the form stating that "The effect of the roof structure on the seismic behaviour of the building is high", for the complete cross-section roof and "The effect of the roof structure on the seismic behaviour of the building is moderate" for the decayed one, since the differences are rather low between the two cases. Ultimately, since the queen-post roof structure has a lower influence on the seismic behaviour of the building and is not changing its deformation, the form will state that "The effect of the roof structure on the seismic behaviour of the building is moderate" for the full cross-section case and "The effect of the roof structure on the seismic behaviour of the building is low" for the decayed roof structure.

Table 5-8 Effect of the roof structures on the seismic response of the historic masonry building compared to the same building with no roof structure and subsequent result offered by the assessment procedure

Roof structure	Full/reduced Cross-section -20%	Compared to the same building with no roof structure						Effect (result offered by the assessment methodology)
		Top horizontal displacement	Top inter-story drift	Damage Level historic masonry	Internal Forces (on the historic masonry wall)			
					N	V_z	M_y	
18th century Baroque roof	Full	-55%	-85%	Reduced		Increase	Increase	Very high
	Reduced	-50%	-80%	Reduced	Tension	Increase	Increase	High
19th century Queen post inclined post	Full	-45%	-80%	Reduced		Decrease	Increase	High
	Reduced	-45%	-85%	Reduced	Tension	Decrease	Increase	Moderate
20th century Queen post	Full	-30%	-40%	Reduced		Increase	Increase	Moderate
	Reduced	-30%	-50%	Reduced	Tension	Increase	Increase	Low

5.2 Analysing the results

After selecting the suitable response for each of the assessed criteria, the procedure is automatically analysing each response and calculating its corresponding score. Every level of the assessment can obtain a maximum score of up to 100 points which are equally divided between all the assessed criteria of the considered level.

Based on the selected responses, the procedure offers information about:

1. the value of the roof structure from each point of view;
2. the predominant value of the roof structure;
3. the ideal global value of the roof structure;
4. the real, current value of the roof structure;
5. the vulnerability of the roof structure
6. the influence of the roof structure on the seismic behaviour of the building it belongs.

The value of the roof structure from each point of view is determined separately, based on the selected responses. The score of each assessment level will be determined using the following equation:

$$V_{Level} = \sum_{i=1}^{n_q} \frac{100 \times S_i}{n_q \times \max C_i} \tag{26}$$

Where, V_{Level} is the value of the roof structure for the assessment level which is taken into consideration, S_i the score displayed in the assessment form corresponding to the chosen response for the assessed criterion, n_q the total number of assessed criteria for the considered level and *max* C_i the maximum score which can be obtained for the assessed criterion. This equation ensures that the maximum of 100 points which can be obtained for each assessment level is equally divided between all the criteria of the considered level and subsequently modified according to the score of the chosen response.

The procedure is, therefore determines:

- The urban value of the roof structure, considering the seven assessed criteria of the assessment level:

$$V_{Urb} = \sum_{i=1}^{7} \frac{100 \times S_i}{7 \times \max C_i} \tag{27}$$

- The architectural value of the roof structure, considering the nine assessed criteria:

$$V_{Arh} = \sum_{i=1}^{9} \frac{100 \times S_i}{9 \times \max C_i} \tag{28}$$

- The symbolic value of the roof structure, considering the six assessed criteria:

$$V_{Sym} = \sum_{i=1}^{6} \frac{100 \times S_i}{6 \times \max C_i} \tag{29}$$

- The structural value of the roof structure, considering the eleven assessed criteria:

$$V_{Str} = \sum_{i=1}^{11} \frac{100 \times S_i}{11 \times \max C_i} \tag{30}$$

These values are subsequently compared with each other in order to identify which feature is determining the predominant value of the roof structure.

This comparison offers a preliminary point of view about the most important feature of the roof structure (Fig. 5.1). In this way, the assessor is informed about its role in its urban context, link to the building, symbolic features and structural characteristics, bringing forward which one of them is having an essential role in defining its features and helping defined future decisions and strategies.

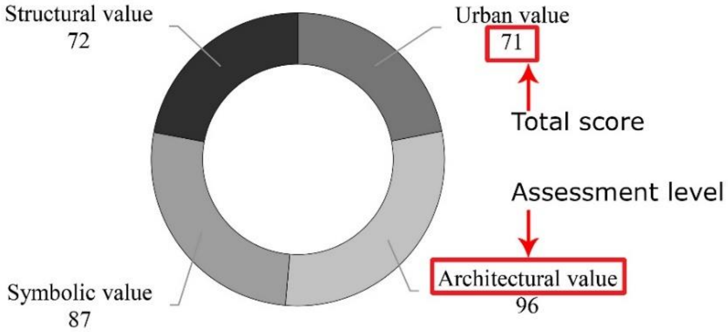


Fig. 5.1 Value of the roof structure – comparison of the assessed levels (example)

Therefore, roofs with significant importance in the urban context are additionally defining the aesthetics of a building are not only structures meant to protect the building from meteorological factors. They are complex systems which complete the image of an urban area, or a building and their appearance should be preserved. Roof structures with symbolic features, highlight that craftsman used to build according to complex principles and that the roof structure is a sophisticated system of structural elements placed according to their knowledge about the universe. These types of roof structures have to be preserved as a whole, highlighting the features which make them unique and the used ratios should not be altered. Ultimately, roof structures which prove to have a high structural value, present the use of unique structural elements or are complex types adapted to suit the needs of the building. In this case, the structures should be preserved as a testimony of the skills of the craftsman and the quality of the craftsmanship visible in every detail.

After the predominant value of the roof structure is determined, its ideal global value is brought forward. The ideal value of the roof structure represents the hypothetical value of the roof structure, without any decay, based on its urban context, the link with the building and its aesthetical value, its symbolic features and unique structural elements. It is determined using the following equation, which was developed considering the involvement of various professionals, architects, engineers or other professions, in the assessment of heritage structures:

$$V_i = 0.25 \times V_{Urb} + 0.25 \times V_{Arh} + 0.15 \times V_{Sym} + 0.35 \times V_{Str} \quad (31)$$

The obtained score, which can reach up to 100 points, is subsequently compared to value level thresholds in order to determine if the roof structure has a very low, low, moderate, high or very high value. The higher the score, the higher the value of the roof structure is (Table 5-9).

The assessment form offers a visual representation of the value levels and the ideal value of the assessed roof structure, in order to make the obtained results clearer for the assessor. The visualisation is based on two concentric doughnut charts, the exterior one representing the ranges of the value levels with the corresponding thresholds, while the interior one is only presenting the score obtained for the ideal value. In this way, the assessor can observe the value level of the assessed roof structure and its position to the other value levels without actually knowing the corresponding value index of each level (Fig. 5.2).

Table 5-9 Value index and levels of historic timber roof structures

Value index	Value level
0-25	The roof structure has very low value
25-50	The roof structure has low value
50-75	The roof structure has moderate value
75-90	The roof structure has a high value
90-100	The roof structure has a very high value

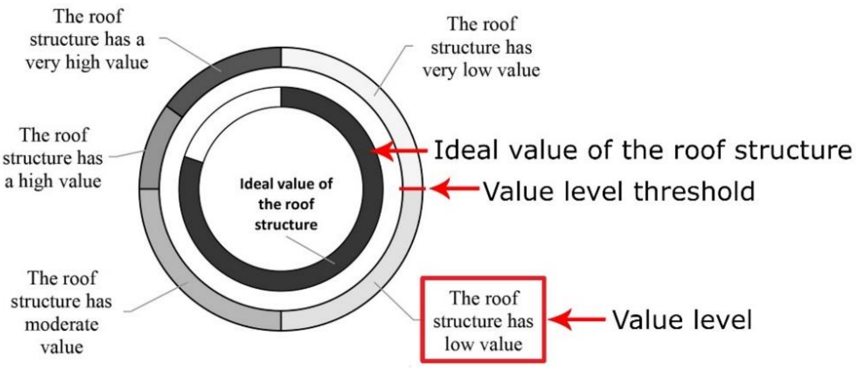


Fig. 5.2 Ideal value of the roof structure – the visual representation of the ideal value and value level thresholds (example)

After assessing the roof structure from all the points of view and determining its global value, the focus is shifted towards the visual assessment of the state of conservation of the roof structure. By doing this, the assessor can determine the real value of the roof structure and its vulnerability.

First, the responses from the “Value reduction factors” assessment level are processed by the assessment form, and the decay index of the roof structure offered to the assessor. The decay index (D) is determined using the same equation as previously, by dividing the maximum of points equally between all the criteria of the level and then changing the score according to the chosen response:

$$D = \sum_{i=1}^{12} \frac{100 \times S_i}{12 \times \max C_i} \tag{32}$$

The real, current value of the roof can be determined, considering the ideal value of the roof structure and its state of conservation. Since the decay of the structural elements is not only affecting its structural integrity but also its aesthetical and symbolical value, in order to determine the real value of the roof structure, the decay factor is subtracted from the ideal value of the roof.

Since the ideal value of the roof structure is determined by considering a series of complex features, linked to the context or the history of the craftsman, in order to determine its real value, a correction coefficient of 15% was considered for the decay value, and the real value is determined using the following equation:

$$\mathrm{Vr} = \mathrm{Vi} - 0{,}15\mathrm{D} \tag{33}$$

Where V_r is the current value of the roof structure. By using the correction coefficient, it is also assured that the real value of the roof structure will be a positive number, and no thresholds below 0 had to be set up.

The thresholds of the value levels are the same as in the ideal value case (Table 5-9), with wider boundaries of 25 points for the first three value levels and tighter ones for the roof structures with high and very high values. The visual representation of the obtained value index also respects the same principles as previously presented (Fig. 5.3).

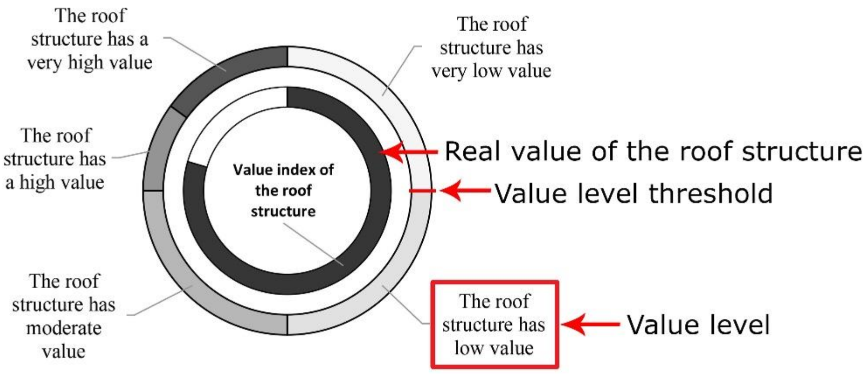


Fig. 5.3 Real value of the roof structure – the visual representation of the real value and value level thresholds (example)

Next, the vulnerability index is determined by considering the ideal value of the roof structure and the factors which are increasing its vulnerability, the decay of the exterior part of the roof, the decay of the timber structure and the climate change vulnerability.

Since a structure is more vulnerable if its value is high, 65% of the vulnerability index was taken from the score of the ideal value and the other 35% from the vulnerability increasing factors - 25% from the decay and 10% from the climatic threats score:

$$\mathrm{V} = 0{,}65V_i + 0{,}25\mathrm{D} + 0{,}10S_{Climate} \tag{34}$$

Where *V* is the vulnerability index of the roof structure, $\mathrm{V_i}$ the ideal value of the roof structure, D the decay index of the roof and $\mathrm{S_{Climate}}$ the climate threat vulnerability previously determined.

Subsequently, considering the obtained vulnerability index, the roof structure can have a very low, low, moderate, high or very high vulnerability. The boundaries between two consecutive vulnerability levels decrease as the vulnerability gets higher from 30 points for the low vulnerability level down to 10 for the very high vulnerability level (Table 5-10).

The visual representation of the obtained vulnerability index also respects the same principles as previously presented for the ideal and real value of the roof structure (Fig. 5.4).

Table 5-10 Vulnerability index and levels of historic timber roof structures

Vulnerability index	Vulnerability level
0-30	The roof structure has very low vulnerability
30-50	The roof structure has low vulnerability
50-70	The roof structure has a moderate vulnerability
70-90	The roof structure has a high vulnerability
90-100	The roof structure has a very high vulnerability

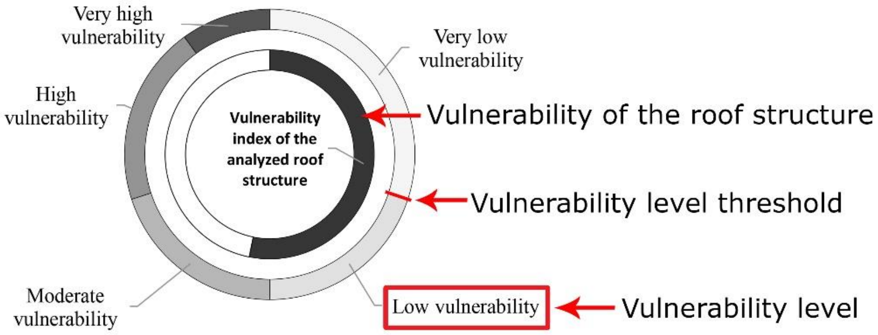


Fig. 5.4 Vulnerability of the roof structure – the visual representation of the vulnerability and vulnerability level thresholds (example)

5.3 Validating the proposed vulnerability assessment procedure

In order to validate the proposed vulnerability assessment procedure, 18 roof structures from various neighbourhoods of the city were chosen: the city centre, the Iosefin district and Fabric district (Fig. 5.5). The roof structures were chosen from different periods, different contexts and belonging to buildings with different functions, in order to capture the changes of the predominant, ideal and real value and their vulnerability better. The roof structures were assessed from all the relevant point of view and all the obtained data introduced in the assessment form.

The context in which the chosen roof structures were built are entirely different and are marked by the character of each district. While the centre had from the beginning a more urban aspect, the other two districts were former villages, which suffered significant changes at the end of the 19th century, beginning of the 20th, when they were connected to the old fortress, and they became neighbourhoods of the new city [179,180]. This is why different types of buildings, roofs and roof structures influenced by different principles were identified in these three areas.

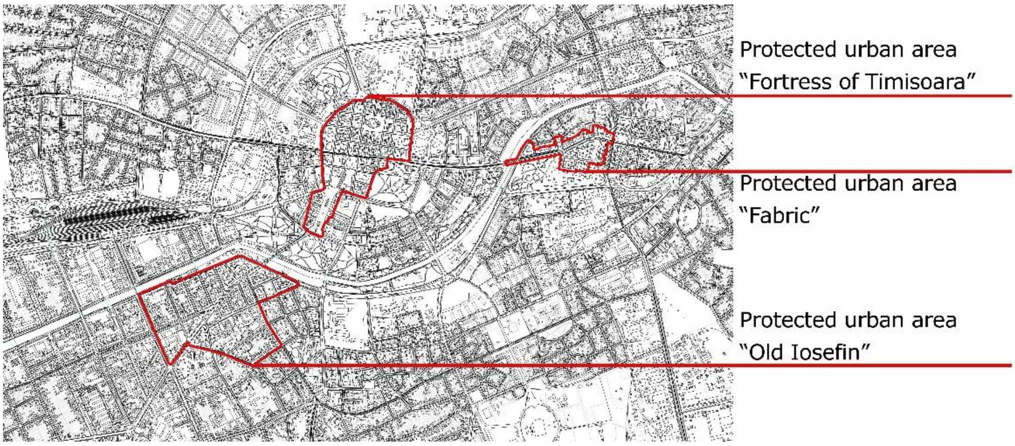


Fig. 5.5 Protected urban areas where the assessed roof structures were chosen

5.3.1 The assessed roof structures

5.3.1.1 Roof structures from the area of the old fortress of Timisoara

In the 18th century, a new project for the city was proposed, which supposed the demolition of all the existing buildings, built during the Ottoman period, except for the castle. The project proposed an entirely new rectangular grid of narrow streets, with brick masonry buildings, organised around three main squares, one military, one religious and the third one administrative.

At the end of the 19th century, the fortification was preventing the city from evolving. At the same time, the surrounding villages developed continuously, which led to the decision to demolish the fortification walls and ensure, through complex urban planning, a connection between the historic city and the surrounding villages. The project was developed by Ludwig von Ybl and engineer Aladar Kovacs Sebestyen [16, 17], who proposed a series of radial streets and a new square.

In this area, 13 roof structures were assessed, from different contexts and periods (Fig. 5.6). The buildings represent residential buildings as well as religious and public buildings, chosen in order to understand better the link between urban planning principles, architecture and structure related to the roof and its timber structure.

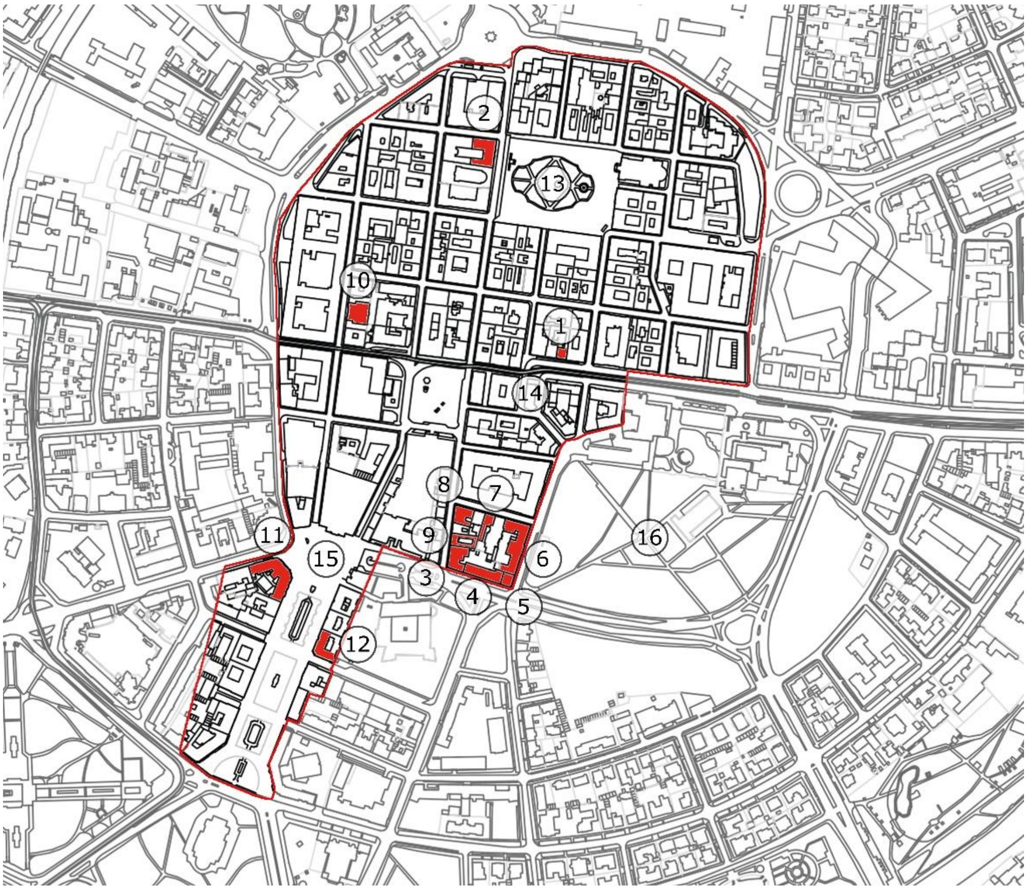


Fig. 5.6 Assessed roof structures from the fortress of Timisoara and landmarks (1 – St. George square building; 2 – Union square building; 3 - I.C. Bratianu high school - wing A; 4 - I.C. Bratianu high school - wing B; 5 - I.C. Bratianu high school - wing C; Chemistry faculty; 7 - Residential building 19th century; 8 - "Victoria" Hotel; 9 - Residential building 20th century; 10 - Great Synagogue of Timisoara; 11 - Lloyd Palace; 12- Löfler Palace; 13 – Union square; 14 – St. George square; 15 – Liberty square; 16 – “Civic” park

Two roof structures from the 18th century were evaluated in this area, one related to the religious Saint George square, closer to the outer side of the fortress and one placed in the Union Square (Fig. 5.7). Despite presenting the same structural typology, with a Baroque straining device, a double layer of exterior structural elements and tie-beams placed under every frame, the context in which they emerged is entirely different.

The roof structure of the Saint George square building is a simple gable roof placed parallel to the street. Since the street is rather narrow, the roof can only be perceived from the other side of the road and the square place nearby. At the same time due to the presence of an alignment of trees, placed near the pedestrian area, today the building and the roof are almost entirely covered, and the passerby cannot perceive their architecture. The building placed in the Union Square, on the other hand, has a corner position and can be seen from the square but also while approaching the square from one of its tangent streets. It presents, therefore, an intersection of two gable roofs, placed towards the square and the street and a shed roof towards the interior courtyard.

Fig. 5.7 Union square 18th century roof structure

Also, from this area, seven early and late 19th century roof structures have been assessed, mainly because this period presents the most significant changes regarding architecture and urban planning principles but also concerning the used structural types. All of these roofs were chosen from a block placed close do the outer limits of the fortress. After the fortress was torn down, their importance rose significantly. From a block placed on the outer limit of the city, it became one that was in a close connection to the new urban developments, and highly visible from the new streets. Because of this, some of the buildings evolved, mainly those facing the new streets and the corner buildings, while others preserved their initial appearance, which leads to a mix of styles and structural types.

Two roof structures from the early 19th century were identified, which present the transition towards the typical queen-post roof structure. Both of them belong to the high school which is placed on the southern part of the block, placed on the corners of the main building. Despite their excellent position facing an open urban space, both of the structures were preserved in their original state having a low height and not standing out at all.

Due to their position, both structures represent half of a hip roof, with one side of the hip facing the street while the main gable roof is directly connected to the neighbouring roof. In both cases, the roofs present sophisticated ways to solve the hip part of the structure, using a complex system of interlinking elements meant to transfer the load from the hip side towards the main gable roof. The central part of the structure is a queen-post roof structure with inclined posts (Fig. 5.8). Still, additional timber elements, meant to connect the rafter with the posts and the passing brace were used, which is highlighting the fact that they are transitional types.

Fig. 5.8 Early 19th century roof structure type

The other five roof structures are different versions of queen-post roofs, but they still highlight the significant changes which take place in this period concerning architectural styles and the influence of the urban context on the roof.

On the southern part of the block, the roof structure of the main wing of the high school was assessed. The building suffered changes in the 19th century being transformed into an eclectic building, connected to the new streets which highlight its importance. It was observed that the building has two different areas when considering its roof structure, one who seems an adaptation or reuse of one from another building due to unused mortise holes. The other roof, placed on the right side of the building, is a clear queen and king-post roof structure (Fig. 5.9). The left roof structure also presents a similar type to the queen and king-post roof structure, with specific elements like hanging posts, passing braces and collar beam but there are still additional elements which are not specific. The main peculiar feature of this roof is that the passing braces and the rafters are not connected in the inferior part by a tie beam, but a fragment of a tie-beam was used to connect one of the posts with the passing braces and the rafters.

Fig. 5.9 I.C. Bratianu high school – late 19th century roof structure (right side)

On the northwestern part of the block, a 1900 Secession style, L shaped building was also assessed, with an impressive round tower, placed on the corner of the building, a specific element of the Secession style.

The roof is composed of two gable roofs which are intersected on the corner. However, what makes this structure special is the round tower. Regarding the structure, a clear queen and king-post type was used with hanging posts, rafters, passing braces and a continuous tie connecting all the elements in the inferior part. For the tower, the half of the same roof type was used, but this time it was rotated around a central axis.

Another unusual roof of this block is placed in the continuous front of its northern side. Despite being purely residential, in order to introduce light into the staircase, an oculus was introduced in the top slab of the building, ensuring the use of the natural light from the attic. In the same time, since the building was placed along a very narrow street, in the vicinity of an heritage church, the building with only three floors would not have been able to stand out, so measures had to be taken. Therefore, considering the two factors, the top wall was built a bit higher, increasing the importance of the building but also enabling light to enter the attic and down the staircase.

Still, the most spectacular roof of the block is the structure of the Chemistry Faculty, placed on the eastern side of the block. The three-story building used to be a Franciscan monastery, which was later on transformed into a school and in the 20th century into the headquarters of the Chemistry Faculty of the Polytechnic University of Timisoara. The building covers almost the whole eastern side of the block, facing one of the main parks of the city, which makes it highly visible for almost everybody passing by.

The building has a complex shape and presents clear eclectic features. The roof structure presents a mix of roof shapes, with shed roofs towards the interior courtyard, gable roofs towards the street, a part of a hip roof on the northern part of the building and an imposing pyramidal roof placed in the central part of the eastern façade. The used structural type represents an adaptation of the queen-post roof structure, which had to be altered because the wall facing the street is slightly higher than the one facing the interior courtyard. Therefore, an additional horizontal element is connecting the higher wall with the passing brace and the hanging post. At the same time, due to the mix of shapes, complex connections had to be solved between its various parts which makes it unique and spectacular from both the inside and outside.

Subsequently, 20th century roof structures were also taken into consideration, from the area of the Victory square. These buildings have completely different aesthetics, being imposing and helping shape the limits of the square and the principal visual directions planned by the architect. Two buildings from this square were assessed, one being a corner building, the Lloyd Palace and the second one a building placed in the perfectly aligned frontage of the square, the Löffler palace. Both buildings present complex queen and king post roof structure types which had to be adapted due to the significant height of the roof and 2 series of collar ties used to connect the queen posts.

5.3.1.2 Roof structures from the area of the Fabric district

The Fabric suburban area (Fig. 5.10) used to be inhabited mainly by craftsman, traders and a small number of peasants. At the end of the 19th century, this area was already more inhabited than the central city. Since the area around the fortress, in which it was forbidden to build, got smaller in time, the village started to develop towards it and finally after the fortification was demolished, new residential buildings were planned along the street connecting the centre with the Fabric district.

In this area three roof structures were assessed, which present almost similar structural principles, belonging to buildings placed on the street connecting the Fabric district with the city centre built at the beginning of the 19th century, in a period in which roofs and roof structures had to be adapted in order to comply to architectural requirements.

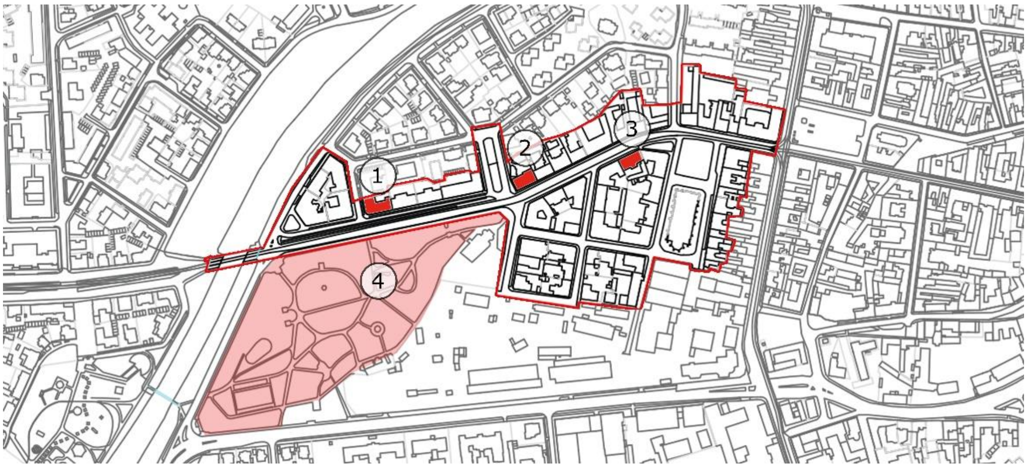


Fig. 5.10 Assessed roof structures from the Fabric district and landmarks (1 - Archduke house; 2 - Archduke house; 3 – Residential building; 4 - "Queen Mary" park)

One of the first buildings from this street is the Archduke house, a three-story corner building. It is a highly visible building for the people passing by, due to its position facing one of the entrances to the nearby "Queen Mary" park. Still, despite the ample space surrounding the building, the roof remains utterly invisible from the pedestrian area, since it has a low pitch and a high attic wall facing the street. The roof structure is, therefore, not completing the aesthetics of the building in any way. The used hip roof is presenting an end of the 19th century roof structure with inclined posts, in a good state of conservation, with a complex solution in the hip area.

The second assessed roof structure also belongs to a corner building, the Karl Kunz palace, built at the beginning of the 20th century. This building is also visible from the nearby "Queen Mary" park and has an ample open space on two of its sides. As in the previous case, the intersection of two gable roofs is not visible from the pedestrian area. The roof structure is a queen-post type which was altered towards the street since the exterior walls facing it are higher in order to increase the monumentality of the building.

The last assessed roof structure from this district is a residential building which is placed in the continuous front of the block, aligned to the street. At the same time, trees are placed on the limit of the pedestrian area. Since the building is not withdrawn from the street and due to the presence of the trees, the roof structure can only partially be perceived by the pedestrians. Like in the case of the Karl Kunz palace, the roof structure is a queen-post type which was altered due to the presence of a higher exterior wall towards the street.

5.3.1.3 Roof structures from the area of the Iosefin district

First developed around the Bega channel, the suburban area of Iosefin (Fig. 5.11) has mainly evolved in time towards the south-west, with a series of new streets, placed perpendicular to the waterfront. Due to the strong connection with the water, this area encouraged the appearance of industrial buildings, while the southern part was mainly residential. In the middle of the 19th century, the appearance of a catholic church in the southern part of the district encouraged the development of a new main street with a significant width.

For this district, roof structures from its southern part were analysed and three buildings placed along its main street chosen. Like for the Fabric district, two corner buildings and one placed in the middle of the continuous frontage of a block were analysed. All the selected buildings were built at the beginning of the 20th century in the Secession style, and present a higher exterior wall placed towards the street no matter the position or importance of the building. Even though all the assessed buildings are presenting a queen post-like roof structure, they still had to be altered in the area of the street walls and additional structural elements used. In order to better connect the roof structure with the top of the wall, an additional horizontal beam was used connecting the tie-beam with the passing brace, the ridge purlin and the wall plate. At the same time, the posts have different heights, and the collar beam is connecting the top of one post with the other post 40 cm below the top.This leads to complex and new, adapted timber roof structures which are unique despite belonging to simple residential buildings.

Fig. 5.11 Assessed roof structures from the Iosefin district and landmarks (1 - 1900 Secession style building; 2 – corner residential building; 3- 20th century residential building; 4 - Mocioni square)

Two of the chosen buildings are placed on the corner of two different blocks of the district, both in a close connection with the nearby Mocioni square. Due to the steep pitch of the roofs, they are highly visible from the square and the pedestrian area across the street.

The first one is a 1900 Secession style, two-story building which has a complex shape caused by the fact that the two intersecting streets are not forming a 90° angle and the corner placed towards the square is additionally chamfered.

The roof is a complex intersection of gable roofs which are slightly altered in the area of the chamfered corner. The structure is a queen-post structure, but the collar beam is placed not at the top of the shorter queen-post, but about 30 cm below its top, slightly below the intersection of the queen posts with the passing brace.

The second corner building is also presenting two stories and a highly visible roof. The roof is, in this case, a clear intersection between two gable roofs without any additional alterations. The roof structure, on the other hand, is also presenting adaptations of the queen post type, adding inclined timber element in the area of the higher wall, connected to the queen post slightly below the joint with the collar beam.

The third building, built at the beginning of the 20th century, is inserted in the middle of a heterogeneous façade of a block with buildings presenting similar style but variable height. Despite also having a ground floor and one upper floor, the roof is in this case only visible from the other side of the street.

The roof is a gable roof parallel to the street with a half hip roof placed towards the inner courtyard. The structure, on the other hand, is of a completely different type as all the other roof structures presented until now. The gable roof part is a purlin roof structure in which every truss is composed only of the two rafters, the tie-beam in the inferior part of the truss and a collar beam in the upper part. Additional ridge and intermediate purlins were used to connect the trusses.

5.3.2 Applying the assessment procedure and calibrating the scores

From the three considered neighbourhoods of the city, after the assessment of each roof structure from all the points of views, four roof structures were selected, which were representative for each of the four assessment levels. The scope was to see if the assessment form will recognise the same predominant value as the performed preliminary assessment.

The selected buildings for the validation of the roof structure assessment procedure were the following

- For the urban value – the Lloyd palace from the Victory square;
- For the architectural value – the "Victoria" hotel;
- For the symbolic value – the main wing of the I.C. Bratianu high school building;
- For the structural value – the St. George building.

For each of the selected buildings and their roof structure the urban context was assessed, the architectural features of the building and its eventual evolution in time, the ratios used, the main structural type and finally the state of conservation of the timber.

5.3.2.1 The Lloyd palace, Victory square – roof with a high urban value

The Lloyd palace has significant importance in shaping the Victory square (Fig. 5.12). Already from the first urban plans of architect von Ybl, the role of this building was highlighted. On the one hand, it is aligned with the other buildings forming the north-western frontage of the square, helping define the ambience of the promenade. On the other hand, due to the clearly defined lentil like urban space, it creates together with other two buildings, it was also designed to visually connect the square with the historical castle from the Ottoman period (Table 5-11).

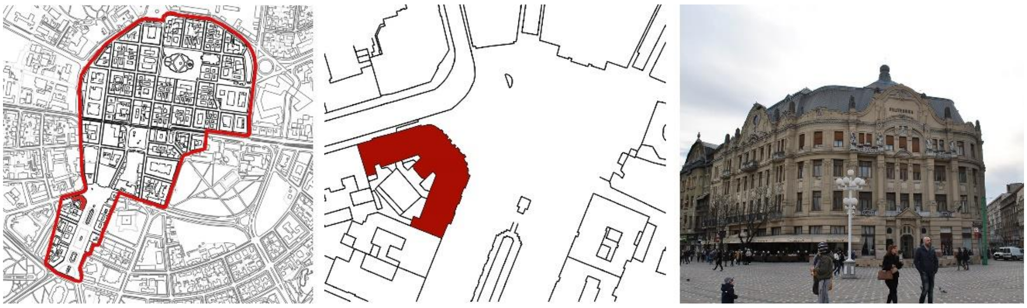


Fig. 5.12 The Lloyd palace – urban context and relationship with the street and building

The roof is accurately expressing these too principles. It presents a mix of roof shapes, used according to their position on the building and related to the context. Therefore, towards the street and the main frontage of the square two gambrel roofs were used, which have a steep slope in the inferior part while in the front section of the building a pyramidal roof was placed, highlighting its central axis by a globe laid on the top of the pyramid. In order to connect the gambrel roof with the pyramid, a semi-circular roof structure was used, while over the two ends of the building, two shed roofs were placed (Table 5-12).

The used roof structure is a queen and king-post type. Due to the significant height of the gambrel and the pyramidal roof, the hanging device of the queen-post structure had to be designed over two levels, intermediately connected by a collar beam. The most peculiar features of this structure are the rounded corners which represent half of the same queen-post roof structure rotated around the hanging post but also the top structural element place above the pyramidal roof meant to sustain the decorative globe (Fig. 5.13).

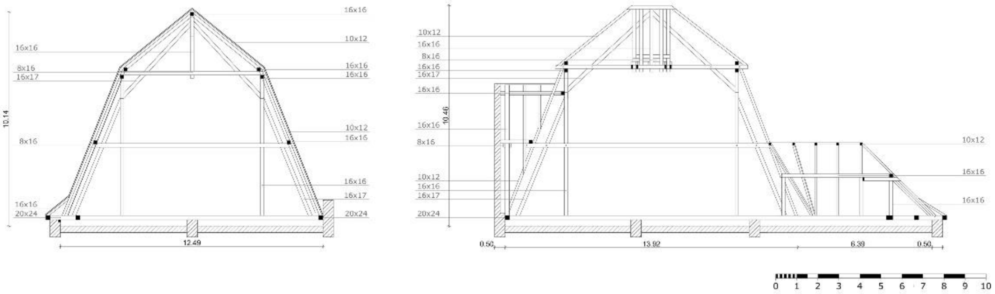


Fig. 5.13 The Lloyd palace – the roof structure

Despite its complexity, the roof structure has no real symbolic value, since no valuable ratios were found as defining principles for the position of the timber elements (Fig. 5.14). Still, there are numbers on each structural element, highlighting that the knowledge of the craftsman was still used even in the 20th century (Table 5-13).

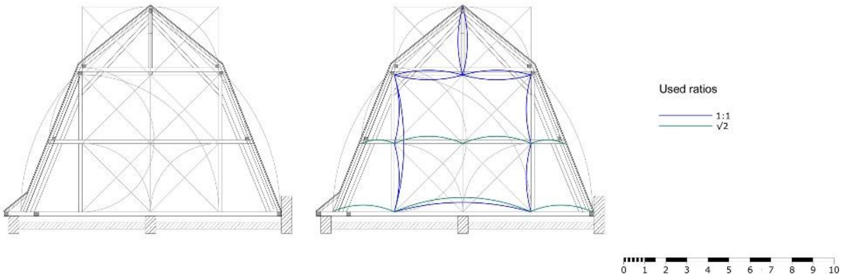


Fig. 5.14 The Lloyd palace – used ratios

Although no real improvements were made concerning the used structural type, this case represents an impressive way to solve connections between different shapes of roofs with different heights. The aesthetics of this roof structure lies, therefore in the details (Table 5-14).

The roof structure is in a good state of conservation, presenting only a little decay caused by water infiltration of the rafters and ridge purlin. Still, the cause of the decay was removed in the meanwhile, since the roof envelope was changed entirely, and no infiltration is now possible. The roof on the outside, its decorative elements and its envelope are also in a very good state of conservation presenting no decay (Table 5-15).

Table 5-11 Lloyd palace – urban value

Criterion		Response	Score
Value of the urban area		Significant role in defining the urban space	4
Urban analysis	Position of the building	Marks an essential urban point	4
	Frontage	Continuous front	2
	Height	Constant Height at cornice	2
	Alignment	Street alignment	4
Geometry	Roof shape	Mix of shapes	4
	Roof pitch	45º-65º	4

Table 5-12 Lloyd palace – architectural value

Criterion		Response	Score
Historical analysis of the building	Dating	1912-1936	2
	Monument	Not a monument	0
Building analysis	Height	<=P+3	2
Functional analysis	Original function	Public function	2
	Changes in function	No	2
	Contemporary function	Public function	2
Aesthetic analysis	Architectural style	Eclectic style	3
Geometry of the roof structure	Roof shape	Mix of shapes	5
Exterior appearance	Roof envelope	Metal roofing	1

Table 5-13 Lloyd palace – symbolic value

Criterion	Response	Score
Ratio between the roof and the building	Incoherent mix of ratios	1

Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
	Position of joints defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
	Elements with great symbolic value	Symbolic roof decorations	4

Table 5-14 Lloyd palace – structural value

	Criterion	Response	Score
Roof structure	Structural typology	Mix of structural typologies	3
	Construction system	Purlin roof structure	2
	Structural style	Eclectic Roof	2
Structural elements	Truss typology	Main and secondary trusses	2
	Tie beam	Only main trusses	1
	Hanging device	Only main trusses	1
	Hanging device with	Collar ties	2
	Special structures	Complex, unique structures	1
	Rigidity enhancing system	Central longitudinal system	1
Joint typology	Joining materials	Mechanical fasteners (nails, screws)	1
	Used traditional joints	Mortise and tenon	3

Table 5-15 Lloyd palace – value reduction factors

	Criterion	Response	Score
Decay visible from the outside	Decay of the ridge	<10%	1
	Decay of the cornice	No decay	0
	Decay at the chimney	No decay	0
	Decay of the envelope	No visible decay of the envelope	0
Decay of the roof structure	Decay of the tie-beam	<10%	1
	Decay of the compound rafter	<10%	1
	Decay of the rafter	10-20%	2
	Decay of the purlins	<10%	1
	Decay of the straining beam	<10%	1
	Decay of the collar beam	<10%	1
	Decay of the counterbrace	<10%	1
	Roof to wall connection	Semi-rigid	1

Results

- The urban value of the roof structure – 100 points;
- The architectural value of the roof structure - 61 points;
- The symbolic value of the roof structure - 53 points;
- The structural value of the roof structure - 70 points;
- The ideal value of the roof structure - 70 points;
- The decay index of the roof structure - 23 points;
- The climate change vulnerability of the roof structure – 51 points;
- The value index of the roof structure - 69 points;
- The vulnerability of the roof structure - 58 points.

Conclusions:

- The roof structure has moderate value;
- The roof structure has a predominant urbanistic value;
- The roof structure is reducing the horizontal displacement of the building;
- The effect of the roof structure on the seismic behaviour of the building is low;

- The roof structure has a moderate vulnerability.

5.3.2.2 "Victoria" Hotel – roof with a high architectural value

The hotel was built at the beginning of the 20th century on the north-western corner of a block of the old fortress area. It was built in the 1900 Secession style, specific for Timisoara and the aesthetics of the building is generally marked by a tower placed on the corner of the building, highlighting the building but also the roof structure (Table 5-16).

Placed at the intersection of two narrow streets, the main roof structure of the building is only slightly visible for a pedestrian, and only from a great distance. Still, due to the presence of the corner tower, the roof is highlighted and automatically brought into the attention of the passerby (Fig. 5.15).

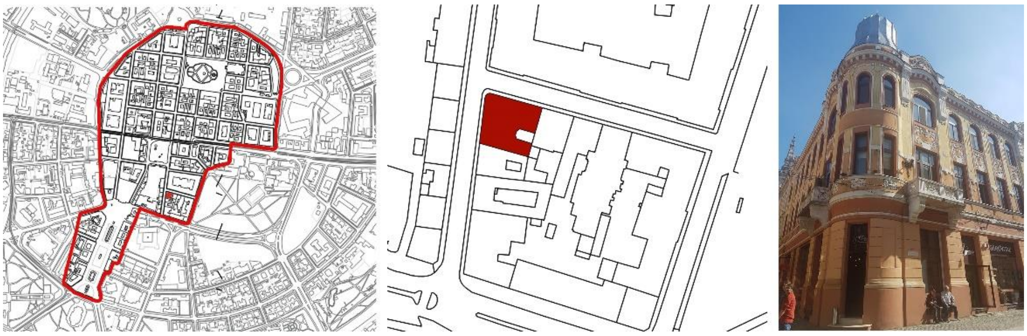


Fig. 5.15 "Victoria" Hotel – urban context and relationship with the street and building

Due to the U shape of the building, the roof structure is a mix of two gable roofs placed towards the street, and one shed roof towards the inner courtyard (Fig. 5.16). The roof structure is in this case strongly influenced by the shape of the building and the geometry of the roof, the gable roof part presenting a clear queen and kingpost roof type while the shed roof is presenting only half of the same typology (Table 5-17).

Due to the type of the roof structure and the significant distance between the tie beam and the straining beam, the attic of the hotel was transformed into apartments so no accurate survey of the roof structure could be made. Still, it could be observed that the timber structure in the gable roof parts is a queen and king-post roof structure type with a queen post truss placed in the inferior part of the roof composed of a tie beam, two queen posts and a collar beam. Due to the height of the roof, an additional king post was placed above the collar beam connecting it with the ridge purlin. In order to better connect the inferior part of the roof to the king post, an passing brace was also included.

For the shed roof, the same structural type was used, but only one queen post was kept while the kingpost was placed in contact with the historic masonry wall connecting the straining beam and the tie beam (Table 5-19).

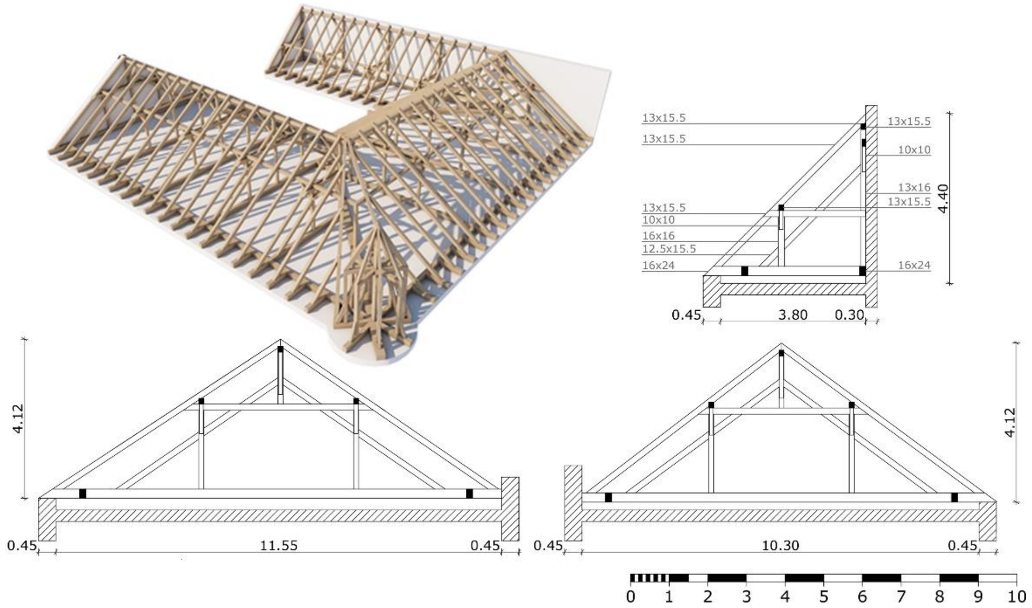


Fig. 5.16 "Victoria" Hotel – the roof structure

The symbolic analysis could only be performed in this case for the shed roof since not all the structural elements of the gable roof structure were visible. The analysis revealed that, since the building is an early 20th century construction, mainly static ratios were used. It was observed that the position of all structural elements is defined by 1/1 ratios, except for the rectangle defined by the posts and the tie and collar beam, which is a √2 rectangle (Fig. 5.17). No additional symbolic elements were identified for this roof structure as a mark of the craftsman (Table 5-18).

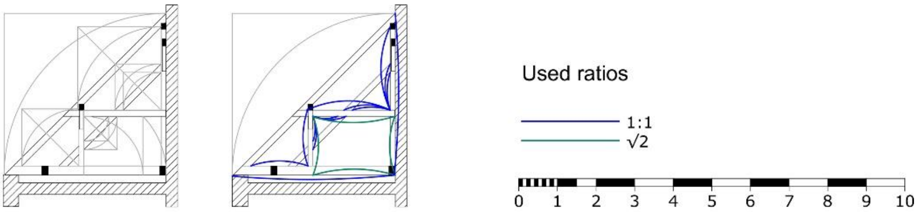


Fig. 5.17 "Victoria" Hotel – used ratios

The roof structure is in a good state of conservation, presenting only a little decay of the rafters and ridge purlin due to water infiltration caused by the poor maintenance of the roof envelope and due to the presence of small gaps between the ceramic tiles (Fig. 5.18). On the outside, the ridge is also presenting a slight decay caused by poor maintenance (Table 5-20).

Fig. 5.18 “Victoria” Hotel – decay of the timber roof structure

The observations were included in the assessment form and the corresponding response selected for each of the evaluated criteria.

Table 5-16 “Victoria” Hotel – urban value

Criterion		Response	Score
Value of the urban area		Protected urban area	2
Urban analysis	Position of the building	Integrated in urban alignment	2
	Frontage	Continuous front	2
	Height	Variable Height at cornice	0
	Alignment	Street alignment	4
Geometry	Roof shape	Mix of shapes	4
	Roof pitch	45°-65°	4

Table 5-17 “Victoria” Hotel – architectural value

Criterion		Response	Score
Historical analysis of the building	Dating	1900-1912	3
	Monument	Not a monument	0
Building analysis	Height	<=P+3	2
Functional analysis	Original function	Public function	2
	Changes in function	No	2
	Contemporary function	Public function	2
Aesthetic analysis	Architectural style	Secession	5
Geometry of the roof structure	Roof shape	Mix of shapes	5
Exterior appearance	Roof envelope	Ceramic tile	5

Table 5-18 "Victoria" Hotel – symbolic value

Criterion		Response	Score
Ratio between the roof and the building		No ratio	0
Ratio between structural elements	Height/width of the roof structure	Dynamic ratio (√2, √3, √5)	3
	Position of joints defined by	Dynamic ratio (√2, √3, √5)	3
	Position of purlins defined by	Dynamic ratio (√2, √3, √5)	3
Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
	Elements with great symbolic value	No symbolic elements	0

Table 5-19 "Victoria" Hotel – structural value

Criterion		Response	Score
Roof structure	Structural typology	Complex single typology roof structure	2
	Construction system	Purlin roof structure	2
	Structural style	Eclectic Roof	2
Structural elements	Truss typology	Main and secondary trusses	2
	Tie beam	Only main trusses	1
	Hanging device	Only main trusses	1
	Hanging device with	Collar beam	1
	Special structures	Towers	2
	Rigidity enhancing system	No rigidity enhancing system	0
Joint typology	Joining materials	Wood dowel	3
	Used traditional joints	Mortise and tenon	3

Table 5-20 "Victoria" Hotel – value reduction factors

Criterion		Response	Score
Decay visible from the outside	Decay of the ridge	<10%	1
	Decay of the cornice	No decay	0
	Decay at the chimney	No decay	0
	Decay of the envelope	No visible decay of the envelope	0
Decay of the roof structure	Decay of the tie-beam	<10%	1
	Decay of the compound rafter	<10%	1
	Decay of the rafter	10-20%	2
	Decay of the purlins	10-20%	2
	Decay of the straining beam	No straining beam / No decay	0
	Decay of the collar beam	<10%	1
	Decay of the counterbrace	<10%	1
	Roof to wall connection	Semi-rigid	1

Results

- The urban value of the roof structure – 71 points;
- The architectural value of the roof structure - 75 points;
- The symbolic value of the roof structure - 44 points;
- The structural value of the roof structure - 69 points;
- The ideal value of the roof structure - 67 points;
- The decay index of the roof structure - 23 points;
- The climate change vulnerability of the roof structure – 51 points;
- The value index of the roof structure - 64 points;
- The vulnerability of the roof structure - 55 points.

Conclusions:

- The roof structure has moderate value;
- The roof structure has a predominant architectural value;

- The roof structure is reducing the horizontal displacement of the building;
- The effect of the roof structure on the seismic behaviour of the building is moderate;
- The roof structure has a moderate vulnerability.

5.3.2.3 I.C. Bratianu high school - roof with a high symbolic value

The main building of the I.C. Bratianu high school was initially built in the middle of the 18th century at the border of the old fortress, close to the city wall. When the fortification was torn down at the end of the 19th century, the importance of the building changed and a new building, a girl school of the Sisters of Notre Dame order, was built in its place (Fig. 5.19). Due to its new position in the city and high visibility from the new main streets, the building was transformed into an eclectic building of the new urban developments, being visually linked to the new Victory square but also the historic Huniade castle (Table 5-21).

The building presents, even from the outside two slightly different sectors. The differences are visible on the façade, in the position of the windows and used decoration but also in the roof structure type (Fig. 5.20). Despite their differences, both sides of the roof are presenting a steep roof slope and a queen and king post roof structure suitable to shape this type of pitch (Table 5-22).

Fig. 5.19 I.C. Bratianu high school – urban context and relationship with the street and building

On the left side of the building, the typical queen and king post roof structure, with tie-beam, hanging posts, collar beam and passing braces is slightly modified. Additional collar ties were used in order to better connect the queen posts to the compound rafters and rafters. The tie beams, in this case, are also part of the wooden beam flooring. The roof structure from this part seemed to be a reused one, due to the presence of different unused mortise wholes placed in the passing braces and queen posts, which are a sign that in the original roof structure also a longitudinal rigidity enhancing system was used.

The other side is presenting a queen and king post roof structure without any alterations. All the specific elements for this type can be found, from hanging device, tie beam which is independent of the timber beam flooring, collar beam, passing braces and rafters (Table 5-24).

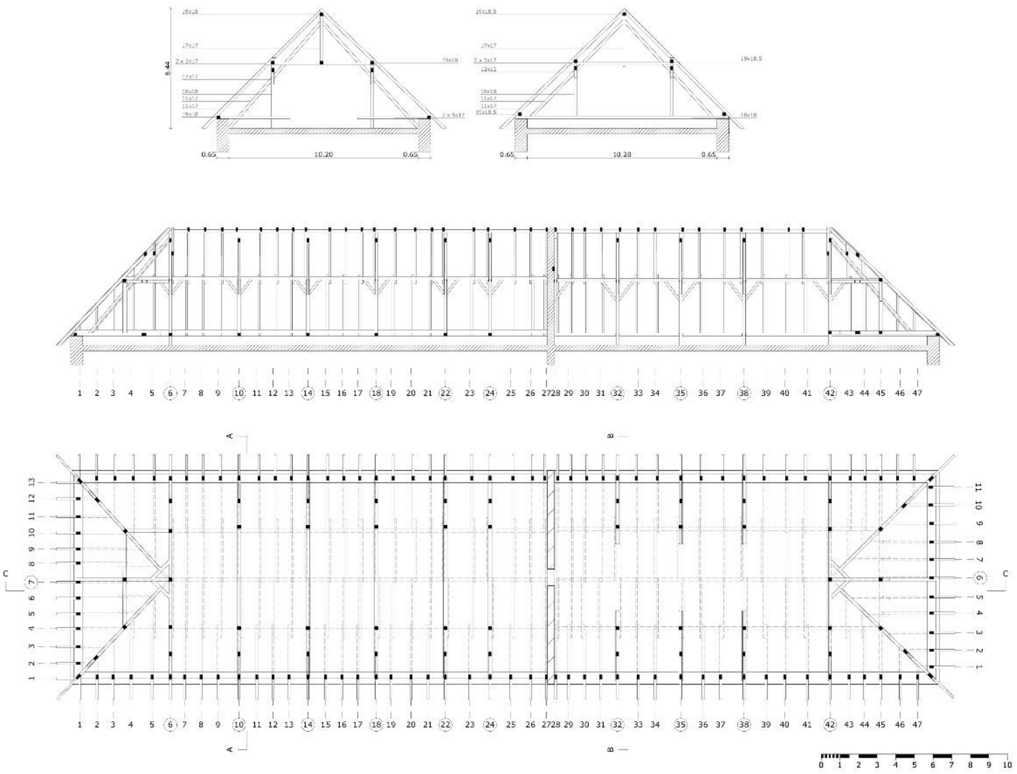


Fig. 5.20 I.C. Bratianu high school – the roof structure

The same changes were also observed during the geometric analysis (Table 5-23) of the roof structure (Fig. 5.21). The analysis showed that even though, the same structural elements defined the ratios, the used proportions are entirely different, marking the changes in the philosophy of the craft guilds.

The left area of the roof structure is mainly marked by golden ratios (Φ), defining the position of the hanging posts related to the tie-beam and collar beam, the intersection of the passing brace or the collar ties with the queen posts and collar beam.

The right side, on the other hand, is marked by both dynamic (√2, √3 and √5) and static ratios (1/1), for the same intersections of structural elements. No sacred ratios were identified in this part.

The roof structure is in a good state of conservation, presenting only a little decay of the rafters and ridge purlin due to water infiltration caused by the poor maintenance of the roof envelope and due to the presence of small gaps between the ceramic tiles. On the outside, the ridge is also presenting a slight decay caused by poor maintenance (Table 5-25).

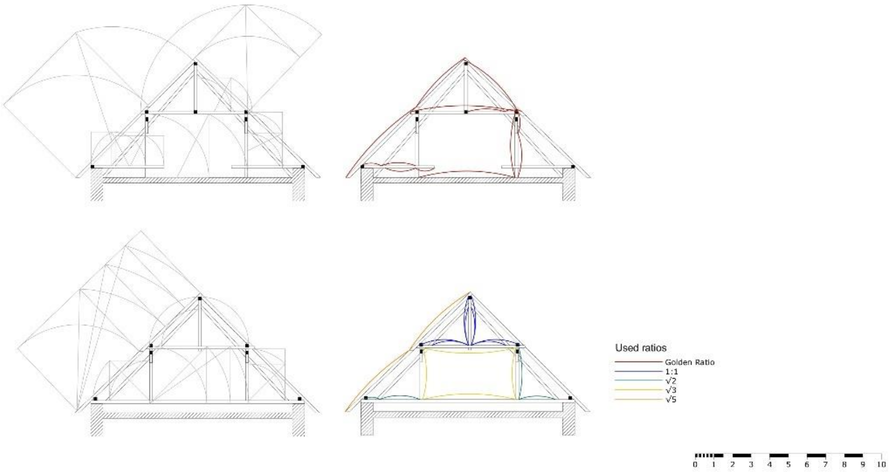


Fig. 5.21 I.C. Bratianu high school – used ratios

Table 5-21 I.C. Bratianu high school – urban value

Criterion		Response	Score
Value of the urban area		Protected urban area	2
Urban analysis	Position of the building	Marks the urban silhouette	3
	Frontage	Continuous front	2
	Height	Variable Height at cornice	0
	Alignment	Street alignment	4
Geometry	Roof shape	Gable roof	2
	Roof pitch	30º-45º	3

Table 5-22 I.C. Bratianu high school – architectural value

Criterion		Response	Score
Historical analysis of the building	Dating	1900-1912	3
	Monument	Not a monument	0
Building analysis	Height	<=P+3	2
Functional analysis	Original function	Public function	2
	Changes in function	No	2
	Contemporary function	Public function	2
Aesthetic analysis	Architectural style	Eclectic style	3
Geometry of the roof structure	Roof shape	Gable roof	2
Exterior appearance	Roof envelope	Ceramic tile	5

Table 5-23 I.C. Bratianu high school – symbolic value

Criterion		Response	Score
Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
Ratio between structural elements	Height/width of the roof	Golden ratio (Φ)	4
	Position of joints defined by	Golden ratio (Φ)	4
	Position of purlins defined by	Golden ratio (Φ)	4
Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
	Elements with great symbolic value	No symbolic elements	0

Table 5-24 I.C. Bratianu high school – structural value

	Criterion	Response	Score
Roof structure	Structural typology	Mix of structural typologies	3
	Construction system	Purlin roof structure	2
	Structural style	Eclectic Roof	2
Structural elements	Truss typology	Main and secondary trusses	2
	Tie beam	Only main trusses	1
	Hanging device	Only main trusses	1
	Hanging device with	Collar beam	1
	Special structures	No special structures	0
	Rigidity enhancing system	No rigidity enhancing system	0
Joint typology	Joining materials	Wood dowel	3
	Used traditional joints	Mortise and tenon	3

Table 5-25 I.C. Bratianu high school – value reduction factors

	Criterion	Response	Score
Decay visible from the outside	Decay of the ridge	No decay	0
	Decay of the cornice	No decay	0
	Decay at the chimney	No decay	0
	Decay of the envelope	No visible decay of the envelope	0
Decay of the roof structure	Decay of the tie-beam	<10%	1
	Decay of the compound rafter	<10%	1
	Decay of the rafter	<10%	1
	Decay of the purlins	10-20%	2
	Decay of the straining beam	No straining beam / No decay	0
	Decay of the collar beam	<10%	1
	Decay of the counterbrace	<10%	1
	Roof to wall connection	Semi-rigid	1

Results

- The urban value of the roof structure – 64 points;
- The architectural value of the roof structure - 64 points;
- The symbolic value of the roof structure - 65 points;
- The structural value of the roof structure - 63 points;
- The ideal value of the roof structure - 64 points;
- The decay index of the roof structure - 19 points;
- The climate change vulnerability of the roof structure – 48 points;
- The value index of the roof structure - 61 points;
- The vulnerability of the roof structure - 51 points;

Conclusions:

- The roof structure has moderate value;
- The roof structure has a predominant symbolic value;
- The roof structure is reducing the horizontal displacement of the building;
- The effect of the roof structure on the seismic behaviour of the building is moderate;
- The roof structure has a moderate vulnerability.

5.3.2.4 The St. George building – roof with a high structural value

The St. George square building is placed close to one of the main historic squares of the city, the St. George square, along a rather narrow street with an alignment of trees placed along the pedestrian area (Fig. 5.22). The building comprises all the specific features of 18th century buildings from the city centre of Timisoara, presenting a clear Baroque architectural style, the main building placed towards the street and a narrow annexe building placed towards the inner courtyard.

Since the street is narrow, the roof can only be perceived from the other side of the road and partially from the square placed nearby. At the same time, due to the presence of the alignment of trees, today, the building and the roof are almost entirely covered, and the passerby cannot perceive their aesthetics. In the meanwhile, due to its bad state of conservation caused by the lack of maintenance, the building was partially demolished, and a new hotel is planned to be built in its place (Table 5-26).

Fig. 5.22 St. George building – urban context and relationship with the street and building

The roof structure is a type characteristic for 18th century buildings in Timisoara, being composed of inner rafters, placed parallel with the rafters, forming together with the straining beam, the straining device of the structure (Fig. 5.23). In order to increase the rigidity of the roof structure also in the longitudinal direction, ridge, intermediate and eaves purlins were used, and diagonal compound rafters were placed in the plane of the rafters (Table 5-27).

In this case, the only difference between the main and secondary trusses of the roof structure is the missing straining device (inner rafters, straining beam and counterbraces) for the secondary frames. Tie beams and collar beams are still used for all the trusses (Table 5-29).

The geometric analysis (Fig. 5.24) of the roof structure showed that the position of the main structural elements is defined by golden ratios (Φ) but also by dynamic ones (√2), highlighting the position of the straining and collar beam, but also that of the position of the intermediate purlins and that of the counterbraces joints (Table 5-28).

The roof structure is in a good state of conservation, presenting only a little decay of the rafters due to water infiltration caused by the poor maintenance of the roof envelope and due to the presence of small gaps between the ceramic tiles. On the outside, the ridge also presents a slight decay caused by poor maintenance (Table 5-30).

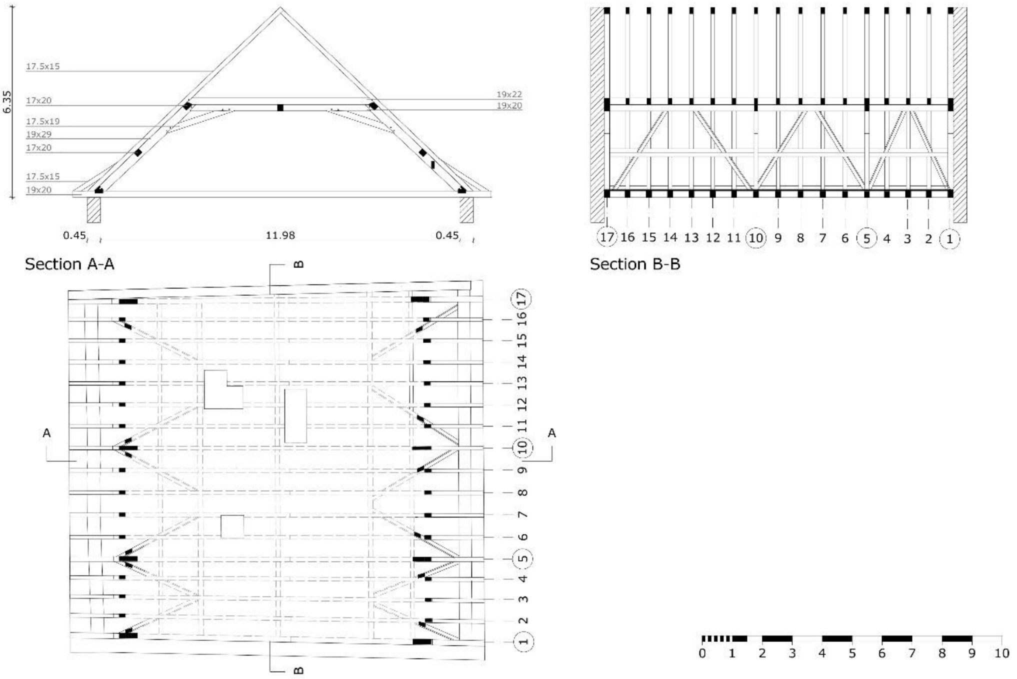


Fig. 5.23 St. George building – the roof structure

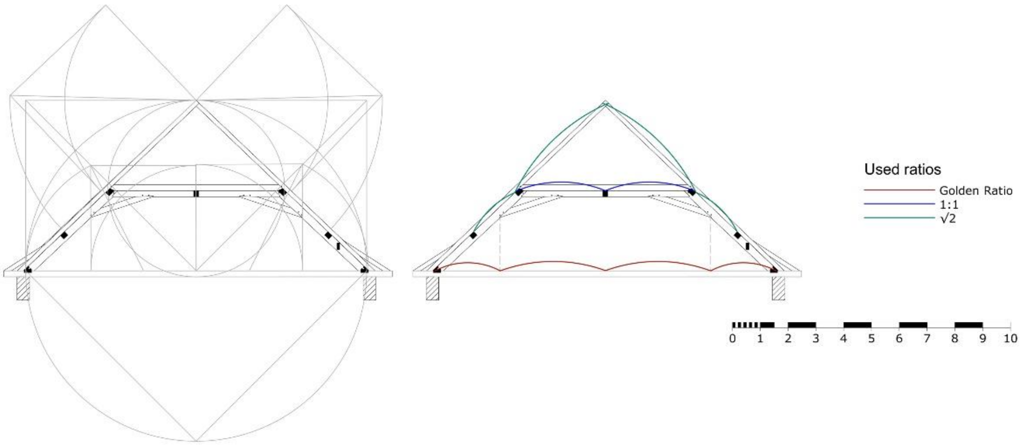


Fig. 5.24 St. George building – used ratios

Table 5-26 St. George building – urban value

Criterion		Response	Score
Value of the urban area		Protected urban area	2
Urban analysis	Position of the building	Integrated in urban alignment	2
	Frontage	Continuous front	2
	Height	Constant Height at cornice	2
	Alignment	Street alignment	4
Geometry	Roof shape	Gable roof	2
	Roof pitch	30º-45º	3

Table 5-27 St. George building – architectural value

Criterion		Response	Score
Historic analysis of the building	Dating	<1893	5
	Monument	Not a monument	0
Building analysis	Height	<=P+3	2
Functional analysis	Original function	Public function	2
	Changes of function	No	2
	Contemporary function	Public function	2
Aesthetic analysis	Architectural style	Baroque	5
Geometry of the roof structure	Roof shape	Gable roof	2
Exterior appearance	Roof envelope	Ceramic tile	5

Table 5-28 St. George building – symbolic value

Criterion		Response	Score
Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
	Position of joints defined by	Golden ratio (Φ)	4
	Position of purlins defined by	Golden ratio (Φ)	4
Symbolic aesthetics	Inscriptions	Craftsmen sign	5
	Elements with great symbolic value	No symbolic elements	0

Table 5-29 St. George building – structural value

Criterion		Response	Score
Roof structure	Structural typology	Complex single typology roof structure	2
	Construction system	Purlin roof structure	2
	Structural style	Baroque Roof	3
Structural elements	Truss typology	Main and secondary trusses	2
	Tie beam	Every truss	2
	Hanging device	Every truss	2
	Hanging device with	Collar beam	1
	Special structures	No special structures	0
	Rigidity enhancing system	Longitudinal system in rafter plane	2
Joint typology	Joining materials	Wood dowel	3
	Used traditional joints	Mortise and tenon	3

Table 5-30 St. George building – value reduction factors

Criterion		Response	Score
Decay visible from the outside	Decay of the ridge	<10%	1
	Decay of the cornice	No decay	0
	Decay at the chimney	No decay	0
	Decay of the envelope	No visible decay of the envelope	0
Decay of the roof structure	Decay of the tie-beam	<10%	1
	Decay of the compound rafter	<10%	1
	Decay of the rafter	<10%	1
	Decay of the purlins	<10%	1
	Decay of the straining beam	<10%	1
	Decay of the collar beam	<10%	1
	Decay of the counterbrace	No counterbrace / No decay	0
	Roof to wall connection	Rigid	0

Results

- Urban value of the roof structure – 75 points;
- Architectural value of the roof structure - 73 points;
- Symbolic value of the roof structure - 79 points;
- Structural value of the roof structure - 80 points;
- Ideal value of the roof structure - 77 points;
- Decay index of the roof structure - 15 points;
- The climate change vulnerability of the roof structure – 46 points;
- Value index of the roof structure - 75 points;
- Vulnerability of the roof structure - 58 points;

Conclusions:

- The roof structure has high value;
- The roof structure has a predominant structural value;
- The roof structure is reducing the horizontal displacement of the building;
- The effect of the roof structure on the seismic behaviour of the building is very high;
- The roof structure has a moderate vulnerability.

5.3.2.5 Result analysis

The form proved out to offer results which correspond with the preliminary performed analysis (Table 5-31). Still, it was observed that when considering the predominant value of a roof structure, 18th century buildings presents high values and only slight differences between the obtained scores, like in the case of the St. George square building. The differences between the obtained scores increase in time, the Lloyd palace being a good example of building with a high urban value but low symbolic value.

The observed changes of the predominant features are consistent with the evolution of architectural principles and the importance of the acknowledgement of the relation between the professionals working on designing a building. 18th century buildings were built by craftsman which considered the building as a whole, linked to its context and saw each structural element as part of a general view. Therefore, urban planning, architectural, symbolic and structural principles have approximately the same influence on the perception of the roof. In time, this equilibrium in architecture faded, and each building presents a dominant feature or is strongly influenced by one principle.

Concerning the ideal, real and vulnerability index of the roof structures, only little differences could be observed between the four chosen reference roof structures, since all of them are important buildings of Timisoara.

Table 5-31 Obtained score for the four chosen reference roof structures

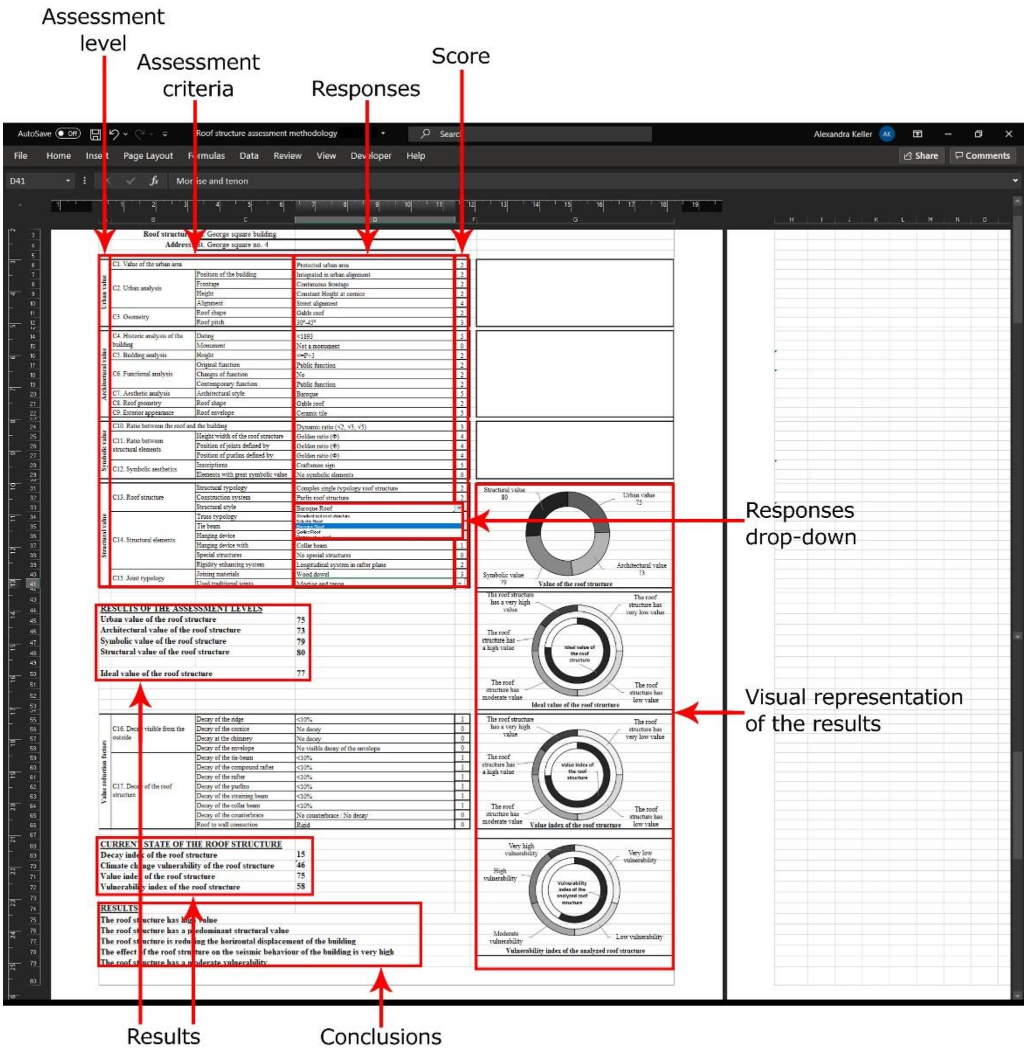
	Lloyd palace	"Victoria" Hotel	I.C. Bratianu high school	St. George building
Urban value	100	71	64	75
Architectural value	61	75	64	73
Symbolic value	53	44	65	79
Structural value	70	69	63	80
Ideal value	73	67	64	77
Decay index	23	23	19	15
Value index	69	64	61	75
Vulnerability	58	55	51	58

5.3.3 Development of an easy-to-use assessment form

In order to be able to perform and quick and easy assessment of a roof structure by using the proposed assessment methodology two different forms were developed, one which could be filled out by using a computer or laptop and the second one which is a mobile application (app) and has the advantage that it can be filled out even on-site.

The computer-based form was developed by using Microsoft Office Excel (Fig. 5.25). It is a simple table which is partially filled out with all the information needed for the assessment. The form is organised considering the five levels of the assessment with each criterion marked in order for the assessor to identify the requested information quickly. For each criterion, a drop-down list of possible responses is offered. The considered response is selected by simply clicking on it in the list, and the form is automatically calculating the corresponding score and displaying it next to the response. For each assessed criterion, the form is also automatically calculating all the value and vulnerability indexes and updating their visual representation, offering the assessor the possibility to see in real-time how each choice is influencing the final results. Finally, at the end of the form, based on all the input data, the form is displaying the assessment conclusions.

If however, the assessment will be performed in a different location with other valuable features and threats, the form can also be easily altered, the responses changed, and the scores adapted. Any changes made to the main list of criteria and scores will automatically be updated, leaving the form ready to be used.



Roof structure: St. George square building
Address: St. George square no. 4

Urban value	C1. Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30°-45°	3
Architectural value	C4. Historic analysis of the building	Dating	<1893	5
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Baroque	5
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	C10. Ratio between the roof and the building		Dynamic ratio (√2, √3, √5)	3
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	Craftsmen sign	5
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Baroque Roof	
	C14. Structural elements	Truss typology		
		Tie beam		
		Hanging device		
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	Longitudinal system in rafter plane	2
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**75**
Architectural value of the roof structure	**73**
Symbolic value of the roof structure	**79**
Structural value of the roof structure	**80**
Ideal value of the roof structure	**77**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	<10%	1
		Decay of the straining beam	<10%	1
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	No counterbrace / No decay	0
		Roof to wall connection	Rigid	0

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**15**
Climate change vulnerability of the roof structure	**46**
Value index of the roof structure	**75**
Vulnerability index of the roof structure	**58**

RESULTS

The roof structure has high value
The roof structure has a predominant structural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is very high
The roof structure has a moderate vulnerability

Fig. 5.25 Excel form of the assessment methodology (editable)

The smartphone app approaches the assessment slightly differently. It was developed by using Microsoft Power Apps, a software which enables the development of a platform without using any code and offering the possibility to link the obtained data to various other platforms, in this case, Google Sheets. The disadvantage of the app builder is that it can only be used if permission is granted to both the app and the Google Drive document by the developer and the assessor owns an institutional Office 365 account.

Since the form had to fit on a smaller screen, in this case, the information is offered to the assessor in steps, and no complete overview of the final results is possible. At the beginning, a welcome message appears on the screen stating: "The following roof structure assessment procedure includes complex criteria which determine the value and vulnerability of the roof structure from a multidisciplinary and transdisciplinary point of view, based on the importance of the roof structure in its context, its valuable features and influence of the surrounding environmental factors on its state of conservation.”. The assessor is subsequently invented to start evaluating the roof structure by touching the button placed on the bottom of the page (Fig. 5.26a).

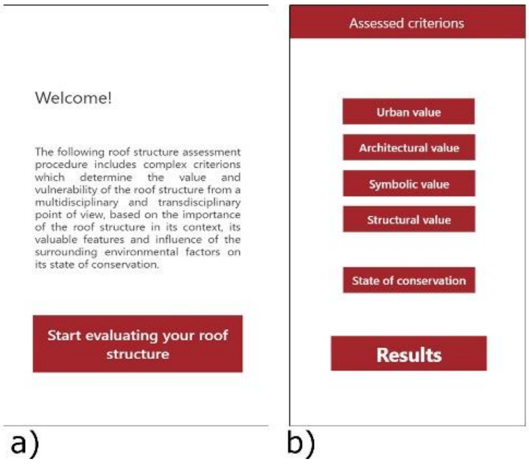
The second screen includes an overview of the considered assessment levels. By clicking on any assessment level button, the corresponding page is opened, and the roof structure can be analysed from the respective point of view (Fig. 5.26b). Throughout the evaluation, the assessor can come back to the page by touching the “Home” button.

Each assessment level (Fig. 5.27) includes a clear list of each criterion of the level and a corresponding drop-down list with the responses to choose. The response is selected by simply touching it in the list. In order to make the assessment more user-friendly, near each criterion, an “Information” button is placed, which includes additional explanation which can be useful during the decision-making process (Fig. 5.28). This feature of the app is useful if, for example, an owner of an attic would like to assess the roof structure and is not familiar with specific terms.

The app is connected to a Google Sheet, which takes over the selected answers, calculates the corresponding score and the value and vulnerability of the roof structure. By touching the “Show results” button (Fig. 5.29a), the app is displaying all the results from the Google Sheet (Fig. 5.29b), and clearly stating the main conclusions of the assessment (Fig. 5.29c).

Subsequently, the results can also be seen in the Google Sheet in a similar format to the Excel form (Fig. 5.30).

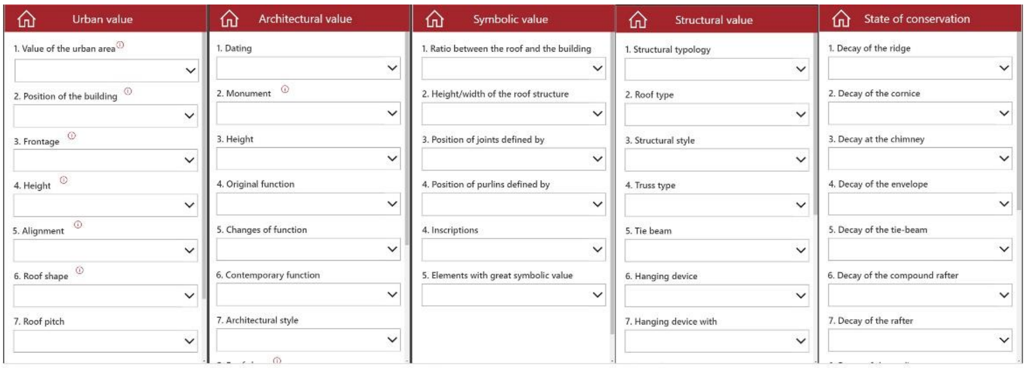
Compared to the Excel form, the app is more user-friendly, offering the possibility to obtain additional information about each assessed criterion. Still, the main disadvantage of the app is that is not able to offer a visual representation directly in the “Results” page and that changes of the considered criteria, responses and corresponding scores are somewhat challenging to make by other users.



Welcome!

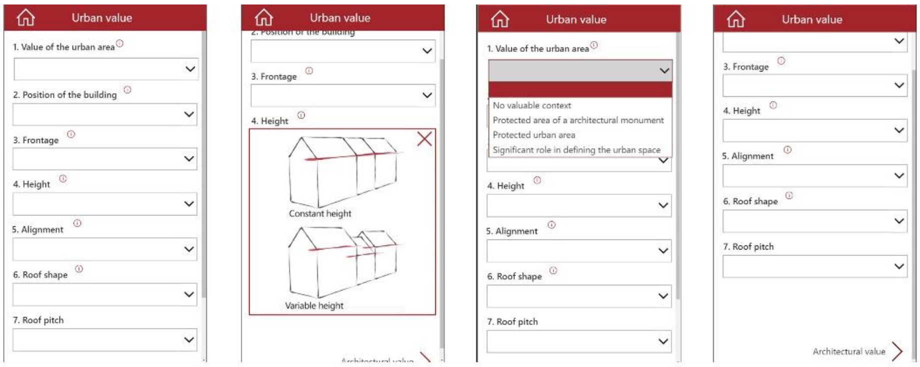
The following roof structure assessment procedure includes complex criterions which determine the value and vulnerability of the roof structure from a multidisciplinary and transdisciplinary point of view, based on the importance of the roof structure in its context, its valuable features and influence of the surrounding environmental factors on its state of conservation.

Fig. 5.26 First pages of the assessment app, a) Welcome page; b) Overview page of the assessment levels and the results button



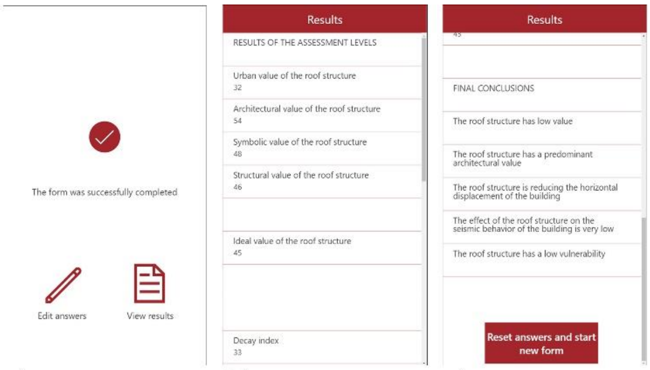
a) b) c) d) e)

Fig. 5.27 Form for each assessment level a) Urban value; b) Architectural value; c) Symbolic value; d) Structural value; e) Value reduction factors



a) b) c) d)

Fig. 5.28 How to use the app a) Main page of the assessment level; b) Displaying addition information about the criterion; c) Drop-down list of possible answers; d) End of the assessment level and "Next page" button



The form was successfully completed

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure
32

Architectural value of the roof structure
54

Symbolic value of the roof structure
48

Structural value of the roof structure
46

Ideal value of the roof structure
45

Decay index
33

FINAL CONCLUSIONS

The roof structure has low value

The roof structure has a predominant architectural value

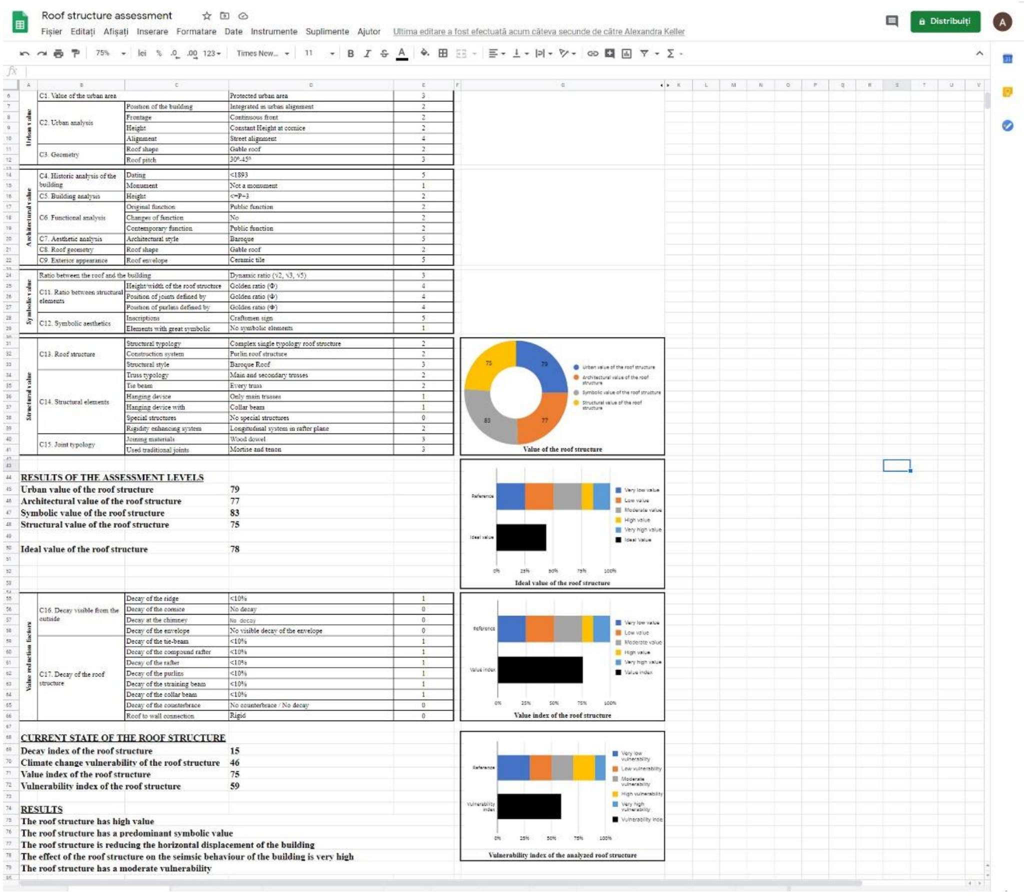
The roof structure is reducing the horizontal displacement of the building

The effect of the roof structure on the seismic behavior of the building is very low

The roof structure has a low vulnerability

a) b) c)

Fig. 5.29 How to use the app a) Confirmation of the finished assessment and submission of the responses; b Displaying the results; c) Main conclusions and "Reset answers and start new form" button



Urban value	C1. Value of the urban area		Protected urban area	3
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous front	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30°-45°	3
Architectural value	C4. Historic analysis of the building	Dating	<1893	5
		Monument	Not a monument	1
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Baroque	5
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		Dynamic ratio (√2, √3, √5)	3
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	Craftsmen sign	5
		Elements with great symbolic	No symbolic elements	1
Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Baroque Roof	3
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Every truss	2
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	Longitudinal system in rafter plane	2
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**79**
Architectural value of the roof structure	**77**
Symbolic value of the roof structure	**83**
Structural value of the roof structure	**75**
Ideal value of the roof structure	**78**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	<10%	1
		Decay of the straining beam	<10%	1
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	No counterbrace / No decay	0
		Roof to wall connection	Rigid	0

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**15**
Climate change vulnerability of the roof structure	**46**
Value index of the roof structure	**75**
Vulnerability index of the roof structure	**59**

RESULTS

The roof structure has high value
The roof structure has a predominant symbolic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seimsic behaviour of the building is very high
The roof structure has a moderate vulnerability

Fig. 5.30 Google Sheet form of the assessment methodology (only for visualization of the results)

5.3.4 Results

Subsequently, all the other 14 assessed roof structures (Table 5-32) were analysed in the same way as the reference roof structures and all the observations introduced into the assessment form.

In order to better understand the importance of the roof structures from all the point of views, the buildings were sorted based on their construction year, and the obtained results compared for each assessment level. In this way, the evolution in time of the assessed principles could be observed and identified if specific trends are visible.

Table 5-32 Assessed roof structures and used abbreviations

	Roof structure	Abbreviation used
1	St. George square building	Roof 1812
2	Union square building	Roof 1821
3	I.C. Bratianu high school - wing A	Roof 1828 (1)
4	I.C. Bratianu high school - wing C	Roof 1828 (2)
5	Great Synagogue of Timisoara	Roof 1863
6	Archdukes house	Roof 1870
7	Residential building 4	Roof 1889
8	Residential building 2	Roof 1900 (1)
9	Residential building 3	Roof 1900 (2)
10	I.C. Bratianu high school - wing B	Roof 1900 (3)
11	Residential building 6	Roof 1900 (4)
12	Karl Kunz palace	Roof 1903 (1)
13	Residential building 5	Roof 1903 (2)
14	Chemistry faculty	Roof 1905
15	"Victoria" Hotel	Roof 1907
16	Lloyd Palace	Roof 1912
17	Löfler Palace	Roof 1913
18	Residential building 1	Roof 1940

First, the urban value of each roof structure was assessed (Table 5-33, Fig. 5.31). For this level of the assessment, the most evident evolution of the importance of urban planning principles in time is visible. 18th and beginning of the 19th century buildings, despite being placed along narrow streets and being partially invisible, their context is coherent, and the roof structures are placed in urban areas with a high historical value. Therefore, their urban value is high. In the same period, but towards the exterior of the old fortress, roof structures have a lower pitch and are only slightly visible despite their new urban context. Still, buildings with a high cultural and historical value like the great synagogue from the old fortress which is an individual building, not integrated into the continuous frontage formed by the neighbouring buildings, stands out despite the narrow street. This offers the building automatically a high urban value.

Towards the end of the 19th century, the importance of the roof structure in defining urban space is low since in this period buildings focus mainly on their aesthetics. Despite being placed in new urban developments which could favour the visibility of the roof, wide streets, small squares or urban parks, the roofs are most of the time hidden either behind the alignment of trees placed along the pedestrian area or behind high walls, placed towards the street, specific for the 1900 architectural style.

The importance of urban value principles and the urban context is once again highlighted at the beginning of the 20th century when the maximum urban value index is obtained for the considered buildings from the Victory square. Their position and the dimension of the roofs mark the urban space, which ultimately leads to their high urban value.

Residential buildings, on the other hand, present, no matter the period in which they were built, a low urban value. Since they have a low height and are integrated into the street alignment, their roofs do not stand out, and the roof does not influence the perception of the building in any way.

Table 5-33 Score obtained for each criterion and total urban value score

Criterion		Roof 1812	Roof 1821	Roof 1828 (1)	Roof 1828 (2)	Roof 1863	Roof 1870	Roof 1889	Roof 1900 (1)	Roof 1900 (2)	Roof 1900 (3)	Roof 1900 (4)	Roof 1903 (1)	Roof 1903 (2)	Roof 1905	Roof 1907	Roof 1912	Roof 1913	Roof 1940
C1.	1.	2	4	2	2	4	2	0	2	0	2	2	0	2	2	2	4	4	2
C2.	2.	2	2	2	3	1	2	2	2	2	3	2	2	2	3	2	4	2	2
	3.	2	2	2	2	2	2	2	0	2	2	2	2	2	2	2	2	2	0
	4.	2	2	0	0	2	0	0	0	0	0	0	2	2	0	0	2	2	0
	5.	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
C3	6.	2	2	2	2	3	2	2	2	2	2	4	4	2	4	4	4	4	2
	7.	3	3	3	3	3	3	3	2	3	3	3	3	3	3	4	4	4	2
Urban value		75	82	61	64	82	61	54	43	54	64	68	75	75	71	71	100	93	43

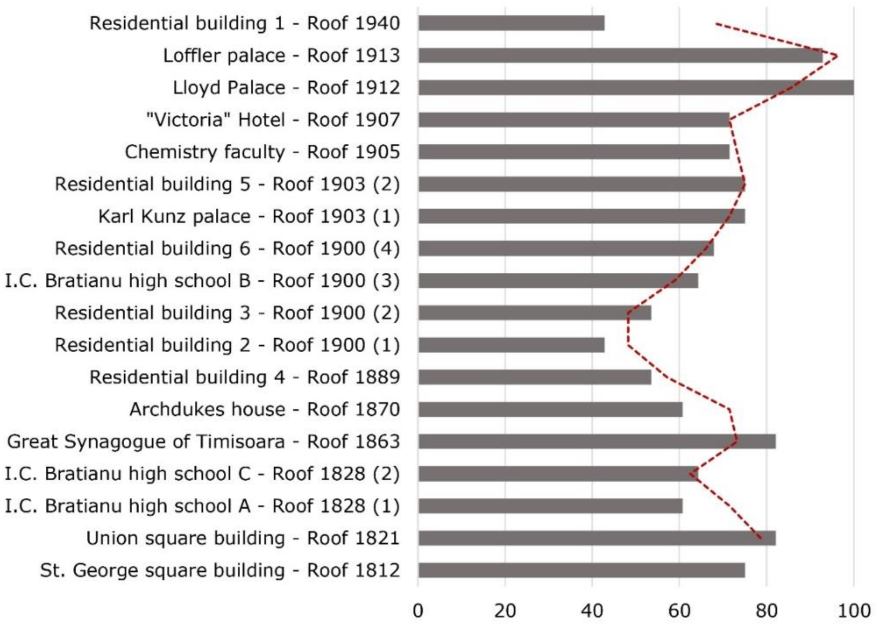


Fig. 5.31 Urban value score for each roof structure and trendline

The architectural value of the roofs and roof structures presents a completely different trend, partially connected to the way the roof is connected to the general appearance of the building and related to the way architectural principles are influencing the choice and shape of the roof structure (Table 5-34, Fig. 5.32). Since the chosen buildings have a high historical value and are key buildings of the history of Timisoara, the differences between the obtained scores are low. Still, it could be observed that residential buildings are presenting the lowest score while highly important buildings like the Great Synagogue where the presence of a central oculus is strongly influencing the roof structure type, obtain a greater score.

At the same time, it could be observed that the architectural value of the roof structure is also slightly rising at the beginning of the 20th century when the architectural requirements highly influence the used roof structure typology.

Table 5-34 Score obtained for each criterion and total architectural value score

Criterion		Roof 1812	Roof 1821	Roof 1828 (1)	Roof 1828 (2)	Roof 1863	Roof 1870	Roof 1889	Roof 1900 (1)	Roof 1900 (2)	Roof 1900 (3)	Roof 1900 (4)	Roof 1903 (1)	Roof 1903 (2)	Roof 1905	Roof 1907	Roof 1912	Roof 1913	Roof 1940
C4.	1.	5	5	4	4	5	4	2	4	3	3	3	3	3	3	3	2	2	1
	2.	0	0	0	0	3	0	0	0	0	0	0	0	0	0	0	0	0	0
C5.	3,	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
C6.	4.	2	2	2	2	3	1	1	2	1	2	1	1	1	2	2	2	2	2
	5.	2	2	2	2	2	2	2	1	2	2	2	2	2	2	2	2	2	2
	6.	2	2	2	2	3	1	1	1	1	2	1	1	1	2	2	2	2	2
C7.	7.	5	5	3	3	5	2	3	3	4	3	3	4	3	3	5	3	4	3
C8.	8.	2	2	2	2	4	2	2	5	2	2	5	5	2	5	5	5	5	2
C9.	9.	5	5	5	5	5	5	5	5	5	5	5	5	5	5	5	1	5	5
Architectural value score		73	73	66	66	96	57	54	64	59	64	63	66	57	71	75	61	71	60

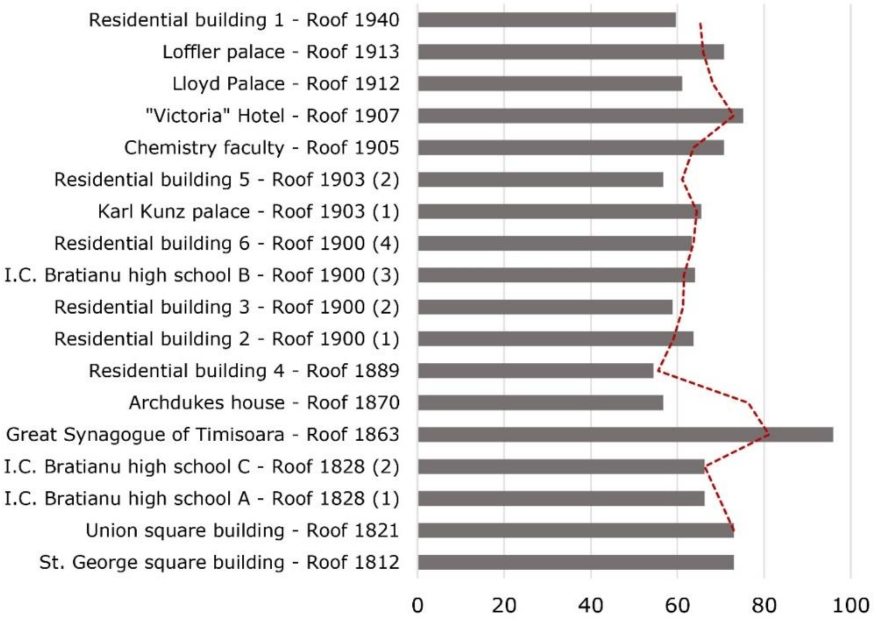


Fig. 5.32 Architectural value score for each roof structure and trendline

The symbolic value of the roof structure also presents an evident change in time, from roofs and roof structures highly completing the geometry of the buildings where the position of each joint is determined by complex ratios down to buildings where the roof only has a functional role, with few or no used ratios in defining the structure (Table 5-35, Fig. 5.33).

Table 5-35 Score obtained for each criterion and total symbolic value score

Criterion		Roof 1812	Roof 1821	Roof 1828 (1)	Roof 1828 (2)	Roof 1863	Roof 1870	Roof 1889	Roof 1900 (1)	Roof 1900 (2)	Roof 1900 (3)	Roof 1900 (4)	Roof 1903 (1)	Roof 1903 (2)	Roof 1905	Roof 1907	Roof 1912	Roof 1913	Roof 1940
C10.	1.	3	2	3	3	4	2	0	0	2	2	0	2	0	2	0	1	2	0
C11.	2.	4	4	4	4	4	2	2	4	2	4	2	2	2	4	3	2	2	0
	3,	4	4	4	4	4	3	2	4	3	4	2	3	2	3	3	2	2	3
	4.	4	4	4	4	4	2	2	2	2	4	2	2	2	2	3	2	2	0
C12	5.	5	5	2	2	0	2	0	2	2	2	2	2	0	2	2	2	2	0
	6.	0	0	0	0	4	0	0	0	0	0	0	0	0	0	0	4	0	0
Symbolic value score		79	75	69	69	83	44	25	48	44	65	32	44	25	53	44	53	40	13

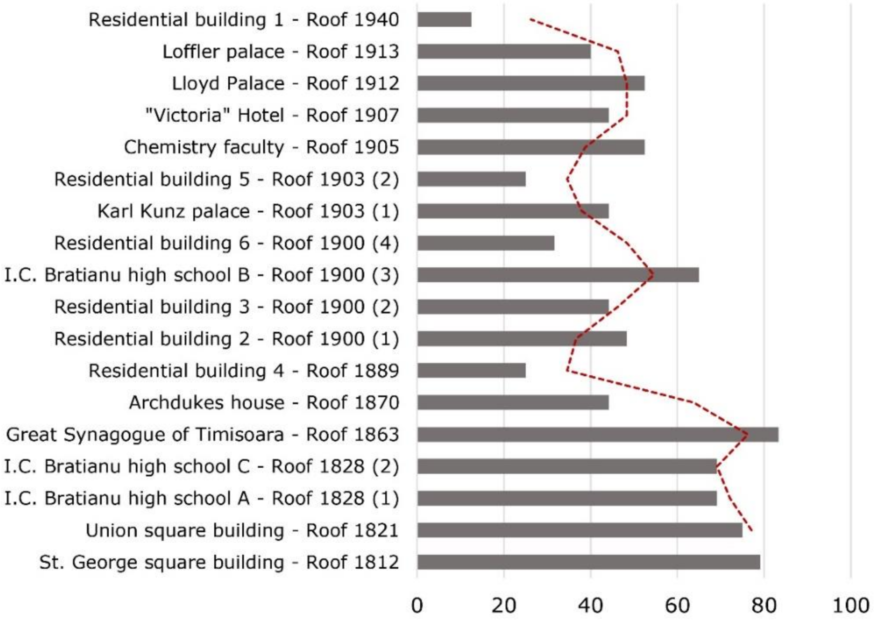


Fig. 5.33 Symbolic value score for each roof structure and trendline

A slight change in the trend can be observed at the beginning of the 20th century, in the case of the buildings from the Victory square, where, despite the use of static ratios, the symbolic value of the roof is increased due to the presence of additional decorative elements. At the same time, residential buildings prove out to have less symbolic value than buildings with public functions, no matter the period in which they were built.

The structural value presents a similar evolution in time as the connection to the architectural features (Table 5-36, Fig. 5.34). According to the analysis, it could be observed that 18th and early 19th century roof structures present structures with high complexity, while late 19th and 20th century roof structures bring forward a mix of structural types, being highly influenced by the exterior appearance of the roof.

Table 5-36 Score obtained for each criterion and total structural value score

Criterion		Roof 1812	Roof 1821	Roof 1828 (1)	Roof 1828 (2)	Roof 1863	Roof 1870	Roof 1889	Roof 1900 (1)	Roof 1900 (2)	Roof 1900 (3)	Roof 1900 (4)	Roof 1903 (1)	Roof 1903 (2)	Roof 1905	Roof 1907	Roof 1912	Roof 1913	Roof 1940
C13.	1.	2	2	2	2	3	2	1	3	3	3	3	3	2	3	2	3	3	2
	2.	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	3,	3	3	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
C14.	4.	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2	2
	5.	2	2	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
	6.	2	2	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1
	7.	1	1	0	2	1	1	1	1	1	1	1	1	1	1	1	2	2	1
	8.	0	1	1	1	1	1	0	0	1	0	0	1	0	1	2	1	1	0
	9.	2	2	2	2	0	2	0	0	0	0	0	0	0	0	0	1	1	0
C15.	10.	3	3	3	3	3	3	1	3	3	3	1	3	1	1	3	1	1	3
	11.	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
Structural value score		80	84	69	78	67	73	51	63	67	63	57	67	54	61	69	70	70	60

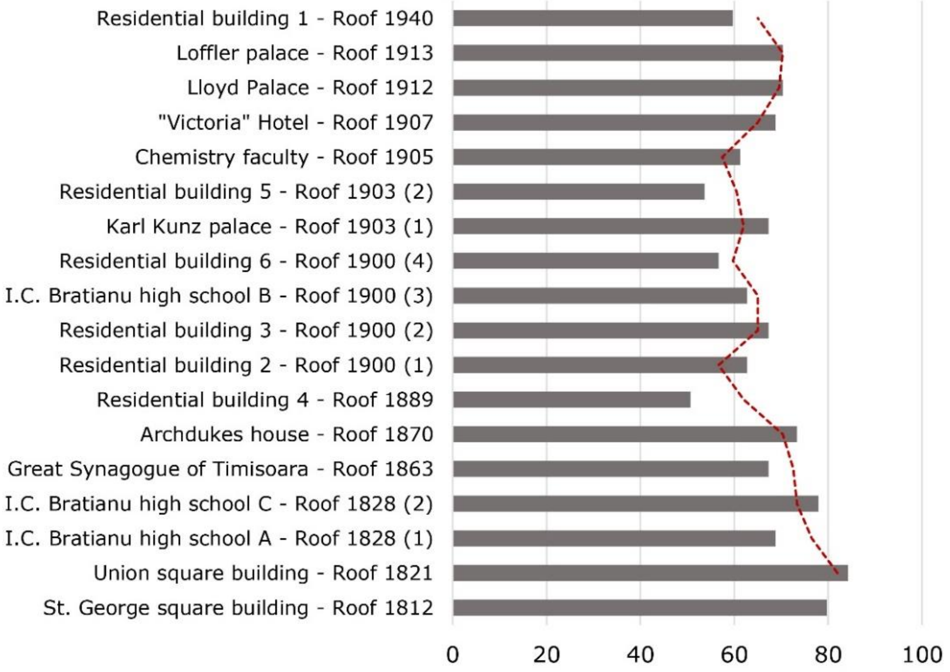


Fig. 5.34 Structural value score for each roof structure and trendline

When comparing the obtained scores for each of the four assessment levels, an apparent change of the predominant value of the roofs and a clear connection between various levels of the assessment can be observed (Fig. 5.35).

The comparison showed that early 19th century buildings present an equilibrium between the features influencing their total value. It is therefore highlighted that in this period no dominant feature is influencing the choice of roof and roof structure type and that roofs are meant to shape the top of the building in the urban context, complete the aesthetics of the building, contribute to the philosophical meaning of the structure, while presenting a complex structure.

Starting with the end of the 19th century, the importance of features starts to change. Roofs with high urban value prove out to lack in significant symbolic features and use no valuable ratios. Roof with high architectural value, on the other hand, also present a high structural value, since the architectural features prove out to significantly influence the structural characteristics.

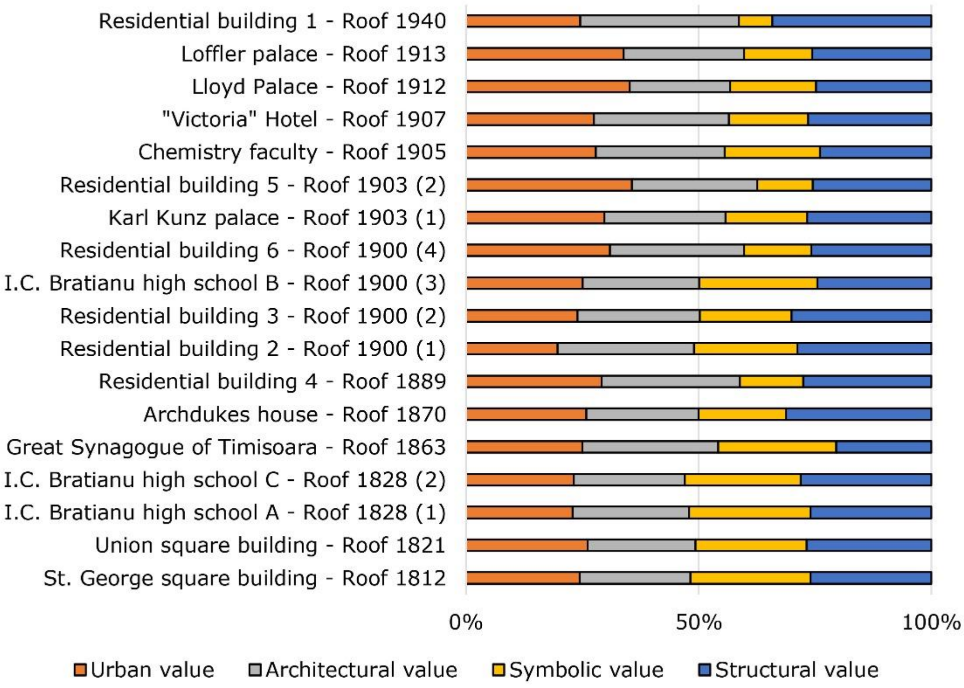


Fig. 5.35 Comparison of the obtained scores for all the assessment levels

The analysis of the decay showed that the assessed roof structures have a good state of conservation (Table 5-37, Fig. 5.36).

On the exterior, most of the assessed roofs are presenting no decay or slight decay in the ridge and cornice area. The roof structures, on the other hand are presenting decays on the rafter and the ridge purlins and wall plates being in connection to the historic masonry walls, in contact with humid surfaces. These decays are mostly present in residential buildings and are caused by the lack of maintenance and various damages of the roof envelope. The decay is, however, only local and reduce the cross-section of the timber elements under 20%. For all the other structural elements, a general loss of 10% of the cross-section, due to ageing, was considered even if no decay was visible.

Table 5-37 Score obtained for each criterion and total decay index

Criterion		Roof 1812	Roof 1821	Roof 1828 (1)	Roof 1828 (2)	Roof 1863	Roof 1870	Roof 1889	Roof 1900 (1)	Roof 1900 (2)	Roof 1900 (3)	Roof 1900 (4)	Roof 1903 (1)	Roof 1903 (2)	Roof 1905	Roof 1907	Roof 1912	Roof 1913	Roof 1940
C16.	1.	1	0	0	0	0	2	0	1	0	0	0	0	0	0	1	1	1	1
	2.	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0
	3,	0	0	0	1	0	1	0	0	0	0	0	0	0	0	0	0	0	0
	4.	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0	0	2	0
C17.	5.	1	1	1	1	0	1	1	1	2	1	1	2	1	1	1	1	1	1
	6.	1	1	1	1	0	2	1	1	2	1	1	2	1	1	1	1	1	1
	7.	1	2	1	1	1	2	2	2	2	1	2	2	1	1	2	2	2	2
	8.	1	1	1	2	1	2	1	2	3	2	2	3	1	1	2	1	2	2
	9.	1	1	0	0	0	0	0	1	0	0	0	0	0	0	0	1	1	1
	10.	1	1	0	1	0	2	1	0	2	1	1	2	1	1	1	1	2	0
	11.	0	0	0	0	0	1	1	1	1	1	1	1	1	1	1	1	1	1
	12.	0	0	0	1	0	1	1	1	1	1	1	1	1	1	1	1	1	1
Decay index		15	15	8	21	4	38	19	23	29	19	21	29	17	17	23	23	31	23

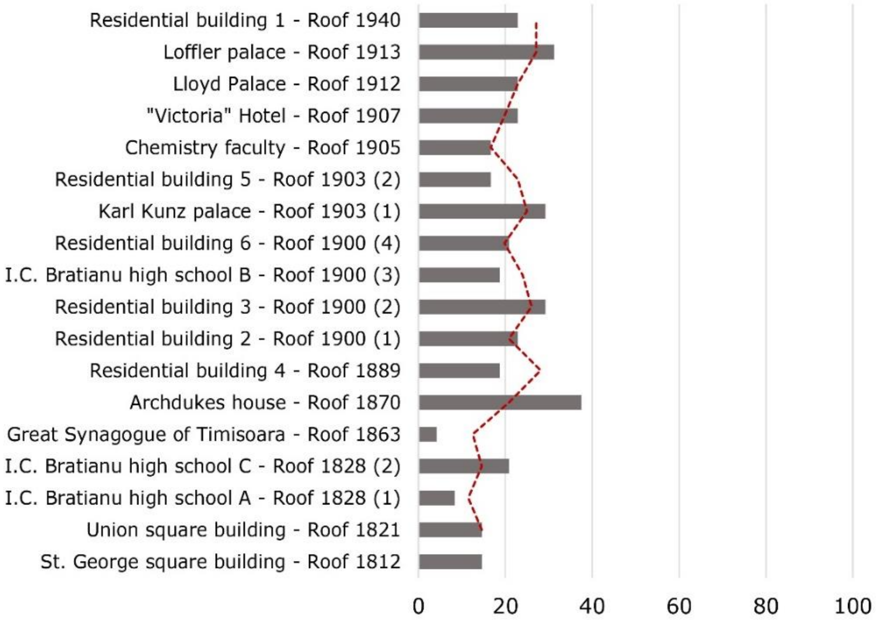


Fig. 5.36 Decay index for each roof structure and trendline

Subsequently, the final results were compared (Table 5-38): the ideal value of the roof structures (Fig. 5.37), their real value (Fig. 5.38) and their vulnerability (Fig. 5.39).

As previously defined, the ideal value of the roof structure is determined by considering the obtained score for each of the assessment levels (Fig. 5.37). It was observed that roof structures which proved out to have a high or very high idea value, present high scores for all of the four assessment levels which means that the scores are approximately similar like in the case of the early 19th century buildings. Early 20th century buildings prove out to have a moderate value since they are presenting high urban, architectural or even structural value, but lack in symbolic features.

Ultimately, all the assessed roof structures belonging to residential buildings have low or moderate value, depending on their position in the urban context and their link to the building since they do not present important symbolic features and the structural types are approximately similar adaptations of queen-post structures.

The real value of the roofs is also taking the decay and the climate change vulnerability into consideration (Fig. 5.38). Since the decay index presents insignificant variations between the assessed roof structures, the real value index is presenting the same trendline as the ideal value analysis, but slightly lower values. The real value is about 5% lower than the ideal value for most of the assessed roof structures, except for three of them, which have a high decay index and present, therefore, a 10% lower real value.

Ultimately, the vulnerability index was determined and analysed (Fig. 5.39). Since it depends on the ideal value of the roof structures, their decay index and climate change vulnerability, which is also related to the decay of the roof structure and its envelope, it also presents a similar trendline to the ideal value but with lower peaks and slopes. Still, it can be observed that roof structures with high value, are also the ones presenting a moderate or even high vulnerability. This index becomes, therefore, a useful decision-making tool, highlighting the roof structures where proper maintenance is necessary and where interventions are needed immediately.

Table 5-38 Value and vulnerability indexes of the assessed roof structures

	Roof 1812	Roof 1821	Roof 1828 (1)	Roof 1828 (2)	Roof 1863	Roof 1870	Roof 1889	Roof 1900 (1)	Roof 1900 (2)	Roof 1900 (3)	Roof 1900 (4)	Roof 1903 (1)	Roof 1903 (2)	Roof 1905	Roof 1907	Roof 1912	Roof 1913	Roof 1940
Ideal value	77	80	66	70	81	62	48	56	58	64	57	65	55	65	67	73	72	48
Climate change vulnerability	46	47	42	49	38	59	48	51	57	48	50	57	47	47	51	51	56	51
Value Index	75	77	65	67	80	56	46	52	54	61	54	61	53	62	64	69	67	45
Vulnerability Index	58	60	49	56	57	55	41	47	51	51	48	55	45	51	55	58	60	42

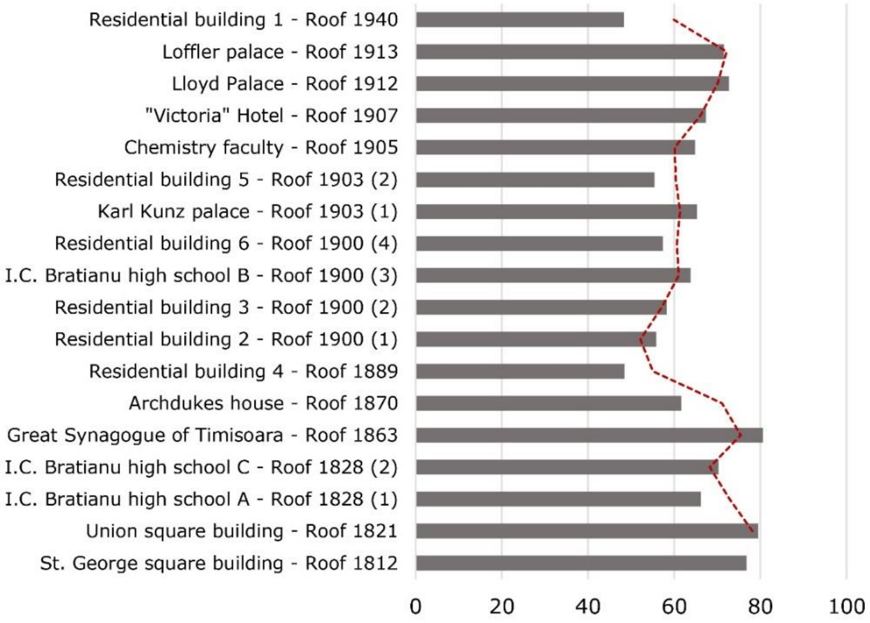


Fig. 5.37 Ideal value for each roof structure and trendline

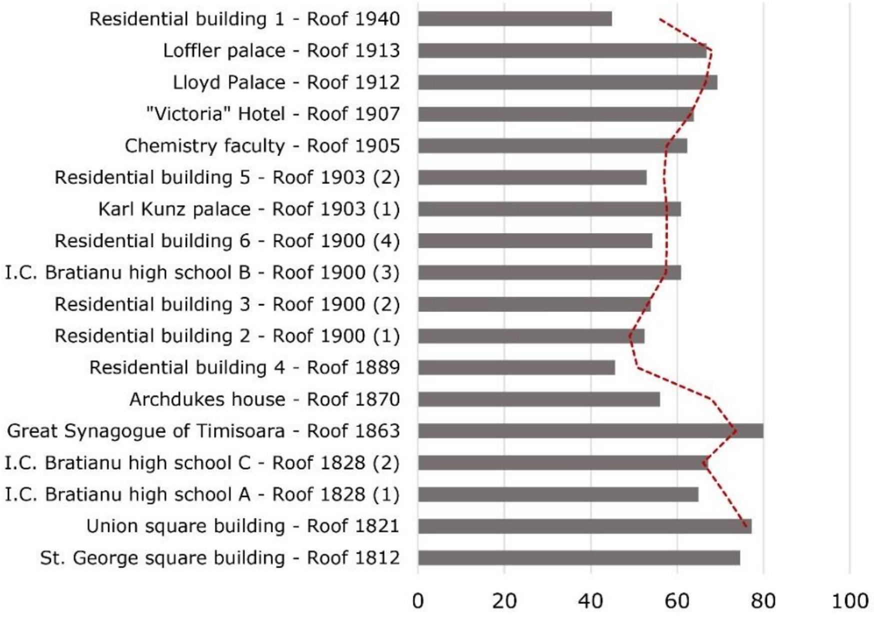


Fig. 5.38 Value index for each roof structure and trendline

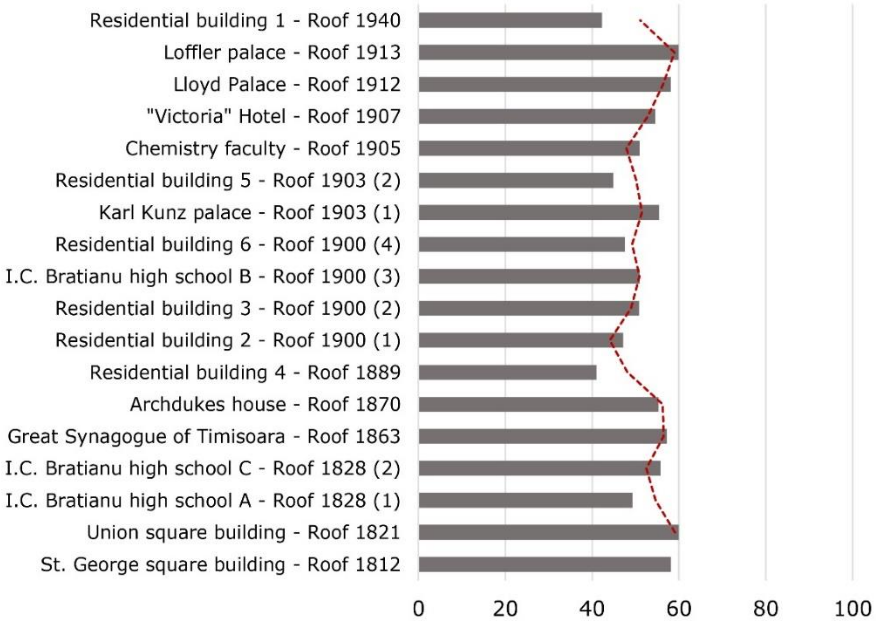


Fig. 5.39 Vulnerability index for each roof structure and trendline

5.4 Conclusions

It was observed, by analysing existing assessment methodologies, that there are specific criteria which are not taken into consideration, mainly related to the multidisciplinary and interdisciplinary analysis of historic timber roof structures. In order to be able to assess them as a whole, additional criteria have been identified and introduced in a complex assessment procedure, which could be used as a preliminary assessment tool, before the actual structural analysis of the roof structure.

The proposed procedure objectively approaches roof structures by using a multidisciplinary, transdisciplinary and interdisciplinary analysis, and addressing their link to the immediate urban space, its relationship to the building, its symbolical features and structural characteristic. It considers, therefore:

1. The role of the roof in its context, by analysing urban planning principles which could have influenced the shape of the roof, its pitch, but also the effect of the roof in shaping urban space and increasing the importance of the public area.
2. The link between the building and the roof by highlighting its role in completing the aesthetics of the building but also the effect of specific architectural requirements or principles in shaping the roof and ultimately defining the roof structure.
3. The presence of various symbolic elements and defining geometric principles which could confirm the authenticity of the roof and its structure, but also the link between the composing structural elements

4. The complexity of the roof structure by highlighting the presence of commonly used but also rare structural elements, information which can be used in defining the effect of the roof structure on the seismic vulnerability of the building.
5. The state of conservation of the roof and the roof structure which can influence the vulnerability of the roof.
6. Therefore, by using the procedure:
    - 6.1. A list of features is brought in the attention of the assessor which are relevant in properly making a preliminary assessment of a historic roof structure
    - 6.2. The roof structure can be assessed objectively, by looking at it from different angles, for a multidisciplinary point of view;
    - 6.3. The main features influencing the shape of the roof and roof structure type can be identified after a preliminary analysis of the urban development of the area and the building;
    - 6.4. The decay and climate change vulnerability can be determined, after a preliminary visual inspection of the roof exterior and the state of conservation of the timber elements;
    - 6.5. The effect of the roof structure on the seismic behaviour of the building highlighted, based on performed numerical simulations for 18th, 19th and 20th century roof structures.

More than this, the procedure offers a quick and easy way to perform a preliminary analysis of the value and vulnerability of the roof structure and help prioritise future interventions. Therefore, a roof structure with a high value and moderate vulnerability would encourage earlier repair and retrofit intervention than a roof structure with moderate or low value and moderate vulnerability.

The assessed criteria and responses to choose from were developed and confirmed based on historic roof structure from Timisoara and local climatic conditions and risks. Still, the framework can also be used in other cities by replacing specific assessed criteria or responses with local ones.

Still, this type of approach is also slightly subjective, and the point-of-view of the assessor can influence the chosen responses. Therefore, this type of preliminary visual assessment cannot replace a proper structural assessment of the structure and should be viewed as a fast and cost-efficient decision-making tool.

The described methodology is a first step in defining a holistic assessment of historic timber roof structures. The assessed criteria and responses to choose from were developed and confirmed based on the analysis of historic roof structure from Timisoara and local climatic conditions and risks but also based on other assessment methodologies identified in literature. It can, therefore, be developed in the future from all points of view and additional criteria added based on future observation. At the same time, the framework can also be used in other cities, but an adaptation of the score and replacement of specific assessed criteria or responses with local ones might be necessary.

5.5 Published research outcomes

The research outcomes presented in this chapter have been published in the following journals and conference proceedings:

1. M. Mosoarca, A. Keller, "A complex assessment methodology and procedure for historic roof structures", International Journal Of Architectural Heritage, ISSN 1558-3058, vol. 12(4), pp. 578-98, 2018, DOI: https://doi.org/10.1080/15583058.2018.1442519, WOS:000431697600007 (Web of Science indexed paper, Impact factor - 1.853)
2. A. Keller, M. Moșoarcă, "Assessment methodology for historic timber roof structures", Journal of Architecture, Urbanism and Heritage, ISSN 2668-2249, vol. 2, 2018.
3. A. Keller, M. Moșoarcă, "Historic timber roof structures value and influence on the seismic behavior of heritage buildings", Structures and Architecture-Bridging the Gap and Crossing Borders: Proceedings of the Fourth International Conference on Structures and Architecture (ICSA 2019), 24-26 July, Lisbon, Portugal, 907-914, 2019

6 CONCLUSIONS AND PERSONAL CONTRIBUTIONS

6.1 Conclusions

The research presented in the thesis represents an extensive study concerning the assessment of historic timber roof structures which is not only looking at the structure as an isolated element but by considering it as a part of a complex system.

Throughout the thesis, a series of features are brought forward which are highlighting the need of looking at historic timber roof structures from a multidisciplinary point of view respecting in this way the principles and recommendations of the Venice Charter and ICOMOS principles. It is therefore highlighted that urban planning principles, architectural styles, symbolic and geometric ratios and complex structural features are ultimately influencing the value of a roof structure. At the same time, the vulnerability of these structures to various threats is also brought forward by acknowledging the effect of current meteorological conditions and future climatic changes on the state of conservation of the roof structure elements and roof structures as a whole.

More than this, in the following two chapters, the focus is shifted on their structural behaviour. First, based on analysed experimental tests from literature, the structural behaviour of a roof structure is calibrated, and parameters named which must be considered during numerical simulations. The influence of the cross-section of the timber elements, material and joint axial stiffness was, therefore, considered and the obtained displacements compared with the ones recorded during the experimental tests.

The calibration process showed that it is necessary to:

1. consider a 20% reduction of the cross-section of the timber elements in the case of the models involving rigid, hinged or component method semi-rigid joints;
2. consider four times increase of the calculated axial stiffness of the joints calculated using the Heimeshoff and Köhler or the Hölzer method.

The observations were later on extended to 3D models of three characteristic 18th, 19th and 20th century roof structures in Timisoara in order to observe their influence on the seismic behaviour of a characteristic 18th century historic masonry building. Additionally, the cross-section of the timber elements and their state of conservation, roof to wall connection and joint axial stiffness were considered, performing therefore 15 simulations for each chosen roof structure. The performed numerical simulations are bringing forward that, if comparing the results with a reference building without roof structure, depending on the roof structure type, its state of conservation and the considered properties of the supports and joints:

1. In all the scenarios, the differences between the effect of each roof structure are highly influenced by the roof structure type and its state of conservation.
2. It was observed that in a good state of conservation, the presence of the roof structure is:
    - 2.1. Reducing the top horizontal displacement between 10 and 55%.
    - 2.2. Reducing the inter-story drift on the last floor between 5 and 85%.
3. In the case of a significantly, up to 20% decayed roof structure, its effect on the seismic behaviour of the historic masonry building is slightly different:

   3.1. Reducing the top horizontal displacement between 25 and 50%.
   3.2. Increasing the horizontal displacement with 10 up to 20% on the lower floors.
   3.3. Reducing the inter-story drift on the last floor between 25 and 85%.
   3.4. Increasing the inter-story drift with 10 up to 25% on the lower floors.
4. In both cases the presence of the 18th century roof structure has a better influence on the reduction of the top horizontal out-of-plane displacement and inter-story drift, than the other two types, the 20th century one presenting the lowest reduction.
5. Significantly reducing the damage level on all floors of the building.
6. Changing the deformed shape of the building from flexural, as recorded in the no roof structure case, to shear, in the case of the 18th and 19th century roof structure.
7. Concerning the internal forces recorded on the masonry wall, it was observed that:
   7.1. Tensile axial forces can appear at the top of the building.
   7.2. Shear forces perpendicular to the wall and out-of-plane bending moments can suffer an increase at the top of the building.
   7.3. Shear forces with a reverse direction appear in the area of the cross-vault
   7.4. Out-of-plane bending moments present an apparent increase at the top of the building but lower values at the base of the top floor.

Ultimately, a multi-, inter- and transdisciplinary assessment procedure was proposed which can be used as an objective and efficient tool to perform a preliminary assessment of a historic roof structure. It is considering its history, link to the surrounding urban space, its relationship to the building, its symbolical features and structural characteristics while also taking into consideration its state of conservation, vulnerability to meteorological factors and influence on the seismic behaviour of the building.

The procedure, organised in a tree-like structure, offers the assessor a list of features which were determined to be relevant for the assessment of historic timber roof structures and a list of possible answers based on the studies performed on roof structures in Timisoara, their evolution and main structural features. It then, based on the selected answers, automatically offers insight on its ideal value, by only considering its valuable features and ignoring the decay, its current value and vulnerability clearly and graphically. The procedure is, therefore, a quick and easy tool, which can be used to perform a preliminary analysis of the value and vulnerability of the assessed roof structure, based on a visual inspection, and help prioritise future interventions. Compared to other assessment methodologies which only consider the structural features of the roof structures, by also including the aesthetical and context related features the value and vulnerability of the structure can be significantly increased.

By applying the procedure on characteristic roof structures from Timisoara, an analysis of the obtained results was made which highlights that each of the considered features (urban planning, architecture, symbolism and structure) are dominant in specific periods. At the same time, it brings forward that each importance is changing in time, being influenced by the active periods of the craft guilds, changes in urban planning principles and architectural styles and ultimately by the technological developments which lead to more efficient roof structures.

6.2 Personal contributions

The main achievements and personal contributions are:

1. A thorough analysis of current international assessment methodologies and procedures;
2. An extensive desk and on-site survey of selected roof structures from Timisoara, from different periods and contexts;
3. Identification of additional features which have to be taken into consideration when assessing the value and vulnerability of historic timber roof structures:
    - 3.1. The analysis of context/urban planning related features on the value of historic timber roof structures and their changing influence on the roof structure shape and aesthetics, starting with the 18th century;
    - 3.2. Analysis of the link between architectural style and roof structure appearance;
    - 3.3. The geometric/symbolic analysis of 18th, 19th and 20th century roof structures from Timisoara highlighting their development in time and identification of an evolution pattern of the ratios used to define historic timber roof structures;
    - 3.4. Acknowledgement of the effect of climate-change-related threads (high wind velocities, hail and precipitation quantity increase) on the state of conservation and the integrity of historic timber roof structures;
    - 3.5. Analysis of the way the roof to wall connection is influencing the response of a roof structure when subjected to extreme meteorological events like high wind velocities;
4. Analysis of various semi-rigid modelling methods suitable for traditionally crafted joints and identification of the main differences between them;
5. Proposal of a calibrated historic roof structure model based on an analysed experimental test from the literature:
    - 5.1. Analysis of performed full-scale laboratory tests and numerical simulations performed on historic timber roof structures;
    - 5.2. Identification of parameters which must be considered during linear finite element simulations of historic timber roof structures.
6. Acknowledgement of the effect of selected historic timber roof structure specific for Timisoara on the seismic behaviour of a characteristic historic masonry building from the 18th century
    - 6.1. Finite element numerical modelling of a historic masonry building and three different roof structure types;
    - 6.2. Analysis and comparison of 5 different parameters and highlighting of the different effects of the three roof structures: the out-of-plane horizontal displacement, inter-story drift, deformed shape of the building, recorded damage level and internal forces on the wall;
    - 6.3. Analysis and comparison of the effect of the timber elements cross-section loss on the seismic behaviour of the considered historic masonry building of the same parameters.
7. Development of a preliminary assessment procedure, based on historical and visual analysis of the roof and roof structure, which determines the value of the assessed roof structure and its vulnerability which can be used as a decision-making tool for the planning and hierarchisation of future interventions and comprehensive structural assessments.

   7.1. Development of a corresponding score for each considered answer based on the observations of the performed analysis and formulas for the calculation of each value, decay and vulnerability index;
   7.2. Calibration of the developed assessment procedure based on a selection of analysed roof structures with significant urban, architectural, symbolic and structural value;
   7.3. Analysis of selected roof structure using the proposed assessment procedure on all the surveyed roof structures and analysis of the obtained results;
   7.4. Identification of the way the influence of each considered feature (urban planning, architecture, symbolism and structure) is changing over time in Timisoara.
8. Development of a comprehensive and easy-to-use Excel form and mobile application which can be used on site for the assessment of a historic timber roof structure.

6.3 Future researches

Studies concerning historic timber roof structures, their assessment and understanding of their structural behaviour are few, and future developments of the topics presented in this thesis are necessary in order to understand the complexity of these structures properly.

1. Extensive studies have to be performed in the future in order to understand all the features which influence the value and vulnerability of roof structures:
   1.1. How urban planning and architecture related principles are connected to the exterior appearance of roofs and how they ultimately affect the configuration of the interior timber structure
   1.2. How the geometrical analysis is defining other roof structure types and structures from other cities in order to validate the existence of the same patterns and acknowledge the influence of the beliefs of the craftsman in shaping them;
   1.3. How roof structure types were adapted considering their context;
   1.4. How current and future climatic threats are affecting roofs and roof structures and including their climatic vulnerability in climate change adaptation and mitigation strategies;
2. Development of value and vulnerability maps and clear intervention prioritising guides for roof structures in Timisoara by using the proposed roof structures assessment methodology;
3. Full-scale tests have to be also performed on local types of roof structures in order to properly understand their behaviour and determine how various roof structure types are influencing the seismic behaviour of heritage buildings. At the same time, it is important to understand how these two parts of buildings are interlinked and introduce these new data in seismic vulnerability assessment methodologies developed for heritage buildings;
4. Laboratory tests have to be performed on local traditional roof structure joints in order to understand the load transfer and their stiffness;
5. Additional numerical simulations have to be performed in order to understand the effect of other roof structure types on the seismic behaviour of historic masonry buildings. The main scope of any future studies would be the acknowledgement of the importance of considering the presence of roof structures in seismic

vulnerability assessment methodologies. The study was performed, until now, using linear finite element simulations, on three characteristic roof structure types from Timisoara in order to observe their effect on the seismic behaviour of a masonry building. Since roof structures are diverse and adapted based on the period and area in which they were built, further studies, have to be made;

6. In order to properly understand historic timber roof structures, their behaviour and influence on the seismic behaviour of historic masonry buildings, the study has to be also extended, and non-linear analysis performed;
7. The in-plane behaviour of the masonry building with the roof structures has to be also analysed;
8. Regarding the assessment procedure, currently, the assessed criteria and answers to choose from where developed and confirmed based on characteristic historic roof structure from Timisoara, from historically significant areas of the city, and local climatic conditions and risks. Therefore, it is necessary to further validate the procedure in a broader array of roof structures from various neighbourhoods of the city. At the same time, the framework can also be adapted and used in other cities, by replacing specific assessed criteria or answers with local ones and adapt the corresponding assigned scores.

6.4 Published papers

The research outcomes presented in this thesis have been published journals and conference proceedings:

- 5 papers in Web of Science indexed journals;
- 6 papers in Web of Science indexed proceedings;
- 1 paper in international database journals (SCOPUS);
- 2 papers in international database proceedings (SCOPUS);
- 11 papers in international proceedings;
- 1 paper in national proceedings.

6.4.1 Papers in Web of Science indexed journals

1. M. Mosoarca, A. Keller, C. Petrus and A. Racolta, “Failure analysis of historical buildings due to climate change”, Engineering Failure Analysis, ISSN: 1350-6307, vol. 82, pp. 666-680, 2017, DOI: 10.1016/j.engfailanal.2017.06.013, WOS:000413323400056. (Web of Science indexed paper, Impact factor - 2.897)
2. A. Keller, N. Chieffo, E. Opritescu, M. Mosoarca and A. Formisano, “Resilience of historic cities and adaptation to climate change”, Urbanism Architecture Constructions, ISSN: 2069-0509, vol. 8(1), pp. 15-26, 2017, WOS:000388684600002.
3. M. Mosoarca, A. Keller, “A complex assessment methodology and procedure for historic roof structures”, International Journal Of Architectural Heritage, ISSN 1558-3058, vol. 12(4), pp. 578-98, 2018, DOI: https://doi.org/10.1080/15583058.2018.1442519, WOS:000431697600007 (Impact factor - 1.853)
4. M. Mosoarca, A. Keller, C. Bocan, “Failure analysis of church towers and roof structures due to high wind velocities”, Engineering Failure Analysis, ISSN: 1350-6307, vol 100, pp. 76-87, 2019, DOI: 10.1016/j.engfailanal.2019.02.046, WOS:000463165000007 (Impact factor - 2.897)

5. A. Keller, M.A. Parisi, E. Tsakanika, M. Mosoarca, “Influence of historic roof structures on the seismic behaviour of historic masonry structures”, Proceedings of the Institution of Civil Engineers - Structures and Buildings, ISSN 0965-0911, https://doi.org/10.1680/jstbu.19.00098, 2019 (Impact factor – 0.965) – Published Online: August 23, 2019 (Ahead of print)

6.4.2 Papers in Web of Science indexed proceedings

1. I. Andreescu, A. Keller and M. Mosoarca, “Complex Assessment of Roof Structures”, Procedia Engineering, ISSN: 1877-7058, vol. 161, pp. 1204-1210, 2016, DOI: 10.1016/j.proeng.2016.08.542, WOS:000387566500185.
2. A. Keller, M. Moșoarcă, “Modern Historic Timber Structure Consolidation Technologies - A State of the Art Review”, Modern Technologies For The 3rd Millennium, Proceeding of the 16th National Technical-Scientific Conference on Modern Technologies for the 3rd Millennium, pp. 179-184, March 2017
3. I. Andreescu, A. Keller “Architecture as "Gesamtkunstwerk" – The Role of the Roof in Defining Architecture in the 19th and 20th Century in Timisoara”, IOP Conference Series: Materials Science and Engineering, ISSN: 1757-8981, Proceedings of the 3rd World Multidisciplinary Civil Engineering, Architecture, Urban Planning Symposium (WMCAUS 2018), 18–22 June 2018, Prague, Czech Republic, vol. 471, DOI: 10.1088/1757-899X/471/7/072034, WOS:000465811803030.
4. D. Bocan, A. Keller, C. Bocan, I. Apostol, M. Mosoarca, „Potential Results of Using Current Thermal Rehabilitation Techniques on a City Block of Timisoara and their Structural Strengthening Opportunities”, IOP Conference Series: Materials Science and Engineering, vol. 471, 2019, WOS:000465811802088.
5. D. Bocan, A. Keller, I. Apostol, M. Mosoarca, R. Bradeanu, „The impact of insulating plaster on the energy performance of historical buildings”, Modern Technologies For The 3rd Millennium, pp. 179-184, 2018, WOS:000491484600031.
6. I. Apostol, M. Mosoarca, N. Chieffo, A. Keller, D. Bocan, C. Bocan, R. Bradeanu, „Solutions for improving seismic vulnerability of historic masonry buildings”, Modern Technologies For The 3rd Millennium, pp. 131-136, 2018, WOS:000491484600023.

6.4.3 Papers in international database journals (SCOPUS)

1. A. Nariţa, V. Gurza, R. Opriţa, A. Keller, I. Apostol, M. Moşoarcă and C. Bocan, “New vulnerabilities of historic urban centers and archaeological sites: Extreme loads”, Pollack Periodica, vol. 11(3), pp. 15-26, 2016

6.4.4 Papers in international database proceedings (SCOPUS)

1. D. Bocan, C. Bocan, A. Keller, „Possibilities of Using Fiber Reinforced Mortar and Textile Reinforced Mortar for Strengthening Masonry Columns in Rehabilitation Projects”, RILEM Bookseries, vol 18, pp. 1651-1660, 2019.
2. M. Mosoarca, I. Apostol, A. Keller, A. Formisano, ”Consolidation methods of Romanian historical building with composite materials”, in Proceedings of International Conference on Mechanics of Masonry Structures Strengthened with Composites Materials, Key Engineering Materials, vol 747, pp. 406-413, 2017.

6.4.5 Papers in international proceedings

1. N. Chieffo, I. Apostol, A. Keller, M. Mosoarca and A. Marzo, “Global behavior of historical historic masonry structures and timber roof framework”, 3rd International Conference on Protection Of Historical Constructions (PROHITECH’17), Mazzolani, F. Lamas, A. Calado, L. Proenca, J. and Faggiano, B. eds., 2017.
2. A. Keller, N. Chieffo and M. Mosoarca, “Influence of roof structures on seismic behavior of historic buildings”, 3rd International Conference on Protection Of Historical Constructions (PROHITECH’17), Mazzolani, F. Lamas, A. Calado, L. Proenca, J. and Faggiano, B. eds., 2017
3. I. Andreescu I., A. Keller, “Complex features in assessing historic roof structures” 3rd International Conference on Protection of Historical Constructions (PROHITECH’17), Mazzolani, F. Lamas, A. Calado, L. Proenca, J. and Faggiano, B. eds., 2017.
4. A. Keller, M. Mosoarca “A complex assessment of historic roof structures” In 4th International Conference on Structural Health Assessment of Timber Structures (SHATIS’17), Arun, G. ed., 157–168, 2017.
5. A. Keller, M. Mosoarca “Influence of roof structures on seismic vulnerability of historic buildings” in 5th International Conference on Structural Health Assessment of Timber Structures (SHATIS’19), Branco J., Sousa H. and Poletti E. eds, 171-178, 2019
6. A. Keller, M. Moșoarcă, “Historic timber roof structures value and influence on the seismic behavior of heritage buildings”, Structures and Architecture-Bridging the Gap and Crossing Borders: Proceedings of the Fourth International Conference on Structures and Architecture (ICSA 2019), 24-26 July, Lisbon, Portugal, 907-914, 2019
7. D. Bocan, C. Bocan, A. Keller, „Energy efficiency study applied on a monumental building”, in Structures and Architecture-Bridging the Gap and Crossing Borders: Proceedings of the Fourth International Conference on Structures and Architecture (ICSA 2019), Lisbon, Portugal, July 2019, pp. 288-295.
8. I. Apostol, A. Keller, M. Mosoarca, ”Climate Change Risk Assessment Methodology for Historic Urban Centers”, Proceedings of the Green Development Infrastucture Technology Internation Conference (Gredit’2016), 2016.
9. A. Keller, C. Petrus, M. Mosoarca „The impact of climate change on the energy performance of historical buildings”, in Proceedings of the 3rd International Academic Conference on Places and Technologies, pp. 14-15, 2016.
10. B. Isopescu, A. Keller, M. Mosoarca, V. Stoian, “Interdisciplinary assessment, analysis and diagnosis of a historic timber roof structure from the 20th century”, in Proceedings of the 12th International Conference on Structural Analysis of Historical Constructions (SAHC 2020). *(accepted for publication)*
11. A. Keller, M. Mosoarca, “Effect of historic timber roof structures on the structural behaviour of masonry buildings during seismic events”, in Proceedings of the 12th International Conference on Structural Analysis of Historical Constructions (SAHC 2020). *(accepted for publication)*

6.4.6 Papers in national proceedings

1. A. Keller, M. Moșoarcă, “Assessment methodology for historic timber roof structures”, Journal of Architecture, Urbanism and Heritage, ISSN 2668-2249, vol. 2, 2018.

6.5 Citations

The up-to-date citation indexes are (including sef-citations):

- Web of Science: 16
- Scopus:27
- Google Scholar: 46

6.5.1 Citations of papers published in Web of Science indexed journals

M. Mosoarca, A. Keller, C. Petrus and A. Racolta, "Failure analysis of historical buildings due to climate change", Engineering Failure Analysis, ISSN: 1350-6307, vol. 82, pp. 666-680, 2017, DOI: 10.1016/j.engfailanal.2017.06.013, WOS:000413323400056. (Web of Science indexed paper, Impact factor - 2.897)

- Papers in Web of Science indexed journals and proceedings: **5**
  1. Ariffin, A. B., Zahari, M. S. M., and Hanafiah, M. H. "Adaptive reuse of historic buildings: connecting the links between tourist appreciation and visitation", Property Management, vol. 38, pp. 531-541, 2020
  2. Stagrum, A. E., Andenæs, E., Kvande, T., and Lohne, J. "Climate Change Adaptation Measures for Buildings—A Scoping Review", Sustainability, vol. 12(5), 2020
  3. Gonzalez, C.M.M., Rodriguez, A.L.L., Casas, J.N., Jaramillo, J.R., Teeling, C. "Risk assessment, monitoring and simulation of heritage buildings", Ge-Conservacion, vol. 17, pp. 215-225, 2020
  4. Kobetičová, K., & Černý, R. "Terrestrial eutrophication of building materials and buildings: An emerging topic in environmental studies", Science of the Total Environment, vol. 689, pp. 1316-1328, 2019
  5. Hulimka, J., Kałuża, M., & Kubica, J. "Failure and overhaul of a historic brick tower", Engineering failure analysis, vol. 102, pp. 46-59, 2019
- Papers in SCOPUS indexed journals and proceedings: **1**
  1. Cercel, P., and Vorovei, C. "Rehabilitation of hydrotechnical structures in the actual conditions of climate and anthropic changes", Environmental Engineering and Management Journal (EEMJ), vol. 18(7), 2019
- Papers in Google Scholar indexed journals and proceedings: **3**
  1. Özodabaş, A., and Karakurt, C. "Lightweight Natural Pumice Composites For Rehabilitation Of Historical Buildings And Landscape Works. Eskişehir Technical University Journal of Science and Technology A-Applied Sciences and Engineering, 21(2), 269-281, 2020
  2. Mwizerwa, F., and Buddhi, D. "Analysis of daily maximum temperature of Kigali city to be used in climate responsive building design. International Journal of Management", Technology And Engineering, vol. IX(I), 2855-2861, 2019
  3. Posani, M., Veiga, M. D. R., and De Freitas, V. P. "Historic buildings resilience: A view over envelope energy retrofit possibilities" Proceedings of the 8th International Conference on Building Resilience – ICBR Lisbon'2018, 2018

Keller, N. Chieffo, E. Opritescu, M. Mosoarca and A. Formisano, "Resilience of historic cities and adaptation to climate change", Urbanism Architecture Constructions, ISSN: 2069-0509, vol. 8(1), pp. 15-26, 2017, WOS:000388684600002.

- Papers in Web of Science indexed journals and proceedings: **1**
  1. Chieffo, N., Formisano, A., & Miguel Ferreira, T. "Damage scenario-based approach and retrofitting strategies for seismic risk mitigation: an application

to the historical Centre of Sant’Antimo (Italy)”. European Journal of Environmental and Civil Engineering, pp. 1-20, 2019

- Papers in Google Scholar indexed journals and proceedings: **3**
  1. Habibi, K., Hoseini, S. M., Dehshti, M., Khanian, M., & Mosavi, A. “The Impact of Natural Elements on Environmental Comfort in the Iranian-Islamic Historical City of Isfahan”, International Journal of Environmental Research and Public Health, vol. 17(16), 2020
  2. Adjim, H., Djedid, A., & Hamma, W. “urbanism, climate change and floods: case of Tlemcen city in Algeria”. Urbanism. Architecture. Constructions/Urbanism. Arhitectura. Constructii, vol. 9(1), 2018.
  3. Hamma, W. “Forecasting and risk management in Tlemcen: Legislation and urban master plans”, Urbanism. Arhitectură. Construcţii, vol. 9(1), pp. 5-22, 2018

M. Mosoarca, A. Keller, “A complex assessment methodology and procedure for historic roof structures”, International Journal Of Architectural Heritage, ISSN 1558-3058, vol. 12(4), pp. 578-98, 2018, DOI: https://doi.org/10.1080/15583058.2018.1442519, WOS:000431697600007 (Impact factor - 1.853)

- Papers in Web of Science indexed journals and proceedings: **2**
  1. Mosoarca, M., Onescu, I., Onescu, E., & Anastasiadis, A. Seismic vulnerability assessment methodology for historic masonry buildings in the near-field areas. Engineering Failure Analysis, vol. 115, 2020
  2. Mosoarca, M., Onescu, I., Onescu, E., Azap, B., Chieffo, N., & Szitar-Sirbu, M. “Seismic vulnerability assessment for the historical areas of the Timisoara city, Romania”, Engineering Failure Analysis, vol. 101, pp. 86-112, 2019
- Papers in SCOPUS indexed journals and proceedings: **1**
  1. Onescu, I., Onescu, E., & Mosoarca, M. “The impact of the cultural value on the seismic vulnerability of a historical building”. In IOP Conference Series: Materials Science and Engineering, vol. 603, IOP Publishing, 2019

M. Mosoarca, A. Keller, C. Bocan, “Failure analysis of church towers and roof structures due to high wind velocities”, Engineering Failure Analysis, ISSN: 1350-6307, vol 100, pp. 76-87, 2019, DOI: 10.1016/j.engfailanal.2019.02.046, WOS:000463165000007 (Web of Science indexed paper, Impact factor - 2.897)

- Papers in Web of Science indexed journals and proceedings: **1**
  1. Vincenzi, L., Bassoli, E., Ponsi, F., Castagnetti, C., & Mancini, F. “Dynamic monitoring and evaluation of bell ringing effects for the structural assessment of a masonry bell tower”, Journal of Civil Structural Health Monitoring, vol. 9(4), pp. 439-458, 1019
- Papers in Google Scholar journals and proceedings: **1**
  1. Shabani, A., Kioumarsi, M., Plevris, V., & Stamatopoulos, H. “Structural Vulnerability Assessment of Heritage Timber Buildings: A Methodological Proposal”, Forests, vol. 11(8), pp. 881, 2020

6.5.2 Citations of papers published in Web of Science indexed proceedings

I. Andreescu, A. Keller and M. Mosoarca, “Complex Assessment of Roof Structures”, Procedia Engineering, ISSN: 1877-7058, vol. 161, pp. 1204-1210, 2016, DOI:

10.1016/j.proeng.2016.08.542, WOS:000387566500185. (Web of Science indexed paper)

- Papers in Web of Science indexed journals and proceedings: **1**
    1. Faggiano, B., Marzo, A., Grippa, M. R., Iovane, G., Mazzolani, F. M., & Calicchio, D. "The inventory of structural typologies of timber floor slabs and roofs in the monumental built heritage: the case of the Royal Palace of Naples", International Journal of Architectural Heritage, vol. 12(4), pp.683-709, 2018
- Papers in SCOPUS indexed journals and proceedings: **3**
    1. Onescu, E., Onescu, I., & Mosoarca, M. "The Impact of Timber Roof Framework Over Historical Masonry Structures", In IOP Conference Series: Materials Science and Engineering, vol. 603(4), 2019
    2. Mosoarca, M., Stoian, V., Florea, M., & Niculescu, M. "Structural Balance of Historical Aggregates" In Structural Analysis of Historical Constructions, pp. 2448-2456, Springer, Cham, 2019
    3. Arlanova, M., Pavlov, S., & Yablonskii, L. "Restoration of historical mansions built in Russian province at the beginning of XX century". In MATEC Web of Conferences, vol. 193, 2018
- Papers in Google Scholar indexed journals and proceedings: **1**
    1. Makutėnienė, D., Šostak, O. R., & Maceika, A. Delphi method application for the roofing projects. Engineering Graphics BALTGRAF-14, pp. 32-36, 2017

6.5.3 Citations of papers published in international database journals (SCOPUS)

A. Nariţa, V. Gurza, R. Opriţa, A. Keller, I. Apostol, M. Moşoarcă and C. Bocan, "New vulnerabilities of historic urban centers and archaeological sites: Extreme loads", Pollack Periodica, vol. 11(3), pp. 15-26, 2016

- Papers in SCOPUS indexed journals and proceedings: **3**
    1. Quagliarini, E., Lucesoli, M., & Bernardini, G. "Rapid tools for assessing building heritage's seismic vulnerability: a preliminary reliability analysis", Journal of Cultural Heritage, vol. 39, pp. 130-139, 2019
    2. Mosoarca, M., Stoian, V., Florea, M., & Niculescu, M. "Structural Balance of Historical Aggregates" In Structural Analysis of Historical Constructions, pp. 2448-2456, Springer, Cham, 2019
    3. Sabareanu, E. "Assessment and Rehabilitation Issues Concerning Existing 70's Structural Stock", IOP Conference Series: Materials Science and Engineering, vol. 209(1), 2017

6.5.4 Citations of papers published in international proceedings

N. Chieffo, I. Apostol, A. Keller, M. Mosoarca and A. Marzo, "Global behavior of historical historic masonry structures and timber roof framework", 3rd International Conference on Protection Of Historical Constructions (PROHITECH'17), Mazzolani, F. Lamas, A. Calado, L. Proenca, J. and Faggiano, B. eds., 2017.

- Papers in Google Scholar indexed journals and proceedings: **1**
    1. Onescu, E., Onescu, I., & Mosoarca, M. "The Impact of Timber Roof Framework Over Historical Masonry Structures", In IOP Conference Series: Materials Science and Engineering, vol. 603(4), 2019

6.1 Acknowledgements

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Part of the research connected to the structural analysis of historic timber roof structures and the axial stiffness of traditional timber joints (presented in chapter 3) was done under the supervision of Prof. Dr. Ing. Thierry Descamps, during an Erasmus+ training mobility at the University of Mons in Belgium in 2017.

APPENDIX A

Out-of-plane displacement analysis - Roof structures with complete cross-section timber elements

Story	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
3.60	1.32	2.18	2.18	0.06
7.30	4.41	7.28	5.10	0.19
10.05	12.04	19.86	12.58	-0.13

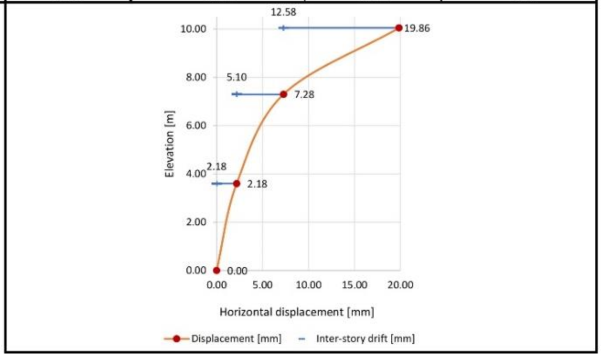


Fig. A. 1 Displacement and inter-story drift–building with first roof structure with rigid joints

	Rigid support				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.12	1.84	1.84	0.05	1.35	2.22	2.22	0.06	1.18	1.94	1.94	0.05
7.30	3.29	5.43	3.59	0.10	4.26	7.02	4.80	0.13	3.56	5.87	3.93	0.11
10.05	5.30	8.75	3.31	0.12	6.41	10.57	3.55	0.13	6.23	10.28	4.42	0.16

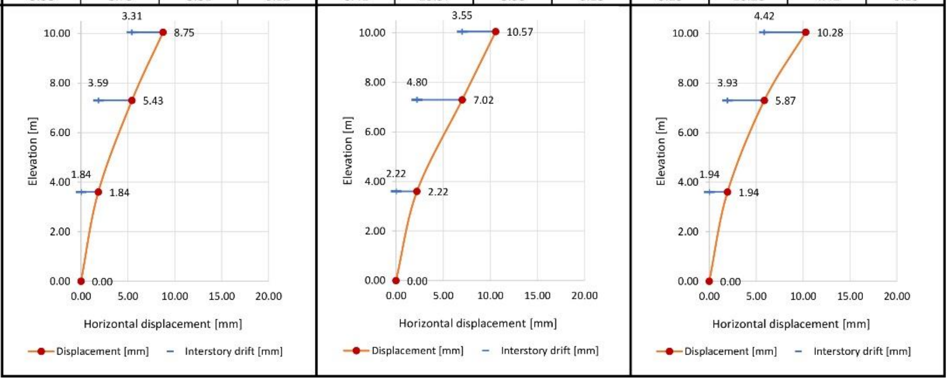


Fig. A. 2 Displacement and inter-story drift–building with first roof structure with rigid joints

	Rigid support				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.32	2.17	2.17	0.06	1.36	2.24	2.24	0.06	1.22	2.01	2.01	0.06
7.30	4.18	6.90	4.73	0.13	4.31	7.10	4.87	0.13	3.89	6.42	4.40	0.12
10.05	5.44	8.97	2.08	0.08	6.74	11.12	4.02	0.15	6.05	9.98	3.56	0.13

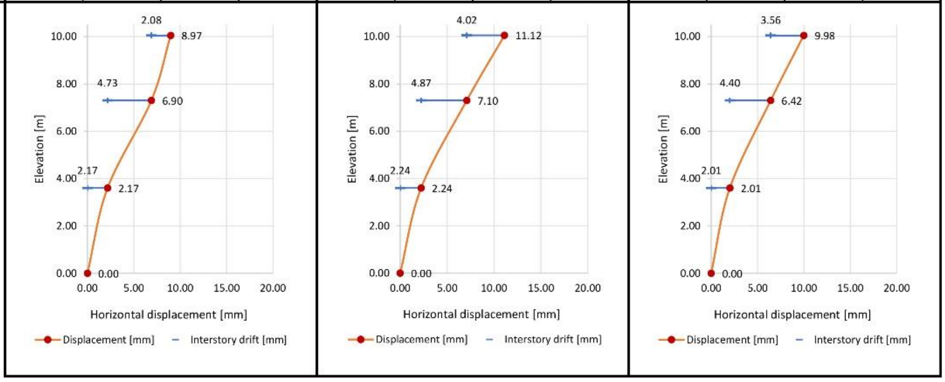


Fig. A. 3 Displacement and inter-story drift-building with first roof structure with hinged joints

	Rigid support				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.37	2.26	2.26	0.06	1.35	2.22	2.22	0.06	1.25	2.06	2.06	0.06
7.30	4.44	7.33	5.07	0.14	4.30	7.09	4.87	0.13	4.06	6.69	4.63	0.13
10.05	5.56	9.17	1.84	0.07	6.74	11.13	4.03	0.15	6.09	10.04	3.35	0.12

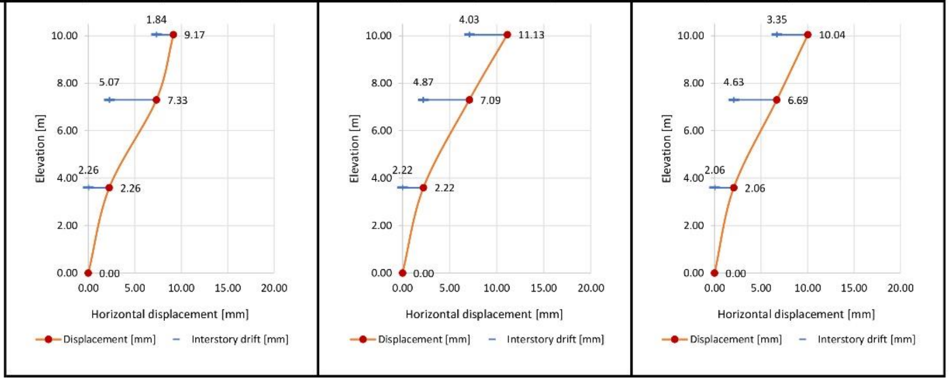


Fig. A. 4 Displacement and inter-story drift-building with first roof structure with Hölzer method determined joints

	Rigid support				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.33	2.20	2.20	0.06	1.32	2.17	2.17	0.06	0.99	1.63	1.63	0.05
7.30	4.25	7.01	4.81	0.13	4.15	6.84	4.67	0.13	2.82	4.66	3.02	0.08
10.05	5.50	9.08	2.07	0.08	7.49	12.36	5.52	0.20	6.00	9.91	5.25	0.19

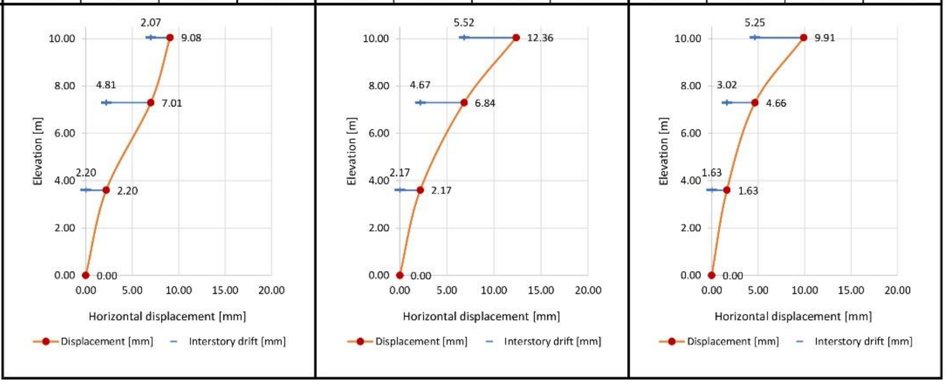


Fig. A. 5 Displacement and inter-story drift-building with first roof structure with component method determined joints

	Rigid support				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.35	2.23	2.23	0.06	1.29	2.13	2.13	0.06	1.15	1.89	1.89	0.05
7.30	4.39	7.25	5.02	0.14	4.07	6.71	4.58	0.12	3.58	5.90	4.01	0.11
10.05	5.56	9.17	1.92	0.07	7.39	12.19	5.47	0.20	6.40	10.56	4.66	0.17

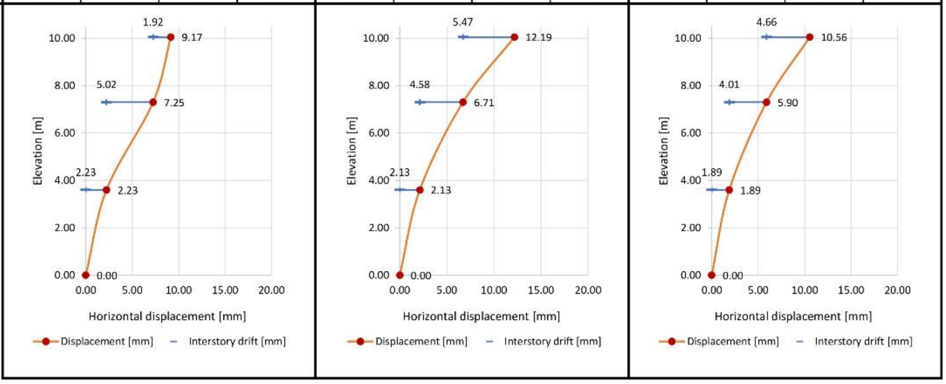


Fig. A. 6 Displacement and inter-story drift-building with first roof structure with Heimeshoff and Köhler method determined joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.32	2.18	2.18	0.06	1.32	2.17	2.17	0.06	1.33	2.19	2.19	0.06
7.30	4.30	7.09	4.91	0.13	4.27	7.05	4.88	0.13	4.31	7.12	4.93	0.13
10.05	7.16	11.81	4.72	0.17	7.50	12.37	5.32	0.19	7.42	12.25	5.13	0.19

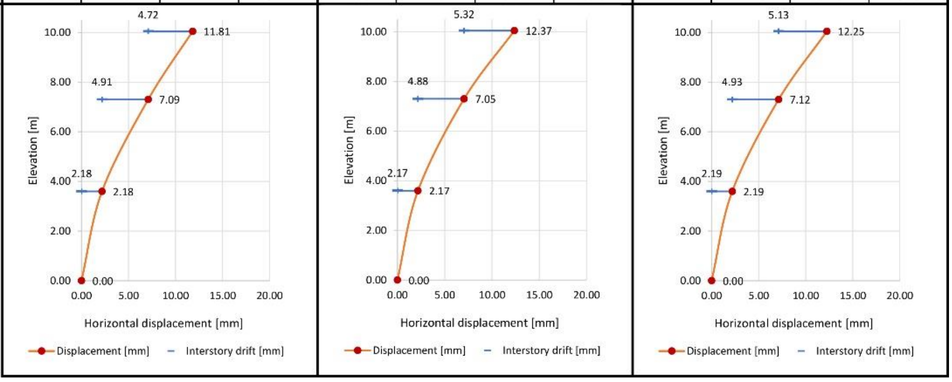


Fig. A. 7 Displacement and inter-story drift–building with second roof structure with rigid joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.36	2.25	2.25	0.06	1.40	2.31	2.31	0.06	1.40	2.31	2.31	0.06
7.30	4.63	7.64	5.39	0.15	4.80	7.91	5.60	0.15	4.79	7.90	5.59	0.15
10.05	6.98	11.52	3.88	0.14	6.86	11.32	3.41	0.12	6.76	11.16	3.26	0.12

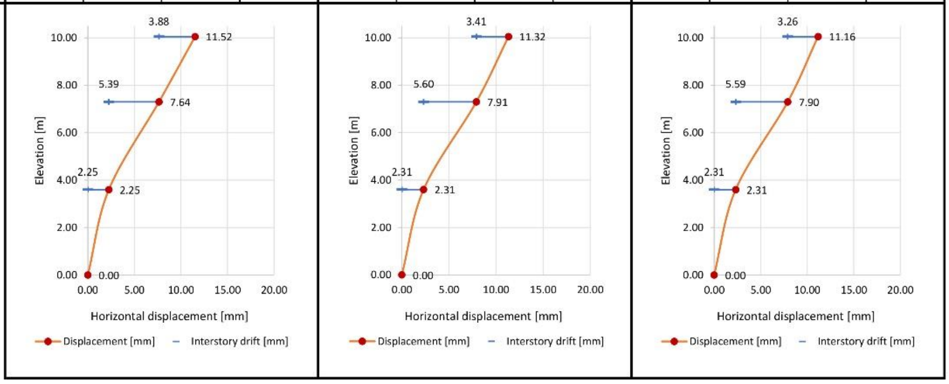


Fig. A. 8 Displacement and inter-story drift-building with second roof structure with hinged joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.37	2.26	2.26	0.06	1.40	2.31	2.31	0.06	1.40	2.32	2.32	0.06
7.30	4.65	7.67	5.42	0.15	4.81	7.93	5.62	0.15	4.80	7.92	5.61	0.15
10.05	6.96	11.48	3.80	0.14	6.85	11.30	3.37	0.12	6.75	11.14	3.21	0.12

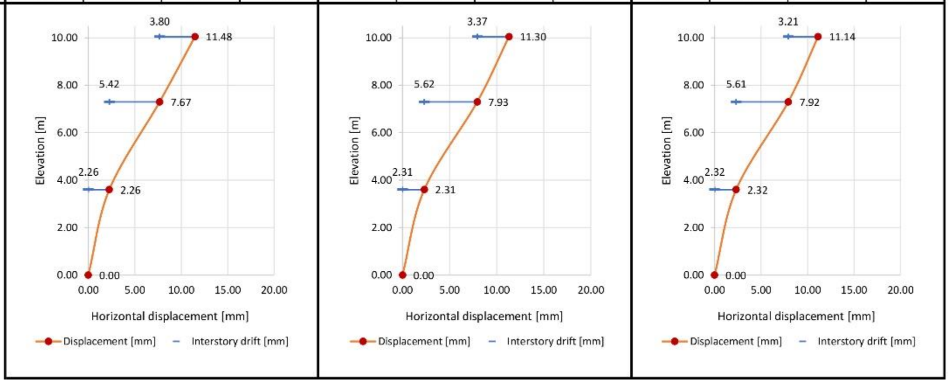


Fig. A. 9 Displacement and inter-story drift-building with second roof structure with Hölzer method determined joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.37	2.25	2.25	0.06	1.40	2.31	2.31	0.06	1.40	2.31	2.31	0.06
7.30	4.65	7.67	5.41	0.15	4.81	7.93	5.62	0.15	4.80	7.92	5.61	0.15
10.05	6.97	11.51	3.84	0.14	6.87	11.33	3.40	0.12	6.77	11.16	3.24	0.12

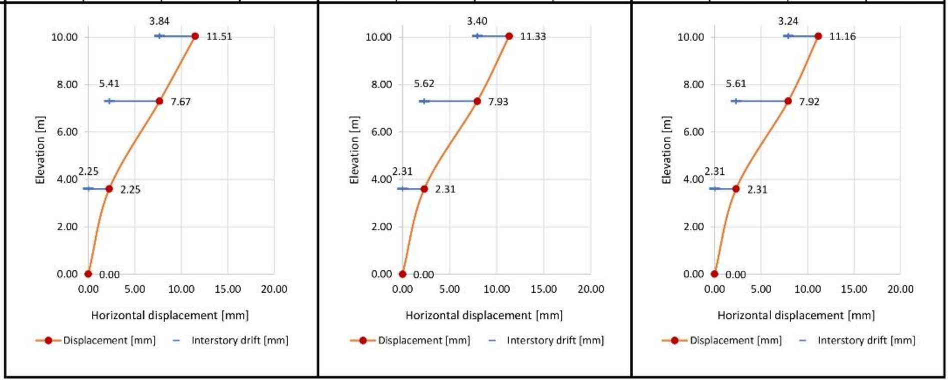


Fig. A. 10 Displacement and inter-story drift-building with second roof structure with component method determined joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.42	2.34	2.34	0.07	1.44	2.37	2.37	0.07	1.44	2.38	2.38	0.07
7.30	4.96	8.18	5.84	0.16	5.03	8.30	5.92	0.16	5.02	8.29	5.91	0.16
10.05	6.71	11.07	2.89	0.11	6.78	11.18	2.88	0.10	6.69	11.03	2.74	0.10

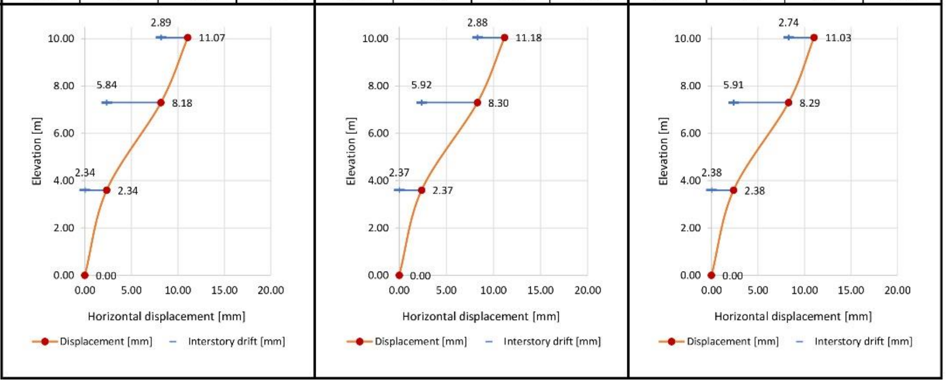


Fig. A. 11 Displacement and inter-story drift-building with second roof structure with Heimeshoff and Köhler method determined joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.07	1.77	1.77	0.05	1.14	1.88	1.88	0.05	1.08	1.78	1.78	0.05
7.30	3.59	5.92	4.15	0.11	3.83	6.32	4.44	0.12	3.57	5.90	4.12	0.11
10.05	10.65	17.57	11.65	0.42	10.94	18.06	11.74	0.43	10.90	17.99	12.10	0.44

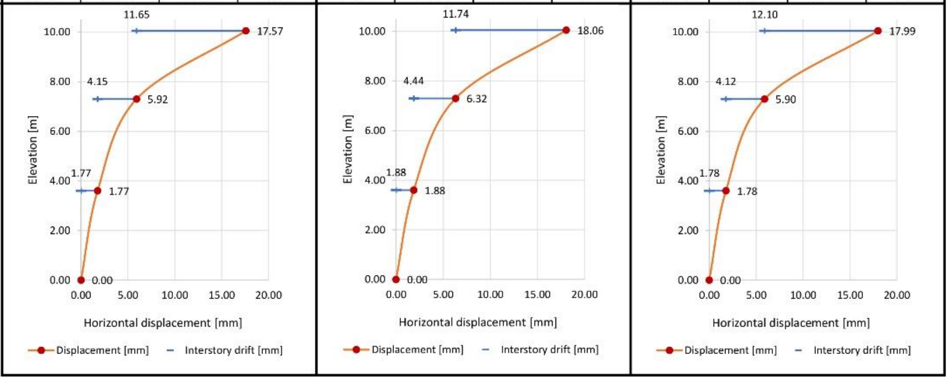


Fig. A. 12 Displacement and inter-story drift–building with third roof structure with rigid joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.18	1.94	1.94	0.05	1.20	1.98	1.98	0.06	1.21	1.99	1.99	0.06
7.30	3.86	6.38	4.44	0.12	3.82	6.30	4.31	0.12	3.82	6.30	4.31	0.12
10.05	9.13	15.06	8.69	0.32	9.37	15.46	9.16	0.33	9.33	15.39	9.09	0.33

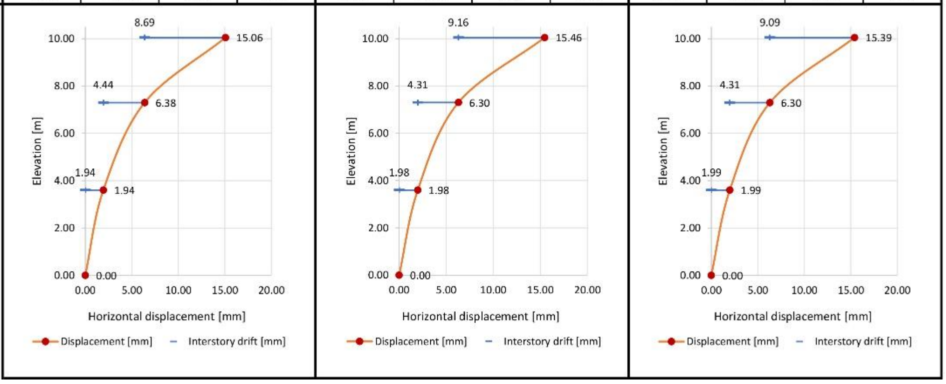


Fig. A. 13 Displacement and inter-story drift-building with third roof structure with hinged joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.22	2.01	2.01	0.06	1.20	1.98	1.98	0.06	1.20	1.98	1.98	0.06
7.30	3.90	6.43	4.43	0.12	3.80	6.27	4.29	0.12	3.80	6.27	4.29	0.12
10.05	9.22	15.22	8.78	0.32	9.31	15.36	9.09	0.33	9.27	15.30	9.02	0.33

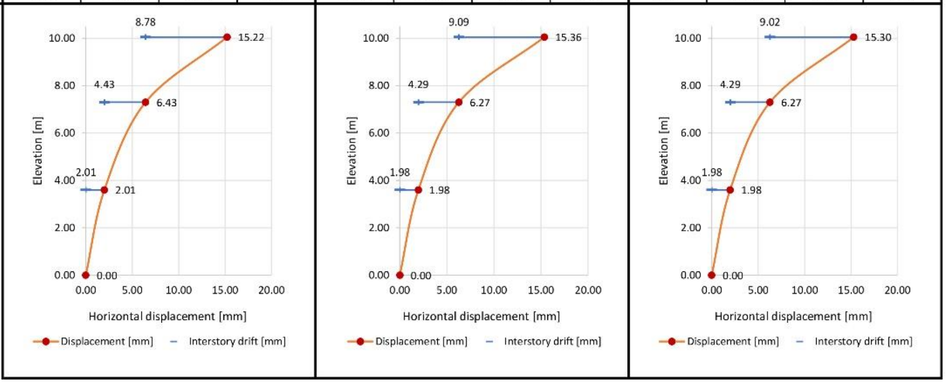


Fig. A. 14 Displacement and inter-story drift-building with third roof structure with Hölzer method determined joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.15	1.90	1.90	0.05	1.21	1.99	1.99	0.06	1.21	1.99	1.99	0.06
7.30	3.84	6.34	4.44	0.12	3.81	6.29	4.29	0.12	3.81	6.28	4.29	0.12
10.05	9.33	15.40	9.06	0.33	9.59	15.83	9.54	0.35	9.55	15.76	9.47	0.34

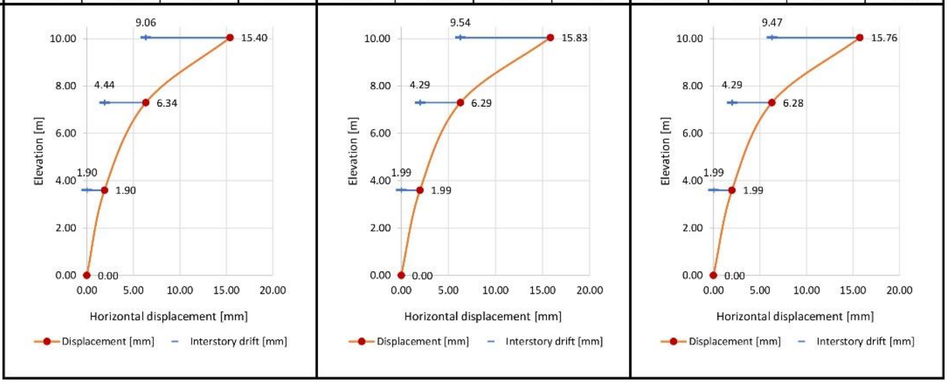


Fig. A. 15 Displacement and inter-story drift-building with third roof structure with component method determined joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.24	2.04	2.04	0.06	1.22	2.01	2.01	0.06	1.22	2.02	2.02	0.06
7.30	4.07	6.72	4.68	0.13	3.96	6.54	4.52	0.12	3.96	6.53	4.51	0.12
10.05	8.73	14.40	7.68	0.28	8.81	14.53	7.99	0.29	8.65	14.28	7.75	0.28

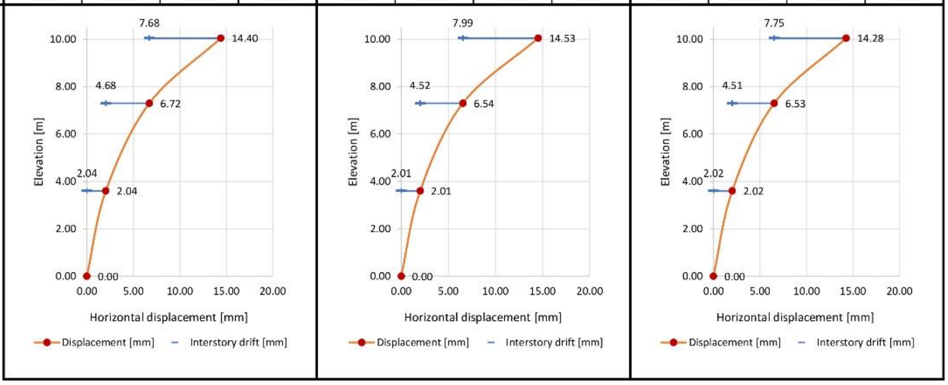


Fig. A. 16 Displacement and inter-story drift-building with third roof structure with Heimeshoff and Köhler method determined joints

Out-of-plane displacement analysis - Roof structures with a reduced cross-section

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.35	2.23	2.23	0.06	1.28	2.11	2.11	0.06	1.21	2.00	2.00	0.06
7.30	4.38	7.23	4.99	0.13	4.10	6.77	4.66	0.13	3.85	6.34	4.34	0.12
10.05	6.03	9.95	2.73	0.10	6.31	10.41	3.64	0.13	6.52	10.75	4.41	0.16

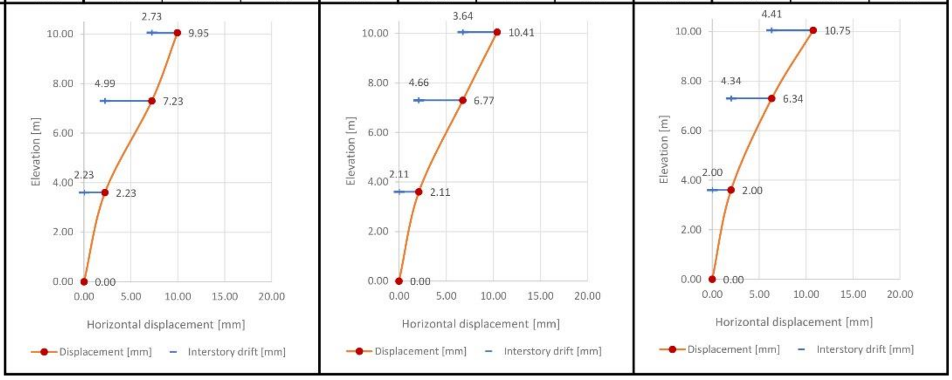


Fig. A. 17 Displacement and inter-story drift–building with first decayed roof structure with rigid joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.42	2.35	2.35	0.07	1.30	2.14	2.14	0.06	1.28	2.12	2.12	0.06
7.30	4.71	7.77	5.42	0.15	4.14	6.83	4.69	0.13	4.26	7.02	4.90	0.13
10.05	6.34	10.46	2.69	0.10	7.73	12.75	5.92	0.22	7.06	11.64	4.62	0.17

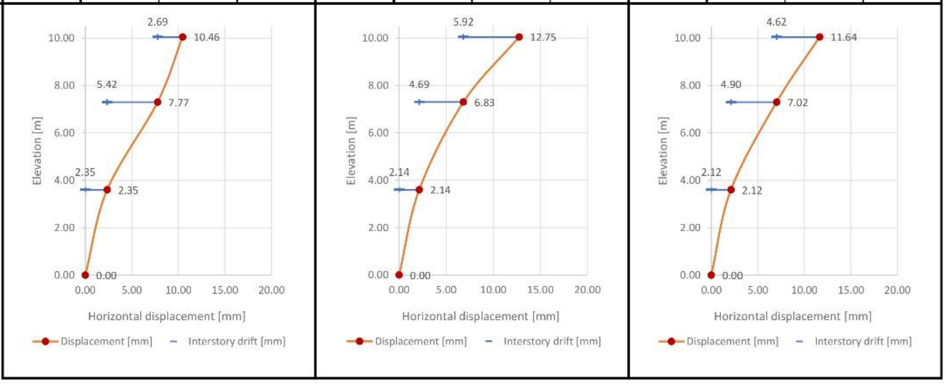


Fig. A. 18 Displacement and inter-story drift-building with first decayed roof structure with hinged joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.43	2.35	2.35	0.07	1.28	2.11	2.11	0.06	1.28	2.11	2.11	0.06
7.30	4.78	7.88	5.53	0.15	4.16	6.87	4.76	0.13	4.31	7.11	5.00	0.14
10.05	6.40	10.56	2.68	0.10	7.71	12.73	5.86	0.21	7.01	11.56	4.45	0.16

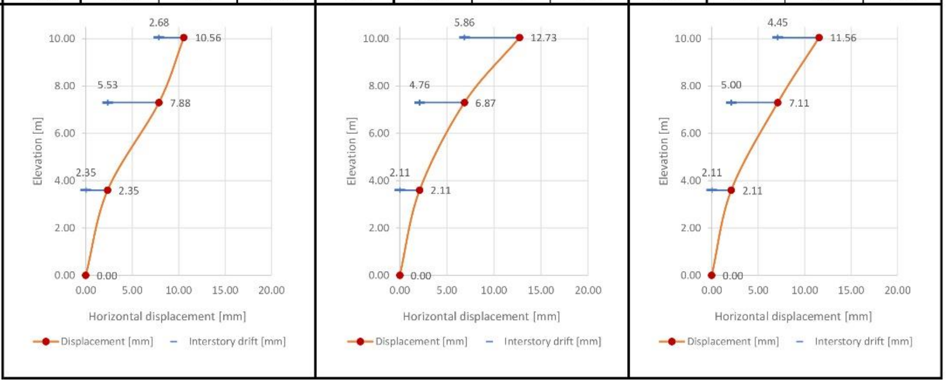


Fig. A. 19 Displacement and inter-story drift-building with first decayed roof structure with Hölzer method determined joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.42	2.33	2.33	0.06	1.23	2.03	2.03	0.06	1.25	2.05	2.05	0.06
7.30	4.72	7.79	5.45	0.15	3.93	6.49	4.46	0.12	4.15	6.84	4.79	0.13
10.05	6.50	10.72	2.93	0.11	8.88	14.65	8.16	0.30	8.04	13.27	6.43	0.23

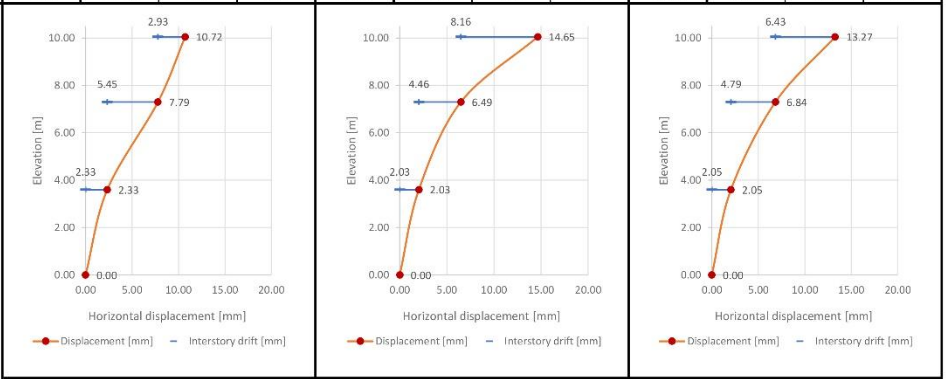


Fig. A. 20 Displacement and inter-story drift-building with first decayed roof structure with component method determined joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.38	2.28	2.28	0.06	1.21	2.00	2.00	0.06	1.24	2.04	2.04	0.06
7.30	4.69	7.74	5.46	0.15	3.84	6.33	4.34	0.12	4.13	6.81	4.77	0.13
10.05	6.42	10.59	2.85	0.10	8.55	14.10	7.77	0.28	7.88	13.00	6.19	0.23

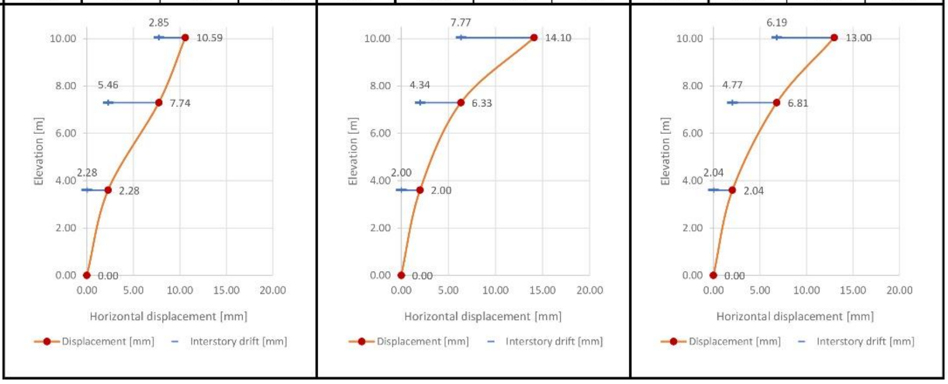


Fig. A. 21 Displacement and inter-story drift-building with first decayed roof structure with Heimeshoff and Köhler method determined joints

Story	Rigid support				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.44	2.37	2.37	0.07	1.44	2.38	2.38	0.07	1.44	2.38	2.38	0.07
7.30	5.00	8.25	5.87	0.16	5.01	8.27	5.89	0.16	5.01	8.27	5.89	0.16
10.05	7.35	12.13	3.88	0.14	7.59	12.53	4.26	0.15	7.50	12.37	4.10	0.15

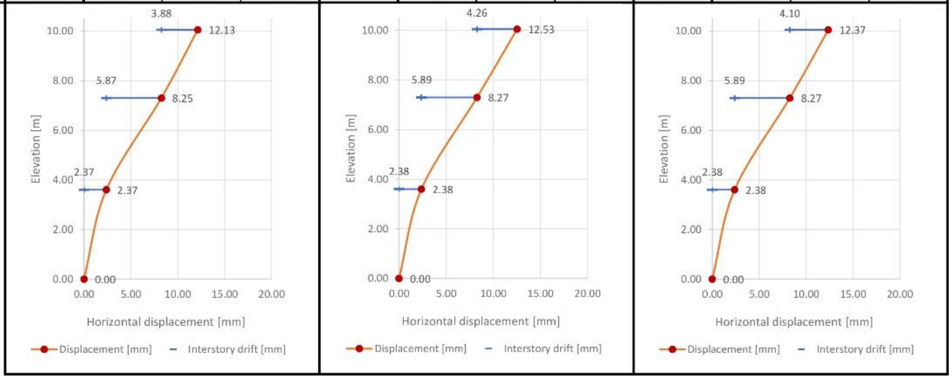


Fig. A. 22 Displacement and inter-story drift–building with second decayed roof structure with rigid joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.46	2.41	2.41	0.07	1.48	2.44	2.44	0.07	1.48	2.45	2.45	0.07
7.30	5.27	8.69	6.28	0.17	5.35	8.82	6.38	0.17	5.34	8.81	6.37	0.17
10.05	6.68	11.02	2.33	0.08	6.80	11.23	2.40	0.09	6.71	11.07	2.26	0.08

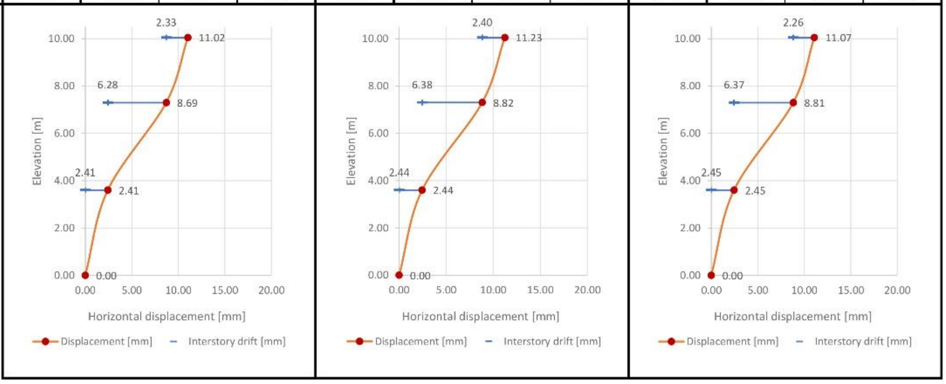


Fig. A. 23 Displacement and inter-story drift-building with second decayed roof structure with hinged joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.46	2.41	2.41	0.07	1.46	2.41	2.41	0.07	1.48	2.45	2.45	0.07
7.30	5.27	8.70	6.29	0.17	5.27	8.70	6.29	0.17	5.34	8.81	6.37	0.17
10.05	6.68	11.02	2.32	0.08	6.75	11.13	2.43	0.09	6.71	11.07	2.26	0.08

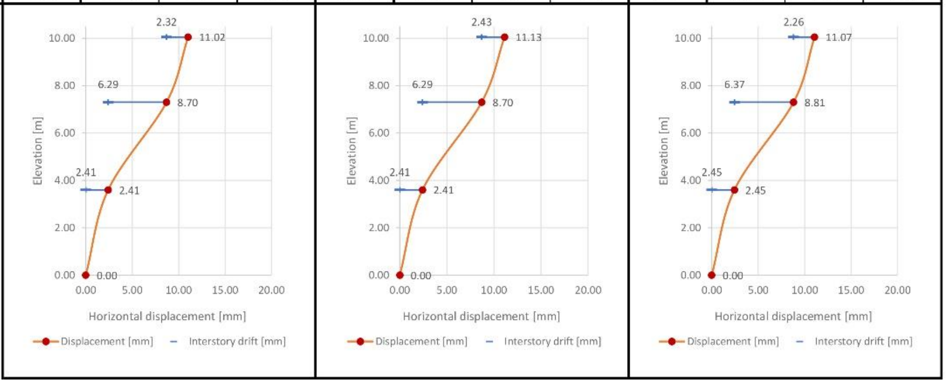


Fig. A. 24 Displacement and inter-story drift-building with second decayed roof structure with Hölzer method determined joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.46	2.41	2.41	0.07	1.48	2.44	2.44	0.07	1.48	2.45	2.45	0.07
7.30	5.28	8.70	6.29	0.17	5.35	8.83	6.39	0.17	5.35	8.82	6.37	0.17
10.05	6.69	11.03	2.33	0.08	6.81	11.24	2.41	0.09	6.72	11.08	2.26	0.08

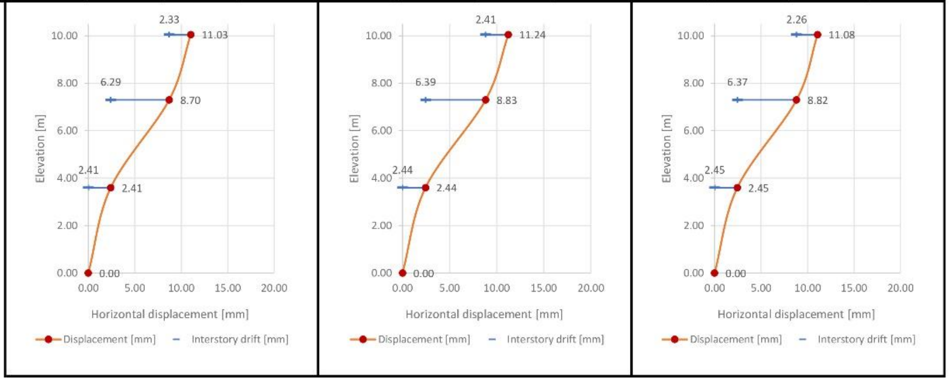


Fig. A. 25 Displacement and inter-story drift-building with second decayed roof structure with component method determined joints

Story	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]	Displacement [mm]	Displacement [mm]*q=1.65	Inter-story drift [mm]	Inter-story drift ratio [%]
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.48	2.44	2.44	0.07	1.49	2.46	2.46	0.07	1.49	2.46	2.46	0.07
7.30	5.36	8.85	6.41	0.17	5.40	8.91	6.44	0.17	5.40	8.90	6.44	0.17
10.05	6.67	11.01	2.16	0.08	6.62	10.93	2.02	0.07	6.73	11.10	2.19	0.08

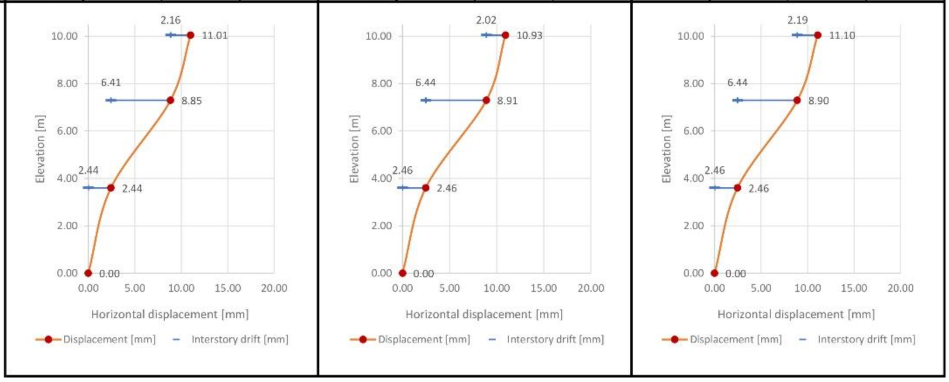


Fig. A. 26 Displacement and inter-story drift-building with second decayed roof structure with Heimeshoff and Köhler method determined joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.32	2.17	2.17	0.06	1.29	2.13	2.13	0.06	1.30	2.14	2.14	0.06
7.30	4.42	7.29	5.12	0.14	4.39	7.24	5.10	0.14	4.40	7.26	5.12	0.14
10.05	8.51	14.04	6.74	0.25	8.66	14.29	7.06	0.26	8.62	14.23	6.96	0.25

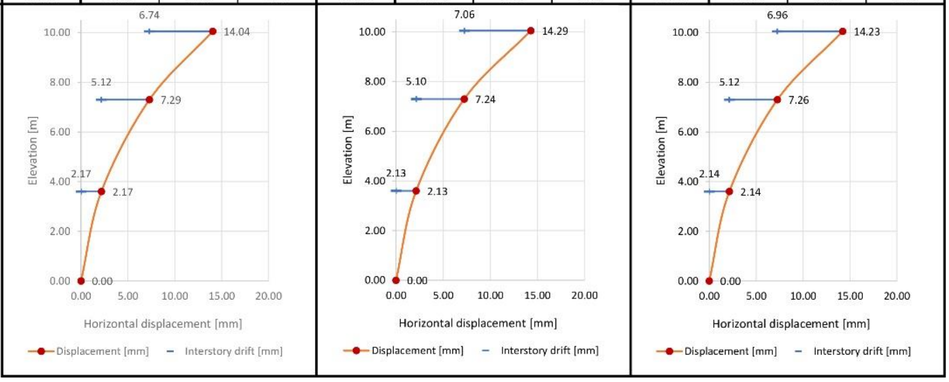


Fig. A. 27 Displacement and inter-story drift–building with third decayed roof structure with rigid joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.36	2.25	2.25	0.06	1.35	2.23	2.23	0.06	1.35	2.23	2.23	0.06
7.30	4.75	7.84	5.59	0.15	4.68	7.72	5.49	0.15	4.68	7.73	5.49	0.15
10.05	8.74	14.41	6.58	0.24	9.07	14.97	7.25	0.26	9.09	15.00	7.27	0.26

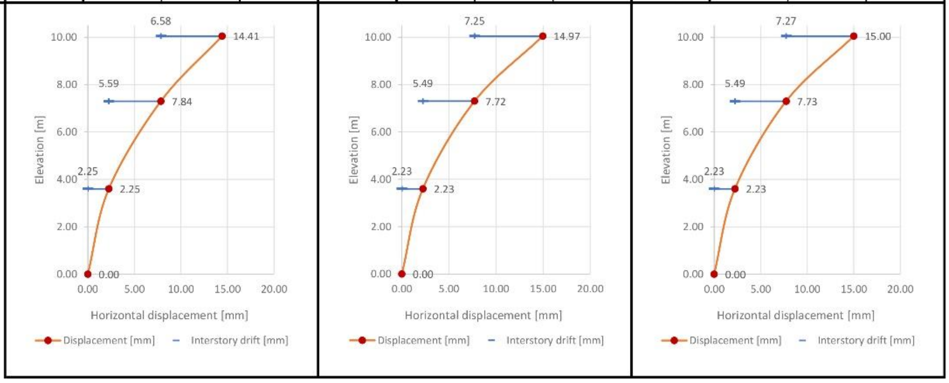


Fig. A. 28 Displacement and inter-story drift-building with third decayed roof structure with hinged joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.35	2.23	2.23	0.06	1.34	2.20	2.20	0.06	1.34	2.21	2.21	0.06
7.30	4.71	7.78	5.55	0.15	4.62	7.62	5.42	0.15	4.63	7.64	5.43	0.15
10.05	8.55	14.10	6.32	0.23	8.89	14.66	7.04	0.26	8.89	14.67	7.03	0.26

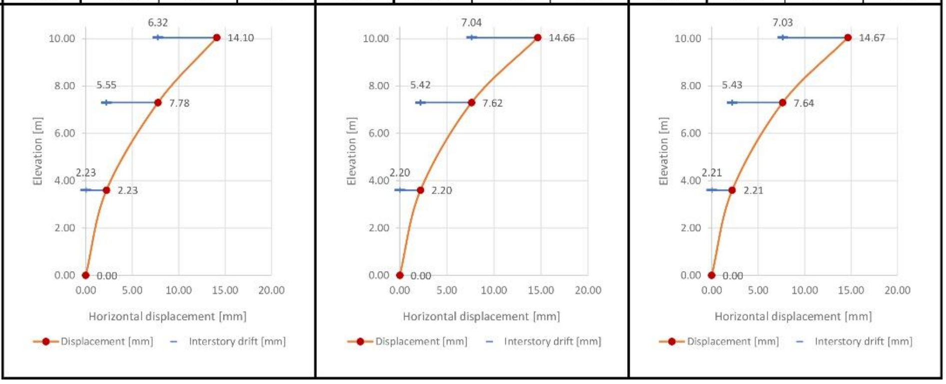


Fig. A. 29 Displacement and inter-story drift-building with third decayed roof structure with Hölzer method determined joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.36	2.24	2.24	0.06	1.35	2.22	2.22	0.06	1.35	2.23	2.23	0.06
7.30	4.72	7.79	5.55	0.15	4.65	7.67	5.45	0.15	4.66	7.69	5.46	0.15
10.05	8.45	13.94	6.15	0.22	8.96	14.78	7.10	0.26	8.97	14.79	7.10	0.26

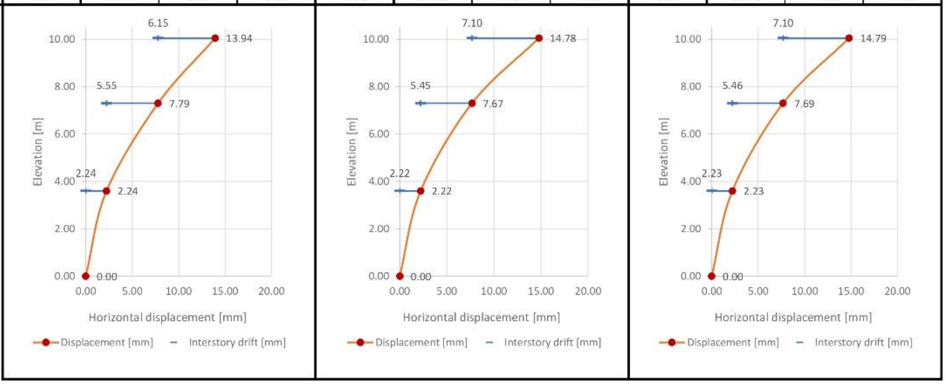


Fig. A. 30 Displacement and inter-story drift-building with third decayed roof structure with component method determined joints

	Rigid suport				Sliding support (X - 50N/mm; Y - 10N/mm)				Hinged and sliding support			
Story	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**	**Displacement [mm]**	**Displacement [mm]*q=1.65**	**Inter-story drift [mm]**	**Inter-story drift ratio [%]**
0.00	0.00	0.00			0.00	0.00			0.00	0.00		
3.60	1.34	2.20	2.20	0.06	1.32	2.18	2.18	0.06	1.32	2.18	2.18	0.06
7.30	4.73	7.80	5.60	0.15	4.61	7.61	5.43	0.15	4.62	7.62	5.44	0.15
10.05	8.42	13.88	6.08	0.22	8.72	14.39	6.78	0.25	8.72	14.38	6.77	0.25

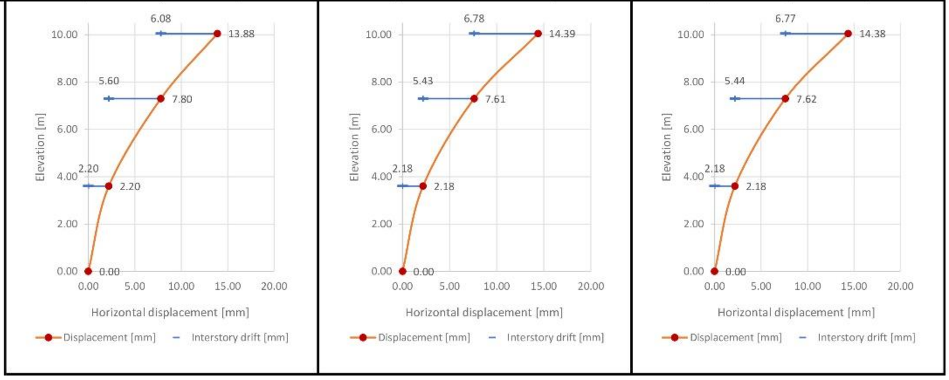


Fig. A. 31 Displacement and inter-story drift-building with third decayed roof structure with Heimeshoff and Köhler method determined joints

APPENDIX B

Roof structure 1812

Fig. B. 1 Urban context and relationship with the street and building

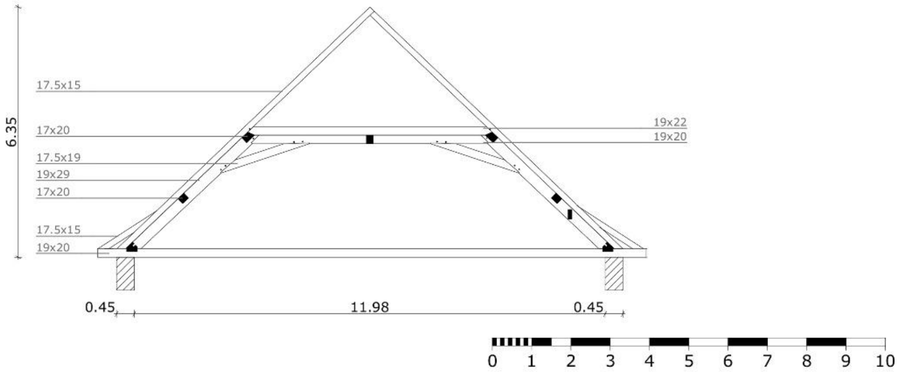


Fig. B. 2 The roof structure – section A-A

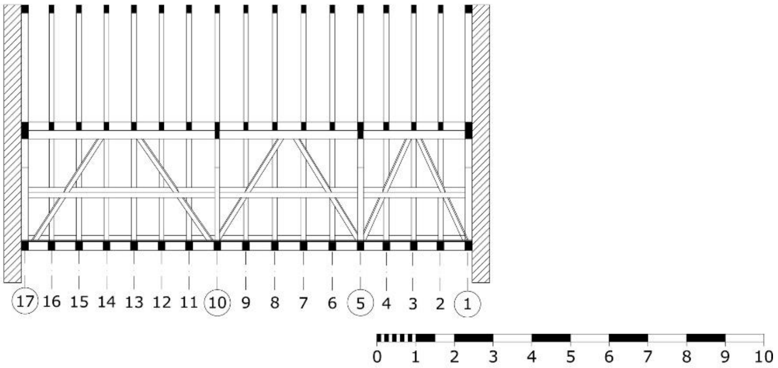


Fig. B. 3 The roof structure – section B-B

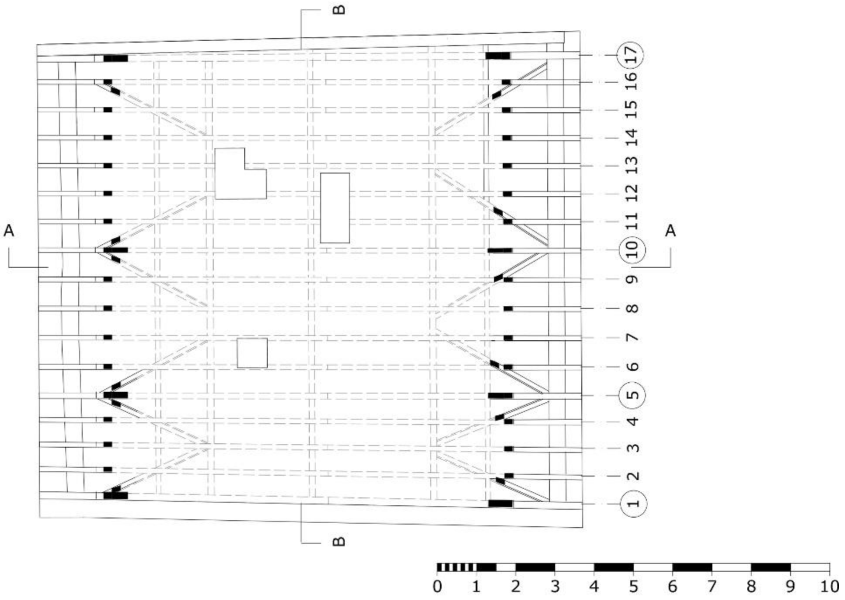


Fig. B. 4 The roof structure – plan

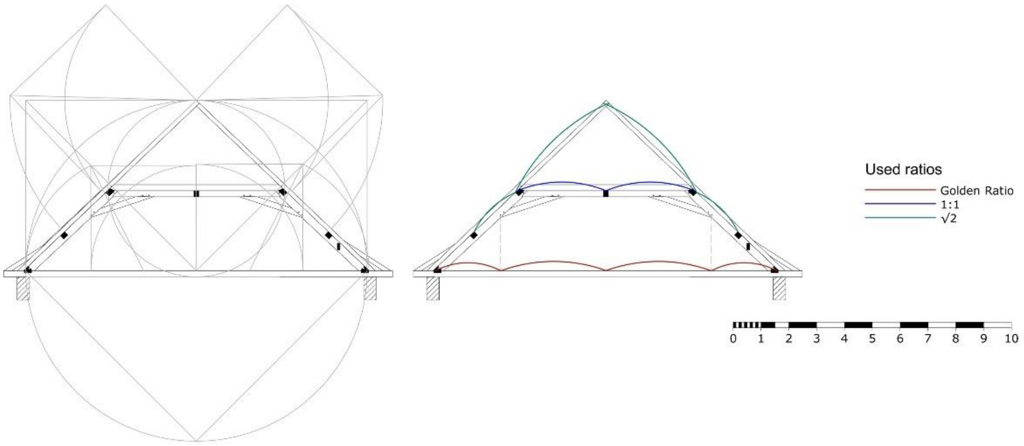


Fig. B. 5 Geometric analysis

Roof structure 1828 (1)

Fig. B. 6 Urban context and relationship with the street and building

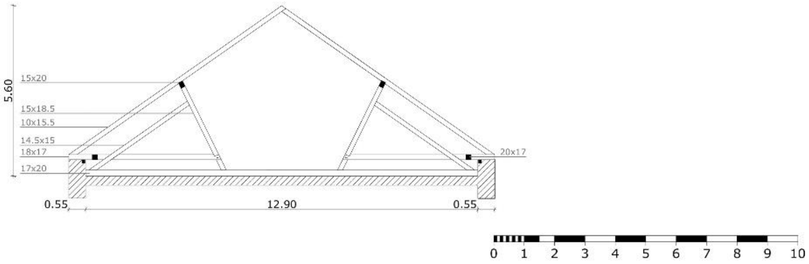


Fig. B. 7 The roof structure – section A-A

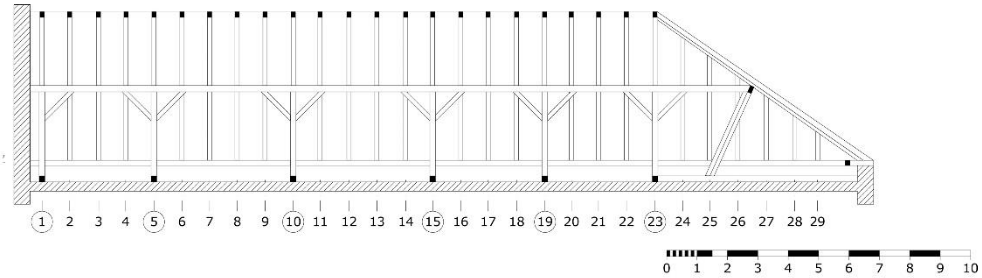


Fig. B. 8 The roof structure – section B-B

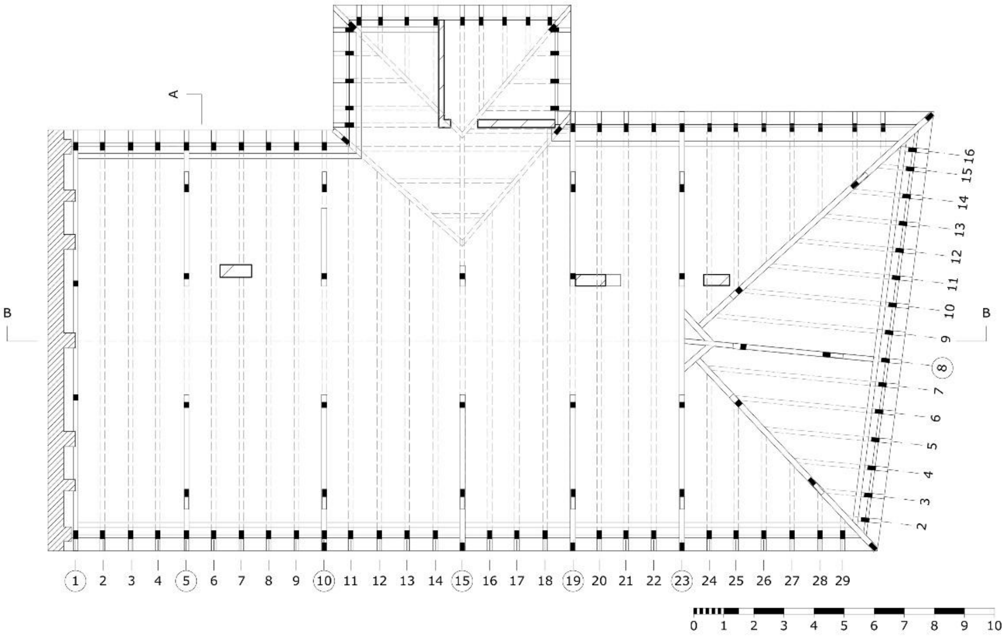


Fig. B. 9 The roof structure - plan

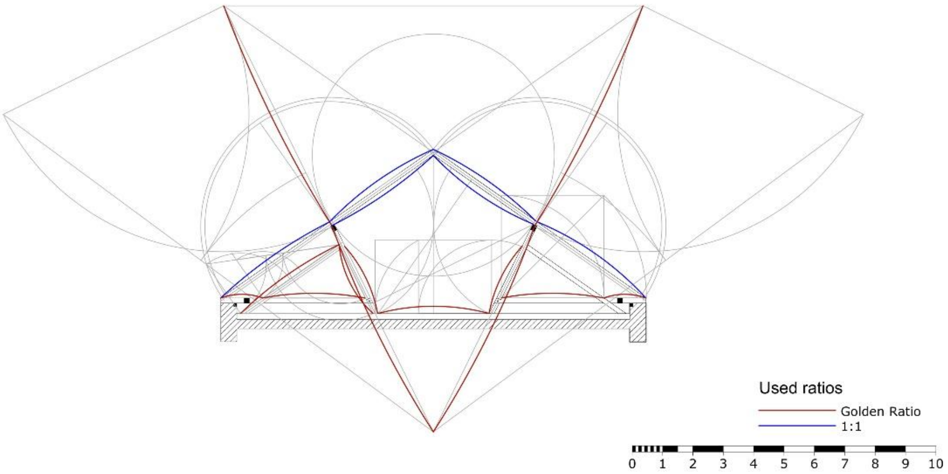


Fig. B. 10 Geometric analysis

Roof structure 1828 (2)

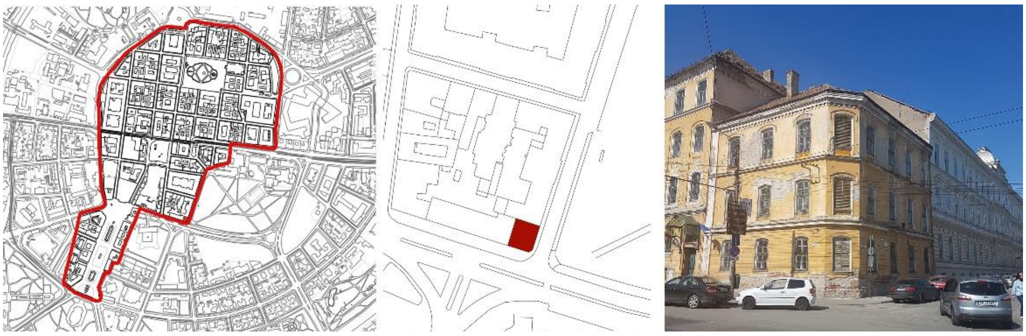


Fig. B. 11 Urban context and relationship with the street and building

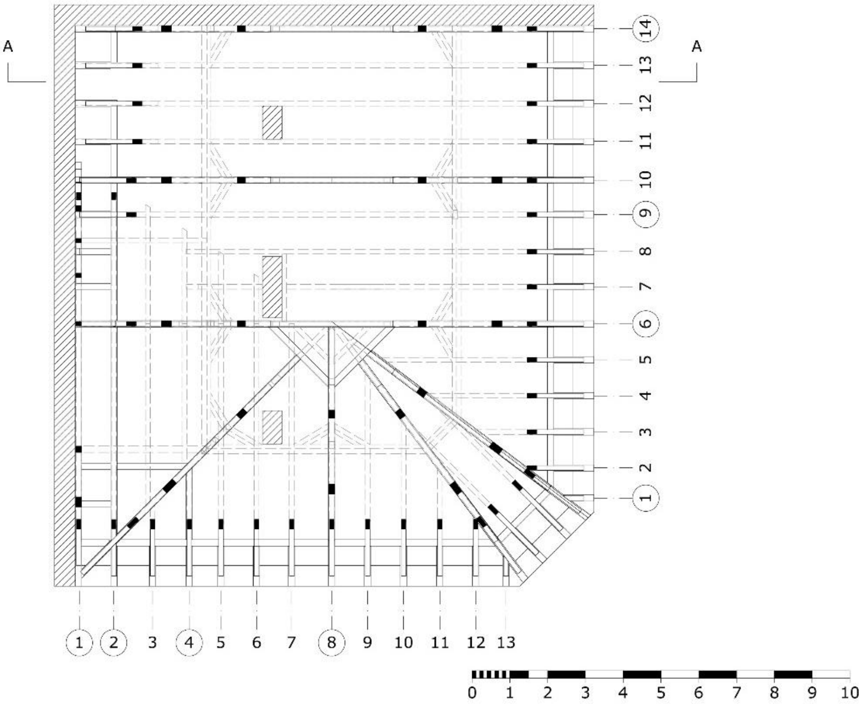


Fig. B. 12 The roof structure – plan

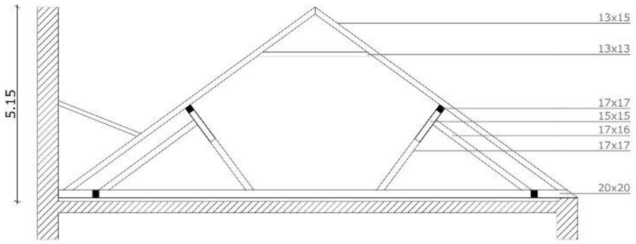


Fig. B. 13 The roof structure – section A-A

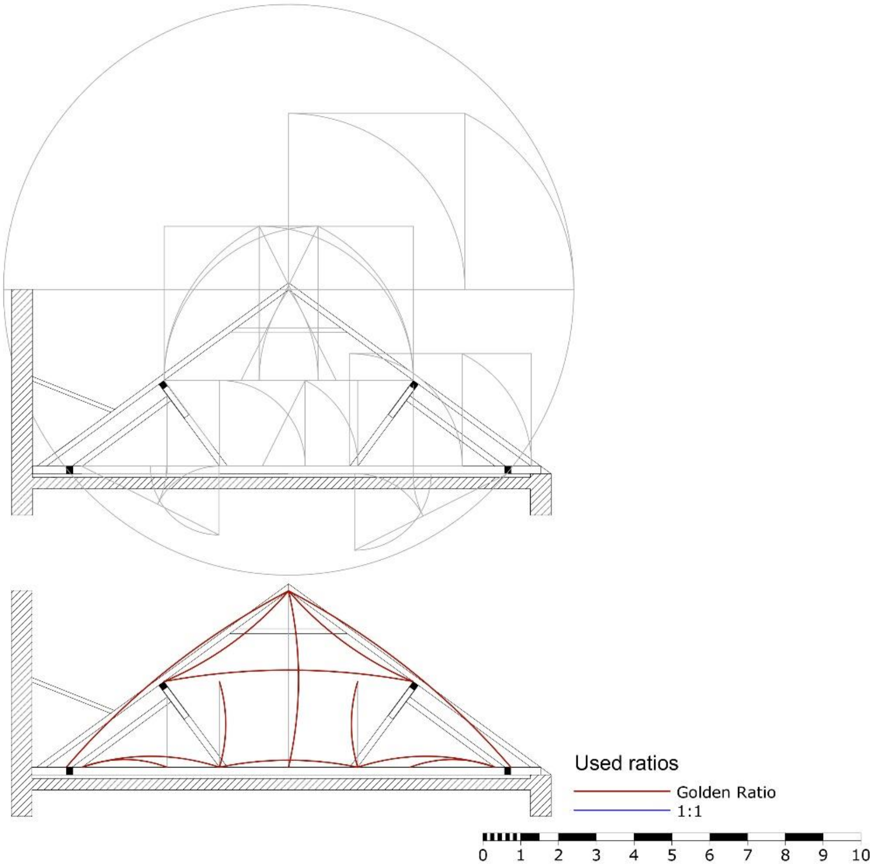


Fig. B. 14 Geometric analysis

Roof structure 1889

Fig. B. 15 Urban context and relationship with the street and building

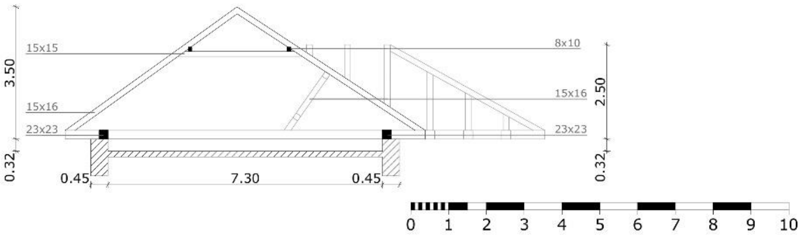


Fig. B. 16 The roof structure – section A-A

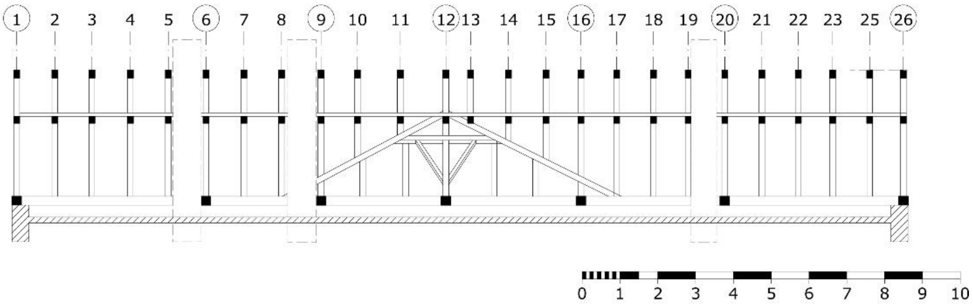


Fig. B. 17 The roof structure – section B-B

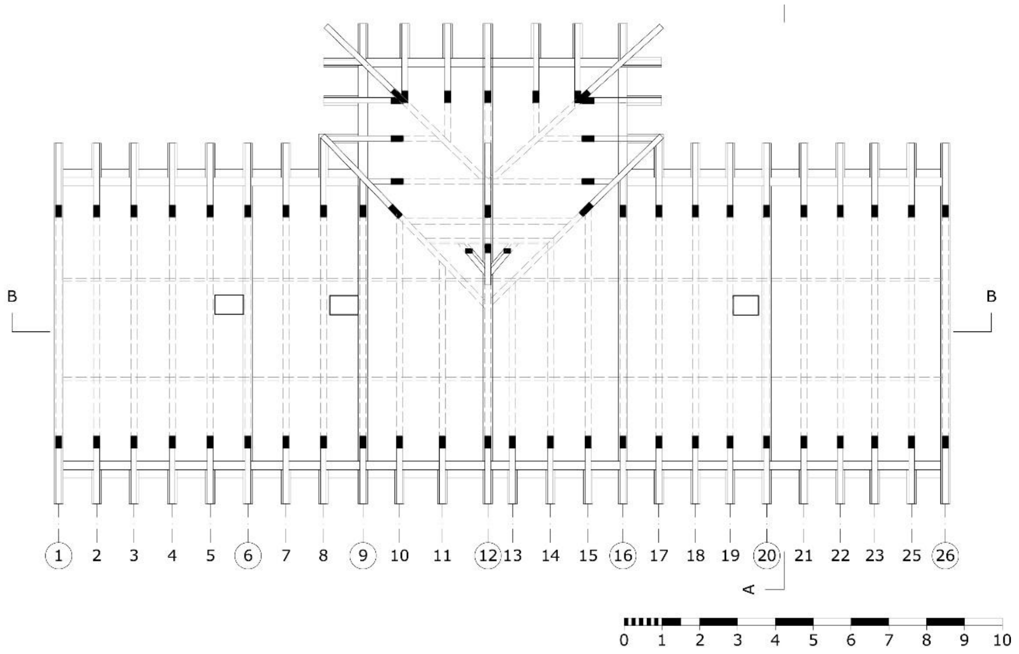


Fig. B. 18 The roof structure – section B-B

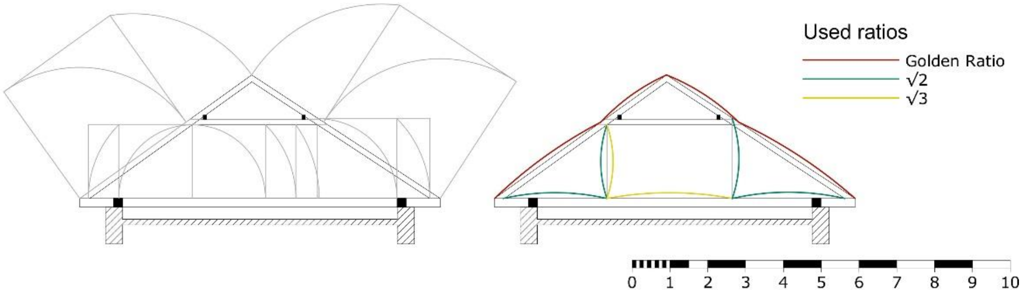


Fig. B. 19 Geometric analysis

Roof structure 1900 (1)

Fig. B. 20 Urban context and relationship with the street and building

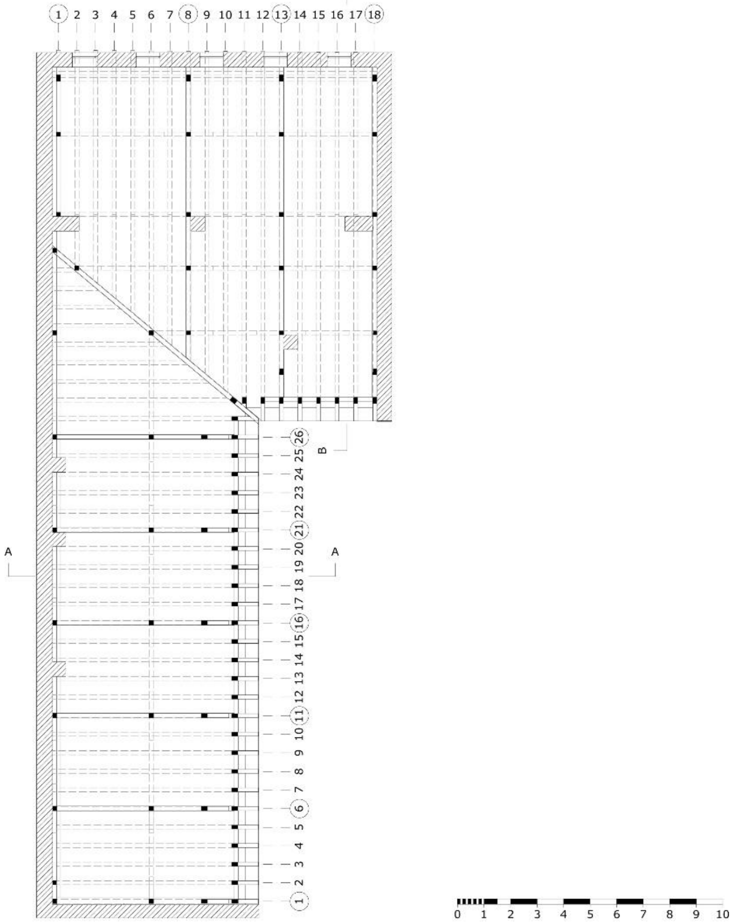


Fig. B. 21 The roof structure – plan

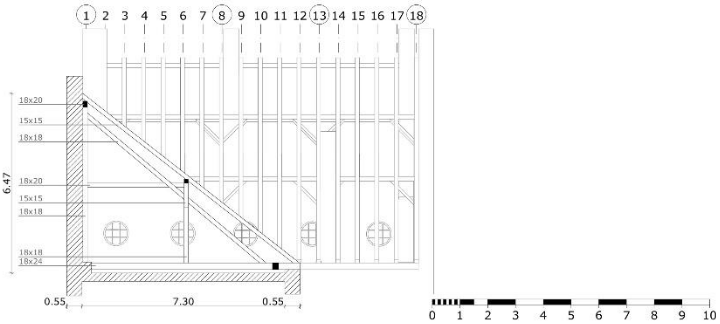


Fig. B. 22 The roof structure – section A-A

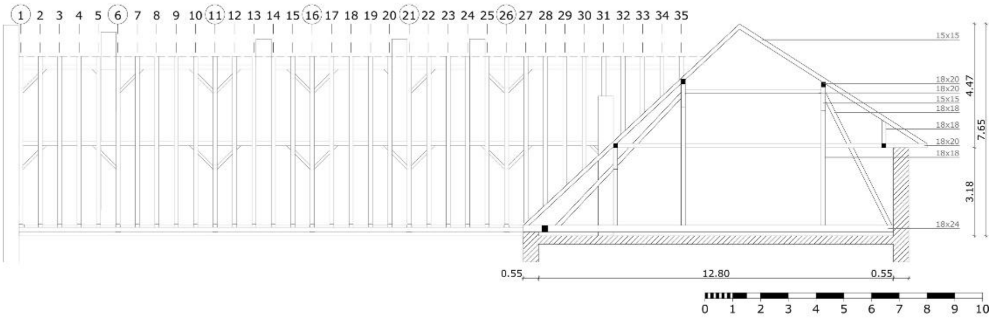


Fig. B. 23 The roof structure – section B-B

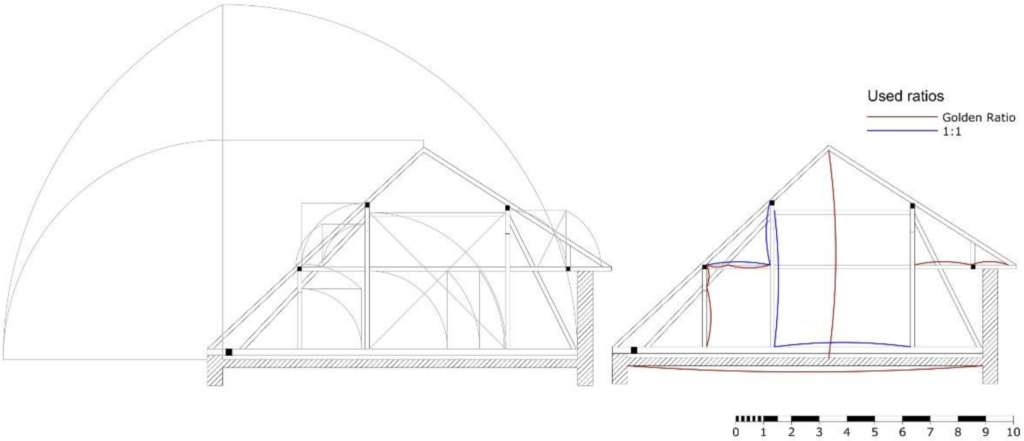


Fig. B. 24 Geometric analysis

Roof structure 1900 (2)

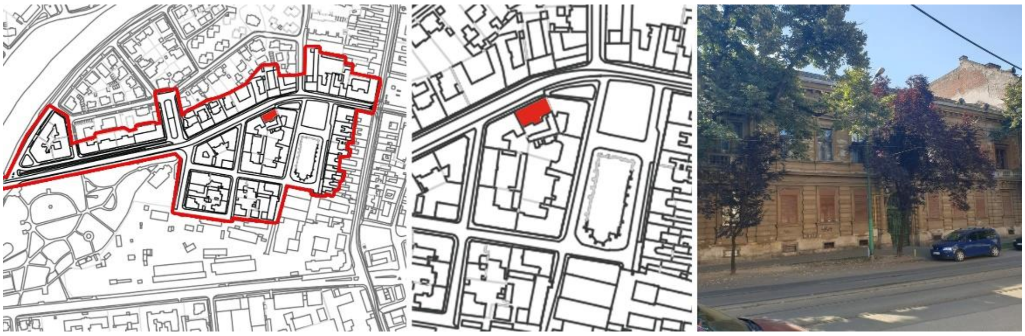


Fig. B. 25 Urban context and relationship with the street and building

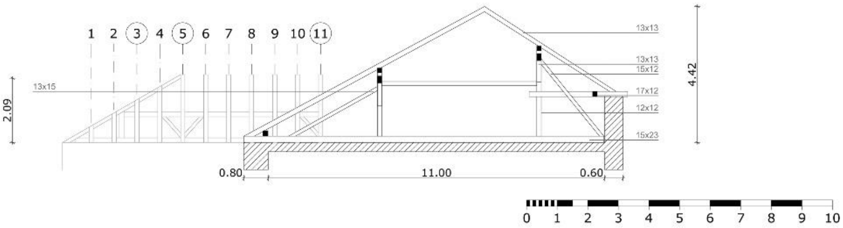


Fig. B. 26 The roof structure – section A-A

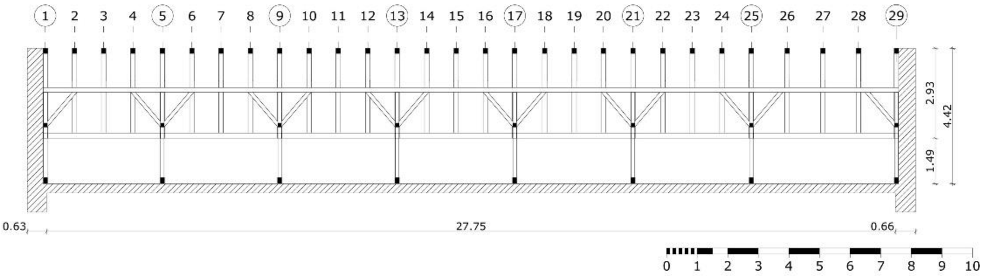


Fig. B. 27 The roof structure – section B-B

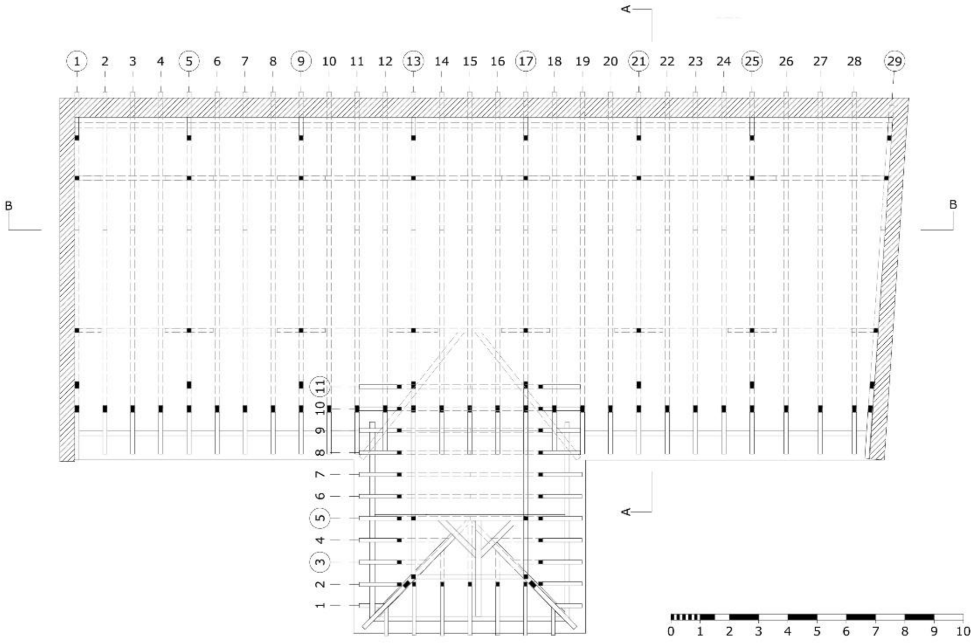


Fig. B. 28 The roof structure – plan

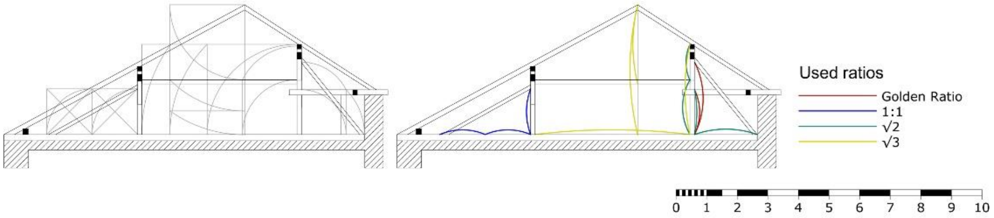


Fig. B. 29 Geometric analysis

Roof structure 1900 (3)

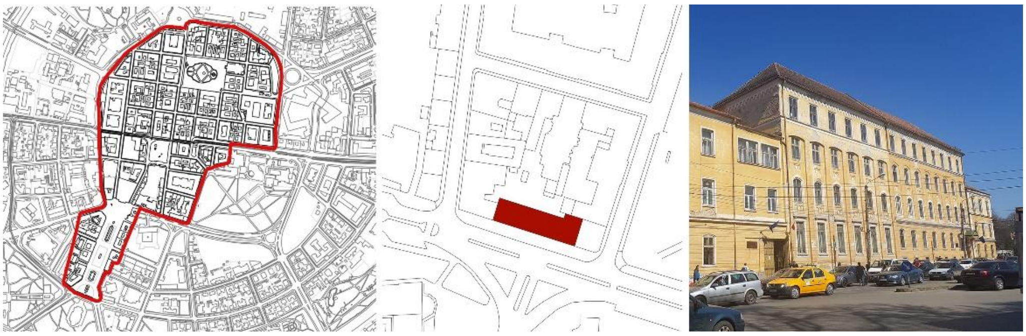


Fig. B. 30 Urban context and relationship with the street and building

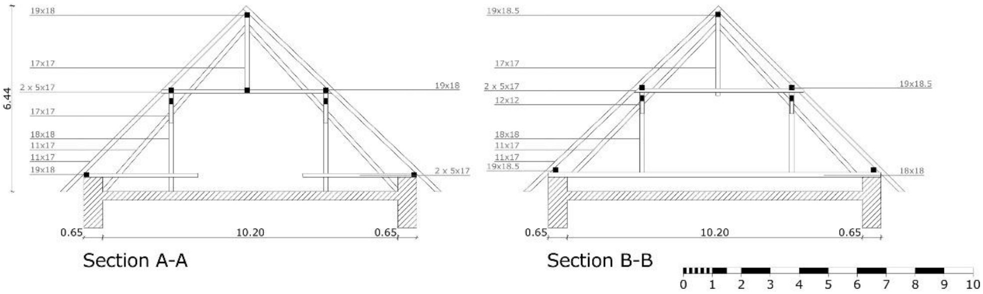


Fig. B. 31 The roof structure - sections

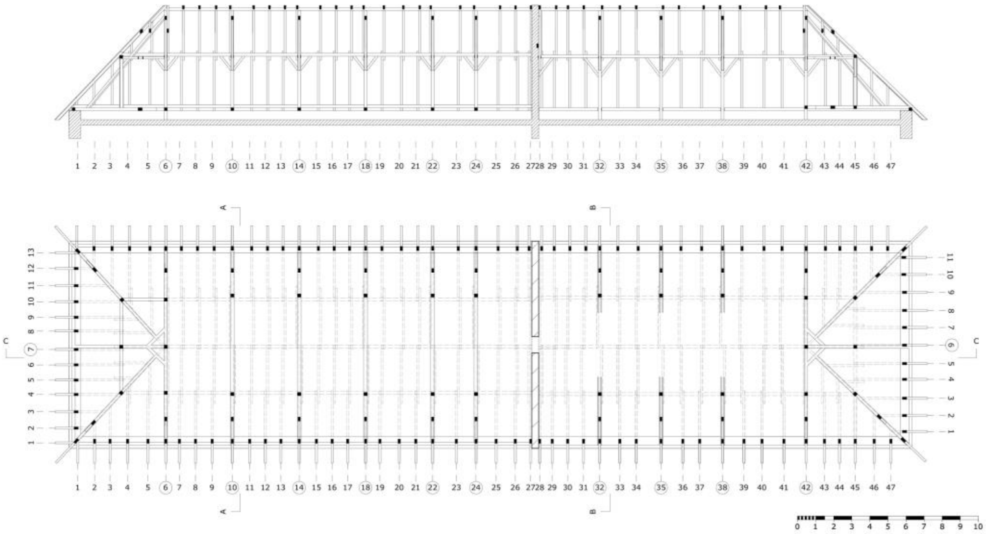


Fig. B. 32 The roof structure

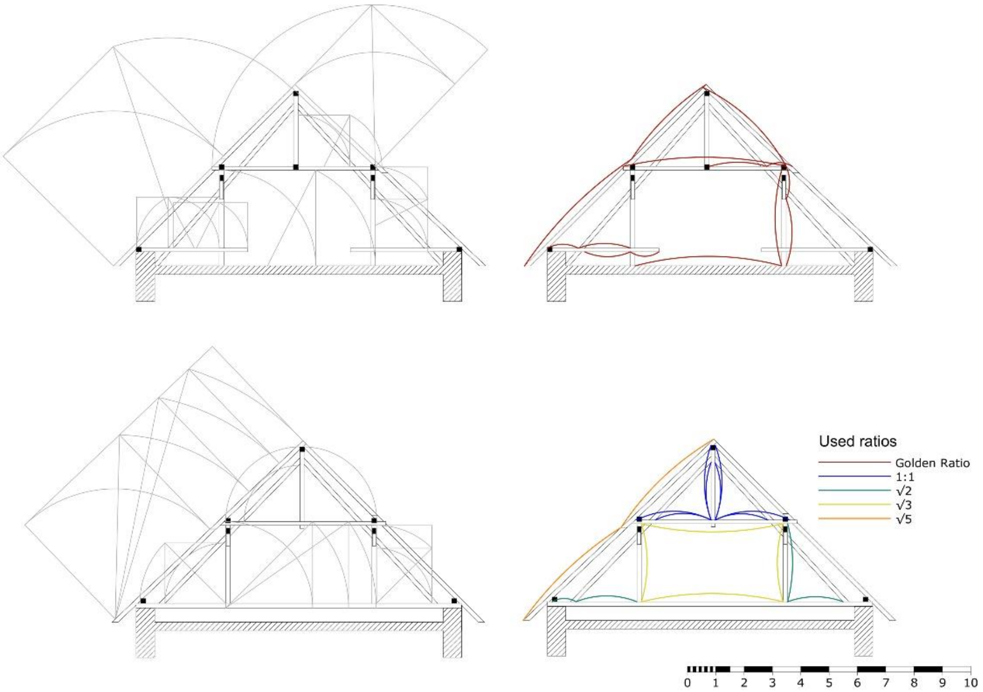


Fig. B. 33 Geometric analysis

Roof structure 1900 (4)

Fig. B. 34 Urban context and relationship with the street and building

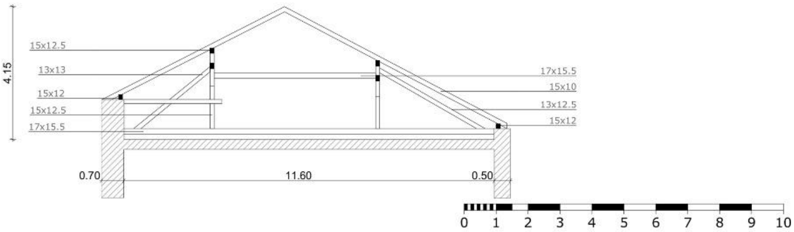


Fig. B. 35 The roof structure – section A-A

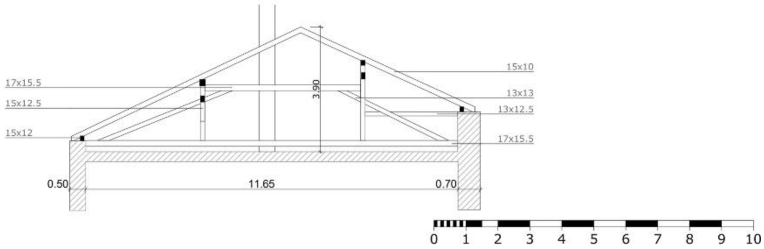


Fig. B. 36 The roof structure – section B-B

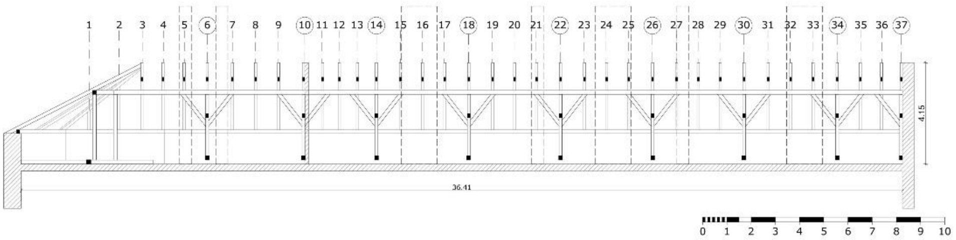


Fig. B. 37 The roof structure – section C-C

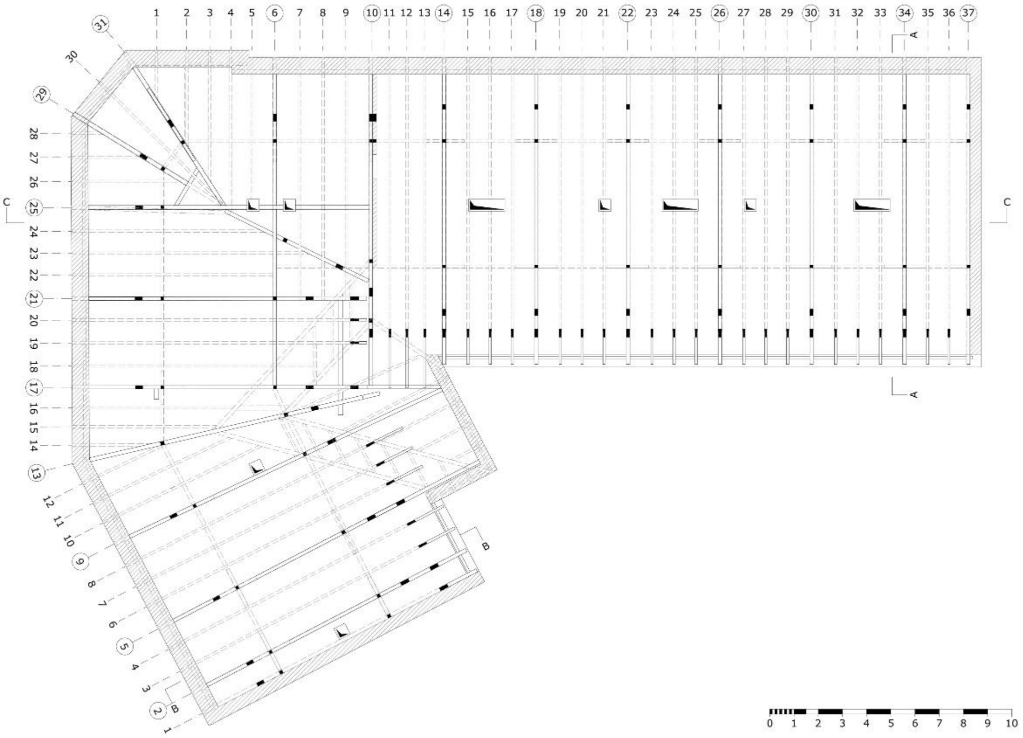


Fig. B. 38 The roof structure – section B-B

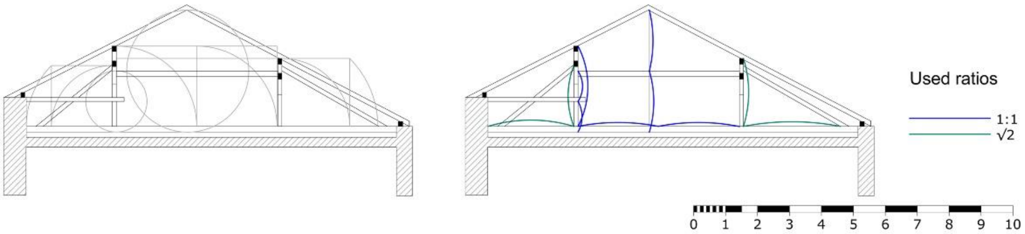


Fig. B. 39 Geometric analysis

Roof structure 1903 (1)

Fig. B. 40 Urban context and relationship with the street and building

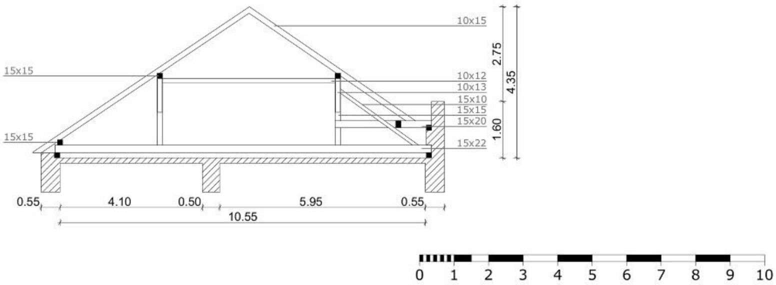


Fig. B. 41 The roof structure – section A-A

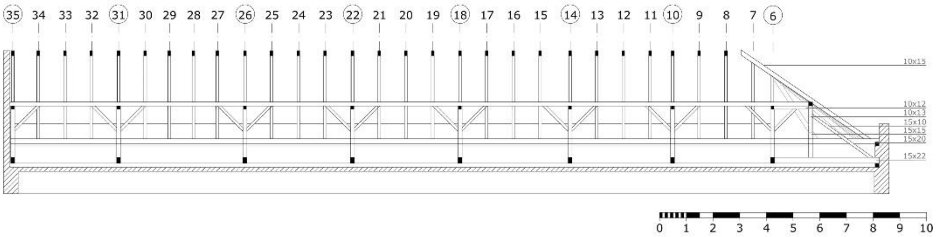


Fig. B. 42 The roof structure – section B-B

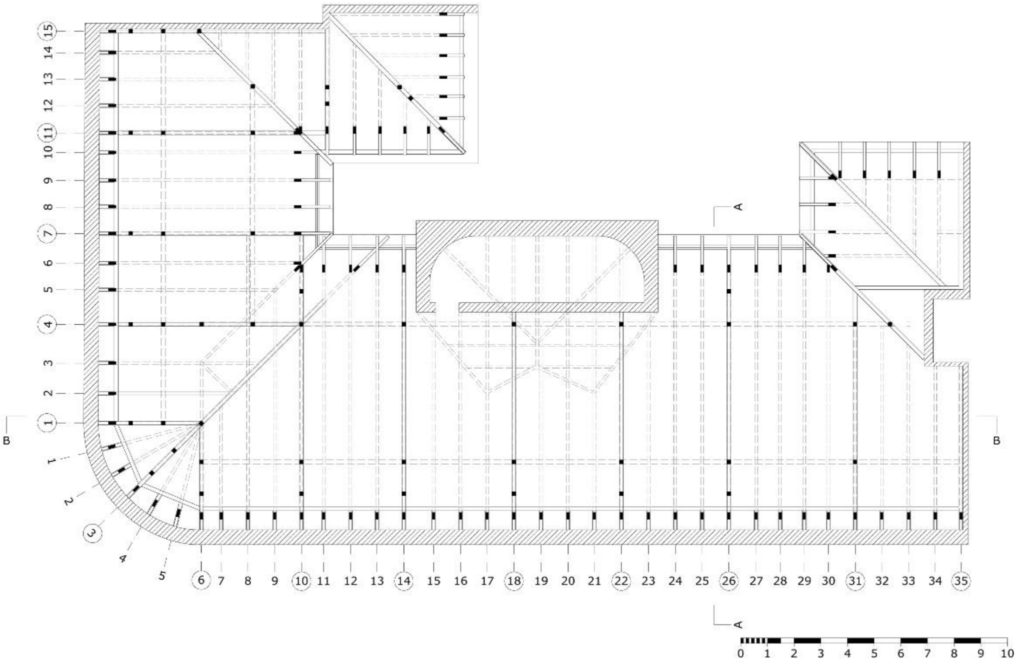


Fig. B. 43 The roof structure – plan

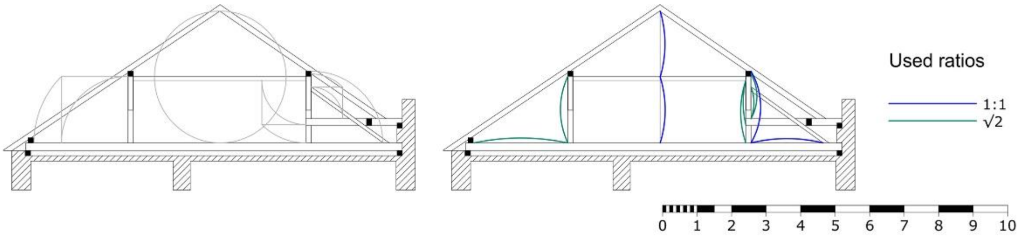


Fig. B. 44 Geometric analysis

Roof structure 1905

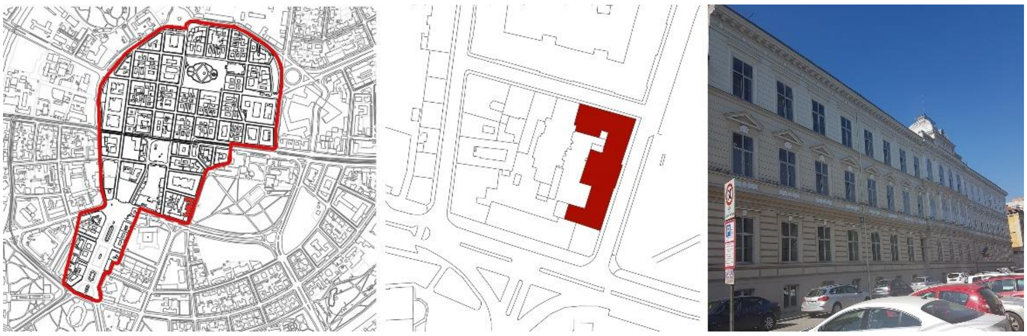


Fig. B. 45 Urban context and relationship with the street and building

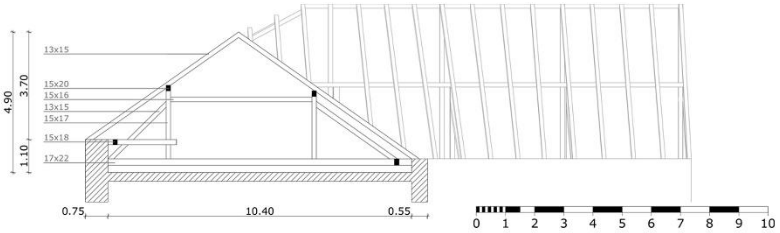


Fig. B. 46 The roof structure – section A-A

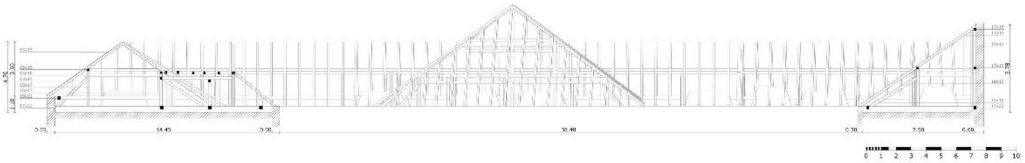


Fig. B. 47 The roof structure – section A-A

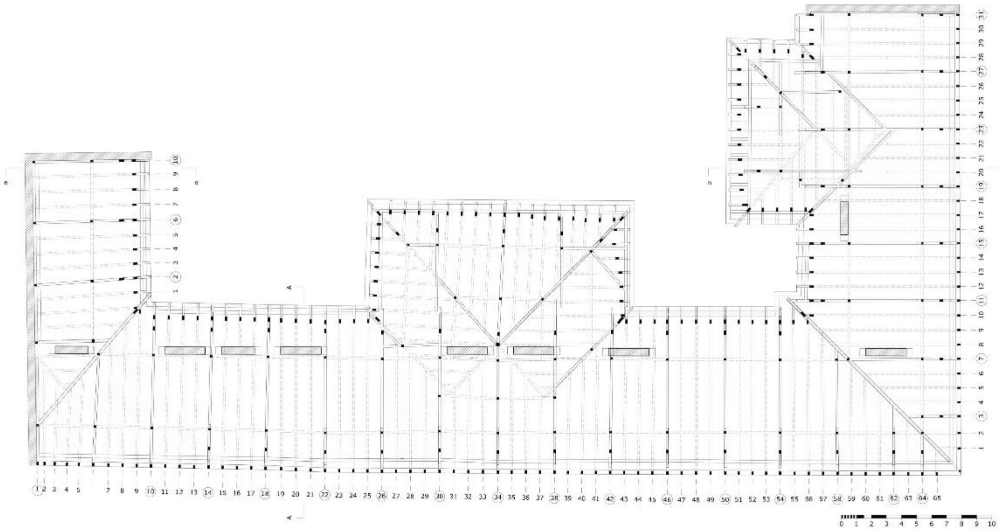


Fig. B. 48 The roof structure – section A-A

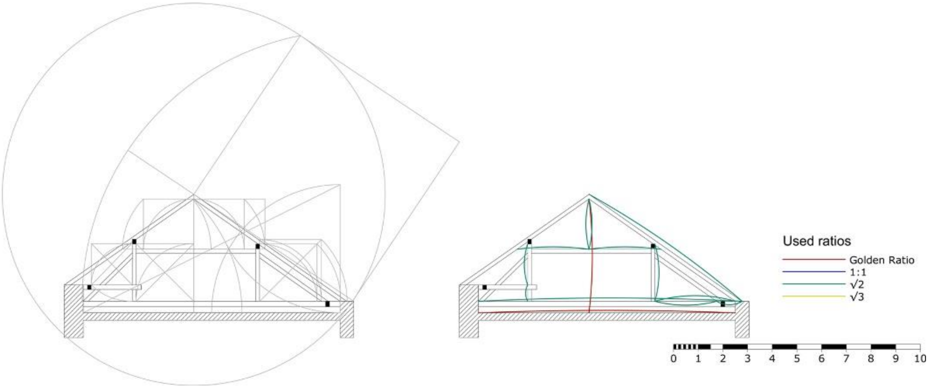


Fig. B. 49 Geometric analysis

Roof structure 1907

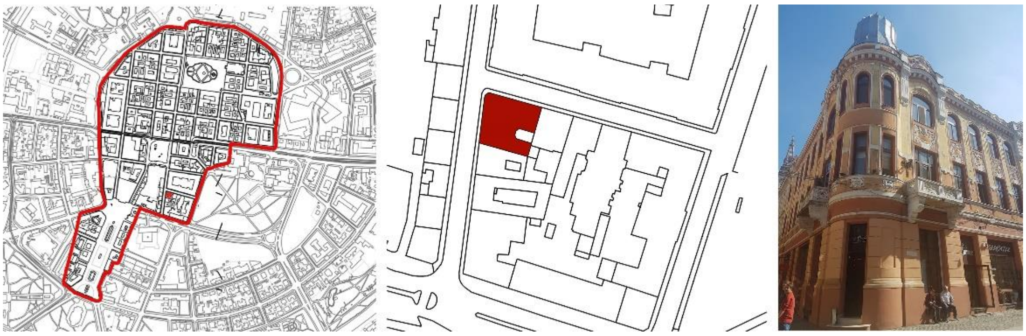


Fig. B. 50 Urban context and relationship with the street and building

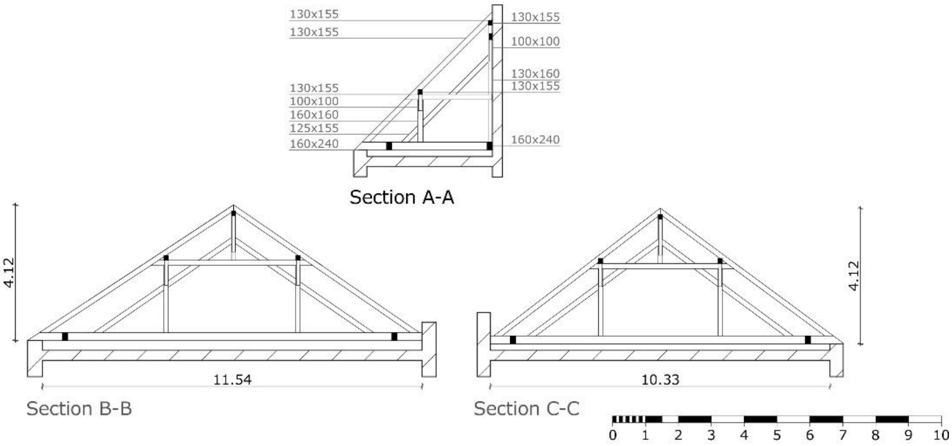


Fig. B. 51 The roof structure - sections

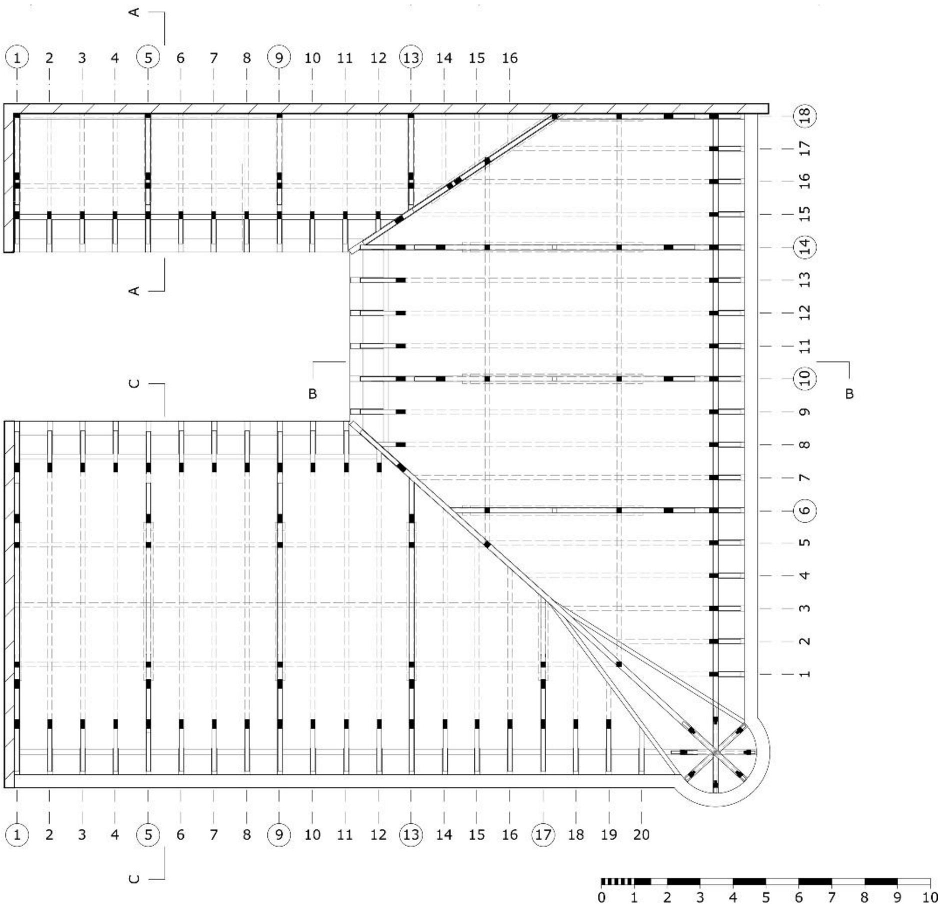


Fig. B. 52 The roof structure - plan

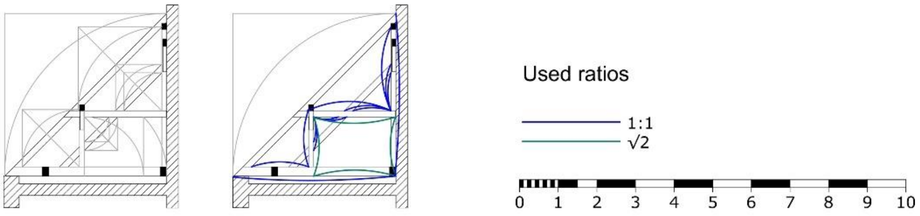


Fig. B. 53 Geometric analysis

Roof structure 1912

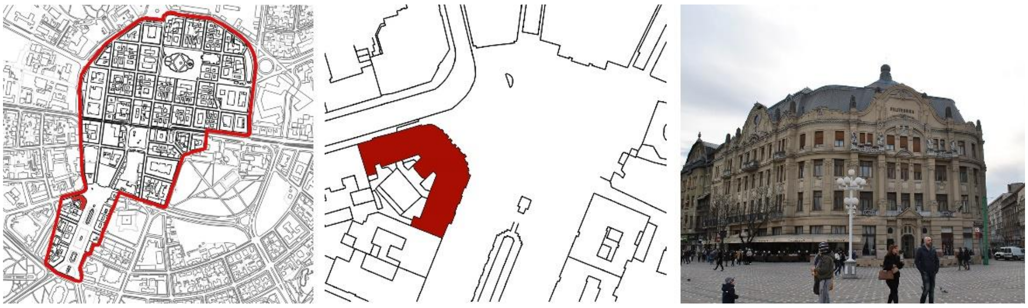


Fig. B. 54 Urban context and relationship with the street and building

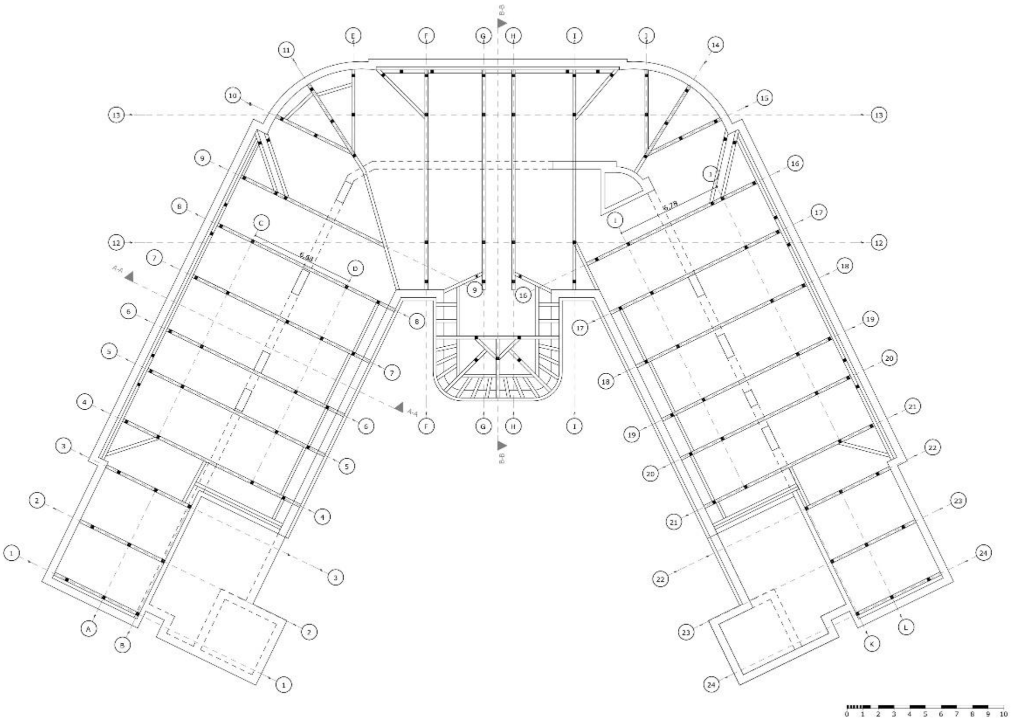


Fig. B. 55 The roof structure – plan

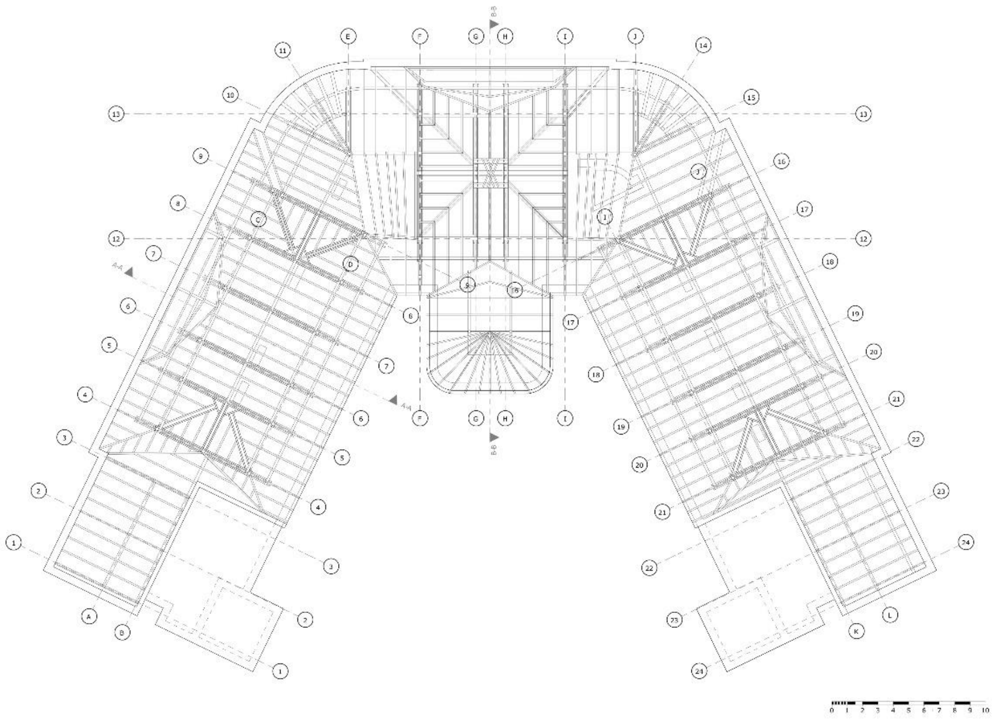


Fig. B. 56 The roof structure – top view

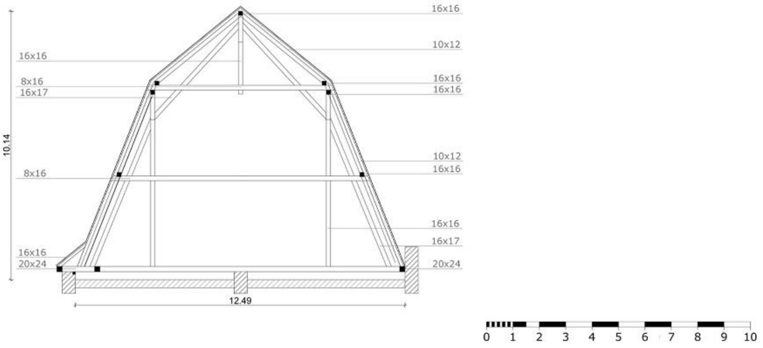


Fig. B. 57 The roof structure – section A-A

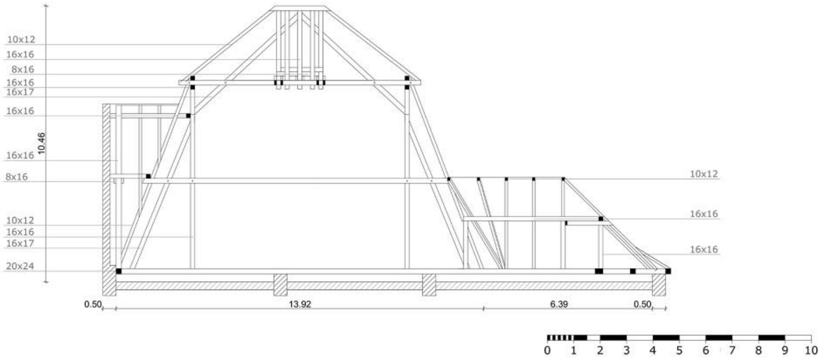


Fig. B. 58 The roof structure – section A-A

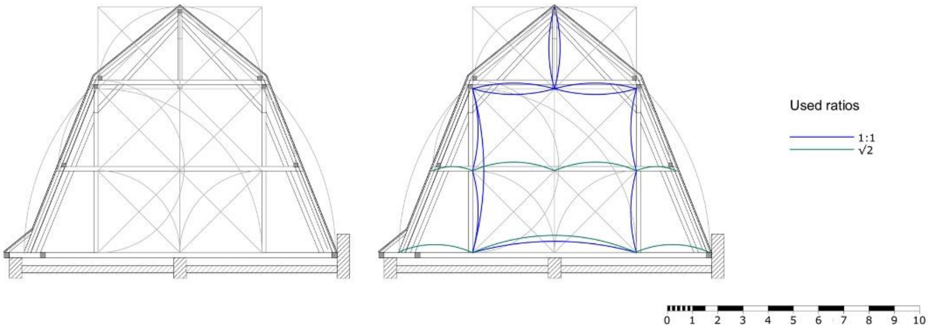


Fig. B. 59 Geometric analysis

Roof structure 1940

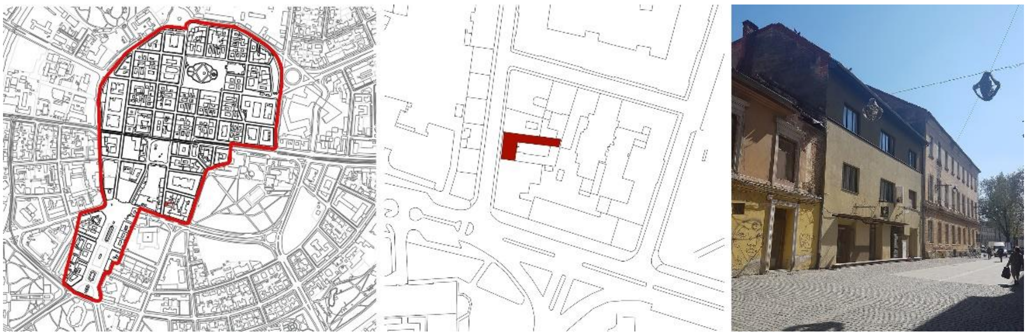


Fig. B. 60 Urban context and relationship with the street and building

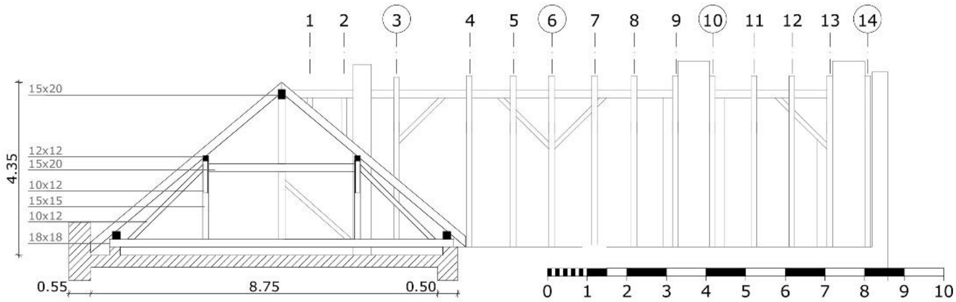


Fig. B. 61 The roof structure – section A-A

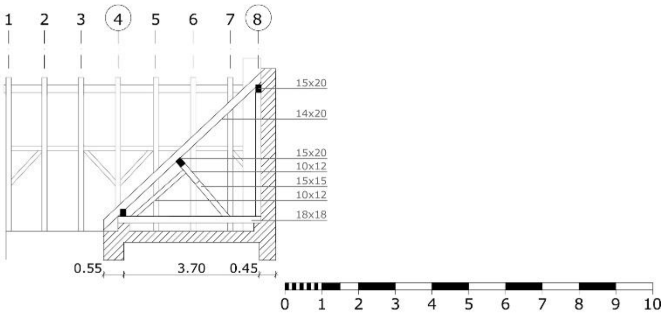


Fig. B. 62 The roof structure – section B-B

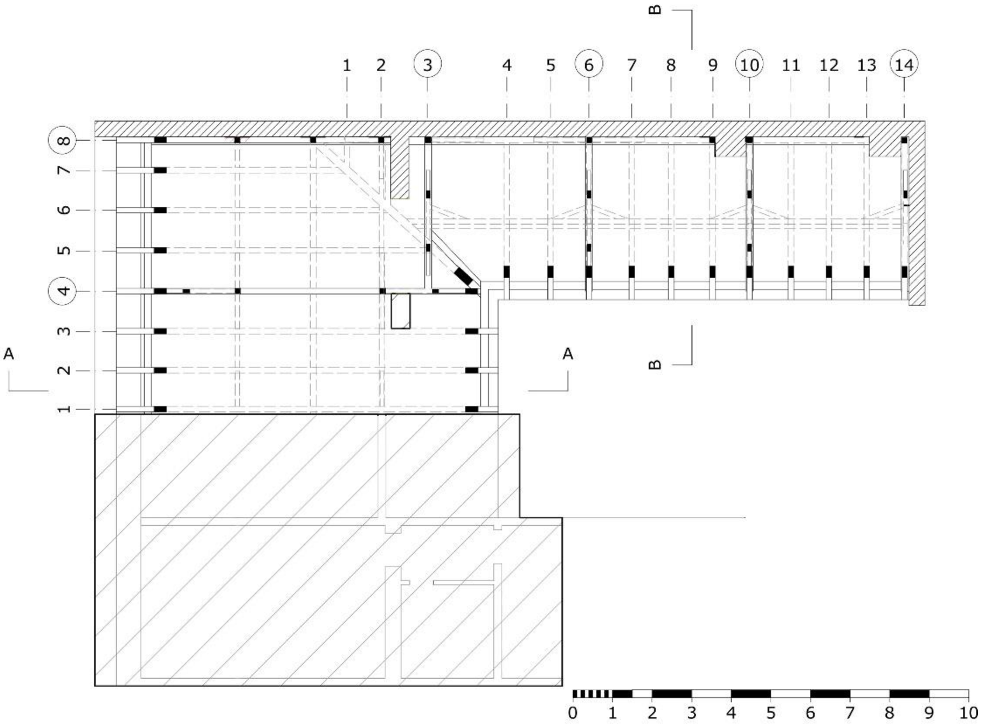


Fig. B. 63 The roof structure – plan

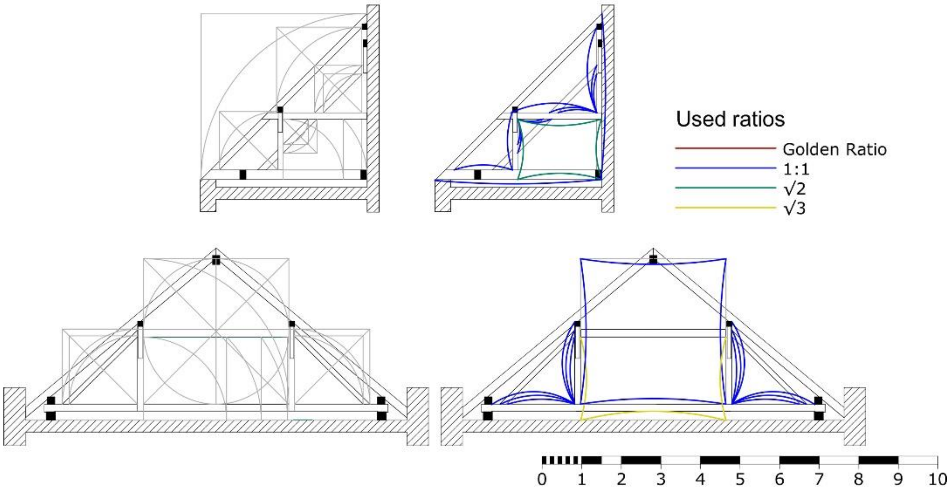
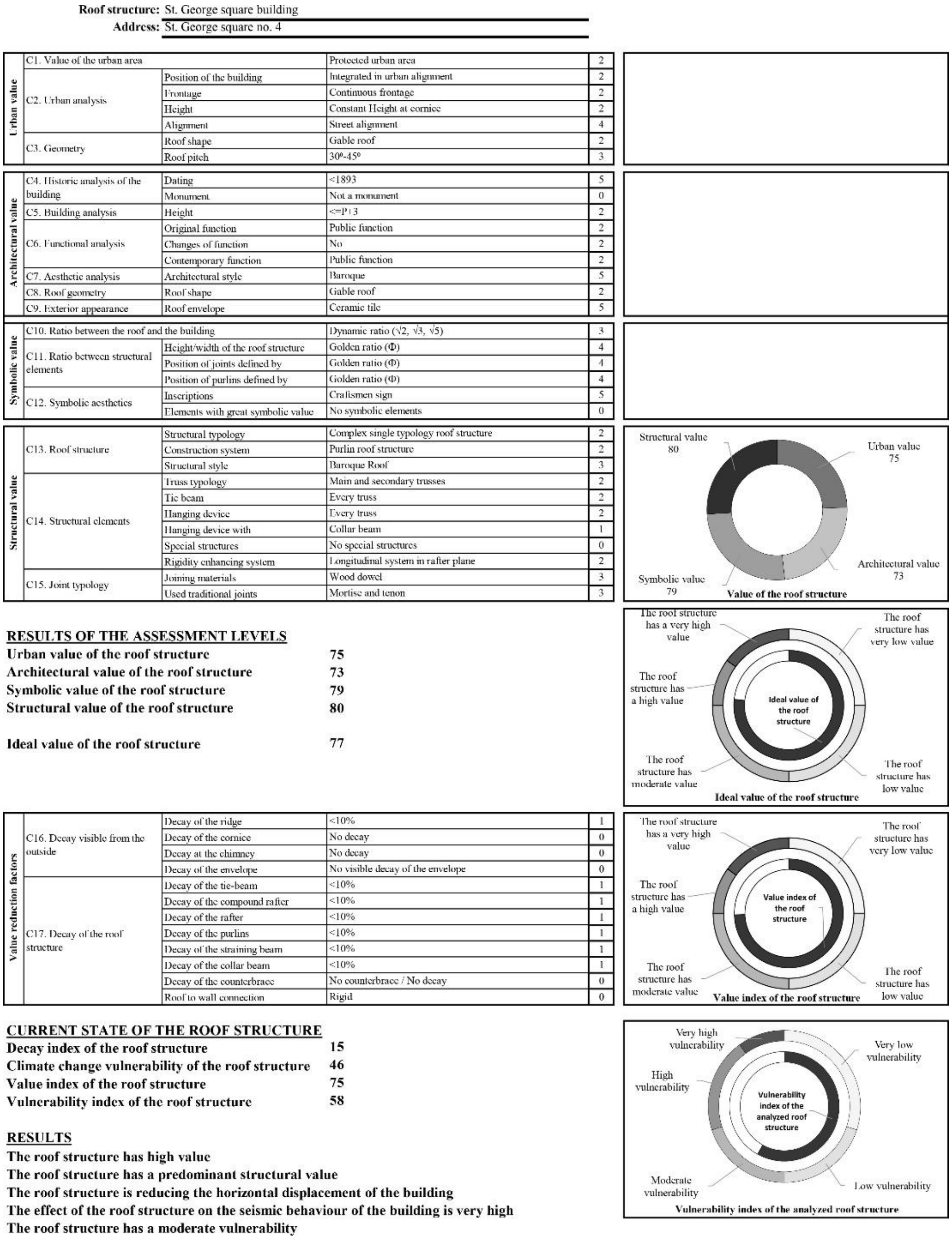


Fig. B. 64 Geometric analysis

APPENDIX C



Roof structure: St. George square building
Address: St. George square no. 4

Urban value	C1. Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30º-45º	3

Architectural value	C4. Historic analysis of the building	Dating	<1893	5
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Baroque	5
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

Symbolic value	C10. Ratio between the roof and the building		Dynamic ratio (√2, √3, √5)	3
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	Craftsmen sign	5
		Elements with great symbolic value	No symbolic elements	0

Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Baroque Roof	3
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Every truss	2
		Hanging device	Every truss	2
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	Longitudinal system in rafter plane	2
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**75**
Architectural value of the roof structure	**73**
Symbolic value of the roof structure	**79**
Structural value of the roof structure	**80**
Ideal value of the roof structure	**77**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	<10%	1
		Decay of the straining beam	<10%	1
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	No counterbrace / No decay	0
		Roof to wall connection	Rigid	0

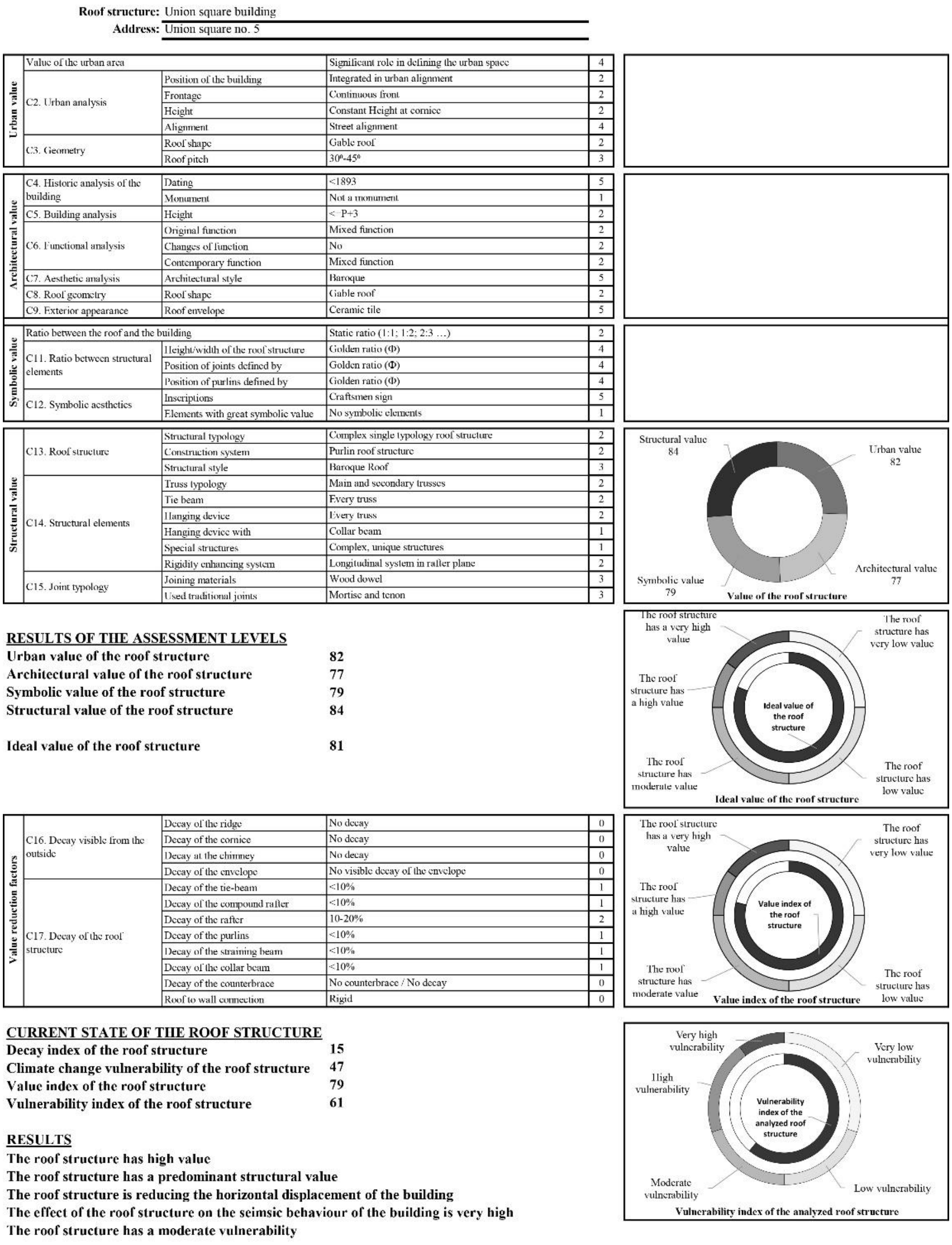
CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**15**
Climate change vulnerability of the roof structure	**46**
Value index of the roof structure	**75**
Vulnerability index of the roof structure	**58**

RESULTS

The roof structure has high value
The roof structure has a predominant structural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is very high
The roof structure has a moderate vulnerability

Fig. C. 1 Assessment sheet for the St. George building roof structure



Roof structure: Union square building

Address: Union square no. 5

Urban value	Value of the urban area		Significant role in defining the urban space	4
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous front	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30⁰-45⁰	3
Architectural value	C4. Historic analysis of the building	Dating	<1893	5
		Monument	Not a monument	1
	C5. Building analysis	Height	≤P+3	2
	C6. Functional analysis	Original function	Mixed function	2
		Changes of function	No	2
		Contemporary function	Mixed function	2
	C7. Aesthetic analysis	Architectural style	Baroque	5
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		Static ratio (1:1; 1:2; 2:3 …)	2
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	Craftsmen sign	5
		Elements with great symbolic value	No symbolic elements	1
Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Baroque Roof	3
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Every truss	2
		Hanging device	Every truss	2
		Hanging device with	Collar beam	1
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	Longitudinal system in rafter plane	2
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**82**
Architectural value of the roof structure	**77**
Symbolic value of the roof structure	**79**
Structural value of the roof structure	**84**
Ideal value of the roof structure	**81**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	<10%	1
		Decay of the straining beam	<10%	1
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	No counterbrace / No decay	0
		Roof to wall connection	Rigid	0

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**15**
Climate change vulnerability of the roof structure	**47**
Value index of the roof structure	**79**
Vulnerability index of the roof structure	**61**

RESULTS

The roof structure has high value
The roof structure has a predominant structural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seimsic behaviour of the building is very high
The roof structure has a moderate vulnerability

Fig. C. 2 Assessment sheet for the Union square building roof structure

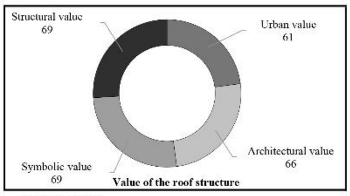
Roof structure: I.C. Bratianu high school - wing A
Address: Iancu Huniade square, no. 2

Urban value	Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Hipped roof	2
		Roof pitch	30º-45º	3

Architectural value	C4. Historic analysis of the building	Dating	1893-1900	4
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Hipped roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

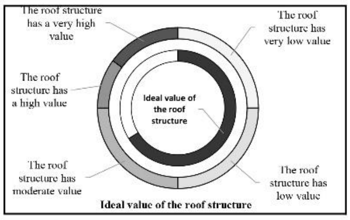
Symbolic value	Ratio between the roof and the building		Dynamic ratio (√2, √3, √5)	3
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0

Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	No hanging device	0
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	Longitudinal system in rafter plane	2
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

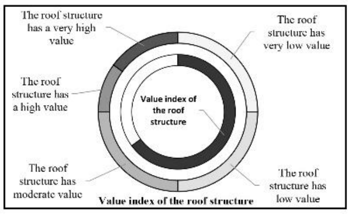


RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**61**
Architectural value of the roof structure	**66**
Symbolic value of the roof structure	**69**
Structural value of the roof structure	**69**
Ideal value of the roof structure	**66**



Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	<10%	1
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	No collar beam / No decay	0
		Decay of the counterbrace	No counterbrace / No decay	0
		Roof to wall connection	Rigid	0



CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**8**
Climate change vulnerability of the roof structure	**42**
Value index of the roof structure	**65**
Vulnerability index of the roof structure	**49**

RESULTS

The roof structure has moderate value
The roof structure has a predominant symbolic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is high
The roof structure has a low vulnerability

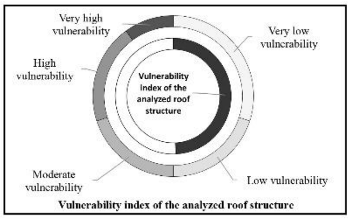
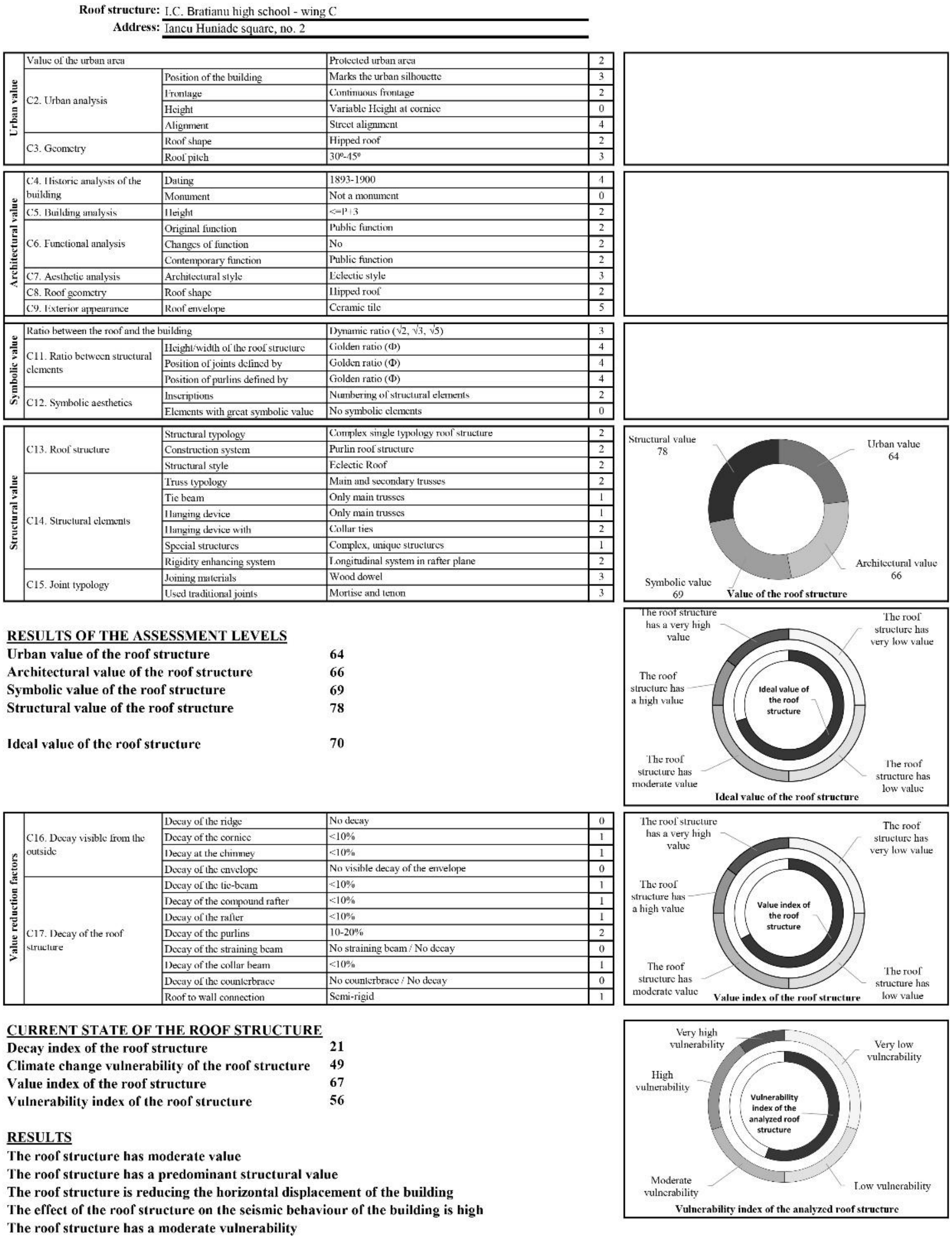


Fig. C. 3 Assessment sheet for the I.C. Bratianu high school – wing A roof structure



Roof structure: I.C. Bratianu high school - wing C
Address: Iancu Huniade square, no. 2

Urban value	Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Marks the urban silhouette	3
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Hipped roof	2
		Roof pitch	30º-45º	3

Architectural value	C4. Historic analysis of the building	Dating	1893-1900	4
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Hipped roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

Symbolic value	Ratio between the roof and the building		Dynamic ratio (√2, √3, √5)	3
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0

Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar ties	2
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	Longitudinal system in rafter plane	2
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**64**
Architectural value of the roof structure	**66**
Symbolic value of the roof structure	**69**
Structural value of the roof structure	**78**
Ideal value of the roof structure	**70**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	<10%	1
		Decay at the chimney	<10%	1
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	10-20%	2
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	No counterbrace / No decay	0
		Roof to wall connection	Semi-rigid	1

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**21**
Climate change vulnerability of the roof structure	**49**
Value index of the roof structure	**67**
Vulnerability index of the roof structure	**56**

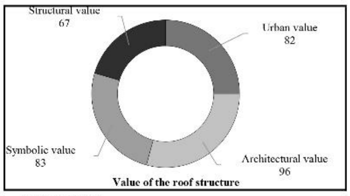
RESULTS

The roof structure has moderate value
The roof structure has a predominant structural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is high
The roof structure has a moderate vulnerability

Fig. C. 4 Assessment sheet for the I.C. Bratianu high school – wing C roof structure

Roof structure: Great Synagogue of Timisoara
Address: Mărăşeşti street no. 6

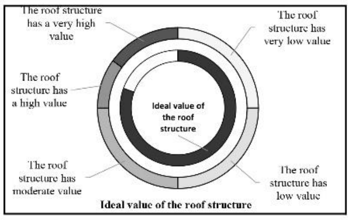
Urban value	Value of the urban area		Significant role in defining the urban space	4
	C2. Urban analysis	Position of the building	Independent building	1
		Frontage	Continuous frontage	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Pyramid Hip roof	3
		Roof pitch	30º-45º	3
Architectural value	C4. Historic analysis of the building	Dating	<1893	5
		Monument	Class A monument	3
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Religious	3
		Changes of function	No	2
		Contemporary function	Religious	3
	C7. Aesthetic analysis	Architectural style	Baroque	5
	C8. Roof geometry	Roof shape	Pyramid Hip roof	4
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		Golden ratio (Φ)	4
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	No inscriptions	0
		Elements with great symbolic value	Symbolic roof decorations	4
Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3



Value of the roof structure

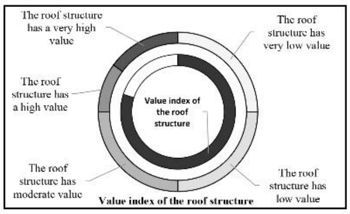
RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**82**
Architectural value of the roof structure	**96**
Symbolic value of the roof structure	**83**
Structural value of the roof structure	**67**
Ideal value of the roof structure	**81**



Ideal value of the roof structure

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	No tie-beam / No decay	0
		Decay of the compound rafter	No compound rafter / No decay	0
		Decay of the rafter	<10%	1
		Decay of the purlins	<10%	1
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	No collar beam / No decay	0
		Decay of the counterbrace	No counterbrace / No decay	0
		Roof to wall connection	Rigid	0



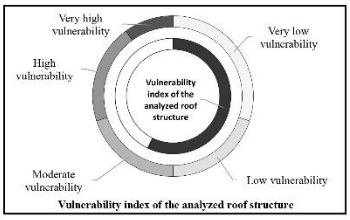
Value index of the roof structure

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**4**
Climate change vulnerability of the roof structure	**38**
Value index of the roof structure	**80**
Vulnerability index of the roof structure	**57**

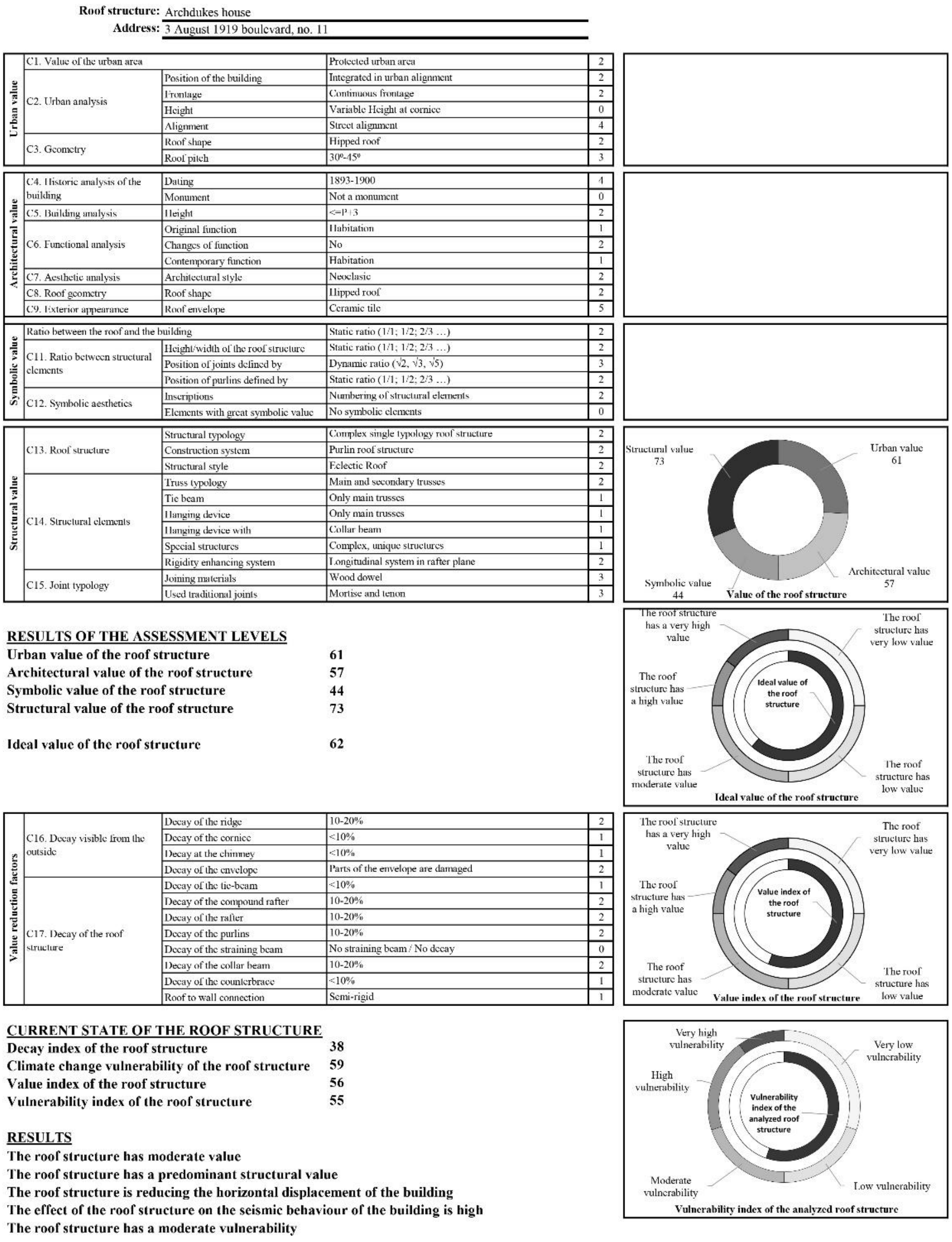
RESULTS

The roof structure has high value
The roof structure has a predominant architectural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is very high
The roof structure has a moderate vulnerability



Vulnerability index of the analyzed roof structure

Fig. C. 5 Assessment sheet for the Great Synagogue of Timisoara roof structure



Roof structure: Archdukes house
Address: 3 August 1919 boulevard, no. 11

Urban value	C1. Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Hipped roof	2
		Roof pitch	30º-45º	3
Architectural value	C4. Historic analysis of the building	Dating	1893-1900	4
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Habitation	1
		Changes of function	No	2
		Contemporary function	Habitation	1
	C7. Aesthetic analysis	Architectural style	Neoclasic	2
	C8. Roof geometry	Roof shape	Hipped roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Dynamic ratio (√2, √3, √5)	3
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	Longitudinal system in rafter plane	2
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**61**
Architectural value of the roof structure	**57**
Symbolic value of the roof structure	**44**
Structural value of the roof structure	**73**
Ideal value of the roof structure	**62**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	10-20%	2
		Decay of the cornice	<10%	1
		Decay at the chimney	<10%	1
		Decay of the envelope	Parts of the envelope are damaged	2
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	10-20%	2
		Decay of the rafter	10-20%	2
		Decay of the purlins	10-20%	2
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	10-20%	2
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**38**
Climate change vulnerability of the roof structure	**59**
Value index of the roof structure	**56**
Vulnerability index of the roof structure	**55**

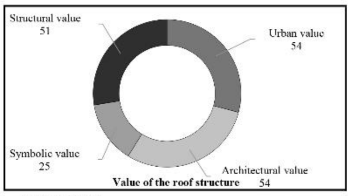
RESULTS

The roof structure has moderate value
The roof structure has a predominant structural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is high
The roof structure has a moderate vulnerability

Fig. C. 6 Assessment sheet for the Archduke building roof structure

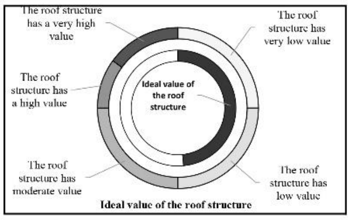
Roof structure: Residential building 4
Address: King Carol I boulevard, no. 4

Urban value	C1. Value of the urban area		No valuable context/ No defining role in the urban space	0
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30º-45º	3
Architectural value	C4. Historic analysis of the building	Dating	1912-1936	2
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Habitation	1
		Changes of function	No	2
		Contemporary function	Habitation	1
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		No ratio	0
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	No inscriptions	0
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Simplified and standardized roof structure	1
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Mechanical fasteners (nails, screws)	1
		Used traditional joints	Mortise and tenon	3

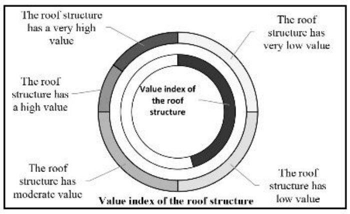


RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**54**
Architectural value of the roof structure	**54**
Symbolic value of the roof structure	**25**
Structural value of the roof structure	**51**
Ideal value of the roof structure	**48**

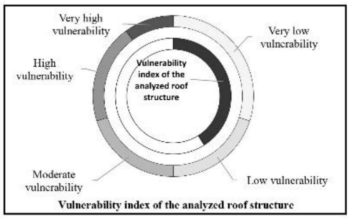


Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	<10%	1
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1



CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**19**
Climate change vulnerability of the roof structure	**48**
Value index of the roof structure	**46**
Vulnerability index of the roof structure	**41**



RESULTS

The roof structure has low value
The roof structure has a predominant architectural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is low
The roof structure has a low vulnerability

Fig. C. 7 Assessment sheet for the residential building no.4 roof structure

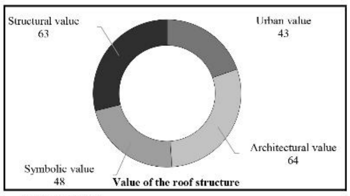
Roof structure: Residential building 2
Address: Bolyai János street

Urban value	Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Discontinuous frontage	0
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	15º-30º	2

Architectural value	C4. Historic analysis of the building	Dating	1893-1900	4
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	Yes	1
		Contemporary function	Habitation	1
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Mix of shapes	5
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

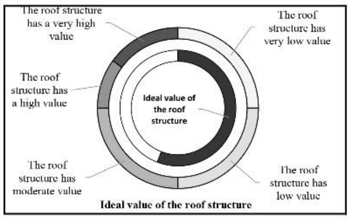
Symbolic value	Ratio between the roof and the building		No ratio	0
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0

Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

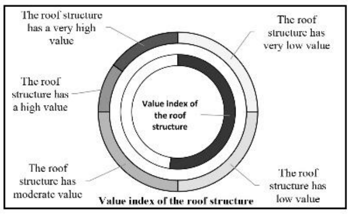


RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**43**
Architectural value of the roof structure	**64**
Symbolic value of the roof structure	**48**
Structural value of the roof structure	**63**
Ideal value of the roof structure	**56**



Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	10-20%	2
		Decay of the straining beam	<10%	1
		Decay of the collar beam	No collar beam / No decay	0
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1



CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**23**
Climate change vulnerability of the roof structure	**51**
Value index of the roof structure	**52**
Vulnerability index of the roof structure	**47**

RESULTS

The roof structure has moderate value
The roof structure has a predominant architectural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is high
The roof structure has a low vulnerability

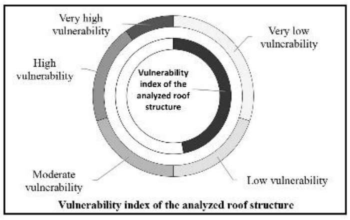


Fig. C. 8 Assessment sheet for the residential building no.2 roof structure

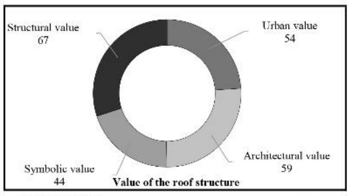
Roof structure: Residential building 3
Address: 3 August 1919 boulevard

Urban value	C1. Value of the urban area		No valuable context/ No defining role in the urban space	0
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30º-45º	3

Architectural value	C4. Historic analysis of the building	Dating	1900-1912	3
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Habitation	1
		Changes of function	No	2
		Contemporary function	Habitation	1
	C7. Aesthetic analysis	Architectural style	Seccesion	4
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

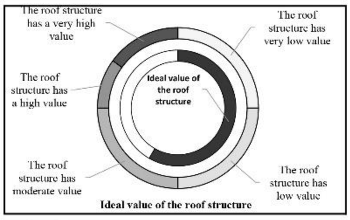
Symbolic value	Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Dynamic ratio (√2, √3, √5)	3
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0

Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

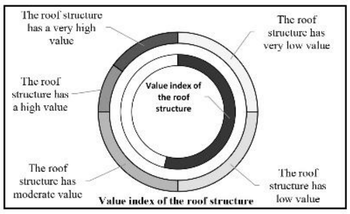


RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**54**
Architectural value of the roof structure	**59**
Symbolic value of the roof structure	**44**
Structural value of the roof structure	**67**
Ideal value of the roof structure	**58**



Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	10-20%	2
		Decay of the compound rafter	10-20%	2
		Decay of the rafter	10-20%	2
		Decay of the purlins	20-30%	3
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	10-20%	2
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1



CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**29**
Climate change vulnerability of the roof structure	**57**
Value index of the roof structure	**54**
Vulnerability index of the roof structure	**51**

RESULTS

The roof structure has moderate value
The roof structure has a predominant structural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is moderate
The roof structure has a moderate vulnerability

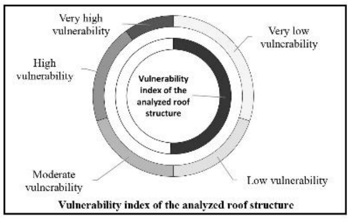
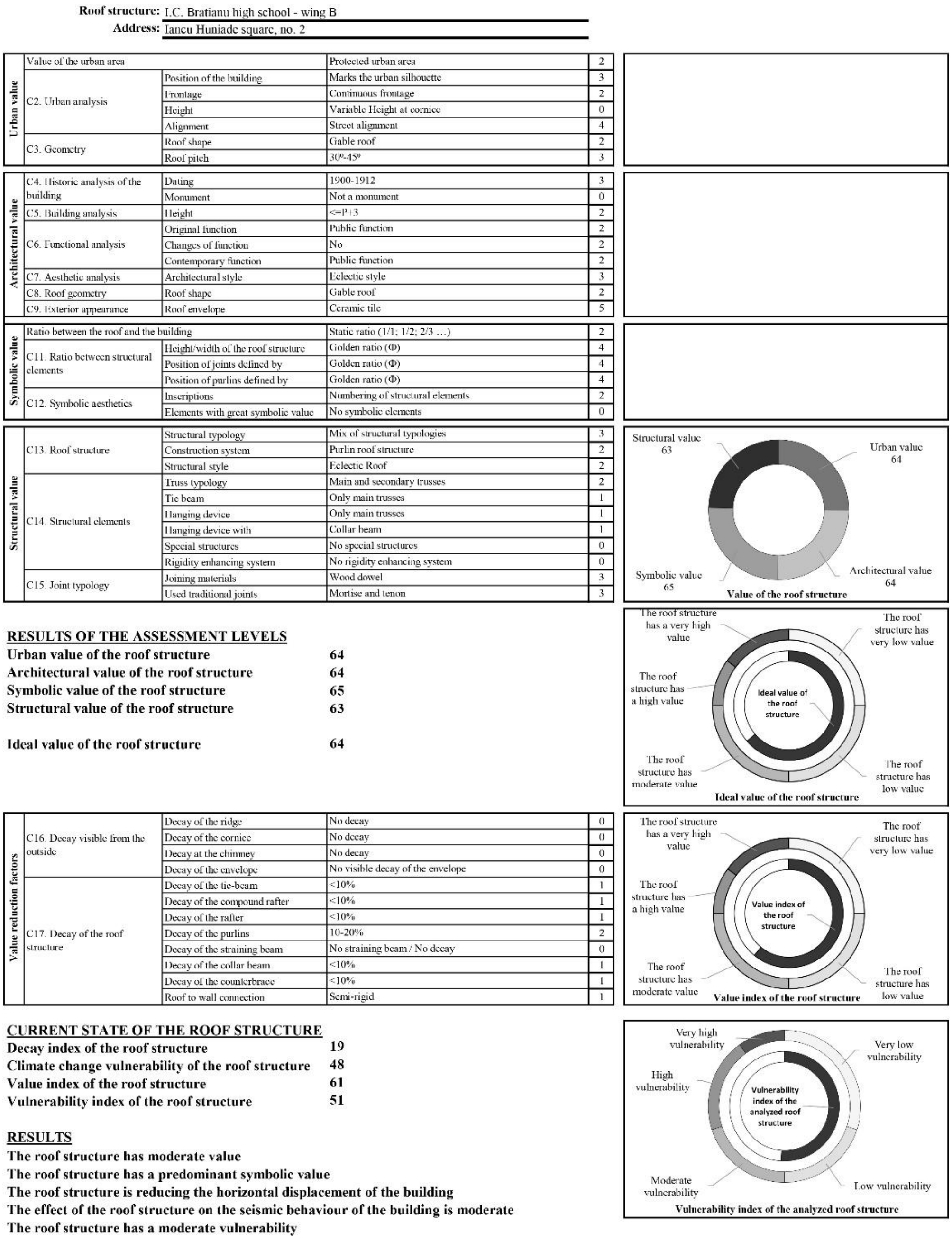


Fig. C. 9 Assessment sheet for the residential building no.3 roof structure



Roof structure: I.C. Bratianu high school - wing B
Address: Iancu Huniade square, no. 2

Urban value	Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Marks the urban silhouette	3
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30º-45º	3

Architectural value	C4. Historic analysis of the building	Dating	1900-1912	3
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

Symbolic value	Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Golden ratio (Φ)	4
		Position of purlins defined by	Golden ratio (Φ)	4
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0

Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**64**
Architectural value of the roof structure	**64**
Symbolic value of the roof structure	**65**
Structural value of the roof structure	**63**
Ideal value of the roof structure	**64**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	10-20%	2
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1

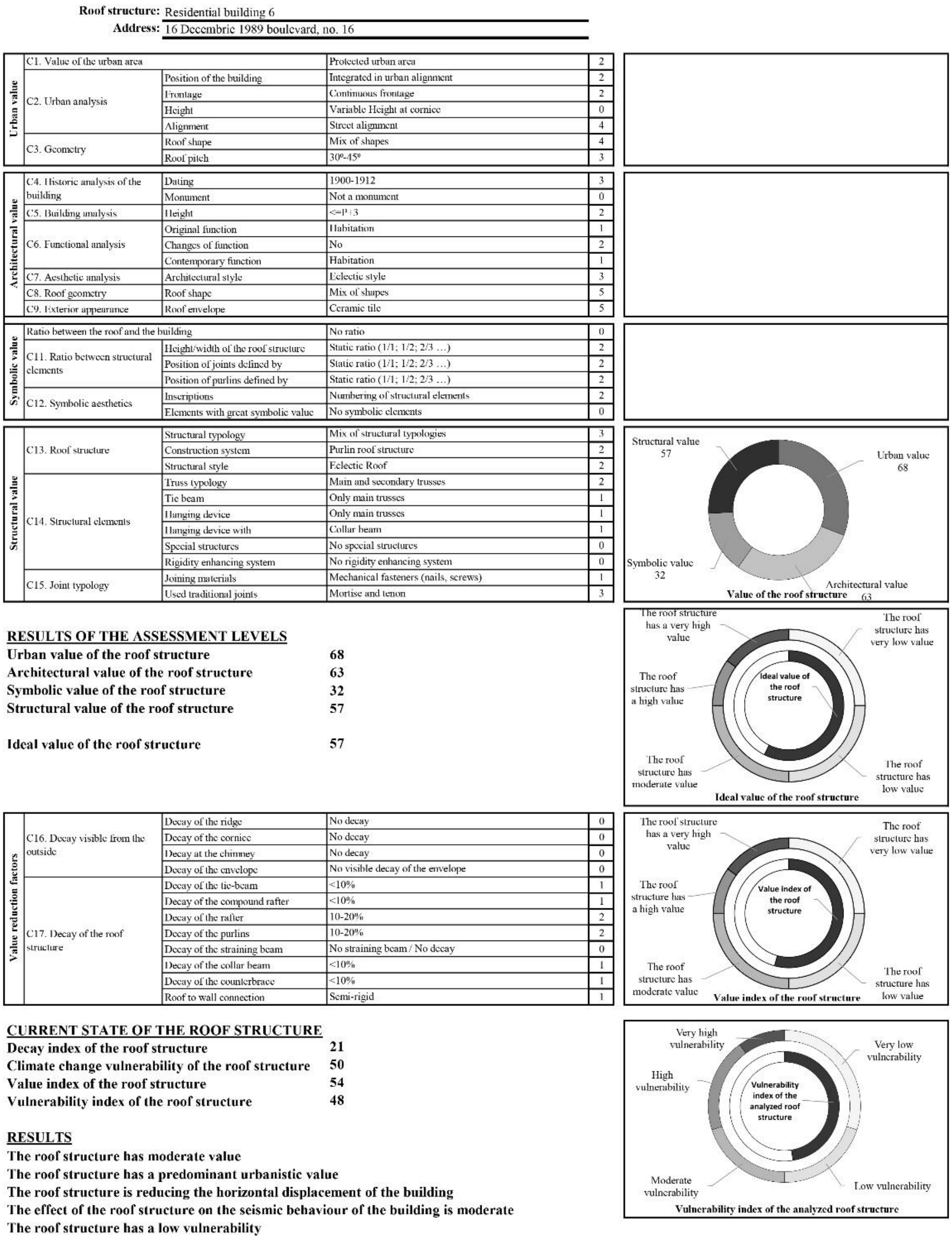
CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**19**
Climate change vulnerability of the roof structure	**48**
Value index of the roof structure	**61**
Vulnerability index of the roof structure	**51**

RESULTS

The roof structure has moderate value
The roof structure has a predominant symbolic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is moderate
The roof structure has a moderate vulnerability

Fig. C. 10 Assessment sheet for the I.C. Bratianu high school – wing B roof structure



Roof structure: Residential building 6
Address: 16 Decembrie 1989 boulevard, no. 16

Urban value	C1. Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Mix of shapes	4
		Roof pitch	30º-45º	3
Architectural value	C4. Historic analysis of the building	Dating	1900-1912	3
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Habitation	1
		Changes of function	No	2
		Contemporary function	Habitation	1
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Mix of shapes	5
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		No ratio	0
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Mechanical fasteners (nails, screws)	1
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**68**
Architectural value of the roof structure	**63**
Symbolic value of the roof structure	**32**
Structural value of the roof structure	**57**
Ideal value of the roof structure	**57**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	10-20%	2
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1

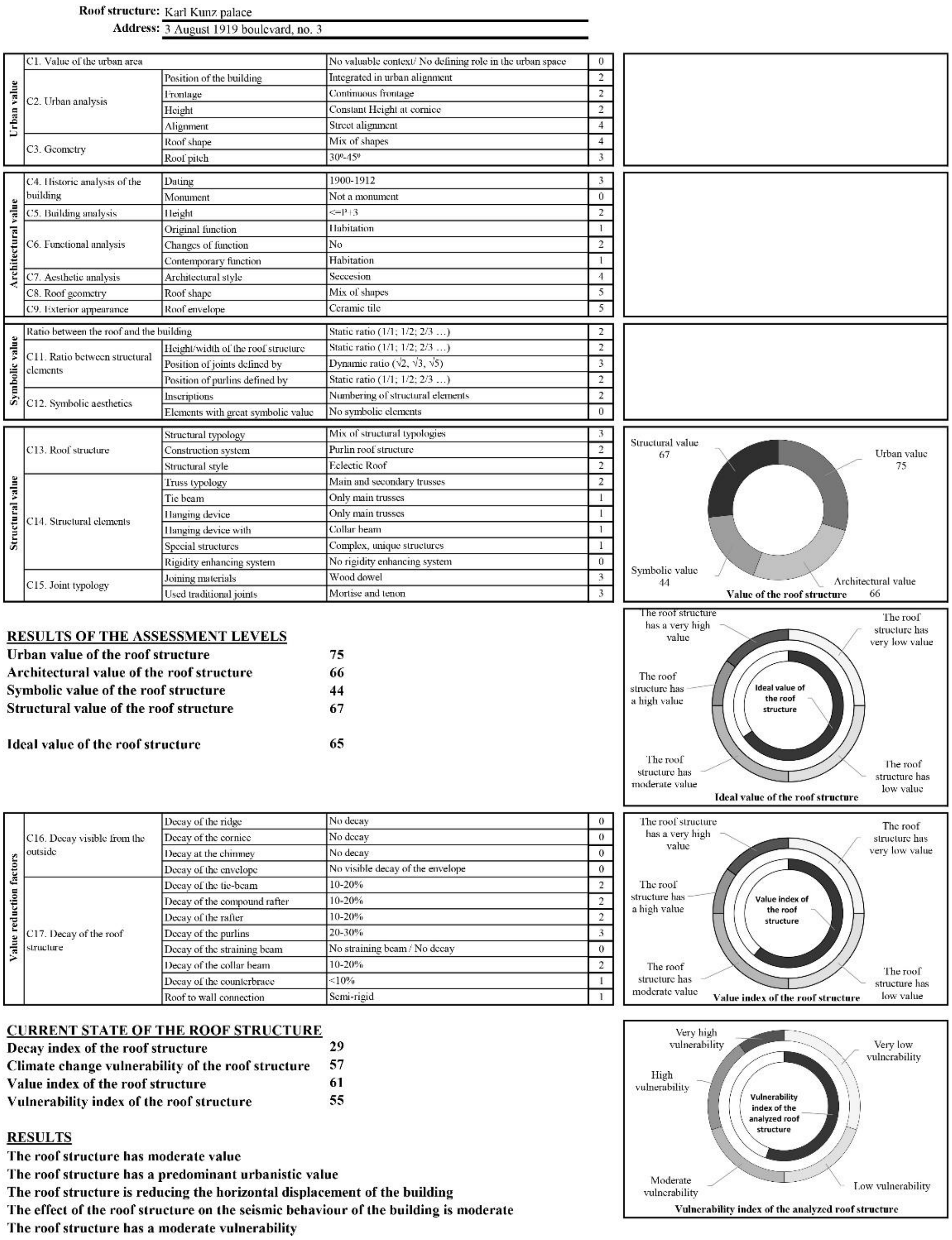
CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**21**
Climate change vulnerability of the roof structure	**50**
Value index of the roof structure	**54**
Vulnerability index of the roof structure	**48**

RESULTS

The roof structure has moderate value
The roof structure has a predominant urbanistic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is moderate
The roof structure has a low vulnerability

Fig. C. 11 Assessment sheet for the residential building no.6 roof structure



Roof structure: Karl Kunz palace
Address: 3 August 1919 boulevard, no. 3

Urban value	C1. Value of the urban area		No valuable context/ No defining role in the urban space	0
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Mix of shapes	4
		Roof pitch	30º-45º	3
Architectural value	C4. Historic analysis of the building	Dating	1900-1912	3
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Habitation	1
		Changes of function	No	2
		Contemporary function	Habitation	1
	C7. Aesthetic analysis	Architectural style	Seccesion	4
	C8. Roof geometry	Roof shape	Mix of shapes	5
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Dynamic ratio (√2, √3, √5)	3
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**75**
Architectural value of the roof structure	**66**
Symbolic value of the roof structure	**44**
Structural value of the roof structure	**67**
Ideal value of the roof structure	**65**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	10-20%	2
		Decay of the compound rafter	10-20%	2
		Decay of the rafter	10-20%	2
		Decay of the purlins	20-30%	3
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	10-20%	2
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**29**
Climate change vulnerability of the roof structure	**57**
Value index of the roof structure	**61**
Vulnerability index of the roof structure	**55**

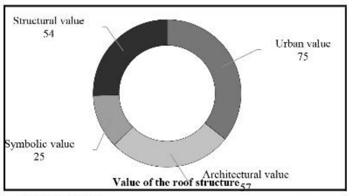
RESULTS

The roof structure has moderate value
The roof structure has a predominant urbanistic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is moderate
The roof structure has a moderate vulnerability

Fig. C. 12 Assessment sheet for the Karl Kunz building roof structure

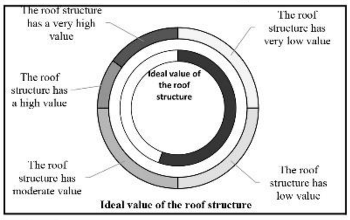
Roof structure: Residential building 5
Address: Alexandru Mocioni square, no. 7

Urban value	C1. Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	30º-45º	3
Architectural value	C4. Historic analysis of the building	Dating	1900-1912	3
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Habitation	1
		Changes of function	No	2
		Contemporary function	Habitation	1
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Gable roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		No ratio	0
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	No inscriptions	0
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Mechanical fasteners (nails, screws)	1
		Used traditional joints	Mortise and tenon	3

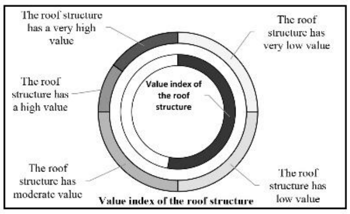


RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**75**
Architectural value of the roof structure	**57**
Symbolic value of the roof structure	**25**
Structural value of the roof structure	**54**
Ideal value of the roof structure	**55**

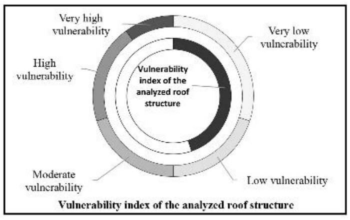


Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	<10%	1
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1



CURRENT STATE OF THE ROOF STRUCTURE

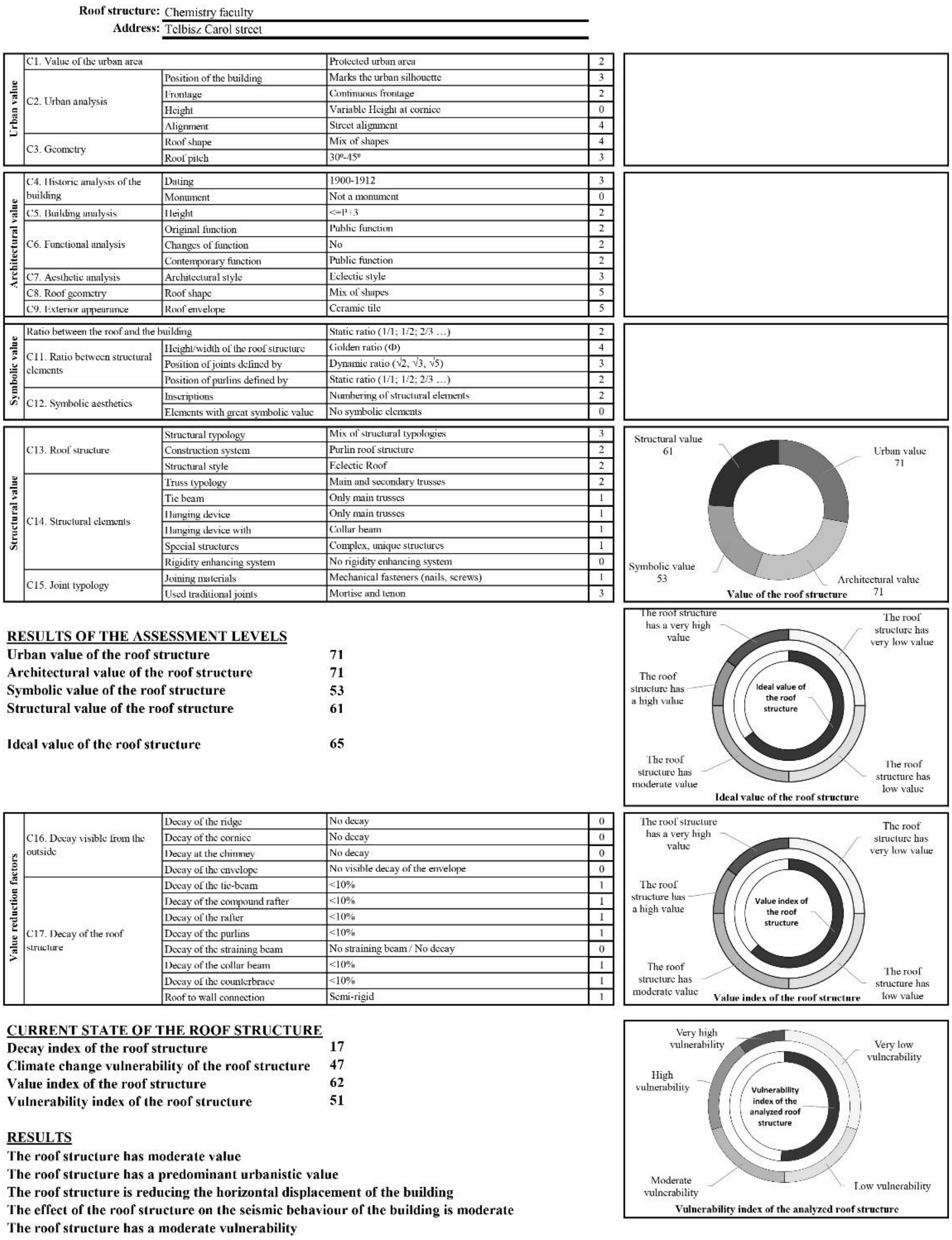
Decay index of the roof structure	**17**
Climate change vulnerability of the roof structure	**47**
Value index of the roof structure	**53**
Vulnerability index of the roof structure	**45**



RESULTS

The roof structure has moderate value
The roof structure has a predominant urbanistic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is moderate
The roof structure has a low vulnerability

Fig. C. 13 Assessment sheet for the residential building no.5 roof structure



Roof structure: Chemistry faculty
Address: Telbisz Carol street

Urban value	C1. Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Marks the urban silhouette	3
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Mix of shapes	4
		Roof pitch	30º-45º	3

Architectural value	C4. Historic analysis of the building	Dating	1900-1912	3
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Mix of shapes	5
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

Symbolic value	Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
	C11. Ratio between structural elements	Height/width of the roof structure	Golden ratio (Φ)	4
		Position of joints defined by	Dynamic ratio (√2, √3, √5)	3
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0

Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Mechanical fasteners (nails, screws)	1
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**71**
Architectural value of the roof structure	**71**
Symbolic value of the roof structure	**53**
Structural value of the roof structure	**61**
Ideal value of the roof structure	**65**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	No decay	0
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	<10%	1
		Decay of the purlins	<10%	1
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**17**
Climate change vulnerability of the roof structure	**47**
Value index of the roof structure	**62**
Vulnerability index of the roof structure	**51**

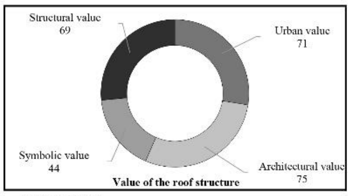
RESULTS

The roof structure has moderate value
The roof structure has a predominant urbanistic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is moderate
The roof structure has a moderate vulnerability

Fig. C. 14 Assessment sheet for the Chemistry faculty roof structure

Roof structure: "Victoria" Hotel
Address: Lucian Blaga, no. 3

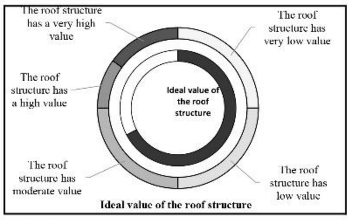
Urban value	Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Mix of shapes	4
		Roof pitch	45º-65º	4
Architectural value	C4. Historic analysis of the building	Dating	1900-1912	3
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Secession	5
	C8. Roof geometry	Roof shape	Mix of shapes	5
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		No ratio	0
	C11. Ratio between structural elements	Height/width of the roof structure	Dynamic ratio (√2, √3, √5)	3
		Position of joints defined by	Dynamic ratio (√2, √3, √5)	3
		Position of purlins defined by	Dynamic ratio (√2, √3, √5)	3
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	Towers	2
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3



Value of the roof structure

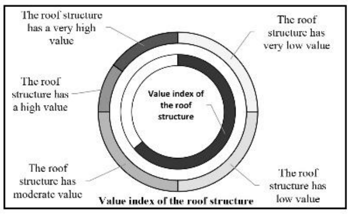
RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**71**
Architectural value of the roof structure	**75**
Symbolic value of the roof structure	**44**
Structural value of the roof structure	**69**
Ideal value of the roof structure	**67**



Ideal value of the roof structure

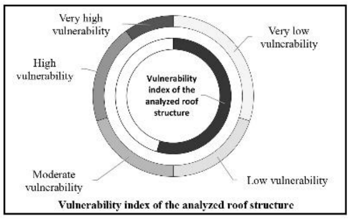
Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	10-20%	2
		Decay of the straining beam	No straining beam / No decay	0
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1



Value index of the roof structure

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**23**
Climate change vulnerability of the roof structure	**51**
Value index of the roof structure	**64**
Vulnerability index of the roof structure	**55**



Vulnerability index of the analyzed roof structure

RESULTS

The roof structure has moderate value
The roof structure has a predominant architectural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is moderate
The roof structure has a moderate vulnerability

Fig. C. 15 Assessment sheet for the Victoria hotel roof structure

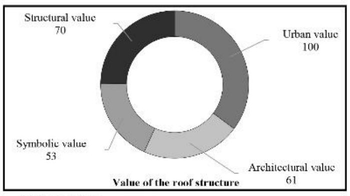
Roof structure: Lloyd Palace
Address: Victory square no. 2

Urban value	Value of the urban area		Significant role in defining the urban space	4
	C2. Urban analysis	Position of the building	Marks an essential urban point	4
		Frontage	Continuous frontage	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Mix of shapes	4
		Roof pitch	45º-65º	4

Architectural value	C4. Historic analysis of the building	Dating	1912-1936	2
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Mix of shapes	5
	C9. Exterior appearance	Roof envelope	Metal roofing	1

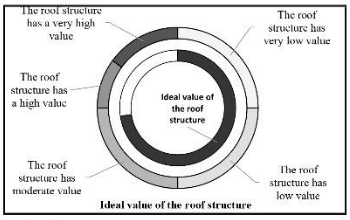
Symbolic value	Ratio between the roof and the building		Incoherent mix of ratios	1
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	Symbolic roof decorations	4

Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar ties	2
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	Central longitudinal system	1
	C15. Joint typology	Joining materials	Mechanical fasteners (nails, screws)	1
		Used traditional joints	Mortise and tenon	3

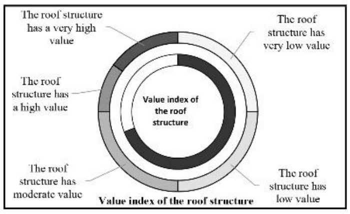


RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**100**
Architectural value of the roof structure	**61**
Symbolic value of the roof structure	**53**
Structural value of the roof structure	**70**
Ideal value of the roof structure	**73**



Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	<10%	1
		Decay of the straining beam	<10%	1
		Decay of the collar beam	<10%	1
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1



CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**23**
Climate change vulnerability of the roof structure	**51**
Value index of the roof structure	**69**
Vulnerability index of the roof structure	**58**

RESULTS

The roof structure has moderate value
The roof structure has a predominant urbanistic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is low
The roof structure has a moderate vulnerability

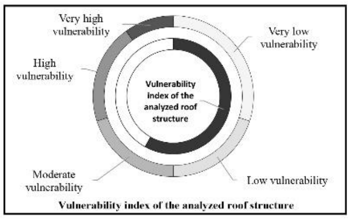


Fig. C. 16 Assessment sheet for the Lloyd palace roof structure

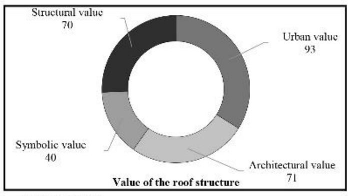
Roof structure: Loffler palace
Address: Victory square

Urban value	Value of the urban area		Significant role in defining the urban space	4
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Continuous frontage	2
		Height	Constant Height at cornice	2
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Mix of shapes	4
		Roof pitch	45º-65º	4

Architectural value	C4. Historic analysis of the building	Dating	1912-1936	2
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Mixed function	2
		Changes of function	No	2
		Contemporary function	Mixed function	2
	C7. Aesthetic analysis	Architectural style	Seccesion	4
	C8. Roof geometry	Roof shape	Mix of shapes	5
	C9. Exterior appearance	Roof envelope	Ceramic tile	5

Symbolic value	Ratio between the roof and the building		Static ratio (1/1; 1/2; 2/3 …)	2
	C11. Ratio between structural elements	Height/width of the roof structure	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of joints defined by	Static ratio (1/1; 1/2; 2/3 …)	2
		Position of purlins defined by	Static ratio (1/1; 1/2; 2/3 …)	2
	C12. Symbolic aesthetics	Inscriptions	Numbering of structural elements	2
		Elements with great symbolic value	No symbolic elements	0

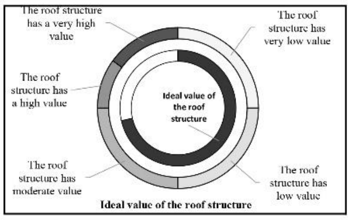
Structural value	C13. Roof structure	Structural typology	Mix of structural typologies	3
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar ties	2
		Special structures	Complex, unique structures	1
		Rigidity enhancing system	Central longitudinal system	1
	C15. Joint typology	Joining materials	Mechanical fasteners (nails, screws)	1
		Used traditional joints	Mortise and tenon	3



Value of the roof structure

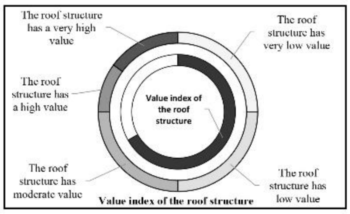
RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**93**
Architectural value of the roof structure	**71**
Symbolic value of the roof structure	**40**
Structural value of the roof structure	**70**
Ideal value of the roof structure	**72**



Ideal value of the roof structure

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	Parts of the envelope are damaged	2
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	10-20%	2
		Decay of the straining beam	<10%	1
		Decay of the collar beam	10-20%	2
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1



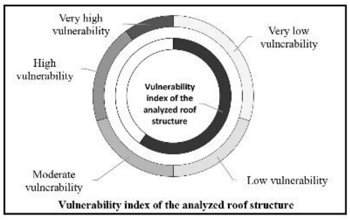
Value index of the roof structure

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**31**
Climate change vulnerability of the roof structure	**56**
Value index of the roof structure	**67**
Vulnerability index of the roof structure	**60**

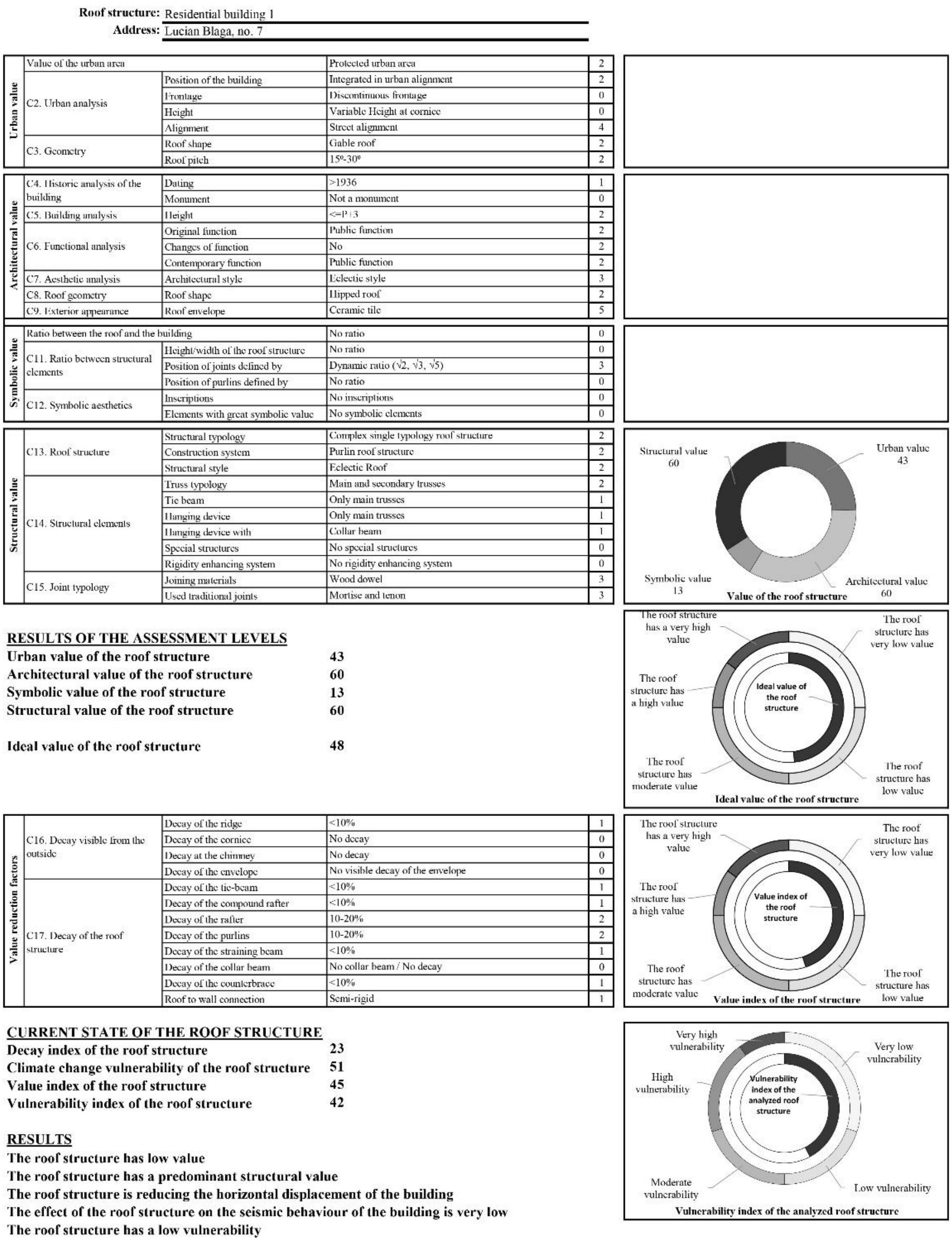
RESULTS

The roof structure has moderate value
The roof structure has a predominant urbanistic value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is low
The roof structure has a moderate vulnerability



Vulnerability index of the analyzed roof structure

Fig. C. 17 Assessment sheet for the Löffler palace roof structure



Roof structure: Residential building 1
Address: Lucian Blaga, no. 7

Urban value	Value of the urban area		Protected urban area	2
	C2. Urban analysis	Position of the building	Integrated in urban alignment	2
		Frontage	Discontinuous frontage	0
		Height	Variable Height at cornice	0
		Alignment	Street alignment	4
	C3. Geometry	Roof shape	Gable roof	2
		Roof pitch	15º-30º	2
Architectural value	C4. Historic analysis of the building	Dating	>1936	1
		Monument	Not a monument	0
	C5. Building analysis	Height	<=P+3	2
	C6. Functional analysis	Original function	Public function	2
		Changes of function	No	2
		Contemporary function	Public function	2
	C7. Aesthetic analysis	Architectural style	Eclectic style	3
	C8. Roof geometry	Roof shape	Hipped roof	2
	C9. Exterior appearance	Roof envelope	Ceramic tile	5
Symbolic value	Ratio between the roof and the building		No ratio	0
	C11. Ratio between structural elements	Height/width of the roof structure	No ratio	0
		Position of joints defined by	Dynamic ratio (√2, √3, √5)	3
		Position of purlins defined by	No ratio	0
	C12. Symbolic aesthetics	Inscriptions	No inscriptions	0
		Elements with great symbolic value	No symbolic elements	0
Structural value	C13. Roof structure	Structural typology	Complex single typology roof structure	2
		Construction system	Purlin roof structure	2
		Structural style	Eclectic Roof	2
	C14. Structural elements	Truss typology	Main and secondary trusses	2
		Tie beam	Only main trusses	1
		Hanging device	Only main trusses	1
		Hanging device with	Collar beam	1
		Special structures	No special structures	0
		Rigidity enhancing system	No rigidity enhancing system	0
	C15. Joint typology	Joining materials	Wood dowel	3
		Used traditional joints	Mortise and tenon	3

RESULTS OF THE ASSESSMENT LEVELS

Urban value of the roof structure	**43**
Architectural value of the roof structure	**60**
Symbolic value of the roof structure	**13**
Structural value of the roof structure	**60**
Ideal value of the roof structure	**48**

Value reduction factors	C16. Decay visible from the outside	Decay of the ridge	<10%	1
		Decay of the cornice	No decay	0
		Decay at the chimney	No decay	0
		Decay of the envelope	No visible decay of the envelope	0
	C17. Decay of the roof structure	Decay of the tie-beam	<10%	1
		Decay of the compound rafter	<10%	1
		Decay of the rafter	10-20%	2
		Decay of the purlins	10-20%	2
		Decay of the straining beam	<10%	1
		Decay of the collar beam	No collar beam / No decay	0
		Decay of the counterbrace	<10%	1
		Roof to wall connection	Semi-rigid	1

CURRENT STATE OF THE ROOF STRUCTURE

Decay index of the roof structure	**23**
Climate change vulnerability of the roof structure	**51**
Value index of the roof structure	**45**
Vulnerability index of the roof structure	**42**

RESULTS

The roof structure has low value
The roof structure has a predominant structural value
The roof structure is reducing the horizontal displacement of the building
The effect of the roof structure on the seismic behaviour of the building is very low
The roof structure has a low vulnerability

Fig. C. 18 Assessment sheet for the residential building no.1 roof structure

BIBLIOGRAPHY

[1] ICOMOS, "Principles for the preservation of historic timber structures", Mexico, 1999.

[2] ICOMOS, "Principles for the Analysis, Conservation and Structural Restoration of Architectural Heritage", Victoria Falls, Zimbabwe, 2003.

[3] ICOMOS, "International Charter for the Conservation and Restoration of Monuments and Sites (The Venice Charter 1964)", Venice, Italy, 1964.

[4] L. Uzielli and J. Gril, "Wood science and conservation: activities and achievements of COST Action IE0601", *Journal of Cultural Heritage*, vol. 13, pp. S1–S5, 2012.

[5] P. Dietsch, J. Köhler, and H. Brüninghoff, *Assessment of timber structures : COST Action E55 "Modelling of the Performance of Timber Structures."* Shaker Verlag, 2010.

[6] D. D'Ayala, J. M. Branco, M. Riggio, A. Harte, J. Kurz, and T. Descamps, "Assessment, reinforcement and monitoring of timber structures: FPS Cost Action FP1101", in *Proceedings of the 9th International Conference on Structural Analysis of Historical Constructions, SAHC 2014*, 2014.

[7] H. Cruz, D. Yeomans, E. Tsakanika, N. Macchioni, A. Jorissen, M. Touza, M. Mannucci, and P. B. Lourenço, "Guidelines for On-Site Assessment of Historic Timber Structures", *International Journal of Architectural Heritage*, vol. 9, no. 3, pp. 277–289, Apr. 2015.

[8] M. Riggio, D. D'Ayala, M. A. Parisi, and C. Tardini, "Assessment of heritage timber structures: Review of standards, guidelines and procedures", *Journal of Cultural Heritage*, vol. 31, pp. 220–235, May 2018.

[9] H. S. Sousa, J. M. Branco, and P. B. Lourenço, "A Holistic Methodology for Probabilistic Safety Assessment of Timber Elements Combining Onsite and Laboratory Data", *International Journal of Architectural Heritage*, vol. 10, no. 5, pp. 526–538, Jul. 2016.

[10] G. Tampone, "Ancient timber structural systems: an attempt to define the main components of the paradigm", in *Proceedings of the 20th International Symposium of the ICOMOS International Wood Committee, IIWC 2016*, 2016, pp. 9–16.

[11] C. Bertolini-Cestari, A. Spanò, S. Invernizzi, and T. Marzi, "Retrofitting Assessment and Numerical Models for the Historical Timber Roof Structures of the Towers of Valentino Castle in Turin", in *Proceedings of the 2nd International Conference on Historic Earthquake-Resistant Timber Frames in the Mediterranean Region, HEaRT 2015*, Springer, Cham, 2016, pp. 321–333.

[12] L. G. Rodrigues, C. A. Aranha, and H. S. Sousa, "Robustness assessment of an ancient timber roof structure", in *Proceedings of the 2nd International Conference on Historic Earthquake-Resistant Timber Frames in the Mediterranean Region, HEaRT 2015*, vol. 1, H. Cruz, J. S. Machado, A. C. Costa, P. X. Candeias, N. Ruggieri, and J. M. Catarino, Eds. Springer, 2016, pp. 447–457.

[13] L. Debailleux, "Indexing System for Recognizing Traditional Timber-Framed Structures", *International Journal of Architectural Heritage*, vol. 9, no. 5, pp. 529–541, Jul. 2015.

[14] A. Meisel, *Historische Dachwerke Beurteilung, realitätsnahe statische Analyse und Instandsetzung*, Monographi. Graz: Verlag der Technischen Universität Graz, 2015.
[15] J. M. Branco, "Portuguese traditional timber structures: survey, analysis and strengthening", in *Proceedings of the 1st International Conference on Protection of Historical Buildings, PROHITECH 09*, 2009, pp. 261–266.
[16] A. Serafini and C. Gonzalez-Longo, "Seventeenth and eighteenth century timber roof structures in Scotland: design, pathologies and conservation", in *Proceedings of the 10th International Conference on Structural Analysis of Historical Constructions, SAHC 2016*, 2016.
[17] M. Piazza, "Behavior and rehabilitation of traditional timber structures", in *Proceedings of the 1st International Conference on Protection of Historical Buildings, PROHITECH 09*, 2009, pp. 75–83.
[18] Ente Nationale Italiano di Unificazione (UNI), *UNI 11119 "Cultural heritage. Wooden artefacts. Load-bearing structures. On-site inspections for the diagnosis of timber members."* Milano, Italy: Ente Nationale Italiano di Unificazione (UNI), 2004.
[19] Ente Nationale Italiano di Unificazione (UNI), *UNI 111138 "Cultural heritage. Wooden artefacts. Load-bearing structures. Criteria for the preliminary evaluation, design and execution of works."* Milano, Italy: Ente Nationale Italiano di Unificazione (UNI), 2004.
[20] International Organization for Standardization (ISO), *ISO 13822:2010(en) Bases for design of structures — Assessment of existing structures*. Geneva, Switzerland: International Organization for Standardization (ISO), 2010.
[21] American Society of Civil Engineers (ASCE), *ASCE 41-13, Seismic Evaluation and Upgrade of Existing Buildings*. Reston, Virginia, USA: American Society of Civil Engineers (ASCE), 2013.
[22] Schweizer Ingenieur sund Architektenverein (SIA), *SIA 269:2011 – Existing structures - Bases for examination and intervention*. Zürich, Switzerland: Schweizer Ingenieur sund Architektenverein (SIA), 2011.
[23] F. Ostendorf, *The history of roof structures (in German)*. Kessinger Publishing, LLC, 1908.
[24] C. Lachner, *Geschichte der Holzbaukunst in Deutschland*. Hannover, 1885.
[25] C. Schaefer, *Die Holzarchitektur Deutschlands vom XIV. bis XVIII. Jahrhundert.* C.R. Vincentz, 1981.
[26] K. Lißner and W. Rug, "Historische Konstruktionen und Verbindungen", in *Holzbausanierung beim Bauen im Bestand*, Springer, 2018, pp. 23–116.
[27] K. Alykow and M. Napiorkowska-Alykow, "The expansion of mansard roofs across europe in the 18th century", in *Proceedings of the 4th International Conference on Structural Health Assessment of Timber Structures, SHATIS'17*, 2017, pp. 25–34.
[28] P. Caston, "Historic Roof Trusses between 1500 and 1700 in German-speaking Central Europe: Documentation, Analysis and Development", *Proceedings of the Second International Congress on Construction History*, vol. 1, pp. 579–598, 2006.
[29] B. Szabó, *Dictionar illustrat de structuri portante istorice*. Cluj, Romania: Kriterion, 2005.
[30] B. Szabó and I. Kirizsán, "Transylvanian Historic Roof Structures Database", in *Proceedings of the 1th International Conference on Structural Health Assessment of Timber Structures, SHATIS'11*, 2011.

[31] B. Szabó and I. Kirizsán, "European historic roof structures: concepts of composition and intervention", in *Proceedings of the 8th International Conference on Structural Analysis of Historical Constructions, SAHC 2012*, 2012, pp. 105–127.

[32] L. Magnus, "Historic timber roof structures construction technology and structural behavior", Catholic University of Leuven, 2008.

[33] A. Serafini and C. Gonzalez-Longo, "The design and construction techniques of eighteenth century timber roofs in Scotland : Glasgow trades hall and tweeddale house in Edinburgh", in *Proceedings of the 5th International Congress on Construction History*, 2015.

[34] B. Köck, "Baroque roof structures: construction and structural behaviour (in German)", München, Univ. der Bundeswehr, 2011.

[35] S. M. Holzer, B. Köck, and S. Stelner, *Meisterwerke barocker Bautechnik Kuppeln, Gewolbe und Kirchendachwerke in Si.idbayern*. 2008.

[36] A. Sabel, "Hölzerne Dachtragwerke im Königreich Bayern", Bundeswehr University Munich, 2016.

[37] S. King and B. Lohrum, *Historic roof structures: an exhibition of models of Medieval roof structures from the cities of Constance, Ravensburg, Rottweil and Villingen (in German)*. Stuttgart, Germany: Universitat Stuttgart, Institut fur Architekturgeschichte, 2000.

[38] H. S. Sousa, J. S. Machado, J. M. Branco, and P. B. Lourenço, "Onsite assessment of structural timber members by means of hierarchical models and probabilistic methods", *Construction and Building Materials*, vol. 101, pp. 1188–1196, Dec. 2015.

[39] J. Gocál, P. Krušinský, E. Capková, and M. Kekeliak, *Geometric and static analysis of the historical truss in village Belá Dulice*, vol. 969. 2014, pp. 199–207.

[40] P. Krušinský, E. Capková, J. Gocál, and M. Holešová, "Geometric and static analysis of the historical trusses in roman catholic church of the Holy Kozma and Damian in the Abramová village", *Civil and Environmental Engineering*, vol. 11, no. 2, pp. 136–141, 2015.

[41] P. Krušinský, J. Gocál, and E. Capková, "Static analysis of historical trusses", *Wiadomości Konserwatorskie*, vol. 47, pp. 120–127, 2016.

[42] P. Dietsch and H. Kreuzinger, "Guideline on the assessment of timber structures: Summary", *Engineering Structures*, vol. 33, no. 11, pp. 2983–2986, Nov. 2011.

[43] T. Tannert, P. Dietsch, C. Bertolini-Cestari, and B. Kasal, "COST Action FP1101 - Assessment, Reinforcement and Monitoring of Timber Structures", in *World Conference on Timber Engineering (WCTE)*, 2012, pp. 515–519.

[44] B. Kasal, "Assessment, reinforcement and monitoring of timber structures-COST FP1101", *Advanced Materials Research*, vol. 778, pp. 1037–1040, 2013.

[45] M. Riggio, M. A. Parisi, C. Tardini, E. Tsakanika, D. D'Ayala, N. Ruggieri, G. Tampone, and F. Augelli, "Existing timber structures: Proposal for an assessment template", in *Proceedings of the 3th International Conference on Structural Health Assessment of Timber Structures, SHATIS'15*, 2015, no. September, pp. 100–107.

[46] A. Serafini, M. Riggio, and C. Gonzalez-Longo, "A database model for the analysis and assessment of historic timber roof structures", *International Wood Products Journal*, vol. 8, no. sup1, pp. 3–8, Aug. 2017.

[47] M. Abels, "Assessment Matrix for Timber Structures Basis for Standardized Building Checks", 2011.

[48] M. Abels, "Begutachtung einer Kirchendachkonstruktion mit der Beurteilungsmatrix für Holzkonstruktionen", pp. 23–31, 2012.

[49] G. Chao, W. Juan, and Y. Qing Shan, "Rapid in-site survey and assessment method for structural members in traditional chinese traditional timber structure", in *Proceedings of the 4th International Conference on Structural Health Assessment of Timber Structures, SHATIS'17*, 2017, pp. 119–130.

[50] M. A. Parisi, C. Chesi, C. Tardini, and M. Piazza, "Seismic vulnerability and preservation of timber roof structures", in *Proceedings of the 6th International Conference on Structural Analysis of Historical Constructions, SAHC 2008*, 2008, pp. 1253–1260.

[51] M. A. Parisi, C. Chesi, C. Tardini, and M. Piazza, "Seismic vulnerability assessment for timber roof structures", in *The 14th World Conference on Earthquake Engineering*, 2008, pp. 12–17.

[52] M. A. Parisi, C. Chesi, C. Tardini, and M. Piazza, "Traditional timber roof structures: seismic vulnerability assessment and preservation", in *Proceedings of the 1st International Conference on Protection of Historical Buildings, PROHITECH 09*, 2009, pp. 391–396.

[53] M. A. Parisi, C. Chesi, and C. Tardini, "The role of timber roof structures in the seismic response of traditional buildings", in *Proceedings of the 15th World Conference on Earthquake Engineering, WCEE 2012*, 2012.

[54] M. A. Parisi and C. Chesi, "Seismic vulnerability of traditional buildings: the effect of roof-masonry walls interaction", in *Tenth U.S. National Conference on Earthquake Engineering Frontiers of Earthquake Engineering*, 2014.

[55] M. A. Parisi and M. Piazza, "Seismic behavior and retrofitting of joints in traditional timber roof structures", *Soil Dynamics and Earthquake Engineering*, vol. 22, no. 9–12, pp. 1183–1191, Oct. 2002.

[56] M. A. Parisi, C. Chesi, and C. Tardini, "Inferring Seismic Behavior From Morphology in Timber Roofs", *International Journal of Architectural Heritage*, vol. 6, no. 1, pp. 100–116, Jan. 2012.

[57] M. A. Parisi, C. Tardini, and E. Maritato, "Seismic behaviour and vulnerability of church roof structures", in *Proceedings of the 10th International Conference on Structural Analysis of Historical Constructions, SAHC 2016*, 2016, pp. 1582–1589.

[58] D. Benedetti and V. Petrini, "On the seismic vulnerability of masonry buildings: proposal of an evaluation method (in Italian)", *L'industria delle costruzioni*, vol. 18, pp. 66–74, 1984.

[59] R. Vicente, D. D'Ayala, T. M. Ferreira, H. Varum, A. Costa, M. da Silva, and S. Lagomarsino, "Seismic vulnerability and risk assessment of historic masonry buildings", in *Structural Rehabilitation of Old Buildings*, 2014, pp. 307–348.

[60] A. Formisano, G. Florio, R. Landolfo, and F. M. Mazzolani, "Numerical calibration of an easy method for seismic behaviour assessment on large scale of masonry building aggregates", *Advances in Engineering Software*, vol. 80, no. C, pp. 116–138, Feb. 2015.

[61] A. Roca, X. Goula, T. Susagna, J. Chávez, M. González, and E. Reinoso, "A simplified method for vulnerability assessment of dwelling buildings and estimation of damage scenarios in catalonia, Spain", *Bulletin of Earthquake Engineering*, vol. 4, no. 2, pp. 141–158, May 2006.

[62] P. Guéguen, C. Michel, and L. Lecorre, "A simplified approach for vulnerability assessment in moderate-to-low seismic hazard regions: Application to Grenoble (France)", *Bulletin of Earthquake Engineering*, vol. 5, no. 3, pp. 467–490, Aug. 2007.
[63] S. Lagomarsino, "On the vulnerability assessment of monumental buildings", *Bulletin of Earthquake Engineering*, vol. 4, no. 4, pp. 445–463, Nov. 2006.
[64] A. Formisano, F. M. Mazzolani, R. Landolfo, and G. Florio, "A quick methodology for seismic vulnerability assessment of historical masonry aggregates", in *COST C26 Final Conference "Urban Habitat Constructions under Catastrophic Events,"* 2010.
[65] M. Mosoarca, I. Onescu, E. Onescu, B. Azap, N. Chieffo, and M. Szitar-Sirbu, "Seismic vulnerability assessment for the historical areas of the Timisoara city, Romania", *Engineering Failure Analysis*, vol. 101, pp. 86–112, Jul. 2019.
[66] I. Apostol, M. Mosoarca, N. Chieffo, and E. Onescu, "Seismic vulnerability scenarios for Timisoara, Romania", in *Proceedings of the 11th International Conference on Structural Analysis of Historical Constructions, SAHC 2018*, 2018, Vol 18., pp. 1191–1200.
[67] M. A. Parisi, C. Chesi, C. Tardini, and D. Vecchi, "Seismic vulnerability of timber roof structures: an assessment procedure", in *Proceedings of the 16th World Conference on Earthquake Engineering, WCEE 2017*, 2017.
[68] B. Nicolescu, *Manifesto of transdisciplinarity*. New York, United States of America: State University of New York Press, Albany, 2002.
[69] B. Nicolescu, "Methodology of transdisciplinarity – levels of reality, logic of the included middle and complexity", *Transdisciplinary Journal of Engineering & Science*, vol. 1, no. 1, pp. 19–38, 2010.
[70] B. Nicolescu, *Transdisciplinarity and Sustainability*. Lubbock, Texas, United States of America: TheATLAS Publishing, 2012.
[71] P. Cilliers and B. Nicolescu, "Complexity and transdisciplinarity – Discontinuity, levels of Reality and the Hidden Third", *Futures*, vol. 44, no. 8, pp. 711–718, 2012.
[72] M. Ghyka, *The Geometry of art and life*. New York, United States of America: Dover Publications Inc., 1977.
[73] R. Lawlor, *Sacred geometry: Philosophy and practice*. Crossroad, 1982.
[74] M. Ghyka, *The Golden ratio. Pythagorean rites and rhythms in the development of Western civilization*. Bucarest: Nemira Pubishing House, 2016.
[75] J. B. Henderson, *The development and decline of Chinese cosmology*. Columbia University Press, New York, 1984.
[76] J. Feng, *Chinese Architecture and Metaphor*. University of Hawai'i Press, 2012.
[77] A. I.-K. Li, "A shape grammar for teaching the architectural style of the Yingzao fashi", 2001.
[78] R. Koolhaas, *Fundamentals: 14th international architecture exhibition*. Marsilio, 2014.
[79] I. Andreescu and A. Keller, "Complex features in assessing historic roof structures", in *Proceedings of the 3rd International Conference on Protection Of Historical Constructions, PROHITECH'17*, 2017.
[80] S. Lipovetsky and F. A. Lootsma, "Generalized golden sections, repeated bisections and aesthetic pleasure", *European Journal of Operational Research*, vol. 121, no. 1, pp. 213–216, Feb. 2000.

[81] L. M. Dabbour, "Geometric proportions: The underlying structure of design process for Islamic geometric patterns", *Frontiers of Architectural Research*, vol. 1, no. 4, pp. 380–391, Dec. 2012.
[82] M. Akhtaruzzaman and A. A. Shafie, "Geometrical Substantiation of Phi, the Golden Ratio and the Baroque of Nature, Architecture, Design and Engineering", *International Journal of Arts*, vol. 1, no. 1, pp. 1–22, Aug. 2012.
[83] J. Loach, "Le Corbusier and the Creative Use of Mathematics", *The British Journal for the History of Science*, vol. 31, no. 2, pp. 185–215, 1998.
[84] P. Wood, "Le Corbusier's Secret Geometry: Speculations on Regulating Lines Hidden in Ronchamp", pp. 1–17, 2015.
[85] J. J. Schübler, *Neue Anweisung zur unentbehrlichen Zimmermannskunst*. Nürnberg, Germany: Weigel, 1782.
[86] P. Krušinský, J. Gocál, and E. Capková, "Geometric and static analysis of the historical truss above presbytery of the Saint Peter of Alcantara monastery church in Okolicne", *Procedia Engineering*, vol. 111, no. TFoCE, pp. 485–490, 2015.
[87] P. Krušinský, E. Capková, and J. Gocál, "Comparison of Two Medieval Trusses from the Viewpoint of Geometric and Static Analysis", *Advanced Materials Research*, vol. 1122, pp. 243–248, 2015.
[88] J. Gocál, P. Krušinský, E. Capková, and M. Kekeliak, "Analysis of the historical truss in village Belá Dulice", *Acta Scientiarum Polonorum Formatio Circumiectus*, vol. 14, no. 1, pp. 57–73, 2015.
[89] I. Andreescu, A. Keller, and M. Mosoarca, "Complex Assessment of Roof Structures", in *Procedia Engineering*, 2016, vol. 161, pp. 1204–1210.
[90] I. Andreescu and A. Keller, "Architecture as "Gesamtkunstwerk" – the role of the roof in defining architecture in the 19th and 20th century in Timisoara", in *Proceedings of the 3rd World Multidisciplinary Civil Engineering - Architecture - Urban Planning Symposium, WMCAUS*, 2018.
[91] A. Keller and M. Mosoarca, "A complex assessment of historic roof structures", in *Proceedings of the 4th International Conference on Structural Health Assessment of Timber Structures, SHATIS'17*, 2017, pp. 157–168.
[92] M. Cohen, "Introduction: Two Kinds of Proportion", *Architectural Histories*, vol. 2(1), no. 21, pp. 1–25, Jun. 2014.
[93] W. Lethaby, *Architecture Mysticism and Myth*. London, Percival & Co., 1892.
[94] H. Saemann, *Dachwerke über den welfischen Residenzbauten der Barockzeit im Kontext des höfischen Bauwesens: Untersuchungen in den ehemaligen Residenzstädten Hannover, Celle, Osnabrück, Wolfenbüttel und Braunschweig*. Michael Imhof Verlag, 2014.
[95] IPCC, "Climate Change 2014: Synthesis Report. Contribution of Working Groups I, II and III to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change", Geneva, Switzerland, 2014.
[96] A. Busuioc, M. Caian, R. Bojariu, C. Boroneant, S. Cheval, M. Baciu, and A. Dumitrescu, *Climate change scenarios in Romania for 2001-2030 [in Romanian]*. Bukarest: Administratia Nationala de Meteorologie, 2009.
[97] L. Guan, "Preparation of future weather data to study the impact of climate change on buildings", *Building and environment*, vol. 44, no. 4, pp. 793–800, 2009.
[98] CCWorldWeatherGen, "Climate change world weather file generator. Version 1.8. Sustainable Energy research group", *University of Southampton*, 2014. [Online].

Available: http://www.energy.soton.ac.uk/ccworldweathergen/. [Accessed: 27-Nov-2015].

[99] S. Glass and S. Zelinka, "Wood Handbook, Chapter 04: Moisture Relations and Physical Properties of Wood", 2010.

[100] C. C. Gerhards, "Effect of moisture content and temperature on the mechanical properties of wood: an analysis of immediate effects", *Wood and Fiber Science*, vol. 14, no. 1, pp. 4–36, 2007.

[101] M. Fragiacomo, S. Fortino, D. Tononi, I. Usardi, and T. Toratti, "Moisture-induced stresses perpendicular to grain in cross-sections of timber members exposed to different climates", *Engineering Structures*, vol. 33, pp. 3071–3078, 2011.

[102] W. T. Simpson, "Predicting equilibrium moisture content of wood by mathematical models", *Wood and fiber science*, vol. 5, no. 1, pp. 41–49, 1973.

[103] A. J. Hailwood and S. Horrobin, "Absorption of water by polymers: analysis in terms of a simple model", *Transactions of the Faraday Society*, vol. 42, pp. B084--B092, 1946.

[104] M. Borrega and P. P. Kärenlampi, "Mechanical behavior of heat-treated spruce (Picea abies) wood at constant moisture content and ambient humidity", *Holz als Roh- und Werkstoff*, vol. 66, no. 1, pp. 63–69, Feb. 2008.

[105] J. Sun, N. Lam, L. Zhang, D. Ruan, and E. Gad, "Contact forces generated by hailstone impact", *International Journal of Impact Engineering*, vol. 84, pp. 145–158, 2015.

[106] M. Mosoarca, A. Keller, C. Petrus, and A. Racolta, "Failure analysis of historical buildings due to climate change", *Engineering Failure Analysis*, vol. 82, pp. 666–680, Dec. 2017.

[107] H. J. Punge and M. Kunz, "Hail observations and hailstorm characteristics in Europe: A review", *Atmospheric Research*, vol. 176, pp. 159–184, 2016.

[108] MDRAP, *Norm concerning the design, construction and use of roof envelopes, NP 069 -2014 [in Romanian]*. 2014.

[109] VKF, "Hail - the underestimated danger". [Online]. Available: http://vkf.ch/VKF/Services/Hagelregister.aspx. [Accessed: 26-Nov-2017].

[110] Baunetzwissen, "Hail impact". [Online]. Available: https://www.baunetzwissen.de/geneigtes-dach/fachwissen/feuchte--witterungsschutz/hagelschlag-4512727. [Accessed: 26-Nov-2017].

[111] D. Mihalca and E. Sanciu, "The particularities of wind conditions in Banat as a reflection of relief influence", *Analele Universitatii de Vest din Timisoara*, vol. 6, pp. 33–39, 1996.

[112] M. Mosoarca, A. Keller, and C. Bocan, "Failure analysis of church towers and roof structures due to high wind velocities", *Engineering Failure Analysis*, vol. 100, pp. 76–87, 2019.

[113] R. D. Brites, L. Neves, J. Saporiti Machado, P. B. Lourenço, and H. S. Sousa, "Reliability analysis of a timber truss system subjected to decay", *Engineering Structures*, vol. 46, pp. 184–192, 2013.

[114] B. Faggiano, M. R. Grippa, A. Marzo, and F. M. Mazzolani, "Mechanical identification by NDT of old chestnut structural timber", *Proceedings of the 1st International Conference on Protection of Historical Buildings, PROHITECH 09*, pp. 295–300, 2009.

[115] H. S. Sousa, J. M. Branco, and P. B. Lourenço, "Characterization of cross-sections from old chestnut beams weakened by decay", *International Journal of Architectural Heritage*, vol. 8, no. 3, pp. 436–451, 2014.
[116] H. S. Sousa, J. D. Sørensen, P. H. Kirkegaard, J. M. Branco, and P. B. Lourenço, "On the use of NDT data for reliability-based assessment of existing timber structures", *Engineering Structures*, vol. 56, pp. 298–311, 2013.
[117] G. De Matteis, F. Campitiello, B. Calderoni, and C. Giubileo, "Identification of the mechanical properties of ancient soft timber by means of non-destructive tests", in *Proceedings of the 1st International Conference on Protection of Historical Buildings, PROHITECH 09*, 2009, pp. 287–293.
[118] M. R. Migliore and F. Ramundo, "Combined non-destructive tests method for the diagnosis of existing wooden structures", *Proceedings of the 1st International Conference on Protection of Historical Buildings, PROHITECH 09*, pp. 869–874, 2009.
[119] M. Riggio, R. W. Anthony, F. Augelli, B. Kasal, T. Lechner, W. Muller, and T. Tannert, "In situ assessment of structural timber using non-destructive techniques", *Materials and Structures*, vol. 47, no. 5, pp. 749–766, 2014.
[120] C. Calderoni, G. De Matteis, C. Giubileo, and F. M. Mazzolani, "Experimental correlations between destructive and non-destructive tests on ancient timber elements", *Engineering Structures*, vol. 32, no. 2, pp. 442–448, 2010.
[121] J. Sandak, A. Sandak, and M. Riggio, "Multivariate analysis of multi-sensor data for assessment of timber structures: Principles and applications", *Construction and Building Materials*, vol. 101, pp. 1172–1180, Dec. 2015.
[122] S. Franke and B. Franke, "X-Ray technology for the assessment of existing timber structures", p. 538, 2013.
[123] A. O. Feio and J. S. Machado, "In-situ assessment of timber structural members: Combining information from visual strength grading and NDT/SDT methods – A review", *Construction and Building Materials*, vol. 101, pp. 1157–1165, 2015.
[124] J. M. Branco, "Influence of the joints stiffness in the monotonic and cyclic behaviour of traditional timber trusses. Assessment of the efficacy of different strengthening techniques", University of Minho and University of Trento, 2008.
[125] T. U. Graz, "Mechanic behavior of timber-timber joints of historic and modern timber structures", Graz, 2014.
[126] B. Heimeshoff and N. Kohler, "Assessment of the structural behaviour of timber joints (in german)", Munchen, 1989.
[127] E. Poletti, G. Vasconcelos, J. M. Branco, and A. M. Koukouviki, "Performance evaluation of traditional timber joints under cyclic loading and their influence on the seismic response of timber frame structures", *Construction and Building Materials*, vol. 127, pp. 321–334, 2016.
[128] J. M. Branco, P. Cruz, M. Piazza, and H. Varum, "Modelling of timber joints in traditional structures", *International Workshop on Earthquake Engineering on Timber Structures*, no. May 2015, 2006.
[129] A. O. Feio, P. B. Lourenço, and J. S. Machado, "Testing and modeling of a traditional timber mortise and tenon joint", *Materials and Structures*, vol. 47, no. 1, pp. 213–225, 2014.
[130] "Timber Structure Analysis". [Online]. Available: https://lpms.dicam.unitn.it/?page_id=176. [Accessed: 01-Jan-2020].

[131] M. Del Senno and M. Piazza, "Behaviour and rehabilitation of queen post timber trusses: A case study", *Transactions on the Built Environment*, vol. 66, pp. 803–814, 2003.

[132] M. Piazza, M. Riggio, and G. Brentari, "Strengthening and control methods for old timber trusses: the queenpost truss of the Trento theatre", in *Proceedings of the 5th International Conference on Structural Analysis of Historical Constructions, SAHC 2006*, 2006, pp. 10–13.

[133] J. M. Branco, P. Cruz, M. Piazza, and H. Varum, "Behaviour of traditional portuguese timber roof structures", in *Proceedings of the 9th World Conference on Timber Engineering 2006, WCTE 2006*, 2006, vol. 2, pp. 1588–1595.

[134] J. M. Branco, P. Cruz, and M. Piazza, "Diagnosis and analysis of two king-post trusses", in *Proceedings of the 6th International Conference on Structural Analysis of Historical Constructions, SAHC 2008*, 2008, vol. 1, pp. 459–467.

[135] J. M. Branco, M. Piazza, and P. Cruz, "Structural analysis of two King-post timber trusses: Non-destructive evaluation and load-carrying tests", *Construction and Building Materials*, vol. 24, no. 3, pp. 371–383, Mar. 2010.

[136] J. M. Branco, H. Varum, V. Ramisote, and A. Costa, "Load-carrying capacity test of a long-span timber truss", *Proceedings of the Institution of Civil Engineers - Structures and Buildings*, vol. 169, no. 5, pp. 373–387, May 2016.

[137] J. M. Branco, P. Cruz, M. Piazza, and H. Varum, "Field tests of a timber queen-post truss and numerical analysis", in *Proceedings of the 6th International Conference on Structural Analysis of Historical Constructions, SAHC 2008*, 2008, pp. 507–514.

[138] P. B. Lourenço, H. S. Sousa, R. D. Brites, and L. Neves, "In situ measured cross section geometry of old timber structures and its influence on structural safety", *Materials and Structures*, vol. 46, no. 7, pp. 1193–1208, Jul. 2013.

[139] D. Cibecchini, N. Bjorngrim, H. S. Sousa, M. Jorge, and J. M. Branco, "Repairing of a timber truss through two different techniques using timber elements and screwed metal plates", in *Proceedings of the 2nd International Conference on Historic Earthquake-Resistant Timber Frames in the Mediterranean Region, HEaRT 2015*, 2015, pp. 371–381.

[140] J. M. Branco, H. S. Sousa, and E. Tsakanika, "Non-destructive assessment, full-scale load-carrying tests and local interventions on two historic timber collar roof trusses", *Engineering Structures*, vol. 140, pp. 209–224, Jun. 2017.

[141] H. S. Sousa and L. Neves, "Reliability-based design of interventions in deteriorated timber structures", *International Journal of Architectural Heritage*, vol. 12, no. 4, pp. 507–515, May 2018.

[142] P. Munafò, F. Stazi, C. Tassi, and F. Davì, "Experimentation on historic timber trusses to identify repair techniques compliant with the original structural-constructive conception", *Construction and Building Materials*, vol. 87, pp. 54–66, 2015.

[143] LNEC, *Timber for structures – Maritime pine for structures. LNEC*. Lisbon, Portugal, 1997.

[144] H. Koch and W. Seim, "Load bearing capacity of traditional roof structures - Modelling of joints", *Proceedings of the 9th World Conference on Timber Engineering 2006, WCTE 2006*, vol. 2, pp. 1038–1044, 2006.

[145] B. Wallner, J. Ortner, S. Gregor, T. Bogensperger, A. Meisel, M. Augustin, and G. Schickhofer, "Timber-Timber joints (in german)", Graz, 2014.

[146] P. Palma, J. Ferreira, and H. Cruz, "Monotonic tests of structural carpentry joints", in *Proceedings of the 11th World Conference on Timber Engineering 2010, WCTE 2010*, 2010.

[147] P. Palma, H. Garcia, J. Ferreira, J. Appleton, and H. Cruz, "Behaviour and repair of carpentry connections – Rotational behaviour of the rafter and tie beam connection in timber roof structures", *Journal of Cultural Heritage*, vol. 13, no. 3, pp. S64–S73, 2012.

[148] M. . Drdacky, F. Wald, and J. Mareš, "Modelling Of Real Historic Timber Joints", *Transactions on The Built Environment*, vol. 39, pp. 169–178, 1999.

[149] F. Wald, J. Mareš, Z. Sokol, and M. . Drdacky, "Component Method for Historical Timber Joints", in *The Paramount Role of Joints into the Reliable Response of Structures*, Dordrecht: Springer Netherlands, 2000, pp. 417–424.

[150] J. M. Branco, P. Cruz, M. Piazza, and H. Varum, "International Workshop on Earthquake Engineering on Timber Structures", *Engineering*, 2006.

[151] M. A. Parisi and M. Piazza, "Mechanics of plain and retrofitted traditional timber connections", *Journal of Structural Engineering*, vol. 126, no. 12, pp. 1395–1403, Dec. 2000.

[152] M. A. Parisi and C. Cordié, "Mechanical behavior of double-step timber joints", *Construction and Building Materials*, vol. 24, pp. 1364–1371, 2010.

[153] T. Descamps and P. Lemlyn, "Effects of the rotational, axial and transversal stiffness of the joints on the static response of old timber framings", in *Proceedings of the 1st International Conference on Protection of Historical Buildings, PROHITECH 09*, 2009, pp. 291–286.

[154] J. M. Branco and T. Descamps, "Analysis and strengthening of carpentry joints", *Construction and Building Materials*, vol. 97, pp. 34–47, Oct. 2015.

[155] S. M. Holzer, *Statische Beurteilung historischer Tragwerke. Band 2, Holzkonstruktionen*. Weinheim, Germany: Wiley-VCH Verlag GmbH & Co. KGaA, 2015.

[156] S. M. Holzer, "Analysis of historical timber structures", in *Proceedings of the 10th International Conference on Structural Analysis of Historical Constructions, SAHC 2016*, 2016, pp. 1203–1210.

[157] European Committee for Standardization (CEN), *EN 518. Structural timber. Grading. Requirements for visual strength grading standards*. 1997.

[158] Ente Nationale Italiano di Unificazione (UNI), *UNI 11035-1. Structural timber – visual strength grading for Italian structural timbers: terminology and measurement of features*. 2003.

[159] Nemetschek, *SCIA Engineer User Manual*. 2013.

[160] European Committee for Standardization (CEN), "EN 338:2016 Structural timber. Strength classes", 2016.

[161] S. M. Holzer and B. Köck, "Investigations into the structural behavior of German baroque timber roofs", *International Journal of Architectural Heritage*, vol. 3, no. 4, pp. 316–338, 2009.

[162] L. Giresini, M. Fragiacomo, and M. Sassu, "Rocking analysis of masonry walls interacting with roofs", *Engineering Structures*, vol. 116, pp. 107–120, 2016.

[163] P. Touliatos, "Traditional aseismic techniques in Greece", in *Proceedings of the Interantional Workshop Les sytèmes nationaux faces aux seismes majeurs*, 1993, pp. 110–124.

[164] P. Touliatos, "The box framed entity and function of the structures: the importance of wood's role", in *Conservation of historic wooden structures: proceedings of the international conference*, 2005, pp. 52–64.

[165] S. Tonna and C. Chesi, "Wood reinforced masonry in poor construction traditions", in *Proceedings of the 3th International Conference on Structural Health Assessment of Timber Structures, SHATIS'15*, 2015.

[166] R. Fonti, A. Borri, R. Barthel, M. Candela, and A. Formisano, "Rubble masonry response under cyclic actions: experimental tests and theoretical models", *International Journal of Masonry Research and Innovation*, vol. 2, no. 1, pp. 30–60, 2017.

[167] A. Keller, M. A. Parisi, E. Tsakanika, and M. Mosoarca, "Influence of historic roof structures on the seismic behaviour of masonry structures", *Proceedings of the Institution of Civil Engineers - Structures and Buildings*, pp. 1–27, Aug. 2019.

[168] A. Keller, N. Chieffo, and M. Mosoarca, "Influence of roof structures on seismic behavior of historic buildings", in *Proceedings of the 3rd International Conference on Protection Of Historical Constructions, PROHITECH'17*, 2017.

[169] M. Marin, L. Roman, and O. Roman, "Earthquakes in the Banat area - Timisoara (in Romanian)", *Buletinul AGIR*, vol. 2, pp. 23–27, 2011.

[170] A. Narita, M. Mosoarca, C. Modena, F. da Porto, M. Munari, S. Taffarel, C. Marson, C. Valotto, and M. Roverato, "Behavior of Historic Buildings in Zones with Moderate Seismic Activity. Case Study: Banat Region, Romania", *Procedia Engineering*, vol. 161, pp. 729–737, 2016.

[171] A. Bala, V. Raileanu, C. Dinu, and M. Diaconescu, "Crustal seismicity and active fault systems in Romania", *Romanian Reports in Physics*, vol. 67, no. 3, pp. 1176–1191, 2015.

[172] E. Oros, *Earthquakes in the Banat plain (in Romanian)*. Timisoara, Romania: Graffiti, 1991.

[173] E. Oros and M. Diaconescu, "Recent vs. historical seismicity analysis for Banat seismic region (western part of Romania)", in *Proceedings of the 5th National Conference on Earthquake Engineering and 1st National Conference on Earthquake Engineering and Seismology*, 2014.

[174] E. Oros, M. Popa, and I. A. Moldovan, "Seismological database for Banat seismic region (Romania)-part 1: the parametric earthquake catalogue", *Romanian Journal of Physics*, vol. 53, no. 7–8, pp. 955–964, 2008.

[175] MDRAP, "Romanian Seismic Design Code P 100-1/2013", 2013.

[176] MDRAP, *Design code - Assessment of snow action on structures*. 2012.

[177] V. Bosiljkov, D. D'Ayala, and V. Novelli, "Evaluation of uncertainties in determining the seismic vulnerability of historic masonry buildings in Slovenia: use of macro-element and structural element modelling", *Bulletin of Earthquake Engineering*, vol. 13, no. 1, pp. 311–329, Jan. 2015.

[178] V. I. Novelli, N. Makhloufi, D. Benouar, and A. Zekagh, "A procedure for the identification of the seismic vulnerability at territorial scale. Application to the Casbah of Algiers", vol. 13, pp. 177–202, 2015.

[179] M. Opris, *Timisoara: small urban monography (in Romanian)*. Bucarest: Editura Tehnica, 1987.

[180] M. Opris, *Timișoara: Urban monograph, vol. I, recent discoveries which changed the urban history of Timisoara (in Romanian)*. Timisoara, Romania: Brumar, 2007.