Miguel Jorge Magina

Licenciado em engenharia civil

Historic timber roof structures

Dissertação para obtenção do Grau de Mestre em
Engenharia Civil – Estruturas e Geotecnia

Orientador: Professor Doutor Balint Szabó
Universitatea Tehnică Cluj-Napoca
Facultatea de Arhitectură şi Urbanism

Co-orientador: Professor Doutor Carlos Chastre Rodrigues
Universidade Nova de Lisboa
Faculdade de Ciências e Tecnologia

Júri:

Presidente: Prof. Doutor Corneliu Cismasiu
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FACULDADE DE CIÊNCIAS E TECNOLOGIA
UNIVERSIDADE NOVA DE LISBOA

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LOMBADA

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Dedicatória

À minha família a quem devo tudo e fizeram de mim o que hoje sou.

Dedico ainda esta dissertação aos que já não estão fisicamente presentes mas que continuam caminhando comigo no meu coração.

Resumo

Esta dissertação consiste no estudo de estruturas históricas de madeira em coberturas na zona da Transilvânia - Roménia, abordando-se o tipo de estrutura, os seus elementos e variedade de ligações entre os mesmos. São referidos aspectos de procedimento para estudo de uma estrutura desta categoria, ensaios semi-destrutivos e não destrutivos que se podem realizar para melhor compreender as características actuais da madeira em causa e possíveis soluções de reparação ou reforço da estrutura caso seja necessário.

Por fim, desenvolveu-se um estudo onde se analisa e se propõem algumas soluções de intervenção numa estrutura estilo gótica situada na nave da igreja de Huedin-Roménia.

Palavras-chave: Coberturas históricas; Estruturas de madeira; Ligações entre elementos de madeira; Estruturas históricas; Coberturas na Transilvânia;

Abstract

This dissertation covers the study of historic timber roof structures in Transylvania area - Romania, the structures type, its elements and connection variety between them. Procedures to study a structure of this category are approached. It is also referred semi and non-destructive tests that can be done to better understand the present wood characteristics, and potential reparation or strengthening solutions for the structure in case it is necessary.

Ultimately a case study is analyzed and some intervention solutions are proposed for a gothic structure type in the nave of Huedin Reformed church.

Keywords: Historic Roofs; Timber structures; Timber elements joints; Historic structures; Transylvania roofs;

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Nomenclature / List of Symbols

Latin upper case letters

A – Area.

A_{fr} - Area swept by the wind.

A_{ref} - Reference area.

C_e - Exposure coefficient.

CFRP - Carbon fiber reinforced polymer.

C_t - Thermal coefficient.

C_{esl} - Coefficient for exceptional snow loads.

E - Young's modulus.

$E_{0,05}$ - Fifth percentile value of modulus of elasticity.

E_d - Design value of modulus of elasticity.

E_{mean} - Mean value of modulus of elasticity.

EC1 – Eurocode 1.

EC5 – Eurocode 5.

FRP – Fiber reinforced polymer.

$G_{0,05}$ - Fifth percentile value of shear modulus.

G_d - Design value of shear modulus.

G_{mean} - Mean value of shear modulus.

I_{tor} - Torsional moment of inertia.

I_v - Turbulence intensity.

I_z - Second moment of area about the weak axis.

Q_k - Characteristic value of a variable concentrated load.

R_d - Design value of a load-carrying capacity.

R_k - Characteristic load-carrying capacity.

W_y - Section modulus about axis y.

X_d - Design value of a strength property.

X_k - Characteristic value of a strength property.

Latin lower case letters

b - Width of the structure; Width.

c_d - Dynamic factor.

c_{dir} - Directional factor.

c_{fr} - Friction coefficient.

c_p - Pressure coefficient.

c_r - Roughness factor.

c_o - Orography factor.

c_s - Size factor.

c_{season} - Seasonal factor.

d - Depth of the structure.

e - Eccentricity of a force or edge distance.

$f_{c,0,d}$ - Design compressive strength along the grain.

$f_{m,k}$ - Characteristic bending strength.

$f_{m,y,d}$ - Design bending strength about the principal y-axis.

$f_{m,z,d}$ - Design bending strength about the principal z-axis.

$ft_{,0,d}$ - Design tensile strength along the grain.

$f_{v,d}$ - Design shear strength.

h - Depth.

$k_{c,y}$ or $k_{c,z}$ - Instability factor

k_{crit} - Factor used for lateral buckling

k_{def} - Deformation factor

k_r - Terrain factor.

k_h - Depth factor.

k_m - Factor considering re-distribution of bending stresses in a cross-section.

k_{mod} - Modification factor for duration of load and moisture content.

k_{shape} - Factor depending on the shape of the cross-section.

k_y or k_z - Instability factor.

l - Span; contact length.

l_{ef} - Effective length; Effective length of distribution.

q_k - Characteristic value of a uniformly distributed load, or line load.

q_p - Peak velocity pressure.

s - Snow load on the roof [kN/m^2].

s_k - Characteristic value of snow on the ground at the relevant site [kN/m^2].

s_{Ad} - Design value of exceptional snow load on the ground [kN/m^2].

v_m - Mean wind velocity.

$v_{b,0}$ - Fundamental value of the basic wind velocity.

v_b - Basic wind velocity.

w - Wind pressure.

z - Height above ground.

z_0 - Roughness length.

z_e, z_i - Reference height for external wind action, internal pressure.

z_{max} - Maximum height.

z_{min} - Minimum height.

Greek Lower case letters

α - Pitch of roof, measured from horizontal [º]; Angle between a force and the direction of grain; Angle between the direction of the load and the loaded edge (or end).

β_c - Straightness factor.

γ_M - Partial factor for material properties, also accounting for model uncertainties and dimensional variations.

λ_y - Slenderness ratio corresponding to bending about the y-axis.

λ_z - Slenderness ratio corresponding to bending about the z-axis.

$\lambda_{rel,y}$ - Relative slenderness ratio corresponding to bending about the y-axis.

$\lambda_{rel,z}$ - Relative slenderness ratio corresponding to bending about the z-axis.

$\sigma_{c,0,d}$ - Design compressive stress along the grain.

$\sigma_{c,\alpha,d}$ - Design compressive stress at an angle α to the grain.

$\sigma_{m,crit}$ - Critical bending stress.

$\sigma_{m,y,d}$ - Design bending stress about the principal y-axis.

$\sigma_{m,z,d}$ - Design bending stress about the principal z-axis.

$\sigma_{t,0,d}$ - Design tensile stress along the grain.

τ_d - Design shear stress.

$\tau_{tor,d}$ - Design shear stress from torsion.

μ_i - Snow load shape coefficient.

Ψ_0 - Factor for combination value of a variable action.

Ψ_1 - Factor for frequent value of a variable action.

Ψ_2 - Factor for quasi-permanent value of a variable action.

ρ - Air density.

σ_v - Standard deviation of the turbulence.

1 – Introduction

With a very favorable weight-resistance ratio, wood is a material capable of transmitting both compressive and tension stresses, it is consequently a material naturally suitable for parts subject to bending [48]. Thanks to its versatility, it is not surprising that, it had acquired utmost importance in structure construction in the past. Some of those timber structures were able, with or without some interventions, to survive until nowadays.

The remaining historical buildings and structures, thanks to their unique or rare characteristics (material, design, construction techniques etc.) and history, have an important cultural significance and they are a part of mankind built heritage. For these reasons the structural engineer is more and more designated to study the structures and to extent their life time, preserving their identity /singularity, techniques and the original material as much as possible.

Differently to concrete or even steel, the wood properties are very sensitive to the environmental conditions. For instance the moisture content has a direct effect in their resistance/strength and stiffness [48]. Consequently a good understanding of physical and mechanical characteristics is essential for the execution of safe and durable structures. Exposed to external agents like insects and fungus, the material properties can decrease, so it is necessary to take appropriate precautions.

1.1 – Dissertation objectives

The aim of this work is the assessment, preservation principles and characterization of historic roof structures, particularly the common ones found in Transylvania-Romania (fig.1.1).

For the historic roof structures characterization in Transylvania the goal is to make a survey of the structural configuration for the different types, evolution, components, techniques and joint technology at that time.

The objective for structure assessment and preservation principles is to examine the documentation related to existing building reliability and historical building preservation, present methods to understand the actual properties of timber and to describe the most common interventions. In result the aim is to have knowledge about the best methods to approach and intervene in such structure.

At last, a case of study with the objective to check the reliability of a historic roof structure with Gothic character located in Huedin church nave using Eurocode 5-1-1 (EC5) [26]. Another objective is to see the elements function in the structure and its stress distribution. In order to prolong the structure lifetime, the aim is to propose some interventions according its needs.

1.2 – Dissertation structure

To achieve proposed objectives, the dissertation was divided in the following chapters:

- The first chapter includes an introduction to the dissertation's topic, the organization and the objectives.
- The second chapter is an overview of historic timber roof structures in Transylvania (Romania) area, their structure characterization, its elements composition and connection variety between them. The connection modelling importance is discussed.
- In third chapter, procedures to study structures of this category are approached in order to verify its reliability for future use. The Principles for the preservation of historic timber structures (1999) [35] adopted by ICOMOS, ICOMOS CHARTER – principles for analysis, conservation and structural restoration of architectural heritage (2003) [36] and the ISO 13822:2010 Bases for design of structures – Assessment of existing structures [38] are considered. The national annexes from Italy (UNI 11119 [77] and UNI 11138 [78]) and Swiss Norm 269 [52] are analyzed as well. A methodology to approach these historic roof structures based on the standards is proposed. It is presented semi and non-destructive tests that can be completed to better understand the present timber characteristics and deterioration state. Potential reparation or strengthening solutions for the structures are mentioned.
- In chapter four a survey of a gothic structure type localized in the nave of Huedin church is made along with its deterioration or/and damage state. It is analyzed by Eurocode 1 (EC1) action values for imposed, wind and snow loads and verified the safety to the ultimate state limits according to EC5, as well as the observation of the element function both to symmetrical and asymmetrical loads, and subsequently some intervention solutions are proposed.
- Ultimately in chapter five the conclusions are made.

Fig. 1.1 – European map with Romania and Transylvania enlightened [81].

2 - Historical timber roof structures in Transylvania

Wood has been used by humans since pre-historic times, from temporary shelters with small trunks to imposing buildings with complex technical features. In this research the aim is to give special emphasis to permanent structures which can be related to other ones built in the same period in geometry, mechanical stresses distribution, material, element composition and construction technology. Historic tower roof structures (Fig.2.1) frequently present in churches, with often unique and complex structures are not considered in this work.

According to Szabó [65] a historic roof structure is made of timber built under empirical-intuitive methods without engineering theoretic support, characterized by lying exclusively on supporting load-bearing sub-units (walls or columns) in the building outline.

Fig. 2.1 – Tower roof structure in Huedin church.

2.1 - Roof structure categories

Historic roof structures in Transylvania are part of Continental Roof structures, also present in Germany, France, Sweden, the Czech Republic, Austria, Slovakia and Hungary [67].

According to Szabó [65,66] they can be classified (in accordance with their mechanical behavior) as roof structures on rafters and tie-beams and roof structures on beams.

Historic roof structures on beams are load-bearing sub-units with little complexity, supporting half-pitched or pitched roofs, and as the name implies, the system works through the bending of the beams.

Historic roof structures on rafters and tie-beams are load-bearing sub-units of ample complexity able to cover large spans. Spatial load-bearing structures with thrusts, made up of elements in triangular outlines (tie-beam – rafter – rafter). The gravity actions are divided into slanted compression components (common rafter direction) and balanced by the stretching of the tie-beam and by the wall plate reaction. Historically the structures can be named Romanesque, Gothic, Baroque and Eclectic characters, having in mind that structures with mixtures of more than one of these characters might exist, either the ones in the technology changing processes (Romanesque-Gothic, Gothic-Baroque, Baroque-Eclectic) or the ones subjected to later interventions. According to the way the bending in the rafter is reduced, the historic roof structures on rafters and tie-beams can be divided into historic roof structures on collar beams or historic roof structures on purlins.

Because Anglo-Saxon historic roof structures (coastal) differ from the continental ones in structural conformation and elements, the existing English terminology cannot fulfill entirely the needed terms [47]. Bálint Szabó's Illustrated Dictionary of Historic Load-Bearing Structures [65] is especially focus on Transylvania region and for this reason its terms will be adopted in this dissertation.

2.1.1 – Schematics examples of roof structure types

In order to a better understanding of the terms used in subchapters 2.1.2 to 2.1.5, describing the historic roof structures types, such as, different elements and its location in the structure, the schematics representing some of these possible structures are presented.

The following figures 2.2 to 2.6 represent the main and secondary trusses for Romanesque, Gothic, Baroque and Eclectic, the various historic roof structure types in Transylvania. The figures 2.7, 2.8 and 2.9 represent the longitudinal bracing systems for Gothic, Baroque and Eclectic respectively. In table 3.1 the elements name is described with references, represented in the figures.

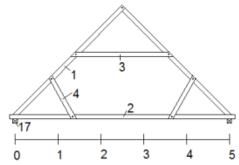


Fig. 2.2 - Romanesque roof structure example.
(Based on [65])

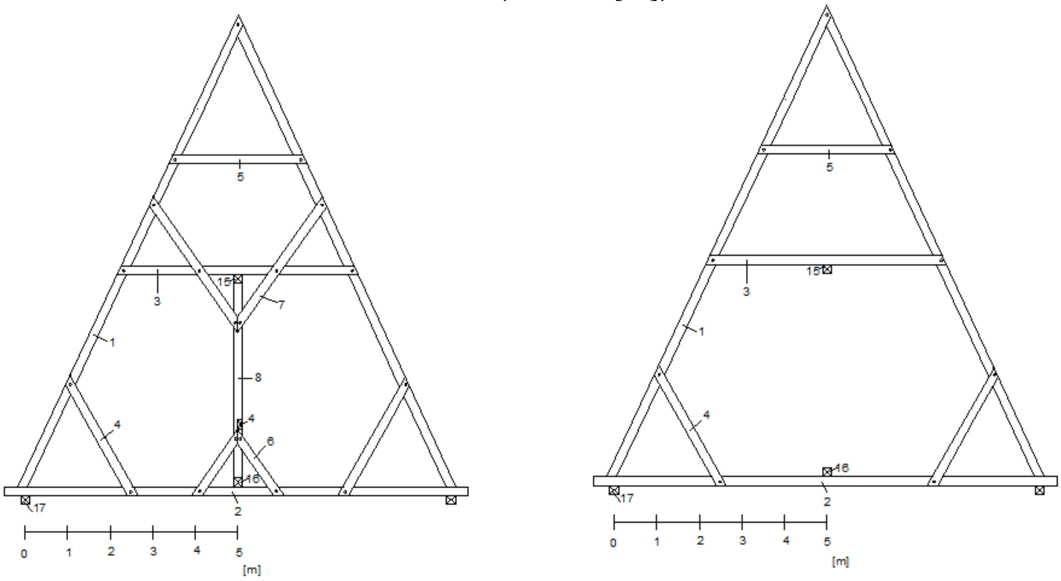


Fig. 2.3 - Gothic main and secondary truss example.
(Based on [65])

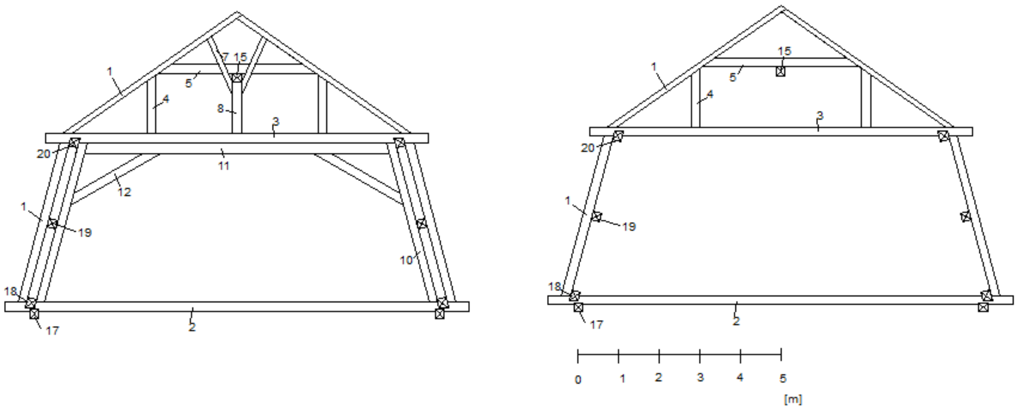


Fig. 2.4 - Mansard Baroque roof structures example.
(Based on [65])

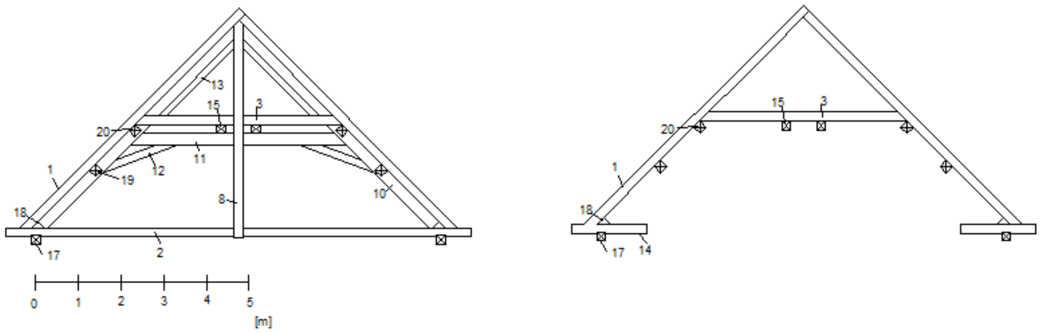


Fig. 2.5 - Baroque roof structures example.
(Based on [65])

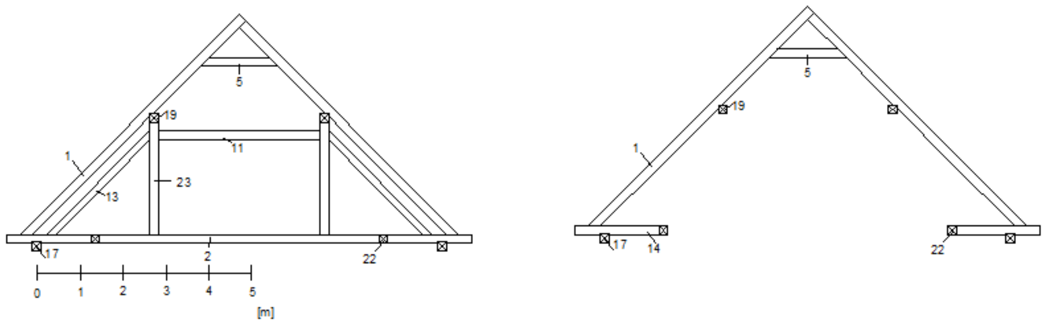


Fig. 2.6 - Eclectic main and secondary truss example.
(Based on [65])

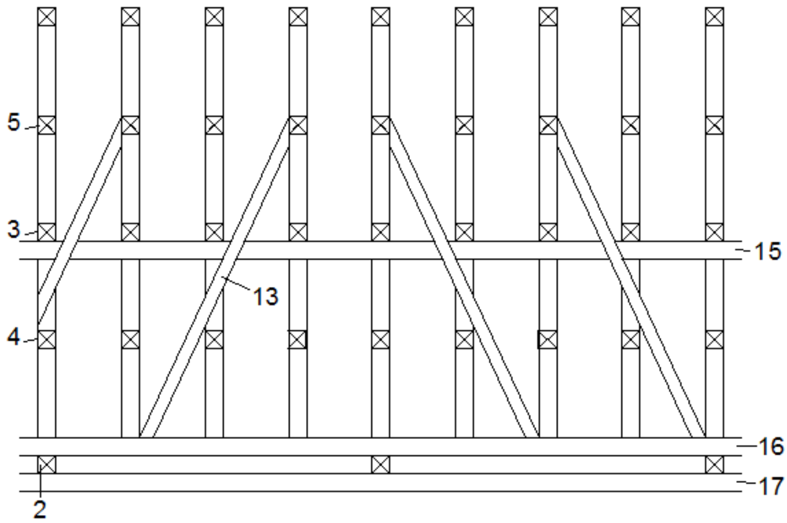


Fig. 2.7 -Gothic longitudinal bracing system example.
(Based on [65])

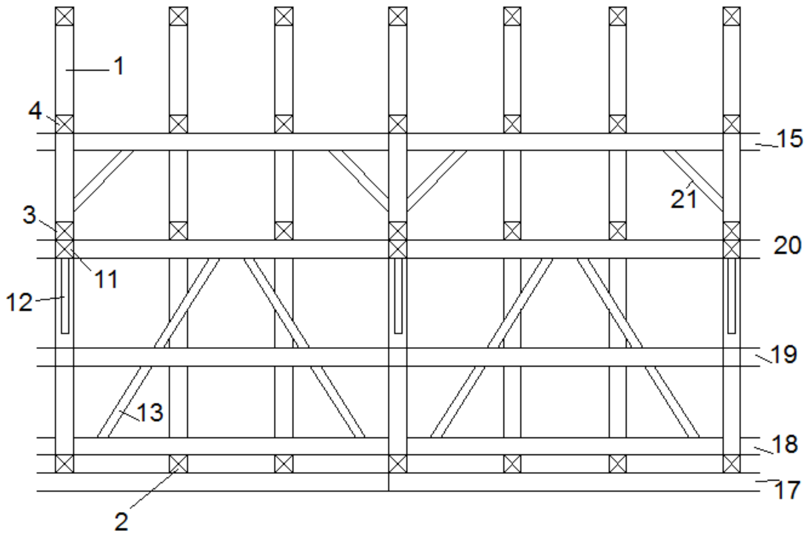


Fig. 2.8 - Baroque longitudinal bracing system example.
(Based on [65])

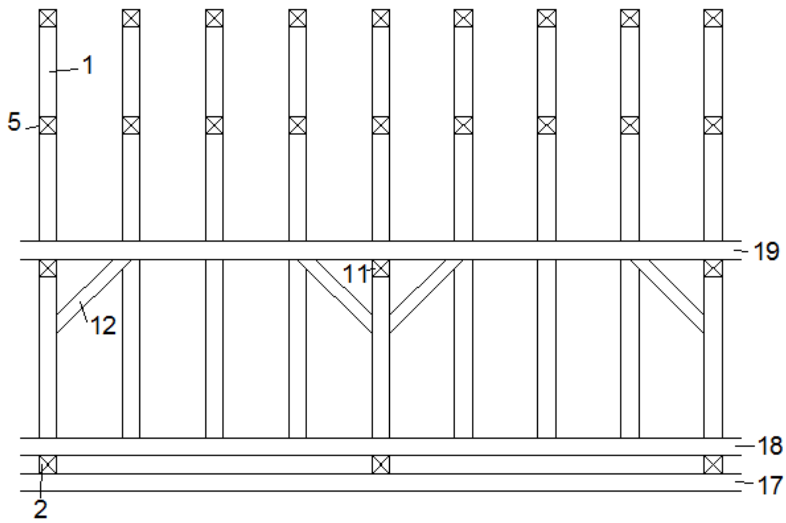


Fig. 2.9 - Eclectic longitudinal bracing system example.
(Based on [65])

Number	Element name
1	Rafter
2	Tie Beam
3	Collar Beam
4	Angle Brace
5	Upper Collar
6	Knee Brace
7	Slanted Strut
8	King Post
9	Queen Post
10	Inner Couple Rafter
11	Straining Beam
12	Counter Brace
13	Passing brace / Compound Rafter
14	Trimmer
15	Upper Plate
16	Lower Plate
17	Wall Plate
18	Eaves Purlin
19	Intermediate Purlin
20	Edge Purlin
21	Bracket
22	Header
23	Hanging Post

Table 2.1 – Element name and numeration.

2.1.2 - Romanesque

These structures were mostly constituted only by one type of transversal frame placed from 0,7 to 1,0 meters between them and with a slope around 40º-60º (not exceeding 45ºusually), their elements were, rafters, tie beam and with the increase of their dimension, also pairs of angle braces and (upper) collar beams (Fig. 2.10 – captions in table 3.1).

The longitudinal bracing system is inexistent unless presented after some structural intervention or roof structures in the Romanesque-Gothic transition. The longitudinal stiffness were guaranteed by the elements supporting the covering, except in rare cases where diagonal elements were located in the outer side or inner side of the rafters or small counterbraces [65,66].

The transversal actions are transmitted from the roof structural elements to wall-plates, that on their turn, transfer the loads almost uniformly distributed (due to the structure symmetry) to the bearing walls that pass them to the foundations. The gables, together with the bearing walls, are responsible for the longitudinal actions passage to foundations [65,66].

The material used is largely hardwood (oak, evergreen oak, etc.) and the construction techniques involve processing by carving the timber into the final form of the element and dovetail half lap joints that contribute to strong connections under axial and bending stresses, connected by wooden pegs to resist shear forces and prevent the elements from separating from each other [65,66].

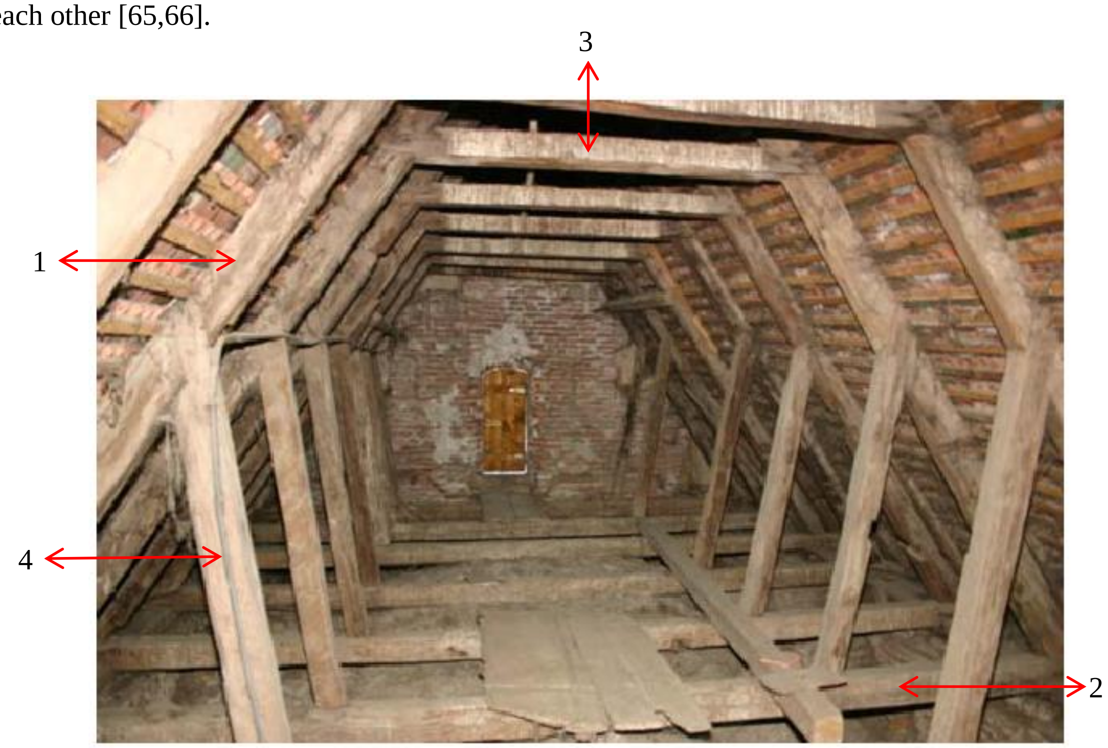


Fig. 2.10 - Romanesque roof structure above the northern portico of the Evangelical church in Sibiu [74].

2.1.3 - Gothic

Regularly constituted by two types of transversal frames (main and secondary trusses), with slopes between 50º-75º. The secondary trusses elements are rafters, tie beam, (upper) collar beam, angle braces and sometimes counter-braces. The main trusses have the same elements in common plus king/queen posts, slanted struts and counter braces (Fig. 2.11 – captions in table 3.1), therefore these trusses are more rigid than the secondary ones, supporting partially them by "absorbing" more stresses than this one, what is more visible under non-gravity loads. Both trusses are self-bearing. The order of these trusses begins and ends with a main one and between them are one or two secondary ones; in rare cases it is possible to have roof structures only with main trusses [65,66].

The longitudinal bracing system is only disposed in the vertical, in the symmetry axis or symmetrically to it. This system is positioned according to needs of the roof structure span and is compost by compound rafters, possibly angle braces and counter-braces, upper and lower plates, that make the connection through king/queen posts (also belonging to the longitudinal system), between the transversal trusses and the rest of longitudinal bracing elements [65,66].

Like the Romanesque the transversal actions are transmitted from the roof structural elements to wall-plates that on their turn transfer the loads to the bearing walls that pass them to the foundations. The gables together with the bearing walls are responsible for the longitudinal actions passage to foundations [65,66].

The material and techniques are similar to the ones used in Romanesque style, normally hardwood and processing the timber by carving it into the final form of the element and dovetail half lap joints that contribute to strong connections under axial and bending stresses, connected by wooden pegs that resisting shear forces, prevent the elements from separating from each other [65,66].

In Romania these kinds of structures were built until and in the 18th century [65,66].

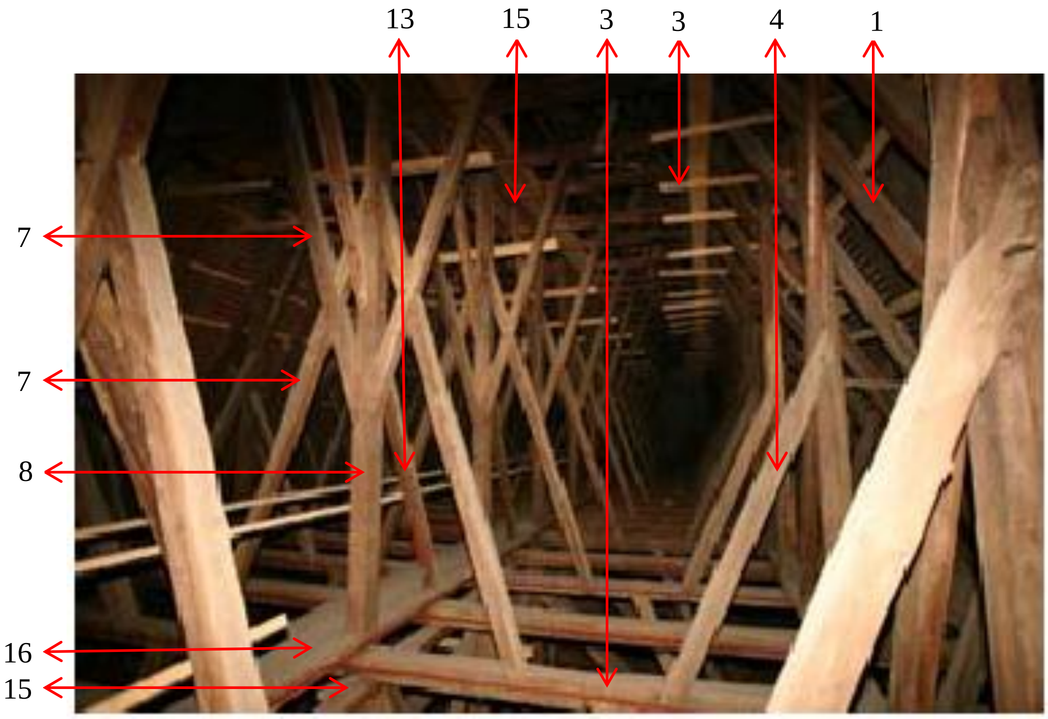


Fig. 2.11 - Gothic roof structure in Evangelical church in Bistrita [74].

2.1.4 - Baroque

Usually constituted by two types of transversal frames (main and secondary trusses), with maximum slopes of 45º, except for mansard roof types where it can reach slopes between 60º-75º. The secondary trusses have rafters, with or without tie beams, (upper) collar beams depending on dimension and trimmers or shoes of eaves purlins. The main trusses besides the secondary elements have inner coupled rafters and angle braces between them and the straining beam, and king post (Fig. 2.12 – captions in table 3.1). Being more rigid, just like in the gothic case, the principal trusses support partially the secondary trusses by "absorbing" more stresses, especially without the tie beams (not self-bearing). Differently from gothic structures this phenomenon is also visible for gravity loads. Commonly four secondary trusses are located between two main ones [65,66].

It is possible to identify four types of main trusses unloading systems after three features: broken or continuous rafters; tie beams in all trusses or just in the main ones; how the horizontal stresses in the rafters of secondary trusses are dealt with [65,66].

The longitudinal bracing system is disposed in the rafter's plane with ordinarily one, being possible two or three levels in rare cases, and is compost by longitudinal bars (often called purlins but with mechanically subjected to different loads) making the connection between longitudinal and transversal frames with the rafters, a pair of compound rafters with different ascending directions between two main trusses and sometimes brackets. Due to its position in the structure and manner of conformation, it is the most effective historic load-bearing structure sub-unit [65,66].

The actions are transmitted to the foundation in the same way as gothic structures are.

The material is largely softwood (fir, spruce, pine, etc.) and the construction techniques involve processing the timber elements trimming them to final form and half lap, notch, grooved, or mortice and tenon joints, connected by pegs of same material to resist shear forces and prevent the elements from separating from each other. The connection between the king posts (in tension) and tie beam is normally made of metal strap-irons. Compared to gothic ones, these connections are not as resistant, when subjected to bending moments [65,66].

In Romania these kinds of structures were built between the 18th and beginning of 19th century.

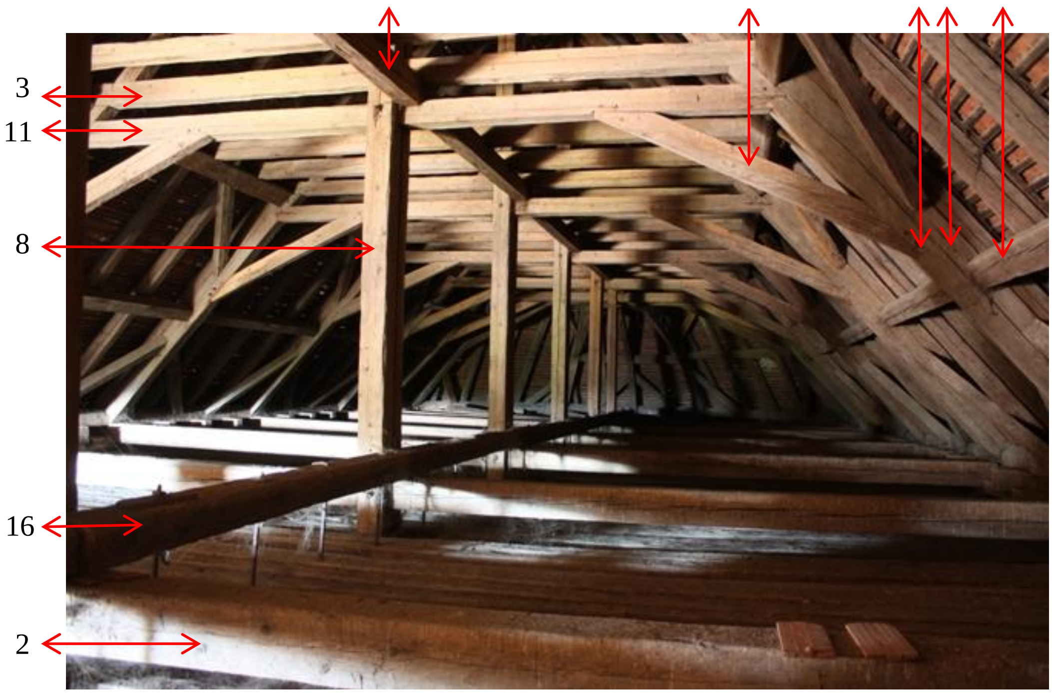


Fig. 2.12 - Baroque roof structure of Cristuru secuisec church, Harghita County [75].

2.1.5 - Eclectic

Normally constituted by two types of transversal frames (main and secondary trusses), with slopes between 30º-45º. The secondary trusses have only rafters, depending on dimension with or without (upper) collar beam, and instead of tie beam they have trimmers. The main frames besides these elements might have more (upper) collar beams, tie beam instead of trimmers, a straining-hanging truss or only a hanging truss and compound rafters (Fig. 2.13 –

captions in table 3.1). Depending on the type of the main truss, king/queen posts and slanted struts can exist. The secondary trusses are not self-bearing and for that reason, they need to rest on the main ones. The trusses disposition have two to five secondary trusses between two main ones [65,66].

The longitudinal bracing system is disposed in the vertical, in the symmetry axis or symmetrically to it with usually one level and in rare cases two. It is compost by upper plates (that can be purlins when the trusses have queen posts), that makes the connection from the transversal with longitudinal bracing frame with the king/queen posts or angle posts (belonging to both systems) and counter-braces [65,66].

The actions are transmitted to the foundation in the same way as gothic structures are.The material and preparing wood techniques are the same as the Baroque style [65,66].

In Romania these kinds of structures were built between the 19th and the Second World War.

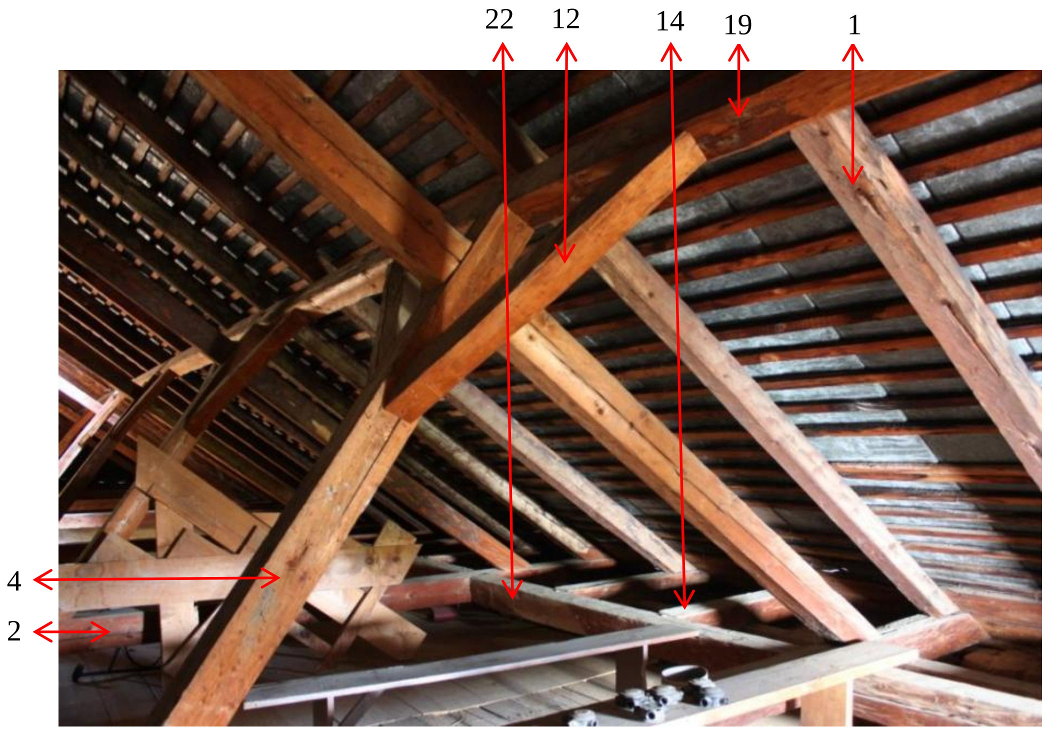


Fig. 2.13 - Eclectic roof structure in Şimleu Silvaniei [76].

2.2 – Timber joints

A historic carpenter’s joint is constructed without design calculations, these connections amongst load-bearing elements of the structure were built based on empirical-intuitive methods. Trimming the timber elements in their joining areas, they take over and transfer mechanical stresses (compression, stretching or shear) from one to the other.

In Transylvania historic roof structures, the connections between its elements can be divided in lapped, notched, grooved and with mortice and tenon [65].

2.2.1 - Lapped joint

Both end and intermediate joints are habitually held together by wooden pegs, they can be lapped on the whole section, 2/3, 1/2 or 1/3 of the element height and they can be straight or squinted. Although they can be found in all historic roof structure characters where the load bearing elements are joined to the wall plate, they are typical for Gothic roof structures, where dovetail lap trimming style allows relative rotation of concurrent elements [65]. Some examples are shown in figures 2.14 and 2.15.

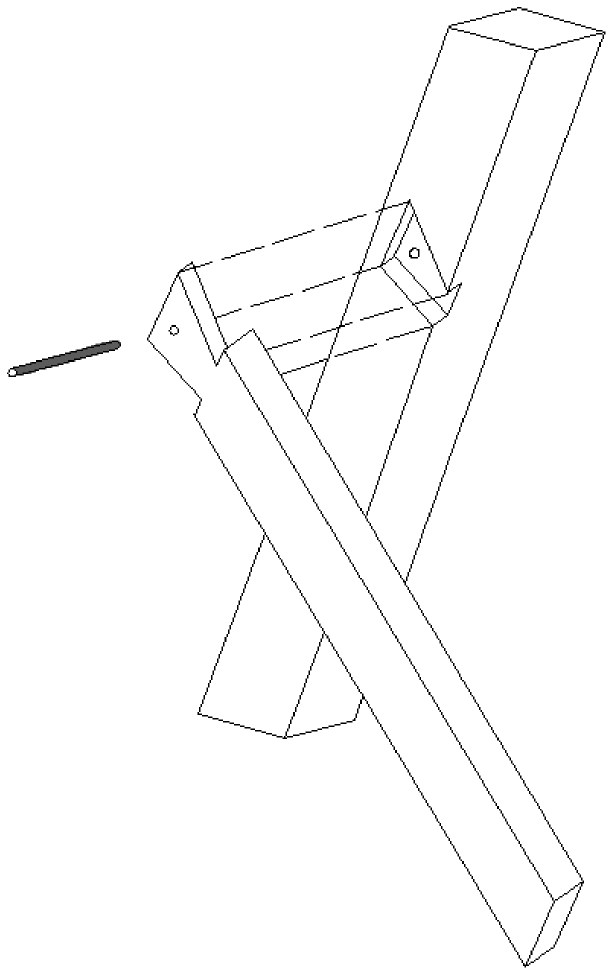


Fig. 2.14 - Dove tail joint
(Based on [65])

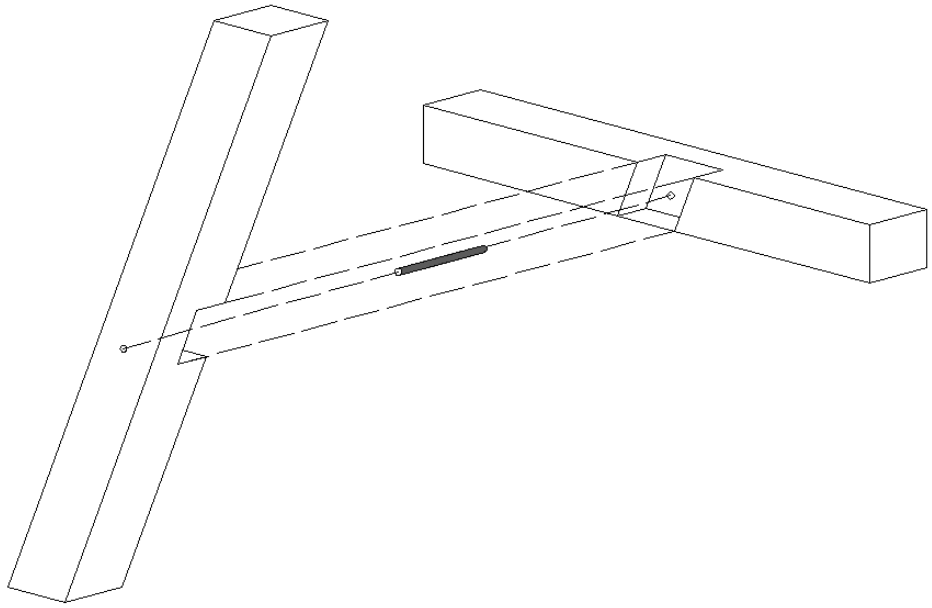


Fig. 2.15 – Half lapped joints
(Based on [65])

2.2.2 - Notched joint

Generally not held together by wooden pegs, these joints can be straight and squinted notches having a simple or double shear threshold and are typical for all historic roof structure characters. The squinted notches are typical for a tie beam connection with rafters [65].

In Gothic roof structures these joints (the straight type) were often used to join longitudinal plates of longitudinal bracing frames to horizontal transversal load-bearing elements like tie beam, collar beam or upper collar. Some examples are shown in figures 2.16 and 2.17.

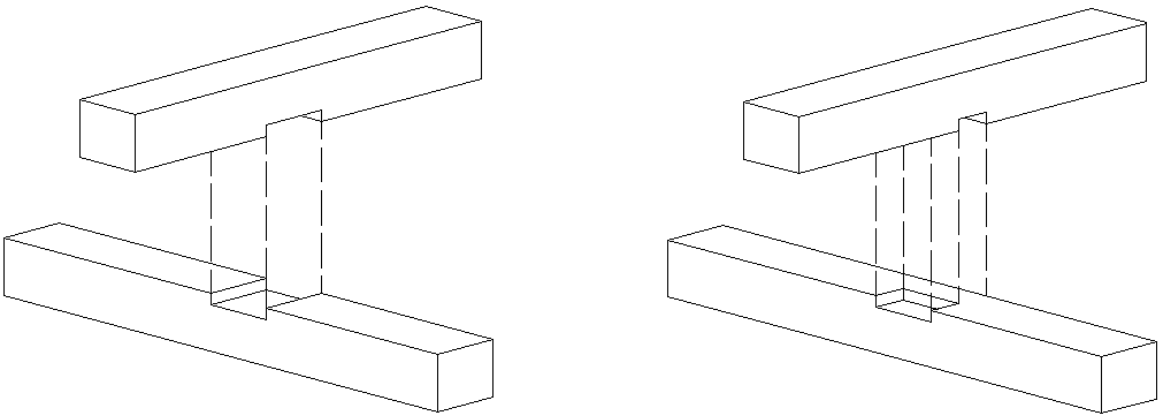


Fig. 2.16 - Notched joints
(Based on [65])

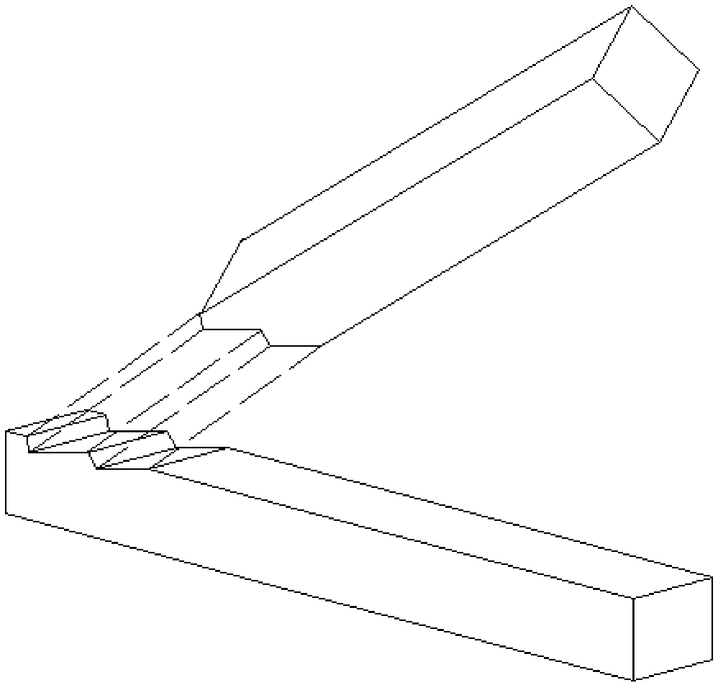


Fig. 2.17 – Squinted notched joint
(Based on [65])

2.2.3 - Grooved joint

Generally not held together by wooden pegs this carpenter's joint is typical for all historic roof structure characters [65]. Some examples are shown in figures 2.18 and 2.19.

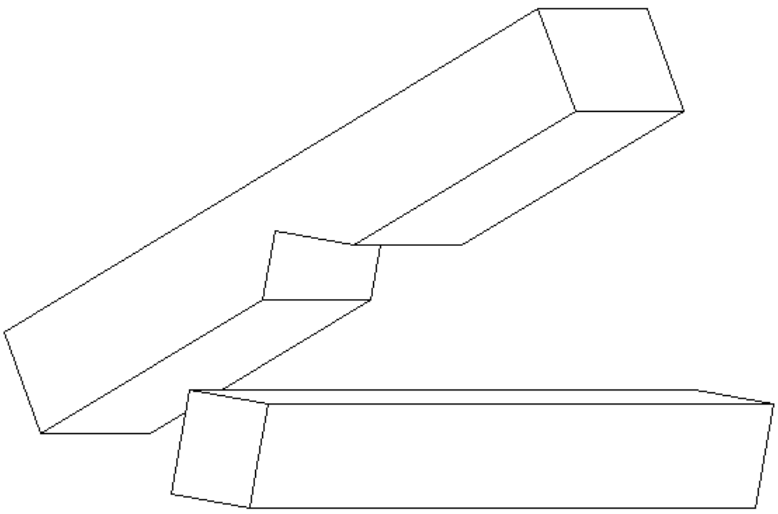


Fig. 2.18 - Grooved joints
(Based on [65])

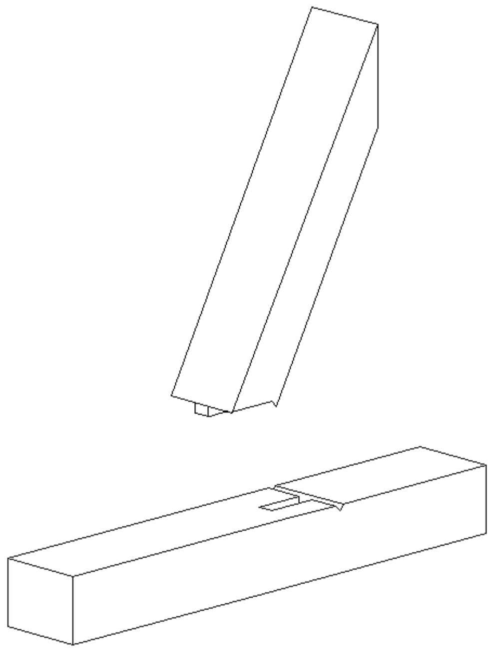


Fig. 2.19 - Grooved joints
(Based on [65])

2.2.4 - Mortice and tenon joint

Operating very well in compressed joints they are usually held by wooden pegs and since the Baroque period aided by strap-irons at stretching stressed joints. They are simple, rectangular or squinted and they appear in all historic roof structures characters. Tie beam – rafter, collar beam – rafter and upper collar – rafter, are some of the connections between load-bearing elements that it are possible to see [65]. A example is shown in figure 2.20.

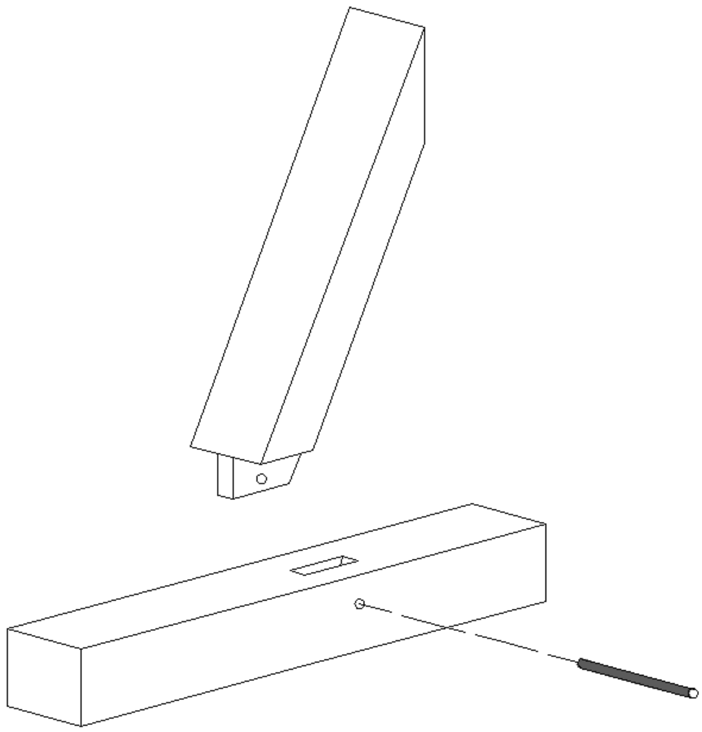


Fig. 2.20 - Mortice and tenon joint
(Based on [65])

2.2.5 – Joint modeling

When dimensioning timber structures it is common to assume the connections as perfect hinged or completely rigid. However in reality these joints, due to the model of the connection itself, gaps between elements and connectors, element crushing and deformation of the connectors, bear a semi-rigid behavior resisting a certain moment.

Adopting perfect hinges in the edge of the elements ensures the modeling of the structure to be conservative. However when analyzing existent structures this (among others) can lead to a failure of the structure according to presently standards, additionally, miscomprehending of the structural behavior can lead to not acceptable stress levels from incorrect joint strengthening [10].

For new or recent existing structures, where the connections are standardized, or the technological processes are properly documented, it is possible to estimate the stiffness mentioned above. However in historical joints without standardization, possible lack of the ideal rigor and precision, inside the same type of connection a lot of scenarios can be found, not to mention the "hidden" joints, where only assumptions of their behavior can be made. These joints (with more or less conservation) could had been there for hundreds of years subjected to different actions (natural and biological) which could compromise their actual integrity.

To evidence the influence of joint stiffening in the global behavior of the truss, Branco, Cruz, Piazza and Varum [8] made two identical models of a traditional Portuguese roof structure in terms of geometry, materials and loads. The models differ only in terms of joint stiffness: one with perfect hinges (Model A) and the other with rigid joints (Model B). The figures on annex 1 shows the bending moment diagrams obtained in both models for a symmetric load case (self-weight) and two non-symmetric load cases (snow and mass force 1). The conclusion they had made, was that for non-symmetric loads, the stiffness influence relevantly the structure stresses distribution.

Although some testing are being made, to better understand the mechanical behavior of historic timber joints with some positive results ([9,29,39]), there is still not consensus about how to model these joints in order to get a more realistic performance. This agreement is not only important to a proper stress distribution of the structure but also as helpful for appropriate repair, conservation and reinforcement [39].

3 – Assessment in historical timber roof structures

3.1 - Standards

In the field of historical timber roof structures there are not available international norms like the Eurocodes specifically made for these cases that guide the engineer through a clear process. In this work the actions and verification of ultimate limit state are made using the EC1 and EC5 respectively. However these standards on their own are not enough in such a complex problem, because they are made for new structures and not for already existing ones.

An overall review of the international ICOMOS (1999) [35], the ICOMOS CHARTER (2003) [36] and the ISO 13822 [38] was accomplished. The first one refers to this study (historical timber structures), the second to structural heritage in general and the last to existing structures of all materials.

The national standards from Italy, UNI 11119 [77] and the UNI 111388 [78], and Switzerland, SIA 269 [52], are also looked into. The Italian standards [77,78] are a series of guidelines purposely for timber load bearing structures and the Swiss [52] like the ISO 13822 [38] to existing structures with a division for timber structures.

3.1.1 - ICOMOS – Principles for the preservation of historic timber structures (1999)

This ICOMOS document [35] aim is to "define basic and universally applicable principles and practices for the protection and preservation of historic timber structures with due respect to their cultural significance." The document refers that historic timber structures are "all types of buildings or construction wholly or partially in timber that have cultural significance or that are parts of a historic area."

The principles of this publication [35] regard to the recognition of the structures from all historical periods as part of the world's heritage as well as their ample variety, material and shortage. The reason of the shortage is due to the susceptibility to external elements resulting in material decline and deficit of skills and knowledge of these construction processes.

Along with the principles, this document also makes some recommendations:

- <u>Inspection, recording and documentation</u> states that before any intervention the condition of the structure and possible causes of decay and structural failure should be recorded. The diagnosis based on a documentary evidence, physical inspection and analysis together with non-destructive tests when necessary;
- <u>Monitoring and maintenance</u> with a consistent plan of action;
- <u>Interventions</u> based on "the optimal intervention is the minimum" concept and with the ultimate objective of integrity and authenticity of cultural heritage, following the traditional means, being reversible if possible and not obstructing future preservation works or access;

- Repair and replacement of the structure should respect historical and aesthetical values to replace decayed or damaged elements or for the sake of restoration requirements. The new material shall be of the same species of the old with same or better grading and with compatible moisture content and physical characteristics, marked for posterior identification and the used construction techniques and tools identical to original ones. The joints between new and old elements should be if appropriate and compatible, a traditional woodwork;
- Contemporary materials and technologies where it affirms that these should only be applied if their durability and behavior is proven for a long period of time.

3.1.2 - ICOMOS CHARTER – Principles for the analysis, conservation and structural restoration of architectural heritage (2003)

The principles of this charter [36] are split in general criteria, researches and diagnosis and remedial measures and controls.

The general criteria declare the need for a multi-disciplinary approach. It refers to the value and authenticity of architectural heritage considering the cultural context which it belongs as long as the integrity of all its components as a unique product of the specific building technology of its time. The division into anamnesis (search for significant data and information), diagnosis (individuation of the causes of damage and decay), therapy (choice of the remedial measures) and controls (efficiency of the interventions) in the examination of a structure is to accomplish the cost saving and minimum consequences on the architectural heritage.

The researches and diagnosis chapter indicates that a multidisciplinary team should be working together since the first steps of study. The understanding of the material and structure characteristics is necessary in conservation practice as long as information about the original and earlier states, its techniques, alterations and effects, the phenomena occurred and its present state. The diagnosis are divided in historical, qualitative (based on observation of decay and damage) and quantitative (material and structural tests, monitoring and structural analysis). At the end of the process, before any intervention, the causes of damage and decay should be identified and a safety evaluation should be made taking in consideration that the application of requirements for new buildings can be inappropriate leading to excessive or impossible measures. In these cases the charter without specifying affirms that specific analyses and appropriate considerations can justify other approaches to safety.

The remedial measures and controls chapter declares that the best treatment is preventive maintenance and it should head to the cause rather than the symptoms based on the safety evaluation and significance of the structure and that these should be kept to a minimum with the least harm to the heritage values and only when demonstrated imperative. The choice of techniques should be the least evasive and most compatible with the structure, reversible, as close as the traditional ones and characteristic qualities of structure and its environment in their original or earlier form should not be destroyed. Deteriorated parts should be repaired every time, if it's possible, instead of replacing them. The dismantling and reassembling should only be considered when no others options are possible or simply harmful. Imperfections and alterations that become part of the structure history should be kept if the safety is applied. When there are difficulties measuring the real safety level of a structure, "observational methods" should be planned, starting with a minimum level of intervention but with the possibility of correction measures in the process. Measures impossible to control shouldn't be allowed to carry out and checks and monitoring plans after intervention shall be made.

Although only the Principles have the status of an approved/ratified ICOMOS [36] on this document, it also includes a section with guidelines for:

- Acquisition of data: Information and Investigation (Generally; Historical, structural and architectural investigations; Survey of the structure; Field research and laboratory testing; Monitoring);
- The structural behavior (General aspects; The structural scheme and damage; Material characteristics and decay processes; Actions on the structure and the materials);
- Diagnosis and safety evaluation (General aspects; Identification of the causes; Safety evaluation; The problem of safety evaluation; Historical analysis; Qualitative analysis; The analytic approach; The experimental approach; Decisions and explanatory report);
- Structural damage, materials decay and remedial measures (General aspects; Masonry building; Timber; Iron and steel; Reinforced concrete).

3.1.3 - ISO 13822 - Bases for design of structures — Assessment of existing structures (2010)

The ISO 13822 standard [38] "provides general requirements and procedures for the assessment of existing structures based on the principles of structural reliability and consequences of failure" trying to limit interventions to a strict minimum. Aside from being applicable to any type of structure or material (not specifically for wood) to check the reliability of the structure, the standard attends cost saving and not the importance of historic building conservation and its special demands. Nevertheless still it can serve as an adequate guide line for historical, structures when proper considerations are made.

The procedures of the standard will depend on the assessment goals but the general steps (with a recommended former site visit) are:

- <u>Specification of the assessment objectives;</u>

- Scenarios described in safety plan processes taking into consideration structural condition alterations or actions;
- Preliminary assessment divided in study of documents and other evidences, preliminary inspection for structural design and damages identification, preliminary checks to determine critical deficiencies, decisions on immediate actions when necessary and recommendations for detailed assessment;
- Detailed assessment divided in detailed documentary search and review, detailed inspection and material testing if there are doubts about the values provided in documents or the absence of these, determination of actions, determination of properties of the structures, structural analysis taking into account the deterioration of the structure and verification to assure a target reliability level;
- Results of assessment in form of a report of the assessment, conceptual design of construction interventions with general proposals in annex H, when the performance of the structure is considered inadequate and control of risk as an alternative to construction interventions, controlling or modifying the risk factors.

The ISO 13822 [38] also gives indications regarding the determination of actions and environmental influence, material and structure properties and dimensions.

The structural analysis and correspondent verification is also mentioned on the standard where an assessment based on satisfactory past performance is covered. Therefore assessment of safety, if a structure was designed with previous codes or good construction practices, it can be considered as safe (except in case of accidental actions) if it doesn't reveal significant damage, distress or deterioration, demonstrated satisfactory performance for a long period of time caused by extreme actions from usage and when external elements have occurred. Future deterioration should also be predicted and the maintenance established to provide adequate durability to the structure; no alterations and changes in the use of the building should be foreseen for a long period of time, which can change its actions and thereafter durability. From the serviceable point of view it can be considered as acceptable if it doesn't reveal significant damage, distress or deterioration or demonstrated satisfactory performance for a long period of time for them and vibrations to occurs, and there will be no future alterations to the structure and its use that can change its loads. The future deterioration must be predicted and planned to maintain established adequate durability.

In Figure 3.1 it is presented a general flowchart for assessment of existing structures, which summarizes in a simplified form the ISO 13822 [38] recommended procedures.

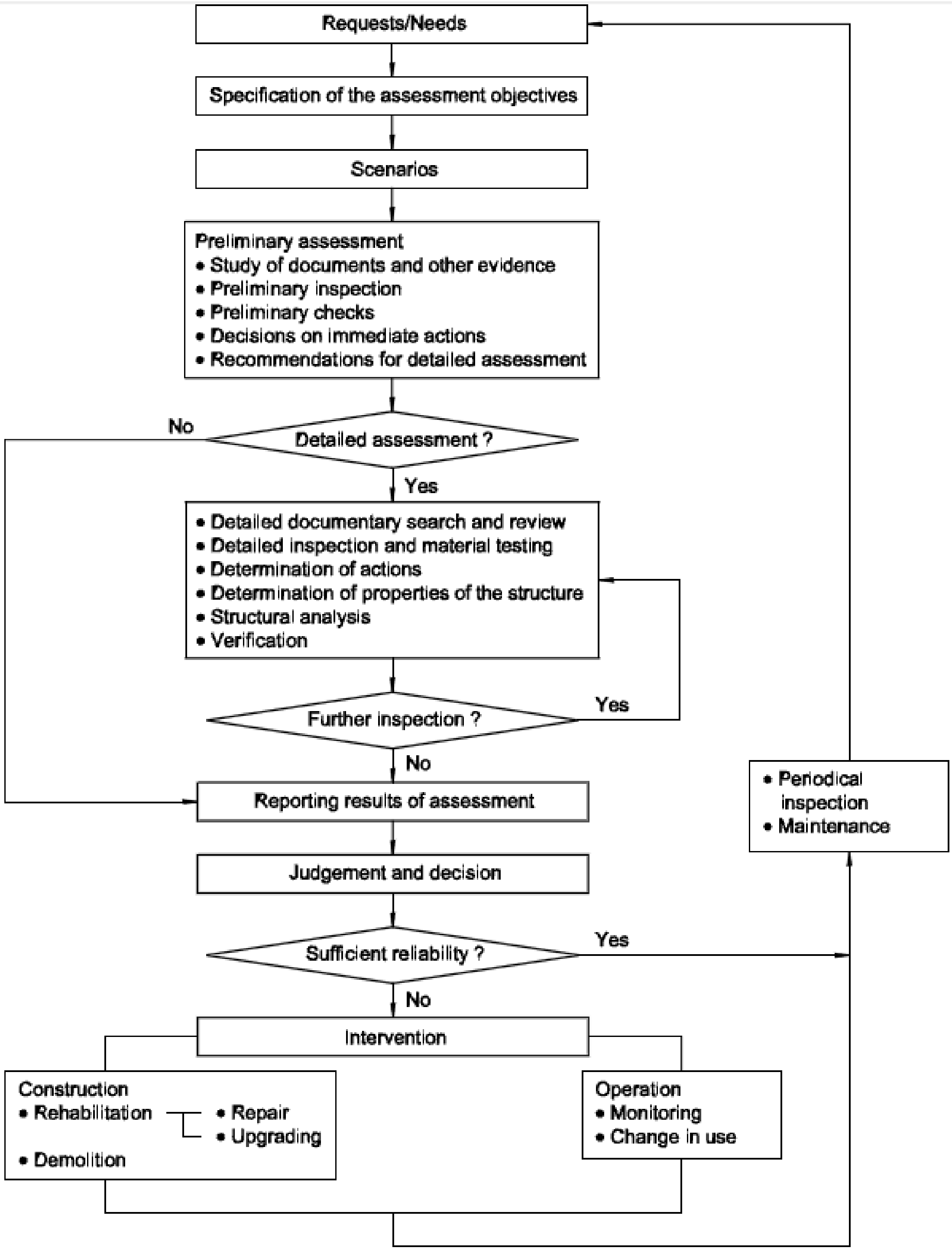


Fig. 3.1 – ISO 13822 - General flowchart for assessment of existing structures [38].

3.1.5 – The Italian standards

With the national necessity of standard techniques to deal with restoration problems of wood and wood based products, the Work Group 20 of the UNI-Normal Technical Committee start to develop standards following two principles:

- The importance of the actual material rather than the typology of the artifact;

- The standards were developed as a guideline, since the vast variety of cases within the field of cultural heritage does not consent the imposition of a single methodology with the criteria “pass-do not pass”.

From this group of standards there are two already approved, related to the dissertation work, the UNI 11119:2004 and the UNI 11138:2004 [44].

The UNI 11119:2004 - Cultural Heritage – Wooden artifacts – Load-bearing structures - On site inspections for the diagnosis of timber members [77], establishes objectives, “procedures and requirements for the diagnosis of the state of conservation and the evaluation of the strength and durability of timber members in load-bearing structures in the area of cultural heritage, through the execution of on site inspections and the use of nondestructive techniques and methods.”

More specifically the objectives of inspection are to obtain information about, wood specie; moisture content; class of biological risk according to EN 335; geometry and morphology of the members indicating its position, defects extension, decay or damage; position, shape and dimension of possible critical areas (part of a timber element with longitudinal axes no less than 150mm, which is considered to be relevant for the diagnosis due to defects, position, state of conservation and high stress conditions revealed by static analysis); strength grading of members and critical areas [43,44,46,77].

For a quality inspection the standard defines that all members should be accessible, cleaned from dust, dirt and others, to make the surface visible and proper lighting should be provided. When a visual grading is not possible (beam parts inside walls for example), non-destructive tests must be performed [43,44,46,77].

Glued laminated elements due to impossibility to achieve the objectives in situ, and decorated or covered timber elements cannot be applicable to this standard.

The UNI 11138:2004 - Cultural Heritage - Wooden artifacts - Load bearing structures of buildings - Criteria for the preliminary evaluation, design and execution of works [78], “provides information on the criteria that should be followed in the preliminary evaluation, planning and eventual execution of conservation, maintenance and restoration of wooden artifacts which serve as load bearing structures in buildings of cultural interest.”

With the main aim of structure conservation from building of cultural, historical and artistic interest, it is obligatory considered to carry out some verifications and evaluations as a preliminary request:

- preliminary verification of the compatibility of use and destination, namely the function of the building or its parts and the structural performance;

- preliminary evaluation of the state of conservation and the service condition of each element;
- preliminary evaluation of the building or monument condition, that may influence the type and entity of an eventual intervention.

These verifications and preliminary evaluations will identify if the structure requires an intervention or does not, and if so, avoid incompatible adjustments with the intervention propose [43,44,46,78].

For the preliminary evaluation of the state of conservation, the standard divides it in historical analysis (structural typologies, their evolution, construction characteristics, traumatic events, used material, construction techniques…), characterization of materials (identification according to UNI 11118 [80] and evaluation of the state of conservation and strength grading according to UNI 11119 [77]), geometric characterization (geometric survey, including deformations, on timber elements individually, joints and the structure as a whole), characterization of decay (recognize, specify and characterize biotic decay and possible environmental poor conditions, that may be the decay reason), structural analysis (evaluation of actual static condition as well as proposed interventions) and presentations of results requirements [43,44,46,78].

The intervention planning must respect the preliminary evaluation phase mentioned above and the type of intervention, material and methodologies must be compatible with the present structure. The aim, verifications and the acceptability of the relative results during interventions should be foreseen and clearly indicated. The planning is divided into:

- Informative criteria for design choices, where it states that the interventions shall not be hidden but harmonized with the present structural content and tendentially reversible. The removal of past interventions (especially the ones that become stratified through time) shall be avoided. Joints, nodes and restrains, unless proved to be inadequate, should be restored to their original stiffness. The removal of decayed parts which can no longer be rescued and its replacement by timber of the same characteristics is only admissible when the element doesn't present works of artistic interest on its surface;
- Definition of actions, as well as the static safety, shall be assigned by UNI EN 1991-1, UNI EN 1995-1-1 and the national law D.M.LL.PP:16/01/1996. However dealing with existing structures of cultural interest, the standard refers that it can be appropriate to evaluate the safety of the artifact, being possible to adapt the values of safety coefficients (in particular for permanent actions), the importance of framework and consequently the acceptable level of risk, provided that this adaptations are justified in detail;

- Interventions of recuperation, that discuss the reintegration of material, the recovery of the structural continuity of a wooden element, the reintegration of the functionality of joints between elements, the interventions by means of pre-stresses, the indirect interventions, the performance improvements with respect to exceptional actions and the treatments;
- Definition of maintenance and inspection program;
- Essential requirements of a project, divided in essential requirements for the preliminary plan, essential requirements for the definitive plan and essential requirements for the execution plan.

The efficiency of any intervention shall be carried out before intervention itself, through direct physical experimentation or numeric simulation, based on trusted and proven mathematical models [43,44,46,78].

The methodologies and techniques in executing an intervention have to be executed by experienced and specialized technical workers [78].

The typology and time intervals of periodic inspections shall be selected. These periodic inspections should take particular attention to biological attacks and possible new or damage/deformation evolution, when compared to previous interventions or inspections [78].

3.1.5 – The Swiss standards – SIA 269 - Basis for examination and interventions

The SIA 269 [52] emerges from the necessity of a standard on existing structures for engineers, to avoid rather cost-intensive or even unnecessary interventions (which are often the result of insufficient know-how and information, about the existing structure). It specifies the principles, the terminology and the appropriate methodology for dealing with existing structures of different materials including timber [12,43].

The standard SIA 269/1 – Actions in existing structures, holds updated models for actions and action effects, focusing in the determination of characteristic values and other more detailed information by means of measurements and refined modeling. It uses geometric configuration of concentrated and uniformly distributed loads based on the standards for new structures, but the characteristic values are updated based on the specific conditions of the existing structure [12,43].

The approach of SIA 269 divides in (i) Requirements (Utilization, Structural safety, Serviceability, Proportionality of intervention), (ii) Updating (Actions and actions effects, Structural and material properties, Model updating), (iii) Structural analysis and verifications (Deterministic approach, Probability approach), (iv) Examination (Condition survey, Condition evaluation, Recommendation of intervention), and (v) Interventions (Planning and design of interventions, Surveying and maintenance, Urgent and supplementary safety measures, Rehabilitation and modification) [12,43].

The standard SIA 269/5 Exiting Timber Structures [53], provides detailed provisions concerning the condition survey in view of determining reliable updated characteristic values of resistance of timber material and connections as well as the corresponding updated resistance factors [12,43].

3.1.6 – Methodology - Assessment, conservation and restoration

Taking in account the above reviewed international standards, plus the reviews of Swiss and Italian national standards, it is visible that all of them share (sometimes disposed in different order) common guidelines.

More specifically the ICOMOS (1999) [35] was made for the kind of structures discussed in the present work but with no mathematical support, only principles/guidelines; the ICOMOS CHARTER (2003) [36] was made for structural heritage in general but with the same "problems" as the ICOMOS (1999). The Italian standards discussed also consist of guidelines for load-bearing timber structures, which direct the mathematical support for the Eurocodes and national laws (with proper updates if needed) and where it is possible a strength grading of the timber. The ISO 13822 [38] and the SIA 269 standards [52,53] are directed to socio-economic interests towards sustainability principles safeguarding and not heritage preservation, however it can be used as guideline. In addition the SIA 269 and the Italian standards are valid only at national level.

Based on these standards, a methodology is proposed (used ahead in the study case), separated in five stages, the information collecting, the in situ survey, the analysis, the interventions and the maintenance plan:

- In the first stage it is important to collect as much information as possible about the building history, its alterations, interventions and its reasons, disaster events, used material and techniques, structural and architectural drawings;
- The second stage has a major importance since an accurate knowledge of the actual structural configuration, its dimension, connections type and material is essential for a later modeling closer to the real structure behavior. The material identification should be based in visual inspection and whenever possible with the use of non-destructive tests, especially in areas of uncertainty. Besides these characteristics mentioned above, the survey should also identify decayed and damaged material and its likely explanations. For the survey, the structure should be cleaned with non-aggressive products and good conditions of luminosity should be given. The number of site visits should be as many as necessary;
- The third stage is the modeling of the structure supported by structural analysis software and correspondent safety evaluation. Taking in account that most standards are for new structures, in case of safety failure, and since these structures survive for

centuries through the different external actions, an update of the characteristic values can be made, especially for the permanent loads. Another possibility is to use a probabilistic approach to evaluate the safety, like the examples of Machado et al. [45] to better predict the properties of timber members in situ, and of Sousa et al. [59] to predict reliability parameters and evolution of timber decay;

- The fourth stage is the interventions (if required) that should always be kept to a possible minimum and be reversible, using only proven techniques that do not endanger the original and earlier states of the structure. The choice between minimal losses of historic fabric, compatibility with heritage values, respect for the original techniques and aesthetical evasion (even knowing that an intervention should be visually detached from the original material), shall be made case by case. These interventions should always be performed by specialized and experienced personnel on this kind of work, and preferably should act on causes rather than decay symptoms;
- The fifth and final stage is the maintenance plan that should specify the typology and time intervals of periodic inspections, taking special attention to possible increasing deformation and biological attacks.

3.2 – Timber possible causes of deterioration

Timber can be affected with natural defects like knots, hardwood eccentricity, cracks/shakes, cross grain, resin pockets, etc., that can influence its resistance and therefore its quality. These defects can be avoided to some extent when choosing the timber, however other phenomenon can cause deterioration and damage, and amongst them, the most common according to the specialized bibliography in general, are the moisture content, and the insect (Fig. 3.2) and fungal (Fig. 3.3) attacks. Environmental problems [Heat (Fig. 3.4), Light (Fig. 3.5), Condensation, Chemical attack (Fig. 3.6)] and others like, improper building changes (Fig. 3.7 and 3.8), mechanical damage and unforeseen events can also cause timber deterioration/damage [1].

Besides the effect in physical and mechanical properties of wood, the high moisture relations in timber is normally the most common danger for the timber, since it creates a propitious environment for biological (insect and fungal) attack. The strength can be related with the moisture, from the green condition until 4-6% of moisture content, the strength increases. The creep (progressive deformation under constant load) is influenced by moisture content. Creep in constant damp conditions exceeds the one in constant dry conditions, and creep under fluctuating moisture conditions exceeds the one under constant damp conditions [13].

Especially with moisture contents above 18%, but also under these values, insects infest wood, destroying it with their nests and for some species it is also a food supply. Sometimes it is possible to have insects where decay is taking place, presumably because this wood is easier to chew. There are three main types of insects infesting wood, the carpenter ants, the boring beetles and the termites [13].

Unlike mold and stain, there are three main types of wood decay (structural degradation due to fungal attack, digesting components of wood cell walls [13]), the soft-rot, the white-rot and the brown-rot that can seriously decrease timber strength [33]. In order to propagate they need besides the wood, high moisture contents (above 20%), oxygen and appropriate temperatures (20-40ºC) [13]. Fungal attack is very difficult to identify in its early stages and brown-rot, for example, can reduce timber mechanical properties up to 10%, before any weight loss is observed. With weight loss from 5% to 10%, the mechanical properties can be reduced from 20% to 80% [33].

According to Beridean [5] an increase of the temperature from 25ºC to 50ºC will decrease the tension and shear resistance with 15-20% and compressive resistance with 20-40%.

The light when combined with wind, water movement and/or freeze and thaw can create stress that causes small surface splits and cracks, that lead to slow material lost (An estimated 1-7mm erosion rate per century, depending of wood specie and exposure degree) [1,16].

Being naturally acidic, most timbers are not affected by mildly acid or salty conditions, however strong alkalis (pH>10) and strong acids (pH<2) can damage the surface. Also structure elements made of water-sensitive metals (such as iron nails) may begin to corrode when in contact with moist timber. This reaction of the metal can cause chemical reactions in the timber elements next to the metal, weakening this area [1].

Taking in consideration that historic timber structures most likely had reparations or even alterations over time, if the quality of these interventions (methods or material) were inappropriate, instated of resolving the issue it could have become part of the deterioration problems. Some of these usual mistakes include, cutting elements or removing them to put doors, windows, staircases or pipes passage, convert two single areas into one without structural consideration and apply external mortar repairs trapping water inside [1].

Accidental actions like fire, flood, earthquake and collisions are probable to happen in a historic roof structure considering their existing age [1]. Even if a roof structure is well preserved and in accordance with security standards, if for instance, the wall plates collapse, deform or sink excessively, the structure will eventually follow the same path suffering deformations or collapsing.

It is of utmost importance to identify not only the decayed/damaged wood to possible interventions but also their origins to prevent the phenomenon to keep occurring.


Fig. 3.2 – Termite attack [4].


Fig. 3.3 – Brown rot [4].


Fig. 3.4 – Heat damage – House of Lords [1].

Fig. 3.5 – Light modifies wood surface, turning it silver-grey. When combined with external elements, it can create small surface splits and cracks [1].

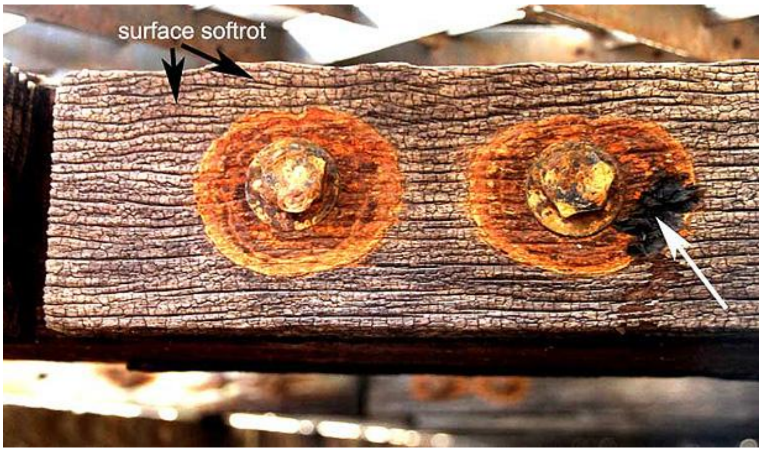


Fig. 3.6 – Chemical damage [34].

Fig. 3.7 – Copious ironwork was amateurishly inserted to arrest its collapsing floors and splaying walls [3].

Fig. 3.8 – Poor workmanship - Pseudo-cruck constructed door frame with every scantling pinned in with wrought-iron nails [3].

3.3 – Non-destructive and semi-destructive timber rehearsals

In addition to visual inspection where experienced inspectors have in overall a more accurate estimation of the strength class then the less experienced ones [17,58] by recognition of the type of wood and identification (mostly superficial) of natural (knots, grain orientation, reaction wood) or biological (decay due to insect and fungus action) defects, deformation, mechanical damage and their position, it is more than reasonable, if possible, to make certain wood tests in order to complement/acquire an improved knowledge of aforementioned class and problems extent.

The EN 14081-1 [20], more specifically the Annex A, gives general requirements for visual strength grading (some of them responsible for considering timber as C18 and below or above C18):

- Limitations for strength-reducing characteristics (Knots, Slope of grain, Density and rate of growth, Fissures);
- Limitations for geometrical characteristics (Wane, Warp);
- Limitations for biological characteristics;
- Other characteristics (Reaction wood, Other criteria).

This Standard is supported by different national grading rules, EN 1912 [21] (complemented by the national standards of each country) which connects the available grades and species to strength classes and EN 338 [79] which provides the characteristic values (used by structural engineers) of visually strength graded timber to the Classes D18 to D70 for hardwoods and C14 to C35 for softwoods [6].

It is possible to identify three types of timber tests, non-destructive, semi-destructive and destructive. Destructive tests are the most truthful to identify the mechanical properties of wood, however because of the historical and architectural value it is not acceptable to subject the structure or its constituents to destructive tests (like the ones in EN 408-1995), unless on elements that due to damage or decay, are no longer suitable and need replacement. Because semi-destructive tests require the extraction of small specimen for testing or that cause small damages to the elements, they should be performed with some reservations. Non-destructive tests, besides being useful for rapid screening of timber for potential problematic areas or internal conditions, typically are not particular reliable for identifying material properties since they suffer from errors resulting from weak correlation between destructive and non-destructive parameters [41].

According to Tannert et al. [69] the development of harmonized test recommendations would have a positive impact on the evaluation of historic structures and decision-making processes in their restoration, with consequent profound effect on costs. They also mention that at the moment more research is required to both, estimate accurate individual member strength as well as, obtain accurate quantification of deterioration.

Non and semi-destructive tests can be grouped into two groups: Global Test Methods (GTM) and Local Test Methods (LTM) [30].

The following table (Table 3.1) resumes some of the test types and methods, with respective information gathered from them according to Magnus [46]:

Method	**Information about:**	**ND / SD**	**GTM / LTM**
Visual inspection	Geometry, rate of growth, natural and mechanical defects, …	ND	GTM
X-ray visualisation	Internal knots and voids, structural defects, decayed wood	ND	GTM
Optical scanning	Surface knots, decayed wood,…	ND	GTM
Infrared thermography	Internal knots and voids, structural defects, decayed wood,…	ND	GTM
Videoscopy	Internal knots and voids, structural defects, decayed wood,…	SD	LTM
Stress wave technique	The dynamic modulus of elasticity, internal defects	SD	GTM
(Ultra)-sonic technique	The dynamic modulus of elasticity, internal defects	ND	GTM / LTM
Static bending test	Load-deflection relationship leads to the static modulus of elasticity	SD	GTM
Extraction of samples	Wood density, moisture content, decay, strength characteristics	SD	LTM
Screw withdrawal test	Density, shear strength and surface damage	SD	LTM
Rod penetration (Pilodyn)	Density and surface damage	SD	LTM
Resistance drilling (Resistograph)	Density and defects	SD	LTM
Legend: ND – non-destructive test; SD – semi-destructive test GTM – global testing method; LTM – local testing method			

Table. 3.1 – Timber tests resume [46]

3.3.1 - X-Ray

Due to the difference in the density or the material capacity to absorb the radiation, it is possible to gather evidence about the decay of the material, structural defects like knots, hidden connections, voids and biological attacks inclusively with the possibility of the agent identification [42,46,69]. In order to produce the image responsible to identify the evidence mentioned above, it is necessary to use a X-ray source that is projected into the element and a recording plate on the opposite side (it is important to have access to both sides of the element) that capture the X-rays that passing through the object, later transformed into image (see Fig. 3.9), using photographic films or digitizing methods [42].

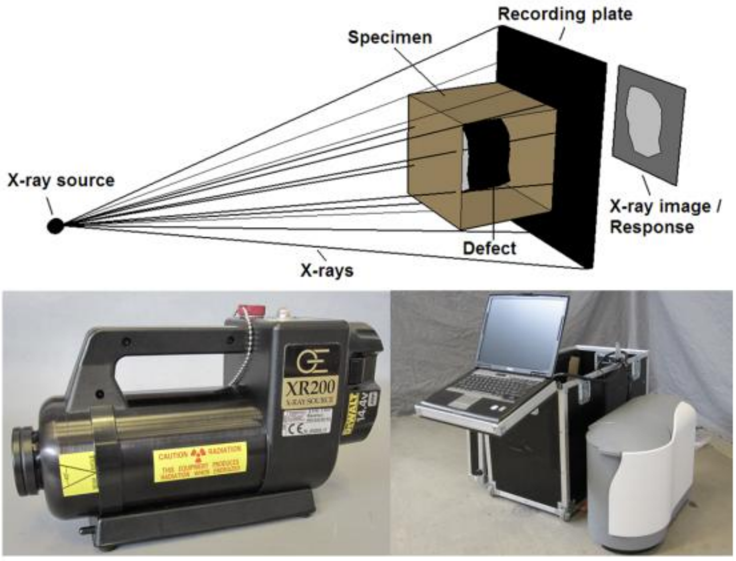


Fig. 3.9 – X-ray machine and process schematic [42].

3.3.2 - Thermographic camera

This device (Fig. 3.10) detects the thermal energy (proportional to the temperature) radiated from objects (surface) in the infrared band (a longer wavelength than visible light) of the electromagnetic spectrum, transforming it into a visible image [49,71].

When under the same environmental conditions, a homogeneous material temperature distribution is uniform, what means that substantial temperature differences shown by thermographic camera can be evidence from deterioration, structural defects, voids, damp conditions (Fig. 3.11), insect attack (Fig. 3.12) or hidden connections. Bigger temperature differences can indicate deeper defects [49].


Fig. 3.10 – Thermographic camera [72].

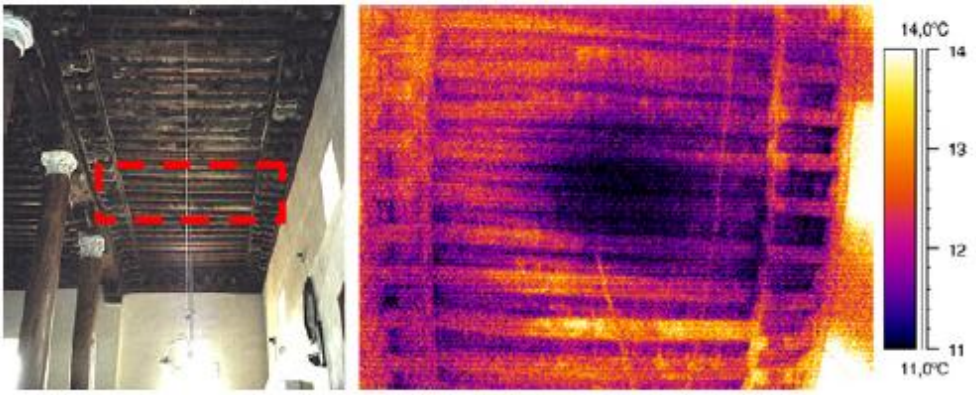


Fig. 3.11 – View of the timber ceiling (left); Infrared image of the selected region (right) showing the damp areas at ceiling. [40].

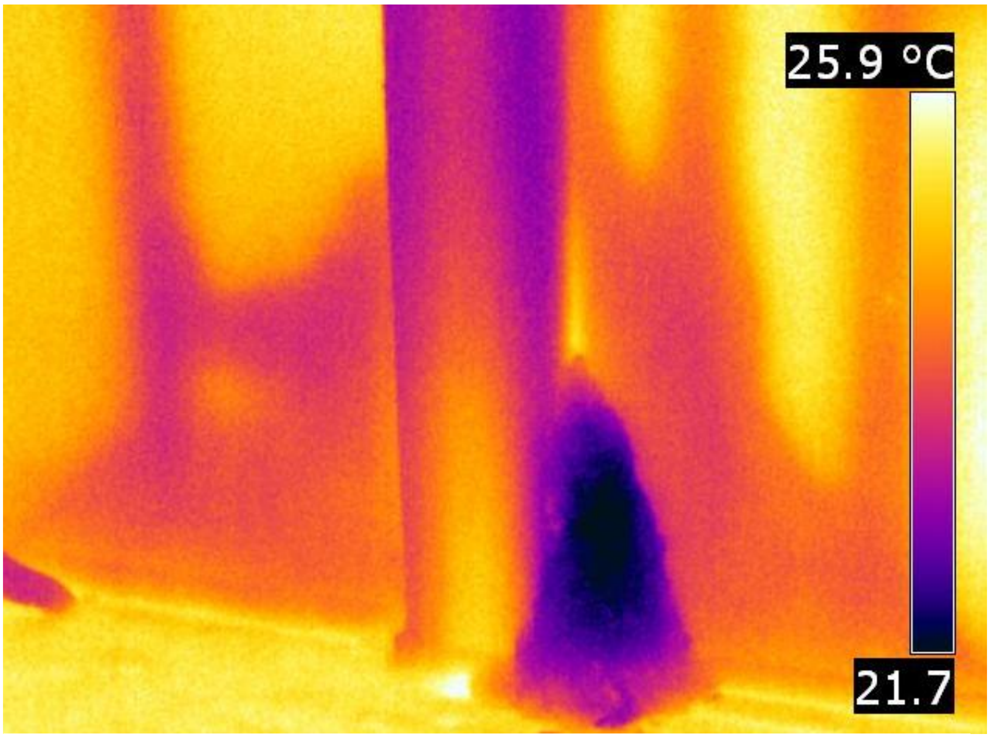


Fig. 3.12 – Termite pack inside and on wall [71].

3.3.3 - Videoscope

The videoscope (Fig. 3.13) is a tool with a small camera fixed at the end of a flexible tube and a monitor displaying the camera images. The indicated equipment can be used either to access inaccessible places or to examine the inside of components, inserting the camera into gaps, crevices or holes previously in situ or made by a drilling device offering knowledge about changes in material or possible voids [19].

Fig. 3.13 – Videoscope [14].

3.3.4 - Wave propagation devices

The basis of these devices is the time that a wave needs to go from point A to point B (Fig. 3.14). Knowing that the fastest way is the shortest way, if the wave is taking longer than it should comparing to others from its class, it means that the wave found deviating conditions expected for its class on their way. These tests can also be used to identify a wood class, if it's unknown, by comparing the propagation velocity to the already known ones.

The propagation of waves from impact, sound or ultra sound will depend on the elasticity of the material. Because rotten/degraded wood is less rigid than sane one, the wave will take longer to cross it. If the wave needs to contour voids left by insect action, it will take longer to go from one point to the other. It is possible this way to identify eventual defects in wood [18].

Another wood property to be measured can be by the attenuation of the wave propagation in the material. A wood degraded by fungus, for example, absorbs more of the wave energy than not an infected one [18].

Fig. 3.14 – Fakopp device - Acoustic wave propagation in clear and decayed trunk [27].

3.3.5 – Resistograph device

The resistograph (Fig. 3.15) is a instrument with a similar action to a drill with small drill bits $\varnothing_{max}$=3mm. As the drill enters the element, depending on the resistance offered by the wood and the progressive penetration of the drill bit, it is possible to identify a typical density variations as well as physical discontinuities like cracks or biological attacks through graphic interpretation (Fig. 3.16) [30,41,70].

Fig. 3.15 – Resistograph [50].

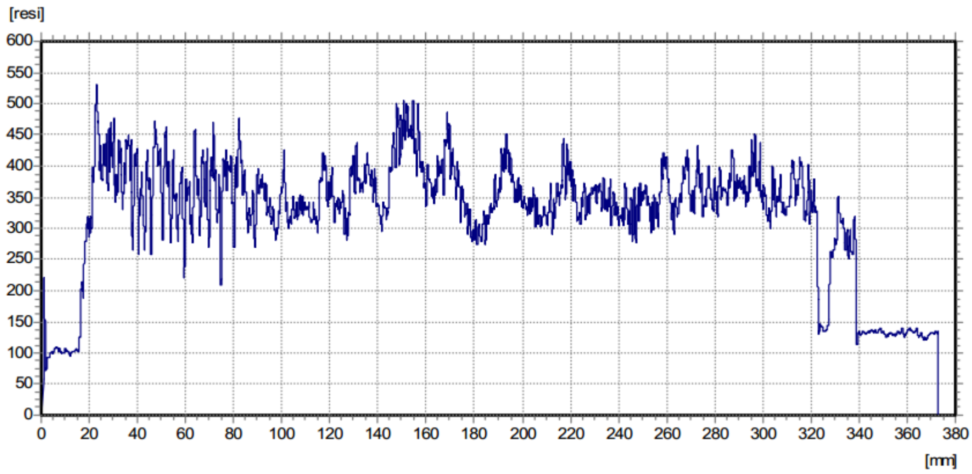


Fig. 3.16 – Resistograph graphic example obtained from a timber element [30].

3.3.6 – Pilodyn device

The Pilodyn (Fig. 3.17) method consists of wood dynamic impact penetration through a blunt steelpin (∅=2,5mm and maximum penetration of 40mm) driven with a precise force, which depth penetration is inversely related to the wood density (see Fi. 3.18) [30,42,51,70].

With this method is also possible to identify superficial deterioration of wood which is related to deeper penetrations [46,70].

The moisture content of the wood as well as the penetration direction (penetration depth is higher in radial direction than in tangential direction), influence the results [46,70]. Correction factors can be applied, so a wider moisture content interval can be tested [46].

This is a simple, quick and not expensive test, which can be done and analyzed in situ. However for better results, several tests should be completed and compared to others done in similar conditions.

Fig. 3.17 – Pilodyn [14].

Pilodyn Depth Penetration Vs. Density

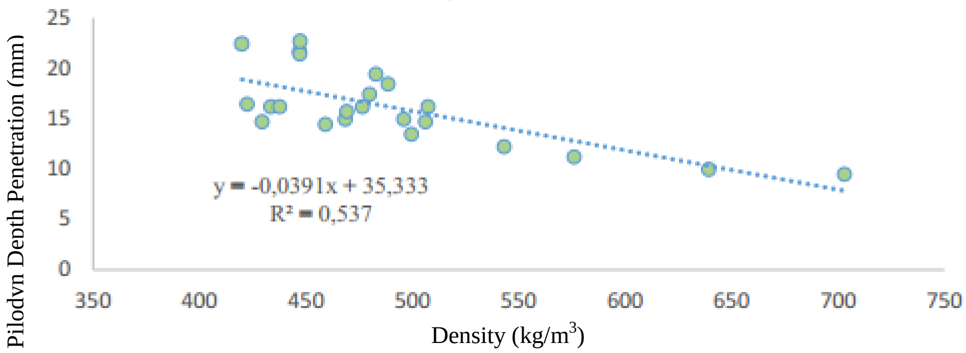


Fig. 3.18 – Pilodyn depth penetration (left) Vs. Density (bottom) – example. (based on [51])

3.3.7 - Core drilling

It consists on extract small wood circular specimens from the timber with core drilling devices (Figs. 3.19 and 3.20) and subject them to testing fixtures to establish material properties such as compressive strength and modulus of elasticity. These samples can be used to identify the moisture content as well as density and age [41,42].

Since wood is an anisotropic material (with material properties directionally dependent), and its strength properties along the fibers (that directly control parameters as bending, tensile and compressive strength along fibers), are the most important since strength across fibers rarely yields to catastrophic failure, the tests are made parallel to the grain [41].

The cores are loaded parallel to grain (perpendicular direction of core longitudinal axis – see Fig. 3.20). The core deformation is measured by two miniature LVDTs (Linear Variable Differential Transformer).From the interpretation of load-deformation curve (3.21), the modulus of elasticity (slope of graphic) and compressive strength of the material (yield point) are obtained. Through correlation it is possible to estimate the properties of full structural members from small specimens [41].

Fig. 3.19 – Core drilling devices [31].

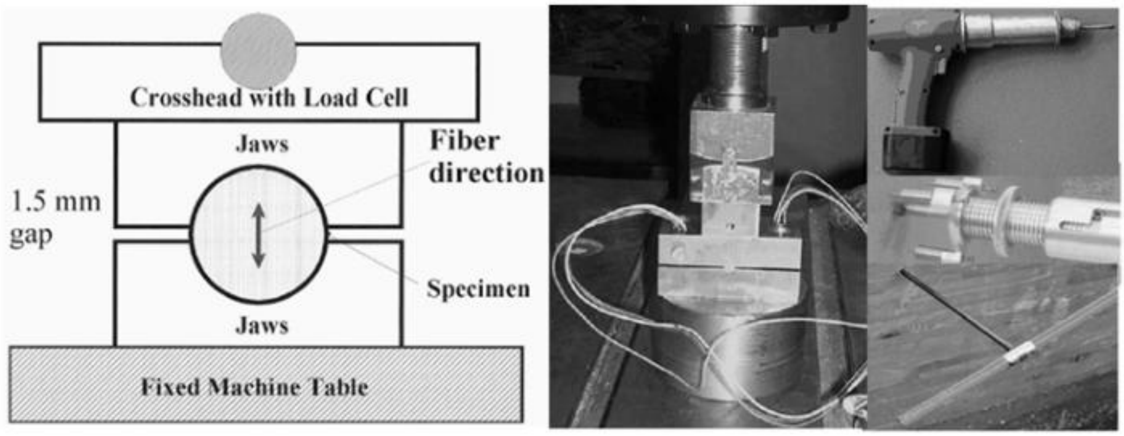


Fig. 3.20 – Schematic testing device and mechanical equipment for core drilling samples [42].

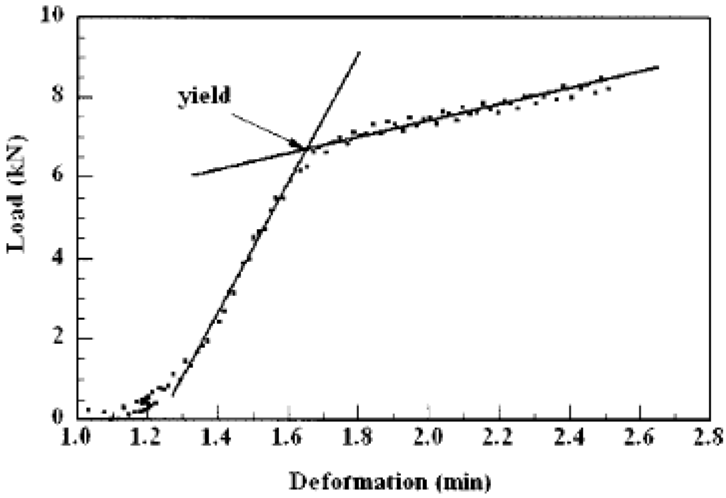


Fig. 3.21 – Typical load-deformation curve for the 4.8-mm core tested in compression parallel to fibers [42].

3.3.8 – Dendrochronology

According to Botár et al. [7] it is the most accurate method for dating archaeological wood finds, leading to impressive results over the years specifying accurately the age of the wood and in many cases the trees origin location even if it is hundreds of kilometers away from the current site.

Since green wood is easier to work with and taking into account that it was handmade to its final shape on site, as the construction progressed (without electric tools as nowadays), the cutting of the tree and the construction date of the roof commonly is not very distinct.

Simplistically, trees from the same specie growing in the same geographical region exposed to the same conditions tend to develop the same unique way. Based on that, the method consists of the study of the ring growth patterns (see Fig. 3.22), measuring the variations of the timber trunks cross section thickness and comparing and identifying it with a reference chronology database (if one exists) [7].

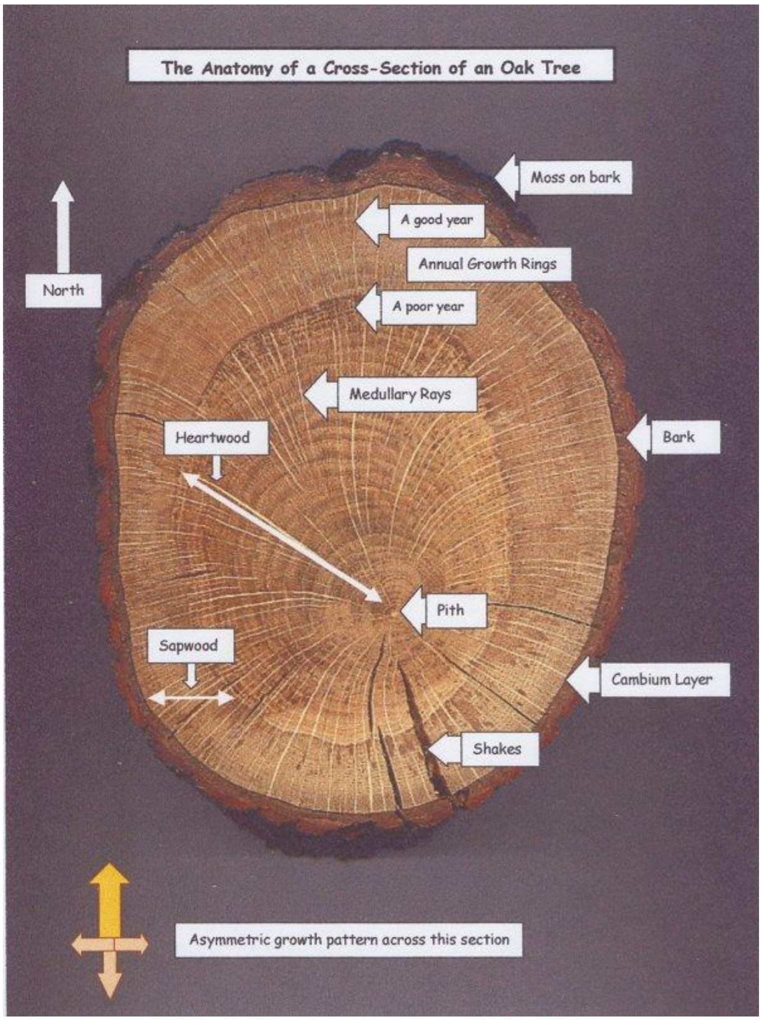


Fig. 3.22 – Anatomy of a cross-section of oak [73].

3.3.9 – Hygrometer

In order to determine the moisture content in timber elements, the most used method, is through hygrometers (Fig. 3.23) that uses its two metallic ends to measure the timber electric resistance and therefore its moisture content. They are more effective between moisture contents from 12% to 22% [11].

Although hydrometers are not designed to detect deterioration, it is recommended as a first method to use in areas where there are suspicions, since moisture contents above the fibers saturation point indicate proper conditions for deterioration development.

Fig. 3.23 – Hygrometer [55].

3.3.10 – Tension Micro-Specimen Technique

To estimate the bending strength (together with compression along the fibers are predominant types of loading), it is necessary to use much larger specimens than core tests (to predict compressive strength) [41].

The extraction of specimens of small triangular cross-sections (significantly smaller than the area of the members – side of the triangular specimen adjusted from 3 to 8mm) achieved by a small diameter thin kerf saw inclined at 45º relative to the surface (Fig.3.24), later attached at the ends by epoxy adhesive to grooved wooden blocks (Fig. 3.25) and subjected to tensile tests performed with special grips (Fig. 3.26), can be used to get the modulus of elasticity in tension and tensile strength. Once the material strength in tension and compression is known, it is possible to design a member under bending [41].

Because the cross-section of these specimens are comparable to the ones required for small-clear specimens of wood of ASTM (American Society for Testing and Materials) (around $8mm^2$), it is possible to compare directly the values from this test with the standard tests without need of correlation [41].

These tests demand the correlations between natural defects and mechanical properties to be known, so the results from small specimens can be converted to properties of full structural members. It is also necessary to use a relatively large number of specimens to ensure reliable date [41], what should be avoided in historical structures.

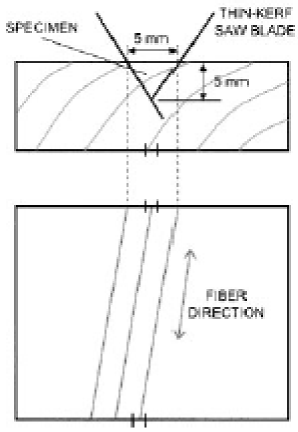


Fig. 3.24 – Schematic of extracting the tension micro-specimen from timber surface [41].

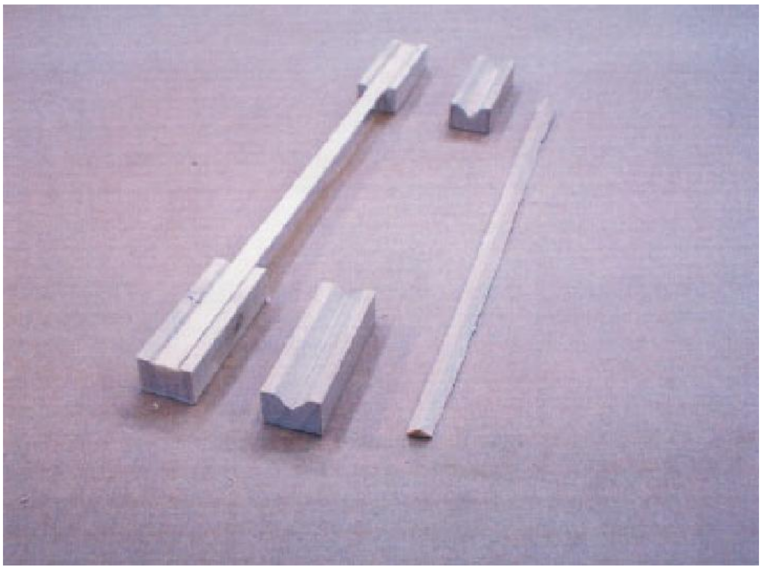


Fig. 3.25 – Tension micro-specimen mounted on the test blocks to minimize potential end effects due to clamping [41].

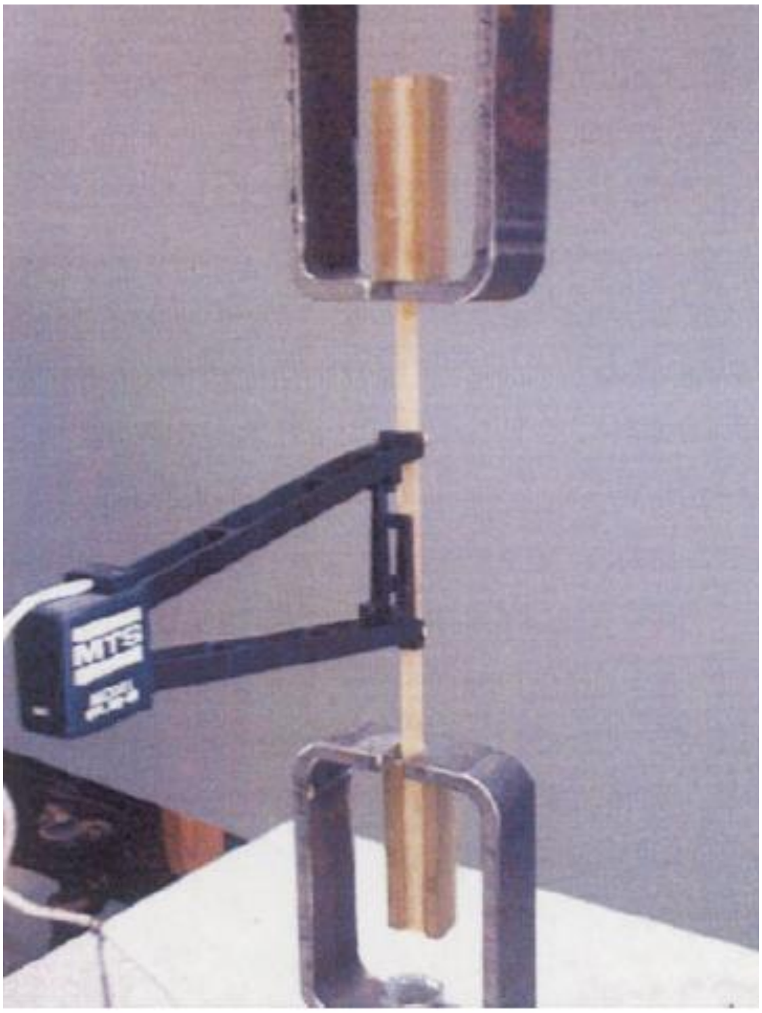


Fig. 3.26 – Testing of the tensile micro-specimen. The central portion defining the gauge length is machined to a smaller cross-section [41].

3.4 – Intervention solutions

According to Feilden (1997) [28] depending on the depth of the intervention, the following degrees can be distinguished (with the possibility of simultaneous degrees in the various parts of the "whole"): prevention of deterioration, preservation of the existing state, consolidation of the fabric, restoration, rehabilitation, reproduction and reconstruction.

- Prevention of deterioration (or indirect conservation) – the action or process of controlling the structure environment, protecting the agents of decay and damage to become active along with regular inspections;
- Preservation – the action or process of taking measures to prevent further decay in order to keep the structure in its existing state;
- Consolidation (or direct conservation) - the action or process of taking necessary measures to maintain the structural integrity and materials of an historic building;
- Restoration - the act or process of accurately recovering the form, features and character of a historic building, as they occur in a defined historical period by

removing features from a different historical period and recovery features sticking in that period of restoration;

- Rehabilitation - the action or process of making the possible use of a historic building by repair, modification/transformation and new parts add, using - case by case - methods of conservation, restoration, reconstruction, and changes or additions needed, keeping the historical, cultural and architectural characteristic properties.
- Reproduction - the action or process of copying an existing element, frequently to replace missing or decayed parts, to maintain its aesthetic harmony;
- Reconstruction - the action or process of restoring - partially or totally - through a new construction shape, features and details of site, landscape, building, structure or object, in order to copy the structure appearance in a specific time period in the history.

Once the assessment of a historical timber roof structure is done, it might come to a conclusion that an intervention is needed in order to preserve, conserve or restore it, if its original shape assures the safety and if it doesn't, improve with additional new elements, change the function of the building/structure or simply limit its loads.

Structural interventions can be separated into two basic groups: structural or existing elements reinforcement; and repairs to correct deterioration related defects or structural failure. In simple terms, they can be categorized as timber-to-timber repairs, sometimes with mechanical fastenings; timber components or structures reinforced with other load-bearing materials (steel, fibre-reinforced polyester rods, epoxy resins); timber replaced with other load-bearing materials; provision of complementary structures which act either combined with the existing frame or independently [1].

The addition of supplementary structures when the original proves to be insufficient sometimes is inevitable either for a particular element(s), truss(es), joint(s) or carrying load system (transversal or longitudinal deficiency to resist loads and conducting them to the bearing elements) or better solutions based on heritage and significance evaluation that can limit the range of valid options.

These adopted solutions are to be analyzed case by case searching for the more effective, reversible and less evasive. The following aspects should be taken in consideration [11]:

- The location of the intervention and its relation with the structure behavior;
- Indoors or outdoors;
- Structural requirements for the repaired element;
- Fire resistance requirements for the repaired area;
- Access conditions to the local intervention;
- Preservation of the structure original identity;

- Intervention costs.

3.4.1 - Timber-Timber repairs

If the decay no longer permits the element function performing but the localized repair is still achievable, it is possible to apply face or patch repairs and whole section repairs otherwise the replace of the entire components is required. In any of these cases, if possible, the shape and construction techniques should be the same as from the original pieces [1,2].

When only the surface is decayed, a face repair, cutting the decayed timber to a sound surface and replacing it for new timber face, can be applied. Considering often dissimilar host movement and new timber even when well fitted, they become over time more and more visual intrusive and create water traps what consequently timber decay, therefore it is necessary to take that in consideration and avoid water contact [1,2].

When a component decay is widespread to the point, where the removal and replacement of deteriorated timber is necessary until sound one is reached in the whole transversal section part, it is important to know the magnitude of the stresses the connections will be subjected to, so suitable joints between the old and new parts can be shaped. No matter how good workmanship and selection of material are, these connections will not be as strong as the original not joined member. These connections can be traditional, originally made to form long continuous members with the junction of two, or others specifically made to meet the requirements on conservative repair [1,2].

According to Brites [11], another common intervention technique, when the structure requires reinforcement, is to increase the elements transversal section (Fig. 3.21), throw new timber pieces junction in the lateral or superior and inferior surfaces. The connections between new and old timber can be made by tapping screws, bolts that cross both, new and old timber, nails or epoxy resins. The EC5 demonstrates the dimensioning method for composed sections, allowing to calculate the stiffness according to the material and connectors type. These techniques although reversible and with few losses of historic fabric, can be unaesthetically.

Figs. 3.27 to 3.33 show some timber-timber repair examples. The first and second ones (Figs. 3.27 and 3.28) are the replacements of a section of a deteriorated timber element by a new one, using only timber technology. In Fig. 3.29 are examples of upper, lower and lateral transversal section increment. In Fig. 3.30 are some examples of metallic connections to join new and old timber. Fig. 3.31 shows a connection which thanks to its squinted section, resists bending moments; its threshold to avoid the splitting along the grain; the fastenings to increase strength and stiffness of joint and resists shear stresses; and folding wedges to tight the connection avoiding gaps. It is designed to resist tensile, bending and compression stresses. Fig. 3.32 shows a simpler connection with a straight section and fastenings to resist tension and compression. Fig. 3.33 shows a scissor scarf joint used in compressed columns, in order to resist to lateral movements.

Fig. 3.27 – Timber to timber repair in situ.

Fig. 3.28 – Old – new timber to timber connection.

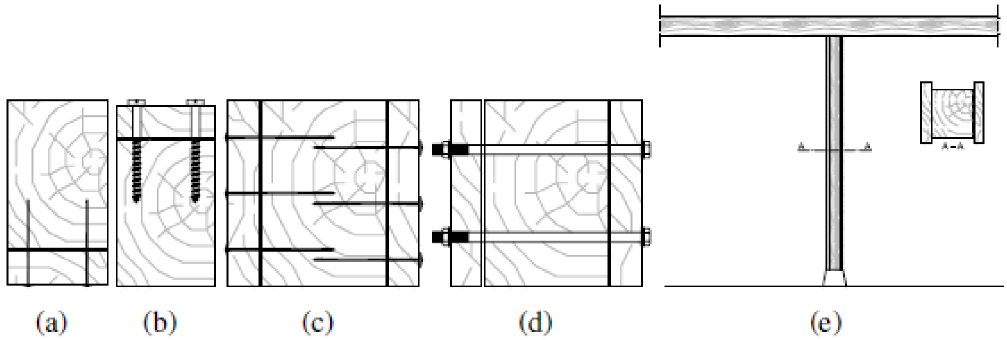


Fig. 3.29– Examples of transversal section increase – reinforcement. a) nailed connection; b) tapping screws connectors; c) nailed connection; d) bolt connectors; e) column reinforcement; [11].

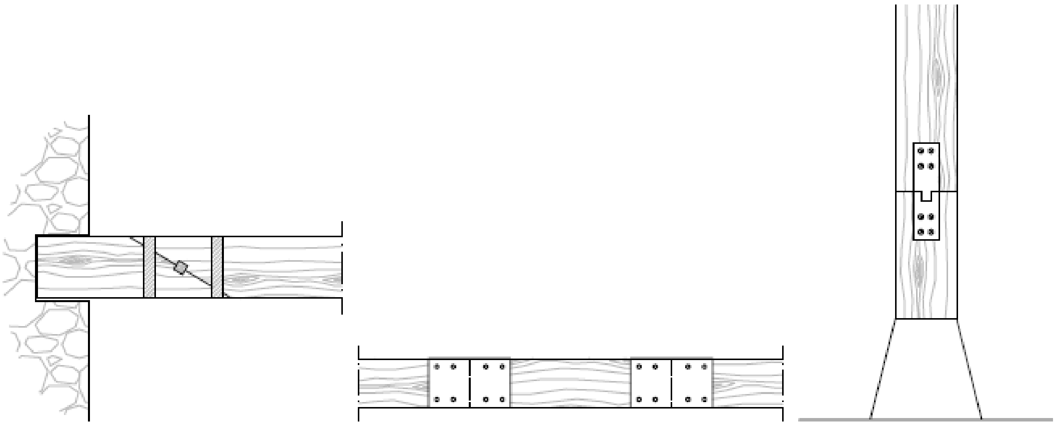


Fig. 3.30 – Examples of section replacement, using metallic connections [11].

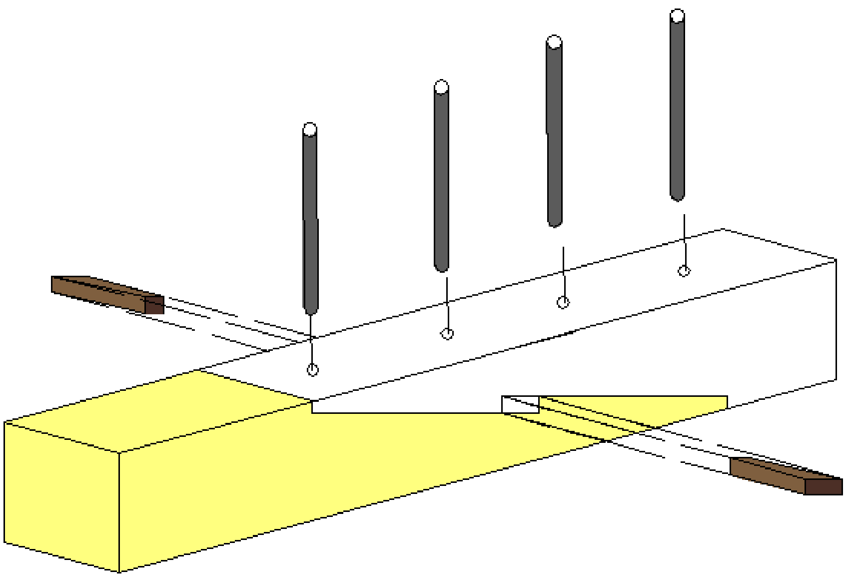


Fig. 3.31 – Example of connection to resist tensile, bending moments and compression. (based on [1])

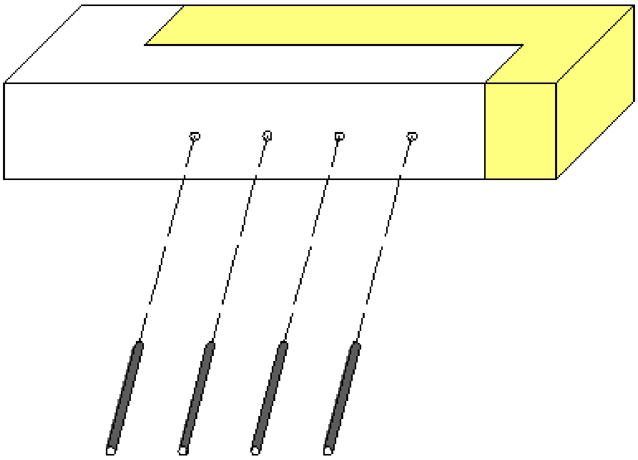


Fig. 3.32 – Example of connection to resist tension or compression.
(based on [1])

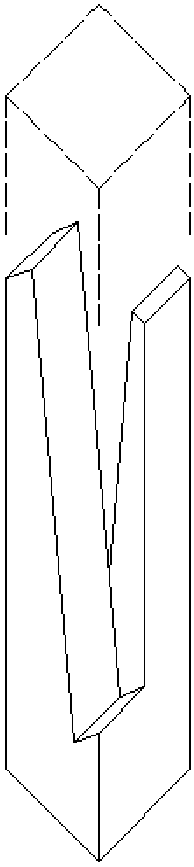


Fig. 3.33 – Scissor scarf joint. Used in compressed columns, resisting lateral movement.
(based on [1])

Selection of timber

According to Ashgate Publishing Limited [1] the timber used for reparation works should match as close as possible to the original one in class quality (or better), specie, grain orientation, moisture content (it should be limited to 15%), characteristics growth and section orientation (it should be cut from the same cross-sectional position as the timber which will be fixed) (Fig. 3.34). The authors of book [1] even claim that if possible, second hand sound timber, which had an opportunity to age under similar circumstances to the one being repaired, is the best choice.

These recommendations are not only to maintain the original content or to preserve the heritage values but also to make sure that both, old and new timber, will have a similar movement with moisture changes or external actions.

Fig. 3.34 – Face repair [1].

3.4.2 - Steel repairs

Allows to carry out local repairs, strengthens the actual structure or provides a new one to carry the excessive or all the loads (except the ancient structure own weight). The metallic interventions can be reversible and involve repairs with minimal loss of historic fabric permitting deteriorated timber not to be removed [1].

The visual impact of the steel components should be taken into consideration and sometimes even if the most viable intervention demands these components, to reinstate the joint capacity using for instance small angle cleats, or to strength elements using components like flitch plates, should be as much as possible concealed in the structure when the work is completed. Simple components such as plates, angle or channels can be fixed alongside the standing timber, subjected or not to its shape, as well as completely new structures next to the old ones like wire steel ropes, requiring good design and specialized workmanship [1].

When the deformation or the bending moment stresses in the elements are too high, it is possible to reinforce them with steel cables, inducing a precamber to the structure or just diminish its bending moments, introducing loads in the opposite direction of the structure deformation. [11]

Figs. 3.35 to 3.38 show some examples of steel usage in timber repair. The first one (Fig. 3.35) is a flitch plate intervention, the second one (Fig. 3.36) are some reinforcements in the connection between elements, the third (Fig. 3.37) is a structural reinforcement of the structure with additional metallic elements and the last one (Fig. 3.38) represents structural reinforcement through metallic cables.

Fig. 3.35 – Flitch plate intervention [1].

Fig. 3.36 – Use of traditional steel elements to improve the connection between timber elements [37].

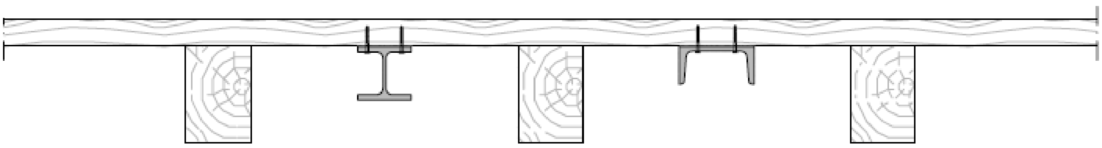
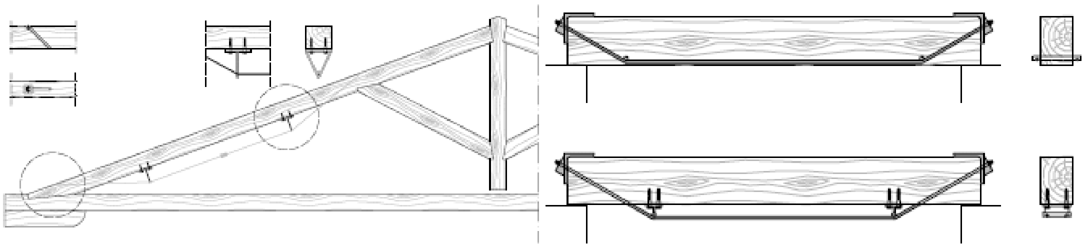


Fig. 3.37 – Reinforcement with additional steel structures [11].



(a) (b)

Fig. 3.38 – Reinforcement with metallic cables [11].

3.4.3 - Synthetic resins

Allow to control the mechanical properties like modulus of elasticity, the viscosity and hardening velocity, it can fix defects with less disturbance and loss of historic fabric than timber to timber repairs which involves removal of parts of elements, conserving noteworthy features which may be present in or on the surface of a decayed timber element, as well as a lower visual impact [1].

They are mainly used to repair beam ends, consolidating decayed timber and replacing missing one using reinforcing connectors of stainless steel or FRP composites, but can also be used as old-new timber connection “glue” or timber section filling.

The FRP’s although materials with excellent mechanical properties [11], cannot be used as in concrete reinforcement, because their non-reversibility is not acceptable by historic material preservation ideals. Nevertheless, Nowak et al. [54] created some original solutions where the CFRP was inserted into the cross sections, which can be (according to the authors) suitable for historic timber building structures.

Knowing that above 40ºC the strength and stiffness of these materials can decrease [1], it is necessary to take in consideration the place and conditions of the intervention. Another thing to be aware of when using resins in connections, is to not change from a hinged or semi-hinged joint to a rigid one, changing the structure behavior.

These interventions should always be carried out by skilled tradesmen, capable of using both traditional and new techniques in an intelligent and effective way and not as a cheap alternative to timber to timber connections that could lead to weaker, ineffectual solutions or less durable when inadequate accomplished [1].

Figs. 3.39 and 3.40 show two examples of synthetic resins usage in timber repairs. The first one is the edge of one element being reinforced with epoxy resin and metallic rods and the second one is a joint intervention, connecting the new and the old timber.

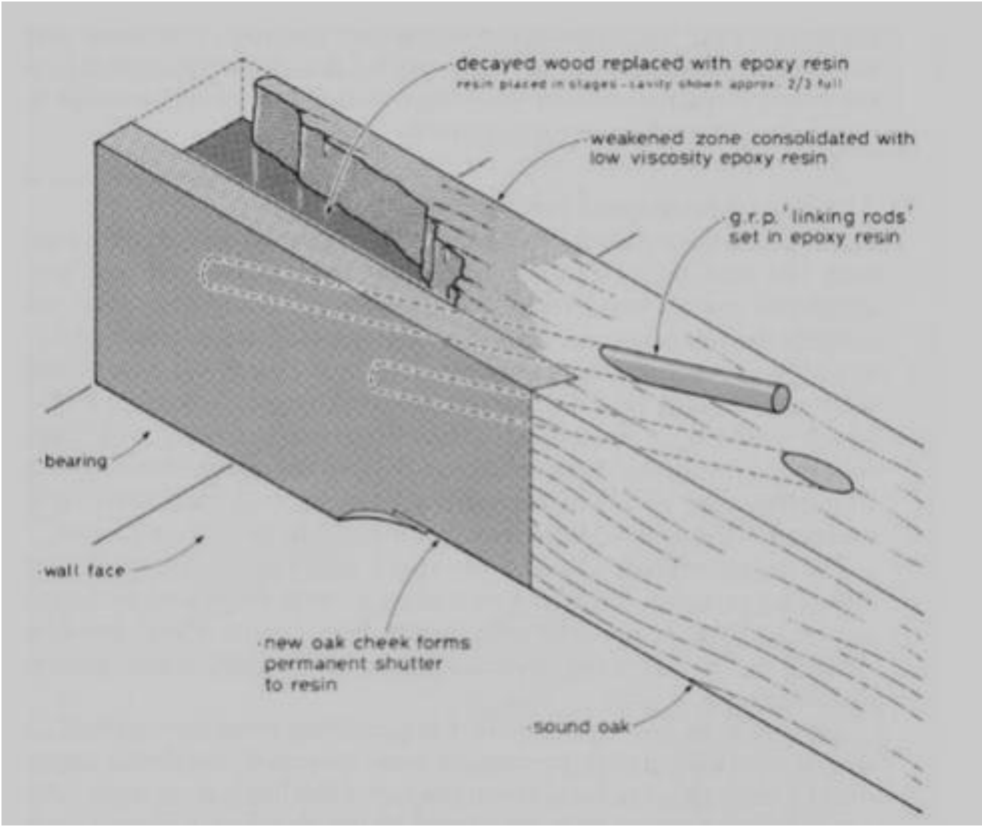


Fig. 3.39 – Timber strengthening example with epoxy resin and metallic rods [32].

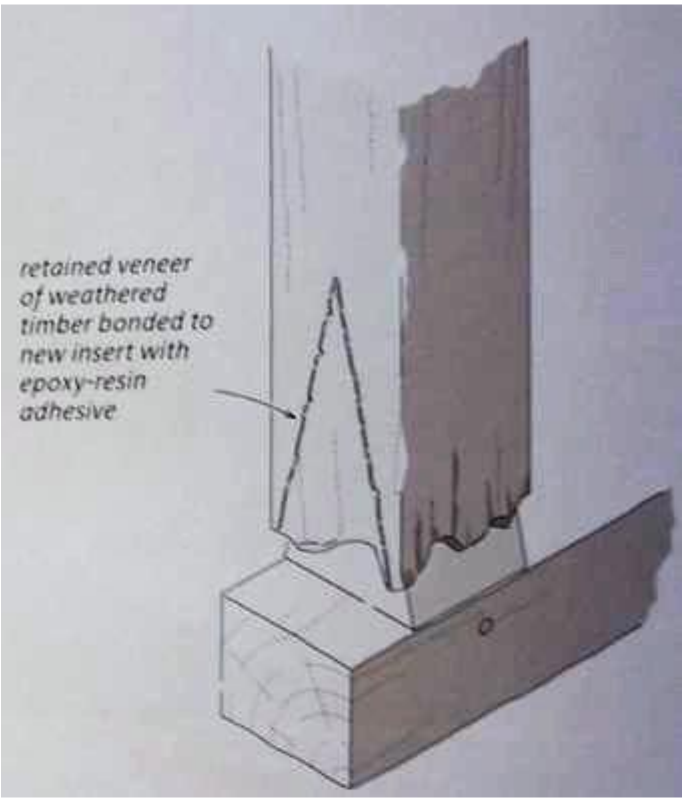


Fig. 3.40 – Joint intervention – New and old timber connected with epoxy resin [1].

4 - Case study – Nave of Huedin Reformed Church

In this chapter a case study is accomplished regarding historic timber roof structures, adopting within the possible referred methodology presented in chapter 3.1.6.

The chapter starts with a brief history of the Huedin Reformed Church, posteriorly a structural survey is presented along with the roof deterioration state. The action values acting on the roof according to EC1 and the combination of actions adopted are presented as well as the modeling considerations and the structure reliability analyzed using the EC5 Ultimate Limit States. The structural function of the different elements is also done to both symmetrical and asymmetrical loads. In order to see the importance and consequences of the connections and material state, the structure without missing elements is modeled. According to the structural analyses some interventions and a maintenance plan are proposed.

4.1 – History

Huedin lies in a popular location (Fig.4.1) next to the Bologa Castle and it became the capital of Țara Călatei due to its location and region of particular ethnographic character [56].

First mentioned in 1332 by written sources, the ancient parish church dedicated to Saint Elizabeth of the House of Árpád in 1235, which resulted after transformations, carried out over the years, into nowadays church (Fig. 4.2 and 4.3) [56].

Tower's inferior levels were built in the gothic period and its four-pinnacle roof with a porch was created in the 18th century. On the level of the nave the medieval times are only reflected by the gothic door of the stairway taking to the gallery. All of the rest of the elements were destroyed in the 17th century and rebuilt later (Fig. 4.4). The lintel of the southern baroque entrance has a written name of the 1772 reconstruction leader Franz Bamer [56].

The painted ceiling (Fig. 4.5 and 4.6) - the biggest heritage significance of the church - was created in 1705 due to the extension of the nave to the south. The immense gothic chancel was probably built before 1483. In the 18th century, on the northern side a crypt was built in the place of sacristy.

The original gothic vaulting of the shrine was replaced with the painted ceiling created in 1780, after the earthquake in 1765. The ceiling represented variations of late renaissance elements. The artwork on the ceiling was created by Lörinc and János Umling [56].

In 1970 the stone pulpit was placed there as it is seen today. Before it stood in the reformatory church of Dragu. It was created by late renaissance master, Dávid Sipos, in the baroque period, around 1750 [56].

The crypt under the shine was created before 1764 on demand of Farkas Bánffy and his wife, Erzsébet Bagosi. The rococo inscription ornamented with their coats-of-arms was the creation of famous sculptor Antal Schuchbauer from Cluj-Napoca [56].

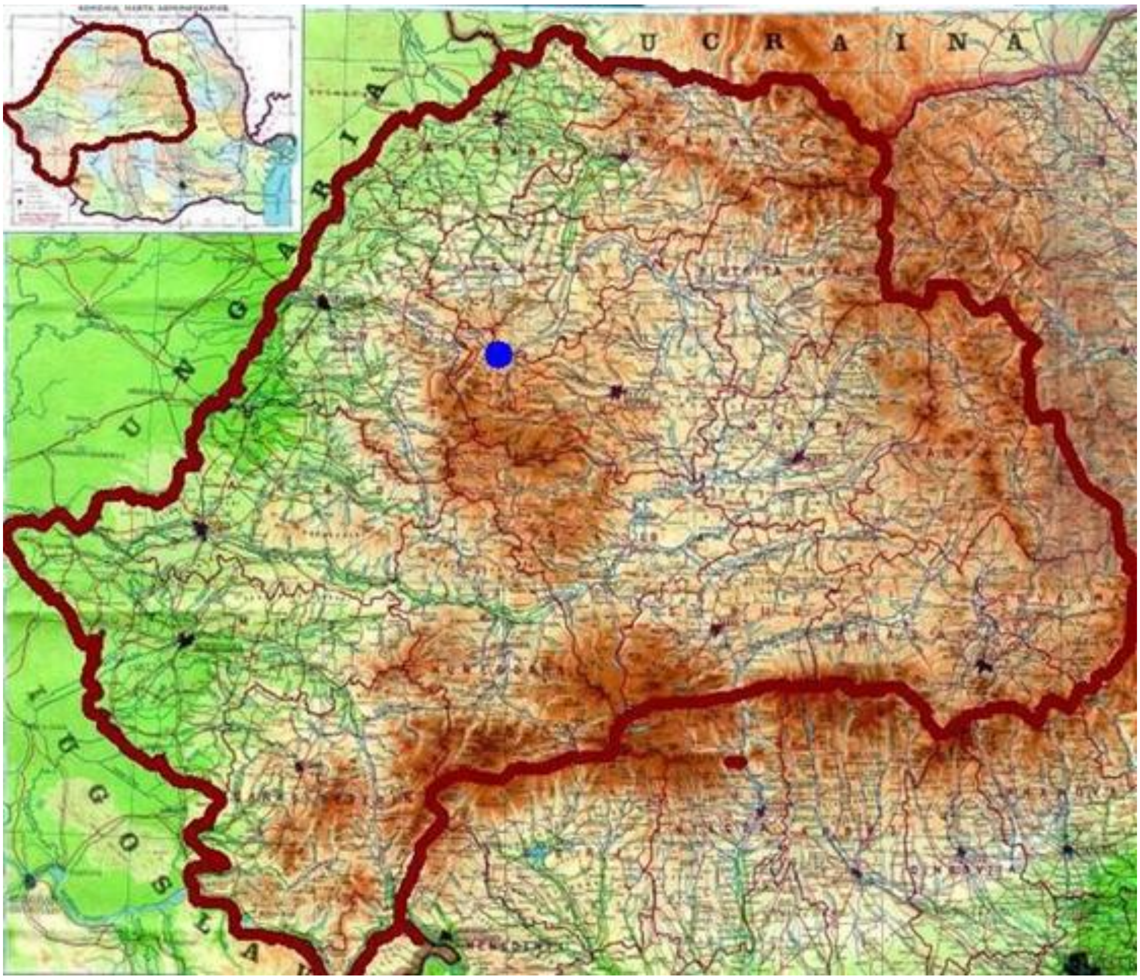


Fig. 4.1 – Huedin Location [57].


Fig.4.2 – South side.

Fig.4.3 – Church nave, north side.

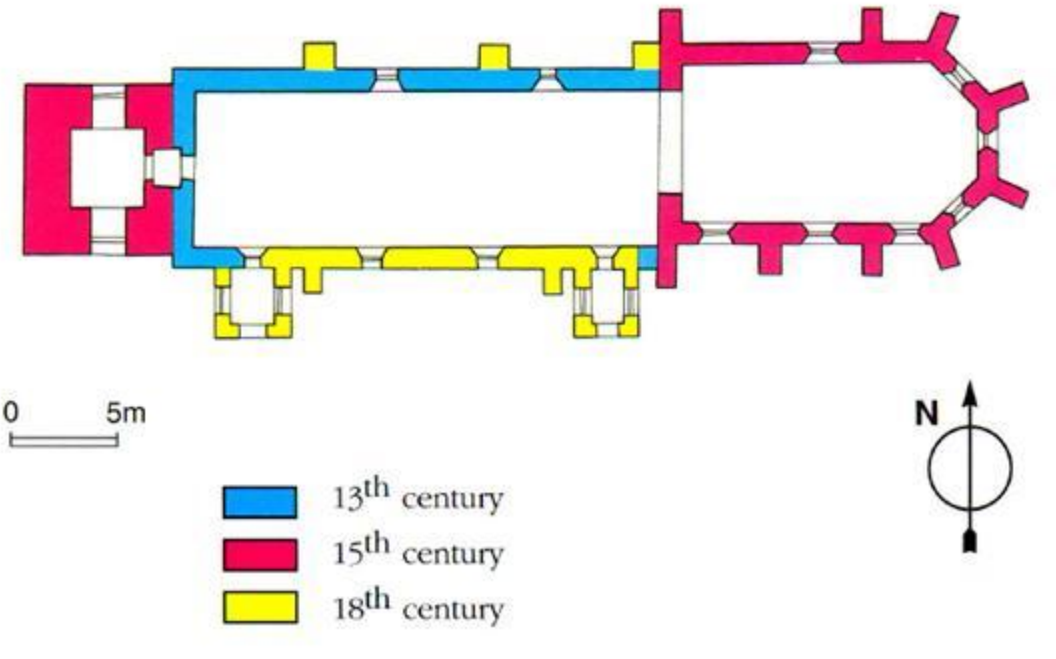


Fig. 4.4 – Church ground plan construction dates
(based on [56])

Fig.4.5 – The nave and the painted paneled ceiling.

Fig. 4.6 – Painted paneled ceiling details.

4.2 - Site survey

4.2.1 – Structural geometry

With a Gothic Character, made of softwood more precisely fir (*Abies Alba*) with a pitch of approximately 58º (Figs. 4.7 to 4.10), the structural configuration of the roof in term of transversal frames is constituted by 27 trusses in total. They are marked from 1 to 27 in Fig. 4.11 from west to east (the nave entrance is from the tower located on west side) and divided in two types, main (M) and secondary (S) trusses. The order of these transversal trusses are two secondary between principal ones (M-S-S-M-S-S-M) except between the last principal truss (27) and the one before last (25) where only one secondary truss is placed.

The common elements in the main and the secondary trusses are rafters, tie beam, collar beam, compound rafters, angle braces, and sprockets. Principal frames, besides mentioned elements, also have a king post and upper and lower knee braces. Three different types of main trusses were identified, the first one with a simple king post ending in the connection with the collar beam and with the compound rafters all the way until their interception with the rafters (MT1 - Fig.4.7), the second similar to the first one but with a double king post (MT1’ - Fig.4.8) and a third one with compound rafters that finishes in the interception with a double king post (MT2 - Fig.4.9). There were recorded 17 original designed secondary trusses (ST - Fig.4.10), 4 main trusses type 2 (trusses number 1, 7, 19 and 25), 4 main trusses type 1 (trusses number 4, 10, 17 and 22) and 2 main frames type 1’ (trusses number 13 and 27). The original design of transversal main frames begins and ends with a double king post truss and between one of these two types of trusses (MT1’ and MT2) there is one with a simple king post (MT1).

The longitudinal bracing system (Fig. 4.11), disposed in vertical on the symmetry axis of the structure, is composed by angle braces and passing braces, lower and upper plates connected to the king post, that is also part of this system, and together they make the connection between the longitudinal and transversal system (see structural layouts in Figs. 4.12 and 4.13). The laths (Fig. 4.14) that make the connection from the rafters to the shingles also contribute to the longitudinal bracing system.

The transversal actions are transmitted from the roof structural elements, through the tie beam to one wall-plate on the south (presumably an alteration made on the wall that was rebuild after the earthquake of 1765) that on its turn transfer the loads to the bearing wall, and to the north side where the tie beams are directly embedded in the bearing wall. The two adjacent parts of the church, the tower and the chancel, together with the bearing walls, are responsible for the longitudinal actions passage to foundations.

The found joints were notched and lapped with wood pegs and presumably mortice and tenon without wooden peg. The notched joints are present between tie beam and wall plate. Mortice and tenon between rafter and tie beam and the rest of the connections between elements are with lapped joints built squinted (dovetail in this case) in connections with element endings and built straight (half lapped) for intermediary joints. The wooden pegs had a quadrangular or hexagonal shape, slightly larger than the approximate 2,5cm circular hole where they were forced to fit in order to guarantee the connection between elements. Further details about joints are made in chapter 4.5.

Figs. 4.7 to 4.11 and Table 4.1 present the configuration of the structure and element considered dimensions (for more details, see Appendix 2 and 3). Figs. 4.12 and 4.13 show the structural 3D perspective in situ

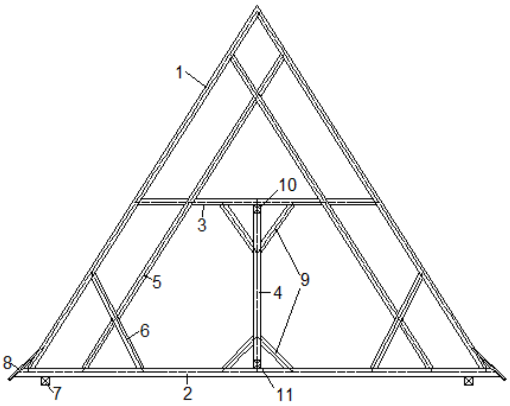


Fig. 4.9 – MT2

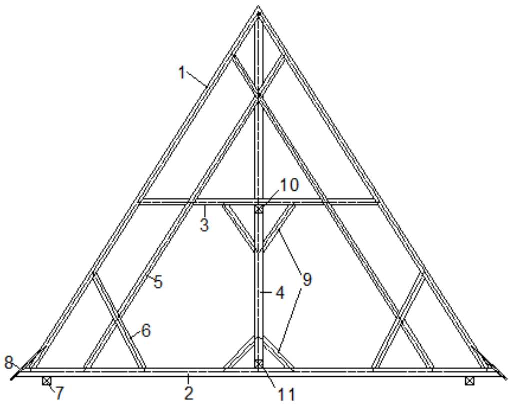


Fig. 4.10 – ST

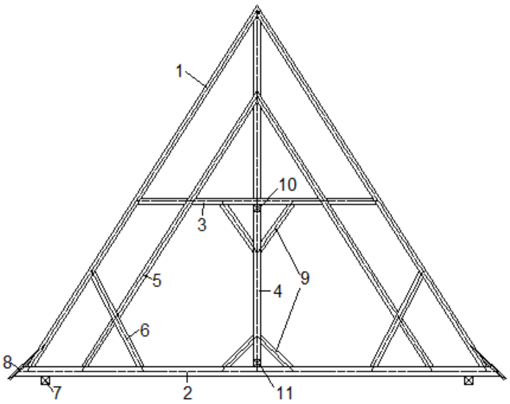


Fig. 4.7 – MT1

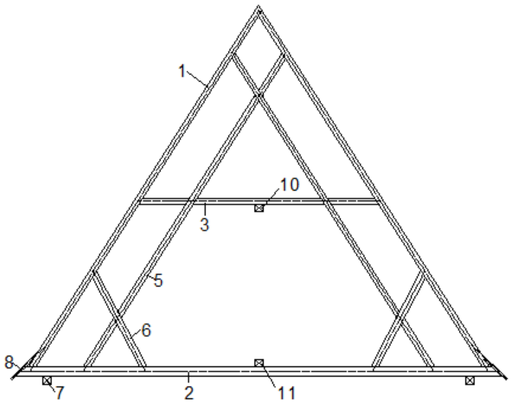


Fig. 4.8 – MT1’

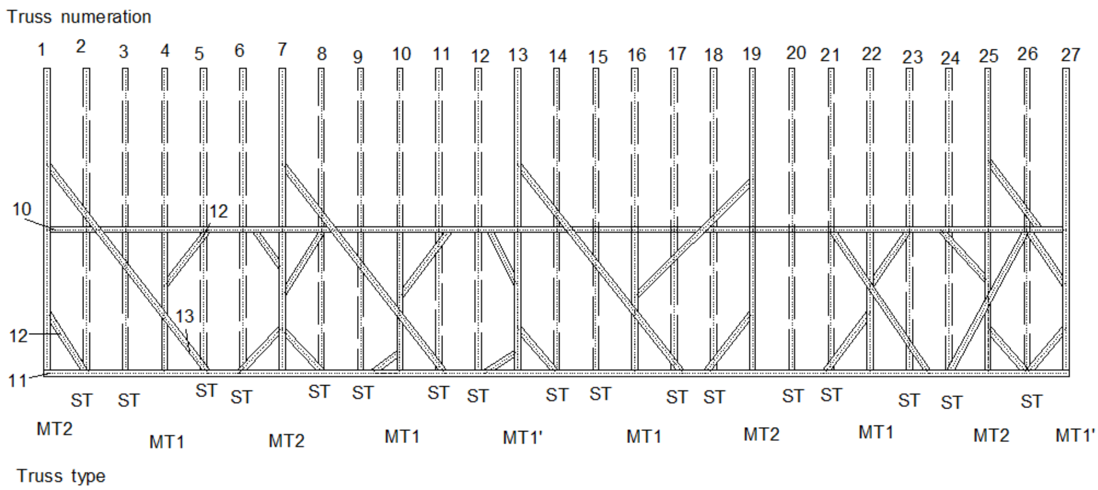


Fig. 4.11 – Longitudinal bracing system

Number	Element	Transversal Dimension (b x h) [cm]
1	Rafter	11,5 x 13
2	Tie beam	17 x 17
3	Collar beam	12 x 13
4	King post	15 x 14
5	Compound rafter	11,5 x 11
6	Angle brace	12,5 x 11,5
7	Wall plate	17 x 18
8	Sprocket	5 x 8,5
9	Knee brace	12 x 12
10	Upper plate	13,5 x 13
11	Lower plate	15 x 16,5
12	(Longitudinal) Angle brace	12 x 12
13	Passing brace	14 x 14

Table 4.1 – Element dimension

Fig. 4.12 – Structural layout.

Fig. 4.13 – Tridimensional aspect in situ

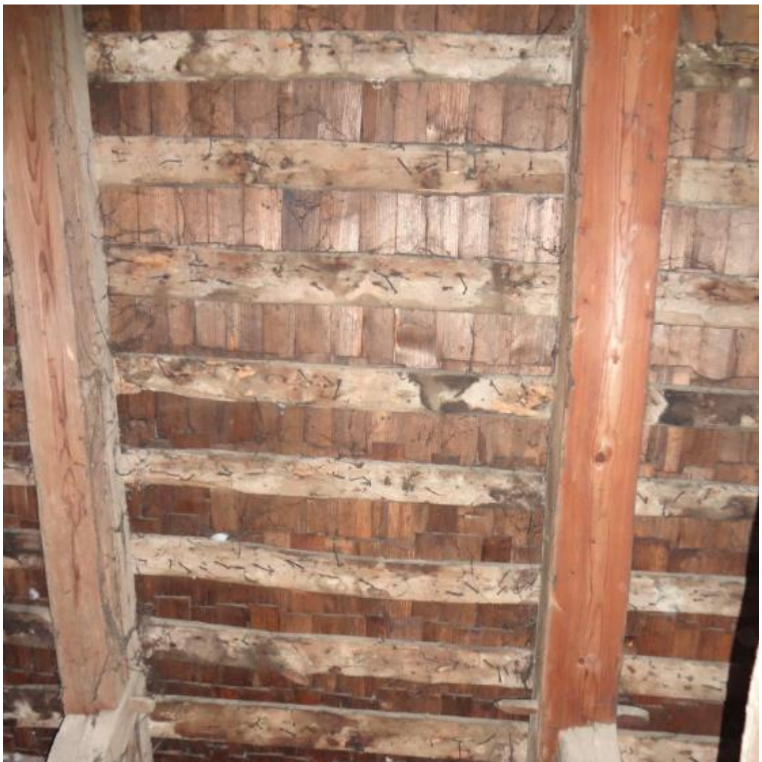


Fig. 4.14 – Laths nailed to rafters

4.2.2 - Roof deterioration state

On the visits to the roof interior the following was noticed:

a) Lack of one upper knee brace in the first truss (Fig. 4.15) for the roof entry reasons, and a small angle brace between the king post and Tie-beam, which for its deteriorated state, especially in the connection with the king post (Fig. 4.15), it is as it was nothing there;

b) The wall plate between the second and third trusses was much deteriorated (Fig. 4.16).

c) The wooden peg joining the longitudinal angle brace hold in the first truss and the lower plate was missing;

d) The wooden pegs on the rafters, especially on the north side were softer than the others what could had indicated that their moisture content is higher or/and that they can be decayed;

e) The shingles were in poor conditions, with some gaps and holes between them allowing water to come in and infiltrate the structure (Fig. 4.17 and 4.18)

f) The original longitudinal passing brace holding the first truss was replaced for a "new" one (Fig. 4.19);

g) It was visible wood dust in two locations of the structure, evidence from active insect attack (Fig. 4.20);

h) The structure was dirty with dust and cobwebs what affects the visual inspection and strength grading, hiding some local timber defects;

i) The first truss instead of one tie-beam has two, next to each other, that are both deteriorated, especially in the extremities (Fig. 4.21);

j) The tie beams on the south side in trusses number 2, 3, 10, 11, 15 were entrapped by one additional timber element in each side (Fig. 4.22);

k) The structure elements have longitudinal cracks (Fig. 4.23) most likely due to the wood shrinkage, since a lot of historic timber structures were made with green wood for its better workability;

l) Some elements although still connected by wooden pegs have "big" gaps between them (Fig. 4.24 and 4.25).

In order to have a better acquaintance of these problematic areas two drawings with their location were made, in Fig. 4.26 for a transversal perspective and Fig. 4.27 for a longitudinal perspective. The letters presented in these Figs. correspond to the letters above explaining the different roof conditions found on the visits.


Fig. 4.15 – Missing upper knee brace and deteriorated angle brace connection.


Fig. 4.16 – Deteriorated wall plate.

Fig. 4.17 – Shingles in bad condition.

Fig. 4.18 – Shingles in bad condition – detail.


Fig. 4.19 – Replaced longitudinal passing brace.


Fig. 4.20 – Wood dust from insect attack.

Fig. 4.21 - Both tie beams are deteriorated in the first truss, especially in the extremities.

Fig. 4.22 – Tie beams entrapped by one additional timber element on each side.


Fig. 4.23 – Longitudinal cracks in the structural elements.


Fig. 4.24 – Gap between rafter and angle brace.

Fig. 4.25 – Gap in upper plate extension joint.

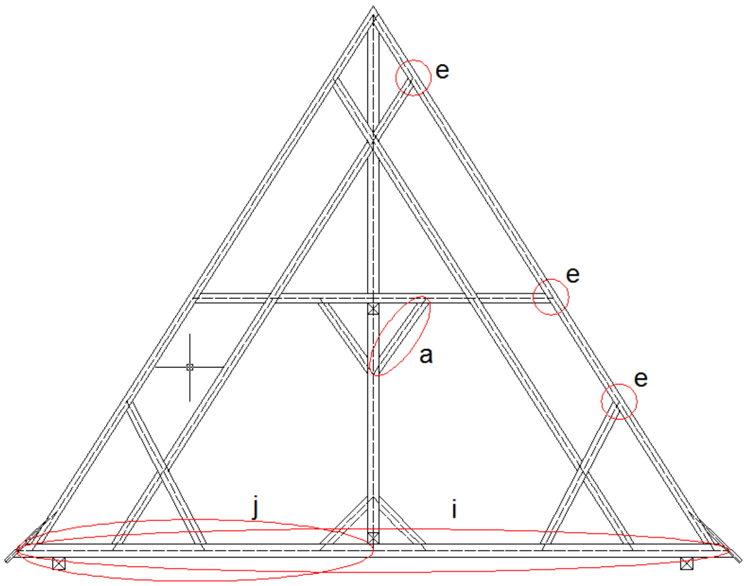


Fig. 4.26 – Problematic areas map – Truss.

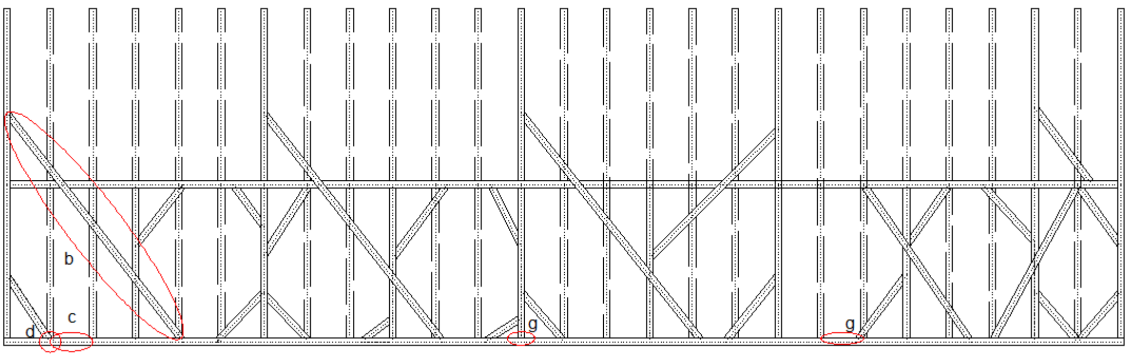


Fig. 4.27 – Problematic areas map – Longitudinal bracing system.

4.3 - Actions

4.3.1 - Permanent actions

The permanent actions in the structure are its self-weight, shingles, laths and the ceiling that is connected to the tie beam. The Romanian standard STAS 10101/1-78 indicates a load of 0.4kN/m^2 for both shingles and laths, however in this work it was considered a 0.5kN/m^2 weight for the shingles and C30 (ρ=4.6kN/m^3) laths distributed from 30 to 30cm horizontally with 3x5cm^2, being not only closer to the reality but also on the safe side. The wood ceiling was considered 0.6kN/m^2 .

4.3.2 - Imposed load

For roofs, separate verifications shall be performed for the concentrated load Q_k and for the uniformly distributed load q_k, acting independently.

With a not accessible roof (Category H – see Table E.1 of annex 2) and with a slope of approximately 58º (pitch bigger than 1:20) the q_k is considered 0.5kN/m^2 (see Table E.2 – Annex 2) [61]. Because repairs can be done from the inside of the roof, the Q_k is not considered. Nevertheless when comparing the concentrate load of 1kN (worst case for the concentrated load – see Table E.2 – Annex 2) and the distributed load of 0.5kN/m^2, shows that the distributed load is the most prejudicial.

4.3.3 – Snow

The snow loads on roofs for the persistent / transient design situations shall be determined as

$$s = \mu_i * C_e * C_t * s_k \tag{4.1}$$

Where

μ_i is the snow load shape coefficient;

s_k is the characteristic value of snow load on tile ground;

C_e is the exposure coefficient;

C_t is the thermal coefficient;

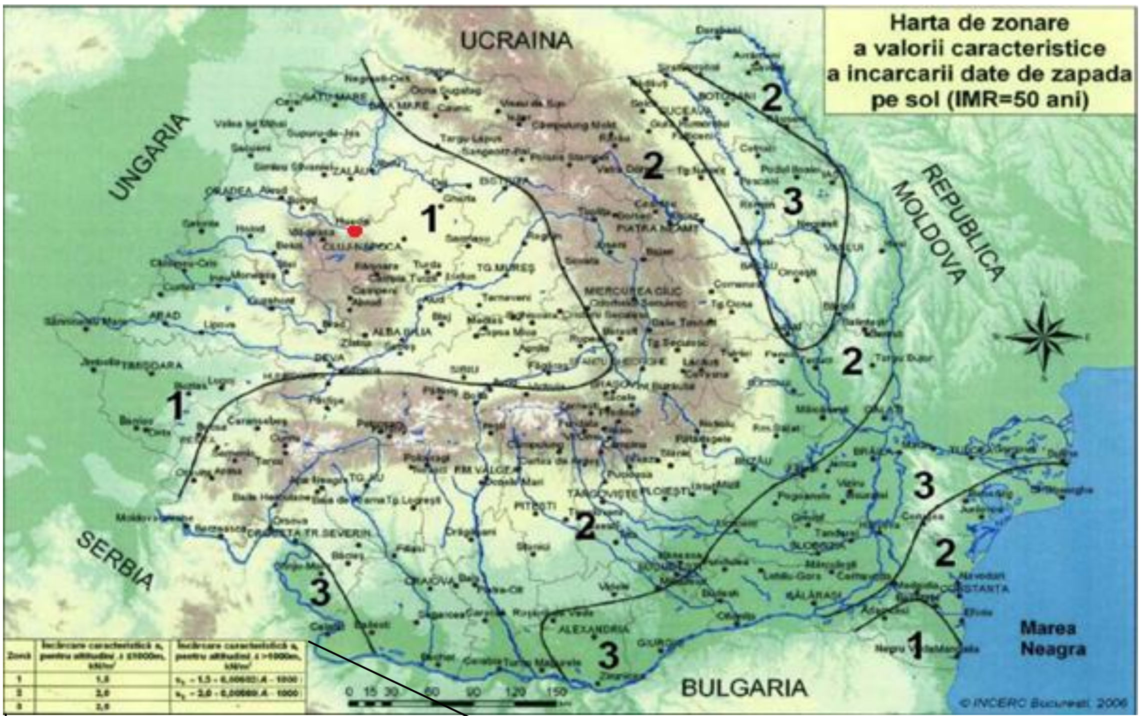
For pitched roofs, according to Figure E.2 of Annex 2, it should be considered the non-drifted load arrangement, case (i), and the drifted load arrangements, cases (ii) and (iii), unless otherwise specified for local conditions. In this study only the non-drifted load type was considered, since the rafters connections are hinged and therefore the roof with non-drifted load is the worst case. Considering this, the only issue would be related with the wind suction, lifting the structure, however that situation is considered in some of the combinations (see chapter 4.4) where the snow loads are not considered at all.

The snow load shape coefficient $\mu1$ is according to table E.4 of Annex 2

$$\mu1 = 0.8 * \frac{60-\alpha}{30} \tag{4.2}$$

Where α is the roof pitch. It leads to a $\mu1 = 0.0652$.

Located in the area 1 according to the Romanian national annex [62], the characteristic value of snow load on tile ground (s_k) is 1.5kN/m^2 (see Fig. 4.25).



Zonă	Încărcare caracteristică s_k pentru altitudini $A \le 1000$m, kN/m^2	Încărcare caracteristică s_k pentru altitudini $A > 1000$m, kN/m^2
1	1,5	$s_k = 1,5 + 0,00602(A - 1000)$
2	2,0	$s_k = 2,0 + 0,00580(A - 1000)$
3	2,5	-

Fig. 4.28 – Romanian national annex map for characteristic value of snow on the ground with Huedin location enlightened [62].

Considering the topography as normal (see Table E.3 of Annex 2), the C_e is 1.0 and not considering the roof with high thermal transmittance (see Annex 2 - Snow), the C_t is 1.0.

From the expression 4.1, the snow loads on roofs is

$$s = 0.0652 * 1.0 * 1.0 * 1.5 = 0.0978 kN/m^2$$

With such a small roof snow load derived from the high pitch, the local effects and snow load shape coefficients for exceptional snow drifts were declined and instead of them and according to the national annex [61] (Table E.2 – Note 2), the imposed load of 0.5kN/m^2 was used as snow load.

4.3.4 – Wind

The wind pressure acting on the external surfaces should be obtained from expression

$$w_e = q_p(z_e) * c_{pe} \quad (4.3)$$

Where

$q_p(z_e)$ is the peak velocity pressure;

z_e is the reference height for the external pressure;

c_{pe} is the pressure coefficient for the external pressure;

The process to get to this pressure is to get the basic wind velocity from Eurocode (national) annexes, calculate the Basic wind velocity then the Mean wind, Wind turbulence and with the last two the Peak velocity pressure (see Annex 2 – Wind). The pressure coefficients are given in the section 7 of EC1-4 for some series of generic cases (see Fig. E.7 and Tables E.7 and E.8 for the study case in Annex 2).

<u>The basic wind velocity</u>

The basic wind velocity is defined by

$$v_b = c_{dir} * c_{season} * v_{b,0} \quad (4.4)$$

v_b – is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category II;

c_{dir} – is the directional factor. The recommended value is 1,0 unless specified contrarily in national annex;

c_{season} – is the season factor. The recommended value is 1,0 unless specified contrarily in national annex;

$v_{b,0}$ – is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category II;

According to the Romanian national annex [63], the basic wind velocity for Huedin (Cluj-Napoca) is 27m/s (Fig. 4.29) and the c_{dir} and c_{season} are both 1.0. Therefore according to expression 4.4 $v_b = 27\ m/s$.

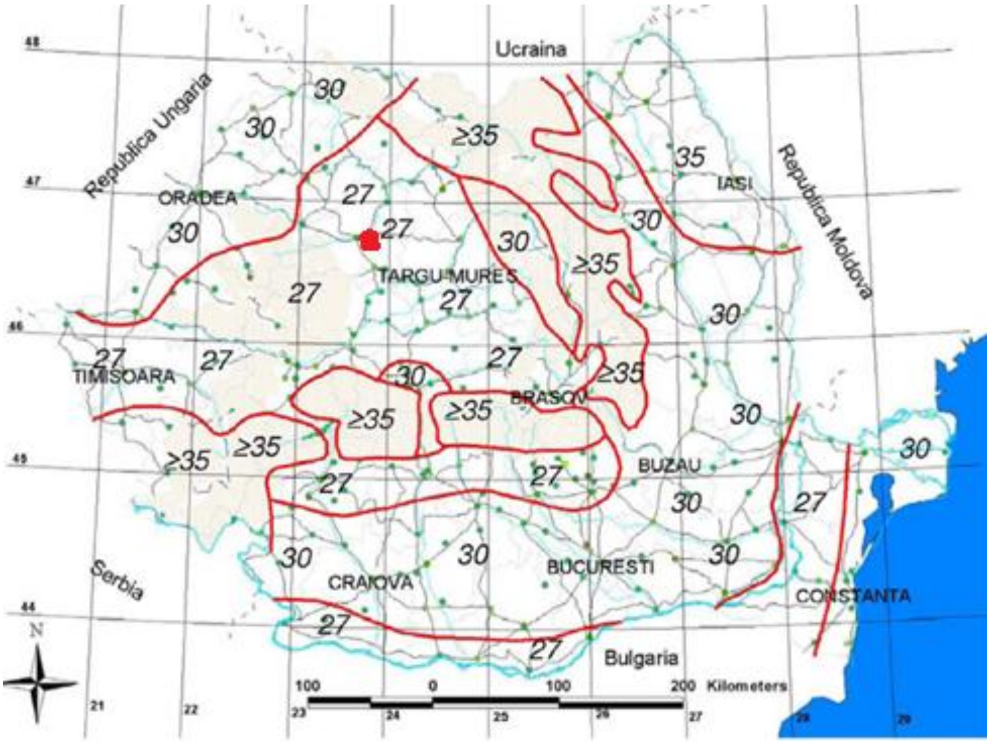


Fig. 4.29 – Romanian wind national annex with Huedin enlightened [63].

Mean wind

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, v_b, and should be determined using the following expression

$$v_m(z) = c_r(z) * c_o(z) * v_b \tag{4.5}$$

Where

c_r is the roughness factor;

c_o is the orography factor;

$$c_r(z) = k_r * \ln(\frac{z}{z_0}) \qquad \text{for } z_{min} \leq z \leq z_{max} \tag{4.6}$$

$$c_r(z) = c_r(z_{min}) \qquad \text{for } z \leq z_{min} \tag{4.7}$$

Where

z_0 is the roughness length;

k_r is the terrain factor depending on the roughness length z_0 calculated using

$$k_r = 0.19 * \left(\frac{z_0}{z_{0,II}}\right)^{0.07} \tag{4.8}$$

$z_{0,II}$ – 0.05 m (terrain category II);

z_{min} - is the minimum height defined in Table E.6 from the Eurocode 1991-1-4;

z_{max} - is to be taken as 200 m, unless otherwise specified in the National Annex;

Located in a suburban area, the terrain category of Huedin church was considered III, meaning z_0=0.3m and z_{min}=5m (see Annex 2 - Table E.6), the orography factor 1.0 (see Annex 2 – wind). With these values and with a building height (z) of 16.38 m, it is possible to determine the roughness factor $c_r(z)$ and subsequently the mean wind velocity.

$$k_r = 0.19 * \left(\frac{0.3}{0.05}\right)^{0.07} = 0.2154$$

$$c_r = 0.2154 * \ln\left(\frac{16.38}{0.3}\right) = 0.8616$$

$$v_m(z) = 0.8616 * 1,0 * 27 = 23.26\text{m/s}$$

Wind turbulence

The turbulence intensity $I_v(z)$ at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity.

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_0(z)*\ln\left(\frac{z}{z_0}\right)} \quad \text{for zmin} \leq z \leq \text{zmax} \qquad (4.9)$$

$$I_v(z) = I_v(z_{min}) \qquad \text{for } z \leq \text{zmin} \qquad (4.10)$$

Where

k_I is the turbulence factor with the recommended value of 1,0 unless contrarily specified in the National annex;

With k_I = 1.0

$$I_v(z) = \frac{1}{1.0 * \ln\left(\frac{16.38}{0.3}\right)} = 0.25$$

Peak velocity pressure

The peak velocity pressure $q_p(z)$ at height z, which includes mean and short-term velocity fluctuations, should be determined through the following expression

$$q_p = (1 + 7 * I_v) * 0.5 * \rho * v_m^2(z) \qquad (4.11)$$

Where

ρ is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms; the recommended value should be 1.25 kg/m^3 or indicated in National annex.

From expression 4.11

$$q_p = (1 + 7 * 0.25) * 0.5 * 1.25 * 23.26^2 * 10^{-3} = 0.9299\ kN/m^2$$

<u>Pressure coefficients for buildings</u>

The external pressure coefficients c_{pe} for buildings and parts of buildings depend on the size of the loaded area (A), which is the area of the structure that produces the wind action in the section to be calculated. The external pressure coefficients are given for loaded areas A of 1 m^2 and 10m^2 in the tables for the appropriate building configurations as $C_{pe,1}$, for local coefficients, and $C_{pe,10}$, for overall coefficients, respectively.

Values for $C_{pe,1}$ are intended for the design of small elements and fixings with an area per element of 1 m^2 or less such as cladding elements and roofing elements. Values for $C_{pe,10}$ may be used for the design of the overall load bearing structure of buildings.

For loaded areas above 1 m^2 and up to 10 m^2, the procedure for calculating external pressure coefficients is based on the following expression (see Annex 2 - Fig. E.5)

$$c_{pe} = c_{pe,1} - \left(c_{pe,1} - c_{pe,10}\right) * \log_{10} A \qquad (4.12)$$

According to Fig. E.7 and Tables E.7 and E.8 of Annex 2, was possible to determine the areas dimension in Figs. 4.30 and 4.31 and the correspondent pressure coefficients in tables 4.2 and 4.3.

For the wind direction θ=0º (south - north)

$$e = \min \begin{cases} b = 23.4 \text{ m} \\ 2h = 2 * 16.38 = 32.76m \end{cases}$$

Where

e is the edge distance;

b is the crosswind dimension;

h is the ridge height;

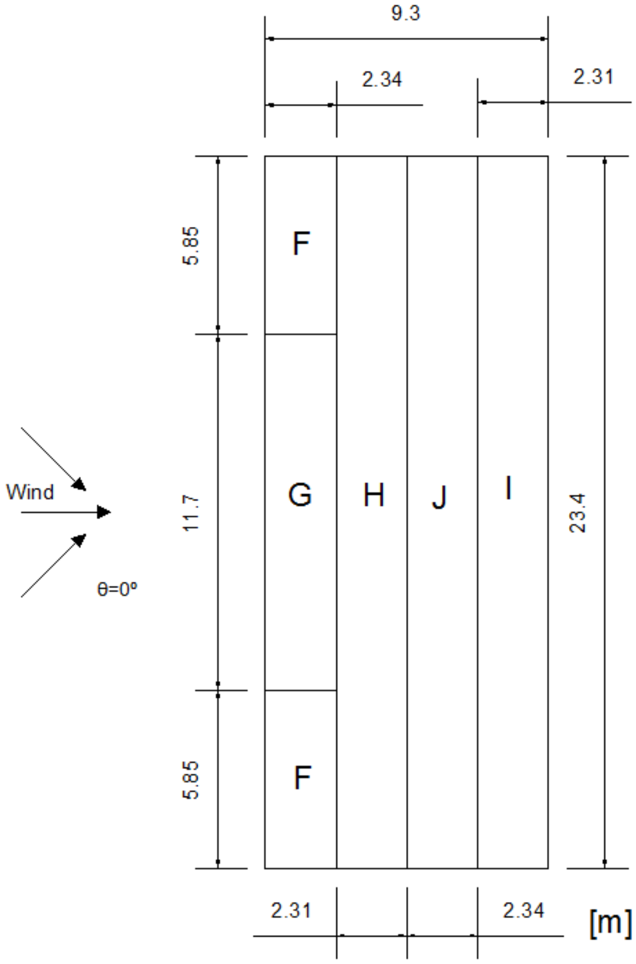


Fig. 4.30 – Areas for wind direction south – north.

All section areas are clearly over $10m^2$, for that reason the C_{pe} used is going to be the $C_{pe,10}$:

0º	F	G	H	I	J
ang	Cpe	Cpe	Cpe	Cpe	Cpe
45	0.70	0.70	0.60	0	0
60	0.70	0.70	0.70	-0.20	-0.30
57.56	0.70	0.70	0.68	-0.17	-0.25

Table 4.2 – Pressure coefficients for south – north wind direction.

For the wind direction θ=90º (East - West)

$$e = \min\begin{cases} b = 9.3\text{ m} \\ 2h = 2 * 16.38 = 32.76m \end{cases}$$

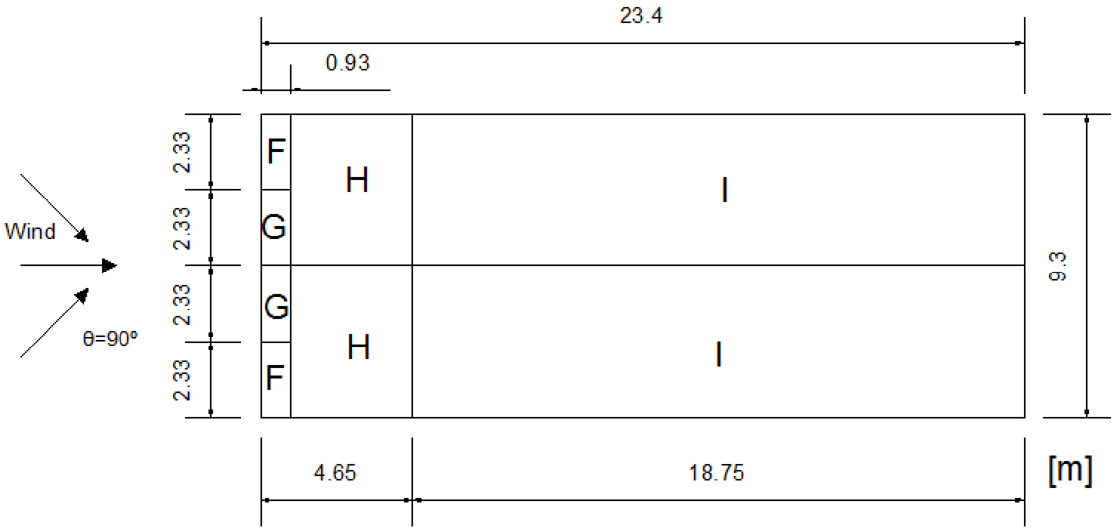


Fig. 4.31 – Areas for wind direction east – west.

In this situation the area of the sections must be calculated in order to know C_{pe}:

90º	F		G		H		I
ang	Cpe,10	Cpe,1	Cpe,10	Cpe,1	Cpe,10	Cpe,1	Cpe
45	-1.10	-1.50	-1.40	-2.00	-0.90	-1.20	-0.50
60	-1.10	-1.50	-1.20	-2.00	-0.80	-1.00	-0.50
57.56	-1.10	-1.50	-1.23	-2.0	-0.82	-1.03	-0.50
Area	2.16		2.16		17.30		-
Cpe	-1.37		-1.74		-0.82		-0.50

Table 4.3 – Pressure coefficients for south – north wind direction.

Wind pressure on surfaces

According to expression 4.3 the wind pressure on the roof surface is obtained (see Tables 4.4 and 4.5).

0º	F	G	H	I	J
Cpe	0.70	0.70	0.68	-0.17	-0.25
We [kN/m^2]	0.65	0.65	0.63	-0.16	-0.23

Table 4.4 – Wind pressure for south – north wind direction.

90º	F	G	H	I
Cpe	-1.37	-1.74	-0.82	-0.50
We [kN/m^2]	-1.27	-1.62	-0.76	-0.46

Table 4.5 – Wind pressure for south – north wind direction.

Internal pressure

The wind pressure acting on the internal surfaces of a structure should be obtained from expression

$$w_i = q_p(z_i) * c_{pi} \quad (4.12)$$

Where

$q_p(z_i)$ is the peak velocity pressure;

z_i is the reference height for the interior pressure;

c_{pi} is the pressure coefficient for the internal pressure;

Internal and external pressures shall be considered to act at the same time.

The internal pressure coefficient, c_{pi}, depends on the size and distribution of the openings in the building envelope (see Annex 2 – Internal pressure).

For buildings without a dominant face, the internal pressure coefficient c_{pi} should be determined from Figure E.8 in Annex 2, and is a function of the ratio of the height and the depth of the building, h/d, and the opening ratio μ for each wind direction θ, which should be determined from the following expression.

$$\mu = \frac{\sum area\ of\ openings\ where\ C_{pe}\ is\ negative\ or -0{,}0}{\sum area\ of\ all\ openings} \quad (4.13)$$

In both south and north part of the protruding roof there are equal openings as illustrated in Fig. 4.32.


Fig. 4.32 – Roof openings

These openings as exemplified in Eurocode (see Annex 2 – Fig. E.6) are subjected to the pressure at the underside found from the wall pressure (see Fig. E.9 and Table E.9 – Annex 2).

For wind directions of 0º there is h/d>1 and half of the openings with a positive value and the other half with negative values (μ=0.5), leading to a C_{pi}=0.114. For wind directions of

90º there is h/d=0.722 and all the openings with a negative value (μ=1.0), leading to a C_{pi}= -0.367 (see Fig. E.8 – Annex 2).

According to expression 4.12 the internal wind pressure for wind direction of 0º is w_i= 0.106 kN/m^2. For wind directions of 90º it is w_i= -0.341 kN/m^2

4.3.5 - Horizontal forces

It was assumed that all the horizontal forces are absorbed by the walls of the church including both, tower and choir.

4.4 - Combination of actions

According to the national annex [60] the values of φ_0, φ_1 and φ_2 are present in the Table 4.6.

Action	φ_0	φ_1	φ_2
Imposed load	0.7	0	0
Snow	0.7	0.5	0.4
Wind	0.7	0.2	0

Table 4.6 – Wind pressure for south – north wind direction.

In this study two situations were contemplated. The permanent and snow loads as non-favorable and as favorable to see the structure reaction to the suction wind forces. The following combinations were considered:

- Leading variable action snow (S) and wind accompanying variable action wind south ↔ north direction (W1). The way (S-N or N-S) is indifferent because of the structure longitudinal symmetry.

$$\sum 1.35 * \mathrm{G} + 1.5 * \mathrm{S} + \sum 1.5 * 0.7 * \mathrm{W1} \qquad (4.14)$$

- Leading variable action snow (S) and wind accompanying variable action wind west → east direction (W21).

$$\sum 1.35 * \mathrm{G} + 1.5 * \mathrm{S} + \sum 1.5 * 0.7 * \mathrm{W21} \qquad (4.15)$$

- Leading variable action, snow (S) and accompanying variable action, wind east – west direction (W22).

$$\sum 1.35 * \mathrm{G} + 1.5 * \mathrm{S} + \sum 1.5 * 0.7 * \mathrm{W22} \qquad (4.16)$$

- Leading variable action, wind south ↔ north direction (W1) and accompanying variable action, snow (S).

$$\sum 1.35 * \mathrm{G} + 1.5 * \mathrm{W1} + \sum 1.5 * 0.7 * \mathrm{S} \qquad (4.17)$$

- Leading variable action, wind west → east direction (W21) and accompanying variable action, snow (S).

$$\sum 1.35 * G + 1.5 * W21 + \sum 1.5 * 0{,}7 * S \quad (4.18)$$

- Leading variable action, wind east → west direction (W22) and accompanying variable action, snow (S).

$$\sum 1.35 * G + 1.5 * W22 + \sum 1.5 * 0.7 * S \quad (4.19)$$

- Leading variable action, wind south ↔ north direction (W1).

$$\sum 1.0 * G + 1.5 * W1 \quad (4.20)$$

- Leading variable action, wind west ↔ east direction (W21).

$$\sum 1.0 * G + 1.5 * W21 \quad (4.21)$$

- Leading variable action, wind east ↔ west direction (W22).

$$\sum 1.0 * G + 1.5 * W22 \quad (4.22)$$

4.5 - Modeling

In order to model the structure (Fig 4.33), some simplifications were made.

The transversal section of elements were considered constant and their length equal for all trusses that in their turn were considered symmetric and equally distanced between each other.

The joints were considered mostly hinged except where the elements cross each other (in this case they were modeled as fixed one to another). This kind of model was done due to the inability to make intermediary connections hinged without separating one element into two acting distinctly one from another. Making the connections hinged the stress results acting on the elements are superior. Therefore the model will be more conservative.

There were some delicate parts in the modeling namely the connection between collar beams and upper plate, the connection between lower plate to tie beam and king post and the connection between the rafters and the king post.

The first connection is problematic because in this situation the collar beam is not attached to the upper plate in the secondary trusses but they simply lay on them (Fig. 4.25). In the main trusses is less problematic since both collar beam and upper plate are connected to the king post (Fig. 4.45 and 4.46) and upper knee braces (Fig. 4.44 and 4.47), preventing the elements to behave separately. The solution was to consider fixed connection in main frames and in the secondary ones it was not considered connection between the elements making them act separately.

The second issue is almost the same as the first problem, in this case the lower plate is only laying down on the tie beam (Fig. 4.60) and the post is not connected to the tie beam but to the lower plate (Fig. 4.59). In the main trusses this issue is less relevant due to the fact that the lower knee braces are connected to the tie beam (Fig. 4.57) and the king post (Fig. 4.58) making more difficult to separate the three elements. Once again the solution was to consider the lower plate – tie beam fixed to each other in the main trusses and in the secondary ones the elements not connected, acting separately from each other. If the tie beam – lower plate connection is not made at least in the main trusses, then the lower plate wouldn't be unloading anywhere. A hinged joint is considered amongst the king post and the tie beam/ lower plate, what is acceptable to use considering the other similar joints in the structure.

The third problem is in the connection between king post and rafters once the king post is not fixed to the rafters (Fig. 4.37) denying the possibility of the rafters to "pull" the post, leaving this one in tension (as first model attempt shows, Fig. 4.34). Two solutions were studied, in the first one it was imposed a maximum tension of zero to the post between rafters and its intersection with compound rafters, the second one was interrupting the king post 1cm before it reached the rafters creating a gap between them. Comparing the results (Appendix 1) it was visible that the values of the first and the second one were very similar, except in the structure deformation, for this reason the solution with the gap was chosen based on a more realistic deformation of the king post (possibility to disconnect itself from the rafters). The first model attempt with the king post and rafters joint together could be used as an alternative in a case where structural alteration is required.

A first analysis will be a simulation of the structure's original behavior, to see if this one was well conceived to resist the values from Eurocode actions (EC1), passing the ultimate limit states (EC5). If the security of the structure, according to EC5 criteria is verified, it means that the interventions will be only to restore the structure original form and resistance. When verification fails narrowly some security factors can be diminished, taking into account that this structure had been there for centuries subjected to external actions and "surviving" them. When security verification fails roundly, then some structural modifications should be made to prevent a collapse, to avoid not only historical material losses but also endanger human lives.

A second analysis will be made to see the structure behavior and element function for one symmetrical (permanent and snow) and one asymmetrical (permanent and wind acting south – north direction) loads.

The Serviceability limit states were not verified since it was considered acceptable as the structure didn't reveal significant damage, distress or deterioration and demonstrated satisfactory performance for a long period of time. Future alterations to the structure and its use, which can change its loads, are not predicted.

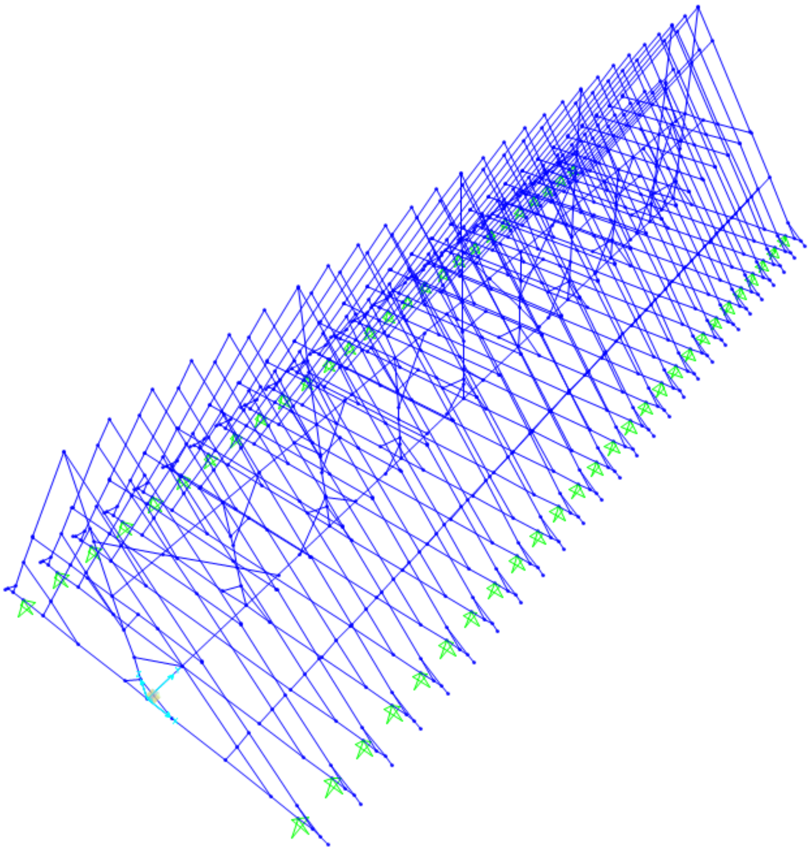


Fig. 4.33 – Schematic of 3D structural modeling

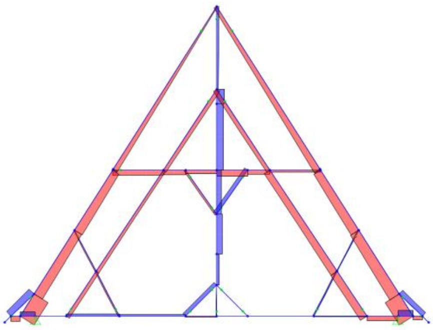


Fig. 4.34 – First model – King Post in tension [kN].

 - Compression.

 - Tension.

4.5.1 - Joints

The way the connections were modeled is already mentioned above. In this subchapter for each connection it will be exhibited one example on site.

Figs. 4.35 and 4.36 are the transversal and longitudinal map respectively, of the different connections shown from Figs. 4.37 to 4.66. The main and secondary frames model in Sap2000 is schematized in figures 4.67 to 4.70.

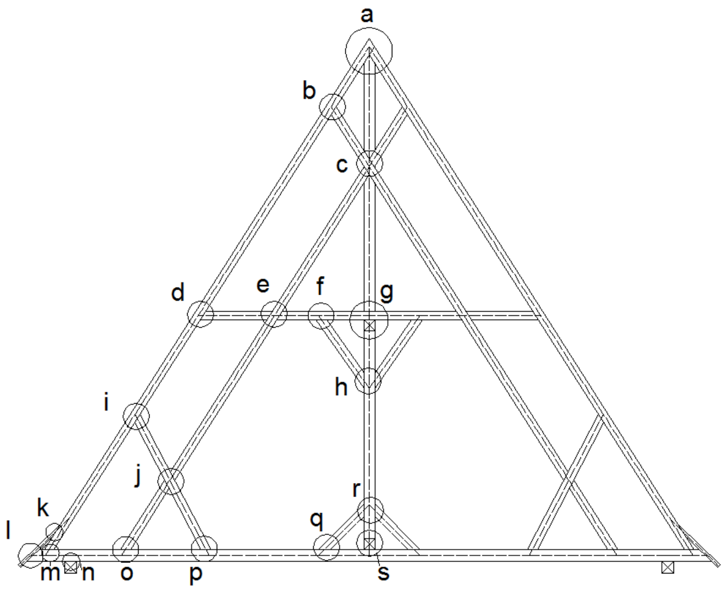


Fig. 4.35 – Joints transversal map.

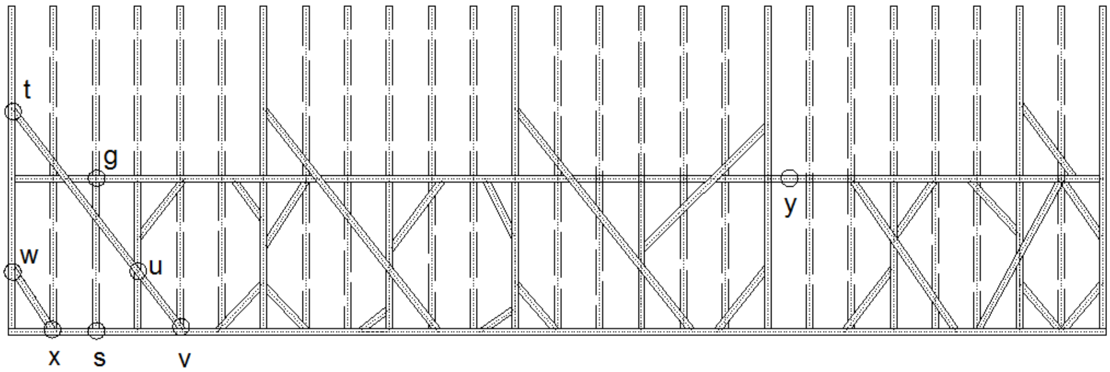


Fig. 4.36 – Joints longitudinal map.


Fig. 4.37 - Ridge + King post (a) - Half lapped joint with wooden peg + Support Hinged joint + Gap.

Fig. 4.38 - Rafter – Compound rafter (b) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.39 - King post – Compound rafters 1(c) - Half Lapped joint with wooden peg
Fixed joint.

Fig. 4.40 - King post – Compound rafters 2(c) - Dove tail lapped joint with wooden peg
Hinged Joint.

Fig. 4.41 - Compound rafter - Compound rafter (c) - Half Lapped joint with wooden peg
Fixed joint.

Fig. 4.42 - Collar beam – Rafter (d) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.43 - Collar beam – Compound rafter (e) - Half Lapped joint with wooden peg
Fixed joint.

Fig. 4.44 - Collar beam – Knee brace (f) - Dove tail lap joint with wooden peg
Hinged joint.

Fig. 4.45 - Collar beam – King post (g)- Half lapped joint with wooden peg
Fixed joint.

Fig. 4.46 - Upper plate – King post (g) - Lapped joint with wooden peg
Fixed joint.

Fig. 4.47 - Knee braces – King post (h) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.48 - Angle brace – Rafter (i) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.49 - Angle brace – Compound rafter (j) - Half Lapped joint with wooden peg
Fixed joint.

Fig. 4.50 - Rafter – Sprocket (k) – Nailed
Hinged joint.

Fig. 4.51 - Tie beam – Sprocket (l) – Supported
Hinged joint.

Fig. 4.52 - Tie beam – Rafter (m) - Mortice and tenon without wooden peg
Hinged joint.


Fig. 4.53 - Tie beam – Wall plate (n) – Notched
Pin support.


Fig. 4.54 - Tie beam – Support wall (n) - Embedded
Pin support.


Fig. 4.55 - Tie beam – Compound Rafter (o) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.56 - Tie beam – Angle brace (p) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.57 - Tie beam – Lower knee brace (q) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.58 - King post – Lower knee braces (r) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.59 - King post – Lower plate (s) - Half lapped joint with wooden peg
Hinged joint.

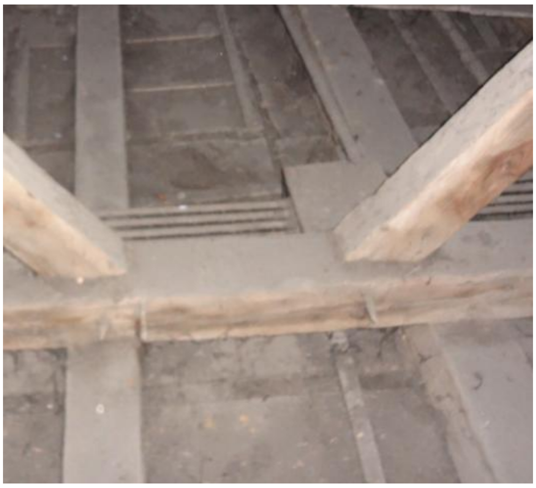


Fig. 4.60 - Tie beam – Lower plate (s).

Fig. 4.61 - Passing brace – King post 1(t) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.62 - Passing brace – King post 2 (u) - Dove tail lapped joint with wooden peg
Fixed joint.

Fig. 4.63 - Passing brace – Lower plate (v) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.64 - (Longitudinal) Angle brace – King post (w) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.65 - (Longitudinal) Angle brace – Lower plate (x) - Dove tail lapped joint with wooden peg
Hinged joint.

Fig. 4.66 - Upper plate –Upper plate (y) - Half Lapped joint with wooden peg
Continuous.

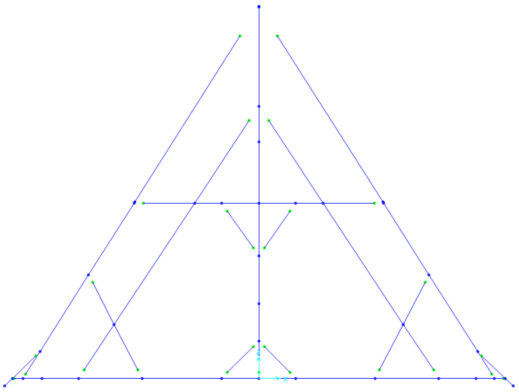


Fig. 4.67 – MT1 – Hinged and Rigid joints.

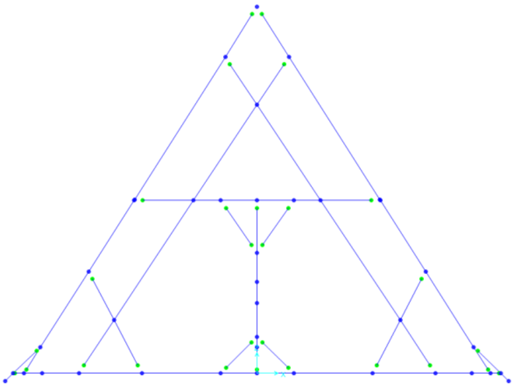


Fig. 4.68 – MT2 – Hinged and Rigid joints.

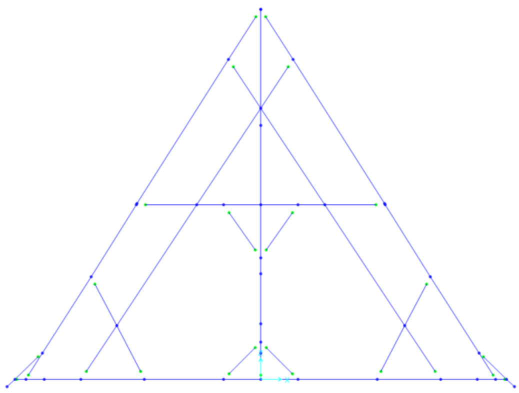


Fig. 4.69 – MT2' – Hinged and Rigid joints.

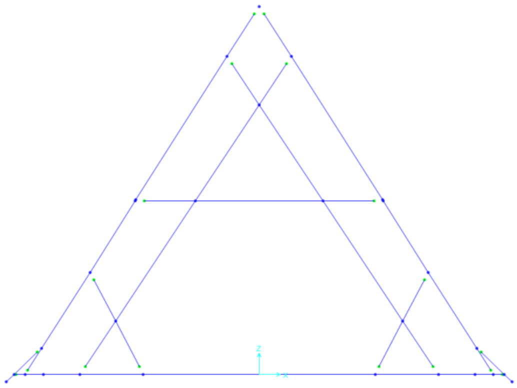


Fig. 4.70 – ST – Hinged and Rigid joints.

4.5.2 - Material

For visual assignment of the strength class of wood specifies according to EN 338 [79], the EN 14081 [20] and EN 1912 [21], can be used allied with the national annexes. However because it was not possible to get the national annex for Romania, an assumption based on Darie, et al. [17] was made. According to these authors as a result of the visual grading by type and magnitude of the defects, the timber is ranked according to the following classes of quality: C10, C18, C24, C30 and C40. Mainly thanks to the elements cracks due shrinkage (drying) and the fact that lower classes were not used for structures, the class of the material was assumed to be C18, a relative conservative measure, since for softwood species like fir the classes in EN 338 ranges goes from C14 to C50.

The Strength classes - Characteristic values adopted are present in Table 4.7 according to EN 338[79].

Strength properties		
Bending	$f_{m,k}$	18 N/mm^2
Tension parallel	$f_{t,0,k}$	11 N/mm^2
Tension perpendicular	$f_{t,90,k}$	0.4 N/mm^2
Compression parallel	$f_{c,0k}$	18 N/mm^2
Compression perpendicular	$f_{c,90,k}$	2.2 N/mm^2
Shear	$f_{v,k}$	3.4 N/mm^2
Stiffness properties		
Mean modulus of elasticity parallel	$E_{0,mean}$	9 kN/mm^2
5% modulus of elasticity parallel	$E_{0,05}$	6 kN/mm^2
Mean modulus of elasticity perpendicular	$E_{90,mean}$	0.3 kN/mm^2
Mean shear modulus	G_{mean}	0.56 kN/mm^2
Density		
Density	ρ_k	320 kg/m^3
Mean density	ρ_{mean}	380 kg/m^3

Table 4.7 - Strength classes - Characteristic values

The service class adopted was 2 based on the work of Cornelia and Nandor [15] and moisture content registered in situ in one of the site visits, around 17% in some elements. Therefore the modification factor, k_{mod} (see Table E.4 – Annex 3), will be 0.60, 0.80 and 0.90 for permanent (self-weigh), medium (snow) and short (wind) action respectively.

4.6 - Ultimate Limit states

For the ultimate limit states was used SAP2000 envelope type combination that devolves the higher and lower values of the structure stresses for which the security must be verified.

It was decided to verify design of cross-sections subjected to combined stresses, namely, combined bending and axial tension or, combined bending and axial compression. Even if one element has only one of the stresses (bending only in one direction, axial tension or compression) and the others are negligible, these verifications are for the worst case scenario. The other verifications are for stability of members in particular, for columns subjected to either compression or combined compression and bending and for beams subjected to either bending or combined bending and compression. Looking at the results of the program, it is shown that the torsion values are small and therefore insignificant from the verification point of view.

The verifications mentioned above were made in an excel document for all SAP2000 elements (one structural element in reality can divide itself in several segments in the program, normally in their intersection with each other) what leads to an exhaustive list (8343 lines), that would not be reasonable to attach to the dissertation. Since the safety of all elements was proved, it was decided to show the location and the stresses of the most solicited element for each of these verifications.

Figs. 4.71 to 4.73 represent the local axis of the structure elements, to better understand the meanings of M2 and M3 in the Tables 4.8 to 4.10 with the stresses of the structure in the section where the worst value for the different verifications of the limit states are observed and represented in Figs. 4.74 to 4.76.

Represented in red, the axis 1is in the grain direction of the element, axis 2 is in the height direction with green color and axis 3 in the width direction with blue color. Therefore M2 is the bending moment in 1-3 plane (about the 2 axis) and M3 is the bending moment in 1-2 plane (about the 3 axis).

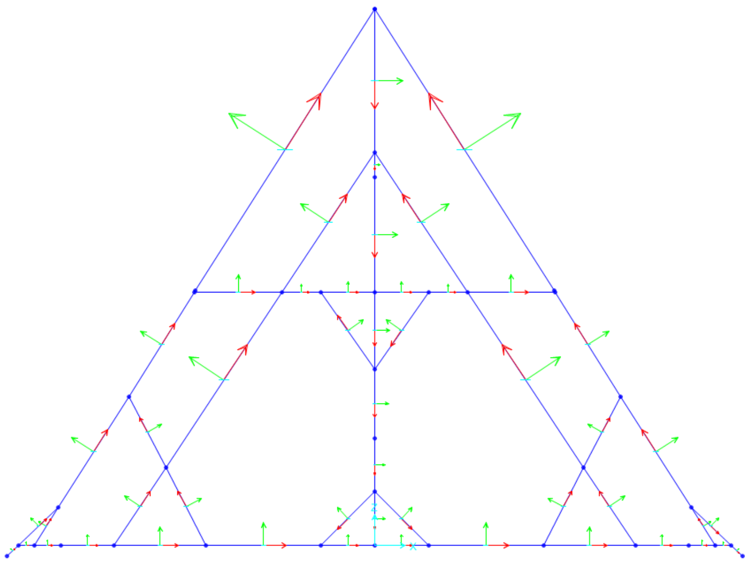


Fig. 4.71 – Local axis representation – transversal perspective of one truss.

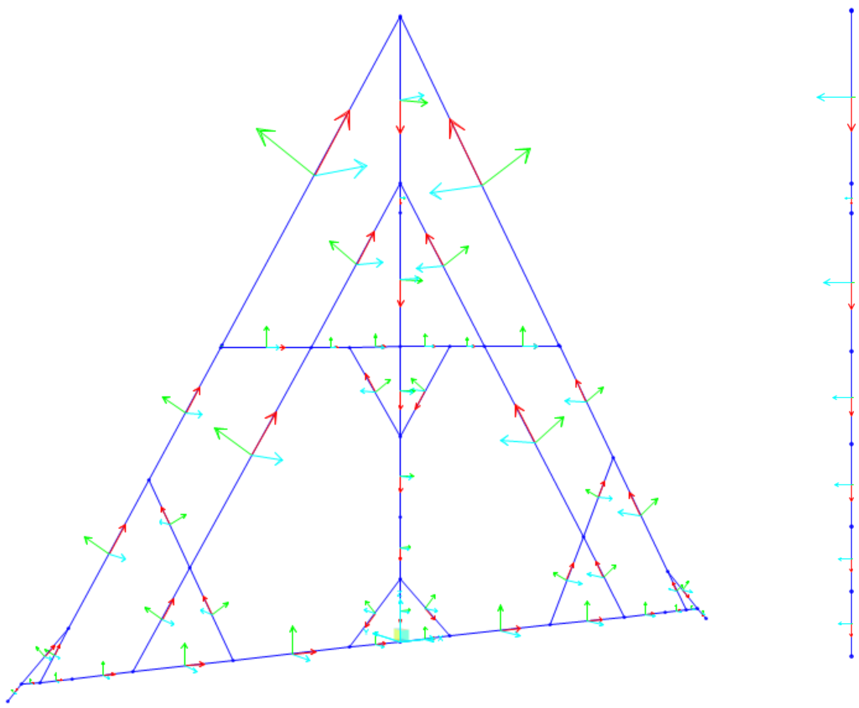


Fig. 4.72 – Local axis representation – 3D and longitudinal perspective of one truss.

Local axis 1 (red).

Local axis 2 (green).

Local axis 3 (blue).

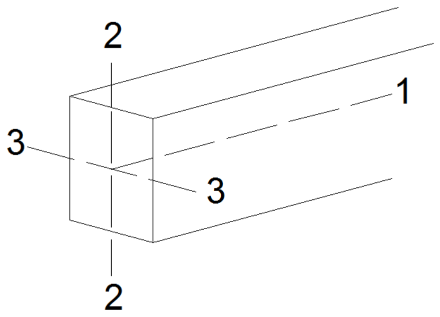


Fig. 4.73 – Axes representation in one element.

4.6.1 - Combined bending with axial tension or compression

For combined bending and axial tension the following expressions shall be satisfied

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \quad (4.23)$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \quad (4.24)$$

For combined bending and axial compression the following expressions shall be satisfied

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \quad (4.25)$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \quad (4.26)$$

Where

$\sigma_{t,0,d}$ is the design tensile stress along the grain;

$f_{t,0,d}$ is the design tensile strength along the grain;

$\sigma_{c,0,d}$ is the design compressive stress along the grain;

$f_{c,0,d}$ is the design compressive strength along the grain.

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are the design bending stresses about the principal axes;

$f_{m,y,d}$ and $f_{m,z,d}$ are the corresponding design bending strengths;

The factor k_m makes allowance for re-distribution of stresses and the effect of inhomogeneities of the material in a cross-section. The value of this factor should be 0.7 for rectangular sections and 1.0 for other cross sections.

The biggest value from expressions 4.23 to 4.26 was 0.5825 ≤ 1.0 in the rafter intersection with the collar beam (Fig. 4.74) in the frame number 25 - a main truss without compound rafters intersection with rafters (MT2). In Table 4.8 is possible to see the stress values that originate the biggest value mentioned above.

Element	Axial (kN)	M2 (kN.m)	M3 (kN.m)
930	7.28	0.01	2.18

Table 4.8 – Worst section stress values - Combined bending with axial tension or compression.

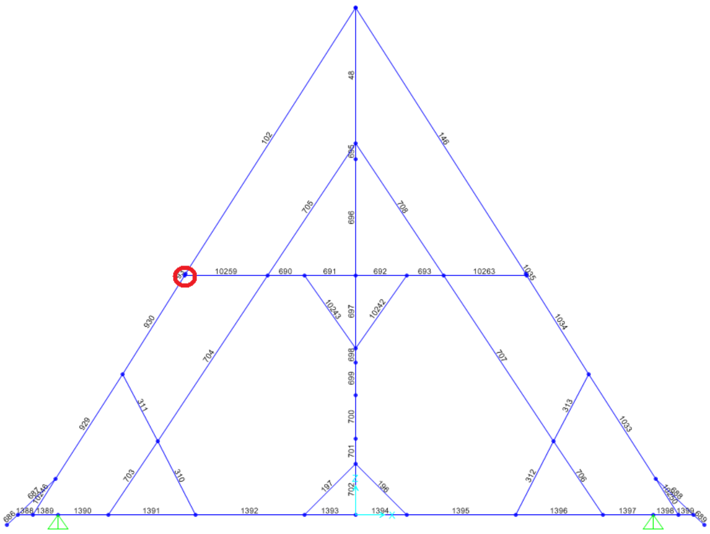


Fig. 4.74 - Combined bending with axial tension or compression worst case.

4.6.2 - Columns subjected to either compression or combined compression and bending

Where both $\lambda_{rel,y} \leq 0.3$ and $\lambda_{rel,z} \leq 0.3$ the stresses should satisfy the expressions from Combined bending and axial compression.

In all other cases the stresses, which will be increased due to deflection, should satisfy the following expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} * f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \qquad (4.27)$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} * f_{c,0,d}} + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \qquad (4.28)$$

Where

$\lambda_{rel,y}$ is a slenderness ratio corresponding to bending about the y-axis (deflection in the z-direction);

$\lambda_{rel,z}$ is a slenderness ratio corresponding to bending about the z-axis (deflection in the y-direction);

$k_{c,y}$ or $k_{c,z}$ are instability factors;

The biggest value from expressions 4.27 and 4.28 was 0.1718 ≤ 1.0 in king post intersection with the upper knee braces (Fig. 4.75) in the frame number 22 - a main truss with a simple king post (MT1). In Table 4.9 is possible to see the stress values that originate the biggest value mentioned above.

Element	Axial (kN)	M2 (kN.m)	M3 (kN.m)
1099	-3.18	-0.05	-1.04

Table 4.9 – Worst section stress values - Columns subjected to either compression or combined compression and bending.

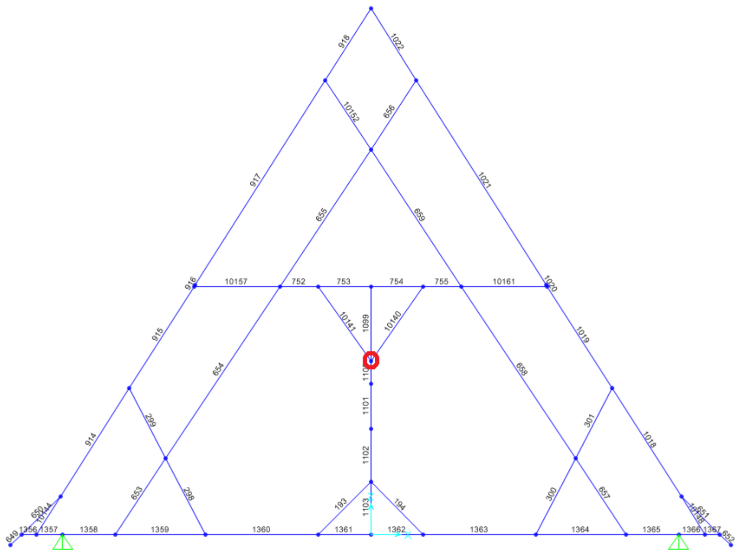


Fig. 4.75 - Columns subjected to either compression or combined compression and bending worst case

4.6.3 - Beams subjected to either bending or combined bending and compression

In the case where a combination of moment M_y about the strong axis y and compressive force N_c exists, the stresses should satisfy the following expression:

$$\left(\frac{\sigma_{m,d}}{k_{crit}*f_{m,d}}\right)^2 + \frac{\sigma_{c,d}}{k_{c,z}*f_{c,0,d}} \leq 1.0 \quad (4.29)$$

Where

$\sigma_{m,d}$ is the design bending stress;

$f_{m,d}$ is the design bending strength;

k_{crit} is a factor which takes into account the reduced bending strength due to lateral buckling.

$\sigma_{c,d}-$ is the design compressive stress;

The biggest value from expression 4.29 was 0.3786 ≤ 1.0 in the rafter intersection with the collar beam (Fig. 4.76) in the frame number 25 - a main truss without compound rafters intersection with rafters (MT2). The same position as the biggest value for combined bending with axial tension or compression. In Table 4.10 is possible to see the stress values that originate the biggest value mentioned above.

Element	Axial (kN)	M2 (kN.m)	M3 (kN.m)
930	7.28	0.01	2.18

Table 4.10 – Worst section stress values - Beams subjected to either bending or combined bending and compression.

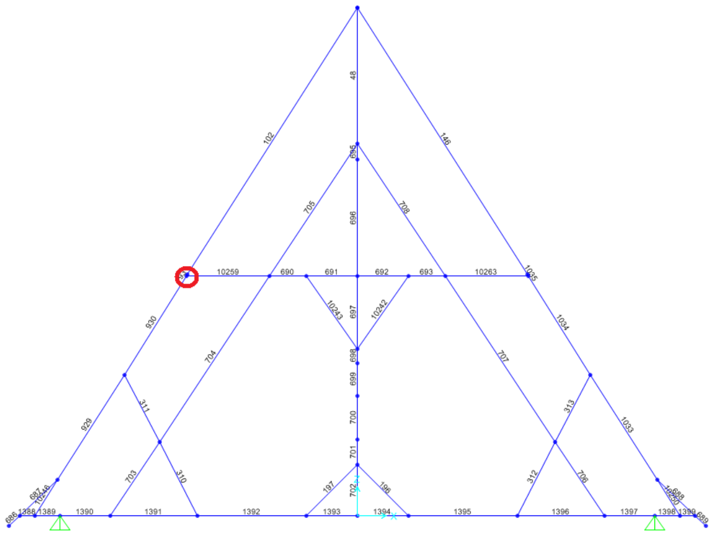


Fig. 4.76 - Beams subjected to either compression or combined compression and bending worst case.

4.7 - Element function / Structural analysis

In order to better understand the structure behavior, the element function in the structure and each ones are more solicited, two combinations were analyzed:

- Symmetrical load, a combination of permanent (G) with snow loads (S)

$$\sum 1.35 * G + 1.5 * S \qquad (4.30)$$

- Asymmetrical load, a combination of permanent (G) with wind loads, south-north direction (W1)

$$\sum 1.35 * G + 1.5 * W1 \qquad (4.31)$$

To illustrate the difference between the trusses, a representative of each type was chosen, given that they have similar stresses to their resembling. For the main truss 1 the alignment number 4 was used, for the main truss 1’ it was used the alignment number 13, for the main truss the alignment number 7 was used, for the secondary truss it was used the alignment number 3 (see Fig. 4.11).

The table 4.11 shows the worst stress values for the common elements in the chosen trusses. Figs. 4.77 to 4.80 represent the location of the loads acting on the elements.

Element	Frame	Symmetrical			Asymmetrical		
		Nmin (kN)	Mmin (kNm)	Mmax (kNm)	Nmin (kN)	Mmin (kNm)	Mmax (kNm)
Rafter	MT1	-14.506	-0.774	0.390	-14.268	-0.964	0.820
	MT1’	-15.716	-0.882	0.482	-14.306	-1.090	0.888
	MT2	-16.367	-1.236	0.785	-13.955	-1.942	1.647
	ST	-19.920	-1.117	0.997	-18.512	-1.238	1.531
Passing braces	MT1	-4.647	-0.421	0.391	-7.318	-0.743	0.628
	MT1’	-6.759	-0.353	0.598	-9.664	-0.639	0.763
	MT2	-6.091	-0.271	0.260	-8.296	-0.792	0.714
	ST	-2.095	-0.634	0.777	-5.253	-1.148	1.153
Angle brace	MT1	0.133	-0.102	0.190	-1.277	-0.101	0.600
	MT1’	0.400	-0.0085	0.3394	-1.038	-0.008	0.745
	MT2	0.548	-0.049	0.175	-0.779	-0.025	0.671
	ST	1.196	-0.563	1.69E-16	-0.586	-0.561	0.323
Collar beam	MT1	-7.227	-0.586	0.663	-7.927	-0.866	0.960
	MT1’	-4.033	-0.111	0.511	-4.737	-0.265	0.660
	MT2	-4.827	-0.107	0.296	-6.322	-0.442	0.660
	ST	-4.454	-0.028	0.616	-5.142	-0.451	1.026
Tie beam	MT1	-3.890	-2.535	0.907	-5.129	-2.271	1.694
	MT1’	-3.901	-2.893	1.328	-5.206	-2.642	2.260
	MT2	-4.121	-2.956	1.048	-5.122	-2.616	1.888
	ST	-6.728	-2.548	1.742	-7.878	-2.642	2.260
Sprocket	MT1	-2.5E-14	-0.016	0.04	-1.2E-14	-0.026	0.065
	MT1’	1.4E-14	-0.016	0.04	1.3E-14	-0.026	0.065
	MT2	-3E-14	-0.016	0.04	-2.2E-14	-0.026	0.065
	ST	-2.4E-15	-0.016	0.04	1.3E-14	-0.026	0.065

Table 4.11 - Worst stress values for the common elements.

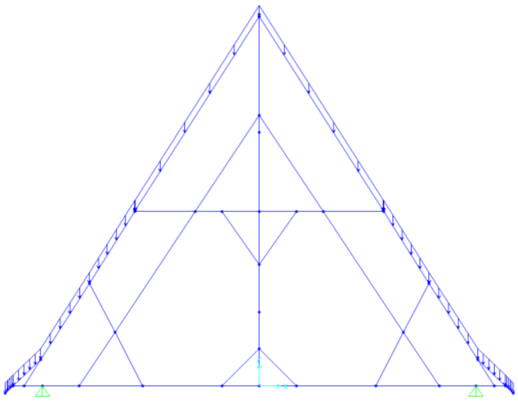


Fig. 4.77 – Snow load - representation.

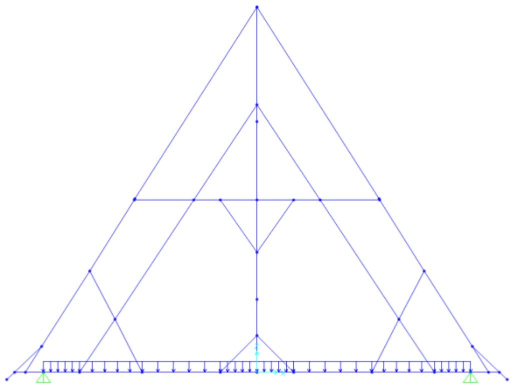


Fig. 4.78 – Ceiling load - representation.

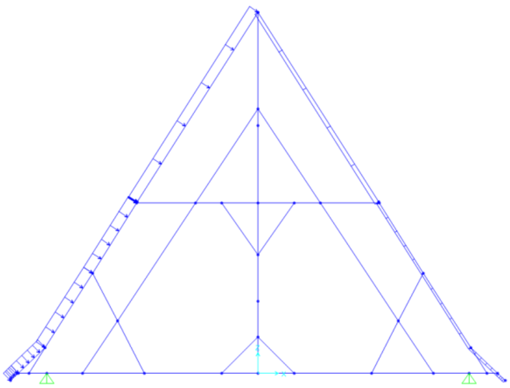


Fig. 4.79 – Wind load acting on external surfaces - representation.

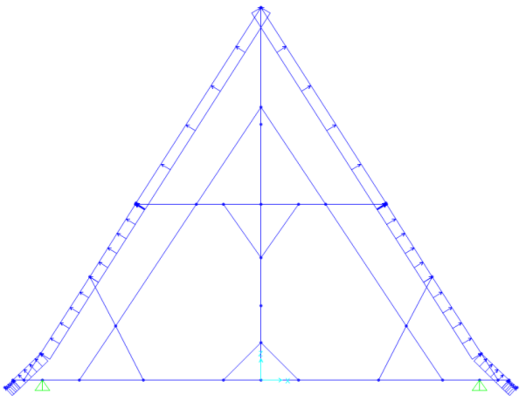


Fig. 4.80 – Wind load acting on internal surfaces - representation.

4.7.1 - Symmetrical loads – Transversal system

From Figs. 4.81 to 4.84 it's represented the stresses in the chosen trusses for axial forces, from 4.85 to 4.88 for bending moments and from 4.89 to 4.92 for the deformations.

<u>Axial forces [kN]</u>

All of them have in the Rafter, Passing braces (except in the secondary trusses) and Collar beam, the main compression forces.

The effective length of the Rafters (the elements subjected to external forces) is reduced by Collar beam, Angle and Passing braces (except in MT2 where Passing braces ends on the King post, not reaching the Rafters) what in terms of preventing buckling is properly done, especially close to the bottom of the structure where axial forces are bigger.

The Tie Beam has only considerable compressions between the Sprockets and Rafter and between the support and Passing braces for all frames.

The Angle braces practically have axial forces close to zero or tension, what can be due to compression in the upper part from the rafter loads but tension in the lower part, due to the Tie beam.

The Sprockets and Knee braces are only subjected to tension forces.

Most of the King post parts are subjected to tension for all main truss cases. Comparing the MT1 and MT1’ it is shown that the King posts have more tension and the Passing braces have more compression in the MT1’.

The scale factor presented by Sap2000 was 0.07.

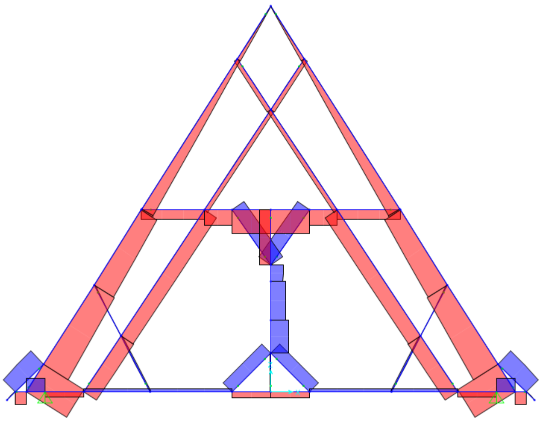


Fig. 4.81 – MT1 – Axial forces when subjected to symmetrical load

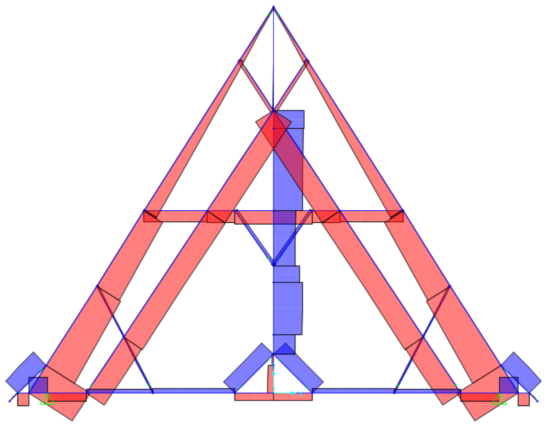


Fig. 4.82 – MT1’ – Axial forces when subjected to symmetrical load

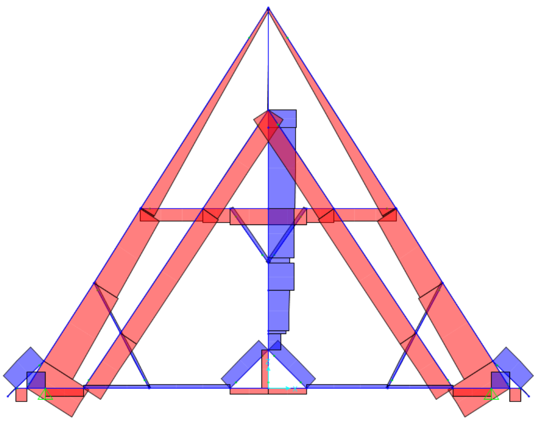


Fig. 4.83 – MT2 – Axial forces when subjected to symmetrical load

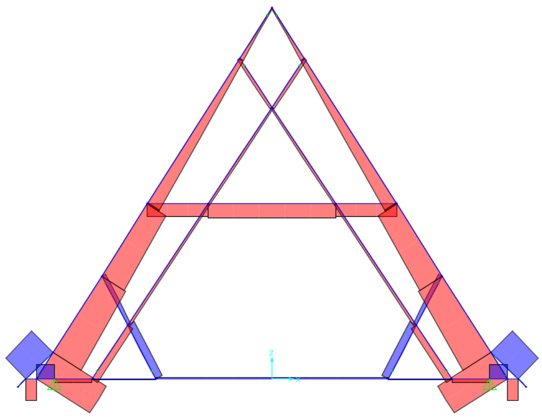


Fig. 4.84 – ST – Axial forces when subjected to symmetrical load

- Compression.

- Tension.

<u>Bending moments [kN.m]</u>

The most solicited elements are the Rafters and the Tie Beam, where the loads are acting.

Due to equal supports on the Rafters of ST, MT1 and MT1’ the bending moment diagrams assume a close shape to each other distinguishing them from the MT2 where the Passing braces are not connected to the rafter. However in the ST the maximum values are higher when compared with the MT1 and MT1’ and reaches values similar to the MT2.

The Tie Beam is the most requested element for bending moments, with the higher negative and positive values when compared with all the other elements and this happens in all truss types. The reason for this is obvious since, besides the ceiling, this element is subjected to the other elements self-weight, which on their turn transmit external actions like snow or wind, transferring them all for the bearing walls.

Looking at the diagrams it is visible that the main trusses have an identical shape, different from the ST one. The reason for this is the non-existing connection between Lower plate and Tie beam in the ST, creating bigger positive moments when compared with the main trusses, where these elements connection with the king post exist. The main trusses with double King post (MF1' and MT2) have bigger moments compared with the MT1 where the King post ends in the Collar beam intersection.

The rest of the elements have some lower moments next to the connections between them.

The scale factor presented by Sap2000 was 0.4.

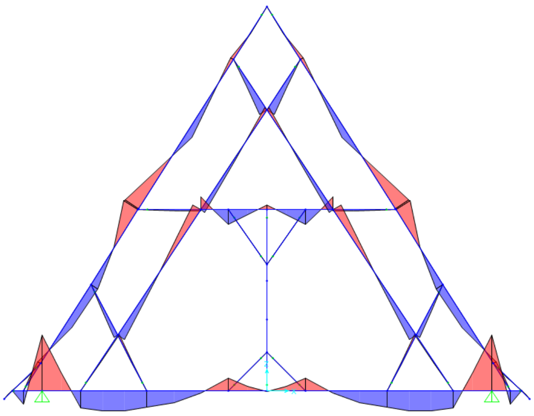


Fig. 4.85 – MT1 - Bending moments when subjected to symmetrical load

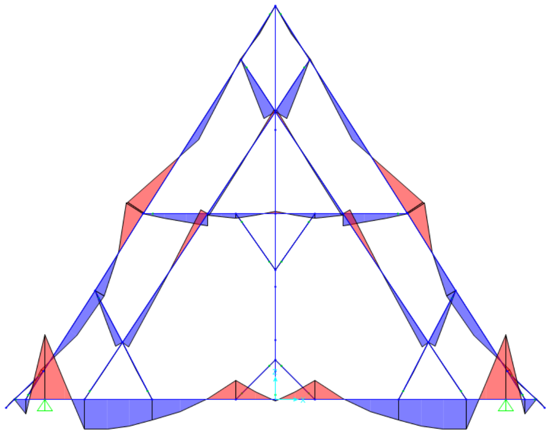


Fig. 4.86 – MT1' - Bending moments when subjected to symmetrical load

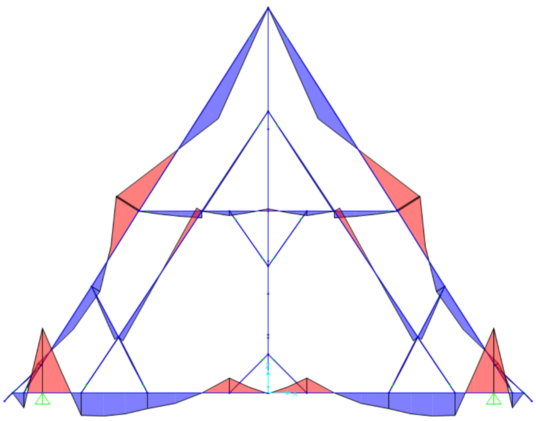


Fig. 4.87 – MT2 - Bending moments when subjected to symmetrical load

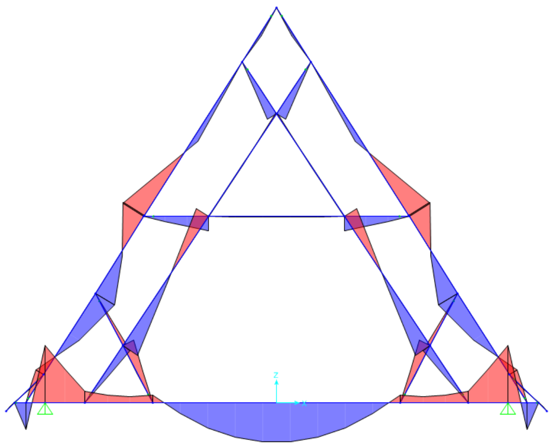


Fig. 4.88 – ST - Bending moments when subjected to symmetrical load

- Negative bending moments.

- Positive bending moments.

Deformation [m]

The most deformed elements are the Rafters, the Tie beam where the loads are acting and the Passing braces.

In the rafters, like in the bending moments, the deformation of ST, MT1 and MT1’ assume a resembling outline (with bigger proportions in the ST) when compared to the MT2.

The same situation is for the tie beam. The main trusses deformation shape is different from the ST for the same reasons as mentioned in the bending moments.

The Passing braces are mostly deformed next to its connection with the Angle brace. This happens when the Angle brace follows the Rafter and Tie beam movements (deformations) dragging the Passing brace and forcing its deformation.

The rest of the elements don’t have such significant deformations as the ones mentioned above.

The scale factor presented by Sap2000 was 200.

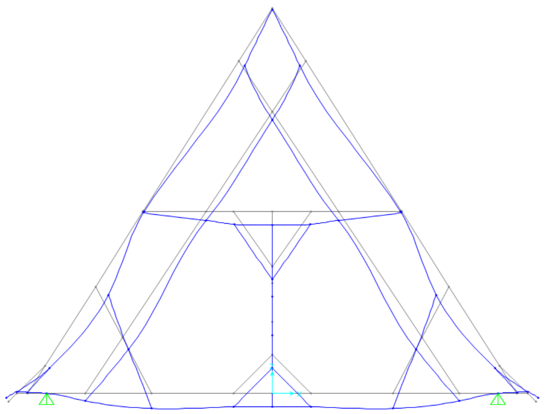


Fig. 4.89 – MT1 - Deformation when subjected to symmetrical load

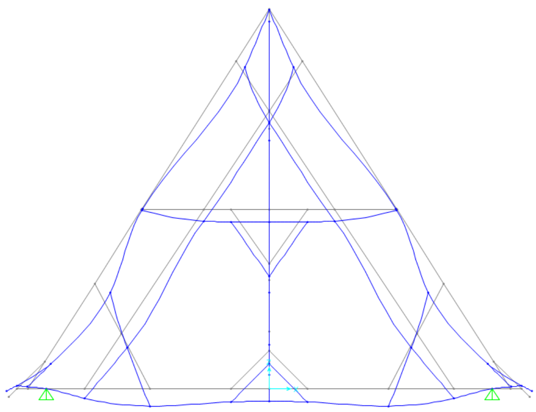


Fig. 4.90 – MT1’ - Deformation when subjected to symmetrical load

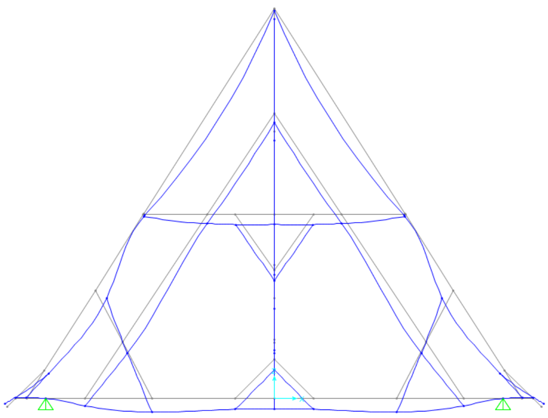


Fig. 4.91 – MT2 - Deformation when subjected to symmetrical load

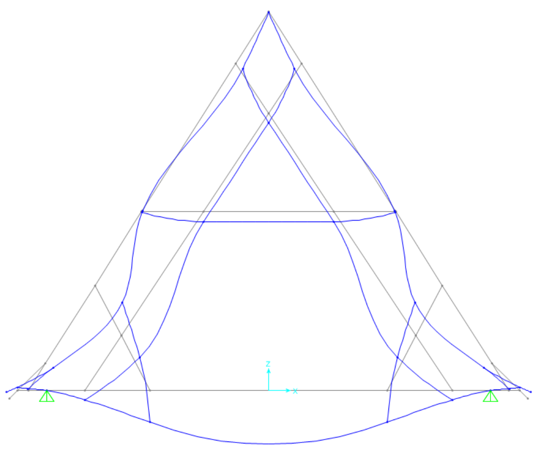


Fig. 4.92 – ST - Deformation when subjected to symmetrical load

4.7.2 - Asymmetrical loads– Transversal system

From Figs. 4.93 to 4.96 it's represented the stresses in the chosen trusses for axial forces, from 4.97 to 4.100 for bending moments and from 4.101 to 4.104 for the deformations.

Axial forces [kN]

All of the trusses have, in the Rafter, Passing brace (the one that intersects the Rafter with the wind compression part or the correspondent in the MT2) and Collar beam (higher where it intersects the wind compression part and after the King post intersection in main trusses) the main compression forces.

Just like in the symmetric load case, the axial forces increase in the Rafters from the upper to the lower part connected to the Tie beam, where the reduction of effective length for the prevention of buckling is guaranteed by the Collar beam and Angle braces.

The Tie Beams have compression only between the Sprockets and Rafter (both sides), between lower Knee braces (in main trusses), and between the support and Passing braces (wind suction side).

The Angle braces in all trusses have compression forces on the wind compression side and tension on the other.

The Sprockets are only subjected to tension forces.

Only the upper Knee brace closer to the wind compression side, have compression in MF1' and MF2 (double King posts), all the others are tensioned.

Comparing the MT1 and MT1' it is shown that similarly to the symmetrical load the King posts have more tension and the Passing braces have more compression in the MT1'. Most of King post parts are in tension.

The scale factor presented by Sap2000 was 0.07.

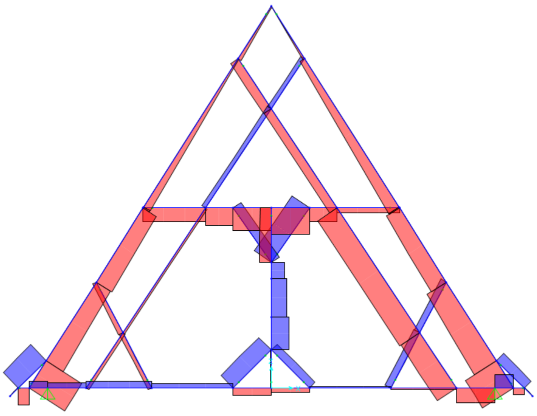


Fig. 4.93 – MT1 – Axial forces when subjected to asymmetrical load

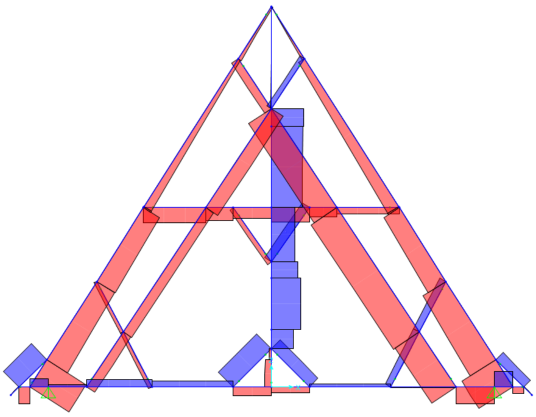


Fig. 4.94 – MT1' – Axial forces when subjected to asymmetrical load

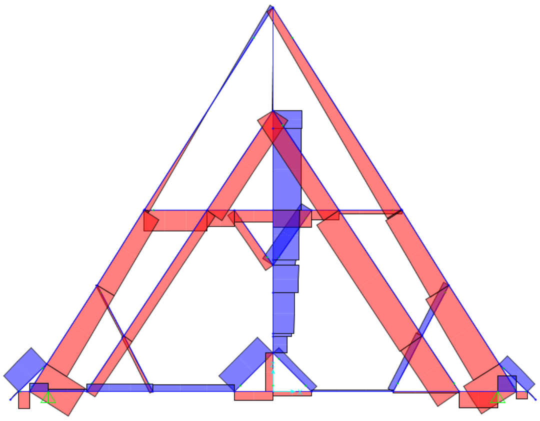


Fig. 4.95 – MT2 – Axial forces when subjected to asymmetrical load

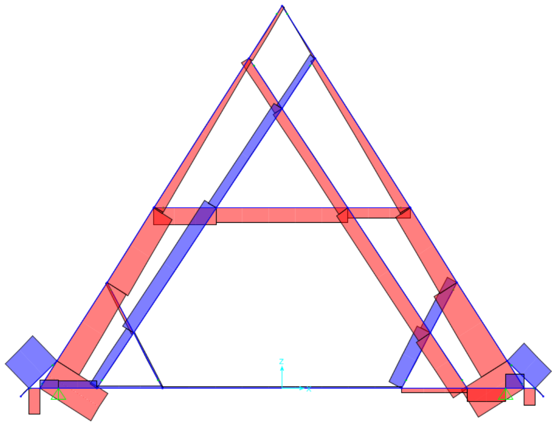


Fig. 4.96 – ST – Axial forces when subjected to asymmetrical load

- Compression.

- Tension.

Bending moments [kN.m]

The most solicited elements are the Rafters and the Tie Beam (where the loads are acting), the Collar beam, King posts and Passing brace (wind suction side).

Due to equal supports on the Rafters of ST, MT1 and MT1’, just like in the symmetrical load, the bending moment diagrams assume a close shape to each other distinguishing them from the MT2 where the Passing braces are not connected to the rafter. However in the ST the maximum values are higher when compared with the MT1 and MT1’. The MT2 not only have bigger values than all the rest trusses but also large differences amongst “wind compression Rafter” where the maximum values are much higher than the other side.

The Tie Beam remains the most requested element for asymmetrical loads.

The main trusses have an identical bending moment shape between them, differently from the ST one. The main trusses with double King post (MF1’ and MT2) have bigger moments compared with the MT1 where the King post ends in the Collar beam intersection.

The King post mostly because of the “movement” created by wind compression in one side and suction on the other, transmitted by the Collar beam, forms moments between the Collar beam and Tie Beam.

The rest of the elements have some lower moments next to the connections between them.

The scale factor presented by Sap2000 was 0.4.

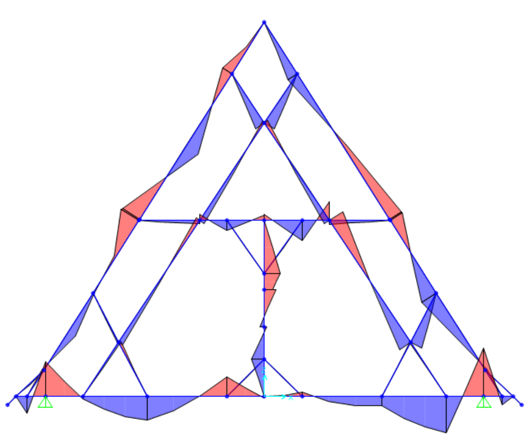


Fig. 4.97 – MT1 – Bending moments when subjected to asymmetrical load

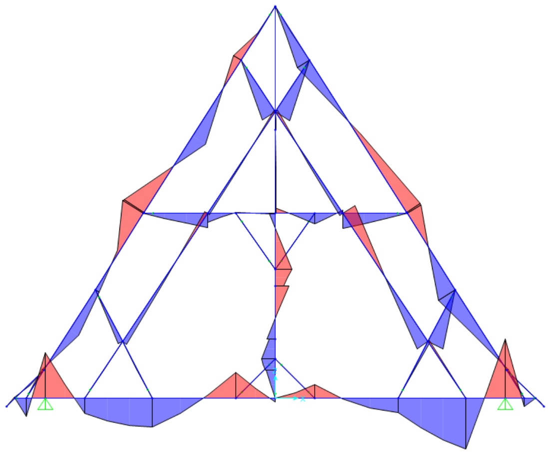


Fig. 4.98 – MT1’ – Bending moments when subjected to asymmetrical load

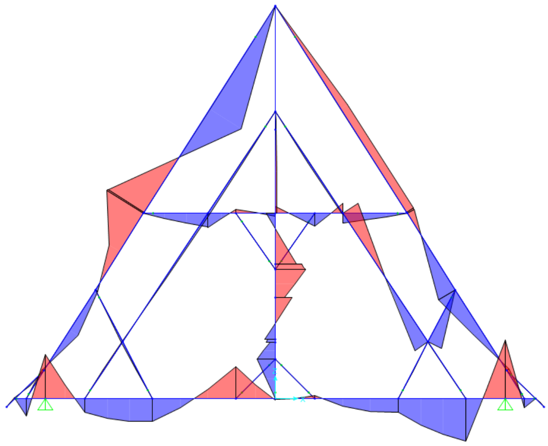


Fig. 4.99 – MT2 – Bending moments when subjected to asymmetrical load

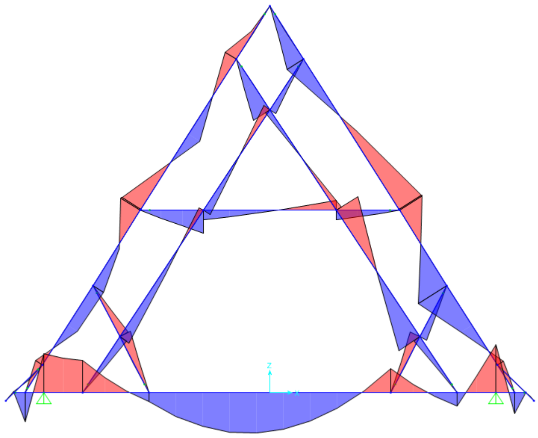


Fig. 4.100 – ST – Bending moments when subjected to asymmetrical load

- Negative bending moments.

- Positive bending moments.

Deformation [m]

The trusses elements are globally deformed (exception are Knee braces with less deformation than the others). It is shown that the structure is more affected with asymmetrical loads then with symmetrical ones.

Like in the bending moments, the deformation of ST, MT1 and MT1’ assume a resembling outline (with bigger proportions in the ST) when compared to the MT2.

For the Tie beam the same situation. The main trusses deformation shape is different from the ST for the same reasons as mentioned in the bending moments.

The rest of the elements are dragged, being forced to deform by the ones with the acting loads, the Tie beam is deforming gravitationally and the Rafters in the wind direction (south → north).

The scale factor presented by Sap2000 was 200.

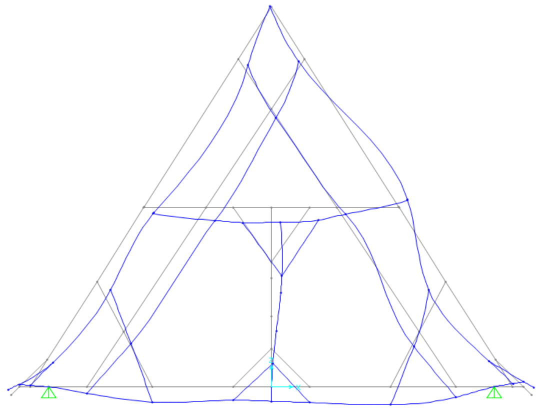


Fig. 4.101 – MT1 – Deformation when subjected to asymmetrical load

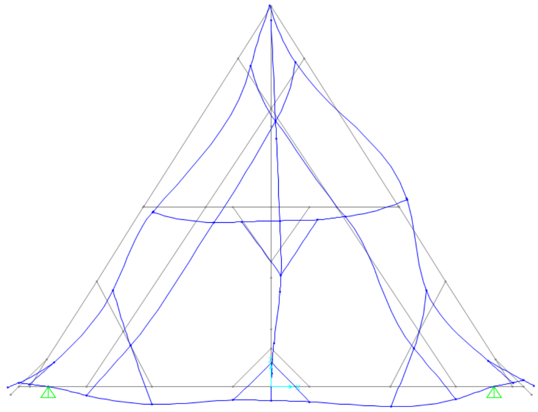


Fig. 4.102 – MT1’ – Deformation when subjected to asymmetrical load

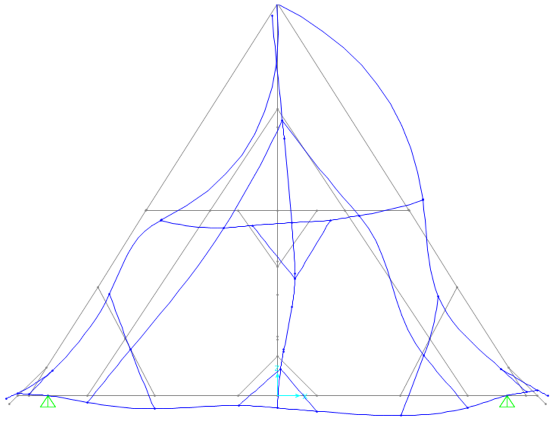


Fig. 4.103 – MT2 – Deformation when subjected to asymmetrical load.

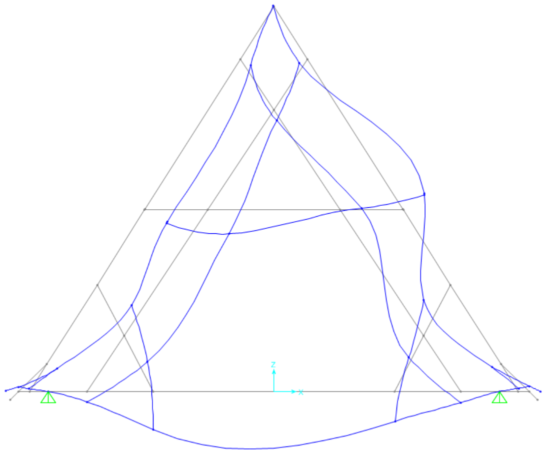


Fig. 4.104 – ST – Deformation when subjected to asymmetrical load.

4.7.3 - Longitudinal system

In the Figs. 4.105 and 4.106 it’s represented the stresses in the chosen trusses for axial forces, 4.107 and 4.108 for bending moments and from 4.109 and 4.110 for deformations.

Axial forces [kN]

For both situations the axial forces distribution is similar. The Upper plate is subjected to compression and the Lower plate and most of the other bracing elements are subjected to tension forces.

The scale factor presented by Sap2000 was 0.07.

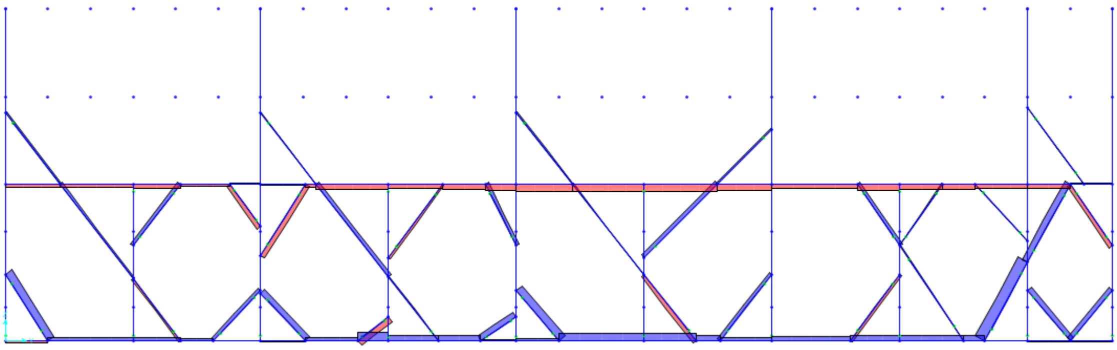


Fig. 4.105 – Longitudinal system - Axial forces when subjected to symmetrical loads.

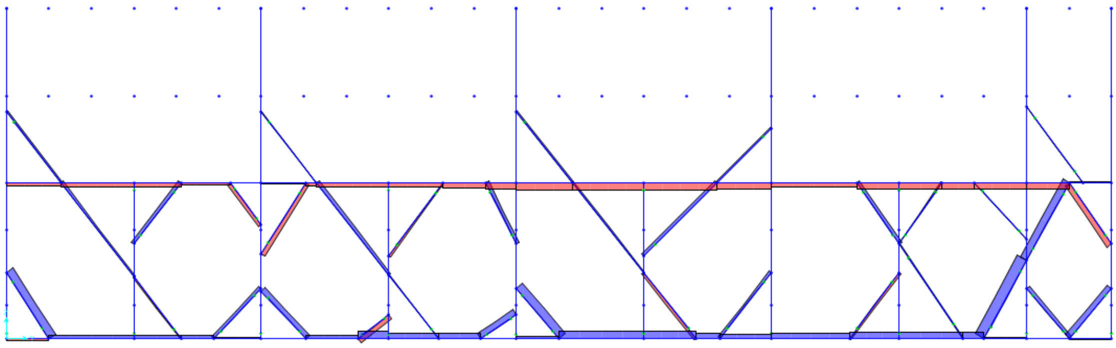


Fig. 4.106 – Longitudinal system - Axial forces when subjected to asymmetrical loads.

- Compression.

- Tension.

Bending moments [kN.m]

Like in the axial forces distribution, the bending moments are very similar either for symmetrical or asymmetrical loads. The Upper plate and the lower plate have bigger moments than the rest.

The scale factor presented by Sap2000 was 0.4.

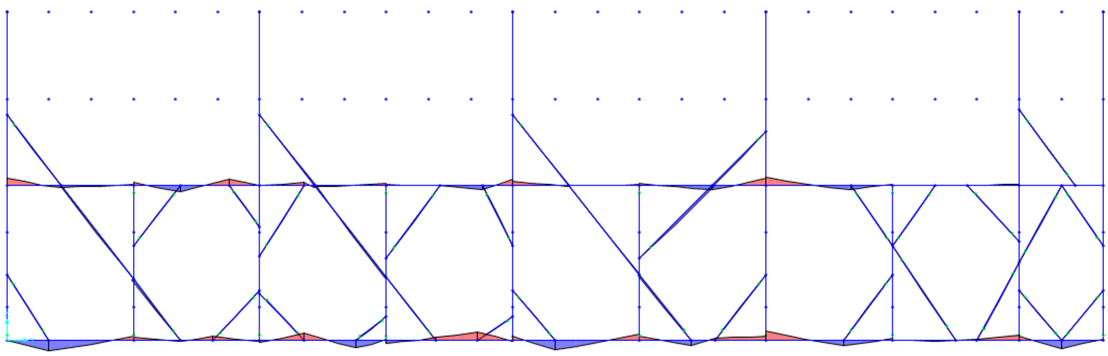


Fig. 4.107 – Longitudinal system – Bending moments when subjected to symmetrical loads.

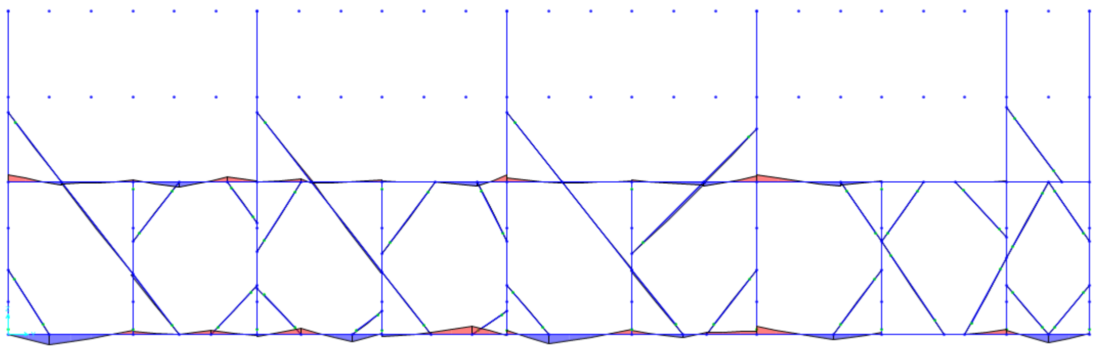


Fig. 4.108 – Longitudinal system – Bending moments when subjected to asymmetrical loads.

- Negative bending moments.

- Positive bending moments.

Deformation [m]

Comparing the main longitudinal plane of the roof structure it is seeable that the deformed shape is very similar, with symmetrical loads or without. The Upper plate and the lower plate have bigger deformations than the rest, as well as the top of double King posts.

The scale factor presented by Sap2000 was 200.

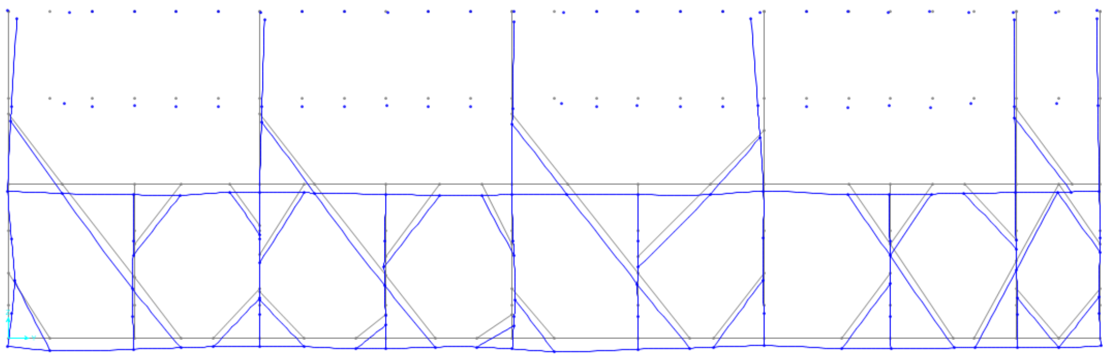


Fig. 4.109 – Longitudinal system – Bending moments when subjected to symmetrical loads.

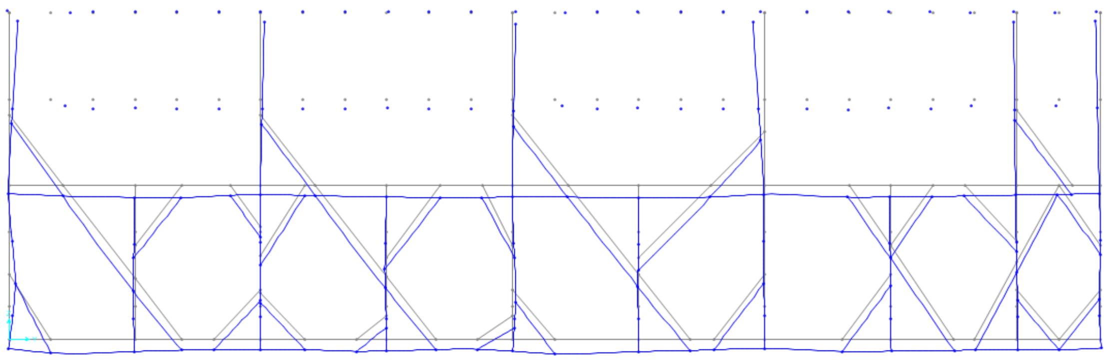


Fig. 4.110 – Longitudinal system – Bending moments when subjected to symmetrical loads.

4.8 – Modeling of the structure without missing elements

In order to see the behavior of the structure without the missing elements (mentioned in the roof conditions), a new model was created. The longitudinal angle brace without wooden peg was considered as not having a connection with the lower plate. Because of the advanced state of deterioration of the wall plate between trusses 2 and 3, their pin supports were considered as not existent.

The result was a failure in the Ultimate limit states in combined bending with axial tension or compression (biggest value from expressions 4.23 to 4.26 was 2.48 > 1.0) and columns subjected to either compression or combined compression and bending (biggest value from expressions 4.27 and 4.28 was 6.15>1.0).

The results are clearly the outcome of a very conservative model, since even with the damaged Wall plate, the Tie-beam will still be supported by the bearing wall (although the long term consequence can be the element's deformation). Despite the not realistic model, the consequences for the elements decay and the importance of proper connections between elements are noteworthy. To avoid structural problems, its original state should be returned.

4.9 - Interventions

As there were no studies accomplished in order to know the present conditions of the timber elements, the modeling was limited to what can be visible to the eye and strength grading assumption. Because all the security verifications prove the structure to be safe, means that no supplementary elements or changes in the structure need to be done and the intention of the interventions will be a return to the structure's original condition.

Until some tests can be done to understand better the qualities and fragilities of the structure (for instance why some of the tie beams on the south side were entrapped by one additional timber element in each side), the following interventions are proposed (always keeping in mind that the painted ceiling is what gives the building its individuality and cultural significance and therefore should not be affected):

- General cleaning of the structure, removing the dust and spider webs for a better vision of the structure and its flaws;
- Verify the character of the decay (active or passive) and the actual extent of the insect attack recorded and its consequences in the wood;
- Spread an anti-insect product to prevent the attack to continue;
- Replace the shingles and laths for new ones, using similar ones that will respect the esthetic of the building and its history;

- Replace the wooden pegs in the rafter connections (most susceptible location) for new ones using the ancient techniques. Some of them are soft to touch, especially in the north side;
- Replace missing wooden pegs and keep an eye on some connections where the elements are still connected with the wood peg but with some gaps;
- Replace the knee brace that was taken away from the first truss for entry reasons and study another way in;
- Removal the two decayed tie beams in the first transversal frame and replace them for a single good one (this procedure have to be done without the shingles and laths);
- Safeguard the elements with fire and insect protection;

The interventions mentioned above should be applied, preferably in proposed order, aiming first to eradicate the most urgent problems and their causes and afterwards concentrate on restoring the elements subjected to them.

Possible solution for tie-beam substitution in the first truss

In order to remove the two deteriorated tie beams in the first truss and its replacement for a new single one in safe conditions, a solution is proposed, without moving the ceiling and therefore not compromising it.

The first step of the proposed solution would be to shore up the ceiling with the aid a transversal beam and polystyrene foam placed under the tie beams (Figs. 4.111 to 4.113). This way when the connecting nails of the ceiling with the tie beams are removed (Fig. 4.114), the ceiling will "rest" on the polystyrene without moving out from its original position.

After the tie beams and all the other elements of the structure are disconnected, the truss should be lifted (at this point the shingles and laths must be removed) and the two deteriorated tie beams shall be replaced with a new single one (Figs. 4.115 and 4.116).

At last the truss shall be placed again in its place, the connections with the ceiling reestablished and the shoring elements removed (Figs. 4.117 and 4.118).

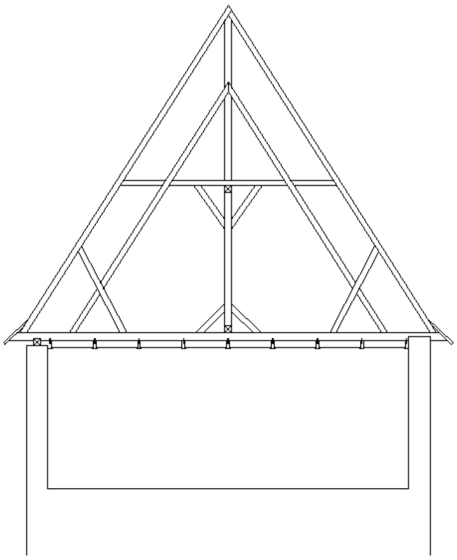


Fig. 4.111 – Initial state.

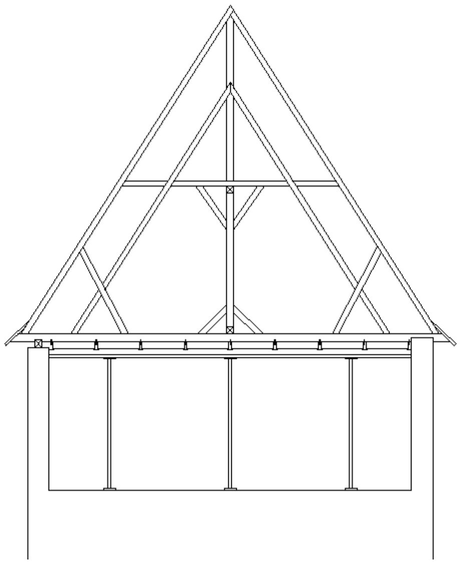


Fig. 4.112 – Positioning of shoring and transverse beam.

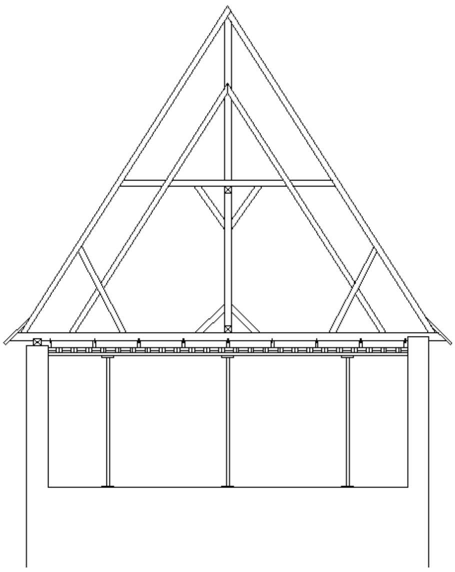


Fig. 4.113 – Positioning of polystyrene foam between the ceiling and transverse beam.

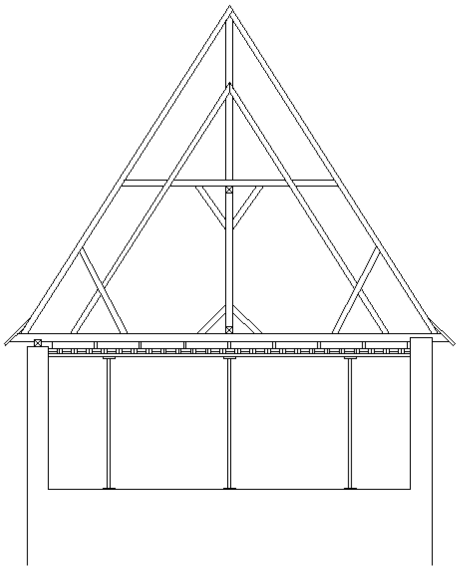


Fig. 4.114 – Removal of connecting nails between Tie-beam and the ceiling.

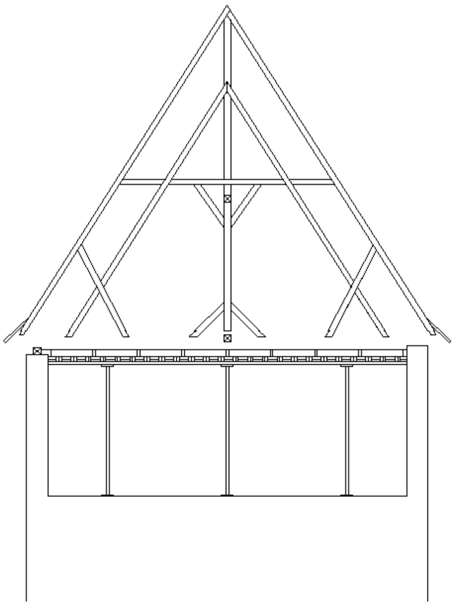


Fig. 4.115 – Connections removal, lifting of the truss to remove the two Tie-beams and its replacement for a new single one.

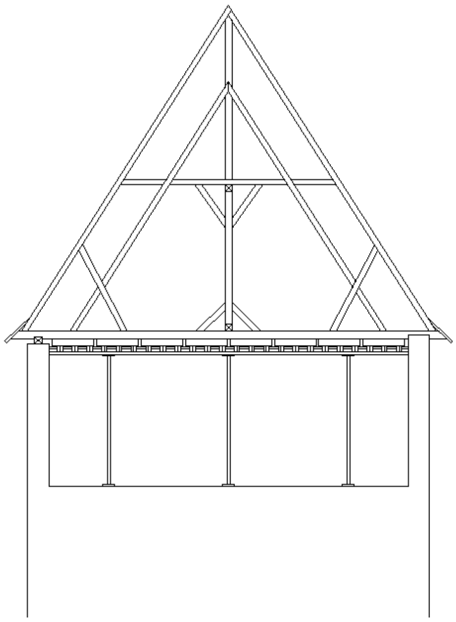


Fig. 4.116 – Truss with new Tie-beam attached.

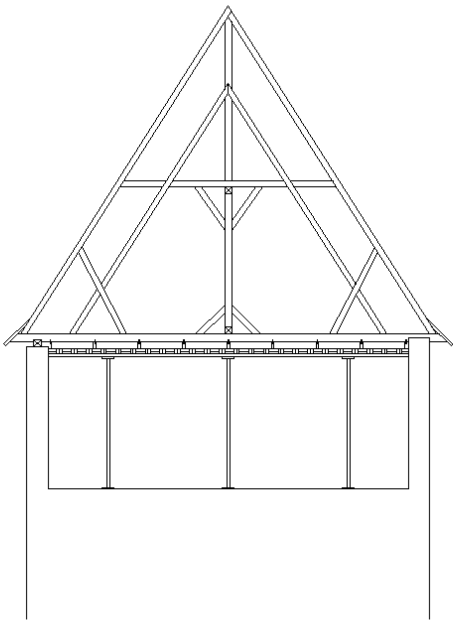


Fig. 4.117 – Setting of new connecting screws/nails between Tie-beam and the ceiling.

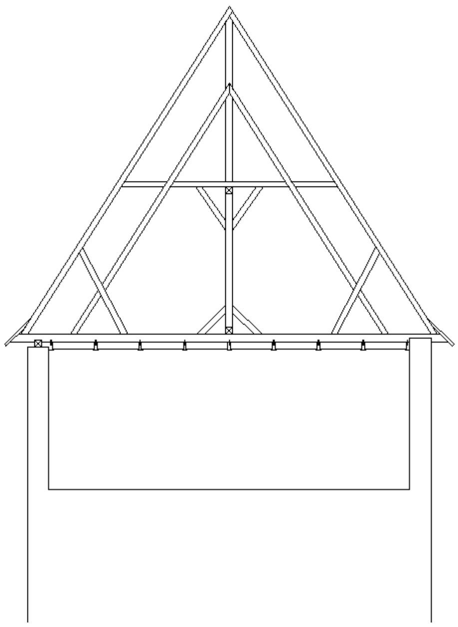


Fig. 4.118 – Removal of polystyrene, shoring and transverse beam.

4.10 – Maintenance plan

In order to ensure a better accessibility and security conditions to inspect and maintain the roof it is proposed to build a platform in timber (Fig. 4.119) or light metallic.

After the proposed interventions are taken care of, at least one visit a year is advised. Preferably in the summer time to identify possible increasing deformation and biological attacks that may occur in the winter.

The fire protection and biological products should be checked for its validity (should be specified in the catalogues) and applied again when necessary.

The shingles and laths due to external elements exposure and material decay, should be replaced every 25/50 years [39], but local replacement should be made whenever the damage or decay, that limit the structure protection against exterior elements like water infiltrations, is identified

To verify the safety of the structure for the maintenance tasks (operative and ladder (M)), a concentrated force of 0.75kN half way between the low Knee brace and Angle brace of the truss with the bigger stresses (25), was considered. This load was considered only acting with the permanent and snow loads. The wind action inclusion would be too conservative (that would mean all actions acting at the same time). The result for the Ultimate limit states in combined bending with axial tension or compression (higher value) was 0.37 < 1.0.

$$\sum 1.35 * \mathrm{G} + 1.5 * \mathrm{M} + \sum 1.5 * 0.7 * \mathrm{S} \qquad (4.32)$$

Fig. 4.119 – Timber maintenance platform [68].

5 - Conclusion

It is noteworthy that for these historical structures of important significance in human civilization as part of human heritage culture, individuality, local authenticity, there are still not European norms of security specifically made to their peculiarities. Only with the possibility of relying on principles like some of the ones debated here, the ICOMOS [35] made for the kind of structures discussed here but with no mathematical support to give, the ICOMOS [36] made for structural heritage in general but with the same problems as the one before and the ISO 13822 [38] applicable to any type of structure or material (not specifically for wood behavior) to check the reliability of the structure, this standard attends cost saving and not the importance of heritage preservation itself and its special demands.

Another concern about historical roof structures is the prevision of the material properties and consequently its mechanical behavior. Such structure/elements can't be subjected to destructive and semi destructive tests and should only be made with major precautions. These kind of tests, however lead to more precise and useful results (specially the destructive ones), when compared with the non-destructive tests. Using only non-destructive tests it is necessary to resort to visual grading, to predict modulus of elasticity and strength that according to Machado et al. [45] underestimate the true values up to 200% or 600% respectively.

The connection between elements is another subject that demands more investigation. For modeling matters it is normal to make them hinged or rigid, being on the safe side or better yet, overestimating the stresses in the elements when considering the connection hinged. However none of the considered connections are correct, as they are capable of resist a certain amount of bending moment. It is necessary to evaluate better the real performance of these connections for a more accurate determination of the structure behavior.

All the subjects mentioned above are of the higher importance to understand properly the structures and prepare a correct intervention plan to protect them from being gone, not only from lack of maintenance but also incorrect interventions induced by lack of knowledge.

In the Huedin reformed church nave, despite the gothic character of the structure, the year of the roof construction is not exactly known. A dating method for the wood like dendrochronology research (now getting established in Transylvania), should be accomplished to clear, if the construction predates or is dated after the earthquake of 1765 which led to severe damage to the church. The dendrological study is planned for 2014, this way the structures construction date can be connected to the date from the painted paneled ceiling.

Due to complications with the rafter- king post connection in the first attempted structural model, it was necessary to find solutions to get a closer structural behavior. The solution was to create a 1cm gap between the rafters and the king post.

Regarding the structural ultimate limit states, they were verified for the original structural configuration thus the interventions to be made should only be to return the structure its earliest qualities. The most solicited elements were the rafters, in particular on the 25^{th} truss with the most unfavorable stresses, what was predictable since the wind and snow loads are acting in these structural elements. Although the structure was constructed so many years ago without engineering theoretic support, it was well thought and designed in a way that it verifies nowadays standards.

The interventions itself in this structure due to its well preserved elements are not extensive but certain semi or non-destructive tests to better understand the timber properties, the decay like an insect attack and fungal extension should be done preceded with a general cleaning of the structure. For the proposed interventions it is noteworthy the one for tie-beam substitution in the first truss due to its higher complexity and necessary extra caution dealing with the historic ceiling of the church.

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Appendix 1 – Different models comparison

In order to create a solution to a first model attempt, where the king post was in tension (what was impossible since the rafters and the king post were not fixed to each other), two solutions were studied. One of the solutions was to interrupt the king post 1cm before it reached the rafters, creating a gap between them (Fig. A.1), the other solution was to impose a maximum tension of zero to the king post between the rafters and its interception with the compound rafters (Fig. A.2). Comparing the results for example for the expression/combination 4.30 on MT1', it was visible that both solutions had very similar results (see Figs. A.3 and A.4, A.5 and A6, A.7 and A.8). However due to a more realistic deformation of the king post (possibility to disconnect itself from the rafters) the solution with the gap was adopted.

$$\sum 1.35 * G + 1.5 * S \quad (4.30)$$

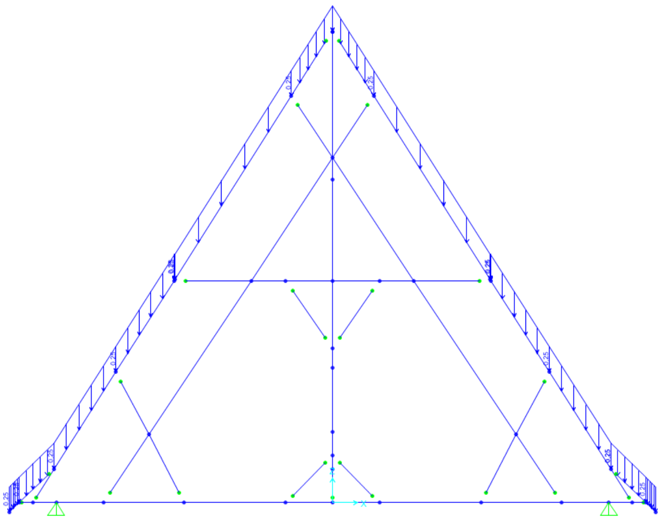


Fig. A.1 - King post not connected to rafters – model with snow loads.

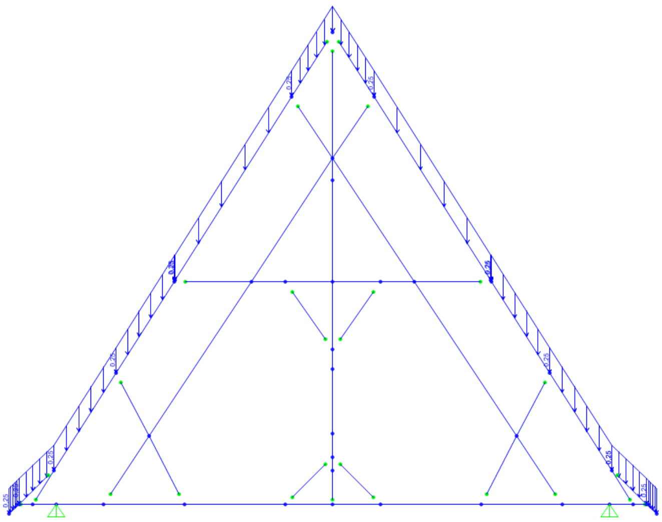


Fig. A.2 – King post connected to rafters but with the tension limited to zero - model with snow loads.

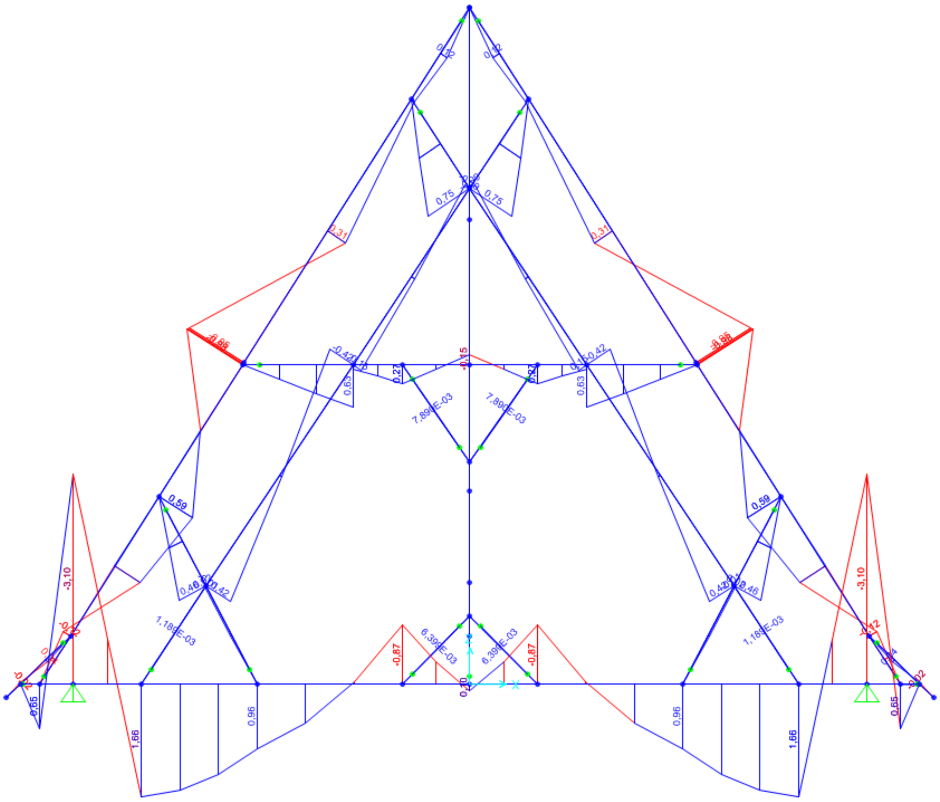


Fig. A.3 - Bending moments [kNm] – king post not connected to rafters.

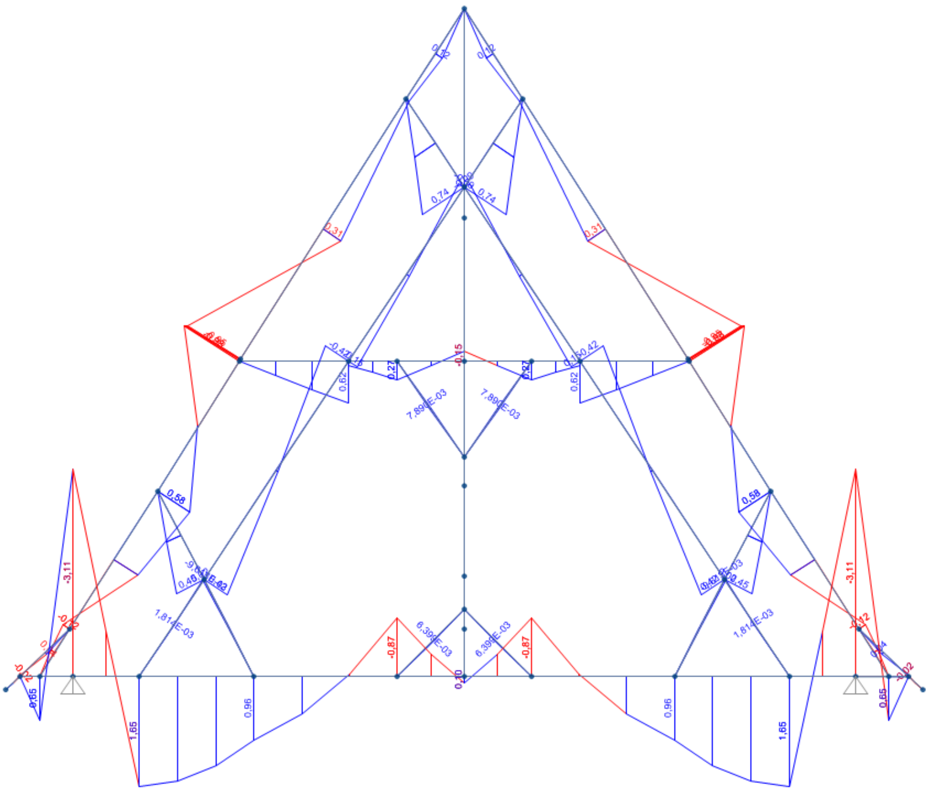


Fig. A.4 - Bending moments [kNm] – king post connected to rafters but with the tension limited to zero.

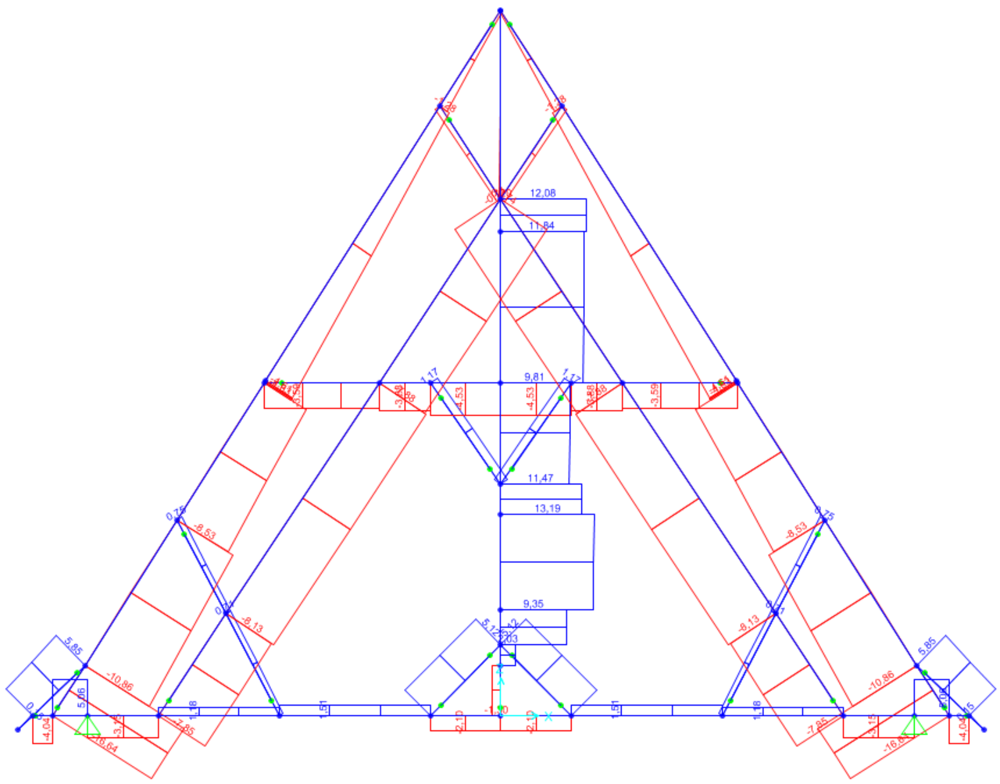


Fig. A.5 - Axial forces [kN] – king post not connected to rafters.

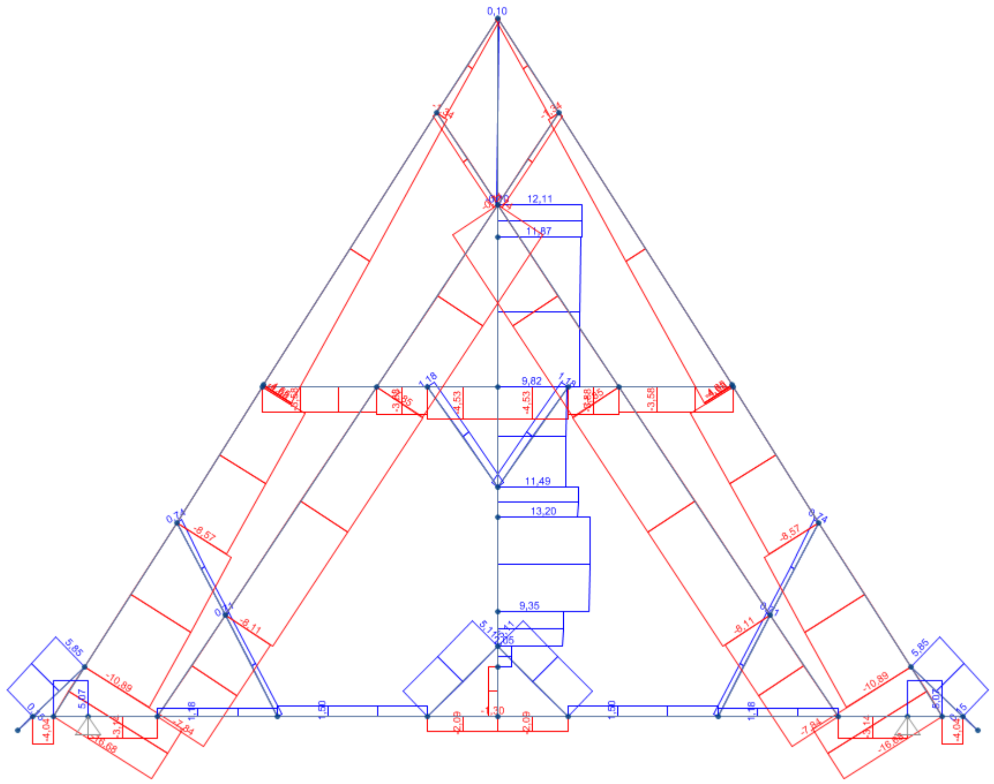


Fig. A.6 - Axial forces [kN] – king post connected to rafters but with the tension limited to zero.

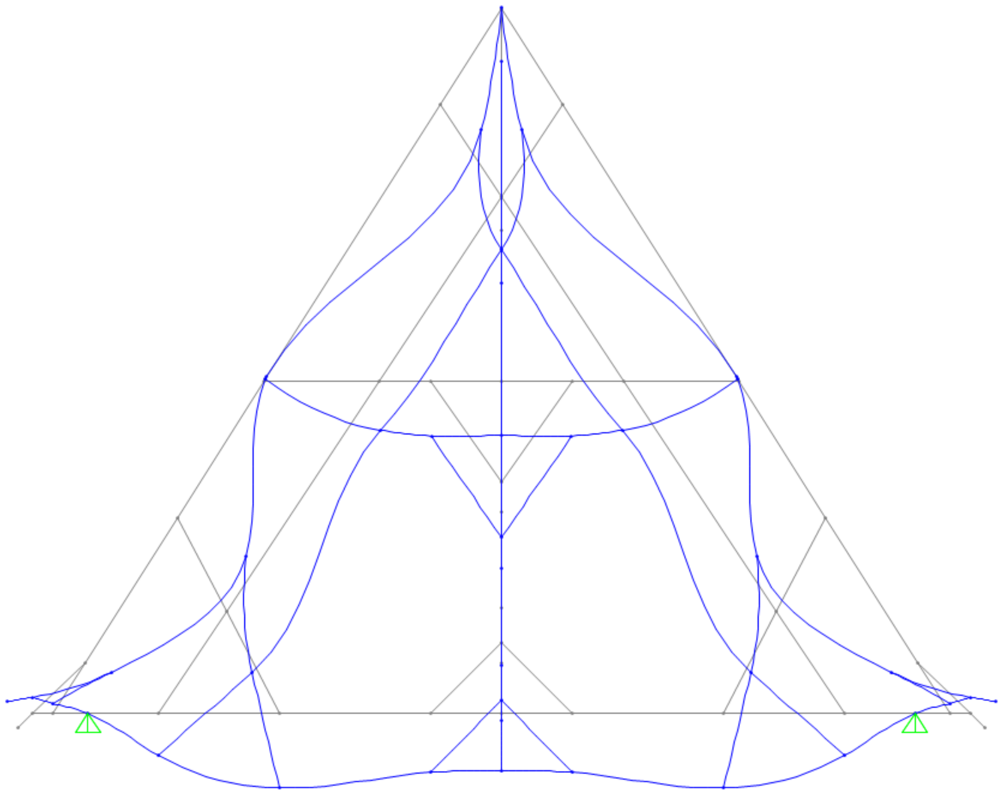


Fig. A.6 - Deformation [m] – king post not connected to rafters.

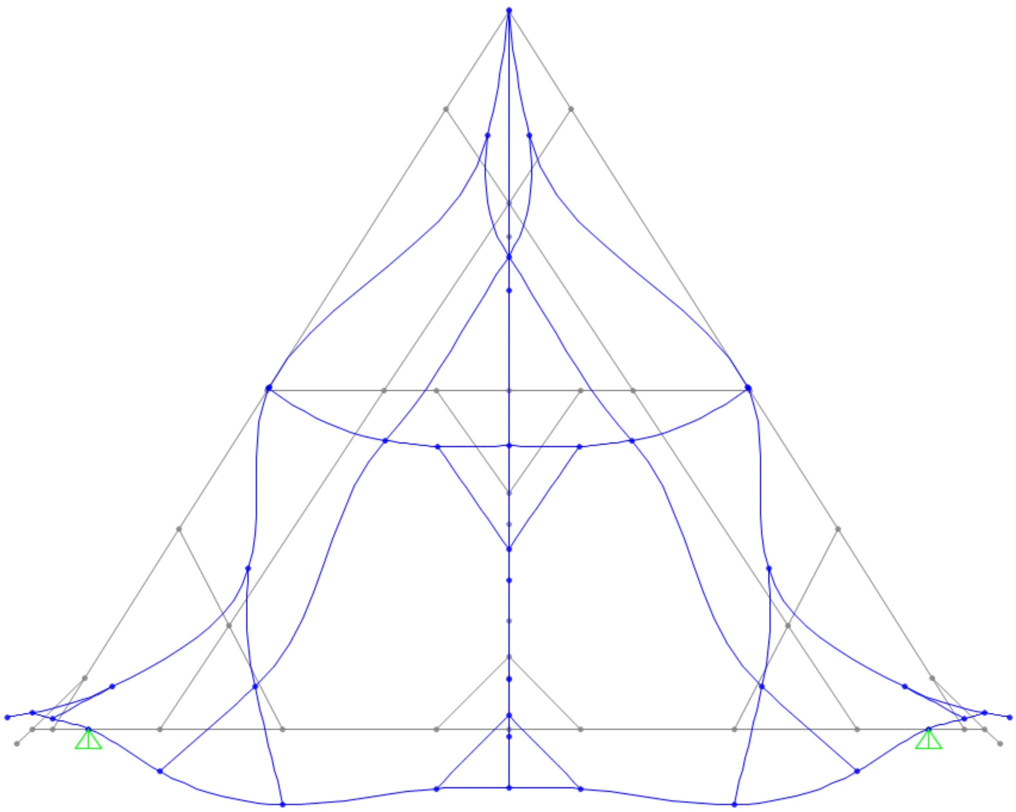


Fig. A.7 – Deformation [m] – king post connected to rafters but with the tension limited to zero.

Appendix 2 – Longitudinal bracing system

Appendix 3 – Main Truss 1’

Annex 1 – Influence of the joints stiffeness

(Branco, Cruz, Piazza and Varum, 2006) [9]

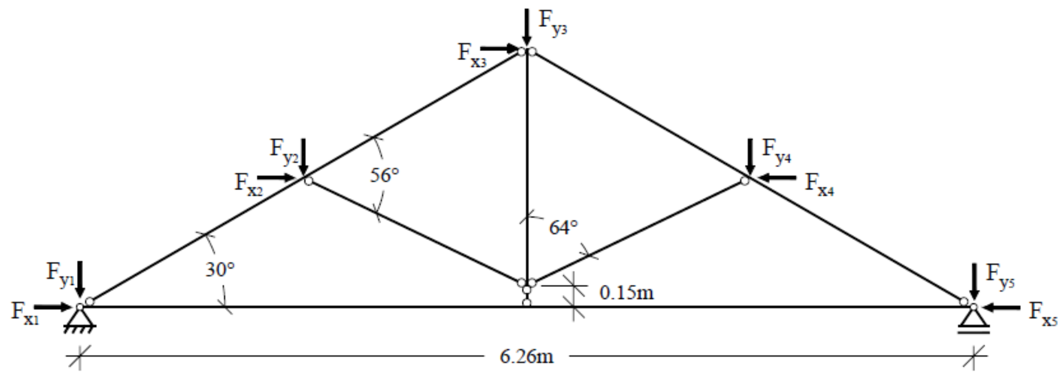
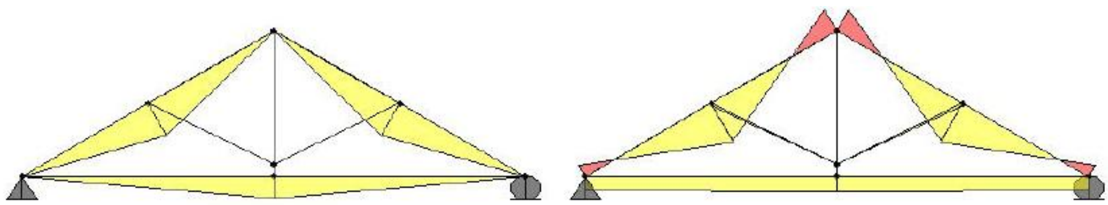


Fig. D.1 – Model adopted for the traditional Portuguese roof structure

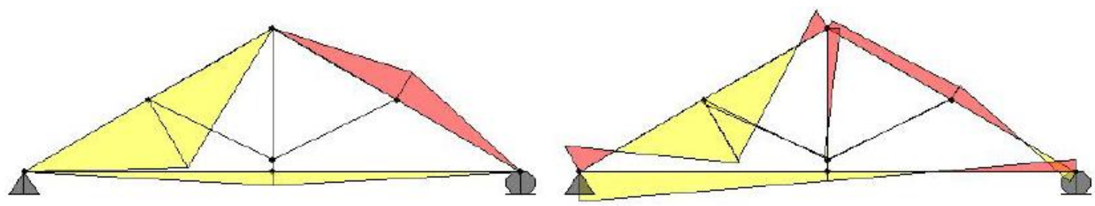
Loading case	Fx1	Fy1	Fx2	Fy2	Fx3	Fy3	Fx4	Fy4	Fx5	Fy5
Self-weight	-	2,51	-	4,64	-	4,64	-	4,64	-	2,51
Snow	-	3,33	-	6,65	-	5,54	-	4,43	-	2,22
Mass Force 1	2,51	-	4,64	-	4,64	-	-4,64	-	-2,51	-

Table A.1 – Loads in kN, for each loading case considered



(a) Perfect hinges *(b) Rigid joints*

Fig. D.2 - Bending moment diagrams for self-weight load case



(a) Perfect hinges *(b) Rigid joints*

Fig. D.3 - Bending moment diagrams for snow load case

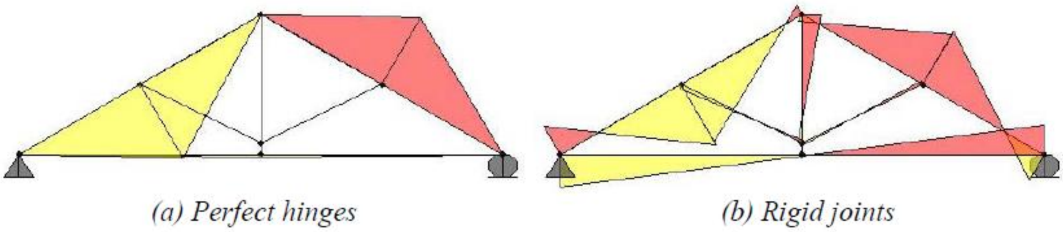


Fig. D.4 - Bending moment diagrams for mass force 1 load case

Annex 2 - Actions

The action values applied on the structure will be according to the specified on the Eurocode 1: Actions on structures (EC1) in particular EC1-1 General Actions: Densities, self-weight, imposed loads for buildings, EC1-3 General Actions: Snow Loads and General Actions: EC1-4 Wind Loads.

The seismic action was not considered in this work since only the roof part of the building was analyzed and it was not gathered information related to the rest of the building.

Imposed loads

On roofs, imposed loads, and snow loads or wind actions should not be applied together simultaneously.

Roof categories

Roofs shall be categorized according to their accessibility into three categories as shown in Table E.1:

Categories of loaded area	Specific Use
H	Roofs not accessible except for normal maintenance and repair.
I	Roofs accessible with occupancy according to categories A to D
K	Roofs accessible for special services, such as helicopter landing areas

Table E.1 - Categorization of roofs [23].

For roofs separate verifications shall be performed for the concentrated load Q_k and the uniformly distributed load q_k, acting independently.

For category H (Roofs not accessible roof except for normal maintenance and repair) according to the national annex from Romania, the q_k can by either 0,5 or 0,75 kN/m^2 depending if the roof pitch is bigger or not than 1:20. The Q_k should be determined based on the support technology presented for the project. If there is possibility to circulate with resource to boards, the Q_k is 0,5kN otherwise it is 1,0kN. The Q_k should be placed in the most unfavorable position in a $10x10cm^2$ surface and if this load is considered the max snow load should be $0,7kN/m^2$.

In the national annex it is referred that if the imposed load is more prejudicial than the snow, then the imposed loads should replace the snow loads (Table E.2 – note 2).

Acoperiş	q_k [kN/m^2]	Q_k [kN]
Categoria H Acoperişuri necirculabile, accesibile numai pentru întreţinere şi reparaţii uzuale	0,5$^{1)}$ sau 0,75$^{1)}$	$Q_k$$^{2)}$

1) Acoperişuri şi terase necirculabile cu:
- panta >1:20 — 0,5 kN/m^2
- panta ≤1:20 — 0,75 kN/m^2

2) Încărcarea concentrata Q_k se determină pe baza datelor tehnologice prezentate în tema de proiectare.
Dacă datele tehnologice nu prevăd valori mai mari, încărcarea concentrată Q_k se consideră:
- acoperişuri şi terase — 1,0 kN
- acoperişuri pe care se poate circula numai cu ajutorul podinelor — 0,5 kN.

NOTA 1 - Încărcarea q_k se raportează la proiecţia orizontală a suprafeţei acoperişului.
NOTA 2 - Încărcarea q_k înlocuieşte încărcarea dată de zăpadă, numai dacă este mai defavorabilă decât aceasta.
NOTA 3 - La acoperişurile cu panta de 1:3 sau mai mică se ia în considerare, suplimentar, încărcarea cu praf industrial, astfel:
a) în vecinătatea oţelăriilor cu agregate de insuflare de oxigen:
- până la 100 m de sursa de praf — 1,0 kN/m^2
- peste 100 m până la 500 m de sursa de praf — 0,5 kN/m^2

b) în vecinătatea furnalelor sau centralelor termoelectrice pe cărbune:
- până la 100 m de sursa de praf — 0,5 kN/m^2
- peste 100 m până la 1000 m de sursa de praf — 0,25 kN/m^2.

NOTA 4 - Încărcarea Q_k se aplică în poziţia cea mai defavorabilă, pe o suprafaţă de 10x10cm^2, fără a se lua în considerare alte încărcări tehnologice sau climatice.
NOTA 5 - La acoperişuri sau terase se ia considerare şi ipoteza de solicitare din încărcarea concentrată Q_k şi încărcarea dată de zăpadă care, în acest caz, nu poate avea o valoare mai mare de 0,7 kN/m^2.

Table E.2 - Imposed loads on roofs of category H – Romanian national Annex[61].

Snow load on roofs

Nature of the load

Properties of a roof or other factors causing different patterns can include:

a) the shape of the roof;

b) its thermal properties;

c) the roughness of its surface;

d) the amount of heat generated under the roof;

e) the proximity of nearby buildings;

f) the surrounding terrain;

g) the local meteorological climate, in particular its windiness, temperature variations, and likelihood of precipitation (either as rain or as snow);

Load arrangements

The following two primary load arrangements shall be taken into account:

- undrifted snow load on roofs ;
- drifted snow load on roofs;

Snow loads on roofs shall be determined as follows:

a) for the persistent / transient design situations

$$s = \mu_i * C_e * C_t * s_k \quad \text{(E.1)}$$

b) For the accidental design situations where exceptional snow load is the accidental action (except for the cases covered in c))

$$s = \mu_i * C_e * C_t * s_{Ad} \quad \text{(E.2)}$$

$$s_{Ad} = C_{esl} * s_k \tag{E.3}$$

c) For the accidental design situations where exceptional snow drift is the accidental action and where Annex B applies:

$$s = \mu_i * s_k \tag{E.4}$$

μ_i – is the snow load shape coefficient;

s_k – is the characteristic value of snow load on tile ground;

s_{Ad} – is the design value of exceptional snow load on the ground for a given location;

C_e – is the exposure coefficient;

C_t – is the thermal coefficient;

C_{esl} – is the coefficient for exceptional snow loads;

The characteristic value of snow load on tile ground s_k, should be found in the Eurocode (national) annexes.

C_e should be taken as 1,0 unless otherwise specified for different topographies.

Topography	C_e
Windswept [a]	0,8
Normal [b]	1,0
Sheltered [c]	1,2

[a] *Windswept topography*: flat unobstructed areas exposed on all sides without, or little shelter afforded by terrain, higher construction works or trees.

[b] *Normal topography*: areas where there is no significant removal of snow by wind on construction work, because of terrain, other construction works or trees.

[c] *Sheltered topography*: areas in which the construction work being considered is considerably lower than the surrounding terrain or surrounded by high trees and/or surrounded by higher construction works.

Table E.3 - Recommended values of C_e for different topographies [24].

The thermal coefficient C_t should be used to account for the reduction of snow loads on roofs with high thermal transmittance (> 1 W/m^{2}K), in particular for some glass covered roofs, because of melting caused by heat loss. For other cases should be considered 1.0.

The recommended value for C_{esl} is 2.0 unless specified in the national annex.

The load should be assumed to act vertically and refer to a horizontal projection of the roof area.

Roof shape coefficients

The snow load shape coefficients depend either if the roof is monopitch, pitched, Multi-span roofs, cylindrical or for roofs abutting and close to taller construction works. Each of two μ_1 and μ_2 coefficients should be taken from the table below.

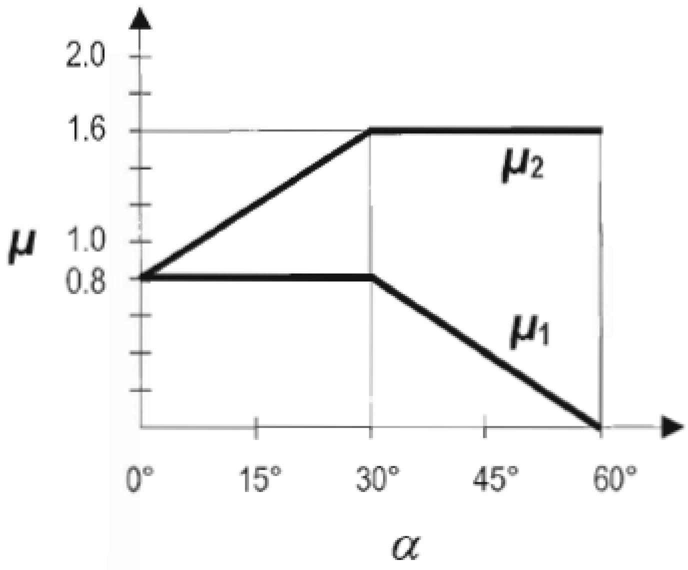


Fig. E.1 – Snow load shape coefficients [24].

Angle of pitch of roof α	$0° \leq \alpha \leq 30°$	$30° < \alpha < 60°$	$\alpha \geq 60°$
μ_1	0,8	0.8(60 - α)/30	0,0
μ_2	0,8 + 0,8 α/30	1,6	--

Table E.4 - Snow load shape coefficients [24].

For pitched roofs:

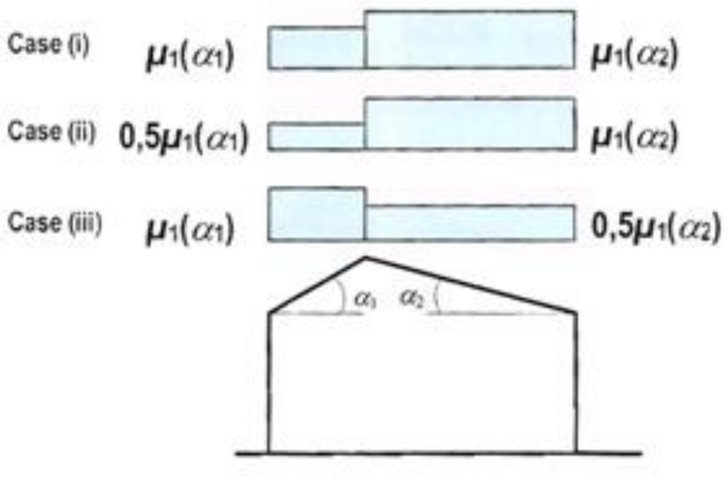


Fig. E.2 – Snow load shape coefficients – Pitched roofs [24].

Local effects

The following local effects should be taken in consideration:

- Drifting at projections and obstructions
- Snow overhanging the edge of a roof
- Snow loads on snowguards and other obstacles

Wind

Wind pressure on surfaces

The wind pressure acting on the external surfaces should be obtained from expression:

$$w_e = q_p(z_e) * c_{pe} \tag{E.5}$$

$q_p(z_e) -$ is the peak velocity pressure;

$z_e -$ is the reference height for the external pressure given ahead;

$c_{pe} -$ is the pressure coefficient for the external pressure;

The wind pressure acting on the internal surfaces of a structure should be obtained from expression:

$$w_i = q_p(z_i) * c_{pi} \tag{E.6}$$

$q_p(z_i) -$ is the peak velocity pressure;

$z_i -$ is the reference height for the interior pressure given ahead;

$c_{pi} -$ is the pressure coefficient for the internal pressure;

The process to get to this pressure is to get the basic wind velocity from Eurocode (national) annexes, calculate the Basic wind velocity then the Mean wind, Wind turbulence and with the last two the Peak velocity pressure. The pressure coefficients are given in the section 7 of EC1-4 for some series of generic cases. Large and considerable higher neighbouring structures and closely spaced buildings and obstacles should be considered.

The net pressure on a wall, roof or element is the difference between the pressures on the opposite surfaces taking due account of their signs. Pressure, directed towards the surface is taken as positive, and suction, directed away from the surface as negative. Examples are given in Figure E.3.

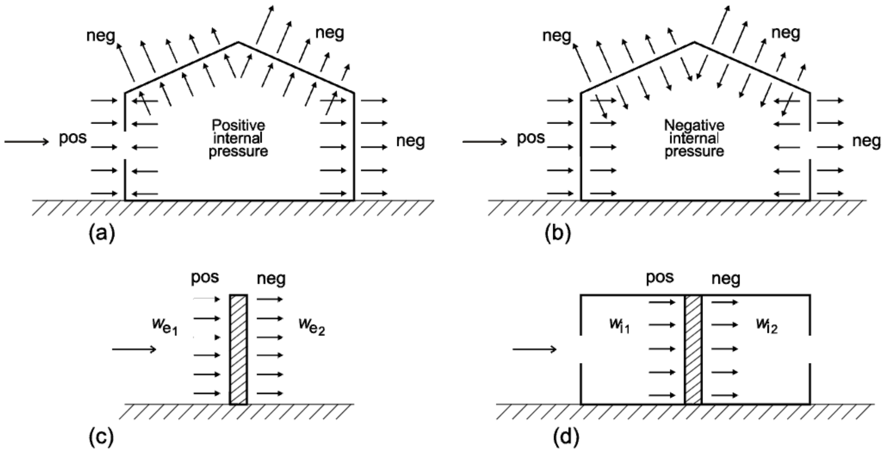


Fig. E.3 – Pressure on surfaces [25].

The basic wind velocity

$$v_b = c_{dir} * c_{season} * v_{b,0}$$

v_b – is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category II;

c_{dir} – is the directional factor. The recommended value is 1.0 unless specified contrarily in national annex;

c_{season} – is the season factor. The recommended value is 1.0 unless specified contrarily in national annex;

$v_{b,0}$ – is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category II;

The fundamental value of the basic wind velocity is the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights.

Mean wind

The mean wind velocity $v_m(z)$ at a height z above the terrain depends on the terrain roughness and orography and on the basic wind velocity, v_b, and should be determined using the following expression:

$$v_m(z) = c_r(z) * c_o(z) * v_b \quad \text{(E.7)}$$

c_r – is the roughness factor;

c_o – is the orography factor;

$$c_r(z) = k_r * \ln(\frac{z}{z_0}) \qquad \text{for } z_{min} \leq z \leq z_{max} \quad \text{(E.8)}$$

$$c_r(z) = c_r(z_{min}) \qquad \text{for } z \leq z_{min} \tag{E.9}$$

z_0 - is the roughness length;

k_r - terrain factor depending on the roughness length z_0 calculated using:

$$k_r = 0.19 * \left(\frac{z_0}{z_{0,II}}\right)^{0.07} \tag{E.10}$$

$z_{0,II}$ – 0.05 m (terrain category II);

z_{min} - is the minimum height defined in Table E.6 from the Eurocode 1991-1-4;

z_{max} - is to be taken as 200 m, unless otherwise specified in the National Annex;

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
NOTE: The terrain categories are illustrated in A.1.			

Table E.6 – Terrain categories and terrain parameters [25].

The terrain roughness to be used for a given wind direction depends on the ground roughness and the distance with uniform terrain roughness in an angular sector around the wind direction. Small areas (less than 10% of the area under consideration) with deviating roughness may be ignored. (Figure E.4.)

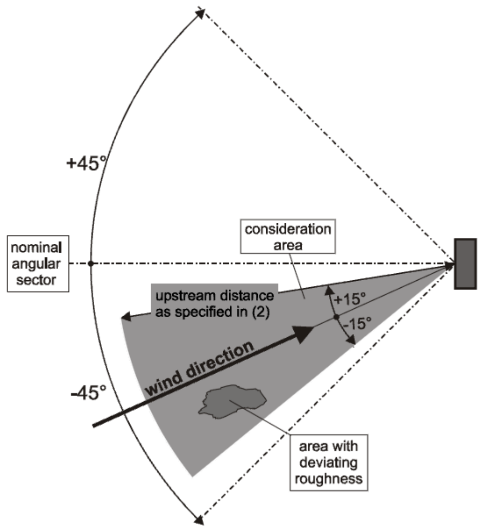


Fig. E.4 – Assessment of terrain roughness [25].

Where orography (e.g. hills, cliffs etc.) increases wind velocities by more than 5% the effects should be taken into account using the orography factor c_0. The procedure to be used for determining c_0 may be given in the National Annex. The recommended procedure is given in A.3 of the Eurocode.

The effects of orography may be neglected when the average slope of the upwind terrain is less than 3°. The upwind terrain may be considered up to a distance of 10 times the height of the isolated orographic feature.

Wind turbulence

The turbulence intensity $I_v(z)$ at height z is defined as the standard deviation of the turbulence divided by the mean wind velocity.

$$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_0(z) * \ln\left(\frac{z}{z_0}\right)} \qquad \text{for } z_{min} \leq z \leq z_{max} \qquad (E.11)$$

$$I_v(z) = I_v(z_{min}) \qquad \text{for } z \leq z_{min} \qquad (E.12)$$

k_I - is the turbulence factor with the recommended value of 1,0 unless contrarily specified in the National annex.

Peak velocity pressure

The peak velocity pressure $q_p(z)$ at height z, which includes mean and short-term velocity fluctuations, should be determined through the following expression:

$$q_p = (1 + 7 * I_v) * 0.5 * \rho * v_m^2(z) \qquad (E.13)$$

ρ - is the air density, which depends on the altitude, temperature and barometric pressure to be expected in the region during wind storms; the recommended value should be 1.25 kg/m^3 or indicated in National annex.

Pressure coefficients for buildings

The external pressure coefficients C_{pe} for buildings and parts of buildings depend on the size of the loaded area (A), which is the area of the structure that produces the wind action in the section to be calculated. The external pressure coefficients are given for loaded areas of 1 m^2 and 10m^2 in the tables of Eurocode 1-4 chapter 7 [25] (for duopitch roofs see Tables E.7 and E.8) for the appropriate building configurations as $C_{pe,1}$, for local coefficients, and $C_{pe,10}$, for overall coefficients, respectively.

Values for $C_{pe,1}$ are intended for the design of small elements and fixings with an area per element of 1 m^2 or less such as cladding elements and roofing elements. Values for $C_{pe,10}$ may be used for the design of the overall load bearing structure of buildings.

For loaded areas above 1 m^2 and up to 10 m^2, the procedure for calculating external pressure coefficients was based on external pressure coefficients $C_{pe,1}$ and $C_{pe,10}$ (Fig. E.5).

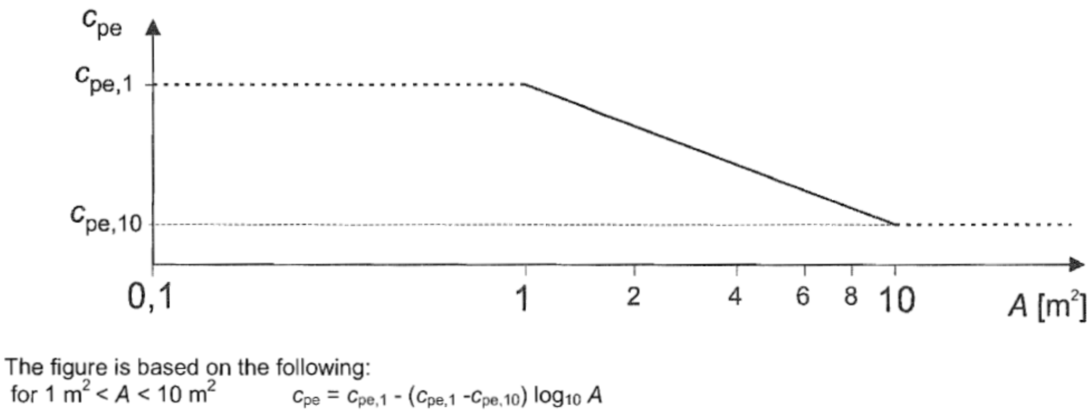


Fig. E.5 – Recommended procedure for determining the external pressure coefficient c_{pe} for buildings with a loaded area A between 1 m^2 and 10 m^2 [25].

The values $C_{pe,10}$ and $C_{pe,1}$ in Eurocode tables should be used for the orthogonal wind directions 0°, 90°, 180°. These values represent the most unfavorable values obtained in a range of wind direction θ= ± 45° either side of the relevant orthogonal direction

For protruding roof corners the pressure on the underside of the roof overhang is equal to the pressure for the zone of the vertical wall directly connected to the protruding roof; the pressure at the top side of the roof overhang is equal to the pressure of the zone, defined for the roof (see Fig. E.6).

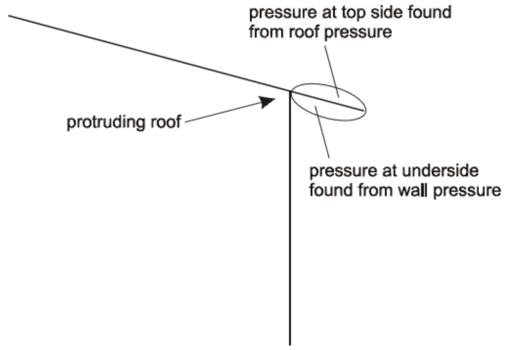


Fig. E.6 – Illustration of relevant pressures for protruding roofs [25].

Duopitch roofs:

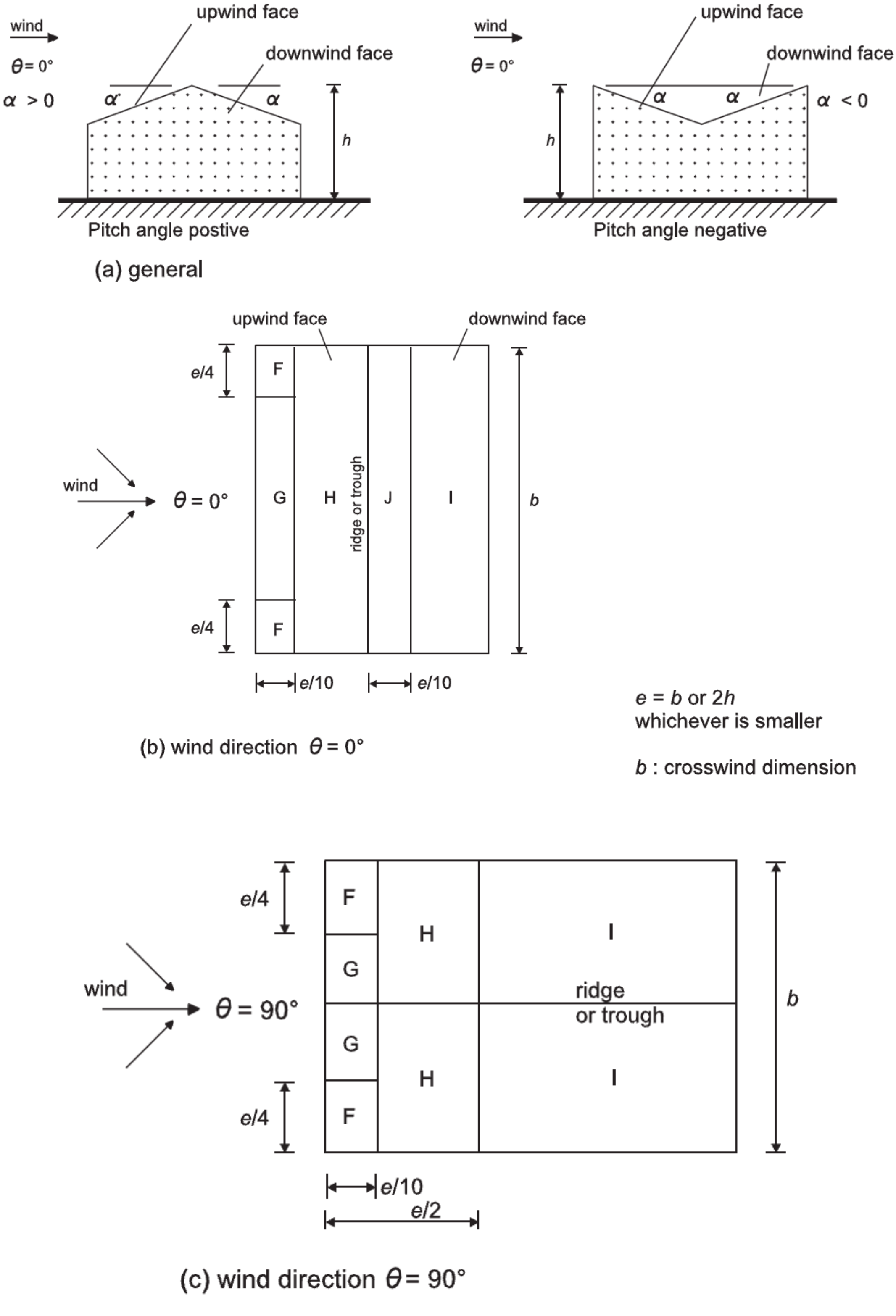


Fig. E.7 – Key for duopitch roofs [25].

Pitch Angle α	Zone for wind direction $\theta = 0°$									
	F		G		H		I		J	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-0,6		-0,6		-0,8		-0,7		-1,0	-1,5
-30°	-1,1	-2,0	-0,8	-1,5	-0,8		-0,6		-0,8	-1,4
-15°	-2,5	-2,8	-1,3	-2,0	-0,9	-1,2	-0,5		-0,7	-1,2
-5°	-2,3	-2,5	-1,2	-2,0	-0,8	-1,2	+0,2		+0,2	
							-0,6		-0,6	
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-0,6		+0,2	
	+0,0		+0,0		+0,0				-0,6	
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-0,4		-1,0	-1,5
	+0,2		+0,2		+0,2		+0,0		+0,0	+0,0
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-0,4		-0,5	
	+0,7		+0,7		+0,4		+0,0		+0,0	
45°	-0,0		-0,0		-0,0		-0,2		-0,3	
	+0,7		+0,7		+0,6		+0,0		+0,0	
60°	+0,7		+0,7		+0,7		-0,2		-0,3	
75°	+0,8		+0,8		+0,8		-0,2		-0,3	

NOTE 1 At $\theta = 0°$ the pressure changes rapidly between positive and negative values on the windward face around a pitch angle of $\alpha = -5°$ to $+45°$, so both positive and negative values are given. For those roofs, four cases should be considered where the largest or smallest values of all areas F, G and H are combined with the largest or smallest values in areas I and J. No mixing of positive and negative values is allowed on the same face.

NOTE 2 Linear interpolation for intermediate pitch angles of the same sign may be used between values of the same sign. (Do not interpolate between $\alpha = +5°$ and $\alpha = -5°$, but use the data for flat roofs in 7.2.3). The values equal to 0,0 are given for interpolation purposes

Table E.7 – Recommended values of external pressure coefficients for duopitch roofs – 0º [25].

Pitch angle α	Zone for wind direction $\theta = 90°$							
	F		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-1,4	-2,0	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-30°	-1,5	-2,1	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-15°	-1,9	-2,5	-1,2	-2,0	-0,8	-1,2	-0,8	-1,2
-5°	-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	-0,6	-1,2
5°	-1,6	-2,2	-1,3	-2,0	-0,7	-1,2	-0,6	
15°	-1,3	-2,0	-1,3	-2,0	-0,6	-1,2	-0,5	
30°	-1,1	-1,5	-1,4	-2,0	-0,8	-1,2	-0,5	
45°	-1,1	-1,5	-1,4	-2,0	-0,9	-1,2	-0,5	
60°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	
75°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	

Table E.8 – Recommended values of external pressure coefficients for duopitch roofs – 90º [25].

Internal pressure

Internal and external pressures shall be considered to act at the same time. The worst combination of external and internal pressures shall be considered for every combination of possible openings and other leakage paths.

The internal pressure coefficient, C_{pi}, depends on the size and distribution of the openings in the building envelope. When in at least two sides of the buildings (facades or roof) the total area of openings in each side is more than 30 % of the area of that side, the actions on the structure should not be calculated from the rules given in this and previous sections but the rules of 7.3 (Canopy roofs) and 7.4 (Free-standing walls, parapets, fences and signboards) of the Eurocode should instead be used.

Where an external opening, such as a door or a window, would be dominant when open but is considered to be closed in the ultimate limit state, during severe windstorms, the condition with the door or window open should be considered as an accidental design situation in accordance with EN 1990 [22].

A face of a building should be regarded as dominant when the area of openings at that face is at least twice the area of openings and leakages in the remaining faces of the building considered.

When the area of the openings at the dominant face is twice the area of the openings in the remaining faces:

$$C_{pi} = 0.75 * C_{pe} \tag{E.14}$$

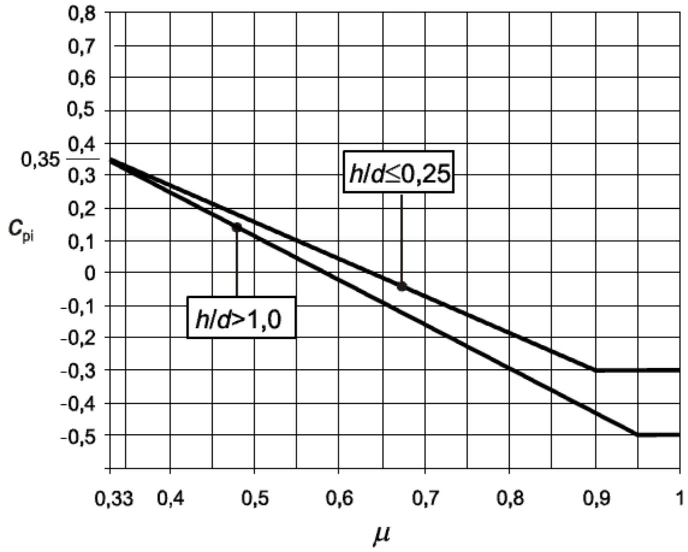
When the area of the openings at the dominant face is at least 3 times the area of the openings in the remaining faces,

$$C_{pi} = 0.90 * C_{pe} \tag{E.15}$$

When these openings are located in zones with different values of external pressures an area weighted average value of C_{pe} should be used.

When the area of the openings at the dominant face is between 2 and 3 times the area of the openings in the remaining faces linear interpolation for calculating C_{pi} may be used.

For buildings without a dominant face, the internal pressure coefficient C_{pi} should be determined from Figure E.8, and is a function of the ratio of the height and the depth of the building, h/d, and the opening ratio μ for each wind direction θ, which should be determined from the following expression.



NOTE For values between *h*/*d* = 0,25 and *h*/*d* = 1,0 linear interpolation may be used.

Fig. E.8 – Internal pressure coef'ficients for uniformly distributed openings [25].

$$\mu = \frac{\sum area\ of\ openings\ where\ C_{pe}\ is\ negative\ or -0{,}0}{\sum area\ of\ all\ openings} \tag{E.16}$$

Where it is not possible, or not considered justified, to estimate μ for a particular case then C_{pi} should be taken as the more onerous of +0.2 and -0.3.

The internal pressure coefficient of open silos and chimneys should be -0.6.

Vertical walls of rectangular plan buildings:

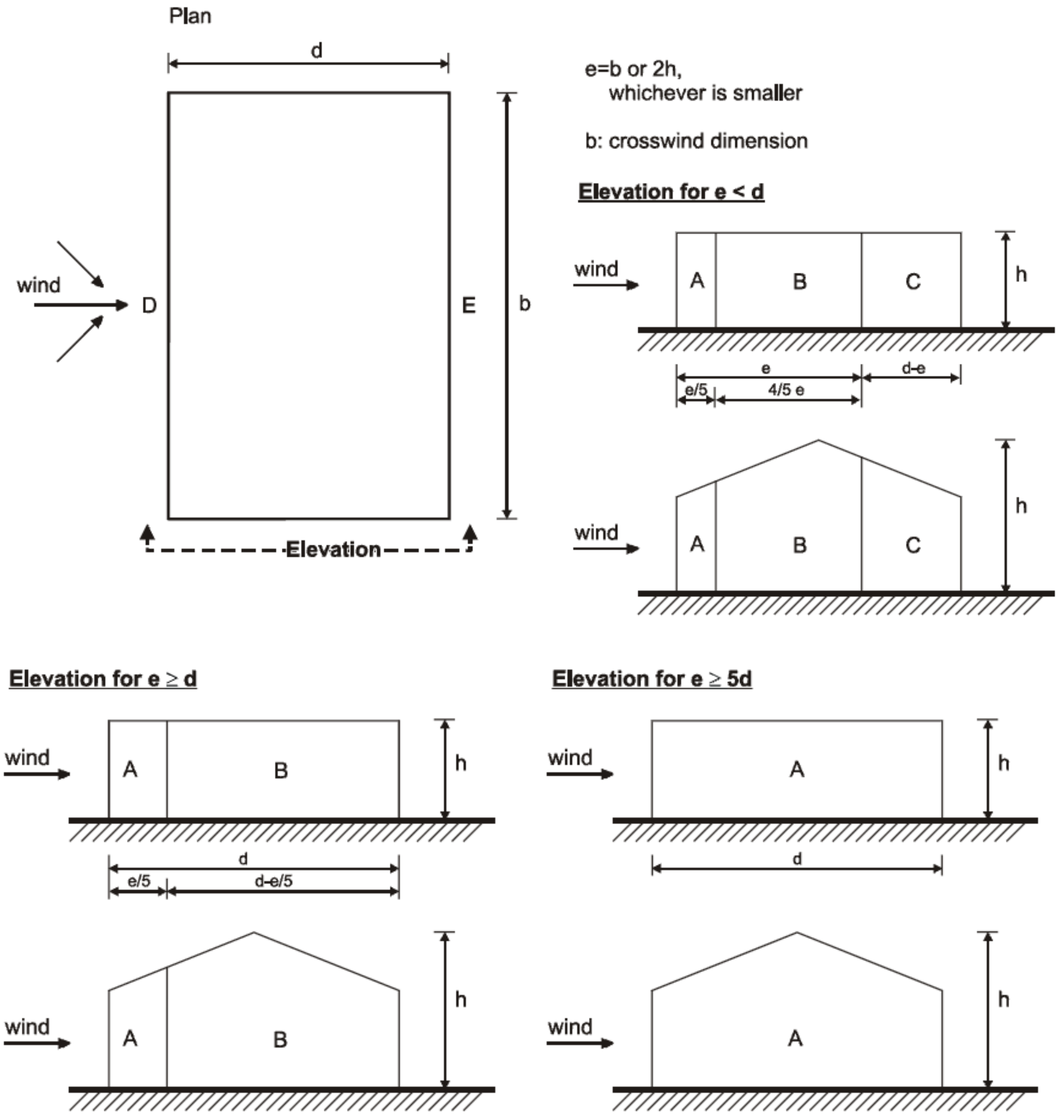


Fig. E.9 – Key for vertical walls [25].

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

Table E.9 – Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings [25].

Wind forces

The wind forces for the whole structure or a structural component should be determined:

by calculating forces using force coefficients (see (a)) or

by calculating forces from surface pressures (see (b))

(a) The wind force Fw acting on a structure or a structural component may be determined directly by using the following expression:

$$F_w = c_s c_d * c_f * q_p(z_e) * A_{ref} \tag{E.17}$$

or by vectorial summation over the individual structural elements (as shown in 7.2.2 of Eurocode) by using the following expression:

$$F_w = c_s c_d * \sum_{elements} c_f * q_p(z_e) * A_{ref} \tag{E.18}$$

$c_s c_d -$ is the structural factor as defined in Section 6 of eurocode;

$c_f -$ is the force coefficient for the structure or structural element, given in Section 7 or Section 8 of EC1-4;

$q_p(z_e) -$ is the peak velocity pressure at reference height z_e (defined in Section 7 or Section 8 of EC1-4);

$A_{ref} -$ is the reference area of the structure or structural element, given in Section 7 or Section 8 of EC1-4;

(b) The wind force, Fw acting on a structure or a structural element may be determined by vectorial summation of the forces F_w,e, $F_{w,i}$ and F_{fr} calculated from the external and internal pressures and the frictional forces resulting from the friction of the wind parallel to the external surfaces:

- external forces:

$$F_{w,e} = c_s\, c_d * \sum_{surfaces} w_e * A_{ref} \tag{E.19}$$

- internal forces:

$$F_{w,i} = \sum_{surfaces} w_i * A_{ref} \tag{E.20}$$

- friction forces:

$$F_{fr} = c_{fr} * q_p(z_e) * A_{fr} \tag{E.21}$$

$c_s\, c_d -$ is the structural factor as defined in Section 6 of EC1-4;

$w_e -$ is the external pressure on the individual surface at height z_e;

$w_i -$ is the internal pressure on the individual surface at height z_i;

$A_{ref} -$ is the reference area of the individual surface

c_{fr} − is the friction coefficient derived from EC1-4 7.5;

A_{fr} − is the area of external surface parallel to the wind, given in EC1-4 7.5;

The structural factor $c_s\, c_d$ can be considered iqual to 1.0 for:

a) For buildings with a height less than 15 m;

b) For facade and roof elements having a natural frequency greater than 5 Hz;

c) For framed buildings which have structural walls and which are less than 100 m high and whose height is less than 4 times the in-wind depth;

For other cases a detailed procedure is indicated in EC1-4 6.3.1.

Annex 3 - Eurocode 5 – Design of timber structures

Considering the nature of the studied structure, it will only be approached EC5-1-1: Common rules and rules for buildings, and within this Eurocode only the applications alluding to solid timber.

Basic variables

Actions and environmental influences

Duration of load and moisture content affect the strength and stiffness properties of timber and wood-based elements and shall be taken into account in the design for mechanical resistance and serviceability.

Actions caused by the effects of moisture content changes in the timber shall be taken into account.

Load duration classes

The load-duration classes are characterized by the effect of a constant load acting for a certain period of time in the life of the structure. For a variable action the appropriate class shall be determined on the basis of an estimate of the typical variation of the load with time.

Actions shall be assigned to one of the load-duration classes given in Tables F.1 and F.2 for strength and stiffness calculations.

Load-duration class	Order of accumulated duration of characteristic load
Permanent	more than 10 years
Long-term	6 months – 10 years
Medium-term	1 week – 6 months
Short-term	less than one week
Instantaneous	

Table F.1 – Load-duration classes [26].

Load-duration class	Examples of loading
Permanent	self-weight
Long-term	storage
Medium-term	imposed floor load, snow
Short-term	snow, wind
Instantaneous	wind, accidental load

Table F.2 – Examples of load-duration assignment [26].

Service classes

The service class system is mainly aimed at assigning strength values and for calculating deformations under defined environmental conditions.

Service class 1 is characterized by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 65% for a few weeks per year. In service class 1 the average moisture content in most softwoods will not exceed 12 %.

Service class 2 is characterized by a moisture content in the materials corresponding to a temperature of 20°C and the relative humidity of the surrounding air only exceeding 85 % for a few weeks per year. In service class 2 the average moisture content in most softwoods will not exceed 20 %.

Service class 3 is characterized by climatic conditions leading to higher moisture contents than in service class 2.

Materials and product properties

When a structure is made from different material properties, this should be taken into consideration and the proper adjustments in modification and deformation factors, modulus of elasticity, shear modulus and slip modulus shall be taken.

Verification by the partial factor method

The recommended partial factors for material properties (γ_M) are given in Table F.3.

Fundamental combinations:	
Solid timber	1,3
Glued laminated timber	1,25
LVL, plywood, OSB,	1,2
Particleboards	1,3
Fibreboards, hard	1,3
Fibreboards, medium	1,3
Fibreboards, MDF	1,3
Fibreboards, soft	1,3
Connections	1,3
Punched metal plate fasteners	1,25
Accidental combinations	1,0

Table F.3 - Recommended partial factors γ_M for material properties and resistances [26].

Design value of material

The design value X_d of a strength property shall be calculated as:

$$X_d = k_{mod} * \frac{X_k}{\gamma_M} \tag{F.1}$$

X_d – is the characteristic value of a strength property;

γ_M – is the partial factor for a material property;

k_{mod} – is a modification factor taking into account the effect of the duration of load and moisture content.

The design member stiffness property E_d or G_d shall be calculated as:

$$E_d = \frac{E_{mean}}{\gamma_M} \tag{F.2}$$

$$G_d = \frac{G_{mean}}{\gamma_M} \tag{F.3}$$

E_{mean} – is the mean value of modulus of elasticity;

G_{mean} – is the mean value of shear modulus.

Design resistances

The design value R_d of a resistance (load-carrying capacity) shall be calculated as:

$$R_d = k_{mod} * \frac{R_k}{\gamma_M} \tag{F.4}$$

R_d – is the characteristic value of load-carrying capacity;

γ_M – is the partial factor for a material property;

k_{mod} – is a modification factor taking into account the effect of the duration of load and moisture content.

Material properties

Strength modification factors for service classes and load-duration classes

If a load combination consists of actions belonging to different load-duration classes a value of k_{mod} should be chosen which corresponds to the action with the shortest duration, e.g. for a combination of dead load and a short-term load, a value of k_{mod} corresponding to the short-term load should be used.

For solid timber the values of the modification factor are given in Table F.4.

Material	Standard	Service class	Load-duration class				
			Permanent action	Long term action	Medium term action	Short term action	Instanta-neous action
Solid timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90

Table F.4 - Values of k_{mod} – Solid timber [26].

Deformation modification factors for service classes

The values of the deformation factors k_{def} given in Table F.5 should be used

Material	Standard	Service class		
		1	2	3
Solid timber	EN 14081-1	0,60	0,80	2,00

Table F.5 - Values of k_{def} – Solid timber [26].

Deformation modification factors for service classes

For rectangular solid timber with a characteristic timber density $\rho_k \leq 700$ kg/m^3, the reference depth in bending or width (maximum cross-sectional dimension) in tension is 150mm. For depths in bending or widths in tension of solid timber less than 150 mm the characteristic values for $f_{m,k}$ and $f_{t,0,k}$ may be increased by the factor k_h, given by:

$$k_h = \min\begin{cases}\left(\frac{150}{h}\right)^{0.2} \\ 1.3\end{cases} \tag{F.5}$$

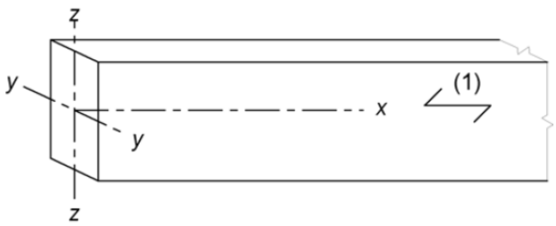
$h -$ is the depth for bending members or width for tension members, in mm;

For timber which is installed at or near its fibre saturation point, and which is likely to dry out under load, the values of k_{def}, given in Table F.5, should be increased by 1.0.

Ultimate limit states

Design of cross-sections subjected to stress in one principal direction

The member is assumed to be subjected to stresses in the direction of only one of its principal axes (Fig. F.1).



Key:
(1) direction of grain

Fig. F.1 – Member axes [26].

Tension parallel to the grain

The following expression shall be satisfied:

$$\sigma_{t,0,d} \leq f_{t,0,d} \quad \text{(F.6)}$$

$\sigma_{t,0,d}$ – is the design tensile stress along the grain;

$f_{t,0,d}$ – is the design tensile strength along the grain;

Tension perpendicular to the grain

The effect of member size shall be taken into account.

Compression parallel to the grain

(1)P The following expression shall be satisfied:

$$\sigma_{c,0,d} \leq f_{c,0,d} \quad \text{(F.7)}$$

$\sigma_{c,0,d}$ – is the design compressive stress along the grain;

$f_{c,0,d}$ – is the design compressive strength along the grain.

Compression perpendicular to the grain

The following expression shall be satisfied:

$$\sigma_{c,90,d} \leq k_{c,90,d} * f_{c,90,d} \quad \text{(F.8)}$$

$\sigma_{c,90,d}$ – is the design compressive stress in the contact area perpendicular to the grain;

$f_{c,90,d}$ – is the design compressive strength perpendicular to the grain;

$k_{c,90,d}$ – is a factor taking into account the load configuration, possibility of splitting and degree of compressive deformation;

The $k_{c,90,d}$ factor varies from 1,0 to 4,0. For examples and process of determination the section 6.1.5 of EC5-1-1 should be consulted.

Bending

The following expressions shall be satisfied:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \quad \text{(F.9)}$$

$$k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \quad \text{(F.10)}$$

$\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ – are the design bending stresses about the principal axes;

$f_{m,y,d}$ and $f_{m,z,d}$ – are the corresponding design bending strengths;

The factor k_m makes allowance for re-distribution of stresses and the effect of inhomogeneities of the material in a cross-section. The value of this factor should be 0,7 for rectangular sections and 1,0 for other cross sections.

Shear

For shear with a stress component parallel to the grain, see Figure F.2(a), as well as for shear with both stress components perpendicular to the grain, see Figure F.(b), the following expression shall be satisfied:

$$\tau_d \leq f_{v,d} \qquad \text{(F.11)}$$

$\tau_d -$ is the design shear stress;

$f_{v,d} -$ is the desig

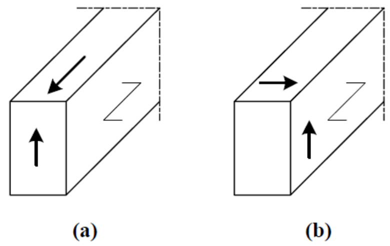


Fig. F.2 – (a) Member with a shear stress component parallel to the grain (b) Member with both stress components perpendicular to the grain (rolling shear) [26].

At supports, the contribution to the total shear force of a concentrated load F acting on the top side of the beam and within a distance h or h_{ef} from the edge of the support may be disregarded (see Figure F.3). For beams with a notch at the support this reduction in the shear force applies only when the notch is on the opposite side to the support.

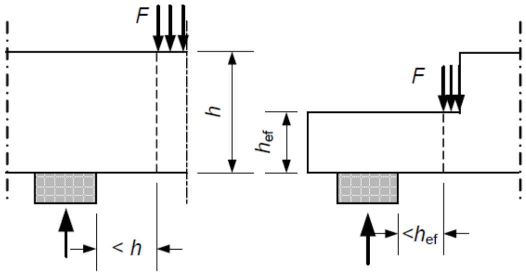


Fig. F.3 – Conditions at a support, for which the concentrated force *F* may be disregarded in the calculation of the shear force [26].

Torsion

The following expression shall be satisfied:

$$\tau_{tor,d} \leq k_{shape} * f_{v,d} \qquad \text{(F.12)}$$

$\tau_{tor,d} -$ is the design torsional stress;

$f_{v,d} -$ is the design shear strength;

$k_{shape}-$ is a factor depending on the shape of the cross-section;

$$k_{shape} = \begin{cases} 1.2 \; for\ a\ circular\ cross\ section \\ min \begin{cases} 1 + 0.15 * \frac{h}{b} \\ 2.0 \end{cases} \quad for\ a\ rectangular\ cross\ section \end{cases} \tag{F.13}$$

h - is the larger cross-sectional dimension;

b - is the smaller cross-sectional dimension;

Design of cross-sections subjected to combined stresses

Compression stresses at an angle to the grain

The compressive stresses at an angle α to the grain, (see Figure F.4), should satisfy the following expression:

$$\sigma_{c,\alpha,d} \leq \frac{f_{c,0,d}}{\frac{f_{c,0,d}}{k_{c,90} * f_{c,90,d}} * \sin^2\alpha + \cos^2\alpha} \tag{F.14}$$

$\sigma_{c,\alpha,d}-$ is the compressive stress at an angle α to the grain;

$k_{c,90}-$ is a factor taking into account the effect of any of stresses perpendicular to the grain.

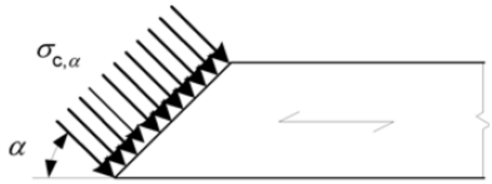


Fig. F.4 – Compressive stresses at an angle to the grain [26].

Combined bending and axial tension

The following expressions shall be satisfied:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{F.15}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{F.16}$$

Combined bending and axial compression

The following expressions shall be satisfied:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{F.17}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{F.18}$$

Stability of members

Columns subjected to either compression or combined compression and bending

The relative slenderness ratios should be taken as:

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} * \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \tag{F.19}$$

$$\lambda_{rel,z} = \frac{\lambda_z}{\pi} * \sqrt{\frac{f_{c,0,k}}{E_{0,05}}} \tag{F.20}$$

λ_y and $\lambda_{rel,y}-$ are slenderness ratios corresponding to bending about the y-axis (deflection in the z-direction);

λ_z and $\lambda_{rel,z}-$ are slenderness ratios corresponding to bending about the z-axis (deflection in the y-direction);

$E_{0,05}-$ is the fifth percentile value of the modulus of elasticity parallel to the grain;

Where both $\lambda_{rel,y} \leq 0.3$ and $\lambda_{rel,z} \leq 0.3$ the stresses should satisfy the expressions from Combined bending and axial compression.

In all other cases the stresses, which will be increased due to deflection, should satisfy the following expressions:

$$\frac{\sigma_{c,0,d}}{k_{c,y} * f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{F.21}$$

$$\frac{\sigma_{c,0,d}}{k_{c,z} * f_{c,0,d}} + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1.0 \tag{F.22}$$

Where:

$$k_{c,y} = \frac{1}{k_y + \sqrt{k_y^2 - \lambda^2_{rel,y}}} \tag{F.23}$$

$$k_{c,z} = \frac{1}{k_z + \sqrt{k_z^2 - \lambda^2_{rel,z}}} \tag{F.24}$$

$$k_y = 0.5 * \left(1 + \beta_c * \left(\lambda_{rel,y} - 0.3\right) + \lambda^2_{rel,y}\right) \tag{F.25}$$

$$k_z = 0.5 * \left(1 + \beta_c * \left(\lambda_{rel,z} - 0.3\right) + \lambda^2_{rel,z}\right) \tag{F.26}$$

$\beta_c -$ is a factor for members within the straightness limits defined in Section 10 of Eurocode; for solid timber is 0.2;

Beams subjected to either bending or combined bending and compression

Lateral torsional stability shall be verified both in the case where only a moment M_y exists about the strong axis y and where a combination of moment M_y and compressive force N_c exists.

The relative slenderness for bending should be taken as:

$$\lambda_{rel,m} = \sqrt{\frac{f_{m,k}}{\sigma_{m,crit}}} \tag{F.27}$$

$\sigma_{m,crit}$ – is the critical bending stress calculated according to the classical theory of stability, using 5-percentile stiffness values.

The critical bending stress should be taken as:

$$\sigma_{m,crit} = \frac{M_{y,crit}}{W_y} = \frac{\pi * \sqrt{E_{0,05} * I_z * G_{0,05} * I_{tor}}}{l_{ef} * W_y} \tag{F.28}$$

$E_{0,05}$ – is the fifth percentile value of modulus of elasticity parallel to grain;

$G_{0,05}$ – is the fifth percentile value of shear modulus parallel to grain;

I_z – is the second moment of area about the weak axis z;

I_{tor} – is the torsional moment of inertia;

l_{ef} – is the effective length of the beam, depending on the support conditions and the load configuration, acccording to Table F.6;

W_y – is the section modulus about the strong axis y;

For softwood with solid rectangular cross-section, $\sigma_{m,crit}$ should be taken as:

$$\sigma_{m,crit} = \frac{0.78 b^2}{h * l_{ef}} * E_{0.05} \tag{F.29}$$

b is the width of the beam;

h is the depth of the beam;

Beam type	**Loading type**	ℓ_{ef}/ℓ [a]
Simply supported	Constant moment Uniformly distributed load Concentrated force at the middle of the span	1,0 0,9 0,8
Cantilever	Uniformly distributed load Concentrated force at the free end	0,5 0,8

[a] The ratio between the effective length ℓ_{ef} and the span ℓ is valid for a beam with torsionally restrained supports and loaded at the centre of gravity. If the load is applied at the compression edge of the beam, ℓ_{ef} should be increased by 2*h* and may be decreased by 0,5*h* for a load at the tension edge of the beam.

Table F.6 – Effective length as a ratio of the span [26].

In the case where only a moment M_y exists about the strong axis y, the stresses should satisfy the following expression:

$$\sigma_{m,d} \leq k_{crit} f_{m,d} \tag{F.30}$$

$\sigma_{m,d}$ – is the design bending stress;

$f_{m,d}$ – is the design bending strength;

k_{crit} – is a factor which takes into account the reduced bending strength due to lateral buckling.

For beams with an initial lateral deviation from straightness within the limits defined in Section 10 of eurocode, k_{crit} may be determined from the following expression:

$$k_{crit} = \begin{cases} 1 \; for \; \lambda_{rel,m} \leq 0.75 \\ 1.56 - 0.75 * \lambda_{rel,m} \quad for \; 0.75 < \lambda_{rel,m} \leq 1.4 \\ \frac{1}{\lambda_{rel,m}^2} \; for \; 1.4 < \lambda_{rel,m} \end{cases} \tag{F.31}$$

The factor k_{crit} may be taken as 1.0 for a beam where lateral displacement of its compressive edge is prevented throughout its length and where torsional rotation is prevented at its supports.

In the case where a combination of moment M_y about the strong axis y and compressive force N_c exists, the stresses should satisfy the following expression:

$$\left(\frac{\sigma_{m,d}}{k_{crit} * f_{m,d}}\right)^2 + \frac{\sigma_{c,d}}{k_{c,z} * f_{c,0,d}} \leq 1.0 \tag{F.32}$$

$\sigma_{m,d}$ – is the design bending stress;

$\sigma_{c,d}$ – is the design compressive stress;

$f_{c,0,d}$ – is the design compressive strength parallel to grain;