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PART 2 - Investigation, analysis, and interventions

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Cover illustration: Auxiliary truss for the strengthening of the roof of San Giovanni Battista church, Borno, Brescia, Italy, 1771-81/2020. © Emanuele Zamperini (2020)

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EDITORIAL
REMARKABLE HISTORIC TIMBER ROOFS. KNOWLEDGE AND CONSERVATION PRACTICE
Part 2 - Investigation, analysis, and interventions

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Following the first volume – published in November 2022 – which contains the contributions on construction history, this second volume of the TEMA special issue on remarkable historic timber roofs is focused on investigations, analyses, and interventions for preservation.

The topic in question has been the subject of an impressive growth of interest in recent decades, consequent to a significant change in the approach to the material authenticity of architecture. The theories of restoration inspired by neo-idealistic or phenomenological philosophy – prevailing at least until the early 1980s – gave an absolute pre-eminence to the image of architecture, to the detriment of its material consistency, especially with reference to all those parts – such as usually happens to the timber roofs – which are hidden from view.

Still in the year 1990, Roberto Di Stefano – an authoritative Italian professor of architectural restoration, at the time president of ICOMOS (International Council on Monuments and Sites) – in his book on structural strengthening for restoration [1], cited as a virtuous example of restoration the complete demolition of the 14th-century timber roof of the Cathedral of Naples. In line with the aforementioned theories of restoration, the roof replacement with steel trusses carried out between 1969 and 1972 was declared admissible because the wooden structures «did not interest the interior architectural spaces».

However, as early as the 1970s, innovative intervention techniques for the restoration of timber structures had begun to be elaborated: both punctual interventions for the reintegration of decayed wood [2–3] and interventions for the strengthening of beams, trusses [4–5], or structural complexes. Whether they were interventions based on the use of innovative materials (e.g., prostheses made with the use of synthetic resins and composite materials) or the rediscovery and reinterpretation of historical techniques (e.g., the trussed beam [6]), they required laboratory and in situ analyses to evaluate their effectiveness. These studies launched a new research sector that could make use of the recent progress of the investigations on newly produced wooden elements for structural use, which in those years were advancing due to the requests connected to the new methods of assessing the safety of the structures.

In 1975, ICOMOS established its International Wood Committee, which has organized a series of international symposia. More recently, other international conferences have been held, like the one organized in Florence in February 2005 on Conservation of Historic Wooden Structures and the SHATIS series (Structural Health Assessment of Timber Structures), which since 2011 have usually taken place every two years.

From a technical point of view, the progressive increase of interest in existing timber structures, the accumulation of a significant amount of research, and international collaboration have led to the affirmation of reliable investigation methods and the drafting of a European standard containing guidelines for the on-site assessment of timber structures (EN 17121:2019). From a cultural point of view, in October 1999 – during the *12th General Assembly* of the ICOMOS held in Mexico City – the extension of interest in wooden structures heritage led to the adoption of a document drafted by the ICOMOS International Wood Committee (chaired by the late Gennaro Tampone between 2005 and 2016), entitled *Principles for the preservation of historic timber structures*. In December 2017, thanks to the continuous attention to the theme, the document was updated – with the new title *Principles for the conservation of wooden built heritage* – and adopted by ICOMOS at the *19th General* Assembly held in Delhi (India).

While having a significant function in disseminating culturally and methodologically valid approaches, these international documents – as all other restoration charters – are not exempt from accusations of cultural relativism: being written as a methodological tool to be used throughout the world, they do not take a clear-cut position on the intervention philosophy underlying the restoration or reinforcement works.

Indeed, if the investigation techniques are usually perceived as characterized by certain technical-scientific objectivity and the controversies on the preference to be attributed to one or another are limited to the diagnostic experts, the restoration and strengthening interventions are often the subject of lively discussions. These debates rise between two different – in some ways opposite – points of view on restoration. On one side, those who consider it more important to preserve the conception and behavior of the roof structure unaltered, even at the cost of partially sacrificing its material authenticity (among them, in Italy, we can mention Franco Laner). On the other side, there are those who, instead, believe that the material constituting the structure is the only guarantee of its authenticity and must therefore be preserved in its entirety, even when partially decayed (among these, we cannot forget the aforementioned Gennaro Tampone). The latter approach brings about the consequent need to modify the structural behavior of the existing structure with the addition of all those elements that are necessary to obviate its shortcomings.

Far from proposing a unified vision or an overall picture of the subject, this special issue of the scientific journal TEMA intends to propose a series of interesting studies on investigations, analyses, and interventions for the preservation of timber structures.

The article by Martina Diaz, Louis Vandenabeele, and Stefan Holzer offers the report of an articulated system of investigations on the timber domes of the Basilica of St. Anthony in Padua, which passes from an extensive geometrical survey of the entire church (carried out with laser scanning of the whole building and traditional hand-measurements of joints, repairs and other traces), to laboratory analyses, to a thorough examination of historical documentary sources. The investigations carried out make it possible to reconstruct the events relating to the construction and restoration of the wooden structures of the domes, also in relation to the underlying masonry structures and the more general technical and economic context of the region, and to provide a fundamental contribution for future preservation and strengthening works.

Davide Prati, Angelo Massafra, and Luca Guardigli present some conclusions they have been able to draw from research they have been carrying out for years on wide-span timber trusses in the area of Bologna. Also, in this case, the starting point is an accurate digital survey (made with a laser scanner) integrated by the in situ collection of other geometric and construction data and by the archival investigation. Thanks to 3D parametric modelling software, they could elaborate a series of analyses on the deformations of the timber trusses and then draw considerations on their stability in and out of the plane. The comparison of the various case studies also allowed them to reflect on the relationship between the geometric proportions of the structures and their structural efficiency.

Paolo Vannucci instead offers us an interesting study on the static behaviour of the Gothic age timber roof structures in France; starting from a static analysis of the roof of the church of Notre-Dame in Paris – tragically destroyed by the fire of 2019 – he elaborates some hypotheses on how medieval carpenters conceived the three-dimensional structural behaviour of the large timber roofing complexes built according to the so-called *chevrons formant ferme* system.

A series of articles proposes projects for the strengthening of specific timber roof structures, some of which have already been concretely realized. All these studies present a correct methodology which starts from the integration of the results of the historical study, of the geometric survey – as accurate as possible in relation to the concrete situation of the places before the intervention – of the recognition of the wood species, of the estimation of the material strength and decay, in order to define the most suitable preservation and reinforcement interventions. These articles are particularly interesting due to the variety of the investigation techniques used and – most of all – in relation to the construction types of the analyzed buildings and their histories, from which the variety and specificity of intervention techniques follow.

Lia Ferrari presents the case study of the 16th-century Palazzo Costabili in Ferrara (Emilia-Romagna, Italy). It is a large palace whose uncommon roof structures are the expression of local building techniques. The presence of strengthening interventions carried out at the end of the 1990s – that the author studied thanks to archival documents – and the damages suffered by the building in the 2012 earthquake make the structure even more complex and so its assessment and particularly interesting the designed strengthening intervention.

The article by Marco Zerbinatti, Alessandro Grazzini, Sara Fasana, and Giovanni Vercelli presents the realized strengthening work on the roof of a 16th-century chapel at the Sacro Monte of Orta (Piedmont, Italy), a UNESCO site. The geometric articulation of the chapel's hipped roof and the building stratifications that have occurred over time have determined a complicated structure, a few parts of which were decayed due to fungal attack. The strengthening work described is exemplary of the complexity that even the intervention on a small building can present.

Tanja Marzi, Clara Bertolini-Cestari, and Olivia Pignatelli wrote about the case study of diagnosis, dendrochronological dating, and strengthening intervention on the roof of the 16th-century church of San Giovanni Battista in Salbertrand (Piedmont, Italy). One aspect on which the authors focus their attention is that knowledge of historical construction techniques can be an important source for drawing inspiration for the development of intervention techniques. Indeed, the technique used for the strengthening of the ridge beam of the church is the trussed beam invented by Polonceau (1839) and described together with its variations by the handbook authors at least until the beginning of the 20th century. Since then, it gradually fell into oblivion with the extensive diffusion of steel and reinforced concrete structures. The investigations described in the article were carried out in the late 1990s and the strengthening works in the early 2000s; far from being a negative point, this allowed the authors to critically evaluate the diagnostic operations carried out more than twenty years ago, verifying their persistent methodological validity – despite the obsolescence of some of the instruments and techniques used – and to test the effectiveness of the strengthening techniques used over time.

The research presented by Marta Casanova, Stefano Musso, and Stefano Podestà in their article deals with the investigation, analysis, and project for the restoration of the roof structure of a part of the Albergo dei Poveri in Genoa. This part of the building – the men's oratory – is particularly interesting due to the simultaneous presence of masonry arches and timber structures supporting a vault made of plastered reed mats. The knowledge of this vault's construction technique – very problematic due to the inaccessibility of the attic – was obtained thanks to thermography and taking advantage of the holes in the vault caused by the collapse of small portions of the reed mats. The proposed reinforcement project starts from the deficiencies and vulnerabilities highlighted by the investigations – particularly the poor reliability of the connection between reed mats and superior structures – and is based on the principle of minimum intervention.

The contribution written by Jorge Branco, Filipa Serino, Eleftheria Tsakanika, and Paulo Lourenço is an interesting and systematic review, although non-all-encompassing, given the limited space available, about the reinforcement methodologies for timber elements in historic roofs.

The positive picture shown in this special issue could lead to being overly optimistic about the future of historic timber roof structures: the real challenge for the future is to move from scientific research and a design activity conducted by a few specialized professionals – mostly linked to the academic world – to the reality of current professional practice. This can only happen through an improved connection between university and extra-university teaching activities in the sector of historic timber structures, which – at least in Italy – are still largely neglected in academic course programs in favour of masonry, reinforced concrete and steel structures.

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THE ROOFING STRUCTURES OF THE GOTHIC AGE IN FRANCE

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Abstract

The structural solution called *chevrons formant ferme* is a typical invention of the French Gothic age. In this paper, the structural functioning of such kind of structures is considered and analyzed, referring to the two original different structures of the roof of Notre-Dame of Paris, destroyed by the fire of 2019. The results show the incredible skill of the builders of the Middle Ages in designing very effective timber structures and how these structures were conceived to respond to criteria of different nature.

Keywords

Timber structures, Structural analysis, Design reconstruction, Gothic architecture.

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1. INTRODUCTION

The unexpected birth and rapid explosion of what we call, from the Renaissance, Gothic architecture, is an unprecedented architectural phenomenon that appeared in the North of France during the 12th century, so huge and wide that, according to Gimpel [1], during less than three centuries, the French builders carried and used more stones than the Egyptians during the whole period of their civilization. The typical Gothic construction is the cathedral that, beyond the peculiarities of the Gothic structural elements, like flying buttresses, pointed arches, and rib vaults, is characterized by its dimensions: a Gothic cathedral is a high and large stone building.

What is often forgotten in the description and architectural studies of the Gothic cathedrals is that these buildings were covered by a timber structure. Though not as apparent and visible as the stone body of a cathedral, these timber constructions, in French, the *charpentes*, are impressive constructions, often having huge dimensions. This fact, along with other constraints of different natures, detailed below, forced the carpenters of the Middle Ages to invent innovative structural solutions, peculiar to this period and geographical area, that time proved to be very effective.

The true invention of the carpenters of the Gothic period is what in French is called the *charpentes à chevrons formant ferme*. This expression, whose translation could be "a carpentry with rafters forming common frames", indicates a particular structural solution that appeared in the French region around Paris during the first part of the 13th century.

Though this kind of structure has been studied extensively in the past by historians of architecture, rather curiously, no serious and complete structural studies of these constructions seem to exist in the literature. This fact has probably contributed to creating misunderstandings and false ideas about the structural functioning of these structures.

A detailed study of two roofing structures with *chevrons formant ferme* has been done recently on the two original *charpentes* of the choir and of the nave that covered Notre-Dame of Paris and were destroyed by the fire that occurred on April the 15th, 2019 [2].

The objective of the present paper is to try to give a reconstruction of the structural ideas of the Gothic builders, using to this end the results of the cited paper, i.e., considering the destroyed roof of Notre-Dame as a representative paradigm of a *chevrons formant ferme* roofing structure. The indications given by the structural calculations show some facts describing clearly the statics of the timber structure and, rather likely, the structural thought of the builders. From these facts, it is also evident that some ideas carried on by architecture historians are inaccurate. All these points are discussed below.

Finally, this research aims to go beyond the mere descriptive analyses done so far and try to shed light on its real static behavior, how, presumably, it was thought by its ancient master-builders. In some way, it is an attempt to retrace the constructional thinking of the Gothics, their ideas in designing their *charpentes*, and to check whether or not some of the more common ideas on this matter are sound.

The paper is so structured: first, a description of a *chevrons formant ferme* structure is given, with some examples. Then, the structural analysis results on the destroyed *charpentes* of Notre-Dame are recalled and analyzed. On the basis of these results, we try, on one side, to refute or confirm some ideas of the past about this kind of structure and then to try to respond to the most delicate question: why the carpenters of the gothic age invented such a sophisticated and innovative structure?

2. DESCRIPTION OF A *CHEVRONS FORMANT FERME* ROOFING STRUCTURE

A covering structure with *chevrons formant ferme* is basically composed of four main structural elements, see Fig. 1: the main frame, in French *ferme principale* or *chevron maître*, the common frames (*fermes secondaires* or *fermettes* or *chevrons*), the bracing system (*contreventement*) and the wall plates (*sablières*). All these elements work together to carry the vertical (dead load) and horizontal (wind loads) actions. This modular unit, composed of the main frames and a few common frames, is then repeated to form the whole roofing structure. Usually, the *charpente*, so constituted, was covered by a wooden decking (the *voligeage*) and by tiles or lead plates. The correspondence between English and French technical terms is given in Tab. 1.

Of course, the static scheme of the main and common frame, as well as of the bracing system, can change, each roofing structure being a peculiar case, but all of them share the same fundamental characteristics of having a main frame and some common frames that are placed at a short distance (say, of the order of 80÷100 cm) and by the absence of purlins. The rafters forming the common frames are not supported by purlins but are a part of the whole structure. The fundamental difference between the main and common frames is the absence, in these last, of the tie-beam: only collar ties are present in the common frames. This is a key point of the structural functioning of *chevrons formant ferme charpentes*. The reason for this choice made by the carpenters of the Middle Ages is analyzed below. Still, since now, it should be kept in mind that these choices (the absence of the tie-beam in the common frames and that of the purlins, with the consequence that the rafters are not carried but self-supporting) completely characterize this kind of structure: the rafters of the common frames actually form, with their collar ties, a (secondary) frame, whence the name of *charpente à chevrons formant ferme*. The whole system is then typically characterized by rafters (of the main and common frames) that are close together, which allows posing the wooden decking (the *voligeage*) directly on the structure, unlike in the structures composed of main frames, purlins, and rafters, see, e.g. [3, 4], that as a consequence have a higher global thickness of the covering structure.

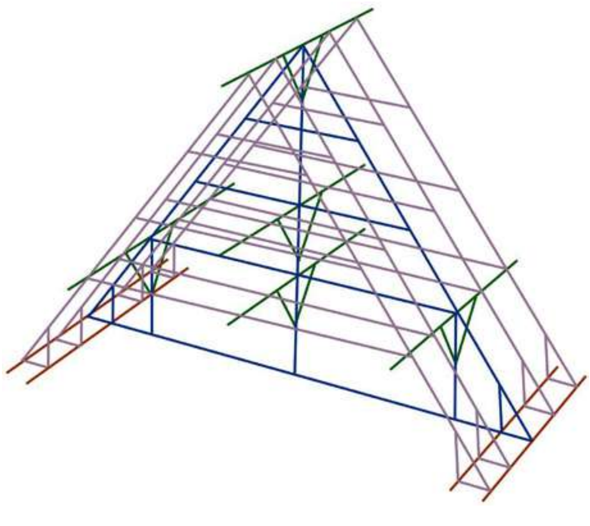

Fig. 1. *The modular unit of a charpente with chevrons formant ferme; in blue, the main frame (chevron maître); in grey, the common frames (fermettes); in green, the bracing system and in brown, the wall plates (sablières).*

Fig. 2. *Views of the combles: (a) in the choir (image source: [7]); (b) in the nave (image source: [8]); (c) in the transept (photo by the Author).*

This structural organization of the *charpente* gives it a three-dimensional structural functioning, which is not the case for the *charpentes* with main frames, purlins, and rafters. This is a fundamental aspect of the *charpentes à chevrons formant ferme*, which is detailed below. Before, a brief description of the *charpente* of Notre-Dame , used as a paradigm for the structural analysis, is given in the next section.

2.1. THE ANCIENT *CHARPENTES* OF NOTRE-DAME

The roof of Notre-Dame of Paris that burnt in the fire of April the 15th, 2019, was composed of three distinct *charpentes*, built at different periods ([5, 6] and Fig. 2): the choir *charpente*, made after 1220, probably from 1225 to 1230; the nave *charpente*, slightly subsequent, presumably built from 1230 to 1240; the transept *charpente*, entirely rebuilt during the restoration work of Lassus and Viollet-le-Duc after 1843, along with the spire and the first frames of the nave and choir nearby the spire. The *charpente* built in the 19th century was of the type with main frames, purlins, and rafters and will not be considered here.

The whole timber structure (also named the *combles* in French) had an overall length of 115.6 m, was 13 m wide, and was 9.75 m high. The two *charpentes* of the nave and choir were both à *chevrons formant ferme* but with different schemes. This fact, considered in detail below, suggests an evolution in the constructive thinking of the ancient carpenters [6, 8]. The longitudinal scheme of the *combles* is presented in Fig. 3a. The three parts of the *charpente* are clearly indicated as well as the notation of the main frames, as usually adopted. The longitudinal structure relying together the frames is schematically presented too. The parts of the *charpente* object of this analysis are those between frames FC4 and FC9 for the choir, and between FN1 and FN11, for the nave. They correspond to the regular part of the medieval *charpente*, the rest being the part of the structure constituting the apse of the choir that Lassus and Viollet-le-Duc reconstructed during the 19th century. Above the frames was a wooden decking, the voligeage, which was supporting the lead plates, nailed on it.

French term	English term
Charpente	Timber structure
Combles	Roofing structure
Ferme principale, chevron maître	Main frame
Ferme secondaire, fermette, chevron	Common frame
Panne faîtière	Ridge beam
Poinçon	King post, crown post
Suspente	Queen post
Poteau	Post
Entrait	Tie-beam
Faux entrait	Collar tie
Arbalétrier	Rafter
Faux arbalétrier	Secondary rafter
Jambette	Hammer post, ashlar piece
Aisselier	Brace, wind brace
Blochet	Hammerbeam
Sablières	Wall plate
Lierne	Girt
Jambe de force	Bracket
Contreventement	Bracing
Voligeage	Wooden decking
Passerelle	Catwalk
Mur gouttereau	Guttering wall
Console, corbeau	Cantilever, corbel

Tab. 1. *Correspondence between the French and English technical terms for the components of a charpente.*

In the choir of Notre-Dame , the modular unit was composed of one main frame and four common frames, spaced ~82 cm, for a whole length of ~4.1 m. In the nave, this distance has been reduced to ~3.5 m, with frames spaced ~75 cm. The main frames of the choir and nave of Notre-Dame , as well as the model of a *fermette*, which is sensibly the same in the two cases, are shown in Fig. 3b-d. These schemes have been reconstructed using mainly [9], who made the first dendrochronological analysis of the charpente in her MSc thesis, and [7]. In the same figure, the French names for the different pieces of the structure are also indicated. Unlike the choir's *charpente*, relatively homogeneous from FC4 to FC9, the nave's *charpente* presents minor differences in some frames. This is probably due to maintenance operations, done or not done, during the centuries. Here, being interested in analyzing the static functioning of the structure, we will consider just the modular structural unit around FN7, considered the most representative frame for the nave's *charpente*.

Though Dubu [10] said that the *charpente* was made with chestnut wood, it is certain that it was realized with the wood of oak trees, while the *voligeage* was made of fir wood, cf. [9], [8]. The number of trees employed for the structure was vast, so the *charpente* was called *la forêt*, the forest. According to F. Épaud, who

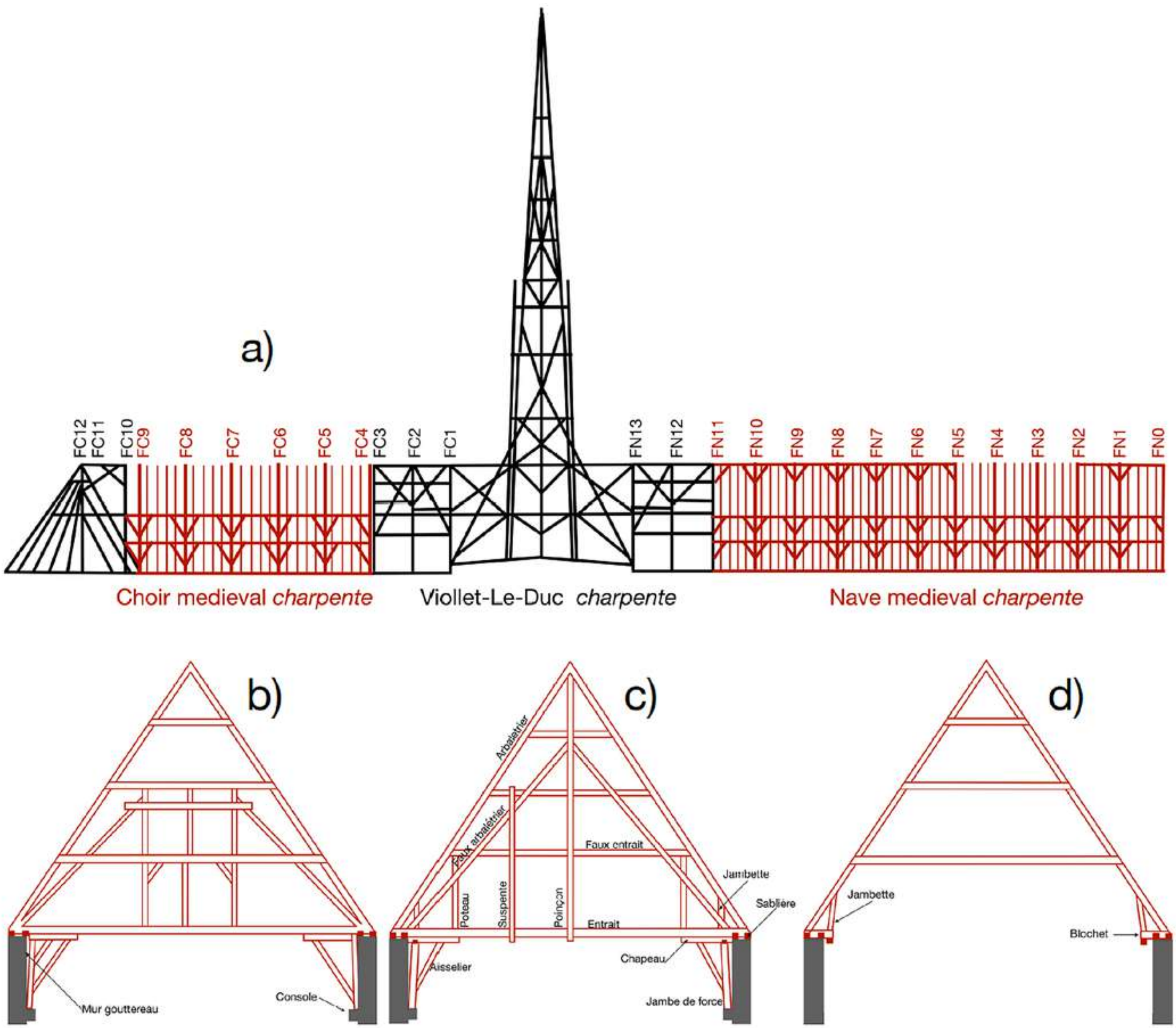

Fig. 3. *Schemes of the charpentes of Notre-Dame: (a) longitudinal scheme; the parts of the structure studied in this paper are in red; thick lines indicate the main frame and the longitudinal bracing system, while thin lines represent the common frames; (b) scheme of a main frame of the choir; (c) scheme of a main frame of the nave; (d) scheme of a common frame.*

has deeply studied the timber structures of the Middle Ages, it was composed of the wood of about 1000 oaks, almost all of them with a diameter of ~25 to 30 cm and 12 m high, a small part with a diameter of ~50 cm and 15 m high, corresponding to about 3 hectares of forest.

2.2. BRIEF HISTORICAL ACCOUNT OF THE NOTRE-DAME *CHARPENTES*

The first choir's *charpente* was probably finished before 1182 ([11], page 16; [12]) when the choir was consecrated. Subsequently, a new *charpente* was erected: on the choir between 1225 and 1230 and in the nave between 1230 and 1240 [5]. These new *charpentes* reused some timber beams of the original roof; in fact, several pieces of them showed unused mortices or *mi-bois* notches, a clear sign of reuse [11].

The reconstruction of the *charpente* was the consequence of a set of changes made to the cathedral during its construction. In particular, the guttering wall, i.e., the upper part of the clerestory, was raised about 2.70 m above its original height, a fact that had several consequences on the structure of the new *charpente*, as discussed below.

The reasons for this reconstruction are not well known, and historians still debate on this point: according to [13], the changes were done just for a matter of style, while [14] suggest that they were made to improve the structural response. Whatever the reasons for these modifications on the still unfinished cathedral, they are significant, do not concern the charpente uniquely, and are still a matter of historical debate today, cf. [12–15]. What is important for the purposes of the present study is that these transformations necessarily forced the carpenters to adopt a structural scheme different from the previous one. This is a crucial point to be analyzed below.

3. STRUCTURAL ANALYSIS OF THE *CHARPENTES À CHEVRONS FORMANT FERME* OF NOTRE-DAME

3.1. THE MECHANICAL MODELS

The *charpentes* of the choir and the nave are studied through a Finite element (FE) analysis. Each of the two structures is modeled as a truss, i.e., as an assembly of elastic rods pinned at the ends and each intermediate intersection with another beam. This choice is motivated by, on the one hand, the great uncertainties that exist in the ancient *charpentes*, where the gaps between the timber struts can be rather crucial because of the long drying of the wood pieces (as shown by the research of F. Épaud [16], the *charpentes* of the 12th and 13th centuries were realized with green wood), which implies a practically null couple at the joints, see also [17]. On the other hand, this assumption does not substantially affect the general structural functioning of the *charpente*, which is the study's objective.

The scheme of the main frames of the two *charpentes* and the common frames are shown in Fig. 4. The support points are denoted by the labels S1 to S10; all of them are modeled as frictionless unilateral supports, i.e., able

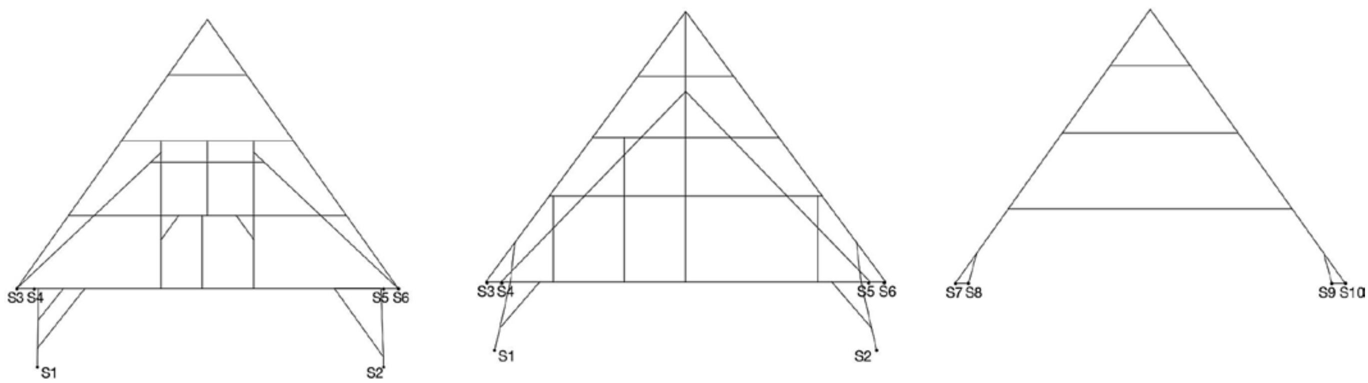

Fig. 4. *From left to right: schemes of the main frames of the choir, the nave, and the fermettes. The supports are indicated by letters S1 to S10.*

to exert only a purely contact reaction. The question of frictionless contact is examined below in section 3.5. In particular, the support points S3 to S10 can exert only upward vertical forces, while in S1 and S2, two unilateral reactions are exerted, an inward horizontal one and a vertical upward one. It is important to notice that as a consequence of the deformations produced by the loads, some of the support points can detach from their footing. The two structural units, the choir and the nave, are modeled through three-dimensional structural schemes shown in Fig. 5. The *sablières* are wall plates running from one main frame to another; they are just posed onto the top of the guttering walls and transmit the horizontal forces from the common to the main frames. They are hence bent in the horizontal plane. Each beam of the FE model is modeled as an Euler-Bernoulli rod. The boundary conditions imposed on the points at the ends of the *liernes* and *sablières* specify the continuity of displacements and rotations with the corresponding elements of the adjacent structural units. In this way, the simulation done on a singular structural unit represents the global structural response of the *charpente* for each one of its parts, in the assumption of uniform loading all over the *charpente*, which is the case for the own weight and, at least to a first approximation, for the wind action.

The dimensions of the beams composing the *charpente* are reported in Tab. 2; the minimum diameter d_{min} of a trunk to obtain the corresponding cross section is also indicated. As observed in [8], most parts of the *charpentes* can be obtained by trunks with a moderate diameter, less than ~30÷35 cm. A thickness of 2 cm for the *voligeage*, made of fir wood (density ~500 kg/m^3), has been considered. For the catwalk, the mass has been evaluated to ~240 kg for each structural unit of both the *charpentes*. The global volumes and masses of wood for each structural unit are summarized in Tab. 3.

The data above show that the quantity of wood is practically the same for the two *charpentes*, though the total mass per unit length is ~19% greater for the nave's *charpente*. In consideration of these data, what can be said is that the change of the structural scheme was not dictated by economical issues. The results of the structural analysis, shown below, suggest another possible reason: the carpenters of the 13th century probably searched for a better structural response. In fact, the main frame of the nave has an improved mechanical behavior than that of the choir; in short, it needs less wood to obtain the same stiffness: the nave's main frame is lighter than the choir's one. However, the nave's *charpente* unit is heavier than that of the choir

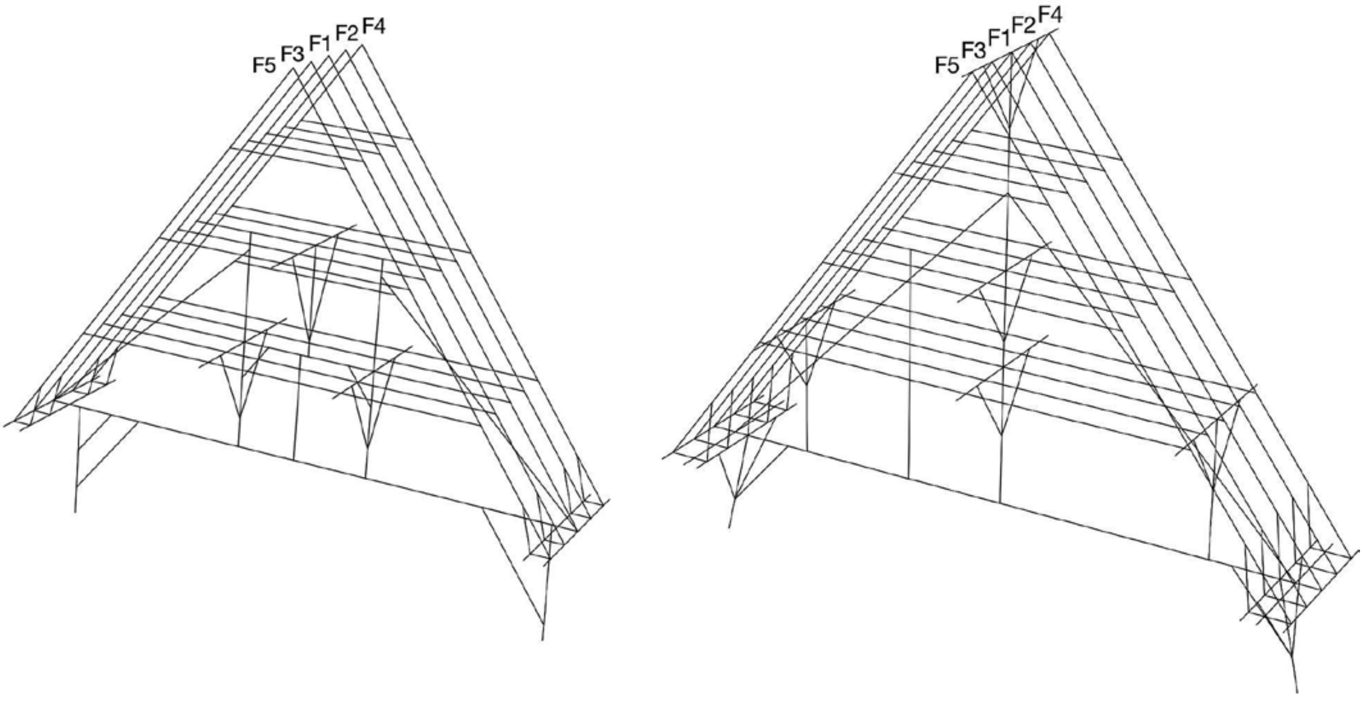

Fig. 5. *Schemes of modular structural units of the choir, left, and nave, right.*

because of the greater weight of the bracing system and of the common frames of the nave.

3.2. MATERIAL CHARACTERISTICS

The mechanical characteristics of oak wood are not constant; they change with the state of the wood, green or dry, and with the plants: as a matter of fact, a certain degree of uncertainty is unavoidable. In order to fix the necessary material parameters, the scientific publication of CIRAD [18] and the European norm EN338 [19] have been used. However, these data are not sufficient to completely characterize a cylindrically orthotropic material like oak wood [20]. In particular, the data taken from EN338 correspond to the wood class D50. Considering this insufficient data, the wood has been modeled as an isotropic material with Young's modulus E = 12500 MPa and a Poisson's ratio ν equal to 0.25. Because the structure is almost exclusively solicited by axial forces and bending moments, this choice does not substantially affect the way the structure works. For what concerns the density ρ, the value of 710 kg/m^3, generally accepted for dry oak wood, has been taken.

	b [cm]	h [cm]	d_{min} [cm]	A [cm^2]	J_1 [cm^4]	J_2 [cm^4]	J_0 [cm^4]	μ [kg/m]
Choir's *charpente*								
Entrait	30	35	46.1	1,050	107,188	78,750	185,938	74.550
1st *faux entrait, liernes*	13	27	30.0	351	21,323	4,943	26,267	24.921
2nd *faux entrait*	17	19.5	25.9	332	10,504	7,984	18,488	23.537
3rd *faux entrait*	15	23	27.5	345	15,209	6,469	21,678	24.495
4th *faux entrait*	15	19	24.2	285	8,574	5,344	13,918	20.235
Arbalétriers	18	19	26.2	342	10,289	9,234	19,523	24.282
Faux arbalétriers	28	17	32.8	476	11,464	31,099	42,562	33.796
Poteaux	19	15	24.2	285	5,344	8,574	13,918	20.235
Poteau central haut	14	14	19.8	196	3,201	3,201	6,403	13.916
Aisseliers faux entrait	14	17	22.0	238	5,732	3,887	9,619	16.898
Jambe gauche, jambettes	18	23	29.2	414	18,251	11,178	29,429	29.394
Aiss. j. gauche and *liernes*	14	18	22.8	252	6,804	4,116	10,920	17.892
Jambe droite	30	19	35.5	570	17,148	42,750	59,898	40.470
Aisselier jambe droite	30	18	35.0	540	14,580	40,500	55,080	38.340
Blochet	15	15	21.2	225	4,219	4,219	8,438	15.975
Sablières	19	14	23.6	266	4,345	8,002	12,347	18.886
Nave's *charpente*								
Entrait	26	29	38.9	754	52,843	42,475	95,318	53.534
Faux entraits	17	24	29.4	408	19,584	9,826	29,410	28.968
Arbalétriers	16	25.5	30.1	408	22,109	8,704	30,813	28.968
Faux arbalétriers	17	19	25.5	323	9,717	7,779	17,496	22.933
Poinçon	23.5	18.5	29.9	435	12,399	20,008	32,407	30.867
Suspente	12	12	17.0	288	3,456	3,456	6,912	20.448
Poteaux	17	20	26.2	340	11,333	8,188	19,522	24.140
Jambettes	15	16	21.9	240	5,120	4,500	9,620	17.040
Liernes	15	18	23.4	270	7,290	5,063	12,353	19.170
Aisseliers and *blochets*	15	15	21.2	225	4,219	4,219	8,438	15.975
Jambe de force	20	15	25.0	300	5,625	10,000	15,625	21.300
Chevrons secondaires	17	24	29.4	408	19,584	9,826	29,410	28.968
Sablières	22	14	26.1	308	5,031	12,423	17,453	21.868

Tab. 2. *Dimensions of the wood beams, as deduced from [7]; for each section, b is the width, h its height, d_{min} is the minimum diameter of the trunk, A the area, J_1 and J_2 the moments of inertia, J_0 the polar moment of inertia and μ the linear density of mass.*

3.3. LOADING CONDITIONS

Two loading conditions have been considered: own weight and own weight plus the wind. Following the European norm EUROCODE 1 [21], the wind action has been modeled as a static load, orthogonal to the surface, and distributed on the windward (overpressure) and leeward (suction) sides of the roof. The actions applied to the *charpente* are sketched in Fig. 6, and the values of the loads are detailed in Tab. 4.

	Choir		Nave	
	Mass [kg]	Volume [m^3]	Mass [kg]	Volume [m^3]
Ferme principale	3,168	4.46	2,920	4.11
Fermette	1,050	1.48	1,220	1.72
Contreventement and *sablières*	856	1.21	1,190	1.68
Total for the structure	5,074	7.15	5,330	7.51
Voligeage	920	1.84	766	1.53
Passerelle	240	0.34	240	0.34
Total for the structural unit	6,234	9.33	6,336	9.38
Total per unit length	1,520	2.27	1,810	2.68

Tab. 3. *Global quantities of wood for the choir and nave structural units.*

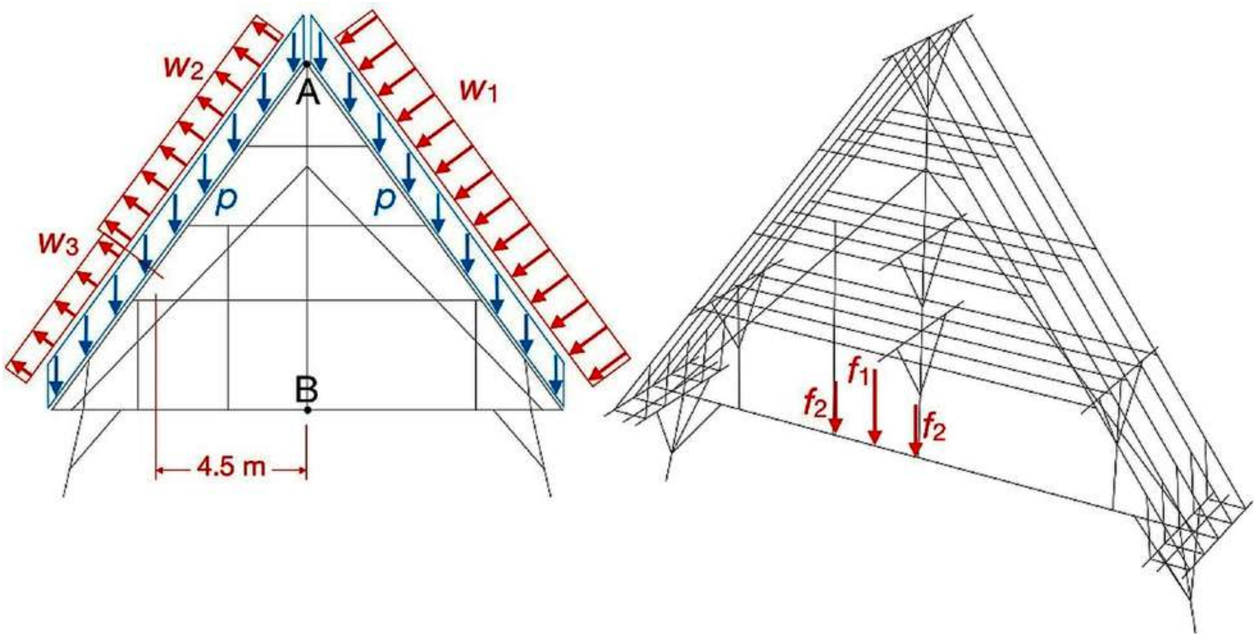

Fig. 6. *Scheme of the actions on the charpente; p: lead tiles and voligeage load; w_1, w_2, w_3: wind load; f_1, f_2: the load of the catwalk.*

	w_1	w_2	w_3	p_0	p_1	p	f_1	f_2
	[N/m]						[N]	
Choir	480.1	351.4	269.4	80.4	479.0	559.4	947	700
Nave	410.0	300.0	230.0	68.7	408.8	477.5	1,037	668

Tab. 4. *Loads on the charpente; p_0 is the linear load of the voligeage, p_1 that of the lead tiles, and p their sum; for the other symbols, refer to Fig. 6 (distributed loads are computed for a 1m wide strip of the roof).*

3.4. RESULTS OF THE NUMERICAL SIMULATIONS

The results of the FE simulations on the two *charpentes* are presented below. For understanding the structural functioning of the structure, especially in view of a critical analysis of the existing literature on the subject, introduced further, the following outputs of the numerical simulations have been considered: the distribution of the reaction forces, the displacements, the stresses, the support conditions, and in particular the role played by friction contact, and a modal analysis used to have an appraisal of the global stiffness of the structure. All these aspects are presented hereafter.

3.4.1. REACTION FORCES

The reaction forces at all the support points, S1 to S6 for the main frame F1, S7 to S10 for the secondary frames F2, F3 and F4, F5 (cf. Figs. 4 and 5), are detailed in Tab. 5 for the two *charpentes* and loading conditions. These reactions are also represented in Fig. 7. Observing these data and figures, the following remarks can be made:

- the distribution of the reactions for the loading condition OW (own weight) is almost, but not exactly, symmetric between the South and North sides; this is the consequence of small asymmetries in the main frames structure;
- still, for the OW loading condition, some of the contact forces are null, e.g., for the supports S7 and S10 of the choir's *fermettes*, or S7 for the F4 and F4 *fermettes* and S9 for the F2 and F3 *fermettes* of the nave's *charpente*;
- the vertical reactions in S1 and S2, i.e., at the level of the corbels supporting the *jambes*, are far less than that absorbed by nodes S3 to S6, on the guttering wall's top, for the choir's *charpente*, while it is higher in the nave, due to the different arrangement of the structure;
- for the loading condition OW+W (Own Weight + Wind), the *jambes* play a major role and strongly affect the distribution of the reactions; in particular, the set of supporting nodes changes: in the choir, nodes S3 on the North side are more charged, while nodes S4, S5, S8, and S10 are inactive; nodes S6, S7 and S9 are also active, but with a reaction force far below that of S3 and node S1 is charged only horizontally, to entirely absorb the wind thrust, while S2 is charged in the vertical direction, as an effect of the roof's slope;
- in the nave, nodes S2, S4, S6, S8 and S10 are inactive, while S1, S3, S5, S7 and S9 active; to remark the high reaction at node S5, consequence of the structural scheme;
- the slope of the roof, ~55°, ensures a stabilizing moment of the wind force distribution w_1 on the windward side, Fig. 6, which explains the positive reaction in nodes S2, S6, and S9 of the choir and the high reaction at node S5 of the nave's *charpente*;
- for both the *charpentes* and load conditions, the distribution of the reactions on the top of the guttering walls is far from being uniform: the largest part of the load is transmitted by the main frames, which play a fundamental role in the structural functioning of the *charpente*.

In Tab. 5, the resultant of the reactions are also indicated. This allows an estimation of the global loads and weights, which are summarized in Tab. 6.

3.4.2. DISPLACEMENT FIELD

The deformation of the modular structural units of the two *charpentes* is represented in Fig. 8; the values of the displacements of nodes A and B in Fig. 6 are shown in Tab. 7. In all the cases, the magnitude of the displacements is minimal. To remark that for the nave's *charpente* submitted to only vertical loads, the vertical displacement of point B, in correspondence with the middle of the tie-beam, is only 56% of the same displacement for the choir's structure. Moreover, the horizontal displacement of point A, the frame's top, of the nave is 44% of that of the choir. These data allow assessing the difference in stiffness of the nave and choir's *charpentes*; see section 3.6.

			Choir		Nave	
Frame	Node	Force [N]	OW	OW+W	OW	OW+W
F1	S1	Rx	2,472	3,7020	9,383	30,260
		Ry	5,084	0	30,220	69,940
	S2	Rx	-2,472	0	-9,383	0
		Ry	4,934	12,310	31,100	0
	S3	Rx	0	0	0	0
		Ry	29,830	51,980	12,090	5,984
	S4	Rx	0	0	0	0
		Ry	2,481	0	19,900	0
	S5	Rx	0	0	0	0
		Ry	2,322	0	15,450	45,600
	S6	Rx	0	0	0	0
		Ry	30,520	7,106	14,780	0
F2, F3	S7	Rx			0	0
		Ry	0	15,630	2,206	724
	S8	Rx	0	0	0	0
		Ry	10,230	0	453	0
	S9	Rx	0	0	0	0
		Ry	10,190	13,000	0	4,621
	S10	Rx	0	0	0	0
		Ry	0	0	2,873	0
F4, F5	S7	Rx	0	0	0	0
		Ry	0	1,540	0	2,736
	S8	Rx	0	0	0	0
		Ry	6,892	0	2,046	0
	S9	Rx	0	0	0	0
		Ry	6,883	8,362	1,587	4,287
	S10	Rx	0	0	0	0
		Ry	0	0	43	0
Resultants	Rx		0	37,020	0	30,260
	Ry		143,561	148,460	141,956	146,260
	Ry	South side	71,922	62,140	70,336	63,416
	Ry	North side	71,639	86,320	71,620	82,844
	Ry	on walls' top only	133,543	136,150	80,636	76,320
	Ry	on South wall's top	66,988	49,830	39,236	63,416
	Ry	on North wall's top	66,555	86,320	41,400	12,904

Tab. 5. *Reaction forces for the choir and nave structural units; Rx: horizontal reaction, Ry: vertical reaction. The number of frames and nodes is indicated in Figs. 4 and 5. OW: Own Weight; OW+W: Own Weight+Wind.*

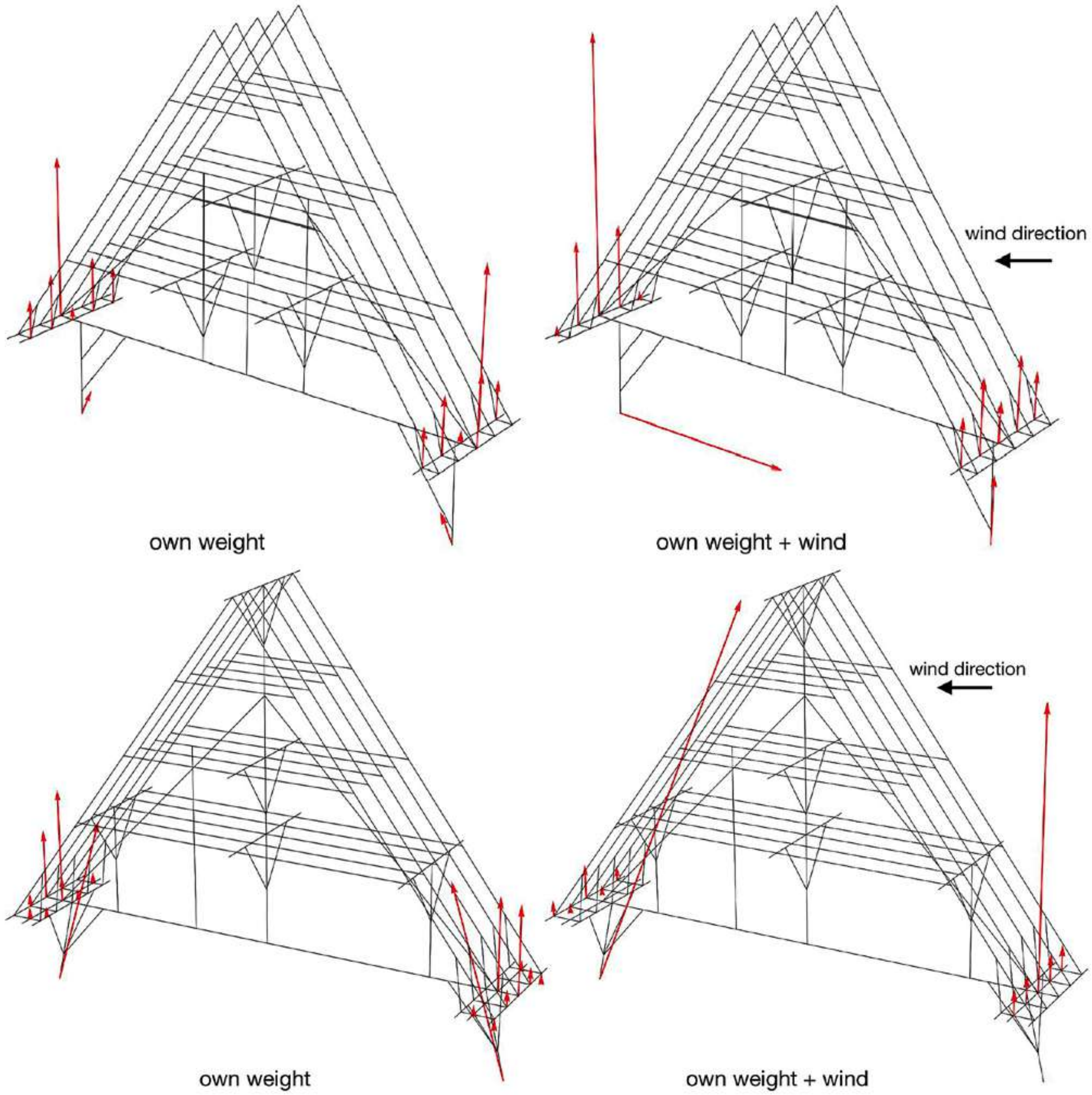

Fig. 7. *Distribution of the reaction forces in the original state; top, the choir's charpente; bottom, the nave's one.*

3.4.3. STRESSES

In Tabs. 8 and 9, the worst combination of internal actions, i.e., the one causing the highest stress value, is given for each different type of beam section, cf. Tab. 2. N is the axial force, positive when tension, M_1 and M_2 are the bending moments, T the shear force, σ_{max} a,d σ_{min} the highest and lowest values of the normal stress in the section, τmax the maximum of the shear stress. The wood being transversely isotropic, a strength criterion for anisotropic materials must be used to check whether or not the material is still in the elastic range [20]. Because wood has a brittle behavior and a different strength in tension and compression, the Hoffman criterion has been used for checking the strength of the structure [22].

By this criterion, for a plane state of stress, which in the case of bent beams, the matter is still in the elastic range if the failure index F_H, defined as

$$F_H = \frac{\sigma_{xx}^2}{X_t X_c} - \frac{\sigma_{xx}\sigma_{yy}}{X_t X_c} + \frac{\sigma_{yy}^2}{Y_t Y_c} - \frac{X_t - X_c}{X_t X_c}\sigma_{xx} - \frac{Y_t - Y_c}{Y_t Y_c}\sigma_{yy} + \frac{\sigma_{xy}^2}{S^2} \quad (1)$$

is not greater than one. In the case of beams, σ_{yy} = 0; moreover, in order to simplify the calculation, for each beam, we calculate F_H for $\sigma_{xx} = \{\sigma_{max}, \sigma_{min}\}$ and $\sigma_{xy} = \tau_{max}$ as reported in Tabs. 8 and 9, though, these stress values generally do not occur at the same point of the same beam. In this way, we obtain, for each beam, an upper bound F_H^{sup} for F_H, and the beam is in the elastic range if

$$F_H^{sup} = \frac{\sigma_{xx}^2}{X_t X_c} - \frac{X_t - X_c}{X_t X_c}\sigma_{xx} + \frac{\sigma_{xy}^2}{S^2} \leq 1. \qquad (2)$$

The characteristic values of X_t, X_c, and S have been chosen once more according to [18] and the European norm EN338, [19]: $X_t = 30$ MPa, $X_c = 29$ MPa, $S = 4$ MPa. The results for F_H^{sup} are shown in Tabs. 8, 9: $F_H^{sup} << 1$ in all the cases. In small words, the structure of both the *charpentes* is very feebly stressed: the matter is everywhere far below the elastic limit state. The most stressed pieces are the *jambes* on the leeward side under the action of the wind: $F_H^{sup} = 0.381$ for the choir's *charpente* and 0.496 for nave's one. The *entraits* are less stressed: the system of the intermediary supports is effective in reducing its bending.

	Weight of a SU	Load on the wall's top		Total wind force	
		South wall	North wall	Wx	Wy
Choir	143,561 35,015	66,988 16,338	66,555 16,233	37,020 9,029	4,900 1,195
Nave	141,956 40,559	39,236 11,210	41,400 11,829	30,260 8,646	4,304 1,230

Tab. 6. *Global loads on a structural unit (SU) of charpente, in [N]. In small: the values per unit length, in [N/m]. W_x and W_y are horizontal and vertical components of the total wind force.*

Node	Choir		Nave	
	δ_x	δ_y	δ_x	δ_y
		Own weight		
A	-0.17	-0.32	0.03	-0.51
B	-0.17	-1.27	0.04	-0.71
		Own weight+wind		
A	-16.60	-0.06	-7.35	-0.02
B	-16.43	-5.13	-7.12	-0.20

Tab. 7. *Displacements of points A and B in Fig. 6, [mm]; δ_x: horizontal displacement, δ_y: vertical displacement.*

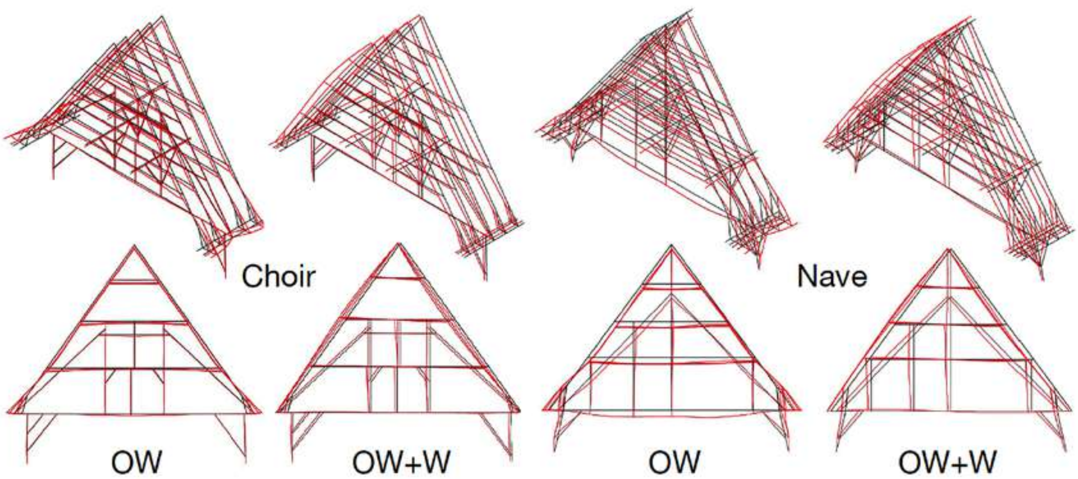

Fig. 8. *Deformation of the structural units of the charpentes, in the original state, for their own weight (OW) and for their own weight plus the wind (OW+W) (displacements magnified).*

	N [N]	M_1 [N m]	M_2 [N m]	T [N]	σ_{max} [MPa]	σ_{min} [MPa]	τ_{max} [MPa]	F^{sup}_{H}
				Own weight				
Entrait	37,422	4,590.06	0	1,915	1.100	-0.393	0.027	0.001
1st faux entrait	-2,453	2,111.70	0	4,298	1.267	-1.407	0.184	0.006
2nd faux entrait	-12,835	640.49	0	187	0.207	-0.982	0.008	0.002
3rd faux entrait	-2,958	1,231.29	0	772	0.840	-1.017	0.034	0.002
4th faux entrait	-1,774	177.97	0	0	0.135	-0.259	0.000	0.001
Arbalétriers	-8,103	1,666.67	0	104	1.302	-1.776	0.005	0.006
Faux arbalétriers	-24,633	629.24	0	1,346	-0.050	-0.984	0.042	0.002
Poteaux	14,368	2,345.56	0	9,240	3.700	-2.788	0.486	0.027
Poteau central haut	2,278	0.24	0	0	0.117	0.116	0.000	0.001
Jambe gauche, jambettes	-5,045	1,570.09	0	2,416	0.867	-1.111	0.088	0.003
Aiss. j. gauche, liernes	-6,988	91.72	0	0	-0.156	-0.399	0.000	0.001
Jambe droite	-4,934	837.54	0	2,323	0.377	-0.551	0.061	0.001
Sablières	0	1,805.65	3537.55	2,202	7.109	-7.109	0.124	0.067
Liernes	0	1,451.47	1.46	3,159	0.921	-0.921	0.135	0.003
				Own weight + wind				
Entrait	76,883	47,712.27	0	20,523	8.522	-7.058	0.293	0.079
1stfaux entrait	7,383	4,611.30	0	4,528	3.130	-2.709	0.194	0.014
2nd faux entrait	11,902	3,694.00	0	10,418	3.788	-3.070	0.471	0.028
3rd faux entrait	-6,224	5,359.04	0	4,839	3.872	-4.233	0.210	0.028
4th faux entrait	-1,991	177.97	0	0	0.120	-0.267	0.000	0.001
Arbalétriers	-2,712	1,345.38	0	777	1.163	-1.322	0.034	0.004
Faux arbalétriers	-29,961	1,578.62	0	873	0.541	-1.800	0.028	0.006
Poteaux	-9,436	4,148.06	0	2,499	5.491	-6.153	0.132	0.052
Poteau central haut	-2,113	266.43	0	148	0.475	-0.690	0.011	0.001
Jambe gauche, jambettes	83,028	24,063.63	0	27,403	17.160	-13.157	0.993	0.381
Aiss. j. gauche, liernes	105,058	91.72	0	0	4.290	4.048	0.000	0.016
Jambe droite	-12,232	131.55	0	363	-0.140	-0.287	0.010	0.001
Sablières	0	2,943.39	6302.16	3,589	12.224	-12.224	0.202	0.188
Liernes	0	620.64	4434.61	0	6.224	-6.224	0.000	0.052

Tab. 8. *Internal actions and stresses in the choir's charpente.*

	N [N]	M_1 [N m]	M_2 [N m]	T [N]	σ_{max} [MPa]	σ_{min} [MPa]	τ_{max} [MPa]	F^{Sup}_{H}
Own weight								
Entrait	29,222	2,399.38	0	12,138	1.046	-0.271	0.241	0.008
Faux entraits	-5,704	1,107.67	0	4,323	0.539	-0.819	0.159	0.005
Arbalétriers	-20,504	937.66	0	2,374	0.038	-1.043	0.087	0.003
Faux arbalétriers	-18,252	441.47	0	2,169	-0.133	-0.997	0.101	0.004
Poinçon	17,586	404.86	0	3,803	0.707	0.102	0.131	0.002
Suspente	2,912	126.84	0	290	0.321	-0.119	0.015	0.001
Poteaux	6,788	259.42	0	896	0.429	-0.029	0.040	0.001
Jambettes	544	371.19	0	1,775	0.603	-0.557	0.111	0.002
Liernes	0	1,178.40	90.85	3,693	1.589	-1.589	0.205	0.011
Aisseliers and blochets	-8,124	50.04	0	0	-0.272	-0.450	0.000	0.001
Jambes de force	-31,443	1,777.49	0	14,055	1.322	-3.418	0.703	0.080
Chevrons secondaires	-6,009	1,096.50	0.02	248	0.525	-0.819	0.009	0.001
Sablières	0	0	728.81	0	0.630	-0.630	0.000	0.001
Own weight + wind								
Entrait	41,322	12,805.76	0	12,138	4.062	-2.966	0.241	0.035
Faux entraits	9,001	5,752.51	0	4,323	3.745	-3.304	0.159	0.031
Arbalétriers	-37,182	4,388.64	0	2,374	1.620	-3.442	0.087	0.020
Faux arbalétriers	-48,625	2,562.83	0	2,169	1.000	-4.011	0.101	0.024
Poinçon	19,250	4,030.64	0	3,803	3.450	-2.564	0.131	0.022
Suspente	6,280	812.21	0	290	1.628	-1.192	0.015	0.004
Poteaux	-10,080	934.12	0	896	0.528	-1.121	0.040	0.003
Jambettes	1,170	866.57	0	1,775	1.403	-1.305	0.111	0.006
Liernes	0	237.72	3,198.68	0	5.032	-5.032	0.000	0.058
Aisseliers and blochets	-44,331	50.04	0	0	-1.881	-2.059	0.000	0.013
Jambes de force	-74,798	10,073.06	0	14,055	10.937	-15.924	0.703	0.496
Chevrons secondaires	-5,241	2,420.75	30.91	248	1.382	-1.638	0.009	0.006
Sablières	0	0	1,613.61	0	1.396	-1.396	0.000	0.004

Tab. 9. *Internal actions and stresses in the nave's charpente.*

3.5. ABOUT THE TRANSMISSION BY THE FRICTION OF THE HORIZONTAL FORCES

The results presented in the previous Sections have been found in the assumption that the horizontal forces cannot be taken up by friction. We ponder now on whether this assumption is or is not correct. To this end, a new simulation, considering the own weight uniquely, has been done, with changed boundary conditions: for each one of the nodes S1 to S10, a fixed support has been considered to simulate a perfect contact, i.e., a support with sufficient friction to stop any sliding. This is the condition implicitly understood when the horizontal thrust of the *charpente* on the top of the guttering walls is supposed to exist. The new simulation results are detailed in Tab. 10, where H is the total horizontal thrust of the *charpente* and V *is* the total vertical reaction for each structural unit applied to the top of the guttering walls.

	Choir		Nave	
	SW	NW	SW	NW
H [N]	46,048	45,356	40,790	43,389
V [N]	68,030	68,354	63,612	63,056
V_w[N]	160,000	160,000	136,100	136,100
V_t[N]	228,030	228,354	199,712	199,156
M [N m]	124,330	122,461	110,133	117,150
e [m]	0.545	0.536	0.551	0.588

Tab. 10. *Total forces, for each structural unit of charpente, on the top of the guttering walls in the assumption of fixed supports; H: horizontal load, V: vertical load, V_w: weight of the wall, V_t: total vertical load, M: overturning moment, e: eccentricity (NW: North wall; SW: South wall).*

We check the global equilibrium of the guttering wall 2.70 m below its top, i.e., at the same level as the corbels supporting the timber *consoles* of the *charpente*. This was the level of the top of the clerestory walls before the modifications started around 1220, and it is, to a good approximation, the free-standing height of the guttering walls. If we consider the thickness of the guttering walls of 60 cm and the density of the limestone of 2,400 kg/m^3, the weight V_w of this part of the guttering wall is ~160,000 N for the choir (length of 4.1 m) and of ~136,100 N for the nave (length of 3.5 m). The total vertical load V_t at the level -2.70 m with respect to the top of the wall can hence be calculated, as well as the overturning moment *M* of the horizontal thrust, and finally, the eccentricity *e* of V_t with respect to the centroid of the wall's section, cf. Tab. 10. The values of the eccentricity *e* so calculated, greater than 50 cm for all the cases, are extremely high and should cause the overturning of the wall. Also, if the wall had a greater thickness, the eccentricity should be too large to ensure a safe equilibrium of the system *charpente*-guttering walls.

We can hence check the physical possibility for the system to develop effective friction forces. The values of the horizontal, R_x, and vertical R_y, contact forces on the top of the guttering walls for the model with fixed bilateral supports are shown in Tab. 11. If we consider a friction coefficient ν = 0.7 for the contact between wood and stone, value usually admitted in such a case for a dry contact, then it appears clearly that the ratio R_x/R_y exceeds ν in several cases. At the same time, it is close to it in other cases. In addition, the contact between the wood of the *charpente* and the stone of the guttering walls is probably far from being perfect: infiltrations of dust and rainwater cannot be excluded, especially if one considers that the contact is actually unilateral and that, as shown in the numerical simulations presented in the section 3.4.1, some nodes of the *charpente* can slightly lift up under the action of the loads (cf. Fig. 8). It should also be mentioned that, as the same word *sablières* indicates (*sable* is the French word for sand), these wall plates were put in place over a layer of sand, to better ensure the contact between the *charpente* and the wall's top. Of course, this greatly decreases the friction forces, and the builders of the Middle Ages were likely aware of that.

Frame	Wall	Own weight			Own weight + wind		
		Rx [N]	*Ry* [N]	*Rx/Ry*	*Rx* [N]	*Ry* [N]	*Rx/Ry*
Choir							
F1	North	22,058	26,586	0,83	30,368	31,337	0,97
	South	-21,342	26,604	0,80	-12,317	22,293	0,55
F2 and F3	North	5,983	10,378	0,58	8,933	10,650	0,84
	South	-5,994	10,339	0,58	-3,232	11,044	0,29
F4 and F5	North	6,012	10,506	0,57	9,254	11,570	0,80
	South	-6,013	10,374	0,58	-2,961	10,185	0,29
Nave							
F1	North	14,523	21,722	0.67	26,051	28,120	0.93
	South	-15,064	22,122	0.68	-3,663	16,145	0.23
F2 and F3	North	8,119	10,281	0.79	9,216	13,483	0.68
	South	-6,490	10,338	0.63	-3,995	7,997	0.50
F4 and F5	North	6,314	10,386	0.61	9,137	13,796	0.66
	South	-6,373	10,407	0.61	-3,808	7,788	0.49

Tab. 11. *Reaction forces on the top of the guttering walls in the assumption of fixed supports; R_x: horizontal reaction, R_y: vertical reaction. For the frame numbering, cf. Fig. 5.*

Finally, the transmission, by friction, of horizontal forces between the *charpente* and the stone structure should be not only dangerous for the structure's safety but also rather uncertain or even impossible, physically speaking.

3.6. MODAL ANALYSIS OF THE *CHARPENTE*

In order to assess the difference in the structural behavior, namely in terms of stiffness, between the *charpentes* of the nave and of the choir, a modal analysis of the two *charpentes* has been performed. In fact, because the weight of the structural units of the two *charpentes* is practically the same (cf. Tab. 6), the higher the frequency, the higher the stiffness. For this analysis, the FE model is the same one used in the previous section for checking the friction mechanism. The first five modes of the choir's and nave's structures are presented in Fig. 9, where the corresponding frequencies are also indicated. What is apparent is that the nave's *charpente* has a greater stiffness than the choir's. In fact, if we compare the fundamental frequencies, mode 1, we can observe that the frequency of the nave's *charpente* is ~2.5 times that of the choir's one. This clearly indicates a better structural conception of the nave's *charpente* with respect to that of the choir.

4. ANALYSIS OF THE RESULTS AND THEIR INTERPRETATION

We now ponder the structural functioning of a *charpente à chevrons formant ferme*. We start with an analysis of the results of the structural study presented above. Then, on this basis, we try to draw some conclusions on the structural thought of the carpenters of the Gothic Age in France and, at the same time, compare these conclusions with the ideas and hypotheses existing in the literature to confirm or refute them.

4.1. THE MAIN RESULTS OF THE STRUCTURAL ANALYSES

The main points arising from the structural analyses presented above are:

- an intense concentration of the vertical reactions in correspondence with the main frames, while the vertical forces in correspondence with the supports of the common frames are much lower;
- the impossibility of transferring the horizontal forces from the charpente to the stone structure

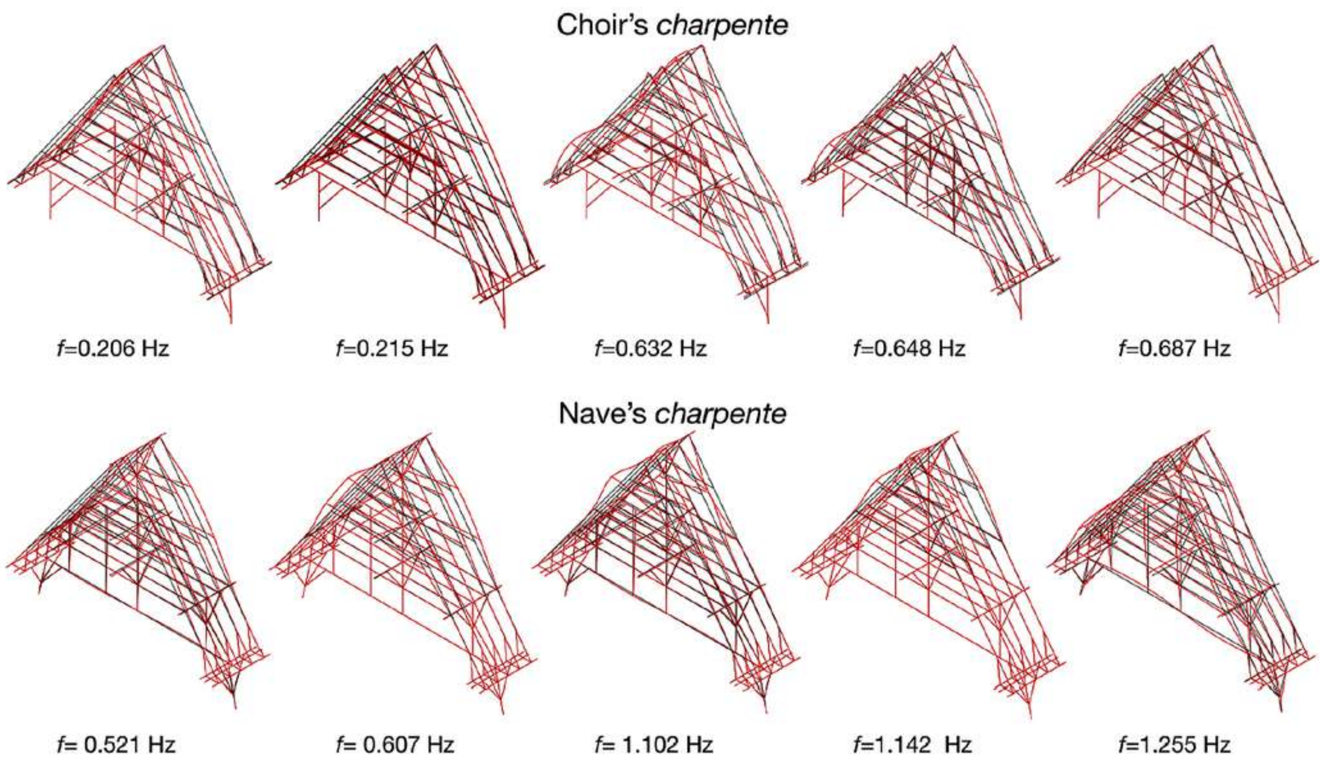

Fig. 9. *First five normal modes and corresponding vibration frequencies.*

below by friction and the fact that such a mechanism of transfer could cause the overturning of the guttering wall;
- the low levels of stress everywhere in the *charpente*;
- the slight deformation of the structure;
- the better design of the main frame of the nave with respect to that of the choir.

4.2. CRITICAL ANALYSIS OF THE RESULTS

Based on these results, the following conclusions can be drawn about the structural functioning of a *charpente à chevrons formant ferme*.

• **A three-dimensional structural functioning**: the structural behavior of a *charpente* with *chevrons formant fermes* is rather complicated and cannot be reduced to a simple planar scheme. The fundamental difference between the main and common frames is the tie-beam, used only for the main frames. Consequently, the horizontal thrust at the base of each secondary frame needs to be equilibrated differently; otherwise, the bending of the rafters would severely deform the frame. This is done by the system composed of the *sablières* and the *blochets*, see Fig. 3d. At the base of any rafter of a secondary frame, a *blochet* transfers the forces from the rafter to the *sablières*, restrained by the tie-beams of two successive main frames. The sablières are hence bent and sheared, in the horizontal plane, by the thrust at the base of the rafters of the secondary frames, and in this way, they transfer these horizontal thrusts, caused by the vertical loads, to the tie-beam of the main frames. Any modular structural unit of the *charpente* is hence self-equilibrated in the horizontal plane and does not apply any horizontal thrust to the stone structure below: a *charpente* with *chevrons formant ferme* is designed to transmit only vertical forces to the top of the guttering wall, while the horizontal thrusts engendered by the vertical loads are self-equilibrated by the system composed by the rafters, *blochets*, *sablières*, and tie-beams of the main frames. In particular, friction is not the mechanism of transfer of the horizontal thrust of the secondary frames to the top of the guttering walls.

In [23], some similar considerations are done, but with some differences. Viollet-le-Duc implicitly admits the role played by the system *blochets-sablières*, but he imputes its adoption for another reason. According to him, the invention of the *charpentes* with *chevrons formant ferme* with such important slopes served to adapt the roofing structure to the reduction of the thickness of the walls of the Gothic cathedrals, with respect to the Romanesque architecture, which rendered challenging to pose a *charpente* with main frames, purlins, and rafters. The only structural consideration made by Viollet-le-Duc concerns the high value of the slopes that decreases the bending of the rafters, so allowing the use of wood beams of relatively small cross sections (an important point, considered further): he never gives an explanation of the global functioning of the *charpente*, though it seems probable that he understood the danger of the transmission of the horizontal thrust from the common frames to the top of the guttering walls by friction.

The point of view of [24] is different: the increase of the roofs' slope in the Gothic period is essentially used to decrease the horizontal thrust. This interpretation cannot be considered correct, cf. Sect. 3.5. Also, the point of view of Pol Abraham [25], as reported in [3], is mechanically wrong: according to him, to explain the deformations of the Gothic vaults, he implicitly admits that there is a transfer, by friction, of the horizontal thrust of the *charpente* to the top of the guttering walls. The results presented above clearly indicate that this is not possible. The point of view of [3] is ambiguous: on the one hand, he strongly supports the idea of Viollet-le-Duc of the increase of the roofs' slope to use trunks of small diameters; on the other hand, he does not make any real structural consideration and tacitly, in the end, he seems to accept the point of view of Pol Abraham.

• **Distribution of the loads**: a common idea about the *charpentes* with *chevrons formant ferme* is that this structural scheme allowed an almost uniform distribution of the loads on the top of the guttering walls. As demonstrated in section 3.4.1, this is far from reality: the loads transferred by the charpente to the guttering walls are unevenly distributed, much higher in correspondence with the main frames than in the secondary ones. The reason for that is exactly the global structural organization that the carpenters gave to the *charpente*, i.e., its three-dimensional functioning.

Probably inspired by a rough two-dimensional analysis of the structure or also, perhaps, suggested by ideological positions, both of them far from the physical reality, the idea that the *chevrons formant fermes* were used to distribute the vertical load almost uniformly on the top of the clerestory walls does not correspond to reality. Moreover, we can affirm that the Middle Ages carpenters did not consider the correspondence of load-points for the *charpente* and the stone structure beneath, simply because they never coincide. In [26], also cited by [3], this discrepancy is severely criticized, based, on the one hand, upon a rather ideological point of view and, on the other hand, on a static idea that cannot be considered as valid, that of a structure that functions as an array of planar independent frames. In addition, the idea that the carpenters on one side and the masons of the stone structure on the other one worked separately, without interacting, is likely to be false: the stone corbels put at the base of the guttering walls exactly in correspondence with the main frames indicate that actually the wall construction of the cathedral was well planned and also conceived for the roofing structure.

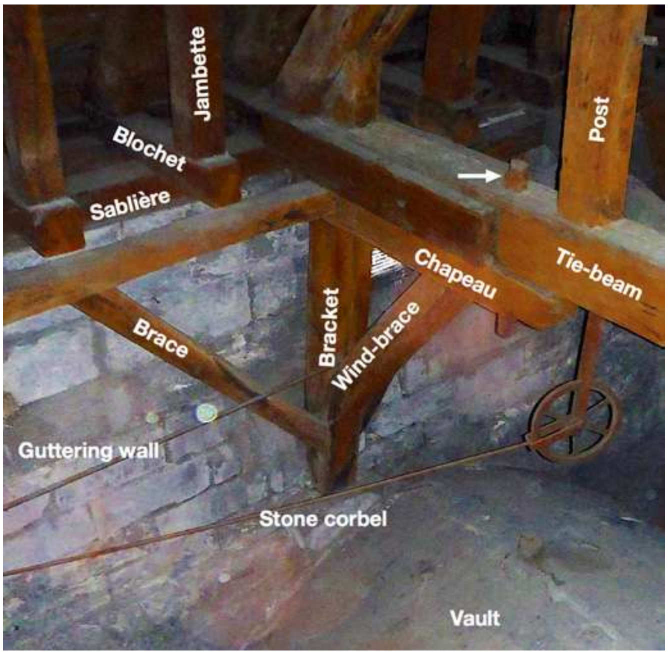

Fig. 10. *The system of the console for the transfer of the wind thrust to the guttering walls; the arrow indicates the shear key between the tie-beam and the chapeau of the console (by the Author).*

- **The wind thrust**: placed above such high constructions, the Gothic builders certainly did not ignore that the wind forces on the roof of a cathedral acted upon the stone structure below with a considerable thrust; also, as seen above, they knew that this thrust was impossible to be counterbalanced by friction, see section 3.5. There are few studies on how Gothic cathedrals, and their roofing structures, withstand the wind forces; namely, for what concerns Notre-Dame [14], with a historical perspective or [27] with nonlinear analysis.

The carpenters of Notre-Dame invented an effective system to transfer the wind action to the underlying stone structure: it is composed of the brackets (*jambes*) of the main frames with their windbraces (*aisseliers*) and *chapeau*, Fig. 10; in [9] this set of struts is indicated as the *console*. It is usually said that the *console* is used as a vertical support of the main frame or also for relieving the bending of the tie-beam. These ideas are not likely: the support given by the *sablières* is largely sufficient, and it should be easier and less expensive to use more posts for decreasing the bending of the tie-beam. In addition, observing the main frames of the nave, we can see that the *console* should support the *entrait* just where there is already a vertical tie, which is redundant. For the choir's frames, the span of the *entrait* is divided into three parts, which is largely sufficient to relieve it in bending, as confirmed by the numerical simulations presented in section 3.4.3. Actually, the true reason that led the carpenters of the 13th century to use the system of the *console* was to transfer the wind force to the lower part of the guttering wall: the horizontal thrust flows as an inclined force to the bottom of the guttering wall through the inclined *aisseliers* and the stone corbels close to the vault level, so improving the strength of the clerestory structure to the action of the wind considerably. This is also corroborated by the presence of a strong shear key connecting the tie-beam and the *console*, Fig. 10, whose role is to transfer the wind thrust from the tie-beam to the *console*, while it is completely useless for transmitting vertical forces. It is likely that the only reason that pushed the carpenters of the cathedral to introduce the *consoles* was to dispose of an excellent device to safely transmit the wind action on the roof to the stone structure below.

The system of the *consoles* is also useful for another reason: the *charpente* of a Gothic cathedral was, typically, built before the construction of the high vault, see, e.g. [3]. During this phase, the *charpente* was essential to counterbalance the flying-buttresses thrust, ensuring the connection between the two sides of the clerestory before the construction of the vault. The *consoles* well assumed this structural role: the tie-beams equipped with the two *consoles* could balance the inward thrust applied to the two opposite clerestory walls by the flying-buttresses. In the case of Notre-Dame, after the rise of the guttering wall, the top of the flying-buttresses was too far below the *charpente* to assume such a kind of thrust balance without a device, the *consoles*, acting down below the *charpente*.

• **The bracing system**: it is generally thought that a system composed of longitudinal beams (the girts) and braces placed at different levels was exclusively used by the carpenters as a bracing system. Indeed, the bracing system was important, especially during the constructive phases, to ensure the global stability of the *charpente* and, once built, to withstand longitudinal horizontal forces (though this is also provided, and more effectively, by the intersection of the nave/choir and transept *charpentes*; also the *voligeage*, acting as a sort of a plate connected to the frames, certainly contributed to the longitudinal stiffness of the structure). However, its true role was another one: the bracing system was used by the Gothic carpenters to relieve the bending of the collar ties (*faux entraits*) of the common frames and to transfer an important part of the vertical load from the secondary frames to the main ones. The numerical simulations (cf. section 3.4.1) confirm the effectiveness of the bracing system in transferring the vertical loads to the main frames, and, most importantly, it also helps in decreasing the bending in the rafters of the *fermettes* and the *sablières* because also the horizontal outward thrust at the base of the rafters is diminished by such a structural organization. Also, thanks to the bracing system, the carpenters did not need to use vertical ties in the *fermettes* to sustain the *faux entraits*. In short, the overall behavior of the *charpente*, conferred by the set of main and common frames and *sablières*, was insufficient to ensure the structure's equilibrium without the bracing system. There are three bracing systems in the choir of Notre-Dame's combles; in the nave, they are five (cf. Fig. 5).

• **The structural differences between the two *charpentes***: if the differences between the two *charpentes* are attentively considered, it appears that all the structural action of the nave's designer is oriented to increase the stiffness, and so the stability of the *charpente*: all the structural changes made with respect to the choir, i.e., the evolution of static scheme for the main frame, the reinforcement of the bracing system and the greater sections used for the *fermettes*, go in the direction of a stiffer structure. Actually, it is not the strength that is substantially improved because the level of stresses in the two *charpentes* is similar, Sect. 3.4.3, but the structure's stiffness, as confirmed by a comparative modal analysis, section 3.6. While a stress analysis was, without doubt, out of the means of the builders of the Middles Ages (the concept of stress was introduced by Cauchy in the 19th century [28]), an embryonic perception of the stability and hence of the stiffness, of a structure can have been in the abilities of the Gothic carpenters. It can be acquired through experience, especially during the construction phases. Thanks to this experience and ability, some particularly wise carpenters can have improved the technique, like in the case of Notre-Dame. Moreover, nowhere in the *combles* did the stresses reach important values: all the structure was scarcely solicited. Hence, all seem to indicate that the Gothics were mainly guided by increasing the structure's stiffness. Anyway, the change of the static scheme from the choir's *charpente* to the nave's one was certainly not dictated by economical reasons because the mass of wood for unit length along the longitudinal axis is greater for the nave.

• **The global structural functioning**: from what was said above, the global structural functioning of a *charpente* with *chevrons formant ferme* emerges clearly: the main frames collect and absorb, through the *sablières*, all the horizontal thrust caused by the vertical loads so that no horizontal action is conferred to the top of the guttering walls. Through the bracing system, the main frames also take on the part of the vertical load of the secondary frames that otherwise should be too much solicited in bending, like the *sablières*. The wind thrust is not transferred to the stone structure by friction on the wall's top, which could cause the tipping of these ones at their base, but by the system of the *consoles* and uniquely in correspondence of the main frames, the wind thrust is passed to the stone structure at the base of the wall on the leeward side as an inclined force. Along with the weight of the upper part of the wall, this allows the stone structure to withstand the action of the wind safely. Globally, the functioning of such a structure is three-dimensional; it is based upon strong collaboration and interaction of all the parts of the structure, and contrarily to what is often said, just like the underlying stone structure of the cathedral, it is a system that mainly transfers the loads at some points, in correspondence of the main frames, and not continuously. On the whole, we can today appreciate the ability of the carpenters of the Middle Ages, that were capable of inventing a very effective structural system well before the discoveries of structural mechanics; such timber structures have spanned the centuries, which witnessed their effectiveness, their only and true danger seems to be the neglect of humans.

A question remains: for what reasons did the Gothics invent the *charpentes* with *chevrons formant ferme*? The following section is devoted to answering this question.

5. CONCLUSION: WHY THE *CHARPENTES À CHEVRONS FORMANT FERME*?

The invention of the *charpentes à chevrons formant ferme* actually responds to an evolution of Gothic architecture, evident just in the case of Notre-Dame [23]: the rise of the guttering walls above the top of the vault. Before such a modification, it was not possible to use frames with tie-beams, because the vault's top was higher than the footing of the frames. The solution adopted by the carpenters was probably that of scissor trusses that do not have a tie-beam. We do not know what the reason for the raising of the guttering walls was, a solution later adopted also in other cathedrals; what is certain is that the carpenters of the 13th century could then adopt a static scheme different from the scissor truss, but they were also faced to the problem of scarcity of trunk of sufficient dimensions (it is famous the adventurous pilgrimage of Abbot Suger to find trees of adequate size for the roof of the Royal Abbey of Saint-Denis), see [3].

This was the true problem for the carpenters of the period: to dispose of wooden pieces of sufficiently large dimensions. In particular, the use of long *entraits* (in Notre-Dame, they have a length of 13 m) must dispose of beams of a significant section to withstand the rod's bending. This was not so easy in the France of the period, and the use of a *charpente* with few *entraits* was hence almost compulsory. So, to solve a structural problem (efficient roofing structures of large dimensions) with the scarcity of sufficiently large timber beams, the carpenters of the beginning of the 13th century invented the *charpente* with *chevrons formant ferme*. Other great cathedrals were covered with this type of structure, among the still-existing ones Amiens [29] and Bourges [30]. With this solution, they brilliantly solved not only the problem of scarcity of sufficiently large wooden beams but transformed the statical problem, adopting a solution that allows eliminating the horizontal thrust at the top of the guttering walls, which are too high to withstand such forces. This solution witnessed a radical change in the structural thought of the carpenters of the period: they were able to pass from a bi- to a three-dimensional scheme, where all the parts of the structure interact together. In the end, we can appreciate how deep and subtle was the structural knowledge of the master builders of the Middle Ages.

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HISTORIC TIMBER ROOFS, A KNOWLEDGE-BASED APPROACH TO STRENGTHENING: THE CASE STUDY OF A RENAISSANCE PALACE IN FERRARA

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Abstract

Knowledge is essential to preserve timber roof structures for their historical and architectural value, since it allows to fully understand their cultural significance and actual structural behavior, thus planning specific strengthening interventions. This paper presents the case study of Palazzo Costabili in Ferrara, Italy, to propose a method to plan architectural strengthening intervention on historical timber roof structures based on a detailed knowledge of the structures' features and state. The palace object of this study is characterized by historical timber structures, an expression of traditional local building techniques, which were partially damaged in the 2012 earthquake. Therefore, an in-depth and careful study of the structure was carried out to identify specific parts that needed to be reinforced. A strengthening of the roof structure was thus designed considering the performances of the timber components and their historical-cultural value. More specifically, the timber roof was first assessed to identify the main wood species, the constructive types, and their related vulnerabilities. At the same time, the parts that showed consistent signs of structural stress were later evaluated by specialists using visual and instrumental analysis. Finally, data collected were critically analyzed to better plan the strengthening intervention, considering both the stress state of the single components and their specific weaknesses, in full compliance with preservation criteria and needs.

Keywords

Historic timber roofs, Traditional construction techniques, On-site assessment, Minimal intervention principle, Preservation of cultural heritage.

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1. INTRODUCTION

In heritage architecture, timber structures are considered witnesses of architectural technology and construction materials of high cultural and historical significance. However, their relevance has been highly neglected in the past, especially compared to the interest and attention paid to the buildings in which they are located [1]. One possible reason for this attitude could be that assessing the actual state of conservation of timber components (and thus their residual mechanical performance) is more complex than simply replacing the wooden elements. Consequently, problems were generally solved in the past by heavily reinforcing the structure with wood, iron, or resins or by replacing not only the decayed parts but the entire timber roof with modern structures and materials. In the 1980s, for instance, the Italian guidelines invited to replace original timber roofs with iron or reinforced concrete structures [2], while preservation of the original roof was allowed only «for reasons of high architectural value» and «in very exceptional situations». This led to consistent alterations of the supporting masonry, strengthened with mortar injections, metal bars, reinforced concrete jacketing, and ring beams [3].

The current approach to historic timber roof conservation is more empirical and respectful of the timber roofs' historical and cultural significance to be preserved as an epitome of old traditional construction techniques with their tangible and intangible features. Thanks to recently developed and improved methods and procedures [4] to empirically analyze existing timber structures, it is now easier to assess their residual mechanical performance and, therefore, to set up more aware and respectful restoration plans. Such a knowledge-based approach needs to be carefully planned at different levels: from the survey of single members and joints (geometry and material consistency) to the interpretation of the structural behavior of the timber structure, considered as a whole and part of the building [5]. Moreover, such peculiar characteristics should be compared with the typical features of similar structures and traditional building skills to better highlight vulnerabilities as well as material (decorations, materials) and immaterial elements to be preserved (construction techniques, structural features).

This approach is useful to assess the actual state of conservation of timber roof structures, to identify the most critical points, and to guide restoration interventions on such structures, usually trying to strike a balance between the preservation of the original material [6] and the respect for the technological conception [7]. Strengthening interventions should always consider the material and immaterial value of the timber roof structure, following the principle of minimum intervention usually recommended in cultural heritage.

This contribution shows how such a knowledge-based methodological approach can be applied to plan timber roof structure strengthening interventions by using the case study of Palazzo Costabili in Ferrara, a historical building featuring timber floors and roofs that are an expression of local construction traditions and techniques. Unfortunately, the palace was severely damaged in the 2012 earthquake and required urgent reinforcement and seismic improvement, especially its historical timber roof. The strengthening intervention started from an in-depth assessment of the actual performances of the timber components, and the reinforcement was specifically designed to preserve the historical value of the roof structures.

2. PALAZZO COSTABILI IN FERRARA

Palazzo Costabili (Fig. 1a) is located in the southeastern and oldest part of the city of Ferrara at the edges of the medieval town, far from the religious and historical center [8]. Despite its marginal position, the palace was a significant building, as testified by the historical figures involved in its construction. The palace was commissioned at the end of the fifteenth century by Count Antonio Costabili (1450-1527), an influential nobleman at the Court of Duke Ercole I of the House of Este. Since he was the ambassador for the Este Family in Milan, the palace is also called "Palazzo di Ludovico il Moro" after Ludovico Sforza, called "il Moro", Duke of Milan from 1494-1499.

2.1. HISTORICAL DEVELOPMENT AND RECENT RESTORATION WORK

According to the date on the original project by Biagio Rossetti (1447-1516), one of the main architects at the court of the House of Este, the palace construction started in 1500 [9, 10]. It was initially conceived as a three-story building organized around a central courtyard of honor with the main entrance on the northern side. It was made up of connected buildings with a double loggia (Fig. 1b), enriched with decorations in white stone by Gabriele Frisoni, who also decorated the monumental staircase with geometric and floral motifs. Next to the stairs, a corridor connects the courtyard to a garden on the eastern side, whereas a grand loggia, on the opposite side of the main entrance, leads to a large park at the back of the building. On the first floor, the upper loggia is composed of a long series of open and blind windows. Nevertheless, only the southern and eastern

Fig. 1. *(a) Aerial view of Palazzo Costabili (image source: Google Earth); (b) three-story building with double loggia organized around the main courtyard.*

sides were completed following the original project. In 1503, Rossetti and Frisoni handed the works over to Maestro Girolamo Pasini and Maestro Cristoforo. Still, one year later, the construction was interrupted, and the northern and western buildings remained single-story buildings without stone decorations [11]. Even if it was never completed, Palazzo Costabili could be considered today as one of Ferrara's most ambitious palaces of the Renaissance period.

After the sixteenth century, the palace changed ownership several times and was split among different owners, thus undergoing substantial modification until it fell into severe decay. In 1920 the palace was confiscated and purchased by the Italian government, and in 1930 it was destined to become the National Archaeological Museum of the Civilization of Spina, officially opened in 1935 [12]. Since then, the museum has showcased all archaeological findings from the province of Ferrara, especially those of the town of Spina, one of the most significant urbanized centers in the region between the sixth and third century BC.

As the building changed its intended use, restoration works were carried out between 1932 and 1935 to fix intrinsic structural deficits due to its architectural conformation [13, 14]. Further strengthening interventions were carried out during the last decades of the twentieth century. Today most of the original vaults and wooden floors were reinforced with metal elements or reinforced concrete slabs.

3. THE TIMBER ROOF STRUCTURE OF THE PALACE

The roof of Palazzo Costabili is characterized by different types of timber structures, while the covering layers are homogeneous, consisting of wooden joists and flat tiles (*pianelle*), while the outside features curved tiles. Such differences in timber seem to be related to the different widths between supports and the articulated configuration of the roof, with complex intersections, differences in height, and misalignment between walls and ridge rather than to the different periods of construction. General architectural features – like the use of open joint trusses [15] and the so-called *alla palladiana* or queen trusses [16] – can be traced back to the 16th century and Ferrara Renaissance architecture. Proof of this is the *trave composta* (using the same technique as a composite beam) employed here in the large-span rafters or purlins typical of local Renaissance construction [17]. This technique consists in connecting more timber beams through precise notches, thus obtaining a single structural element. A recent study showing that the distance between trusses has increased over centuries, changing the configuration of upper layers [19], confirms our dating hypothesis for the timber structures of Palazzo Costabili, as its trusses are shortly distanced (170-210 cm). It should be noted that purlins were not employed here, and joists relied directly on trusses parallel to the ridge. At the same time, flat tiles were arranged with the shorter side parallel to the ridge, which unfortunately caused them to slide down and rotate over time. Over the centuries, the distance between trusses was later increased, and the purlin layer was inserted so that joists became perpendicular to the ridge and the flat tiles more stable [18]. Due to these architectural features, despite the lack of archival documentation, we can posit that most of the timber roofs of Palazzo Costabili date back to the construction of the building itself. However, further analysis – such as a reading of the masonry on which the roof structure rests or dendrochronological studies on the main wooden members – should be carried out to verify the hypothesized dating.

Some parts have, however, been partially modified and consolidated over time. Joints, originally obtained with precise notches and thus featuring few metal connections [16], were progressively reinforced with metallic elements (nails, iron strips, straps, etc.). Even the connection with corbels, widely used in Ferrara's timber structures to reduce deformation, has been improved using metal strips and nails. However, no space was left between the timber members and masonry, generating weak points subject to progressive deterioration because of moisture. More invasive interventions were carried out between 1992 and 1993, as shown by archival documentation [19, 20]. Decayed parts of timber members (especially those in contact with masonry), which deteriorated due to moisture or fungal attack, were removed, and Ø24 metal bars were inserted with epoxy resin to strengthen the joints of trusses or rafters. Moreover, several secondary members have been replaced by new spruce joists (10 cm x 10 cm) or purlins (20 cm x 20 cm) with a bitumen coating at the extremities. One year later, in 1994, strengthening interventions concerned the southern gable roof of the building: a steel cross bracing was inserted [21]. In 1999, the timber structure of the southeastern block was subject to other invasive reinforcements. The decayed extremities of the rafters were replaced by epoxy resins prosthesis and strengthened by internal Ø28 metal bars (Fig. 2a), while Ø24 metal bars were inserted along the rafters to improve the connection between the components of *trave composta* and metal tie-rods were used to enhance the connection between rafters and the central wall (Fig. 2b). The queen trusses of the opposite pitch were also reinforced by the insertion of flitch plates at the joints and a double layer of metal tie-rods for the connection with the central wall.

3.1. IDENTIFICATION OF TIMBER ROOF TYPES

Among timber roof structures, areas with a common morphology (e.g., similar span and type of load-bearing structure) have been identified. In the following sections, a short description of each one is provided to highlight their main features (Fig. 3).

3.1.1. TIMBER PURLIN ROOF

The timber purlin roof is composed of 540-590 cm long beams with a cross-section of 25 cm x 18 cm, arranged parallelly to the ridge at a distance of about 160 cm. This type of roof is employed for small areas. To cover larger spans (790-880 cm), two different solutions were adopted when the palace was built: a rafter beam of a similar section was inserted as intermediate support or the

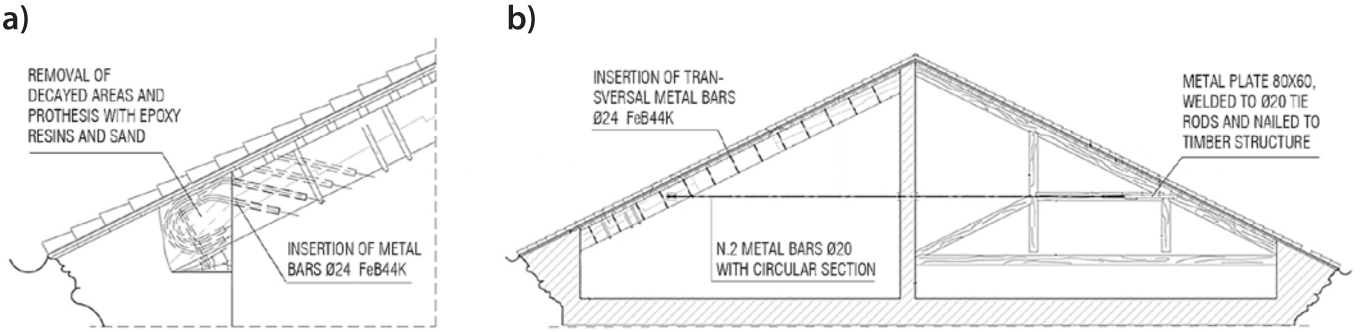

Fig. 2. *Strengthening of the E-S timber roof carried out in 1999: (a) replacement of decayed extremities with epoxy resins prosthesis reinforced with metal bars; (b) insertion of metal bars along "trave composta" and connection between rafters and central wall with metal tie-rods. (Image source: SBAP-FE-ADOC – Mezzadringegneria – FE-ADOC, Tav. 42, Pos. 2955).*

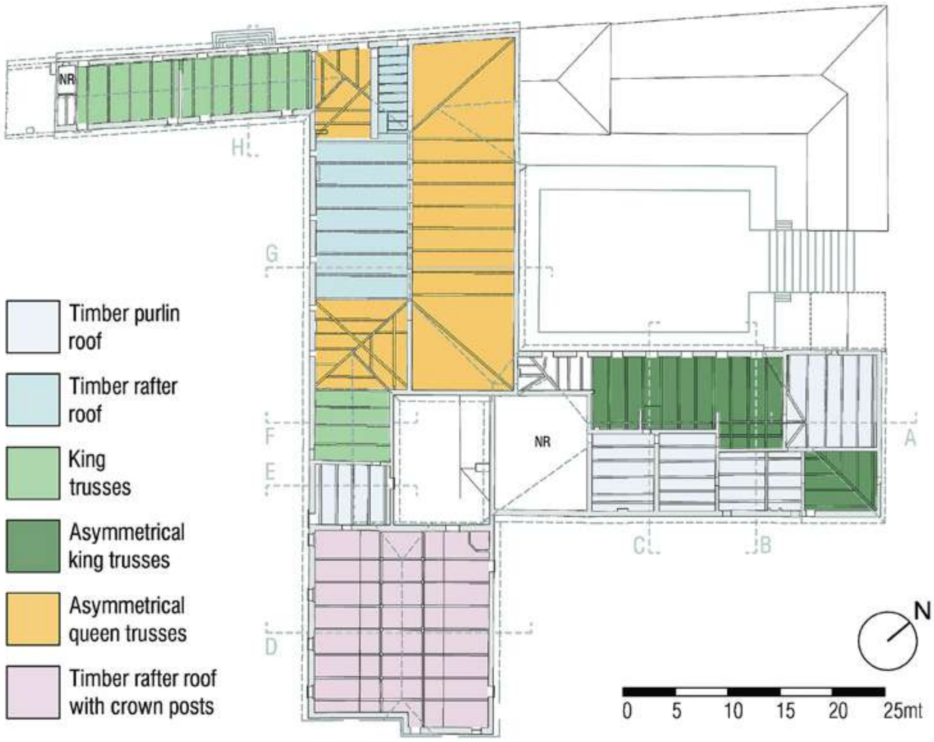

Fig. 3. *Site diagram of the timber roof with different types of timber structures.*

cross-section (17 cm x 50 cm), and the distance (175 cm) of purlins was increased. In this latter case, the technique of *trave composta* is used (Fig. 4a).

3.1.2. TIMBER RAFTER ROOF

The timber rafter roof is composed of beams with a length of 880 cm and a section of 17 cm x 50 cm, arranged perpendicularly to the ridge at a distance of about 220 cm. This type of roof is employed for the roof's southern pitch, and joists are parallel to the ridge. The technique of *trave composta* is used. Corbels are connected with the ends of rafters through u-shaped straps. Some extremities have been strengthened with epoxy resin prostheses or thick wooden boards with iron strips (Fig. 4b). As previously described, metal elements have been inserted, and double tie-rods connect the rafter with the central wall, probably replacing the previous wooden collars.

3.1.3. KING TRUSSES

This type of roof is employed for gable roof portions with small spans (550-750 cm), distanced about 170 cm. Joists are parallel to the ridge. Joints between the rafter and tie-beam were reinforced laterally with C-shaped metal elements connected with masonry. Some trusses are supported by a corbel, connected with the tie-beam by means of iron strips (Fig. 4c).

Fig. 4. *Timber roofs types: (a) large-span timber purlin roof built with the "trave composta" technique; (b) timber rafters roof made with the "trave composta" technique; (c) a small-span king trusses roof with reinforcement.*

3.1.4. ASYMMETRICAL KING TRUSSES

In the case of large-span pitches (680-880 cm), one of the rafters is extended up to the ridge. Moreover, in this case, the ridge is not aligned with the masonry walls, so the posts are off-centered with respect to the tie-beam. A secondary rafter is added to connect the tie-beam end to the extension of the opposite principal rafter. Larger span trusses were reinforced with metal props (Fig. 5a). The distance between trusses is about 170 cm, and the joists are parallel to the ridge. An asymmetrical king truss is also adopted for the N-E hip.

3.1.5. ASYMMETRICAL QUEEN TRUSSES

The queen truss, also called *alla palladiana*, composed of two posts, a tie-beam, a straining beam, and two symmetric rafters (going from the tie-beam and the straining beam), is employed here for large-span pitches (980 cm) but with a variation: an upper rafter is added to connect the straining beam to the ridge, whereas the opposite post is longer to meet the upper of the rafter. The trusses are connected to the central wall with two levels of metal tie-rods that connect, in turn, the rafters with a three-way strap (Fig. 5b). Note that the tie-beams support the timber ceiling of the great hall below, known as *Sala delle Carte Geografiche* (Geographical maps' room) richly decorated with squared coffers. Therefore, the decoration determines the distance between trusses (about 220 cm). Joists are parallel to the ridge. Asymmetrical queen trusses are also adopted for hips and intersections between perpendicular slopes (Fig. 5c).

3.1.6. TIMBER RAFTER ROOF WITH CROWN POSTS

The gabled roof of the southern area of the building consists of long and slender rafters that join the small king trusses towards the ridge area (Fig. 6a). The joint between them is obtained through a groove in the support, which guarantees both compression and traction resistance. Each truss rests on two crown posts by means of collar purlins composed of two beams connected with a particular scarf joint known as *dardo di Giove* (Fig. 6b). The central mezzanine is supported by tie beams that lay on two longitudinal wooden beams resting on pillars. The tie beams are located under the rafters but are not connected to them. The rafters and the tie-beams rely on the external masonry wall, to which they are connected with metal anchorages. Instead, the rafters are nailed to a 2-meter-long wooden corbel protected with a bitumen coating only on the eastern side. Despite the wide span (1,650 cm), the rafters' cross-section is only 15 cm x 30 cm, and the distance between them is about 220 cm. These elements are, therefore, significantly slender compared to the loads they bear (previous interventions added a reinforced concrete slab to the flat tiles layer). Therefore, all the rafters show severe bending and deformation since they were already undersized when the palace was built (Fig. 6c). This timber structure already underwent several strengthening interventions: wooden elements were installed to reinforce existing beams, metal elements (plates, strips, etc.) were inserted to improve the connections both between the beams and among them and the supporting masonry, a cross-bracing system, was added to increase the roof stiffness.

Fig. 5. *Timber roofs types: (a) asymmetrical king trusses; (b) asymmetrical queen trusses reinforced with flitch plates at the joints and a double layer of tie rods; (c) intersection between asymmetrical queen trusses.*

Fig. 6. *(a) Timber rafter roof with crown posts; (b) connection between collar purlins highlighted by the presence of a wooden capital; (c) previous reinforcement for excessive deformation.*

3.2. ON-SITE ASSESSMENT OF DAMAGED TIMBER ROOF

The survey on Palazzo Costabili's timber roof structure was first limited to essential features (geometrical configuration, technology, and general state of conservation) to identify the most critical points. A more detailed analysis was later carried out on the structures that showed significant degradation, more specifically on the timber roof of the southern building, which was the most hit by the 2012 earthquake. The structural seismic response of timber roofs to an earthquake is extremely interesting as it highlights weaknesses that do not appear with ordinary loads [5]. The excessive static deflection of the rafters worsened after the earthquake, and one showed bending cracks due to the vertical component of the seismic action. First aid interventions propped the structural element using metal supports (Fig. 7a). Still, masonry also presented severe crack patterns: vertical cracks in the corners reveal the activation of a tilting mechanism, whereas horizontal cracks in the upper area, all along the longitudinal walls, in correspondence of timber rafters and beams point to a slide of the upper part of the roof (Fig. 7b, c). Previous seismic reinforcements from the 1990s featuring reinforced concrete slabs, tie-rods, metal anchorages, and a bracing system, didn't do much against earthquake damage.

For this reason, further analyses were performed by specialists (Fig. 8a, b), who assessed the materials' consistency and the actual strength with a combination of visual inspection of timber features and defects (visual grading) and instrumental non-destructive measurements of physical-mechanical properties (mechanical grading), according to the UNI 1119:2004 standard [22]. The visual assessment was carried out on all physically accessible parts, whereas the less accessible areas (such as the ends of the beams inserted into supporting walls) were investigated employing alternative inspection methods (e.g., by resistance drilling methods) to define their state of conservation.

3.2.1. THE WOOD SPECIES IDENTIFICATION SYSTEM

The identification of wood species was performed by specialists visually and with laboratory tests (microscope analysis). For the main structural elements (rafters), wooden samples show that the wood used in the roof

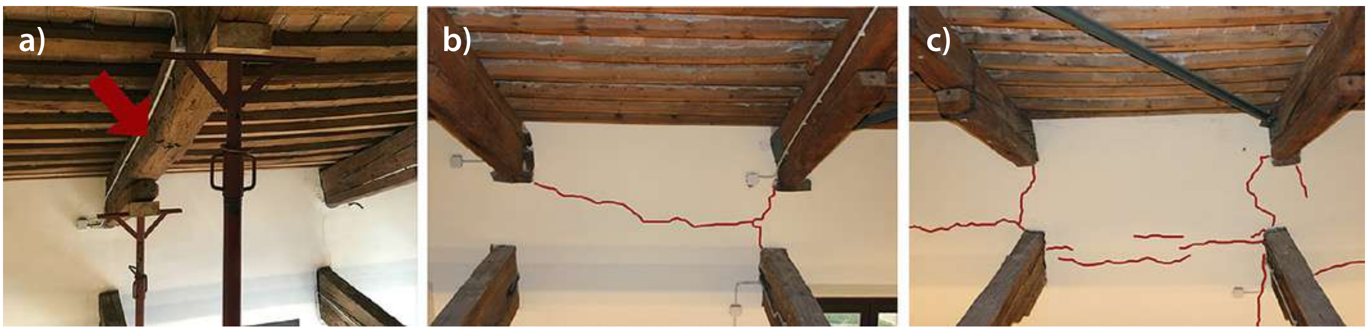

Fig. 7. *Seismic damages: (a) cracked rafter; (b, c) horizontal masonry cracks on longitudinal walls.*

structure is mainly Silver Fir (*Abies alba*) and Spruce (*Picea abies*), in line with the local construction tradition. Conifer was indeed the main species used in the timber structures of Ferrara's monumental buildings, whereas hardwood was mainly used for secondary elements. The use of fir is further proof of the importance that Palazzo Costabili might have had for the family since conifers do not grow locally, and their transport from the mountain regions was a costly practice [16].

3.2.2. WOOD MOISTURE CONTENT AND STATE OF PRESERVATION

The timber's moisture content was investigated with special instruments (drive in *Gann's electric hygrometer*), showing a percentage of humidity between 11-13%, a moisture level low enough to prevent biological and fungal attacks. No active decay or insect damage was detected. Previous deteriorations, such as fungus or xylophagous agents, were already found and solved in the past, as they are located only in previously consolidated timber members. However, the last restoration work removed the space between wood and masonry, preventing the aeration of beam ends and thus favoring moisture-due decay in case of water infiltrations.

3.2.3. EVALUATION OF MECHANICAL PROPERTIES

The assessment of the mechanical properties of timber was carried out by visual and mechanical grading. Non-destructive tests were performed with a drilling device (*IML Resi B-400 dynamometric drill®*) to define the effective (residual) cross-section. The investigation focused on the weakened parts, such as the extremities of the timber beams, close to masonry supports, which are also at higher risk of decay. The analysis found fibers with helical conformation and *cipollature* (ring shake) in several timber components, which is worrying as it compromises their earthquake and static strength performances. Finally, the mechanical properties were defined according to the classification of timber elements, based on wooden defects and decays, following the UNI 11119:2004 standard (Fig. 8c).

4. A STRENGTHENING PROPOSAL FOR STRUCTURAL SAFETY AND CULTURAL PRESERVATION

Inspection and assessments provided the data necessary to evaluate the roof's structural safety and plan potential restoration works that respect the minimal intervention principle and preserve original materials, structural systems, and techniques [4, 23]. An assessment of the areas that are most at risk and, therefore, a priority list for local interventions was defined [24], whose aim is to improve the structural safety of both the roof and the whole building.

Focusing on the southern building of Palazzo Costabili, a tie-rods system has been proposed to enhance the *masonry-box* behavior. This strengthening system,

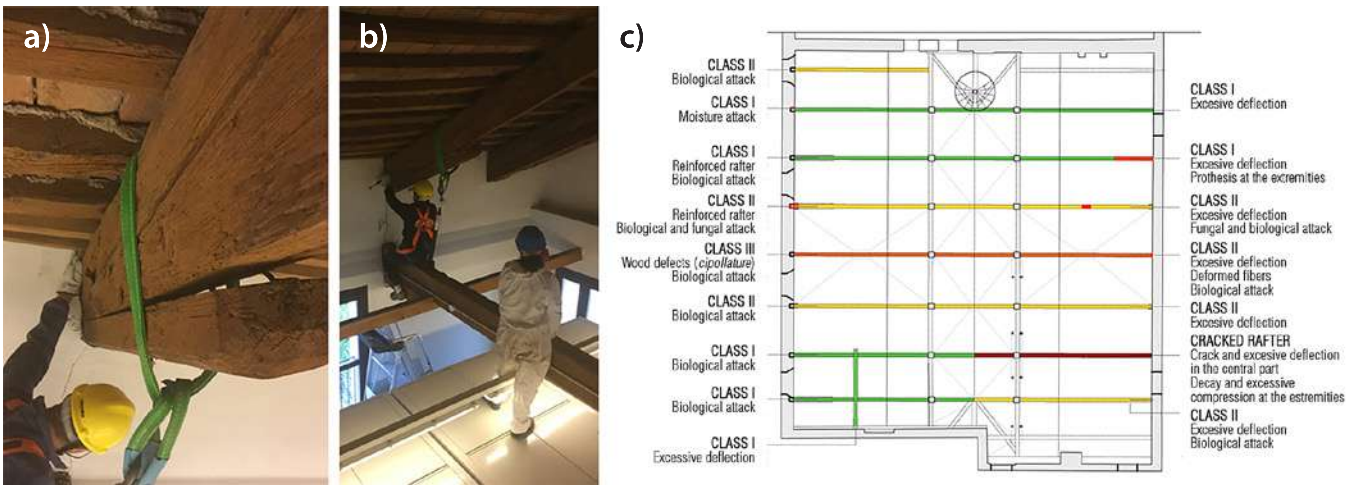

Fig. 8. *(a, b) The on-site assessment; (c) the classification of timber components with different colors (green class I, yellow class II, orange class III, red cracked rafter) according to their state of conservation (excessive deflection, biological attack, and wooden defects).*

connected to the roof structure, could transfer seismic shocks to the perimetral walls, thus preventing the upper part of the roof from sliding down. More specifically, two metal wires can be inserted in the central area of the building – the one most subject to tilting – in order to counter the roof's thrust and connect the lower parts of opposite rafters that are disjointed from horizontal wooden beams.

As per the timber structure, strengthening was calibrated explicitly according to the wooden structures' classification, state of conservation, and stress level. In particular, calculations to assess deformation were carried out to evaluate the real strength of the structure, and loads were surveyed in-situ to consider the structural resources of the existing construction. This analysis highlighted that the strength values of class I members are below the values prescribed by regulations for this kind of structure; therefore, no strengthening interventions were proposed. Class II and III elements showed reduced strength and suffered the most significant structural distress, as their stress values were above normal ranges. Urgent reinforcement is thus needed to avoid structural failure even with a slight increase in the bending stress (e.g., vertical seismic component), as already happened for one of class I or II elements. To preserve as much historical material as possible, the strengthening proposal of the original timber components suggests the addition of an intermediate support to transfer part of the static loads to the lateral supports with post-tensioned metal wires (Fig. 9). This active strengthening can be adjusted over time thanks to the calibration of the wires' tension utilizing telescopic props to guarantee durability. These elements also allow for reversing the deformation of the structure. The anchorage steel plate gives an additional contribution in balancing the inflection for intermediate support, located where the deformation is at its peak. This way, the number of metal surfaces in direct contact with timber could be reduced to a minimum to avoid moisture formation, which promotes surface decay. The only exception is the cracked timber rafter: in this case, the central plate should be enlarged to support the entire damaged area without replacing the original material.

Other interventions are required to improve roof stiffness. Indeed, despite the effectiveness of connections between members, the absence of efficient stiffening planking (the concrete slab over the flat tiles provided an inadequate seismic response in 2012) can cause problems in the case of earthquakes. A steel cross bracing was already inserted, but it is limited only to the central part of the roof, it is not connected to the perimetral walls, and it cannot homogenously transfer the seismic actions to transversal walls, avoiding the synchronous oscillation of the longitudinal ones. In order to enhance its effectiveness, the existing system (Fig. 10) should be completed with steel tubes with a circular hollow section, like the existing ones. The external span requires a specific bracing system because of shape and structural configuration irregularities. For this reason, at the edges, the cross-bracing consists of Ø32 mm metal bars connected to the timber structure in the upper part and the masonry walls at a lower level, in correspondence with the wooden beams of the mezzanine.

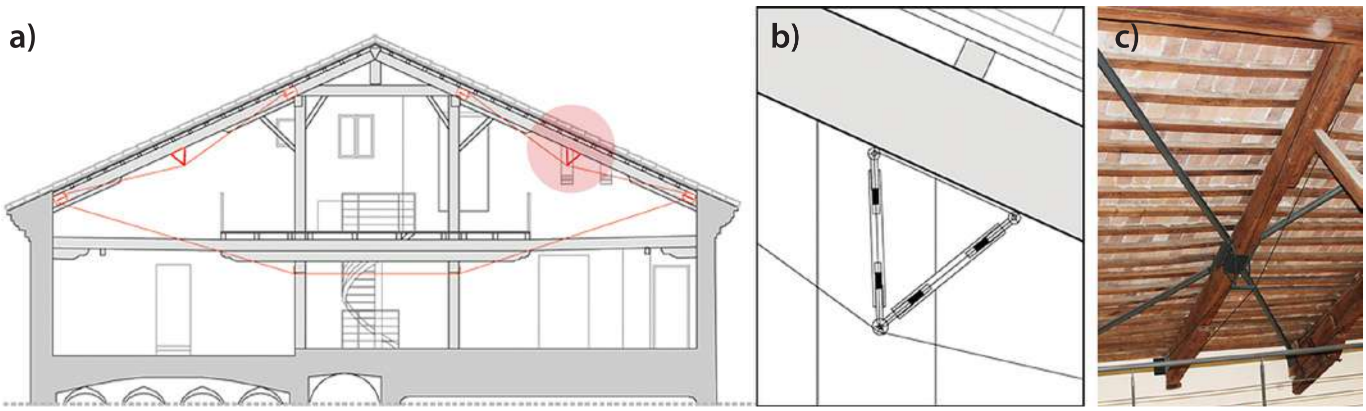

Fig. 9. *(a) A cross-section of the habitable attic room with the strengthening proposal for the roof structure. An intermediate support with post-tensioned metal wires has been added for class II and class III members: (b) a detail of the telescopic props; (c) a render of the reinforcement.*

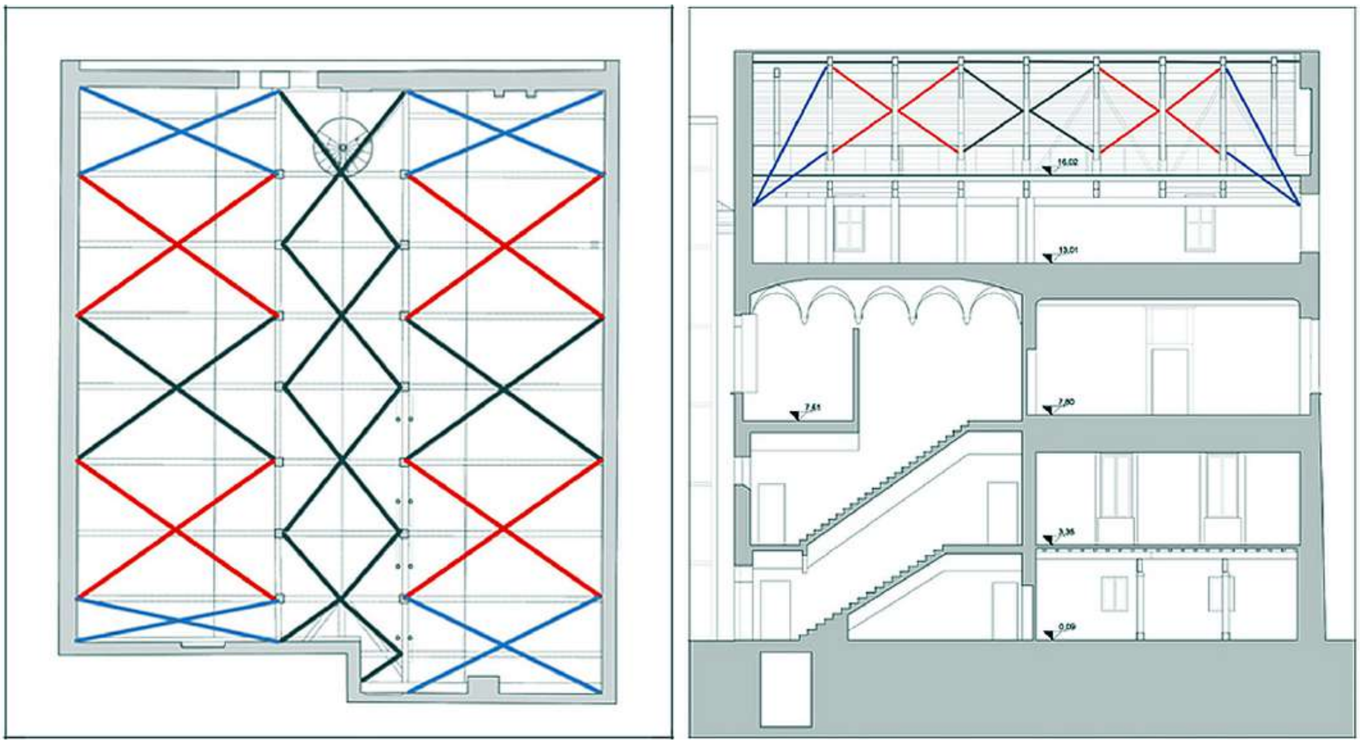

Fig. 10. *Improvement of the existing cross-bracing system of the southern timber roof: in grey are existing elements, in red is the new metal tube circular hollow section, while in blue are the new inclined metal bars.*

Finally, once the problems mentioned above have been solved, it is crucial to constantly control, monitor, and maintain timber structures considering the surrounding environment to minimize decay and subsequent interventions [25]. For the specific case of Palazzo Costabili, periodic direct visual surveys have been planned to assess possible modifications and identify the areas to be checked (bowing or leaning, cracking, wet areas, rot, decayed joints, etc.). Generally, such maintenance inspections should be carried out annually, but they are also needed after one-off events like earthquakes or storms. Moreover, since the current environmental conditions proved favorable for wooden preservation, temperature and humidity sensors should also be installed to detect possible alterations that may cause timber decay. Finally, maintenance activities (cleaning, minor repairs, etc.) should be defined accordingly to the result of regular inspections.

5. CONCLUSIONS

Interventions on historic timber roofs too often consist of widespread and excessively invasive reinforcements, if not in uncritical and complete replacement of the existing structures. This approach is due to the lack of critical knowledge-based analysis, recognition of the cultural value of existing timber structures, and poor understanding of their actual structural behavior (meaning residual mechanical performances and state of conservation). Situation and structural knowledge are thus essential to define strengthening interventions that preserve and reinforce historic timber roofs while preserving their cultural significance. The case study of Palazzo Costabili shows the importance of a critical knowledge-based approach to strengthening historical timber structures that considers the roof's features, its cultural value, and actual weaknesses.

This design of such a strengthening proposal both aims at increasing structural safety and preserving the authenticity of the historical construction. Issues such as compatibility, durability, and removability have been considered, as well as technical and structural requirements. An efficient balance between conservative and safety issues was sought, and strengthening has been precisely calibrated for each timber component. This was made possible by a knowledge-based method, considering the timber roofs' construction features and on-site assessments. It could be extremely useful in the future to preserve the original timber structure and improve its load-bearing function and the seismic behavior of the whole building.

In this regard, an interdisciplinary approach is fundamental: the cooperation between experts enables researchers to further analyze specific aspects, while coordination is key for a critical data analysis that provides a detailed state of the art of the structure object of the strengthening intervention.

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THE TIMBER ROOF STRUCTURE OF CHAPEL XVI AT SACRO MONTE OF ORTA: AN EXAMPLE OF CONSERVATIVE STRENGTHENING WORK

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Abstract

The principles of conservation of existing structures (like evidence of regional construction techniques) pose a significant challenge in the general field of structural safety, increasingly claimed by the new technical codes. The restoration building site of Chapel XVI at Sacro Monte of Orta San Giulio (Italy) testifies to a respectful design approach to enhance the timber elements still recoverable without distorting the original static scheme. The preliminary knowledge phase allowed for correctly interpreting the historical construction phases (including analysis of archival documents and diagnostic investigations), the vulnerabilities of the roof structure, and the peculiarity of the stone roof covering as a construction technique to be preserved and improved. The recovery project has exploited the potential of the laser scanner and micro-invasive diagnostic techniques to move towards limited replacement choices of wood elements that could no longer be recovered. The recovery of the roof of the XVI Chapel, as a pilot building site of an international research project on the maintenance of historical sites (INTERREG ITA-CH “Main10ance”), is evidence of a fruitful multidisciplinary discussion aimed at improving the methodological approach to the strengthening work of historical structures.

Keywords

Maintenance, Historical buildings, Timber roof structures, Strengthening work, Structural safety.

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1. INTRODUCTION

The restoration building sites are increasingly interested (also) by structural safety interventions to reduce static and seismic risk. The recent Italian technical codes [1–3] already provide a gradual approach to the existing built heritage through diversified phases of structural improvement by strengthening works that do not compromise the original structural construction concepts. Following the countless collapses and immeasurable losses of historical buildings severely damaged by the latest Italian seismic events [4–8], structural security becomes more and more a need to be pursued: conservation cannot be guaranteed without safety. However, reaching structural safety levels comparable to those required for new constructions becomes a challenge within a restoration site, where safety and conservation must dialogue together without prevailing one over the other in an aprioristic way. If anything, when timber structures have a high value of technological evidence, conservation has a prevailing hierarchical position in interventions.

This paper describes the methodological approach, the design process, and the execution of the maintenance works for the timber structures and roof covering in the XVI Chapel at the Sacro Monte of Orta. The intervention was scientifically monitored within an international research project (European Research Prog. INTERREG Italy – Switzerland "Main10ance"), which has as its theme the good practices of scheduled maintenance of big architectural sites [9].

The Sacri Monti of northern Italy is a UNESCO heritage site that includes nine complexes consisting of groups of chapels and other buildings with many artistic masterpieces. All buildings are part of a high-quality natural environment. The first complex was founded in the late 15th century in Varallo Sesia, and the others were constructed up to the end of the 17th century and dedicated to different aspects of the Christian faith [10]. The landscape context in which these sites are built is very suggestive and in close contact with the environment. However, it also involves greater sources of degradation on the plasters [11], walls, and roof systems due to many causes, including (principally) humidity. It is, therefore, essential to plan maintenance with good intervention practices, evaluating not only the compatibility of the restoration materials but also developing non-invasive diagnostics methods to monitor the decay of the materials [12].

The Sacro Monte of Orta is located on a wooded peninsula that juts into Orta Lake. The first idea for a new religious complex in Orta dates back to 1583, modeled after the religious itinerary of the Sacro Monte in Varallo. It only became a reality in 1590, thanks to the contribution of Abbot Amico Canobio from Novara and according to the project of Capuchin architect Cleto from Castelletto Ticino. Twenty chapels immersed in a natural surrounding illustrate episodes from the life of St. Francis, with sculptures and paintings, distributed along a path that winds on the top of the hill, opening up spectacular views of the lake and the island of San Giulio. Statues and paintings create scenes of deep realism: the intimate and natural atmospheres typical of early seventeenth-century Lombard art are combined with the lively theatrical Baroque style of the end of the century [10]. The route ends at the Church of St. Nicolao; a proto-Romanesque building completely redesigned during the seventeenth century to recreate the spaces of the lower Basilica of Assisi. In the last decade, some studies have underlined the relevance of the relationship between the architecture, the masterpieces of Sacro Monte, and the landscape (with his anthropic transformations due to centuries of men's work on the natural environment) along with the importance of a planning policy imprinted towards the respect and the safeguard of the stratification of significant elements [13]. Among these chapels is the XVI, dedicated to *St. Francis, who returns to Assisi from the Verna before dying* (Fig. 1). Like all the historical buildings belonging to the monumental complex of Sacro Monte of Orta, the XVI Chapel was characterized by a stone masonry structure, an attic vault, and a stone roof with timber structures.

The timber roofing structure was in a decaying condition and therefore required maintenance interventions, including the partial replacement of the stone slabs and the safety improvement of the timber framework. A survey phase with laser scanners and resistographic tests on the timber elements anticipated the design project. This survey phase revealed a significant number of degraded elements; however, the repair project was oriented towards preserving many parts, limiting replacements, and eventually carrying them out with the same wood species and similar dimensions to the original elements. At the same time, it was necessary to solve some vulnerabilities of the nodes of the roof structures in order to strengthen their static resistance and reduce the seismic risk.

Fig. 1. *The XVI Chapel at the Sacro Monte di Orta.*

For this purpose, the design approach and guided execution of the restoration work in the XVI Chapel represent a virtuous example of the proper dialogue to reach a compromise between safety and conservation. In particular, this pilot site intends to promote a new perspective to reverse the standard logic of total replacement of the structural elements, as usually do when these are not strictly involved in connoting the external image of the building. A more significant design effort can usefully address the conservation of most of the recoverable timber elements, assuming them as a material document of historical techniques.

2. KNOWLEDGE, SURVEY, DIAGNOSTICS

First of all, the correct design of the structural recovery of a historic construction must start from the reconstruction of the building phases, the geometric survey, and non-invasive or micro-invasive diagnostics for the characterization of the materials. The structure survey was carried out by Prof. Andrea Lingua and his team (Politecnico di Torino) by laser scanning with the return of a point cloud of the timber structures (after remove of noisy points, Fig. 2). The remarkable technology allowed determining with precision the geometry of the structure and the dimensions of all the elements, despite the structure itself being apparently disordered. This work enabled successive accurate structural modeling.

The structural scheme consists of the following sequence of mutually supportive elements:

- two main trusses (North and South), resting on the perimeter walls and characterized by tie beam partially resting on the ribs of the vault (red in Fig. 3) and rafters (green in Fig. 3), worked with a *half-lap* joint at the ridge;
- quay beam on the crossing of the rafters (yellow in Fig. 3), characterized by two overlapping elements;
- two cantonal beams (blue in Fig. 3) resting on the perimeter walls through joists;
- ridge beam (purple in Fig. 3), resting on the West side on the masonry. On the East side, the ridge beam is supported by recent props and located below the intersection with the two cantonal beams, with partial support on one of the rafters (transparent violet in Fig. 3).

The rafters of the trusses on the East side also support a cross beam on the East slope. This cross beam supports the rafters of the same roof, and at the cantilevered final part, also similar cross beams on the North and South slopes (all the cross beams are highlighted in dark yellow in Fig. 3). The structure is completed by minor rafters, with sections varying from 16 cm x 16 cm to 12 cm x 12 cm, including some strongly round sections. The arrangement of the minor rafters is laid out on all the slopes with orientation “hut” converging towards the ridge (Fig. 7). These minor rafters lie with fan-shaped planimetric distribution to support the thiny stone slabs covering. In the Western part towards the chapel’s entrance, the fan-shaped distribution of the rafters is arranged to connect the different heights of the main ridge and the ridge of the frontal tympanum.

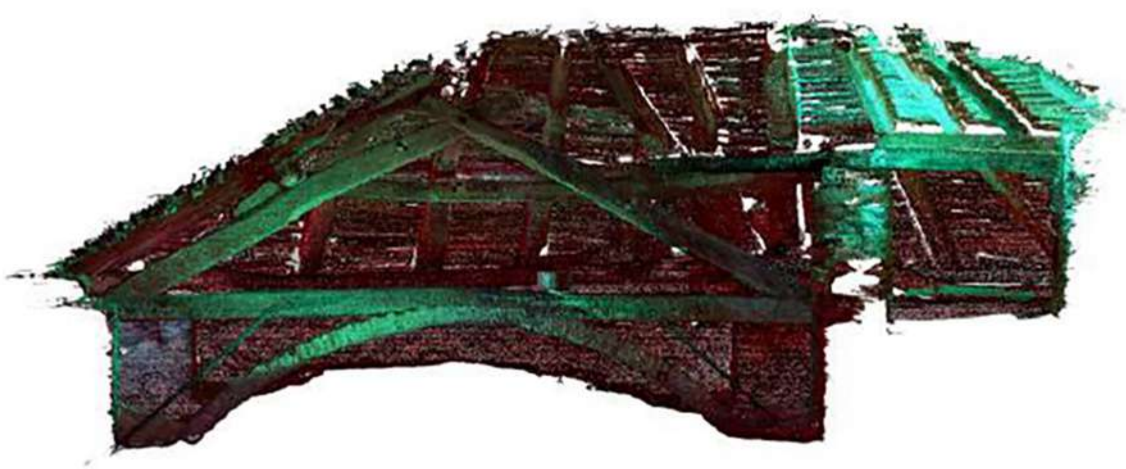

Fig. 2. *Survey image by the laser scanner.*

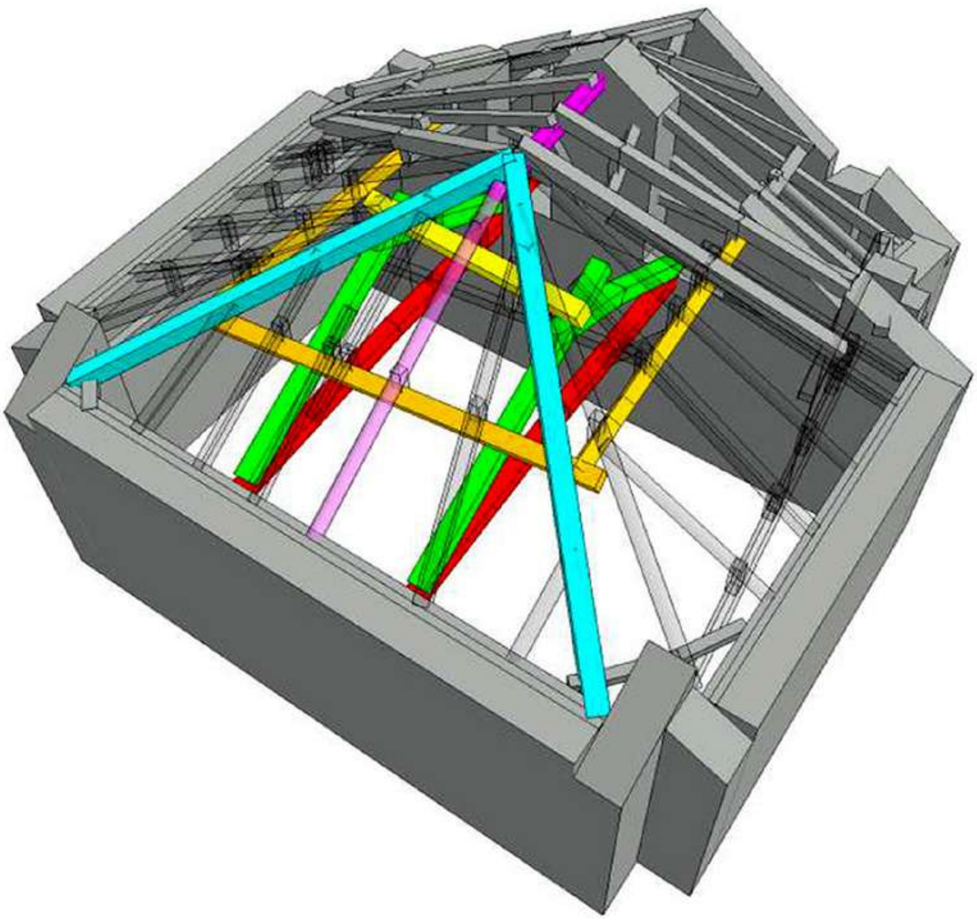

Fig. 3. *The structural frame of existing timber elements.*

2.1. CONSERVATION STATUS OF THE MAIN WOOD ELEMENTS

The timber structures were investigated using a resistographic test conducted at the final part of the elements, repeating the test at a short distance when a potential degradation has been identified. The most significant degradation was found near the support of the truss, (corresponding to point 1 in Fig. 4), strongly compromised by humidity and infiltrations of rain from the roof covering (stone slabs).

Both trusses are made up of timber tie beams, section 20 cm x 20 cm, resting on the East and West walls and timber rafters of a similar section (Fig. 5a). The connections of the rafters on the tie beams are arranged as common practice, with the support surface oriented according to the bisector of the angle of incidence (Fig. 5b). It should be noted that the tie beams of the trusses have metal elements inserted into the masonry walls and connected to anchoring brackets on the external sides, thus forming a metallic tie of the ribs of the masonry vault (Fig. 5a). Both tie beams rest on the extrados of the vaults and are slightly curved (deformation just under 10 cm), with the supports at lower altitudes than the center lines (Fig. 5c). The top of the rafters has a *half-lap* coupling connection, therefore without the presence of the classic connection by the king post, with both rafters continuing in the complete section beyond the coupling area (Fig. 5d). The North truss has the supporting section of the tie beam totally degraded, corresponding on the East side (Fig. 5b), with a partial impairment of the coupling surface of the corresponding rafter. The South truss has conditions of partial degradation of the tie beam in the middle section and at the East final part. The East rafter presents conditions of partial deterioration in the top portion. The trusses support a quay beam on the fork generated by the crossing of the rafters (Fig. 5d). This beam is made up of the overlap of two beams in order to reach the correct support level for the cantonal beams and the

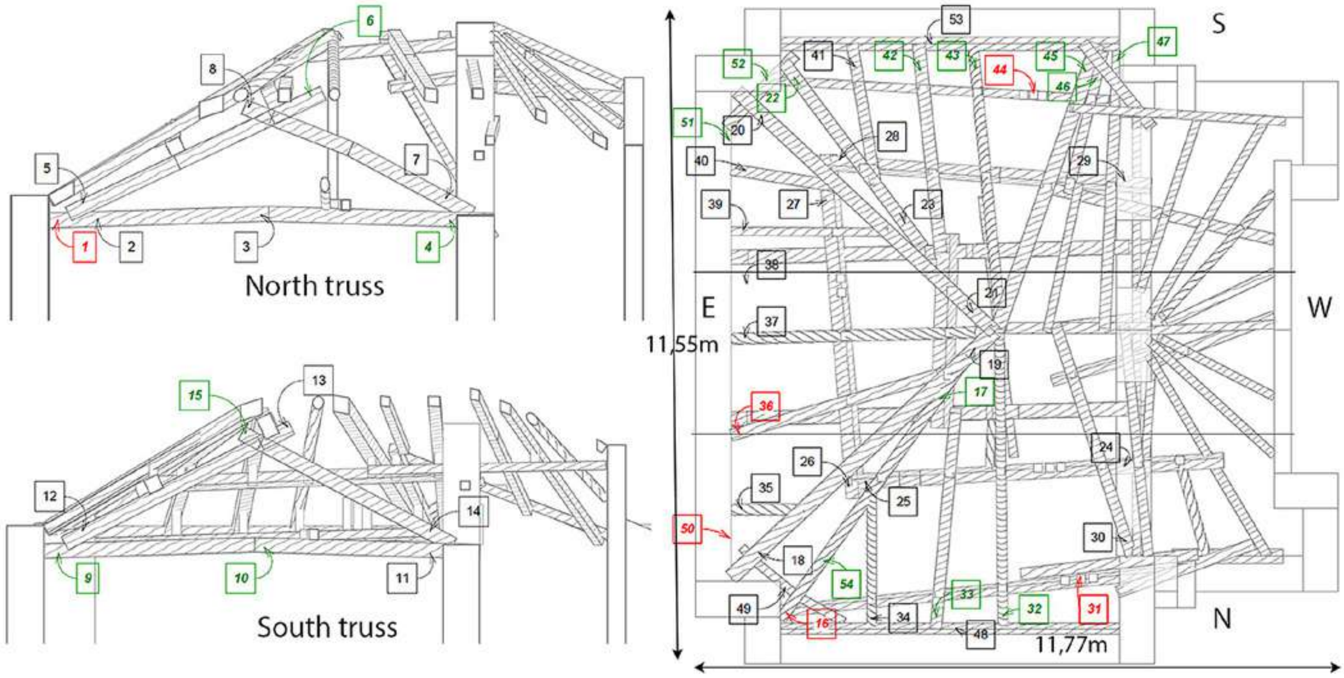

Fig. 4. *Results of resistographic tests carried out on wood structures: in black, the points in good conservation, in green, those corresponding to slight or moderate degradation, in red, those corresponding to advanced degradation.*

Fig. 5. *(a) North truss; (b) detail of the tie beam-rafter connection, in evident degradation inside the masonry support; (c) tie beam rests on the extrados of the vaults; (d) detail of a half-lap coupling connection.*

intermediate rafters of the East slope. The connection of the tops with the quay beam and with the rafters of the North and South slopes guarantees the overturning stability of the trusses.

The two cantonal beams, with 20 cm x 20 cm sections, rest at the bottom on the East masonry near the North and South corners and high on the quay beam, continuing cantilevered until you reach the peak (Fig. 6a).

By observing the conformation of the trusses' rafters and the position of the ridge beam, it would have been possible to hypothesize the presence of a beam resting on the final part of the rafters, supporting the ridge beam (Fig. 6b). To a more careful analysis, it has been understood, instead, that such workmanship corresponds to an early construction phase of the building still without the forepart on the West side, resulting in the symmetrical roof pitches on the East and West side.

The central rafter of the eastern slope of the roof converges with the two cantonal beams in the same position. The ridge beam is in this position of convergence below the converging beams without finding any support. Currently, it is supported by two props of apparent recent insertion, one inclined and resting in the inner masonry on the western side. The other is vertical and rests on a crossbar resting on the tie beams of the trusses (Fig. 6c).

On the eastern layer, a cross beam (20 cm x 20 cm) is resting on the rafters of the two main trusses. The cross beam of the northern layer, with section 16 cm x 16 cm, appears carefully chosen with a deformation that facilitates the support on the main cross-section of the opposite façade.

With regard to wooden frame supporting the stone roof covering, integrative elements are paired to some minor rafters degraded, especially in the terminal section. Along the entire perimeter of the roof, on the top of the walls, there are timber quay elements connected by means of wood processing or by partial overlapping, completed in the corners by diagonally arranged elements.

Fig. 6. *(a) Laser scanner survey of the roof area, including the two cantonal beams; (b) hypothesis of the original configuration of the ridge beam; (c) props to support the ridge beam.*

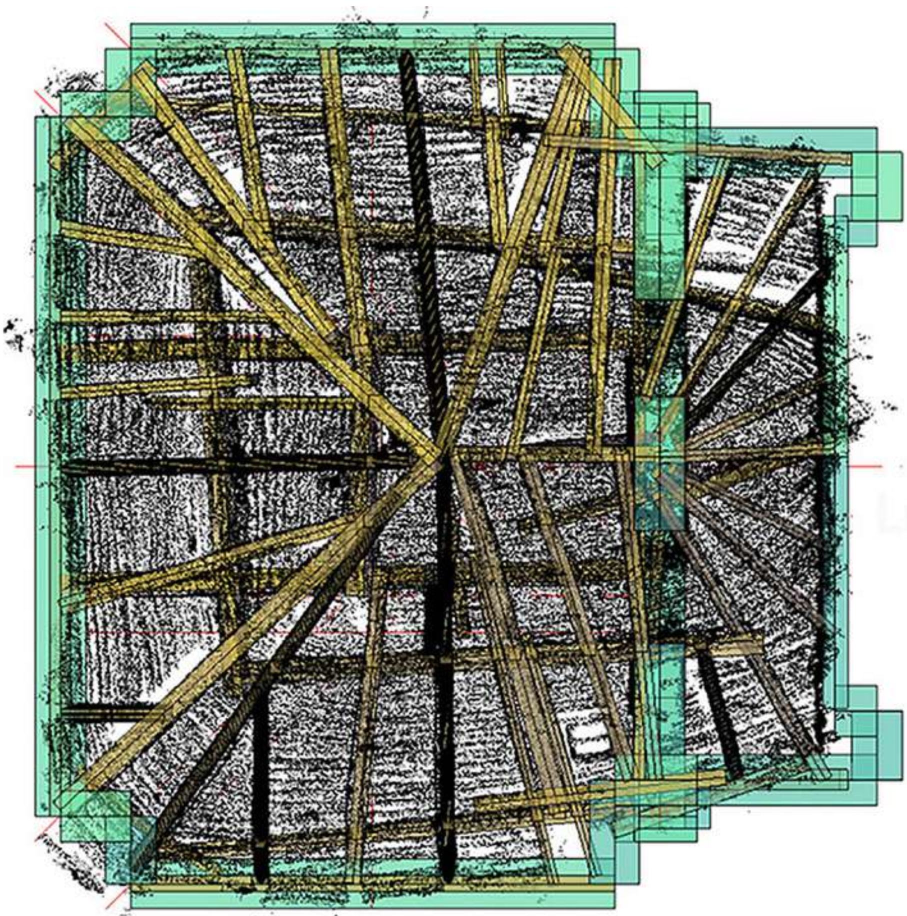

Fig. 7. *Arrangement of the minor rafters in the "hut" roof schema.*

3. STRUCTURAL VERIFICATION OF TIMBER ELEMENTS

A three-dimensional structural model, carried out by finite element software, was made with the positioning of all the wooden elements in perfect match with what was detected by a cloud of points of the survey by the laser scanner. The bonds between the overlapping wooden elements were modeled by inserting vertical connection elements to simulate the reciprocal support connection. The structural modeling has taken into account all integrative elements, resulting in the transfer of a significant portion of the weight of the stone-roof covering on the supporting timber elements. This model allowed the evaluation of the thrusts of the two trusses on their chains and the horizontal forces transmitted by rafters to the quay elements. With separate modeling, the individual components were taken into account to compare the stresses with models that are much simpler and easy to control.

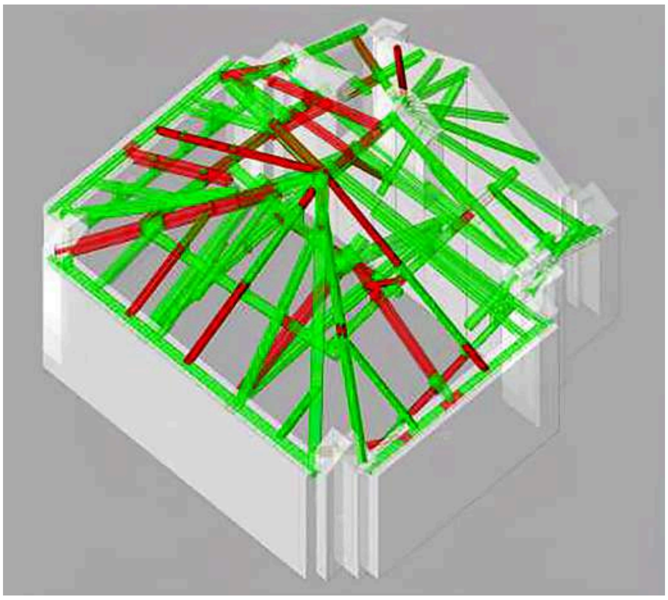

Fig. 8. *Results of static verifications for the roof elements.*

The analyses covered only static load combinations by Italian technical standards [1, 2]. Therefore, the loads considered have the following values: 0.20 kN/m^2 for the actual weight of the timber structure of the roof; 2.8 kN/m^2 for the weight of the roof covering stone (3 layers of slabs, 3 cm thick each one); 1.5 kN/m^2 for the snow overload, for a total of 4.5 kN/m^2. The checks were carried out considering chestnut wood, with the D24 to D18 resistance class [14]. The graphic representation of the verifications of the timber elements is shown in Figure 8, in which the elements with suitable dimensions are represented in green, while those with inadequate sizes are in red.

Since the recovery project has provided only a partial replacement of the roof structure, given the complexity of the geometric distribution of the beams, it was not considered appropriate to insert any top ring beam, even with a compatible masonry technique, so as not to remove the entire scheme of the minor rafters. In fact, the structural work has prioritized the static safety of the original timber structures of the roof, improving, first of all, the connections between the elements, although with a view to reducing the seismic risk in the future. For these reasons, the analysis of seismic vulnerabilities has not yet been carried out in depth. However, further strengthening work of seismic safety can be undertaken later, always favoring punctual reinforcements [15, 16].

4. THE RESTORATION AND STRUCTURAL RECOVERY PROJECT

The restoration project has provided for the conservation and restoration of the elements most characterizing the roof, particularly the two trusses and one main cantonal beam. Through the preliminary surveys, it was possible to identify the undersized or degraded wooden elements, on which the subsequent structural analysis revealed excessive stresses, in particular in some elements spaced by an excessive distance. Therefore, it has been planned to preserve the less affected elements, replace newly or degraded substitute elements, and provide a repositioning for some of them (providing a closer distance where it was necessary) in order to ensure a better distribution of the weight roof covering over them. The adjustments were very modest and only due to the search for the best mutual positioning.

Fig. 9. *(a) Positioning of the timber prosthesis; (b) the timber-timber prosthesis performed.*

Also, it has been planned to replace all quay beams on the North, East, and South slopes and the angular elements, preserving the quay beam incorporated in the West masonry while still ensuring the connection with the new elements of the other layers. The replacement of the quay beams took place by the slight lifting of the minor rafters acting in such a way as not to unbalance the entire structure, prop it up and fan it to ensure the stability of the whole. Moreover was planned to replace the ridge beam and add two cross beams parallel to the same beam, creating a frame/scheme with a more precise structural concept. The supporting knot of the South truss, compromised by the material degradation, was reinforced with a conservative solution by means of timber-wood prosthesis inserted in contrast with clamping screws, without resorting to epoxy resin prosthesis of undoubted/unproven compatibility (Fig. 9). The main connection was made with screws passing from the extrados rafter to the intrados of the tie beam and between the integrative transverse element and the tie beam.

During disassembly, the timber elements were separated by distinguishing elements in good conservation conditions from degraded elements to be eliminated (Fig. 10a). The separation of the elements was submitted to the Work Director for approval. The direction of the work was carried out by eng. Giovanni Vercelli, with the support of the technicians of the Ente di Gestione dei Sacri Monti. Recoverable elements have been cleaned and treated to improve their conservation (Fig. 10b, c). During the repositioning phase, the timber elements were remodeled in order to adapt to the geometry of the bearing structure, alternating the recovery elements with the replacement elements in chestnut wood (Fig. 10a, b).

Fig. 10. *(a) Placement of new timber elements alongside the original ones cleaned up; (b) integration of the new quay beams into the original structural scheme; (c) cleaning and static improvement of the area around the ridge beam; (d) reconstruction of the original stone roof covering.*

The existing stone roof covering has been relocated following the traditional construction techniques (Fig. 10d), replacing the minor support degraded elements and adapting to the new stone slabs.

5. CONCLUSIONS

The roof recovery in the XVI Chapel at the Sacro Monte of Orta (Italy) contemplated critical phases of study and preliminary analysis. These have allowed not to follow a generic road of total replacement of the roof, enhancing the original elements still statically valid. The assessment of the structural safety of a historic building must be carefully interpreted, also calibrated toward the preservation of the original construction technique. Safety and conservation are two important requirements that Italian technical standards can enhance and bring together. The designer must demonstrate the suitable sensitivity to set good static and seismic improvements of the ancient timber structures without compromising their conservation, especially concerning the original construction techniques. The original structural schemes are to be conserved and improved statically and seismically using additional elements in wood or with steel reinforcements.

This recovery building site has been able to enhance a methodological path that has not provided *a priori* total replacements or invasive interventions on the existing building. The design process has been characterized by many technical reflections that have led to the joint effort to preserve and consolidate rather than remove. This methodological approach is, in fact, based on the concept of increased safety made for subsequent steps and interventions, especially in the context of monumental assets.

The historic timber structures represent an inestimable architectural heritage made of architectural history and construction techniques of the past that require simultaneous conservation of authentic elements, enhancement of original static *schema*, and structural safety improvement. The result of the work was excellent in terms of balance in conservation and improvement of the static safety of the timber elements of the roof, also safeguarding the original composition of the stone covering. Moreover, it represents a referring example for the maintenance interventions on similar historic buildings widespread on the territory in the same cultural context.

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Authors contribution

Conceptualisation, MZ, AG, SF, GV; methodology, MZ, AG, SF; software and structural modeling, AG and GV; validation, MZ; investigation, AG, MZ, GV, SF; technical solution for strengthening AG, GV; writing – original draft preparation, AG; writing – review and editing, MZ, AG, SF; supervision, MZ.

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THE ROOF STRUCTURE OF THE MEN'S ORATORY OF THE ALBERGO DEI POVERI IN GENOA

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Abstract

The Genoese roof structures are characterized by the originality of their construction and technological solutions compared to other cultural and geographical areas, including neighboring ones, and, in particular, by the unique connection with the underlying wall structures, by their thrusting nature and by the scarce or non-existent presence of trusses.

From the second half of the 16th century, in the roofs of large spaces such as churches and assembly halls, the wooden elements of the main roof structure are supported by solid brick pillars resting on arches or the ribs of the underlying vaulted structures.

This paper describes the building features, the state of preservation, and the restoration and consolidation project of the roof and the underlying vault with timber ribs and plastered reed mats on the intrados of the Men's Oratory of the Albergo dei Poveri in Genoa, one of the largest still preserved in the monumental complex and one of the most imposing among those still present in the city.

Keywords

Wooden structures, Timber structure vault, Consolidation project.

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1. INTRODUCTION

The paper focuses on the description of a case study that is particularly significant in terms of its size and construction characteristics: the roof structure and wooden vault of the former Men's Oratory of the Albergo dei Poveri in Genoa. The contribution has the twofold aim of documenting the construction features and the state of preservation of one of the largest wooden vaults and the related roof structure, which have preserved the original conception in Genoa, and illustrating the definitive restoration and consolidation project. The study is contextualized and compared with the roof structures of all the monumental complexes referring to the Genoese building tradition. The state of decay, the collapse of a portion of the reed mat vault, and the structural analysis results have determined the need to prepare a structural consolidation intervention to ensure suitable safety standards in relation to the intended use of the Oratory.

2. THE ALBERGO DEI POVERI IN GENOA

The Albergo dei Poveri in Genoa is a monumental complex, raised from 1656 onwards to host the various charities present in the city; the leading supporter and financier of such a project was the Genoese nobleman Emanuele Brignole (1617-1678).

The ex-novo construction of a monumental building intended to house the poor had a strong political and social value. It first affirmed the importance for the ruling classes of the time of finding a solution to the problem of the uncontrolled presence of the poor in the city and, at the same time, glimpsed the possibility of international political redemption for the Republic of Genoa [1].

The Regulation of the charitable institution was based on three rules – order, prayer, and work – which also influenced the unique organization of the vast complex. In fact, the project included a square plan with buildings arranged in a cross shape in the center of which stood the church and, under the dome, the high altar, the intended visual and symbolic center of the complex, visible from all the surrounding areas. However, the original project was not completed, and the construction was interrupted at the beginning of the XIX century so that the western wings were reduced to long corridors connecting the south and north fronts. The magnificence of the complex is attributable more to its immense size rather than to its decorations. These latter are concentrated solely in the entrance hall, the monumental staircases leading to the atrium and the first floor from which there is access to the ante-church and the church, and other reception rooms concentrated mainly in the southern building to the west.

In the XIX century, following social changes, the structure had become a hospice; at the time of the Second World War, it was classified as a Public Assistance and Charity Institution in accordance with Law 6970 of the 17th of July 1890. On the 4th of March 1912, the Ministry of Education declared the building a "valuable monument of art and history" in accordance with Law 185/1902 and Law 364/1909. The complex, owned by the "Emanuele Brignole" Personal Services Agency, was decommissioned at the end of the last century and then loaned to the University of Genoa for fifty years. In recent years, the University has partly restored the building and used three floors of the east wing to house classrooms, a library, and a book depository, as well as the former women's Oratory, i.e., the body of the central cross of buildings to the east of the church, as a lecture hall. In the southern building, part of the Law Faculty Library extends, and other classrooms and departmental offices can be found, in addition to the language center on the floor above the so-called ante-church.

The Laboratory of Analytical Methods for Restoration and the History of Buildings and the Specialization school in architectural and landscape heritage of the University of Genoa have been studying the state of conservation and possible uses of the unrecovered areas of the complex since 2013 [2–5]. The former men's Oratory, i.e., the building to the left of the church, towards the west, is abandoned and in a precarious state of preservation. Still, a project has now been approved for its restoration as the University's second lecture hall, to which this article refers with specific attention to the roof structures.

3. THE ROOF STRUCTURE OF THE ALBERGO DEI POVERI

Compared to other cultural and geographical areas, including neighboring ones, the roof structures in the Genoa area are characterized by their unique connection with the underlying wall structures, their thrusting nature, and the scarce or non-existent presence of trusses [6–9]. From the second half of the 16th century, a distinction was made between the roofs of large spaces, such as churches and assembly halls, and those of more important residential buildings. In the first case, the wooden elements of the main roof structure are carried by solid brick pillars resting on arches or the ribs of the underlying vaulted structures. In palace or villa architecture, on the other hand, the roofs (generally pavilion-shaped) are reminiscent of naval construction techniques, with resistant systems organized according to flat frames, with beams on props, where the diagonal struts are load-bearing elements, like the joists of the secondary beam level, on which the planking is fixed [6]. The diagonals of the pitch, in this way, do not load the edge of the wall box, which, due to the construction characteristics of the walls, is a point that tends to be weak, and the weight of the roof is conveyed onto the perimeter walls through the lame frames described above.

The choice of the wood species used in the roofing structures depends mainly on two factors: the availability in the woods of the territory of the Republic of Genoa and the workability with the tools available at the time. Generally speaking, larch or oak is used for the main load-bearing structures (struts and purlins or beams) and chestnut for the secondary structures (rafters and planking) [10, 11]. Re-used elements from the demolition of ships and hulls of various kinds are not at all rare.

The Albergo dei Poveri offers examples of both types of traditional Genoese roofing identified in the literature [Fig. 1], as it includes both elongated buildings (dormitories, church, oratories, and corridors) and towers located in the corners and the central bodies. The prevailing typology in the arms of the central cross and the wings, i.e., the parts connecting the corner towers to the central ones, are characterized by pitched beams perpendicular to the external fronts, carried by pillars resting on solid brick arches fitted with iron tie-rods. Above the main frame, the rafters follow the slope of the pitch and rest on the dormant beam at the masonry; a fairly regular board is nailed to them, which supports the slate roofing (slate slabs, usually square in shape, 55 ÷ 61 cm wide and 4 ÷ 7 mm thick) fixed with lime mortar. In almost all rooms, there were self-supporting wooden ribbed vaults underneath the roof structures with pendants connected to the roof structures as a static reserve in case of deformation or subsidence.

Inside the Albergo dei Poveri, there are several variations of this basic structural scheme due to the different organization of the underlying masonry or pillars. The basic scheme is found in the arms of the central cross, with five or seven pillars, depending on the width of the underlying arch (9 m and 16.5 m, respectively). In the case of the roof of the central body at the church's nave, instead of having a succession of arches, the roof structure rests directly on the underlying barrel vault of solid brickwork [11].

During the Second World War, the Albergo dei Poveri suffered a great deal of damage, especially to the roof structures, due to the devastating effects of incendiary bombs. Many roof structures were thus rebuilt after the war by the Genio Civile (the roof of the Church or former Women's Oratory, the roof of the east wing with the southern tower, the roof of the eastern part of the north wing, and the advanced central portion of the main building to the south). The Genio interventions were carried out with reinforced concrete structures and prefabricated SACCAI-type beams, repeating the pre-existing structural scheme but using different materials. The roof structure of the former Men's Oratory, with its large trough vault with lunettes consisting of five solid brick arches and, between them, vault fields with wooden ribs and plastered reed mats on the intrados, is one of the largest still preserved in the monumental complex and one of the most imposing among those still present in the city.

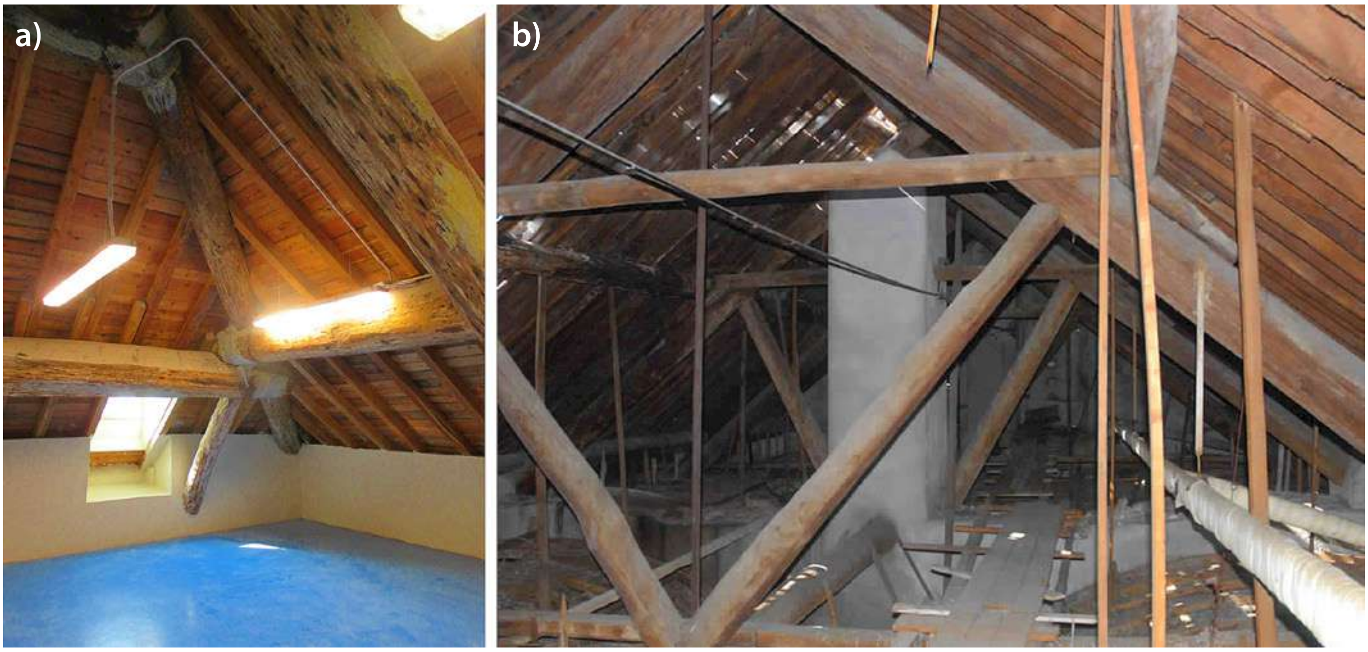

Fig. 1. *Examples of the two types of roof structures present in the Albergo dei Poveri complex: (a) the roof structure of the west tower in the south wing of the complex; (b) the roof structure of the west side of the north wing.*

4. THE ROOF OF THE MEN'S ORATORY: GEOMETRIC AND CONSTRUCTION DESCRIPTION

The former Men's Oratory, which occupies the western arm of the central cross of buildings, is symmetrical in position to the former Women's Oratory in relation to the church but smaller in size (Fig. 2). In the original project of the Albergo dei Poveri, both Oratories were conceived in close connection with the church, as two of the thirteen points from which the guests of the structure could attend the Holy Mass, but, as mentioned, it was never completed. Therefore, the former Men's Oratory is shorter than the Women's.

The large room of the Oratory, 15.5 m wide and 25.6 m long, is connected on one side to the church, from which it is separated only by an imposing wooden and glass door, and on the opposite side to the west wing of the complex (Fig. 3). This wing is, in fact, reduced to a two-level corridor connecting the north wing to the south entrance building. The ground floor corridor is covered by solid brick cross vaults, while on the first floor, the pattern described above is repeated, and the cross vaults are made of wooden ribs and mats, anchored to solid brick transverse arches supporting the roof structures. In the large room of the former Oratory, the corridor of the upper floor opens up with a balcony that allows a view of the room below and the high altar of the church.

A pitched roof covers the Men's Oratory with a ridge in an east-west direction. The roof covering is made of slate laid with lime mortar *alla genovese*. Parallel to the eaves lines, there are two attic walls with barbicans. This part of the roof of the Albergo dei Poveri does not appear to have ever been replaced except for some localized consolidation work. The roof is structur-

Fig. 2. *Location of the Men's Oratory in Genoa's Albergo dei Poveri complex.*

Fig. 3. *East and West views of the men's Oratory.*

ally composed of longitudinal beams (purlins) and the central ridge beam (circular section with a diameter of 30 cm) resting on the terminal masonry and five masonry arches with pillars at the purlins. The masonry arches are made of three-headed masonry 110 cm thick and 110 cm wide (Fig. 4). There are also wooden struts between the purlins to stiffen the system. Joists with a section of 7 cm x 10 cm are placed above the main frame, on which the planking rests and, above it, the covering. Connected to the roof by wooden pendants are the vault ribs to which the reed mat is attached. The wooden ribs have a cross-section of 5 cm by 13-15 cm (and are made up of two connected boards) with a center-to-center distance of approximately 41÷ 47 cm. Due to difficulties accessing the roof structure, it was impossible to identify the elements' wood species.

A barrel vault, with lunettes and a pavilion head towards the church, covers the large room of the former Oratory. The vault, as already mentioned, is made of five solid brick arches and, between them, fields made

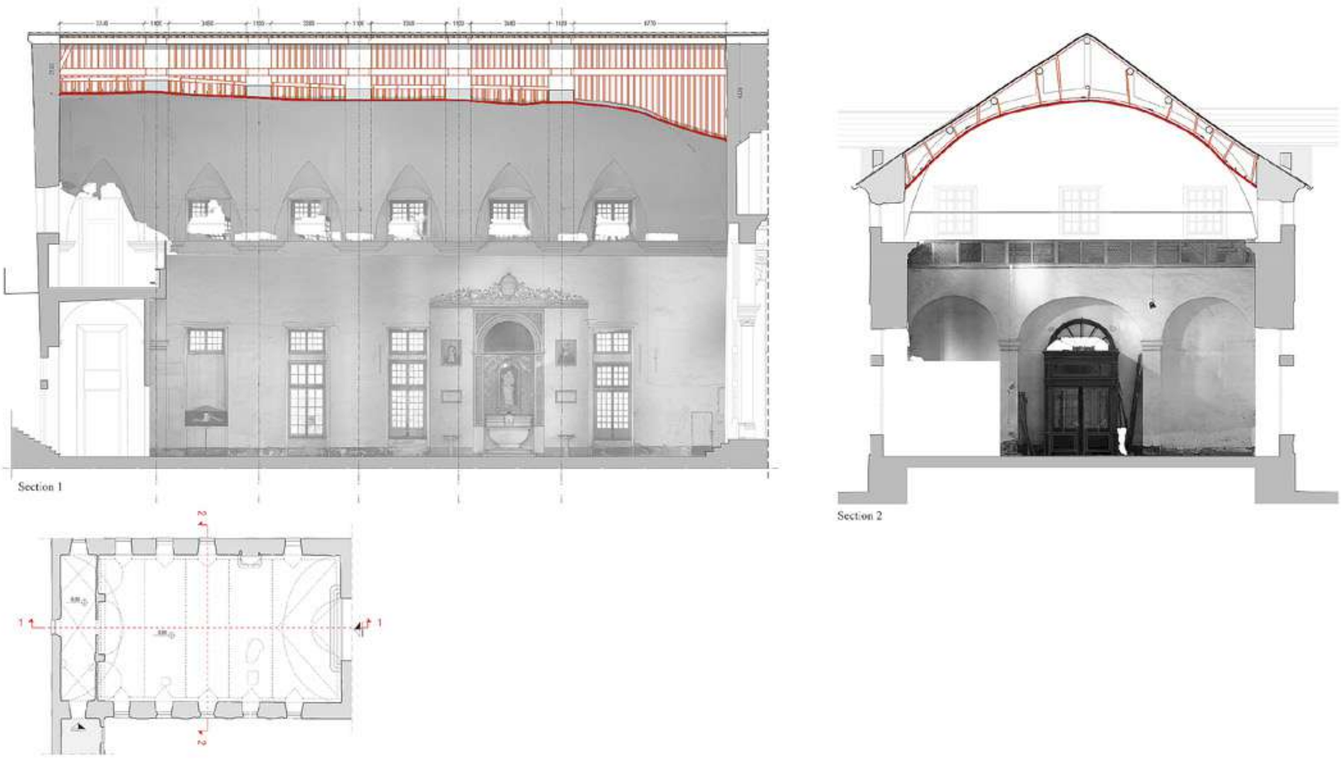

Fig. 4. *The roof structure's current configuration; longitudinal and cross sections.*

of a structure of wooden ribs. These are connected to the roof rafters by wooden pendants, supporting a reed mat covered on the intrados with a thick layer of aerial lime mortar, with a white well-pressed lime surface finish; it has a maximum height from the internal floor of the Oratory of 15.7 m with a shutter at 9.5 m. Wooden vaults, their diffusion, construction techniques, and possible consolidation interventions have been the subject of several studies in Genoa [12, 13] and Italy [14–16]. Still, no interventions have been documented on structures of similar size to the former Oratory.

5. DIAGNOSTICS AND STATE OF PRESERVATION

In order to define the most appropriate restoration intervention for the parts of the vault with wooden structures and reed mat, it was necessary to carry out a preliminary cognitive and diagnostic phase of its peculiarities and criticalities [17].

A thermographic survey was then carried out to highlight the position of the individual constituent elements, hidden from view by the intrados, any anomalies, and particular signs (Fig. 5). A Laser Scanner survey and the

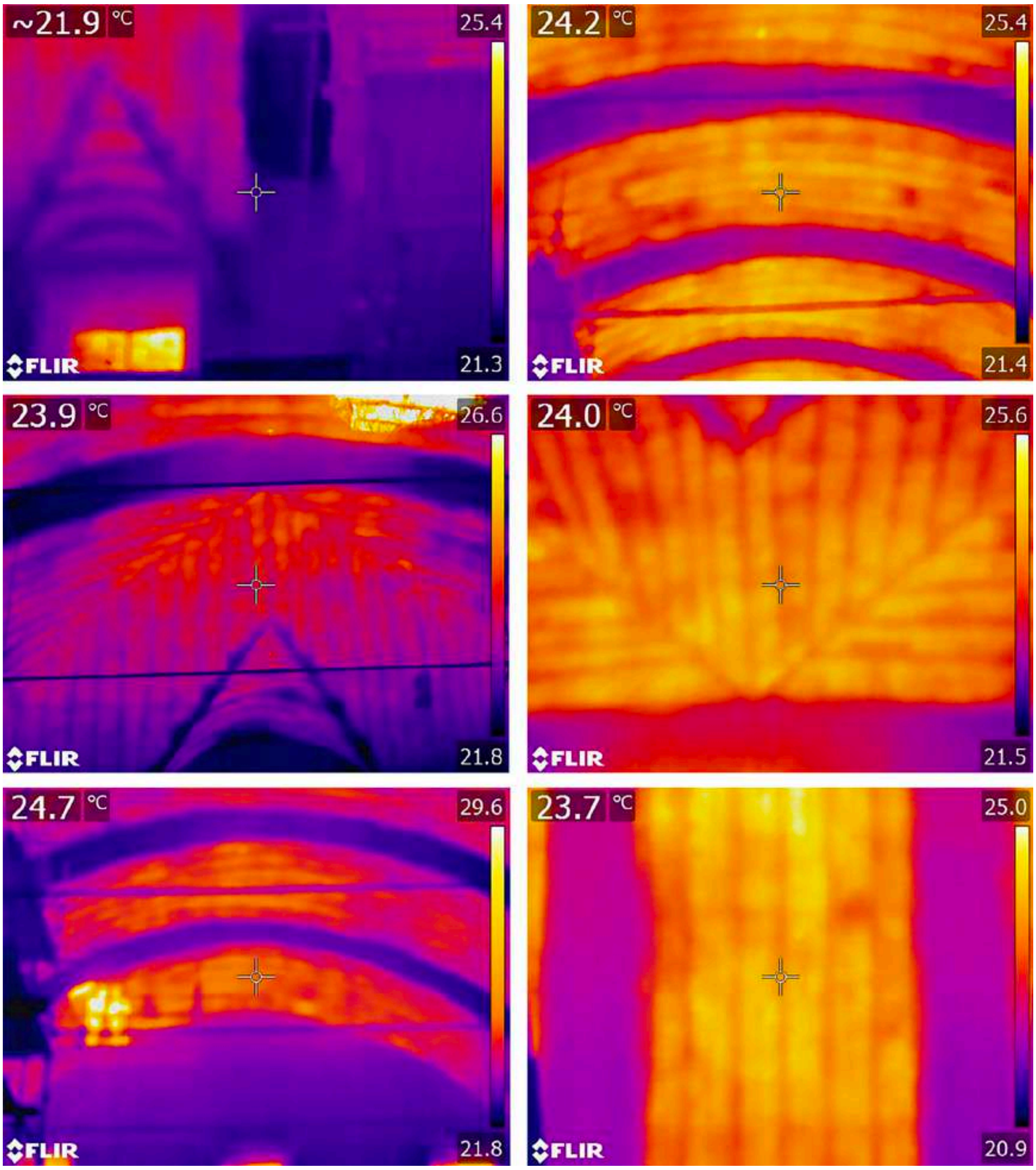

Fig. 5. *Thermographic images of the reed mat vault of the Men's Oratory. The masonry arches, the arrangement of the ribs of the barrel vault, the lunettes, and the pavilion terminal are visible.*

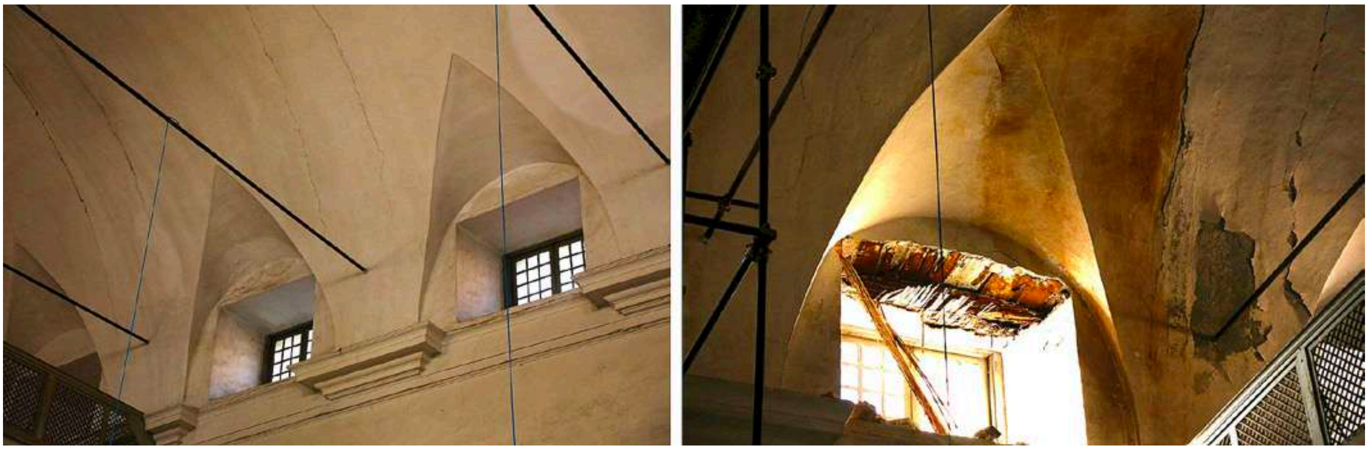

Fig. 6. *Longitudinal cracks in correspondence of the arches and partial lack of the wattle of the ceiling above a window on the south side.*

extraction of the main sections from the relative cloud of points also made it possible to verify the deformations of the vault with the probable partial detachment of the reed mat from the ribs above, in correspondence with the large transversal arches in solid brickwork.

Following the erection of two temporary tower scaffolds, three limited destructive tests were carried out on the southern part of the vault. The tests, complementary to the thermographic surveys and the survey, involved the destruction of small portions of plaster and reed mat to allow the vision of the roof structure and of the vault itself, necessary for the subsequent definition of the consolidation interventions.

In fact, the vault shows numerous cracks near the tie-rods, arranged transversely, parallel to the short side of the Oratory, in correspondence with the masonry arches (Fig. 6). The most cracked portions are, in particular, those along the pavilion head of the vault on the east side, where the central lunette meets the vault. Here, there are also two important bulges on the sides of the lunette, and the plaster is in the process of becoming detached, and on the lower right-hand side near the impost of the pavilion head, where there is also a deficiency affecting not only the layer of plaster but also the underlying supporting reed mat.

The second collapse of a portion of the vault (plaster and reed mat) occurred near the south wall, at the second southeast window, due to rainwater infiltration from the roof (Figs. 7 and 8). In the southeastern corner of the vault, there are drips and stains from infiltration. At the points of rainwater infiltration, especially along the south side of the vault, there are dirt drips and yellowish stains due to the release of tannin from the wooden ribs of the barrel vault. Along the north side of the vault, there are other whitish stains. Between the first two lunettes on the south side, the plaster of the vault has gaps and flaking parts; the paint layers are also subject to exfoliation at this point. In three small ceilings of the high windows on the south side, there are gaps in the plaster and rotting reed mat (Fig. 6). The plaster on the ceiling of the south door, which connects the gallery of the Oratory to the upper west corridor, is cracked, peeling, eroded, and peeling, and there is a gap that reveals the slate slabs underneath.

In addition, the vault observation through the collapsed portions and the assays revealed the presence of inconsistent material at the extrados, the inadequate connection between the ribs and the roof, and the lack of connection between the ribs and the reed mat.

Fig. 7. *Detail of the vault with the collapsed portion of wattle and plaster.*

Fig. 8. *Detailed photo of a portion of plaster of the timber structure vault.*

Only limited parts of the roof structure could be inspected due to the difficulty of access (Fig. 9). The main phenomena of degradation are:

- wall disarticulation in correspondence with the connection of the wooden struts to the masonry;
- cracking due to shrinkage of the wooden struts;
- deterioration of the wooden elements (related to the level of humidity and biotic attacks);
- bending damage to some purlins;
- bending damage to several arches;
- loss of shape of the vault (buckling) in the areas between the masonry arches.

Fig. 9. *Cracks and rotting of the rafters.*

Fig. 10. *The collapsed area of the vault and masonry wattle and disjoining in the support area of the struts.*

Fig. 11. *Degradation of the ribs and presence of loose material on the vault.*

6. THE RESTORATION AND CONSOLIDATION PROJECT

The state of decay, the collapse of a portion of the reed mat vault, and the results of the structural analyses carried out have determined the need to prepare, first of all, a structural consolidation intervention capable of guaranteeing adequate safety standards in relation to the future use of the former Oratory. The conservation intervention proposed aims to combine the permanence of the material and forms of the monument-document, the effectiveness and technical efficiency of the new works in terms of durability, stability, usability, and safety of the artifact, as well as the fulfilment of basic requirements for the use of the environment in relation to the new use.

a. Intervention on the roof

The investigations carried out and described above have highlighted some deficiencies and vulnerabilities of the roofing system. Many wooden elements are also damaged due to the deterioration of the roofing, which has led to widespread rainwater infiltration into the attic. The masonry pillars resting on the arches supporting the roof beams are also particularly slender, and it was, therefore, considered necessary to stiffen the whole system in the transverse direction. The wooden strut supports of the pitch along the perimeter masonry are, in some cases, deficient, and this has led to their partial disconnection and the disarticulation of the masonry.

As for the perimeter masonry, the masonry will be split and stitched in the areas where the struts are supported, where the masonry has become disjointed, creating a lack of support. The masonry pillars above the arches will be stiffened by the insertion of steel frames made of cold-galvanized UPN120 profiles (S275 class steel) and anchored to the pillars using threaded through-bars (Fig. 12). The construction of the reticular system is necessary to inhibit any instability of the pillars and to avoid the loss of support of purlins on the pillars themselves, especially in the case of horizontal actions.

The purlins showing shrinkage cracks will be consolidated by inserting a hooping with metal straps in order to limit the degradation phenomenon in progress and ensure their structural functionality. The hooping will be carried out by adopting a system of 5 mm metal cables connected through a turnbuckle system consisting of two bolted hollow metal rods. To avoid damage to the wooden elements due to the stripping of the metal straps, they will be inserted in transparent PVC tubes. The maximum spacing between the single straps will be 50 cm to guarantee homogeneous reinforcement of the elements.

In addition, a flat reticular ring beam at the top will be constructed on top of the masonry, consisting of class S275 steel plates (60 mm wide and 5 mm thick) welded to form a reticulated grid. The steel plates will be embedded in a thin cast (less than 50 mm) made of hydraulic lime mortar and connected to the underlying wall employing anchor bolts made of threaded rods embedded in the masonry. The advantages of this flat metal reticulated ring beam are many: on the one hand, the negligible increase in mass and rigidity means that the seismic action at the top does not change, and on the other hand, the widespread connection of the roof with the top of the masonry (primary and secondary elements) prevents the occurrence of hammering mechanisms that can trigger overturning mechanisms in the attic strip. All wooden elements will be treated against woodborers and fungi. The existing planking, previously removed, will be replaced by a double planking with crossed metal strips, recovering the existing planks in a good state of preservation. The new package will consist of a double-crossed planking with a thickness of 30 mm + 20 mm screwed mutually and to the joists below. Pre-drilled steel strips of a limited thickness (2 mm) will be placed between the two layers of boards to create a regular criss-cross mesh with a spacing of about 1 m. This intervention will make it possible to connect the various elements, avoiding mutual disarticulation, without, however, determining critical points at an overall level or modifying the overall

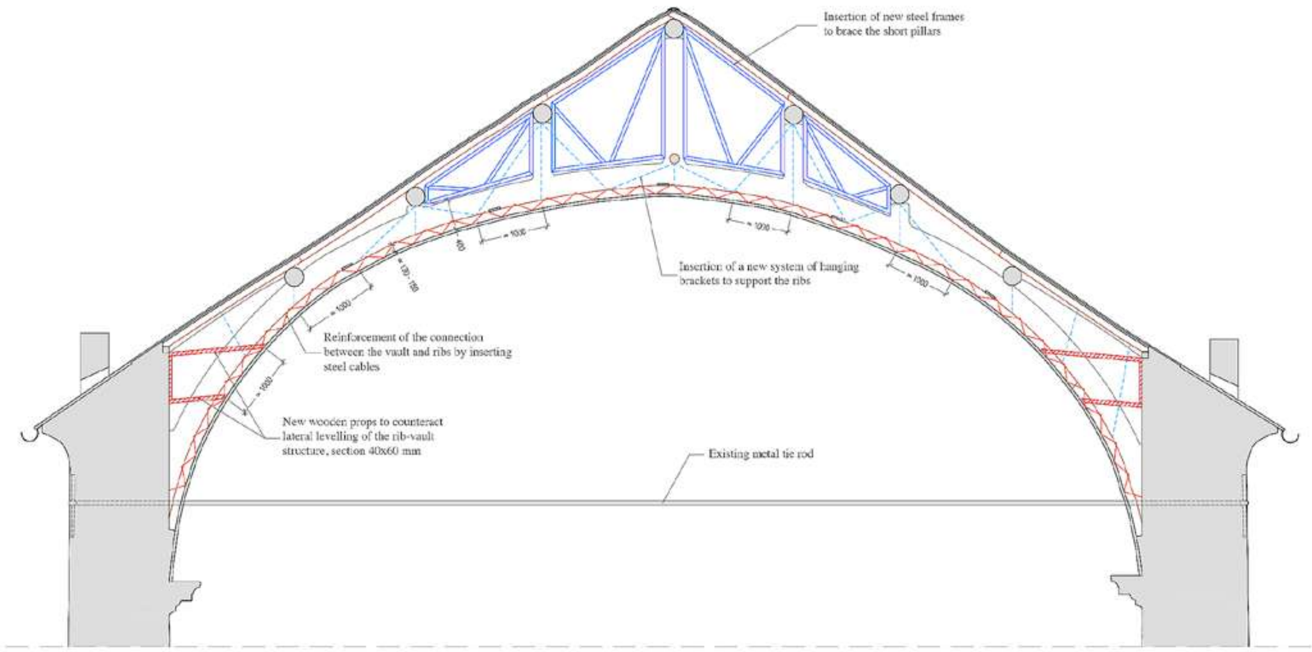

Fig. 12. *Cross-section in correspondence of an arch, consolidation project.*

response. The triple-layer slate roofing will be built with a waterproof sheath over the double planking to prevent infiltration, which in the past has undoubtedly contributed to the current state of deterioration. Openings will be provided in the side walls to allow ventilation of the attic and prevent the accumulation of moisture and condensation.

b. Interventions on the timber-structured vault

The shoring of the vault will precede the work on the vault itself through the creation of continuous ribs (rays of props) with deformable material (such as 10 mm thick polystyrene cakes) placed between the props and the vault since this operation is extremely delicate so as not to compromise the subsequent phases.

The evaluations carried out and described above have made it possible to define the following intervention for the conservation of the timber-structured vault:

- cleaning up the extrados of the vault from the materials currently present. This work will have to be carried out on the entire surface of the vault, allowing, once this preliminary phase has been completed, further verification of the diagnosis made, which has been limited to the accessible areas. Cleaning will be carried out by forced suction of the loose and powdery material deposited. The elimination of loose material on the vault is, in fact, a fundamental operation to permit subsequent consolidation work;
- consolidation of the ribs. The very advanced state of deterioration of many of the wooden elements making up the ribs led to the decision to place new 20 mm wooden boards alongside the damaged existing ribs, where necessary, on both sides, connected by 4 mm diameter screws;
- reconstruction of the collapsed portion of the vault. Reconstruction will be carried out by laying a stainless steel wire mesh plaster holder connected to the wooden ribs, on which a fiber-reinforced hydraulic lime mortar will be applied to reconstruct the portion of the collapsed vault;
- supplementary extrados casting. An extra casting of 10 mm or less will be made on the entire extrados surface of the reed mat of the vault (it is essential to limit the thickness to avoid overloading it). This intervention aims to integrate the irregularities and adequately encompass the portions of the reed mat currently exposed. This intervention will be carried out by infiltration from the extra-

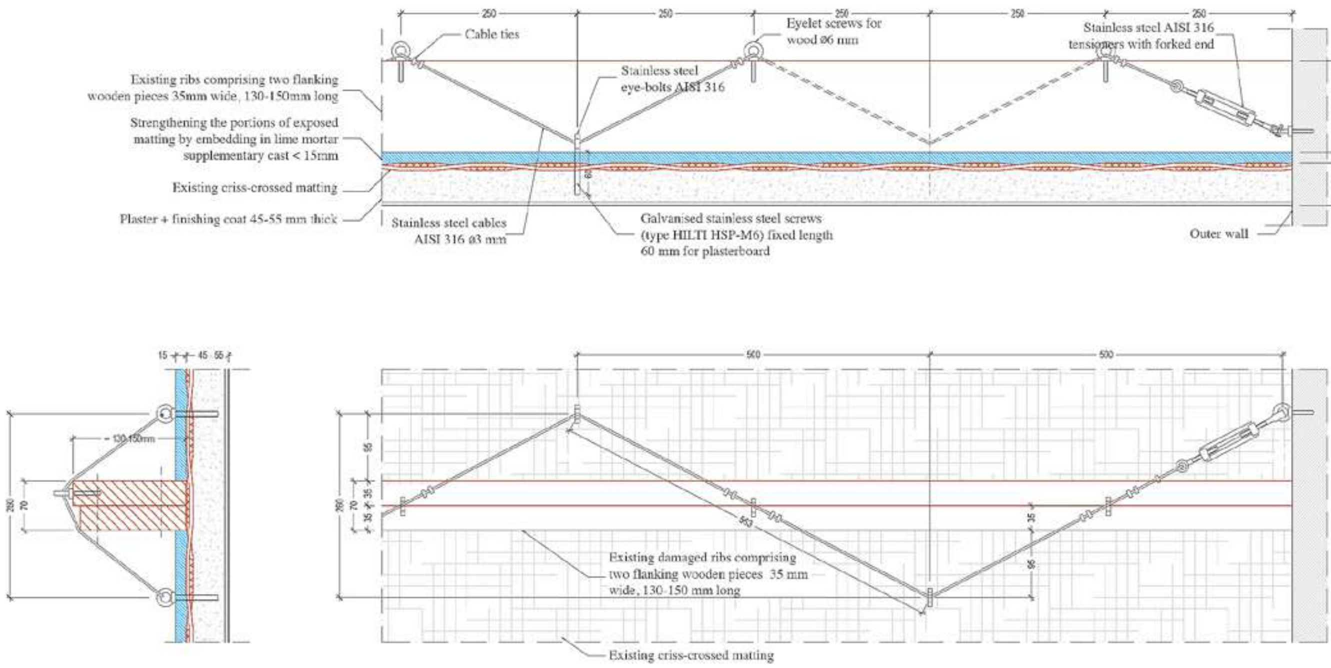

Fig. 13. *Detail of the connection between the vault and the ribs.*

dos in order to recover, as far as possible, the adherence between the underlying plaster and the structure. A cement-free hydraulic lime-based mortar (eco-pozzolan) will be used, with natural siliceous aggregates having a maximum diameter of 2 mm and a compressive strength greater than 15 MPa;

- restoration of the connection between the reed mat and ribs. The connection will be re-established by passing a 3 mm AISI 316 stainless steel stranded cable above the ribs, which will be passed through eyebolts that will be the ends of dowels inserted into the small casting made on top of the reed mat. The cables, continuous along the entire length of the vault, will be anchored to the perimeter walls of the building and tensioned through a tensioner. A tensile spring will be placed between the tensioner and the masonry to keep the cable taut, even in relation to any movements of the vault itself (Fig. 13);
- insertion of a new system of hangers. New hangers will be inserted with galvanized steel ropes (4 mm in diameter), connected to the top of the ribs by inserting wood screws with an upper eyelet (length of the threaded part between 5 cm and 7 cm). A metal element capable of elastic deformation (elastic spring with an elastic constant of 10 to 20 kN/m) is inserted between the tie rod attachment system and the tie rod itself. This new element connected to the turnbuckle will make it possible to dose the tension of the tie rod more appropriately, avoiding concentrations of localized stresses and allowing the vault to deform naturally according to the changes induced by variations in the boundary conditions (variations in humidity, temperature, etc.). Where necessary, it is possible to replace the existing tie rods. This operation must be carried out with the utmost caution, eliminating the existing element only after the new tie rod has been positioned and tensioned so as not to create undesirable deformation during the transitional phase. The presence of the tensile spring with a closed loop, placed between the steel cable and the tensioner, will ensure a more gradual tensioning of the steel cable itself, limiting the possibility of creating states of stress concentrated on the vault and allowing its natural adaptation to climatic conditions. The layout and quantity of hangers will be assessed on-site;
- treatment of wooden elements against woodborers and fungi. Cleaning and protective surface treatment of all exposed wooden elements. If insects are found inside the material, it will be necessary to proceed with punctual disinfestation;
- insertion of reinforcing elements in the areas where the vaults are deformed. At these points, two 40 mm x 60 mm section wooden elements will be inserted, anchored to the perimeter masonry by means of chemical anchors, and screwed to the ribs in such a way as to maintain the shape of the vault and avoid further deformation phenomena (Fig. 12);
- grouting the cracks in the soffit of the vault with mortar made of slaked lime and river sand that is compatible in color, size, and shape with the existing mortar and after stripping the cracks;
- surface consolidation of the plaster on the intrados with nanoclay suspension applied by brush until it is rejected after cleaning the surfaces.

The corniced ceilings of the windows on the upper level will be restored by replacing the damaged and rotting wooden beams/centers, and the portions of the vaulting where the layer of plaster and cornice is missing will be reconstructed by laying a stainless steel mesh plaster holder connected to the wooden ribs, on which hydraulic lime mortar will be applied.

7. CONCLUSIONS

The roof structure of the Men's Oratory is the largest preserved in the Albergo dei Poveri complex and one of the largest in the city of Genoa.

The seventeenth-century mixed structure with transversal masonry arches and a wooden frame made up of longitudinal beams and purlins with a wooden vaulted structure has never been replaced, except for some localized consolidation work.

The difficulties in accessing the extrados of the vault to inspect the construction characteristics and the state of conservation of both the roof structure and the wooden structure vault led to the choice of non-invasive diagnostic techniques (laser scanning and thermography) associated with the execution of three localized tests for the visual inspection of the attic. It will be necessary to carry out an in-depth diagnostic campaign during the construction phase in order to increase the level of knowledge of the structure of the vault, the roof, and above all, the level of deterioration of the individual elements before proceeding with the executive design and implementation of the works.

The project described makes it possible to maintain and restore the architectural elements without altering their structural function by repairing the gaps and improving the structural behavior by eliminating those sources of vulnerability that undermine the structural safety of the building.

Authors contribution

Section 1, 7: M. Casanova, S.F. Musso and S. Podestà; Section 2: S.F. Musso; Section 3, 4: M. Casanova; Section 5, 6: S. Podestà.

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REINFORCEMENT METHODOLOGIES OF TIMBER ELEMENTS IN HISTORIC TIMBER ROOFS

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Abstract

This paper aims to present a comprehensive review of the reinforcement of historic timber roofs, focusing on their main characteristics, advantages, and disadvantages, which would help professionals select and define the design of reinforcement solutions. Cultural heritage issues are taken into consideration. Reinforcement can be done via different methods – traditional and modern – using simple or sophisticated techniques. An overview of the main materials and the techniques used for selected case studies are presented, illustrating how various reinforcement methods are implemented in practice.

Keywords

Historic timber roofs, Architectural Heritage, Reinforcement, Techniques.

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1. INTRODUCTION

Historic timber roofs constitute an important part of the cultural heritage of many countries of the world. The increased sensitivity towards the preservation of cultural heritage has led to the adoption of restoration techniques that comply with generally accepted conservation principles and guarantee, as much as possible, the preservation of authenticity and integrity of the structure, minimal interventions, reversibility, and compatibility with the original parts of the timber [1, 2]. Economic, environmental, historical, and social reasons dictate the aim and scope of intervening in historic structures.

Repair and/or reinforcement is preferred to total structural replacement since the authentic materials and structural systems, the construction technology, and workmanship constitute a significant cultural value of historic buildings. They need to be protected and preserved, even if they may no longer be visible, hidden by plasters or ceilings, after the restoration [1, 3]. This growing sensibility towards the preservation and maintenance of heritage buildings, the various species of wood and the complexity of their structural behavior, their degradation caused by different agents, and the need for rehabilitation to incorporate new uses has led researchers to study various repair and reinforcement solutions.

In most cases worldwide, traditional buildings involve timber used at least as floors and roof systems. The technical and technological development of roof carpentry was very different from country to country. Many different types of roof structural systems and joints exist, which bears testimony to the diversity and richness of the timber cultural heritage and, at the same time, to the difficulty of studying, repairing, and reinforcing them.

Reinforcement or strengthening deals with interventions that increase and upgrade the original or existing load-bearing capacity of the structures, while maintenance and repair try to recover the original load-bearing capacity and return the existing fabric to a known earlier state. Reinforcement is usually applied to extend the use of structures approaching the end of their design life and to ensure that recent requirements for a new use of a building ("heavier" loads or level of safety) and changes in regulations are fulfilled.

Each reinforcement solution has advantages and disadvantages regarding conservation philosophy, architecture, aesthetics, structural performance, and technological and construction quality. Economic issues such as the cost of the intervention and the availability of specialized staff can also determine the choice of the method used [4]. When the reinforcement of a roof is being designed, all of the above has to be taken into account and evaluated carefully to ensure that the proper intervention is chosen. A careful choice of strengthening materials and the reinforcing method is necessary. No material or method can be considered the optimal one. Each case is unique.

This paper will focus on reinforcing historic timber roofs but not reinforcing existing ones, which constitutes a much wider group [5]. It also focuses only on reinforcement methods for the main load-bearing timber elements, not considering the secondary ones (purlins, decking), non-structural parts such as roofing or ceiling materials (clay tiles, timber shingles, ceiling planks, etc.), and reinforcements of connections, which constitute a vast area of research and case studies. Some examples that improve the overall behavior of the roof are included since the decision to use them may indirectly be beneficial for the load-bearing capacity of the timber members of the roof, diminishing, for example, the loads that will carry.

In several cases, roofs have important decorative (woodcarving and polychrome) details, markings, symbols, and finishes, and very often, they carry simple ceilings or ceilings with very high artistic value. In many of these cases, it is required to operate on the spot, without dismantling the carpentry or any of its parts, increasing the difficulty of both the assessment and the restoration procedure.

2. CAUSES OF DAMAGE TO TIMBER ROOFS

The first step before any intervention (repair or reinforcement) is the documentation and the assessment of the existing timber structure: the understanding of the structural system, the damage and the causes, and the residual strength and stiffness properties. Briefly, the most common and major problems of timber roofs are: i) decay problems, usually in parts in which water enters and accumulates, such as in support areas (timber parts embedded in the external walls); ii) insect attack (active or not); iii) damage or lack of strength and/or stiffness of single members (failures, shrinkage cracks, excessive deformations, etc.); iv) damage or lack of strength and/or stiffness of joints (failures, shrinkage cracks, etc.); v) lack of stiffness of the whole timber roof (in-plane or out-of-plane, vertical or horizontal deformations).

Damage and failures of timber roofs can be due to different causes: i) natural defects of wood; ii) biological degradation (rot, insect attack); iii) environmental and atmospheric agents (changes in wood moisture content); iv) fire; v) errors in the original conception/poor original design (lack of adequate sections, poor quality of timber, errors in the original structural system); vi) poor execution; vii) excessive loading (wind, earthquake, etc.); and, viii) maintenance or intervention errors during their lifetime.

Proper assessment with appropriate techniques is obviously of major importance. Therefore, the study of relevant state-of-the-art reports, scientific work, and publications concerned with diagnostic procedures is highly recommended [5–7].

3. REINFORCEMENT METHODS

In the next sections, different examples of reinforcement methods will be presented according to the following categories: i) reinforcement of timber roof members (rafters, tie-beams, posts, end-beams, etc.); ii) reinforcement of the overall load-bearing system of the roof (improvement of the overall stability, e.g., bracing).

3.1. REINFORCEMENT OF TIMBER ROOF MEMBERS

In order to increase the flexural strength and stiffness of timber members (beams), reinforcing elements are usually added to supplement the existing elements. A large variety of reinforcement configurations are available. The reinforcing elements can be in the form of rods, plates, straps, or other structural shapes, which are connected to the beam using mechanical fasteners or structural adhesives. These reinforcing elements can be placed inside or outside the member and may be passive or pre-stressed. Apart from the structural requirements, the strengthening configuration selected for a particular application may depend on other factors, too: aesthetics can limit the use of different materials; the presence of decorative ceilings or painting on beams may require that the reinforcement be restricted to the top or the sides of the timber elements; fire protection, aesthetical issues, and other requirements may exclude the use of externally bonded plates on exposed surfaces; geometrical, architectural or constructional limitations can restrict the use of new elements or elements with certain dimensions; etc. [8].

3.1.1. PRE-STRESSING METAL REINFORCEMENT

Pre-stressing has emerged as one of the most common reinforcement techniques for increasing the bending load-carrying capacity and the stiffness of timber members when large deflections are observed. It is mainly used for rafters and tie-beams and may be required due to an inadequate section or low strength and stiffness properties. An essential advantage of this reinforcement type is its reversibility.

In publications since the second half of the 19th century, it is emphasized how the empirical work of many engineers has created a broad selection of layouts and structural solutions that have worked properly for many decades (Figs. 1 and 2). Technical manuals refer to this kind of reinforcement, mainly pointing out the difficulties of installing the outer tendons at the head of the beams due to the fact that the beams are embedded into the walls [9–11]. Of course, there have been improvements in these techniques since then, boosting confidence in the use of this kind of reinforcement.

It was in the 2nd half of the 20th century that a systematic approach to post-tensioning restoration methods of timber structures was established. Some examples were reported in manuals written during that period and are still a reference for present professionals. Most of the leading restorers belong to the Italian school, which is known as a very active center for restoration theories and projects (Fig. 2) [12].

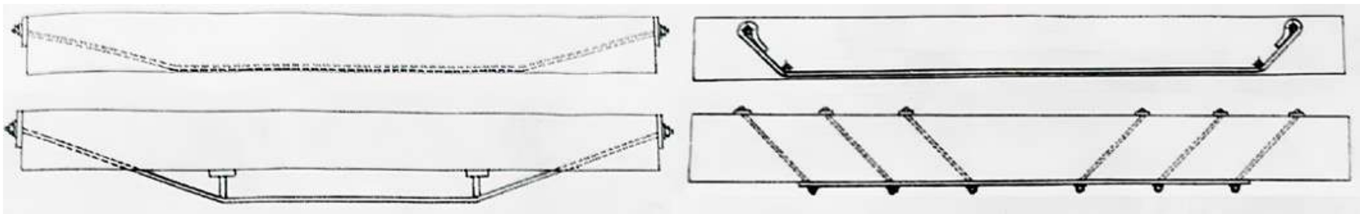

Fig. 1. *Different pre-stressing techniques for strengthening existing timber beams [9–12].*

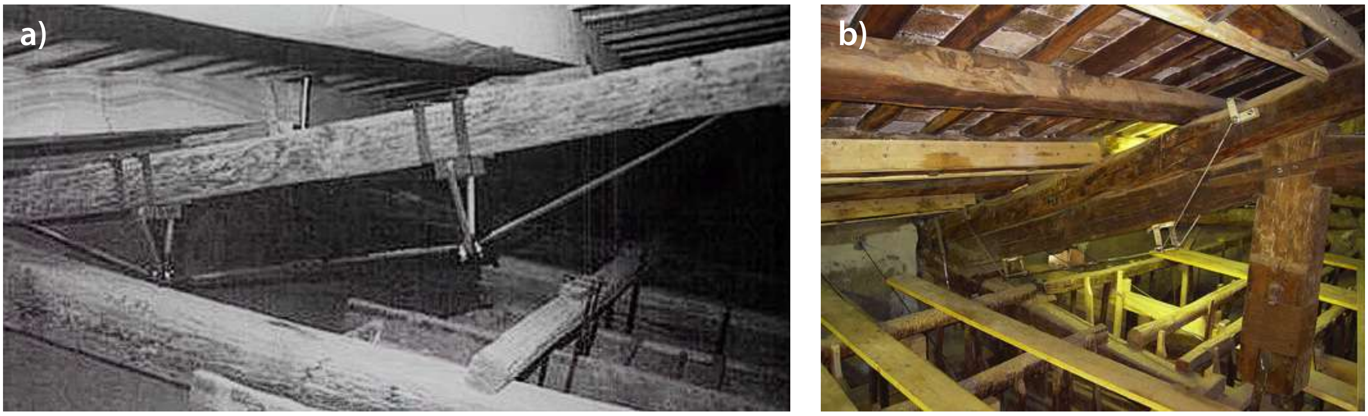

Fig. 2. *Reinforcement of a truss rafter in the theater of Sarteano (a) and a Renaissance palace in Rome by G. Tampone (b) [12].*

Nowadays, for the reinforcing system, high-strength materials with minimum dimensions, such as steel (stainless or not) or titanium, can be used. This reinforcement system can be applied to sound, unbroken beams of very regular shape or to members that have already been repaired [9]. One major issue that needs special attention is how the rheological behavior of the material and the shrinkage of the timber sections may affect the loss of pre-stressing, especially in cases where the application of the load is transferred by perpendicular compression or at an angle to the grain. Tension perpendicular to the grain should be avoided. Periodic inspections are necessary for the pre-stressing methods to ensure the intervention's efficiency according to the design specifications.

3.1.2. NON-PRE-STRESSED METAL REINFORCEMENT

A similar concept to the above, but without pre-stressing, can be used to reinforce tie-beams that present excessive deformation. Steel elements may support the transfer of loads by the tie-beam to other elements (members or joints), which are carefully chosen and verified structurally (Fig. 3) to ensure the safe transfer of the forces through the new load path.

The effect of the environment and the material's initial and existing conditions, even the conditions after the restoration works, must be taken into account, too [9].

3.1.3. CONNECTION OF NEW ELEMENTS (TIMBER-BASED OR STEEL) TO THE EXISTING TIMBER MEMBERS BY STEEL FASTENERS

This is a common technique of reinforcement used to increase the load-carrying capacity of a timber element (e.g., a rafter or a tie-beam) or if the deflection of the beams is too high. Steel sections and plates [10], solid timber sections, or wood-based products (glued-laminated timber, plywood, cross-laminated timber, laminated veneer lumber, etc.), nailed, screwed, or bolted either to the tensile face or the vertical sides of the timber beams, are used to repair or reinforce timber elements (Figs. 4–6). Similar techniques are used for the substitution of the decayed parts of timber members. For a systematic review of the repair and reinforcement of historic timber structures with stainless steel, see Corradi et al. (2019) [10].

As timber beams generally fail in tension in a brittle way, positioning the reinforcement at the tensile face of the beams is very effective for increasing bending strength. The above interventions, in most cases, are not applicable to exposed timber structures. Besides its poor aesthetic appeal, reinforcement with external metal plates may suffer other disadvantages due to condensation on the timber members and their consequent vulnerability to decay. Dimensional changes in the steel parts are caused by changes in temperature and, therefore, additional internal stresses in the wood and fasteners and dimensional changes of the timber that the steel parts

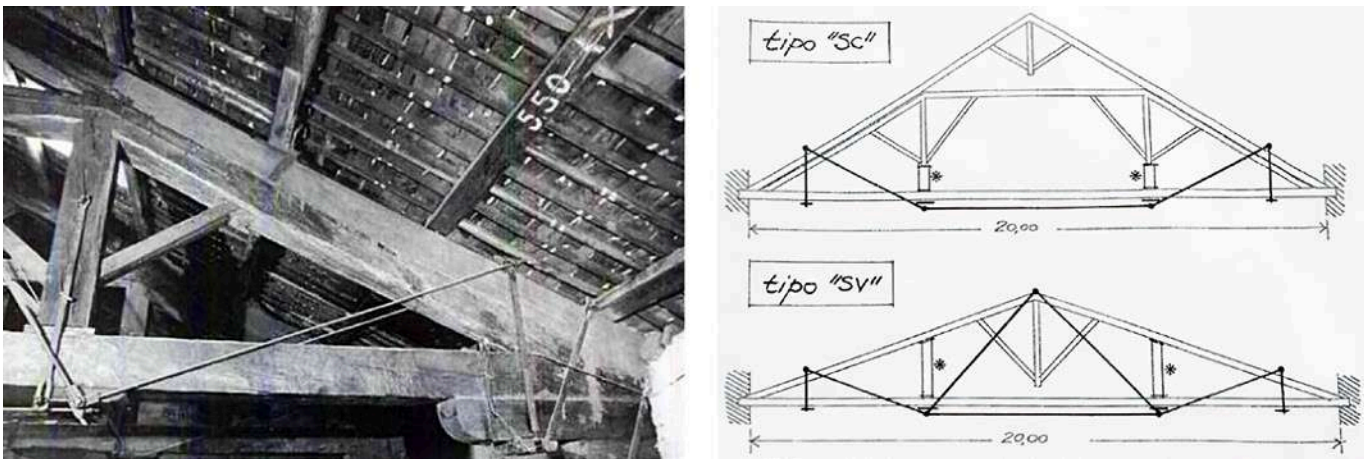

Fig. 3. *Reinforcement of a truss tie-beam. Detail and design schemes of the Savona Theatre project. (Image source: courtesy of the designer, ing. L. Paolini).*

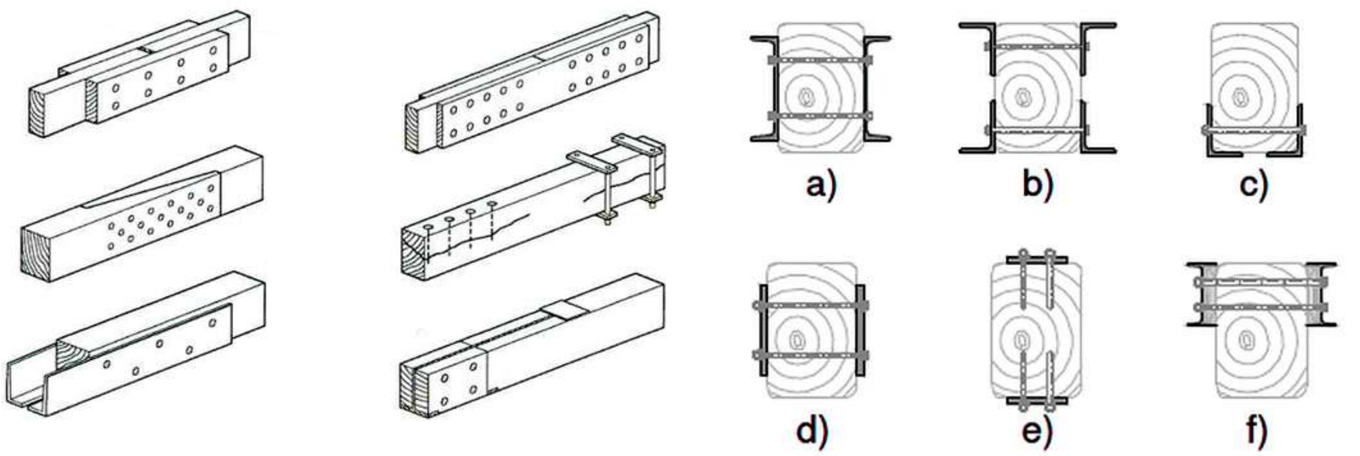

Fig. 4. *Repair or reinforcement of timber members using new timber elements or steel plates and sections connected to the original timbers by metal fasteners [13, 4].*

Fig. 5. *Repair of a timber rafter using large pieces of wood screwed at the sides of the rafter [14].*

cannot adapt to if humidity changes occur. Galvanized steel or protected from corrosion sections, plates, or fasteners would be preferable. On the other hand, when adding steel elements, due to the different modulus of elasticity, it is crucial to take into account that stiffness incompatibility can arise.

Internal elements such as steel plates inserted in the timber members can be used too. They can be connected to timber beams with screws, steel dowels, or bolts (Fig. 6).

The system can be applied to undersized, overloaded, or broken timber members. It requires geometric regularity of the section of the timber member. For a well-sized but broken element, the length of the inserted plate can be limited to the length of the affected section plus an additional length (one and a half the depth of the member), including sound wood at both sides [12]. Strengthening of timbers with steel flitch plates and resin working as a composite member is a method used in several restoration projects in England (Fig. 6d). However, again, it is important to assume that incompatibility issues can arise due to the different stiffness and dimensional stability presented when distinct materials – steel, resin, etc. – are used together in a composite section.

Fig. 6. *Strengthening of a timber member by insertion of steel plates. Invention patent G. Tampone, L. Campa, 1987 (a) [12]. Barn (14th century) in Herefordshire, UK (Sinclair Johnston, 2009). Truss repair using a 20-mm thick steel flitch plate and resin. If resins were not used, many bolts would have been required, further cutting away the fabric and having less aesthetic appeal (b-d) [15].*

3.1.4. ADDITION OF NEW TIMBERS (STRUTS OR POSTS) TO INCREASE THE SUPPORT OF THE EXISTING TIMBER MEMBERS

This is a typical reinforcement method for “post and beam” roofs, the most common type of roofing for buildings in the Balkan and Minor Asia areas during the Byzantine and Ottoman periods [16]. It is a spatial system that functions in a completely different way from the well-known types of king or queen post trusses widely used in Italy and other European countries. The loads are transferred from the rafters through a three-dimensional system of beams, posts, and struts not only to the outer walls but mainly to the internal ones (Fig. 7).

In the “post and beam” system, the connections of the posts and struts can transfer compression forces but not tension. Typical damage of such roofs is the bending failure or excessive deflection of the longitudinal horizontal beams that support the rafters (Fig. 13a) due to the absence of adequate struts or lack of appropriate sections. The reinforcement of the original load-bearing system can be accomplished by the addition of new struts (denser supporting) (Fig. 7c), which is an easy, low-cost and reversible intervention that gives the possibility to keep the original beam in position and retain the concept of the original structural system [15, 17].

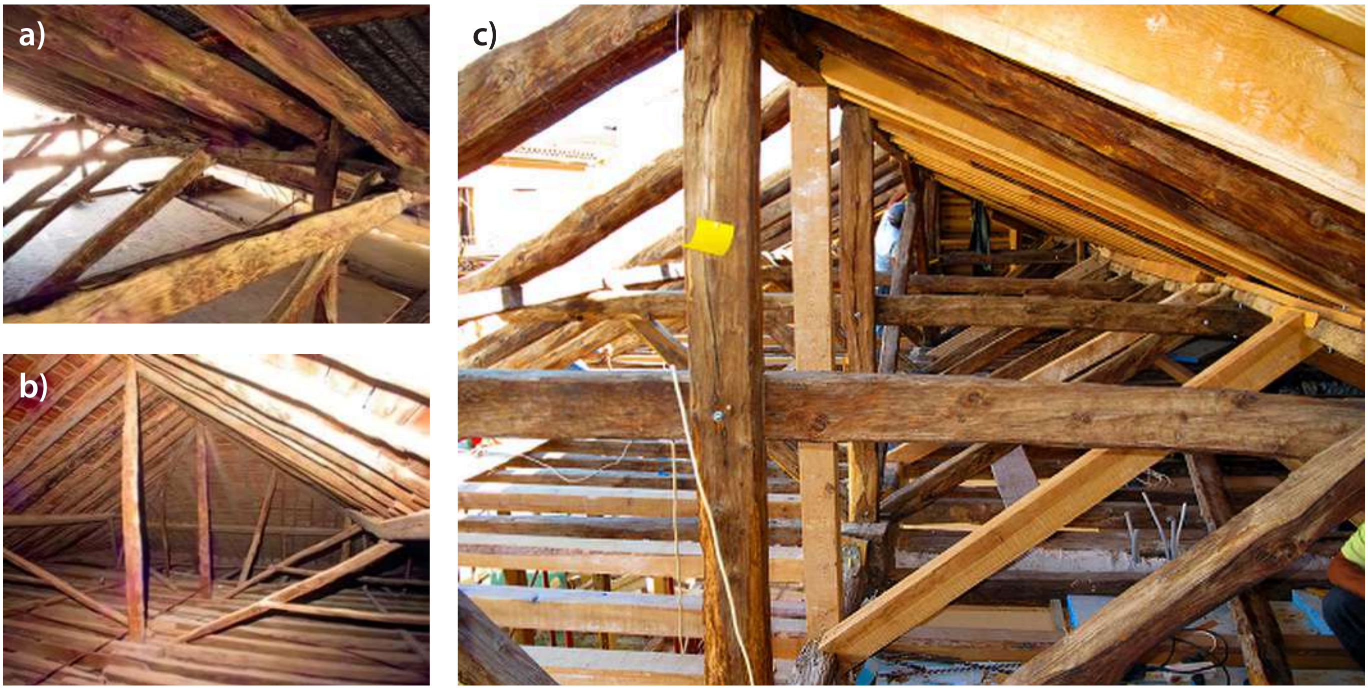

Fig. 7. *“Post and beam” roofs of the Ottoman period mansions in Greece. Excessive deformation of the longitudinal horizontal beams that support the rafters (a, b) [17]. Reinforcement by adding new timber posts and struts to increase the supports of the longitudinal beams that carry the rafters (c) [18].*

3.1.5. ADDITION OF NEW TIMBER OR STEEL MEMBERS NEXT TO THE ORIGINAL TIMBER MEMBERS OF THE ROOF

In some cases, new timber or steel elements can be placed parallel to the original ones without any connection to diminish the loads the existing elements carry (Fig. 8). The concept of this reinforcement is similar to the one described in section 3.2 regarding the whole roof structure.

3.1.6. REPLACEMENT OF THE WHOLE TIMBER MEMBER

There are cases (poor initial construction, severe damage or failure, etc.) where the replacement of the whole timber member (rafter, tie-beam, post, strut, etc.) is the only solution. But this should be resorted to only as a last alternative in any restoration project (Fig. 9).

3.1.7. BEAM-END REINFORCEMENT (PROSTHESIS)

Intervention in roofs very often involves end-beam repairs. End-beams embedded in masonry walls are the most exposed zones to biological agents. If a high level of moisture is present in the masonry, for example, due to infiltration when damage occurs in the roofing elements (e.g., tiles), and the adsorbed moisture cannot evaporate because of poor ventilation, the suitable conditions for biotic attacks (moisture contents above

Fig. 8. *New timber or steel elements placed between the original beams to carry part of the loads (a) [19]. Queen's Tower Estate Ilion, Athens: New steel tie-rod added next to the original timber tie-beam without any connection to transfer the tensile loads (b).*

Fig. 9. *New timber beams used to replace only the severely damaged rafters (a, b) [18]. The Neoclassical School in the Medieval city of Rhodes: New timber glued laminated beam replaced the poorly constructed original horizontal beam (c).*

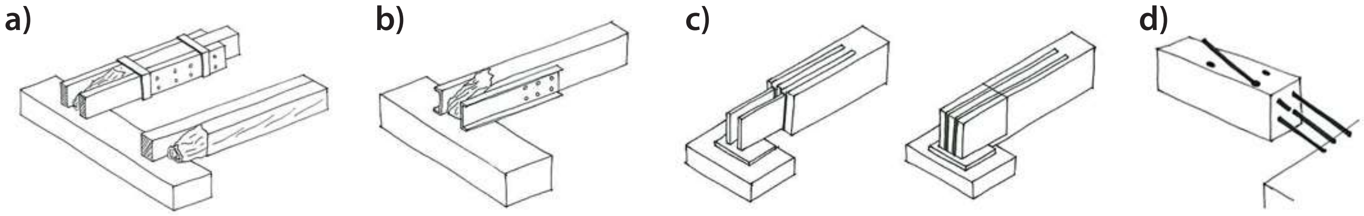

Fig. 10. *Examples of end-beam repair techniques (prosthesis) using different materials and methods [9]: (a) new timber elements connected to the sound part of the original timber with steel straps, plates, and steel fasteners; (b) new steel elements used to replace the decayed timber; (c) glued-in plates connecting the prosthesis to the sound part of the original timber; (d) glued-in rods connecting the prosthesis made either of wood or resin.*

20%) are established, and degradation of end-beams can, therefore, be expected. Nowadays, several techniques and methods can be found in the literature to repair and reinforce rotten timber end-beams. Since the early 1970s, many companies have developed materials and techniques to repair timber elements by partial substitution of the decayed part (design of a prosthesis). All of these techniques aim to restore the load-bearing capacity of the original member and the old part's structural continuity with the new, thereby ensuring their connection and collaboration.

The intervention consists of the substitution of the decayed part by a new element which can be made of solid wood or wood-based products (glued-laminated timber or LVL) (Fig. 10a-d), steel sections or plates (Fig. 10b), or epoxy resin (Fig. 10d). Such new elements are connected to the original timber part by steel fasteners (nails, screws, bolts, dowels, metal straps, etc.), by threaded or ribbed steel glued-in rods (GiR), or by FRP plates, woven fabrics and rods (from glass, carbon, aramid, basalt, etc.). The elements used to substitute the decayed timber may be visible or not, and the elements that connect them to the sound part of the timber can be either external or internal [9].

The use of prostheses became widely accepted mainly due to their low intrusion level, simplicity of design and execution, and the good aesthetic result of some of the techniques used. The type of prosthesis reinforcement method may vary, depending on many parameters (cultural values, aesthetics, presence of decorative elements, access to the damaged timber part, fire protection, on-site application, available expertise, cost, etc.).

The above methods of prostheses, especially the ones that use timber, offer several advantages: high connection stiffness without significant settling; the possibility of ductile design with yielding of the steel or the other types of bars in spite of the adherence based on glue; protection of the glue and the embedded elements from chemicals and fire; unmodified exterior of the reinforced element that maintains the original architectural characteristics [20]. A major advantage of the glued-in rod connections is the transfer of forces directly into the inner part of the members' cross-section [17]. On the other hand, several considerations have been raised for some of the methods used. The use of timber prostheses compared to resin or steel is considered to follow more closely the conservation principles of historic timber structures [1]. For the use of external steel elements (sections or plates) and the problems that may arise, see section 3.1.3 (Fig. 11). The reinforcement design for members or joints should consider the effect the reinforcement can have on the original structural system, which needs to be preserved, except for cases that present important errors in the original conception (Fig. 11). Changes in beam or joint stiffness (resin prosthesis or some cases of steel plates) may have consequences on the overall behavior and load distribution of the entire structure, altering the paths of loads and leading to damage or failures of the weaker elements.

In cases where steel is used, problems may arise due to environmental thermal or humidity changes in steel and timber. As the reinforcing elements generally have different stiffness, thermal expansion, and moisture absorption properties than the timber elements, factors that constrain shrinkage and swelling due to thermal or moisture changes must also be considered. If necessary, additional thermal or moisture-induced stresses should be accounted for in the design (Fig. 12) [8].

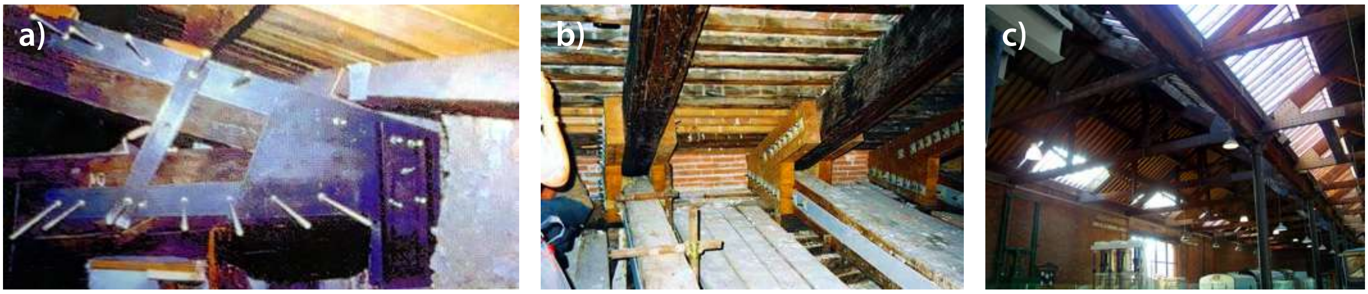

Fig. 11. *Interventions that may change the stiffness properties of the original semi-rigid joints [20].*

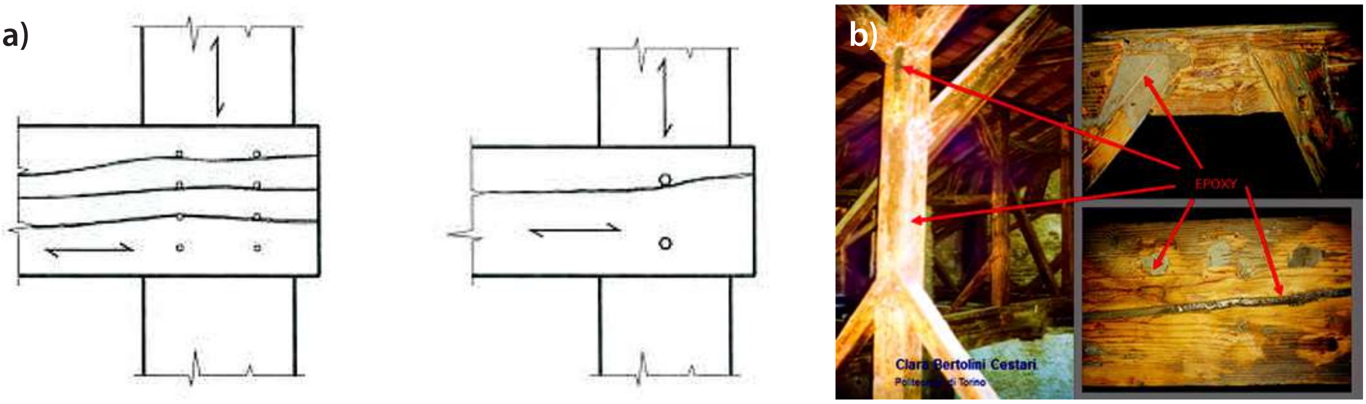

Fig. 12. *Cracks caused by differential shrinkage parallel and perpendicular to the grain in the timber-to-timber connection (a) [21]. Improper filling of the shrinkage cracks with resin (b) [22].*

Some interventions have caused further damage to structures in the past due to a lack of knowledge on selecting and implementing appropriate reinforcement methods. An example is the filling of shrinkage cracks with resins. The excessive stiffness of the adhesives and their subsequent inability to withstand the timber strains, especially strains due to hygrometric variations, can seriously impact the state of the existing cracking and even provoke new cracks (Fig. 12). The same applies to the use of bars glued to timber to stop the further widening of cracks. The use of such a technique can create undesired stress states by preventing the natural movement of the timber. Moreover, questions related to the compatibility of reinforcing and reinforced materials, the significant differences between the properties of wood, a hygroscopic organic material, and those of epoxy, an impermeable plastic, the durability, the low reversibility, the sufficient reliability needed for historic structures, the behavior of the resin under thermal and moisture fluctuations, as well as the long-term performance and fire resistance of the resin are yet to be investigated. Timber experts do not recommend epoxies for external repairs due mainly to moisture issues [23, 24].

3.2. REINFORCEMENT FOR THE OVERALL LOAD-BEARING SYSTEM OF THE ROOF

3.2.1. ADDITION OF A NEW STRUCTURAL SYSTEM IN PARALLEL TO THE ORIGINAL ONE

Rafters or tie beams (see 3.1.5), or even a whole roof truss, can be introduced between the main beams or the main trusses (placed parallel to them). An example of an added parallel system can be found on the roof of the Salone dei Cinquecento in Palazzo Vecchio in Florence (Fig. 13a) and in the main railway station in Wroclaw, Poland (Fig. 13b).

Roofing structures, in several cases, carry ceilings of great artistic value. The reinforcement may concern the increase of the load-bearing capacity and, mainly, limiting the deflection of the roofs since, in many cases, the ceilings support frescos which are brittle without any ductility. This is the case of the king post timber roof designed in 1563 by Giorgio Vasari for the Salone dei Cinquecento in Palazzo Vecchio in Florence (a project carried out 1563-1565, by Battista Botticelli), which had to carry a very

Fig. 13. *Salone dei Cinquecento in Palazzo Vecchio, Florence. General view of the two trusses for the roof at the back of the photo (the original one) and the new one at the front (a) [25]. Main Railway Station in Wroclaw: A new load-bearing structure for the ceiling (a steel space frame) was used to relieve the roof truss from the coffer ceiling loads (b) [26].*

heavy ceiling of great artistic value. The ceiling was soon affected by sagging caused by creep effects. A reinforcement work was carried out in 1854 by Arch. D. Giraldi, who constructed new timber trusses, placed at a lower level as a parallel system between the original ones without any intervention at the old ones (Fig. 13a) [25].

3.2.2. ADDITION OF NEW STRUCTURAL SYSTEMS TO IMPROVE THE OVERALL PERFORMANCE AND STABILITY OF THE ROOF

Besides the behavior of the individual structural elements (members and joints), which has an indirect but considerable impact on the overall behavior of the system, reinforcement may be needed to improve the original structural system that presents either design or execution errors, to confront a severe deformation of the whole roof or an overall stability problem that needs bracing (Fig. 14). The advantage of these solutions is that they are reversible.

Another example of this type of reinforcement was used in Angera Castle (Lago Maggiore, Italy), where all loads are transferred after the intervention to the supports by a new steel truss, although originally, the loads were divided across four supports [28].

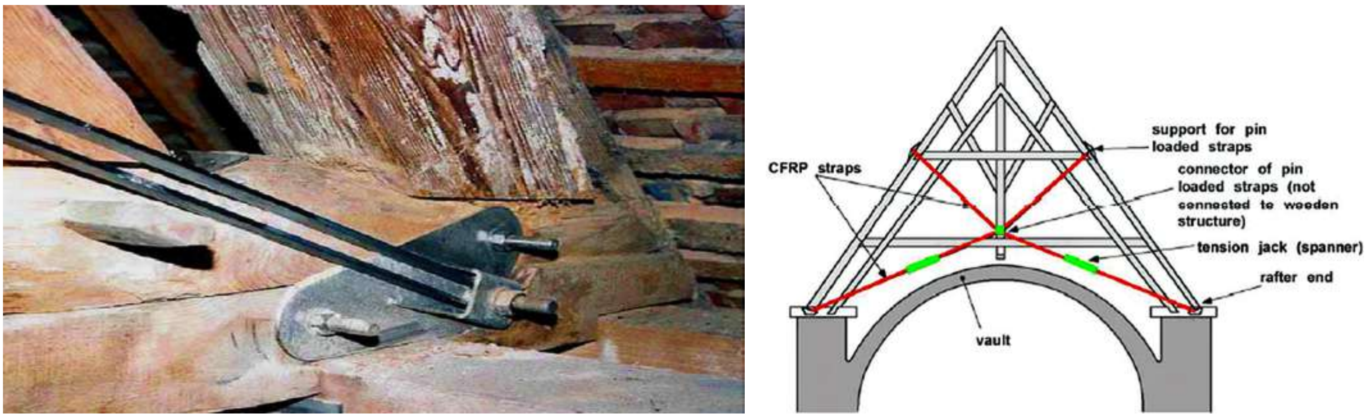

Fig. 14. *St. Marien Church. Reinforcement of the timber roof using unbounded pin-loaded CFRP straps [27].*

4. CONCLUSIONS

When new usage and/or new imposed loads are introduced, reinforcement of traditional timber structures is needed. If the decay of timber elements is huge, then local replacement of the decayed part is the best solution. When interventions are necessary, specific and reliable on-site assessment techniques are required to determine the appropriate level of intervention needed. A critical point remains the evaluation of the replacement, repair, or reinforcing solutions, along with the cultural significance, the know-how, and the associated project costs of each case. Assessment of the durability of the intervention work carried out with new innovative techniques is necessary too. The use of wood to solve problems of wood offers one of the most interesting features in conservation, compatibility.

For timber members, reinforcement helps to restore the load-bearing capacity lost because of the material decay and increases the resistance (strength and stiffness properties). Current knowledge of reinforcement methods of existing structures is largely based on practical experience. Unfortunately, the study of reinforcement techniques still needs to be included in European standards such as the Eurocode 5 (EN 1995-1-1) [29], but only for specific aspects in the National Annexes of some countries.

Investigations on that promising topic have helped experts figure out how to overcome timber weaknesses, resulting in design models and reinforcement methods proposals. Some of the most important and applicable outcomes will be integrated into the revised edition of the Eurocode 5 to help engineers design reinforcements for timber structures. It must be pointed out, though, that these reinforcements are for new timber structures. Since existing structures and, mainly, historic structures still need to be covered by the latest standards, it is urgent to develop relevant European Standards too. Thankfully, in 2017, to close a part of the gap between practical needs and missing standardization, the European Standardization Committee responsible for the Eurocode 8, CEN/TC 250/SC 8 concerning earthquake design, decided to add in chapter 3 for existing buildings a section on existing timber structures, historical or not.

Increased knowledge and research on retrofitting techniques are of great importance to support the Standards that are being or will be developed. When timber structures are reliably repaired or reinforced, structural failures and unnecessary replacements can be avoided, and sustainability, which is essential from economic, environmental, historical, and social perspectives, will be served too. Standards, research, and the constant and continuous dissemination of knowledge, mainly through education, can provide the necessary tools for structural engineers, who are often part of a multi-disciplinary team and have to work together in a restoration project to evaluate the existing condition of a historic timber structure. Moreover, these tools may help in the selection of the proper interventions using innovative and/or simple techniques that will sustain the authenticity of our architectural heritage, including the authenticity of the “invisible”, in many cases, timber load-bearing structures.

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Authors contribution

Filipa S. Serino, writing, investigation and editing; Eleftheria Tsakanika, conceptualization, methodology, writing and review, supervision; Jorge M. Branco, conceptualization, methodology, writing and review, supervision; Paulo B. Lourenço, conceptualization, supervision.

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THE MEDIEVAL CARPENTRY OF THE BASILICA OF ST. ANTHONY IN PADUA

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Abstract

Attics are often the most interesting rooms where to investigate historical construction techniques. Above the eight domes of the medieval Basilica of St. Anthony in Padua, lightweight timber structures attest to the use of rather archaic frameworks. The ongoing research on the domed roofs confirmed the preservation of their 13th-century configuration and the perpetuation of this model during later interventions until the 18th century.
Based on on-site measurements, the study of archival material, and dendro-dating, this paper aims at shedding light on the constructing techniques and dating of the timber domes of St. Anthony. Results from the dendro-sampling campaigns provide evidence of 13th-century elements still in place. Moreover, cross-references between on-site findings and archival materials enable the tracing of the dendro-provenance of replacements from the 16th century. Finally, a short comparison with the timber domes of St. Mark in Venice and St. Justine in Padua enhances the importance of the ancient timber structures of St. Anthony.

Keywords

Timber, Dome, Superstructures, Dendro-analyses, Medieval, Padua.

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1. THE TIMBER DOMES OF ST. ANTHONY

On the upper levels of the pilgrimage Basilica of St. Anthony in Padua, at the turn of a narrow medieval corridor, the visitor may stumble across spectacular attics above the eight domes of the church. Here, relatively light timber frames surmount the extrados of the masonry shells, supporting the delicate ribs of the curved roofs (Fig. 1).

In the last forty years, scholars have proposed several hypotheses on the building's development [1–3]. These imprecisely positioned the erection of the domes in different phases between the second half of the 13th century and around 1310, when historic sources report about a «magna et inmensa mutatio», probably referring to the eastern development of the Basilica [4]. However, there is no common agreement between these theories, which lack tangible proof. Later publications connected the planning of the domes to the original project of the church [5–7], yet without significantly narrowing down a possible dating. So far, the literature has only superficially described the timber structures without analyzing either their constructive features or on-site traces [8, 9].

The lack of scientific evidence regarding the dating of the Basilica and its timber domes motivated the launch of a building-archaeology project in 2019, supervised by Professor Stefan M. Holzer at ETH Zurich, in collaboration with the Pontifical Delegation for Padua and the Venerable Ark of St. Anthony, financed by the Swiss National Science Foundation (SNF). When the project started, fundamental questions were still unsolved about how and when the domes were built or the possibility of retrieving original elements. Ongoing analyses of the lower brick structures have already attached the construction of the western domes to the first building site [10]. In parallel, doctoral research has been seeking to document and date the eight timber structures, contributing to positioning their construction within the timeline of the entire building. From a methodological point of view, the research project includes on-site measurements, laboratory analyses, the recording of traces, and the study of historical sources [11].

The geometrical survey has been achieved through laser scanning of the entire building with 6 mm accuracy (using a Leica RTC360 scanner). From the point clouds, it has been possible to draw sections and build a 3D model, whereas traditional hand-measurements have been essential to document joints, repairs, and other traces.

The dendro-sampling activity focused first on possible original elements in the domes that were supposed to be the oldest. Moreover, a detailed study of the archival sources has provided additional information on the repair history of the domes.

This paper aims at answering the following key-questions: what are the particular features of the timber frames of St. Anthony, which parts are still original, and which elements have been replaced? To do so, the general history of the domes is first unveiled based on archival sources. The following section describes the constructive features of each dome based on detailed on-site surveys. Then, the discussion turns toward the outcomes of dendrochronological dating. Lastly, the timber domes are positioned in their Italian context, and their importance is highlighted in the panorama of Construction History.

2. A HISTORY OF THE DOMES BASED ON ARCHIVAL SOURCES

The most ancient known chronicle citing the domes is a description by Giovanni da Nono, dated back between the last quarter of the 13th and the first half of the 14th century. Although its precise dating is still controversial, the chronicle depicts an internal view of the church

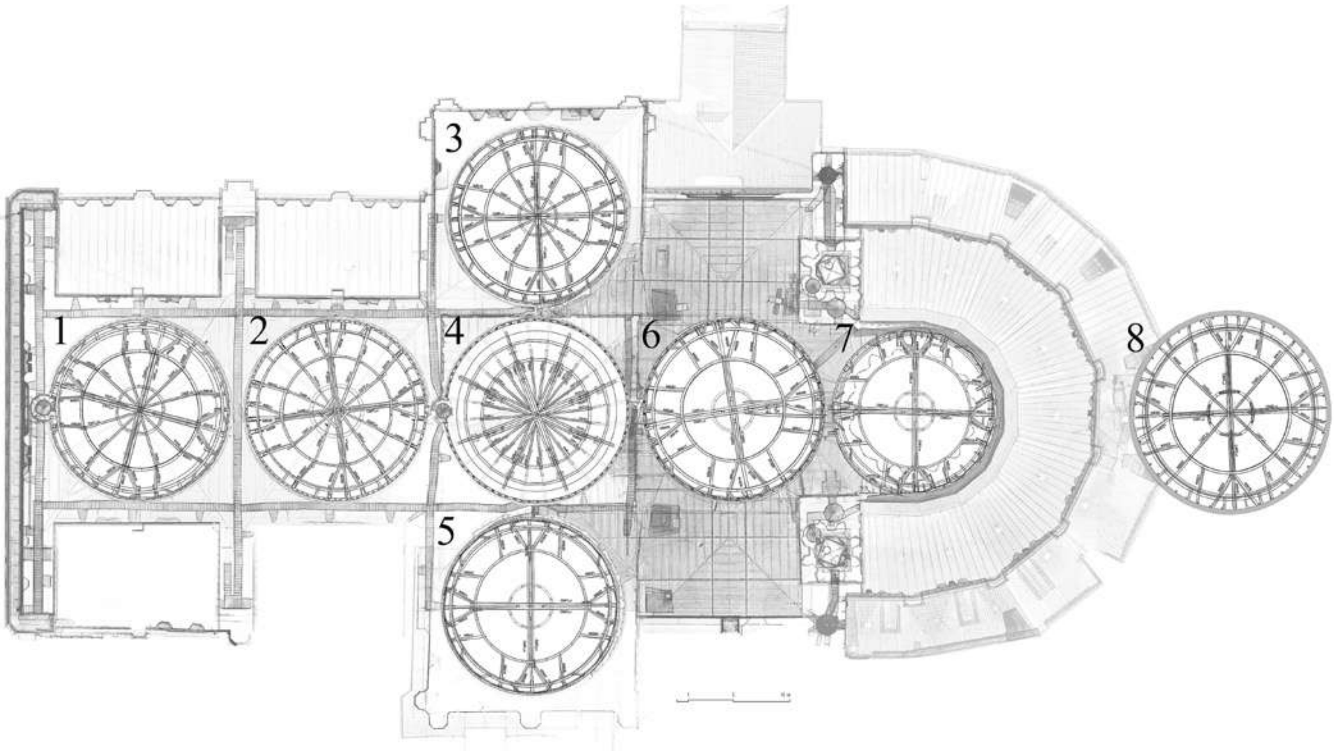

Fig. 1. *Attics plan of the St. Anthony Basilica: (1) façade dome; (2) intermediate dome; (3) St. Anthony dome; (4) angel cone; (5) St. Jacob dome; (6) presbytery dome; (7) choir dome; (8) relics dome.*

topped by seven domes, whereas only six cupolas are described from the outside. The following domes are mentioned: two on the nave, a cone on the crossing, two on the transept arms, and two on the presbytery and the choir. Moreover, another account by Bernardino Scardeone mentions the seven domes in 1330 [12]. Furthermore, two stone reliefs on tombs dated back between the 1330s and 1350s depict the domes, but these are the less reliable sources for counting due to their impreciseness. However, from these sources, one can thus determine that the Basilica had already gained its current silhouette in the first decades of the 14th century.

Due to the loss of archives related to the original worksite, the most ancient documents referring to renovation works on the domes are dated to the 15th century [13]. It is possible to trace a timeline of roofing works through deliberations and payment receipts, although the repairing activities are sketchily cited without precise descriptions. Indubitably, reparations occurred in the façade, St. Anthony, Angel, and choir domes over this century.

In the middle of the 16th century, archival documents mention a general decay state in all seven domes. Lists of building materials provided for repairs include larch beams and boards for the bearing structures and the outer cover. Skilled blacksmiths and carpenters from St. Mark in Venice and the Abbey of St. Justine in Padua took part in the works. In particular, as typical at that time in the Venetian region, carpenters had to travel towards the mountains to select the logs. Timber traders from Padua rafted the material along the Brenta River from establishments mainly located around the Asiago Plateau [14].

At the beginning of the 18th century, a new dome was built on top of the recently built Chapel of the Relics, modeled on the medieval ones. The masonry vault was completed in 1740, and the wooden frame would have been erected by 1745. At about the same time, disrepairs afflicted the façade dome, the St. Anthony dome, and the Angel cone. So far, uncertainty remains as to whether maintenance works took place. Shortly after, in 1749, a dreadful fire destroyed four of the eight domes: the Angel dome, the dome of St. Jacob, and the two above the presbytery and the choir. Historical depictions show the Basilica amputated from four wooden domes, with the exposed masonry shells that remained intact. The four burnt timber structures were quickly reconstructed. The mathematician Giovanni Poleni (1683-1761), previously involved in the structural evaluation of the St. Peter dome in Rome in 1743, contributed to the reparation plan, setting the reconstruction according to the original configuration, «secondo le idee dell'antico architetto, non restare adito a nuovi pensieri» [15]. Between 1749 and 1750, the Venetian Lamberti family provided the timber, likely rafted along the Piave River [16].

The next intense restoration phase took place more than about a century later, between 1853 and 1867, under the ruling of the Austrian government. Interventions occurred in the domes above the nave, St. Anthony and St. Jacob domes, and the dome above the presbytery. The replacements concerned the king-posts, elements of the main structure, parts of the outer skeleton, large parts of the external wooden boards and lead plates, as well as portions of the masonry drums. For some of these interventions, detailed drawings depict the pathways, platforms, and cantilevered galleries scaffolds attached around the drums installed during the worksites. Moreover, the descriptions of the workflow works include details on the elements size of the timbers and the lifting towers used on different occasions to hoist the material.

Although the scattered archival sources leave many open questions on renewals before the fire in 1749, they provide crucial information about damages, repairs, and timber supply in the 18th and 19th centuries. They also indicate that three out of the eight domes did not burn in the fire of 1749. As further discussed, the written sources reveal the perpetuation of the same construction scheme repeated over the centuries with only slight modifications.

3. CONSTRUCTION FEATURES

On the western part of the Basilica, the attics are connected through a rational network of corridors. Apart from the Choir and the Chapel of the Relics, the domes are supported by four brick arches with pendentives. Each dome is composed of an internal masonry shell, the inner diameter of which varies from 13.62 to 14.48 m in diameter from 32 to 43 cm in thickness, surmounted by a seemingly archaic timber structure covered by lead sheets. The wooden frames are installed well above the springing line of the masonry shell and are surrounded by the upper part of the drums. Together with the fact that wooden pieces are never embedded in the masonry, this observation tends to indicate that the brickwork was already in place when the timber frames were assembled.

Taking aside the cone of the Angel, the vertical bearing systems have similar characteristics. Each of them is composed of four symmetrical groups, composed of a central rafter and four secondary struts. All rafters and struts depart from short wooden supports raised from the level of the floor by stones. The four rafters, inclined by an angle of about 50°, lean up on a king-post to form two triangular frames rigidified by one or two levels of collar-beams, the ends of which are supported by the lateral struts. Four concentric rings rest successively on top of the brick parapet, at the levels of the collar-beams and close to the king-post. They support the slender ribs, forming the skeleton on which the outer cover leans. Vertical posts depart from the upper collar-beams to carry the last ring. The cover is composed of a layer of 2.5 cm-thick wooden boards and external lead plates. Similar and further features enable grouping the domes according to their configurations.

3.1. THE FAÇADE, THE INTERMEDIATE, AND ST. ANTHONY CHAPEL’S DOMES

As aforementioned, the timber structures of the Façade, Intermediate, and St. Anthony domes did not burn in 1749, so they have been considered the most ancient preserved (Fig. 2). In these frames, lower collar-beams rest on top of the masonry vault. Spokes form an additional division of the horizontal levels every 45°, except between the lower collar-beams of the St. Anthony dome. The shorter struts resting on the parapet support the lower collar-beams and spokes. In the Intermediate dome, the vertical posts that support the last ring are also set on the upper spokes. King-posts were replaced in these three domes during the 19th century, with a tapered-box profile as a reinforcement at the landing of the ribs. The three outer skeletons are composed of 99, 92, and 94 ribs, respectively, while the number halves in the reconstructed domes. The ribs are attached to the horizontal rings with iron straps and small curved cantilevers from the 19th century (Fig. 6).

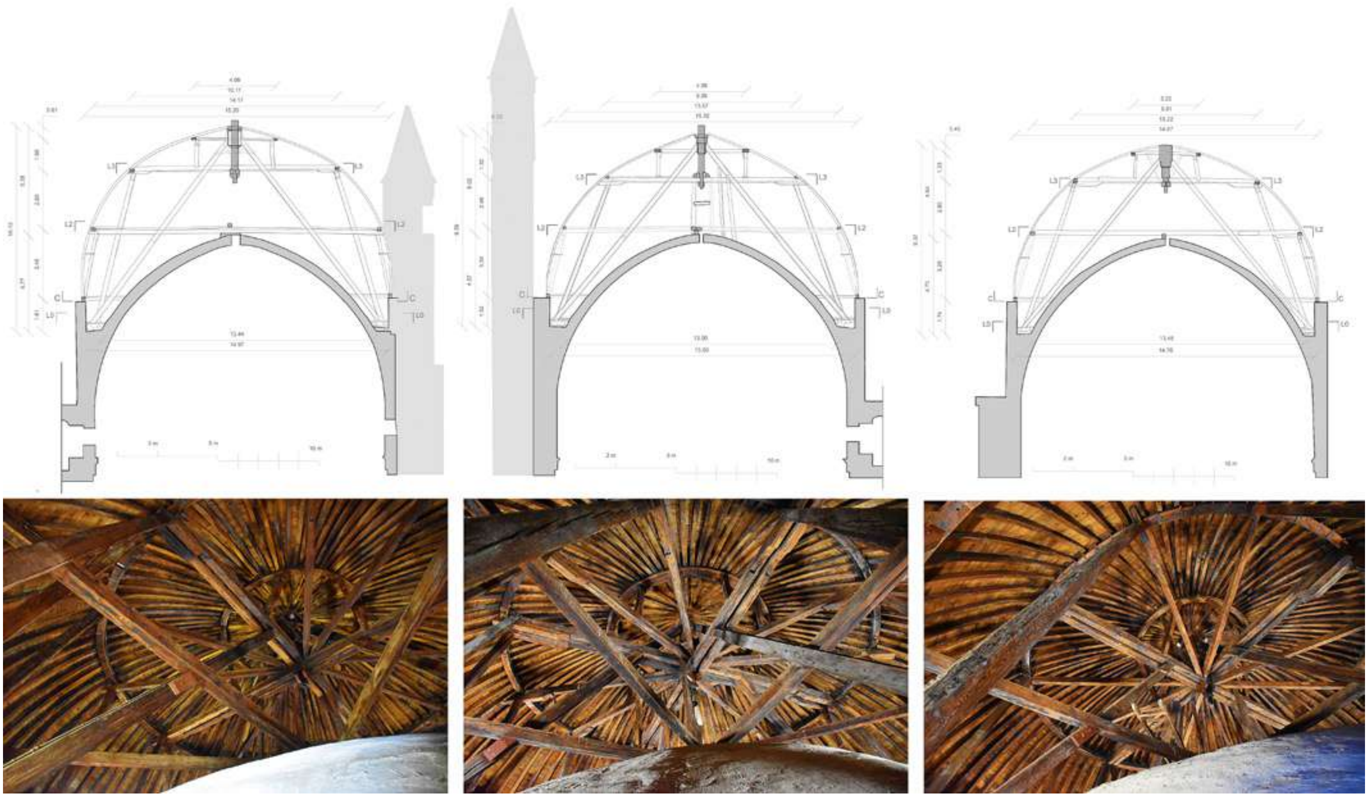

Fig. 2. *From left: the domes of the Façade, Intermediate, and St. Anthony chapel.*

3.2. THE ANGEL CONE

As reported by previous authors, the Angel cone likely evokes the Holy Sepulchre in Jerusalem [7]. Unlike the other domes, its drum is raised on a square masonry base. Most authors tend to date the completion of the masonry shell to the first Saint's translation to the crossing in 1263 [1–3]. Destroyed by fire in 1749, a description dated two years earlier confirms that its reconstruction followed the previous outline. In elevation, it is divided into five stories: the base, three intermediate levels, and an external one on top of the truncated cone (Fig. 3). The vertical bearing system counts 24 struts bearing the first horizontal grid, and as many others running in between the two following upper levels. They rest on short transversal elements, settled between the collar-beams, or directly on the spokes. Eight 13.9-meter-long struts resting on the floor reach the third horizontal level. The king-post is composed of three segments assembled by stop-splayed under-squinted scarf joints with V ends for a total length of 23 meters (Fig. 3). In the third story, a bundle of braces branches out from the king-post and stabilizes the connection with the major struts.

The two lower horizontal levels, L2 and L3, are composed of two couple of collar-beams connected to the rafters with half lap-joints, for a total length of 12 m and 9 m, respectively. In L2, these are composed beams assembled with a scarf joint. In both levels, sixteen spokes are symmetrical split into the quadrants. In the upper levels, L4 and L5, the length of the collar-beams is further reduced to 6.3 m and 3.7 m. Moreover, the number of spokes is limited to a couple in each quadrant. On the last level, vertical posts lean on transversal elements and host wind-bracings linked by lap joints. Except for the first ring on the parapet, the upper rings of the skeleton are segmented into polygonal chains. Seventy-two straight ribs spring from the first ring and lean on the upper ones.

3.3. THE ST. JACOB AND THE CHOIR DOMES

In the St. Jacob (Fig. 4) and the Choir domes (Fig. 5), there is only a high level of collar-beams, with a length of 10.5 and 9.9 m, respectively. In both cases, there are no spokes. The lateral struts, and the shorter ones around the parapet, support the third and the second rings. In the western part of the Choir dome, due to the polyg-

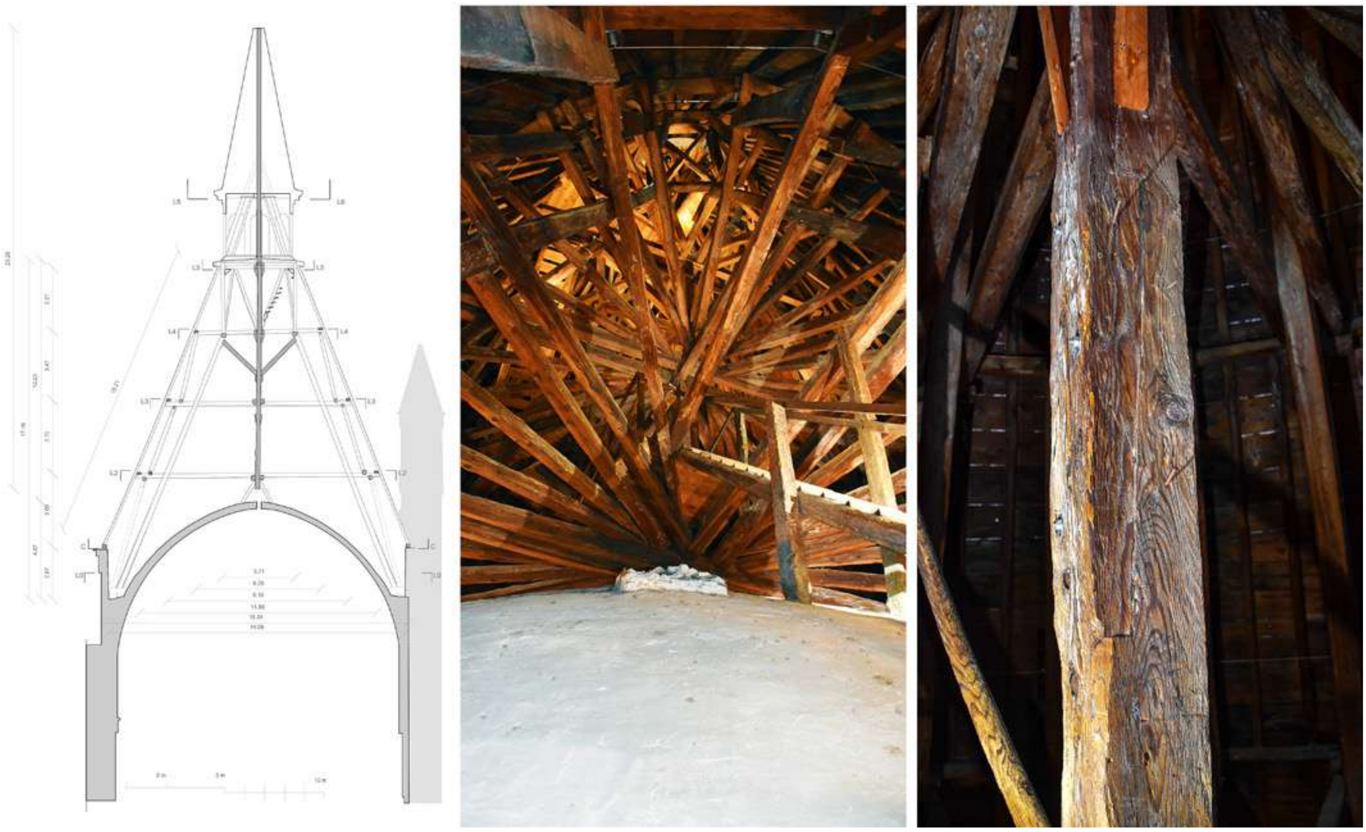

Fig. 3. *The Angel cone and a king-post's joint.*

onal contour of the masonry drum, some main struts that support the collar-beams rest on the first chain. In both frameworks, the king-post has a square profile with sharp edges and a thin cover-box at the top. Forty-eight ribs composed of two layers rise from the inner edge of the first ring. The rings are composed of three layers, the last of which consists of short segments lodged between the ribs. The ribs are tightened to the upper rings with metal straps and curved protruding boards. In general, the similarities between the two skeletons might suggest a common campaign of replacements dated the 19th century (Fig. 6). Eventually, the outer shell on the choir is characterized by a projecting profile, resting on twenty-four putlogs set along with the polygonal masonry and partly covering the gallery on top of the ambulatory.

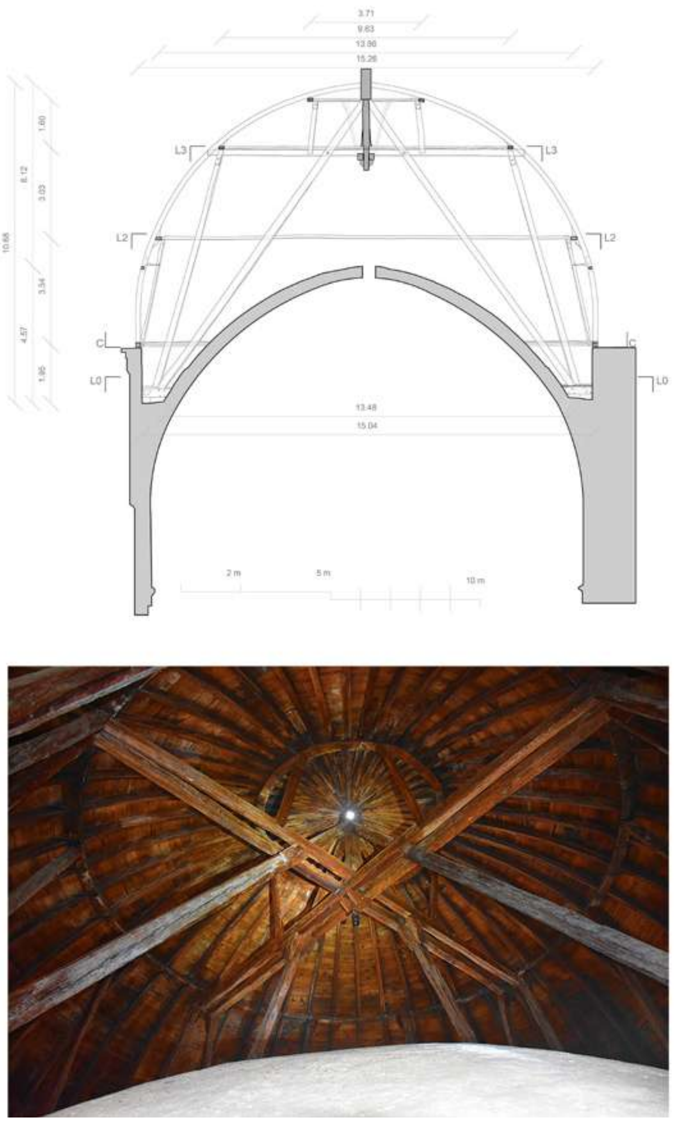

Fig. 4. *The St. Jacob dome.*

3.4. THE PRESBYTERY DOME

The Presbytery dome exhibits two twin horizontal grids, each composed of two overlapping couple of collar-beams (Fig. 5). On the lower level, these are 13.85 m long and are composed of two timbers jointed by means of a scarf joint. The king-post has a tapered profile without upper-box reinforcement and is tight between the lower level of collars. Like the above-described frames, the skeleton counts forty-eight ribs placed along the inner edge of the first ring and attached to the upper rings with the same connections as in the previous frames. Here, a modern wooden reinforcement rest between the ribs, which can also be found in the domes of St. Jacob and the Relics. In the southwestern quadrant, part of a scaffold is suspended with nails to the collar-beams. Its dating is unknown, although it can be positioned between the reconstruction after 1749 and the mid-19th century when its depiction appears in a survey of the church published in 1852 [11]. Due to the size of its elements and their cut, the scaffold would have been assembled at the same time as the reconstructed framework. Its function should relate to *manoeuvres* of liturgical apparatus inside the church through the holes visible in the masonry vault.

3.5. THE DOME ON THE RELICS CHAPEL

There is no parapet in the attic above the Relics chapel, which means that the main bearing structure and the curved ribs rest on the same floor (Fig. 5). The rafters and the major lateral struts stand on small pieces of timber. The ribs and the secondary struts are installed on the first wooden ring, which surrounds the masonry floor. An iron chain surrounds the wooden boards shaping the ring to contain the thrust forces; it likely dates back to the 18th-century construction of the new dome. It is composed of two concentric layers, each 2.5 cm thick and 7.5 cm high.

The main frame includes one level of about 10-meter long collar-beams, with four additional spokes symmetrically distributed into the quadrants. On the heads of the EW-oriented collar-beams, short transversal elements ease the connection to the second ring. The braces are anchored only on the four collar-beams. The king-post

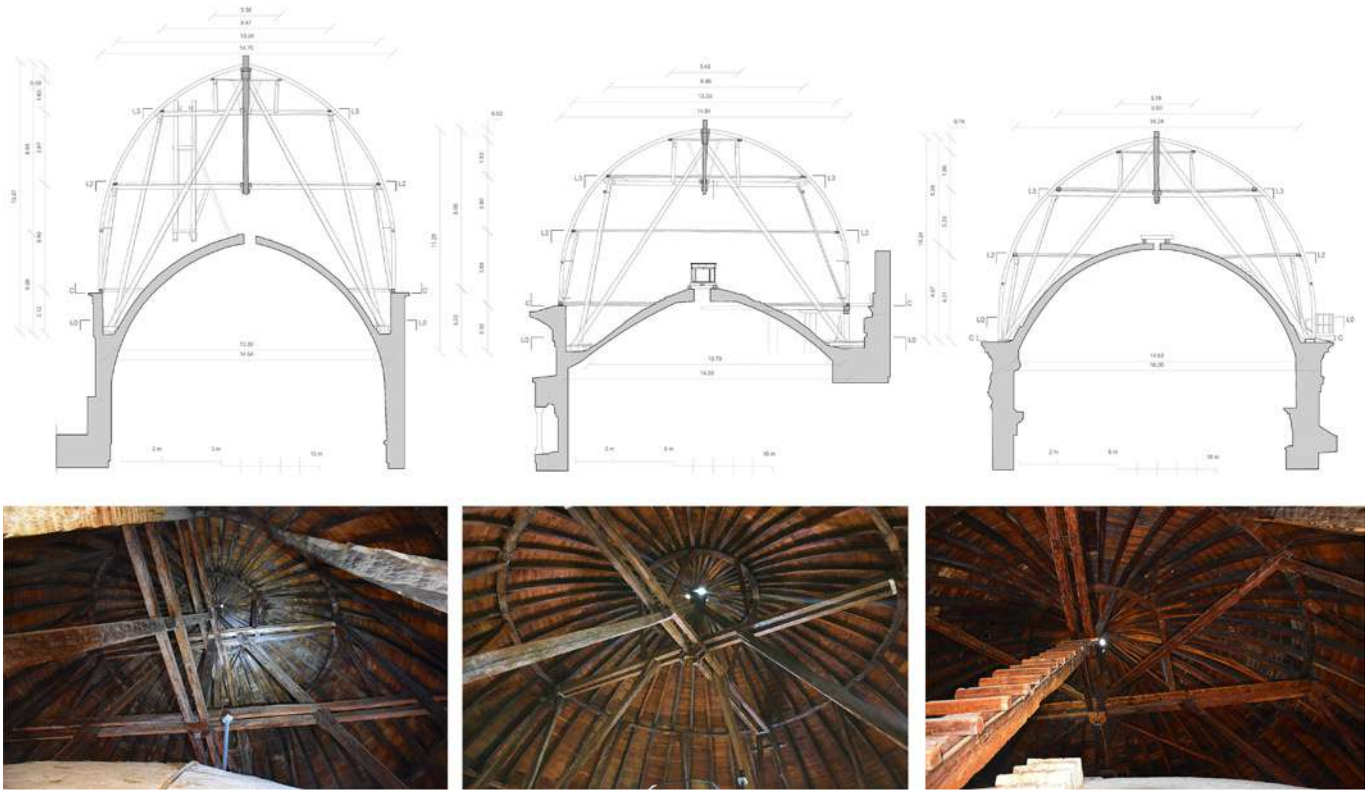

Fig. 5. *From the left: the presbytery, the choir, and the relics chapel domes.*

has an almost straight profile, like in the dome of the presbytery. Other analogies with the foregoing frames are the ensemble of forty-eight double-profile ribs, the iron straps and curved protruding connections between ribs and rings, and the modern reinforcement with wooden segments between the first and second levels of the ribs (Fig. 6). Finally, the wooden boards of the cover do not reach the floor level, they are instead supported by short wooden blocks and follow the projecting profile of the outer shell, like in the Choir dome (Fig. 7).

3.6. JOINERY TECHNIQUES

The structural integrity of the timber frameworks mainly lies in the use of lap joints fastened by iron nails and scarf joints. St.ruts and rafters are installed on short wooden elements through a simple notched joint, in some cases reinforced by an iron nail. The same type of joint is applied at the upper end of the four principal rafters leaning on the central king-post. The triangular frames formed by rafters and king-post, are stiffened by nailed connections with the horizontal collar-beams. These overlap each other through cross-lap joints. The spokes are simply attached to them with a nailed and are supported by lateral or minor struts with a nailed lap joint.

Multiple-profile elements, such as rings and ribs, are based on the layering of boards connected by different joints: tenon and mortise joints, scarf joints, or lap joints. As already anticipated for the individual frames, the connections between rings and ribs can vary at different heights. The ribs are embedded in the first ring in a 2-2.5 cm deep mortise, and modern sandwich-board reinforcements flank them. On the second and third levels, they are attached to the rings through iron straps and/or protruding elements (Fig. 6). These can consist of a curved extension of a rib's segment or small nailed cantilevers. In some cases earlier than 19th-century replacements, the ribs still lean on notches carved on the external edge of the rings. Furthermore, additional short pieces from the 18th century and later periods stiffen various connections between elements.

The simplicity of the joinery and the irregular length of the main structural elements likely facilitated their on-site adjustment and assembly. Long beams were probably cut on the spot, therefore avoiding the complex predefinition of each element forming the three-dimensional structures. However, composite elements such as rings and ribs could have been partially assembled on the ground (or on a working platform) and lifted. Firstly, circular sections of the rings were settled on the horizontal beams and top of the braces. Once rings were placed, the segments of the ribs were hoisted and lodged.

Fig. 6. *Joints between ribs and rings: choir dome (a) and façade dome (b).*

Fig. 7. *Relics dome: cover plates supported by short wooden blocks to follow the projecting profile of the outer shell.*

4. FINDINGS AND OUTCOMES

Findings from dendrochronological investigations and on-site traces help construct a chronology for the erection process of the domed attics. Xylotomy analyses confirmed all the samples and the cover boards are of larch (*Larch Decidua* Mill.). The dendrochronological activity revealed some precise dates and other *termini ante quem non* (t.a.q.n.) depending on the presence of the bark and sapwood on the elements. During four on-site campaigns, 70 samples (of 7 mm diameter) were collected. Results identified the following four time-spans [17] (Fig. 8):

1. between 1282 and 1284;
2. between 1297 and 1305;
3. between 1300 and 1317;
4. between the first and second half of the 16th century;
5. between the first and second half of the 18th century;
6. between the first and second half of the 19th century.

Between the first and second groups, there is a 10-year gap that could be explained by a preliminary stock of material or two consecutive stages of the same erection phase. These groups include rafters, struts, collar-beams, and spokes spread in the intermediate and St. Anthony domes, hereby confirming their original configuration. Elements of the third group are dated according to statistical approximations due to the absence of bark, and they are mostly located in the dome of the façade [18]. Other elements dated back to 1246 (t.a.q.n.) might belong to one of these three clusters or a previous stage. In particular, it is evident that the intermediate and the St. Anthony domes came before the façade dome. The erection process could have occurred in a sequence that started closer to the crossing and followed towards the façade, replacing probably previous roofs on the masonry shells. Eventually, the results of the fourth and fifth groups correspond to interventions reported in the archival sources, as previously discussed.

Analyses identified nine different local mean curves framed between 1023 and 1839, with morphologic similarities, each of which includes a few elements. However, their missing overlapping and chronological contemporaneity stress the timber supply from different areas.

Regarding woodworking, groups 1, 2 and 3 include sawn and axed elements. Even if sometimes hard to distinguish due to the timber age, these traces might indicate pit, cross-cut, and machine saw use. Indeed, despite possible speculations on the sawmill spread earlier, in the 13th century, the Venetian sawmill model spread in the sub-oriental alpine area. However, the length of rafters and struts, reaching up to 14 meters, would exclude a machine sawing since their standard carriage allowed a maximum of 6-meter-long logs. Nevertheless, the records align with the early medieval use of the saw in the

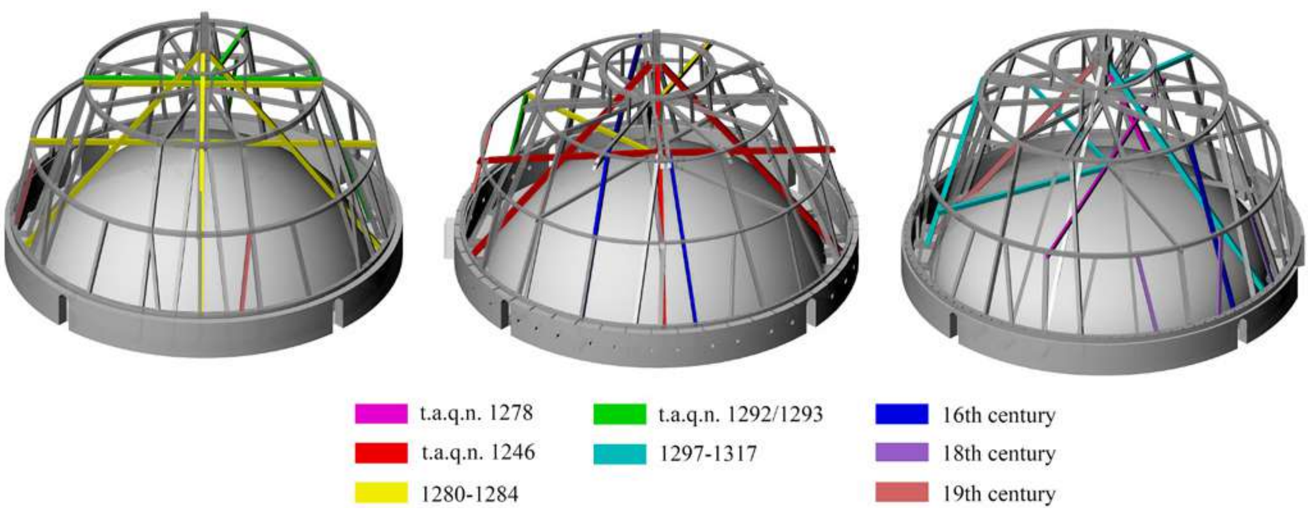

Fig. 8. *Dendrochronology results in the façade, intermediate, and St. Anthony domes.*

European context [19]. Nevertheless, the records align with the early medieval use of the saw in the European context.

From on-site observations, the absence of carpenter marks and the presence of trade signatures came out. In particular, these are evident in replaced elements in the reconstructed frames. Moreover, pegs, holes, and notches bear witness to rafting routes. The written sources, stored at the Archive of St. Anthony Basilica, contributed to deepening the knowledge about the merchants involved in the wood supply since the second half of the 16th century in Padua. The names of actors enable tracking the trade routes and the dendro-provenance. Registers of merchants indicate locations of their sawmills and the forests of their cut-license since the 15th century. The larch, cut in the sub-alpine area, was rafted along the Brenta and the Piave rivers. Furthermore, archival documents also describe some aspects of the building yards, such as sorting wood stocks by standard sizes and sawyers individually paid during building activities.

5. COMPARISONS WITH OTHER DOMES IN VENETO

The lack of preserved medieval domes in the region precludes comparison with coeval models. Nevertheless, it is worth considering carpentry and construction knowledge applied in the closest multiple-dome roofs of the Basilica of St. Mark in Venice and the Abbey of St. Justine in Padua.

Although a mosaic on the façade of St. Mark depicts the superstructures on top of the Basilica around the 1270s, their construction year is far to be known. The timber frameworks were erected as imposing outer covers on the original masonry shells. Still, the possibility of dating them precisely vanished with a fire at the beginning of the 15th century and with heavy restorations in modern times [20]. Yet, the reconstructed domes of St. Mark provide information about different construction techniques spread in Venice from the Renaissance. Like in St. Anthony, St. Mark's five superstructures are composed of a bearing system based on rafters, horizontal collar-beams, a central king-post, and circular ties. However, the frames gain in complexity and differ from each other. In the St. John dome, for instance, the vertical bearing system is repeated in two levels. The upper one leans on a polygonal tie, which is supported by tripods. Another system of tripods leans on the first horizontal level and supports the third ring. Upper struts rest on this first collar-beam level and support the last wooden ring. In the Choir dome, a unique system of struts departs from the first horizontal level to the last ring. The king-post is tightened between the upper collar-beams. A bundle of braces departs from the king-post and supports the last ring, counterbalancing the struts. The same system of braces and struts can be observed in the central Ascension dome. Here, the struts depart from a polygonal base, and the king-post extends until the lower level of collar-beams. The system of braces and struts also occurs in the Leonard dome. Finally, in the Pentecost frameworks, a double order of eight tripods and wind-crossing braces are settled in the intermediate levels. The skeletons and the outer covers recall the same described in the domes of St. Anthony. The widespread use of iron brackets and straps complies with the Renaissance and modern interventions [21] differing from the St. Anthony carpentry.

Wooden domes rose in abundance in the lagoon since the Renaissance [22]; they can be differentiated into two groups. The first relies on a king-post, with vertical rafters and struts inspired by St. Mark prototypes. Tripods or minor struts support the intermediate circular ties. Although the oldest does not exist anymore, these superstructures spread between the 13th and 16th centuries [23]. The second group consists of outer shells leaning on a simplified internal frame. In the dome of St. Mary of Miracles, elements dated back to the original 15th-century model are preserved [24]. Their configuration shows a central king-post flanked by collar-beams connected to an intermediate ring, without rafters support. Unfortunately, the rest of the frame consists of later renovations. The 15th-century timber frame of Saints John and Paul church was renewed in the middle of the 19th century. The timber superstructure of the Baroque church Saint Mary of Health does not provide much information as they were replaced in a renovation [25]. The 18th-century dome of St. Geremia church represents a model of wooden shells, without a king-post, with struts and wind-braced frames crowned by a lantern. At this point, the brief excursus about other timber domes in the Venetian lagoon excludes the exportation of the Paduan model as well as the permeation into it of other influences. Skilled carpenters were involved in Venice in the woodworking of timber traded along the Piave and Adige rivers [26].

Closer to the Basilica of St. Anthony, the Abbey of St. Justine provide another bevy of domes in the skyline further south (Fig. 9). Many architects took part in the long worksite, repeatedly reshaping the silhouette of the church. The erection of the domes started in 1587 with the internal vaulting and finished in the first decade of the 17th century. They vary in size and construction techniques. A central dome rests on the crossing, and three others crown the transept arms and the presbytery. Four additional minor domes surround the crossing at square corners. The larger domes, about 13 meters in diameter, are double wooden shells. The rafters lean on the first wooden ring and support perpendicular trusses. There is a unique horizontal level of collar-beams. The joinery includes mortise and tenon connections, scarf and nailed joints, and iron brackets typical from the late Renaissance period. In short, the joints and frames relate to a trussed roof, unveiling a more modern concept than the structures of St. Anthony. The smaller domes, about 7.5 m in diameter, are composed of a brick shell and a lighter timber superstructure, resembling St. Anthony's ones. Trademarks in the timber elements refer to 19th century replacements that could have changed the frameworks' original configurations. The differences among these seem to reflect changes of carpenters and construction practices over the centuries.

The comparison with nearby models thus enhances the peculiarity of the timber structures of St. Anthony. Their lighter configurations lay out an archaic model that was not copied in later projects. Apparently, the system was only repeated in St. Anthony after the fire of 1749. One cannot say whether they might have been inspired by the older (but lost) timber roofs of St. Mark in Venice.

6. CONCLUSIONS

The paper has described the construction techniques applied in the timber domes, which share a common bearing system yet varied types of collar beams, outer skeletons, and joints. Based on archival research and dendrochronological analyses, these differences could be explained and positioned in time. At this point of the research project, the dendro-dated components have confirmed the preservation of medieval timber elements in the domes that did not burn in the 18th century. Furthermore, there

Fig. 9. *St. Justine Abbey: longitudinal section. (Image source: photo courtesy of Pfister, 2022).*

is clear evidence that the brick shells were built before the timber structures. Consequently, the dating of the timber structures provides a precious *termine ante quem* to frame the medieval worksite of the lower masonries. The results highlight the early spread of sawn beams on the Padua market. Moreover, on-site traces and archival documents bear traces of the timber supply from the 16th century toward Padua.

Finally, the repairs implemented between the 18th century and the second half of the 19th century were carried out in a relatively conservative way. Inspired by the original joinery, with slight variations in carpentry techniques, the approach shows the continuation of medieval construction techniques over the centuries. No major innovative configurations were imported from the nearby context: neither those of the 15th-century St. Mark domes nor those of the 16th-century superstructures of St. Justine, not even those spread in the Venetian lagoon during the Renaissance. Therefore, the timber superstructures of the Basilica of St. Anthony represent a unique testimony of the 13th-century carpentry tradition in Italy.

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Authors contribution

The resources, the investigation, and the text conception and writing were made by M. Diaz. L. Vandenabeele reviewed the manuscript and supported the onsite investigations. S. Holzer reviewed the manuscript and acquired the financial support. All photos and drawings are by M. Diaz (except those for which specific attribution has been provided).

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LEARNING FROM TRADITION: A CASE STUDY OF THE DIAGNOSIS, DENDROCHRONOLOGICAL DATING, AND INTERVENTION ON A 16TH-CENTURY TIMBER ROOF STRUCTURE IN THE WESTERN ITALIAN ALPS

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Abstract

The paper presents a significant case study: the Church of San Giovanni Battista in Salbertrand dates back to the 16th century and constitutes one of the most interesting examples of religious architecture in the Susa Valley of the western Italian Alps.

Its historic timber roof structure was once at risk of demolition, but in 2000 finally became the object of necessary preservation and reinforcement works. Here, the interdisciplinary studies carried out for the diagnosis and assessment of the state of conservation are presented, starting with the identification of the wood species used, the geometrical survey, the visual and NDT diagnosis of the timber elements, and the structural evaluation. A special section is dedicated to the dendrochronological analysis, with a comparison of different case studies regarding larch roof structures of other historic architectures located in the northwest of Italy. The tree-ring sequences obtained from the buildings presented have also been used to define a larch chronology of the Susa Valley in Piedmont.

Following the first assessment phase, a second phase involved defining the restoration and reinforcement interventions. The reinterpretation of historic craftsmanship rules and traditions, which already contemplated the use of steel devices, attempted to offer alternative design solutions. This reinterpretation constituted the basis of the reinforcement interventions carried out in Salbertrand in the early 2000s. This paper highlights the importance of learning from historical treatises, showing how, even in modern reinforcement interventions, the application of traditional carpentry rules can achieve the aims of preservation and structural efficiency with overall cost-effectiveness and durability, resulting in a favorable balance between tradition and innovation.

Keywords

Architectural Heritage, Historic timber structures, Diagnosis, Dendrochronology, Construction Technology.

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1. INTRODUCTION

In building conservation, historic timber structures represent a specific class of handwork. They have particular importance and recognition due to their territorial diffusion, typological articulation, technological characteristics, and artistic and formal value. It is important to recall that these elements are built for structural purposes; however, especially in Italy, in the past, specific load-bearing timber structures in historic buildings did not receive the same attention and consideration than the buildings in which they are located. Therefore, they have not been treated with the same respect. Timber structures have often been demolished, replaced, or altered. Timber roof structures, in particular, have often met a worse fate than other structures because timber is a material that can easily be used, modified, or replaced in buildings. In the past, the neglect of these components was exacerbated due to the difficulty in assessing their actual condition and state of conservation of the historical timber elements and their mechanical performance, along with a lack of methodological pathways for the diagnostic phase.

Regarding their preservation, it was only recently that awareness was raised about restoring and preserving these structures as much as possible without altering their static role [1–3]. According to this idea, interventions should respect the nature of the structural elements and be coherent with their original conception and the material they are made of: wood. A major step forward in the diagnosis of wooden structures in Italy was taken with the definition of the standard UNI 11119:2004 *Cultural heritage - Wooden artifacts - Load-bearing structures - On-site inspections for the diagnosis of timber members* and of the standard UNI 11138:2004 *Cultural heritage - Wooden artifacts - Building load-bearing structures - Criteria for the preliminary assessment, design, and execution of works*, as well as other subsequent standards. Further advances in Italy have been made with the introduction of diagnosis as a mandatory phase within the tender procedure for works on historical roof structures.

In this context of intervention solutions, examining a few recent trends in the rehabilitation of wooden structures is interesting, with construction technologies that respect the guidelines recognized unanimously in the European context. These techniques make appropriate use of available resources in a perspective that tends to favor the concept of “sustainable technology” as a mediation between technological innovation, conservation, and the environment [4]. Special attention is paid to traditional intervention practices (such as replacing deteriorated elements with others made of wood or placing them side by side with other reinforcing elements such as wood or steel elements), as opposed to the use of more recent technologies (such as extensive use of reinforced concrete for the rehabilitation of timber floors, epoxy conglomerate prostheses for the heads of heavily decayed elements, fiberglass reinforcements, etc.).

Through the reinterpretation of historical treatises, this study intends to review traditional building methods to propose alternative design solutions to the repertoire of current intervention techniques. This operation does not mean a “return to the past”. Instead, it aims to take positive inspiration from the past to address a favorable convergence between tradition and innovation, between theory and practice, in compliance with the constraints imposed by economic assessments and needs relating to the sustainability of the built environment. Treatises and manuals of the 19th century are examined – a period in which there were many publications – and the focus is on works mainly referring to the western Alpine area in Italy. From these texts, general indications and specific suggestions proposed by the various authors on how to make interventions on timber structures have been analyzed. Particular attention has been paid to using metal elements to solve localized decay or reinforce the timber structure as a whole, restoring functional and structural efficiency to the wooden joints. Past suggestions, solutions, and rules, retrieved from the art of timber construction can still manifest aspects of excellent design interest today.

This paper analyses a significant case study: the church of San Giovanni Battista in Salbertrand, dating back to the 16th century and constituting one of the most interesting examples of religious architecture in the Susa Valley of the western Italian Alps. After risking demolition, its historic timber roof structure was the object of interdisciplinary studies carried out for the diagnosis and assessment of the state of conservation, involving geometrical surveys, the identification of wooden species, structural evaluation, visual and NDT diagnosis. A particular focus is placed on the dendrochronological analysis, which constitutes the basis for the construction of a larch chronology of the Susa Valley.

In the early 2000s, the reinterpretation of historic carpentry rules and traditions concerning, in particular, the use of steel devices attempted to offer alternative design solutions. It constituted the basis for the execution of the reinforcement interventions carried out in Salbertrand. The interventions on the church of Salbertrand forged a strict link between the preliminary diagnostic phase and the executive project (before the publication of the Italian standards for on-site inspections for the diagnosis of timber members). This approach was forward-looking and not particularly common at the time. The possibility of assessing the overall good state of conservation of these reinforcement interventions nowadays, some twenty years after they were carried out, constitutes an essential reference for future interventions. The long-term durability of these interventions and the need to preserve the original technological principles should be among the fundamental parameters of decision-making regarding the intervention techniques to be applied in structural rehabilitation projects.

2. LEARNING FROM TREATISES IN THE MAINTENANCE, REHABILITATION, AND REINFORCEMENT PROJECT

The construction techniques used in the past reflect a building tradition from which, despite the differences deriving from the various periods and geographical areas, emerges the importance attributed to maintenance work and how this is provided for in the design of a building. This relevance is proved by the historical treatises that contain many technical suggestions on the subject. In the past, maintenance, restoration, and renovation works were often carried out by modifying the existing building. Stone, wood, or metal elements taken from other abandoned buildings were often used. This recycling was achieved by dismantling and reassembling operations carried out by wise use of resources according to their limited availability [5, 6]. Maintenance works, therefore, were aimed at preserving the whole building through a continuity of transmission of traditional building techniques and with a program of interventions that allowed deteriorated elements to be replaced with sound ones.

Emy clearly expressed the concept of replaceability in one of the most precise carpentry treatises of the 19th century. In the *Traité de l'art de la charpenterie*, he suggests: «[…] if decay should be noted in any part of a construction, it must be quickly replaced with good materials […]» [7]. In the organization of the traditional building site, the replacement was an operation that was planned right from the design phase to ensure the easy replacement of a severely decayed part without having to pull down the entire structure. Owing to this ease of replacement, Emy states: «[…] and on this subject I will draw attention to the fact that joinery work that is very large in size, will be perfect and the cost of its construction well invested if, in addition to the conditions imposed by the purpose which has to be satisfied, it also presents the ease of replacing any of its parts which might show such deterioration as they might compromise the soundness of the building and the conservation of the other timbers […]» [7].

The undertaking of this program took place above all through the adequate design of the connections. Many treatise writers have emphasized the possibility of carrying out movements in the different directions of the wooden elements that converge in the joint. The wide range of connections they present required design criteria allowing easy replacement with simple temporary support of the structure.

Another important point about this maintenance system is the attention paid to the possible wood decay caused by biotic micro-organisms (fungi and insects). On this subject, in his manual *Technical elements in architecture* of 1924 [8], eng. Chevalley suggests that: «[…] when laying the roof beams, make sure they will last a long time by not closing them into the masonry (especially the heads of the tie-beams and rafters) so as not to risk the rapid decay of the structure». Along with these suggestions, Chevalley indicates two categories indicating how to position the heads of the beams so that they will last a long time: the first concerns the surface treatment of the head beams, and the second regards the construction technologies to insulate the wooden part from the masonry one.

Reinforcement work is intended to improve the structural performance of a building both in the design phase and in work carried out on specific parts. No matter if these latter have weakened due to phenomena connected with the characteristics of the material (rheological phenomena, movement of the geometry of the framework following cyclical settling of the wood) or on broken connections (caused by severe stress).

Reinforcements can be divided into two main groups: the first category concerns specific and localized parts of the structure (a connection or a section), which includes a series of devices such as bolts, arrow-head bolts, stirrups, brackets and metal laminas, and the second category that concerns the reinforcement of the whole structure or an overall increase in resistance and/or rigidity, such as reinforced beams which stand out for their originality of structural behavior.

2.1. METAL REINFORCEMENTS FOR LOCALIZED INTERVENTIONS

The use of metal elements as reinforcement of timber structures belongs to the tradition. Such practice has often been adopted in the past and may provide an effective way to increase strength and stiffness. Metal elements were used to compensate for the structural inefficiency of wooden components. Therefore tie-beams, strips, clamps, and metal connection elements were proposed to absorb traction stresses, improve faulty connections or restore missing ones. The new components were often placed alongside the original structure for structural collaboration.

The introduction of metal elements in the wooden structures to reinforce the connections took place, according to what the treatise sources testify, during the 19th century, with temporal differences between the different authors. In fact, we can note that, if in the 1830s Cavalieri di San Bertolo [9] considered it unnecessary to insert metal reinforcing, Breymann [10] on the other hand, towards the middle of the 19th century, deemed the exclusively wooden connections to be outdated, when he states: «For this purpose, a varied quantity of connections, partially very ingenious, was conceived […], since iron in the form of strips, screws, and similar, provides an excellent auxiliary element to add solidity combined with the simplicity of connection». Emy gives a synthetic but accurate picture of the gradual introduction of metal elements in wooden structures «[…] iron in timber construction is used in several circumstances to join pieces of timber together, to increase their strength, to consolidate the joints, to provide supports, to serve as an intermediate surface for wooden connections, and finally to replace a few pieces of wood. […] At first it was used to join different pieces together and to reinforce joints, and after a few years, during which the use of iron frameworks became frequent for roofs, attempts were made to use them as supports or tie rods for the timber framework» [7].

Emy's description on this point is also more satisfying as it is not limited to re-proposing pieces of advice drawn from the reading of previous authors. Still, he tries to deepen the reasons for the suggestion and to adopt it differently in response to specific problems, both in use for the reinforced beams and more in general in the connections (Fig. 1).

2.2. METAL REINFORCEMENTS FOR LARGE-SCALE ELEMENTS: REINFORCED BEAMS

This type of device refers at the beginning mainly to the supporting structures of the floors to obtain beams with greater resistance to deflection and thus contain deformations. In the evolution of these types, in the last quarter of the 19th century, reinforced beams were also used in roof structures (an example is the case of the Polonceau truss).

The system of reinforced beams became primarily widespread from the 17th-18th century. Jousse, whom Rondelet indicates as one of the first treatise writers on wooden carpentry, proposes in his *Le théâtre de l'art de charpentier* three examples of beams reinforced by different systems [11]. According to Rondelet these reinforcements that combined strength and economy were adopted «[…] because large-sized timbers are rare and very expensive, and generally of a less safe quality» [12]. Rondelet mentions this system as very frequent at the beginning of the 18th century, also used for the large halls of the Louvre. In its basic structure, the reinforced beam is made up of a wooden element of equal length to the span to be covered, over which two contrasting struts are positioned through different systems of jointing, indentations, bolts, and stirrups.

An effective system for stiffening very long beams, stressed in bending, was to provide them with other intermediate supports, through "columns" or struts, supported by tie rods that connect to both ends of the beam.

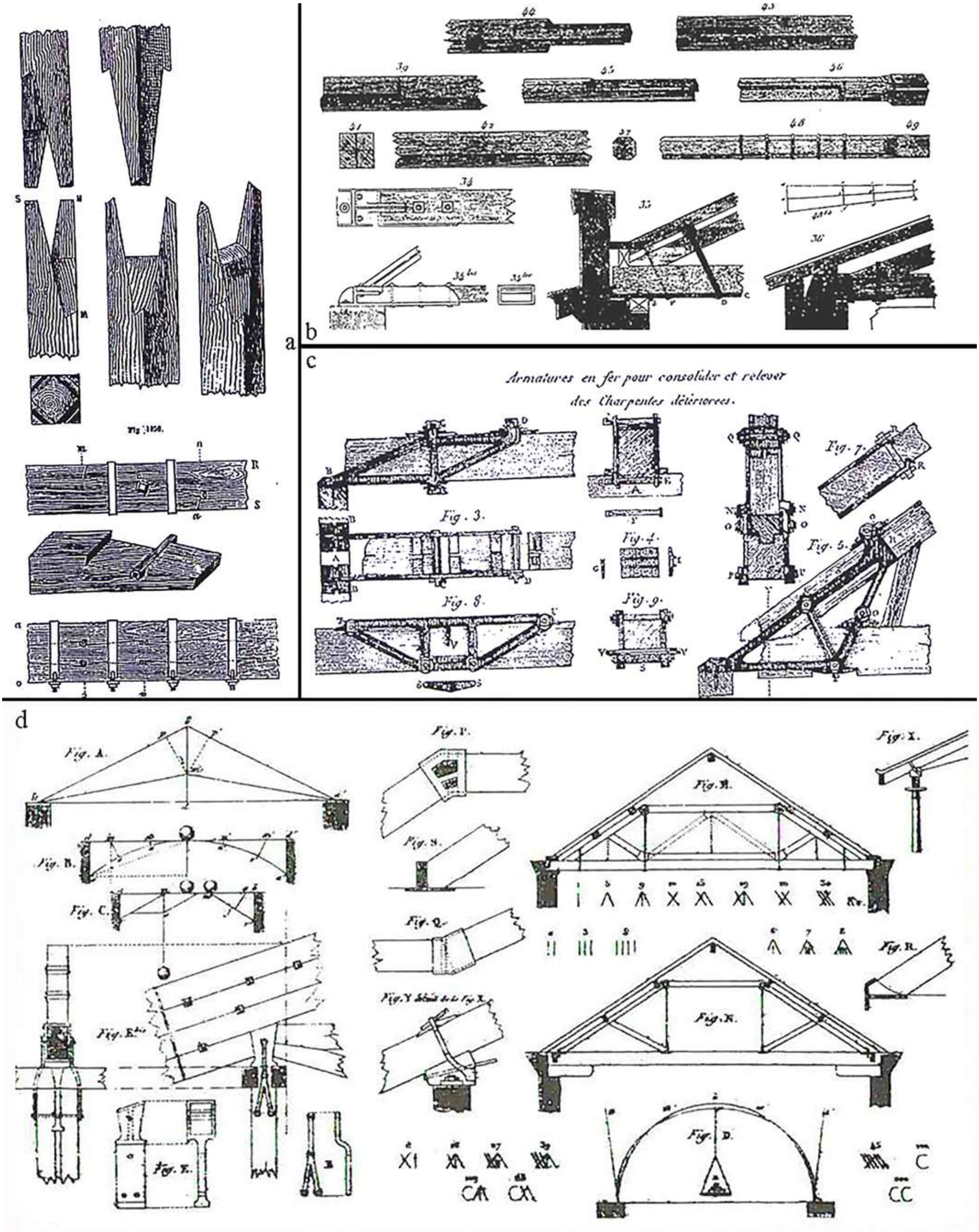

Fig. 1. *Joints timber/timber or with metal reinforcements for decayed elements, drawings from historical treatises and literature: (a) Pareto, Sacheri, 1880 [39, Vol. II, Part III, Cerriana S (ed) headword "Commessure", p 777]; (b) Emy, 1856 [7]; (c, d) Rondelet, 1802-1810 [12].*

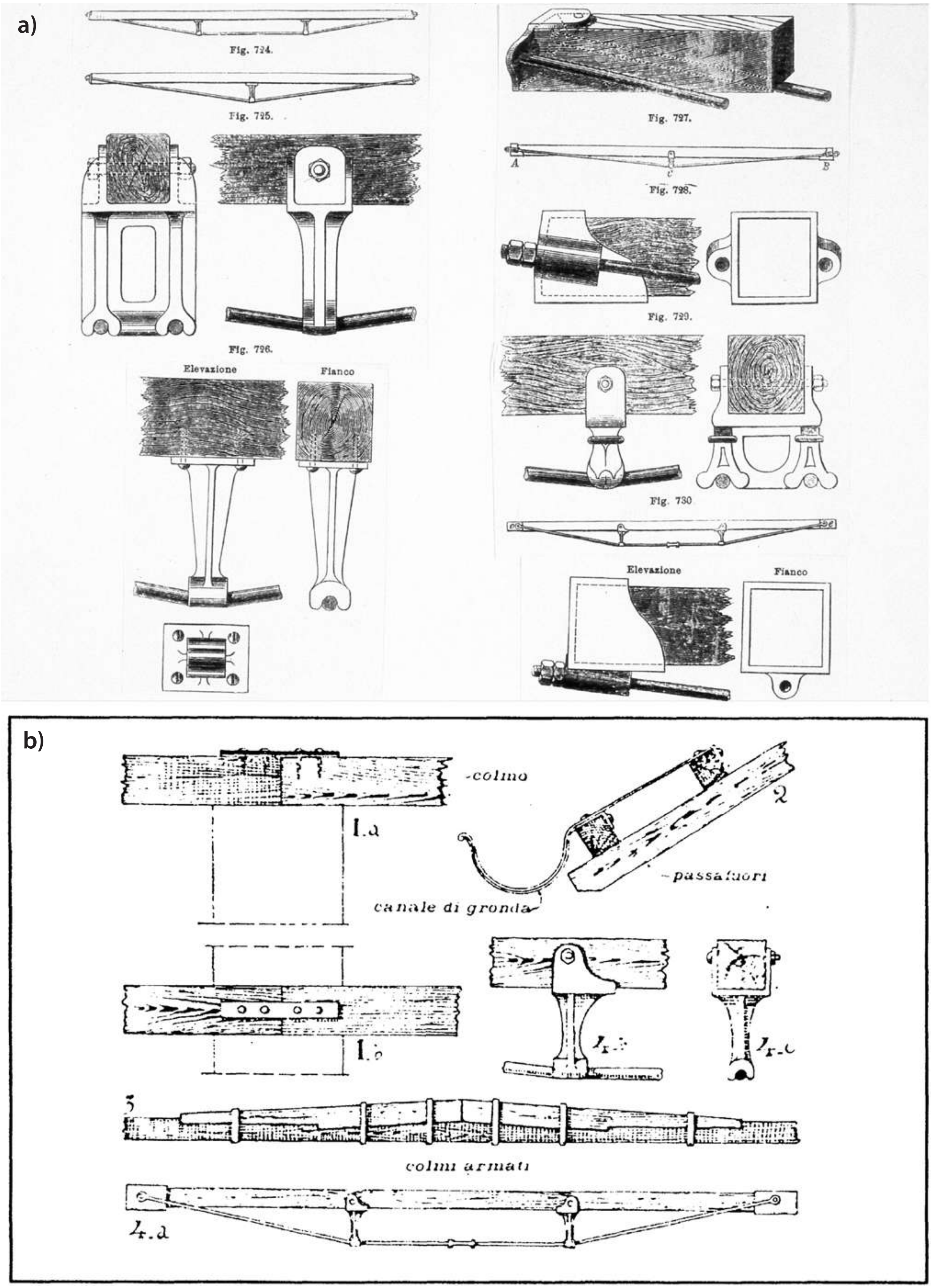

Fig. 2. *(a) Reinforced beams with one or two struts and details of the tie rods and struts in cast-iron, from Pareto, Sacheri, 1889 [39, Vol. VI, Part II-43, Cerriana S (ed) headword "Solai e soffitti", pp 330–331]; (b) reinforced ridge beams, from Chevalley, 1924 [8].*

Thus, were obtained the reinforced beams with one, two, and three struts. In all these typologies, the beam was made of wood, the struts of cast iron, and the tie rods of iron. Among the first solutions, the Polonceau truss [13] stands out for its maturity of conception and extraordinary correspondence between the geometric configuration and the tensional state of the elements. It was designed in 1837 for pitched roofs and was conceived as a mixed structure in wood, iron, and cast iron.

Different types have originated from the evolution of the older form of the reinforced beam described above, whose structural conception is of great appeal and simplicity. All types stemmed starting from the mid-19th century, in which the construction rationality, favored by developments in science and construction technique, tried to make the most of the materials used: the first reinforced beams were in wood, iron, and cast iron, then in wood and iron and finally all in steel (Fig. 2).

3. THE TIMBER ROOF STRUCTURE OF THE SAN GIOVANNI BATTISTA CHURCH IN SALBERTRAND

The municipality of Salbertrand, a small village located in the Susa Valley (Northwest of Italy close to the French border), still preserves its medieval architectural heritage almost intact, among which the parish church of San Giovanni Battista stands out, representing an important example of Gothic architecture. Alfredo D'Andrade referred expressly to some medieval architectures of Salbertrand in the construction of the "Medieval Village" built in Turin for the Turin Exposition of 1884.

Salbertrand is located over 1,000 meters above sea level, and most of the municipal land is covered by wooded areas. The largest site is represented by the *Gran Bosco di Salbertrand* Natural Park consisting of 3,775 hectares that develop on the orographic right side of the Susa Valley, from 1,000 meters to 2,600 meters of altitude, a mixed forest with magnificent specimens of silver fir, spruce and larch, quite unique in the landscape of the Alps of the Piedmont region (Fig. 3). The forest has represented for centuries the economic driving force of the Salbertrand community, differentiating it from neighboring villages mainly for quantitative terms and the high quality of the trees [14]. This high quality was already well known in past centuries. The large wooden beams coming from the Gran Bosco have allowed, starting from the 17th century, the construction of some of the most important military and civil engineering works as well as important architectures of Turin, the Savoy capital. Among them, we can count the Basilica of Superga, the royal hunting palace of Stupinigi, the Regio Theatre, and the Valentino Castle [15].

In recent years Salbertrand has been at the center of a large project supported by the European Union of enhancement and rehabilitation, with specific interven-

Fig. 3. *(a) Historical map of forests around Salbertrand (Mappa dei Boschi di Exilles, 1739, from [14]); (b) view of the larch trees in the Gran Bosco di Salbertrand Natural Park; (c) view of the village of Salbertrand with the Gran Bosco on the background. (Image source: photos courtesy of Gran Bosco di Salbertrand Natural Park).*

Fig. 4. *Church of San Giovanni Battista in Salbertrand. External church views. (Image source: photos courtesy of Gran Bosco di Salbertrand Natural Park).*

tions aimed at conserving the cultural heritage of the historic villages of the Susa valley.

Among its most important buildings is the church of San Giovanni Battista, built between the 12th and 16th centuries. It constitutes one of the most interesting examples of religious architecture of the Susa Valley, a monument of significant interest under the architectural and construction profile. The church was originally a Romanesque building oriented towards the east, of which only the lower part of the bell tower has been preserved up to the present day (Fig. 4).

The first document in which the church is mentioned dates back to 1057. Rearranged in the following centuries, the church underwent an almost total reconstruction during the 16th century. After this intervention, it did not undergo significant changes, and it is still one of the best-preserved and richest examples of late Gothic Alpine churches. The building faces west on the churchyard with a monumental entrance porch, supported by two monolithic octagonal pillars that bear the date of construction: 1536. The interior has three naves separated by simple and multiple columns and complex Gothic pillars on which the capitals lay, diversified from each other on a stylistic and iconographic level. The wider and considerably higher central nave ends in a presbytery where the main altar is located.

Among the numerous architectural values of the church, such as the entrance porch, the portals, the columns with sculpted capitals, frescoes, etc., the covering of the central nave, which can be defined as a "gabled" roof with two inclined pitches, constitutes a masterful example of wooden carpentry of the 16th century [16].

The roofs of the church are organized on different levels and consist of: the roof of the bell tower, the main roof covering the central nave, the two lower-level roofs that cover the side aisles, the roof of the sacristy, the roof of the side chapel on the right of the main altar and the roof of the entrance porch.

All roofs are characterized by the following:

- the main structure in larch wood;
- a natural split stone slate roof covering;
- thick larch planks that support the stone slates.

The pitched roof of the central nave – the object of the present study – has two slopes with a sloping angle of 29°. It is composed of three small timber trusses of about 8 meters of span on which a large ridge of 20 meters (with a diameter of about 50 cm) supports 13 false rafters on each side of the slope displayed at close distance (Fig. 5).

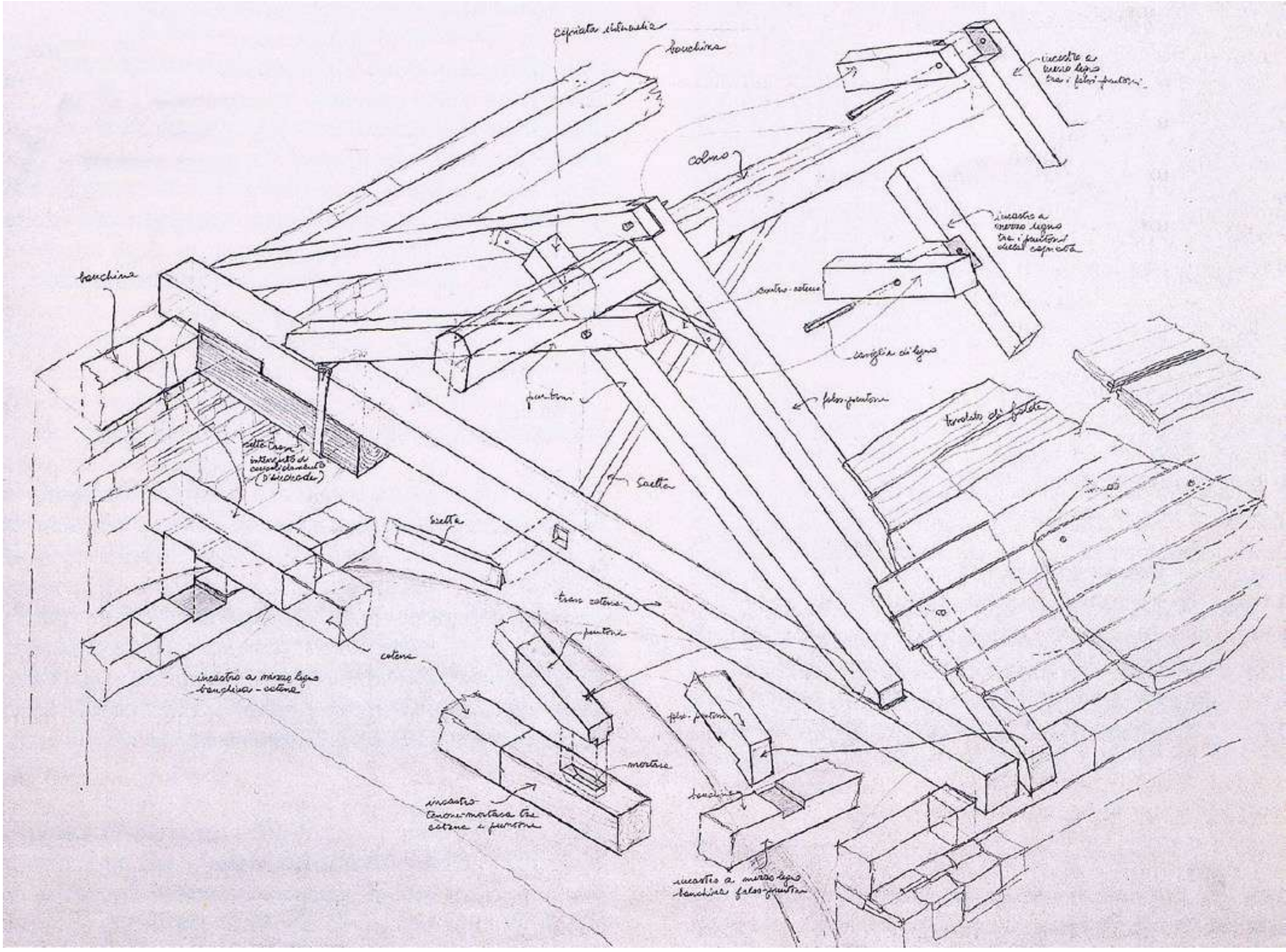

Fig. 5. *Church of San Giovanni Battista in Salbertrand. Construction survey of the timber roof structure and the typical wooden joints. (Image source: drawings courtesy by C. Bertolini-Cestari).*

The specificity of the trusses is the position of the connection between the tie-beam and the rafters, which is set back from the joint between the tie-beam and the longitudinal masonry walls of the central nave. This condition leads to part of the element being subject not only to tension but also to bending. With this arrangement, the ridge assumes a fundamental role in preserving and conserving all the different elements constituting the roof, particularly the inclined elements defined as false rafters. However, the roof structure, which appears poorly braced in the longitudinal direction, is connected on the slopes with two horizontal beams resting on the masonry walls.

The roof covering is realized with split local stones, also called "*lose*", large slates typical of the area fixed with dowels to the wooden boards that rest on the false rafter. Most of the wooden carpentry is in larch (*Larix decidua* Mill.) from the nearby Natural Park of the Gran Bosco of Salbertrand.

On the one hand, carpentry is characterized by skilled and valuable woodworking of the elements and joints. The connections between the rafters are half-timber with wooden dowels, those between the rafters and the tie-beam are tenon and mortise, as well as those between the false rafters in the support on the ridge and on the side platforms. On the other hand, the roof is characterized by the exceptional size of the ridge; a large beam of over 20 m long with an average diameter of 48 cm, which rests on the three trusses. Therefore, realized with a very durable material (larch) and of excellent quality, the carpentry had not undergone substantial restoration over the centuries, except for a reinforcement of a head of the tie-beam of the central truss. This intervention presumably dates to the end of the 19th century, when Alfredo D'Andrade was in charge of the regional office for the conservation of the monuments of the regions of Piedmont and Liguria [16, 1].

4. DIAGNOSIS AND ASSESSMENT OF THE TIMBER ROOF STRUCTURE OF THE CHURCH OF SALBERTRAND

The case of the roof of the parish church of San Giovanni Battista in Salbertrand represents an emblematic example of how a proper diagnosis and assessment of the timber structure constitutes an essential basis for its preservation, being able to avoid unnecessary dismantling or loss of the original wooden carpentry. The research carried out by the Department of Architecture and Design of Politecnico di Torino, in agreement with the National Board of Antiquities, was focused on the preliminary investigations (carried out in the late 1990s) and the project of rehabilitation of the timber structures (carried out in the early 2000s) with reinforcement interventions to allow the maximum preservation of the original elements. A qualifying aspect of the methodological path was the multi-disciplinary activity that has involved various professional figures from different sectors, such as architects, civil engineers, wood technologists, historians, and dendrochronologists, with the supervision of the National Boards of Antiquities responsible for safeguarding structural interventions on buildings belonging to our historical-environmental-cultural heritage.

The diagnostic and assessment phase was focused on the following:

- historical analysis and archival research on the documentary historical sources of the origin of the church and of the successive interventions over time;
- geometrical survey of the structures;
- identification of wood species and their characteristics;
- assessment of the state of conservation of wood material: visual inspections;
- assessment on the state of conservation of wood material: instrumental analysis;
- a further study phase was conducted to date the most important wooden elements through the dendrochronological survey. In view of the intervention, it was considered essential to have a sufficiently reliable dating reference of all the wooden elements of the structure (this investigation is described more in detail in the following section 5).

4.1. IDENTIFICATION OF WOOD SPECIES AND THEIR CHARACTERISTICS

The identification was carried out in collaboration with wood technologists. It was partly carried out visually for the elements with evident characters on the visible parts. In case of doubts, wooden samples were taken and identified in the laboratory with microscope analysis.

The tests that were carried out (macroscopic and microscopic) on the main elements of the roof system have provided the following results:

- the wooden elements constituting the main structure, the trusses (rafters, tie-beams), the ridge, the horizontal beams, the false rafters, and the planks appear as original elements and in larch (*Larix decidua* Mill.);
- some secondary elements, such as the connecting elements, belong to other wood species. The sampling of these elements was also limited to avoid damaging them excessively. For example, the wooden dowels of the rafter joints of the trusses and those of the "*lose*" stone slates are made of laburnum (*Laburnum anagyroides* Medic.). While the bracket, which supports one end of the tie-beam of the central truss, is made of oak (*Quercus petraea* Liebl.), etc.

Larch generally has heartwood resistant to alterations by fungi, while the sapwood is less durable. As regards the attacks of the most common xylophagous insects, this species does not appear to be of high resistance. Attacks by fungi or insects result from prolonged high moisture levels [17]. Also, in this case, due to the lack of portions of roof covering slates, in correspondence with some parts exposed to repeated infiltrations of rainwater, degenerative phenomena occurred in the wood. The knowledge of the specific characteristics of the wood species used in a structure proved once again to be indispensable for a thorough diagnostic study.

4.2. GEOMETRICAL SURVEY

The diagnostic activity involved a preliminary phase during which, simultaneously with the historical analysis, general data collection was carried out on the main geometric characteristics and the construction technology of the roof. Therefore, understanding the structure passed through an accurate geometric and photographic survey.

For the geometric survey, the technique used and the measurements carried out were very detailed as it was necessary to provide indications also on the different types of connections, on the interventions carried out over time, and on the additions, deformations, and decays of the various elements and of the ridge beam in particular. The survey was developed through plans and sections and carried out with traditional techniques. It was also due to the difficulty of accessing under the roof with bulky instruments, as they were at that time. An accurate geometrical survey of the structures, with details of carpentry and joints, has been carried out with drawings in scale 1:50, 1:20, and up to 1:5 for the construction details. The graphical drawings constituted the basis of the instrumental investigations for assessing the decay and state of conservation.

4.3. VISUAL INSPECTIONS

Visual inspections were carried out following an inspection protocol defined by the experience carried out by the same multi-disciplinary research group on the great trusses of the Valentino Castle in Turin. The visual inspection aimed to evaluate the original characteristics of each wooden element and the variations undergone during the structure's service life.

The visual inspection was systematically conducted on all parts of the roof structure (elements and their connections). It led to an initial evaluation of the defects and macroscopic characteristics of the wood, such as: the direction of the grain, the depth, and direction of the cracks, the different colors between sapwood and heartwood, the chromatic alterations due to fungal and xylophagous insect attacks. The determination of the depth and direction of the cracks was carried out with special harmonic steel blades. This survey could also detect the eventual presence and extent of ring-shakes (not present in this case).

As for the ridge beam, it was observed that it is made of a single larch wood element of considerable size and high regularity of shape. The tree trunk areas from which the timber element was obtained have been identified for this beam. In the area close to the facade, both the form of the pentagonal cross-section and its greater diameter reveal that this is the trunk basal part, which presents a higher extension of heartwood. On the other hand, the more regular, cylindrical section towards the apse identifies the trunk's top part. Technological observation, therefore, assumed great importance in obtaining information on the mechanical characteristics and durability of the material.

4.4. INSTRUMENTAL ANALYSIS

The visual inspection was integrated with NDT instrumental inspection. At first, wood moisture content was determined through a portable resistance-type electrical moisture meter. The knowledge of wood moisture content is crucial because it is a limiting factor for the development of fungi and wood-boring insects able to damage the wood. This first instrumental analysis (on the first level), carried out in the dry summer period, gave very variable humidity values along the ridge beam. The average values measured were: 12-13% in healthy areas and generally in the basal part of the trunk; 25% in areas colonized by active fungal mycelia; 17-18% in the areas damaged and affected by the decay of the wood due to brown-rot.

Afterward, visual inspection was integrated with instrumental tests to measure the wood's resistance. At the time (the late 1990s), two portable micro-drilling instruments/resistance drilling were mainly used: Resis-

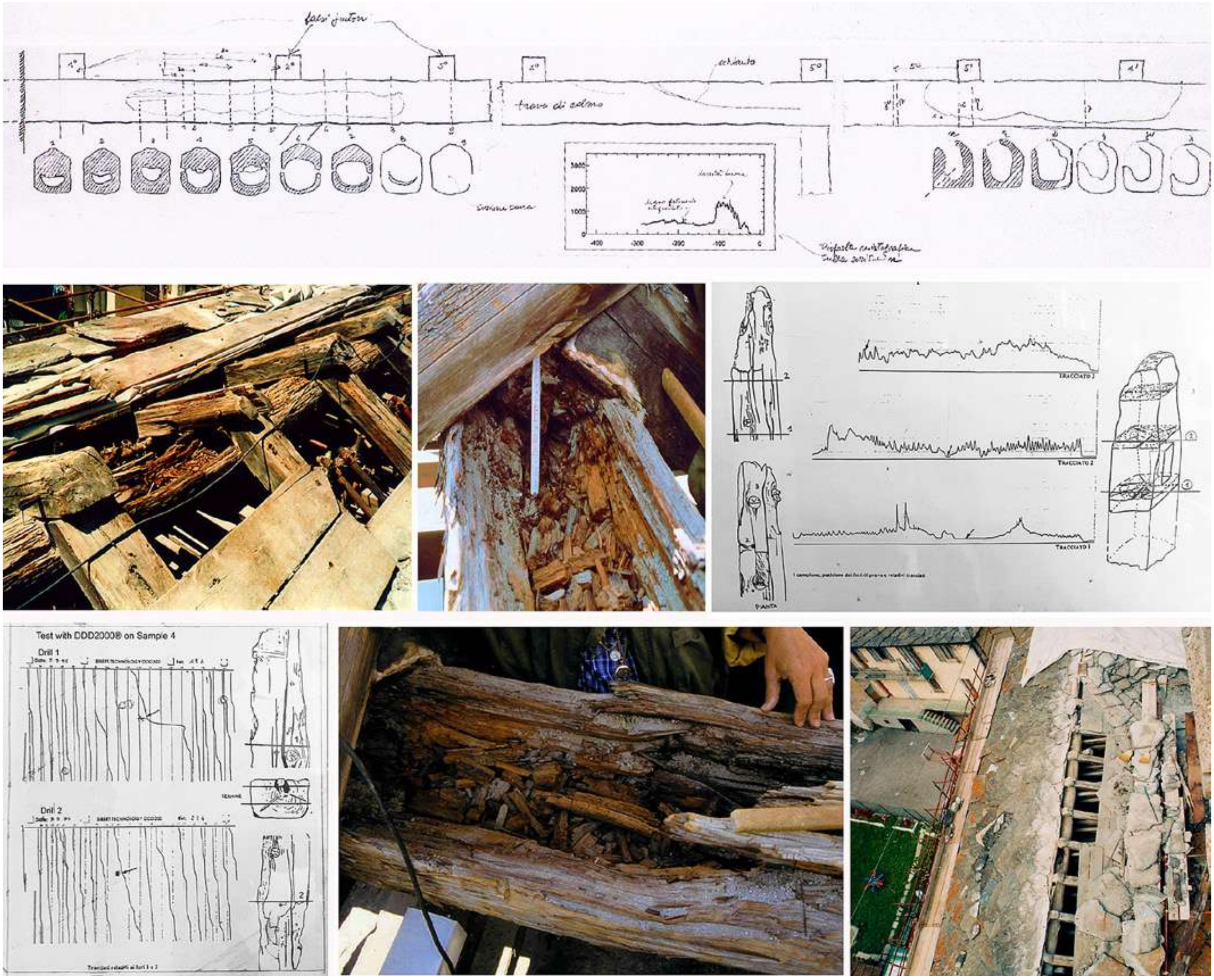

Fig. 6. *Church of San Giovanni Battista in Salbertrand. Geometrical survey with indications on the state of conservation of the ridge beam (decayed and sound beam sections). Examples of resistance drilling graph (Resistograph® and DDD2000® tests) and corresponding test points/directions; Photos of the decay detected on the ridge beam. (Image source: drawings and photos courtesy of C. Bertolini-Cestari).*

tograph® and DDD2000®. Such instruments allow the evaluation and quantification of the decay of the timber elements enclosed inside the masonry. In fact, this type of non-destructive test completed the visual survey to detect alterations that are not visible on the timber member's surface but may be inside (Fig. 6).

At first, some tests were carried out with a Resistograph® instrument, which evaluates the density of the wood through the resistance that the wood itself opposes to the penetration of a very thin drill (which produces a 3 mm diameter hole). This survey generates graphs showing: the depth of the hole to a 1:1 scale along the abscissae axis and the resistance to penetration along the ordinate axis, expressed in the amplitude of both the wooden rings' mechanical resistance and the moisture levels of the wood. Somehow similar is the DDD2000® (Decay Detecting Drill), a drill that makes a hole of only 1.7 mm in diameter. In this case, the drill speed is not kept constant, but the data recording is based on its variation. Tests were carried out on sections affected by the ridge beam's degradation. Tests with DDD2000®, in a total number of 31, were performed radially with respect to the longitudinal axis of the ridge beam. Tests with Resistograph®, in a total number of 21, were performed radially with respect to the longitudinal axis of the ridge beam. In the areas with severe decay, a complete correspondence was found between the results of the Resistograph® and those of the DDD2000®.

Another type of survey that was used, which was not localized or destructive, was the one involving the use of ultrasound instruments (Silvatest®). In this case, the use of electronic equipment allowed the estimation of the mechanical characteristics of the elements investigated. In the tests carried out in Salbertrand, the piezoelectric transducers at 55 kHz were used both transversely and diagonally to the wood. Here, too, the aim was to detect the presence of defects and decay within the wooden artifacts and to locate their position with precision. The limit of this equipment depended on the unlikely probability that the two heads of the timber component would be free. Consequently, ultrasonic assessments conducted on only one side of each element resulted in the variability of the measurement that made them ineffective in the evaluation of mechanical resistance.

4.5. RESULTS OF THE ASSESSMENT ON THE STATE OF CONSERVATION OF WOOD MATERIAL

The different levels of on-site investigation – visual and instrumental – allowed the characterization of the material and the evaluation of the state of preservation of the timber elements [16, 18] and proved to be fundamental for the study of possible reinforcement hypotheses.

Preliminary investigations identified several types of decay, whose main causes can be summarized as follows:

- abundant water leaking caused by the decay on the covering stone slates brought to the presence of xylophagous insects with extensive demolition of the cellular part of the wood (cavities and rot) and subsequent reductions of the resistant sections up to 1/3 of the original section for a total length of 6 m (Fig. 6);
- deformations and twisting from the shrinkage in the head connections of the false rafters, caused by slight structural failures of the ridge beam and by the decay of the contact surfaces.

The inspections carried out in the late 1990s showed an overall good state of conservation of the trusses and the rafters. Still, there was a severe biotic decay of the ridge beam in two areas (4 m length on the apse side and 2 m towards the façade side) (Fig. 6). Furthermore, due to creep (or *fluage*) in these areas of the ridge beam, a substantial deformation of about 30 cm at midspan appeared.

This state of decay and degradation required an urgent reinforcement intervention, considering the snow loads, which are abundant in the mountain area, and the great weight of the roof covering in stone slates.

From the analysis of the results obtained from the surveys, the following documents were prepared, constituting a fundamental basis for the study of possible reinforcement hypotheses:

- thematic charts, based on the construction-geometric surveys which highlighted: identified wood species; material defects; the presence of shrinkage creeks; decay, rot, cavities due to xylophagous agents; disconnections of the joints caused by deformations, shrinkage or partial collapse, etc.;
- first structural modeling of the behavior of the system as a whole;
- hypotheses for the integration and rehabilitation of the damaged parts;
- possible structural reinforcement solutions of the elements strongly compromised by the decay, based on those suggestions provided by historical treatises, particularly mindful of the conservation of the original wooden artifact: ancient suggestions and indications of operational practices that, necessarily updated, could effectively guide the rehabilitation intervention.

5. DENDROCHRONOLOGICAL SURVEY: A COMPARISON OF LARCH IN THE WESTERN ALPS FOR A REFERENCE CHRONOLOGY

Dendrochronology contributes to the knowledge of historical wooden structures, making it possible to date, sometimes with extreme precision, the wood assortments used in buildings. In Italy, dendrochronology is increasingly used in studying historical architecture, even if still not systematically, as in other countries such as Germany and Switzerland. In these countries, it is part of the preliminary diagnostic investigations for the rehabilitation project of a building, and the relationship between these investigations and the project is indeed very close. The publication of the Italian standard UNI 11141:2004 on *Dendrochronological dating guidelines* demonstrates the increasing interest in this discipline achieved in Italy.

The dendrochronological methodology is based on the principle that, in temperate zones, trees of the same wood species, which grow in the same geographical area, give rise in the same period to similar tree-ring series, where each ring corresponds to a calendar year. Therefore, the dating of an artifact of unknown age is obtained by comparing the tree-ring sequence that characterizes it with a reference chronology, representing the growth behavior of the wood species under examination over the centuries in that specific region. Therefore, of extreme importance research is the creation of reference chronologies for different geographical areas and species, which, in addition to increasing the chances of successful dating, also allows for identifying the probable site of origin of the wood used.

The studies conducted by the research group on the church of San Giovanni Battista in Salbertrand and other buildings in Piedmont, including in particular the Valentino Castle, represent interesting examples of the potential of the dendrochronological method, as well as constituting a valuable database for the construction of a reference chronology in this geographical area. Larch, the species used in the examined wooden structures, is widely used in the historical buildings of northern Italy [19–21].

In the presented case studies, an attempt has been made to select elements that preserved at least part of the sapwood and were characterized by a high number of rings. Since the beams were still in place, cores with a diameter of about 0.5 cm were extracted using a Pressler's increment borer. The sampling was always performed starting from the external edge towards the axial center, along two different directions, if possible, considering that a different number of rings could have been removed in the squaring of the beam. Only in some cases it was possible to take a cross-section of a few centimeters. The dendrochronological investigations were carried out following the classic procedures [22–24]. For the dating of the elements, in the absence of local reference chronologies, the chronology of the larch from the Vanoise Park (France) [25] was used since it referred to an area close to Valle di Susa and the possible provenance areas of the wood [15]. Subsequently, comparisons were also made with a chronology under development based on samples taken from living larch trees over five hundred years old from the Alta Valle di Susa (Pignatelli, unpublished data).

The Valentino Castle of Turin (Savoy residence of the early 17th century and Unesco World Heritage Site) represents one of the first dendrochronological studies conducted in Italy on wooden roof structures where extensive sampling was carried out [26, 1]. The investigations, which date back to the early 1990s, involved over sixty wooden elements belonging to the central roof and the four towers of the building. Thanks to the high number of wooden pieces examined and the presence of sapwood on 16 of the dated samples, it was possible to highlight the presence of different construction periods [26]. The oldest wooden elements are those belonging to the central roof, referable to the first phase of construction of the castle, documented by historical sources around 1620, while the beams of the roof structure of the towers belong to the years around 1630 (Fig. 7).

In the church of San Giovanni Battista in Salbertrand, the dendrochronological investigation was carried out on 13 elements of the timber roof structure of the church (truss beams, struts, false rafters, and planks of the roof), including a single plank attributable to the 19th-century restoration carried out by Alfredo D'Andrade, and on the ridge beam of the roof of the porch. The beams, not so accurately squared, allowed the conservation of at least part of the sapwood in most of the elements sampled, and in some cases, even the last ring under the bark seems to be present, allowing a precise dating (Fig. 8).

The dendrochronological investigations made it possible to date all the examined elements, dating back to

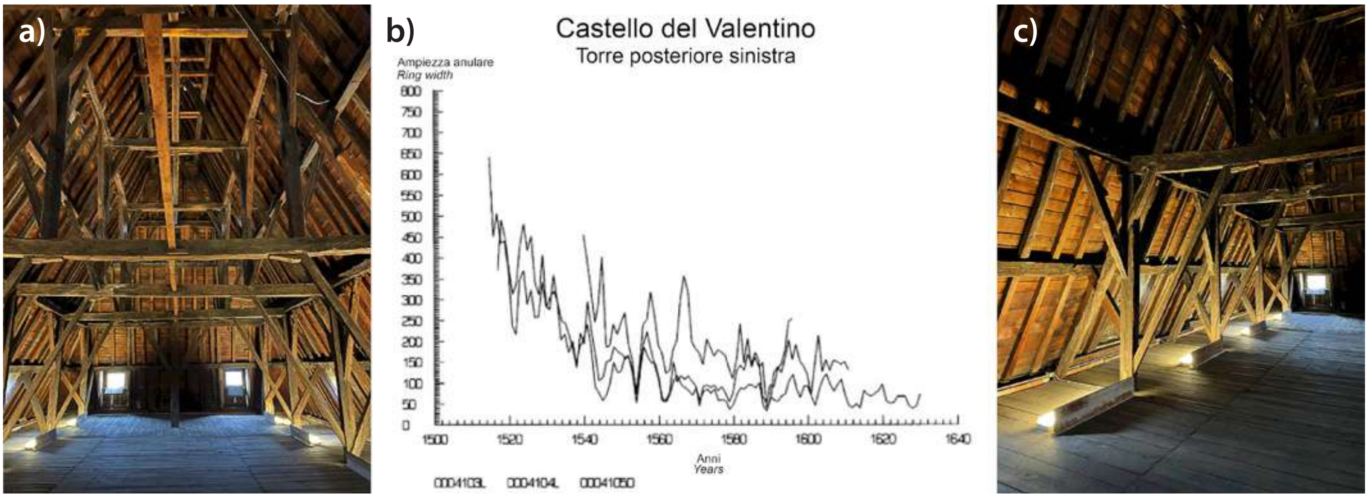

Fig. 7. *Valentino Castle, Turin: (a, c) view of the roof structures of the north-east tower (photos T. Marzi, 2022). (b) Dendrochronological curve of the three trusses of the back tower: the years, corresponding to the succession of the rings measured, are shown on abscissae, while the corresponding ring width is shown on the axis of ordinates.*

the 16th century, excluding the undated restoration plank. The chronology mentioned above of the larch of the Vanoise Park (France) was used for the dating. Comparisons were also made with the chronology elaborated for the samples of the Valentino Castle and the already mentioned chronology of the Alta Valle di Susa. The dendrochronological characteristics of the sequences suggest that the elements considered were obtained from larches probably coming from the same forest and referable to the same falling phase. By calculating the missing sapwood rings, it was possible to place the year in which the felling of the matrix trees may have occurred between AD 1513 and AD 1517. The probable presence in two samples of the cambium terminal ring (the so-called *Waldkante*), consisting only of spring wood, allowed hypothesizing that the felling of the matrix trees used for the roof took place at the end of spring and early summer of the year AD 1513 (Fig. 8). The result obtained for the ridge beam of the porch, whose last ring dated to the year AD 1520, constitutes the *terminus ante quem non* for felling, is in accordance with the date (1536) reported on the column of this part of the building.

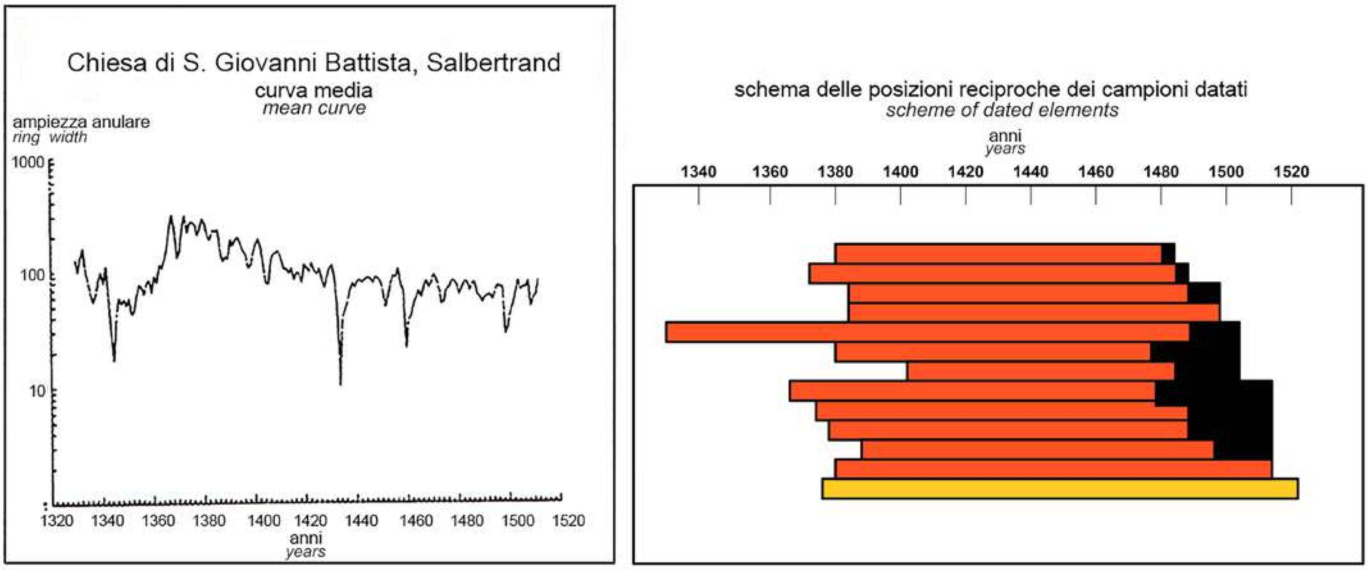

Fig. 8. *Church of Salbertrand. Dendrochronological mean curve. Bar diagram of the dated elements: in black, the part of the existing sap; in red, different samples from the timber roof structure of the church; in yellow, the sample from the ridge beam of the porch roof.*

The mean curve built for the roof covers the period between AD 1330 and AD 1512, overlapping only for a short time with that of the Valentino Castle (AD 1447-AD 1637).

Of consistent interest are the results obtained by comparing the mean curve of the Valentino Castle with the series obtained for the ridge beam of another small church, the Cappella del Seu, located in the Gran Bosco of Salbertrand, which covers the period AD 1490-AD 1674.

In particular, the optical and statistical high agreement (with t_{BP} higher than 8) between this last dendrochronological series and the Valentino Castle curve suggests that the wood used in the castle construction may have come from the woods of Gran Bosco of Salbertrand, as reported in some historical documents. This research results, obtained with extensive sampling, made it possible the dating the wooden structures examined and have become a fundamental part of the database created for the construction of a local chronology of the larch for the western sector of the Alps, from the 14th century to the present day. The systematic investigation of other historical buildings and larch forests in the Valle di Susa will allow the chronology to be consolidated and completed by improving the database relating to the valley.

6. THE SUGGESTIONS OF THE TREATISES IN THE REINFORCEMENT INTERVENTIONS OF THE CHURCH OF SALBERTRAND

A first intervention project, drawn up by the municipal technical office, was oriented towards replacing the ridge beam. But this replacement would have involved a complete reconstruction of the entire roof with a completely new roof. Due to the structure typology and its different parts, it would have been necessary to dismantle all the stone slabs, the underlying wooden planking, and the rafters. All these elements were still in a good conservation state but would have been damaged by the removal, and, therefore, difficult to be reused. That proposed solution was once again the proof that too often, a lack of understanding of building materials such as wood, sometimes leads to solutions that are completely inadequate, if not incorrect, when the skills that the restoration of timber structures requires do not accompany them: technological knowledge of materials and construction. Luckily, after the diagnosis and assessment of the state of conservation of the timber roof structure that was carried out, this initial solution was abandoned. The first diagnostic phase was followed by the second phase involving the definition of the restoration and reinforcement interventions and the execution of the works in 2000.

The intervention, agreed with the National Board of Antiquities, was aimed at the rehabilitation of the complex, with restoration and reinforcement interventions that would allow the original elements to be preserved as much as possible. Following the Principles for the Conservation of Wooden Built Heritage defined by ICOMOS [27], interventions followed the criteria of the minimal intervention capable of ensuring the preservation of the construction, saving as much as possible of its authenticity and integrity, and allowing it to continue to perform its function safely. Repairs of the original elements were carried out on-site, and the structure was strengthened as much as possible with traditional materials and techniques. In this specific case, reinforcements were based on the suggestions provided by the historic treatises that seemed more suitable and effective in guiding the rehabilitation intervention as respectfully as possible of the original artifacts, which, necessarily updated, have effectively led to the reinforcement interventions briefly described below. The techniques suggested by historic treatises have been applied not only for an overall improvement of the static conditions but also in the case of a specific lack of resistance and stiffness of the structure and localized decay of the elements. In particular, the decayed parts of the ridge beam were replaced with solid wood, properly shaped, and connected to the rest of the beam. The system was reinforced with the help of metal rods, thus achieving the strutting of the Polonceau truss [28, 29] (Figs. 9 and 10). Additional ventilation of the level under the roof was also provided with simple devices that allow natural air circulation without having water infiltration. Due to the limitation of the site (narrow streets of the village's historic center), the construction was also optimized by opening the roof from above and transporting the biggest element of the ridge beam (larch beam used for the substitution of the decayed portion) with a helicopter.

6.1. LOCALIZED REINFORCEMENT OF THE RIDGE BEAM WITH COMPOSITE WOOD-WOOD STRUCTURES AND STEEL ELEMENTS

For this intervention, the typology of "head" connections between the wooden elements was used, a technique of particular interest not only from a static point of view but also in the organization of the restoration site. This system was already in current use in the past to obtain beams of considerable length, as evidenced by the various treatises that report a great variety of solutions.

For this intervention, two head connections were realized to "recompose" the decayed structure to give continuity to the ridge beam. The interventions carried out have been:

- removal of the damaged section (length 4 m) and positioning of a sound wooden element (an old beam from a demolition of a wooden floor was used for this). For this phase, provisional support of the two parts of the ridge beam with tubular trellis resting on the perimeter walls was necessary so as not to overload the underlying stone vaults. The connections between the ridge beam and the new element were made on-site with "head" connections, also known as the double "dart of Jupiter" joints, tightened by three bolted stainless-steel collars. In this regard, it should be noted that the use of external stirrups (or through screws), required during installation, presents little guarantee of duration over time. A synthetic structural adhesive was therefore used to provide further unity in the parts where the two surfaces of the connections line up (Figs. 9 and 10);
- reconstruction of a sector of the beam, for the length of 2 m, with laminated wood realized on-site by adapting the boards to the cavity and fixed with a stainless-steel pin bonded with bicomponent resins.

6.2. REINFORCEMENT OF THE RIDGE BEAM WITH "REINFORCED BEAMS"

In addition to the interventions described above, the reinforcement of the ridge beam was completed with the realization of two reinforced beams, with double struts and double centering, with stainless steel rods – arranged in planes parallel to the lateral surfaces of the beam –, for the two spans of the continuous beam on three supports, respectively of 8 m of span. The intervention aims to stiffen the beam, which has an important deformation due to *fluage* (in the middle of the spans, the deflections are respectively 18 and 22 cm). The struts, placed at 1/3 of the span with a height of 0.80 m, are made of stainless steel with a thickness of 15 mm and are connected to the wooden beam with flanged and bolted collars. The ends of the tie rods are fixed to a steel collar whose anchoring is ensured by adjusting screws and stainless-steel pins. For all collars, deformable material (neoprene) has been interposed between wood and steel. In the middle of the beam, the tie rods are connected with a tensioner (threaded coupling) which ensures the initial tension setting, the control, or any eventual periodic adjustment (Figs. 9–11).

This type of work, with the use of two reinforced beams to increase the resistance and rigidity of the ridge beam, partially replaced with a wooden structure, proves that the teachings of Polonceau are still applicable today: a light and reversible intervention in which the combination of steel and wood guarantees the preservation in conditions of safety.

6.3. AN OVERVIEW 20 YEARS AFTER THE INTERVENTION

In Italy, at the time it was performed, the diagnosis and assessment of the timber roof structure of the church of Salbertrand (late 1990's) were particularly innovative to adopt such a multi-disciplinary methodology, involving also instrumental inspections that were relatively new. It was only starting from 2004, with the application in the diagnostic phase of the UNI Italian standards and with the definition of other international standards and guidelines for on-site assessment of historic timber structures

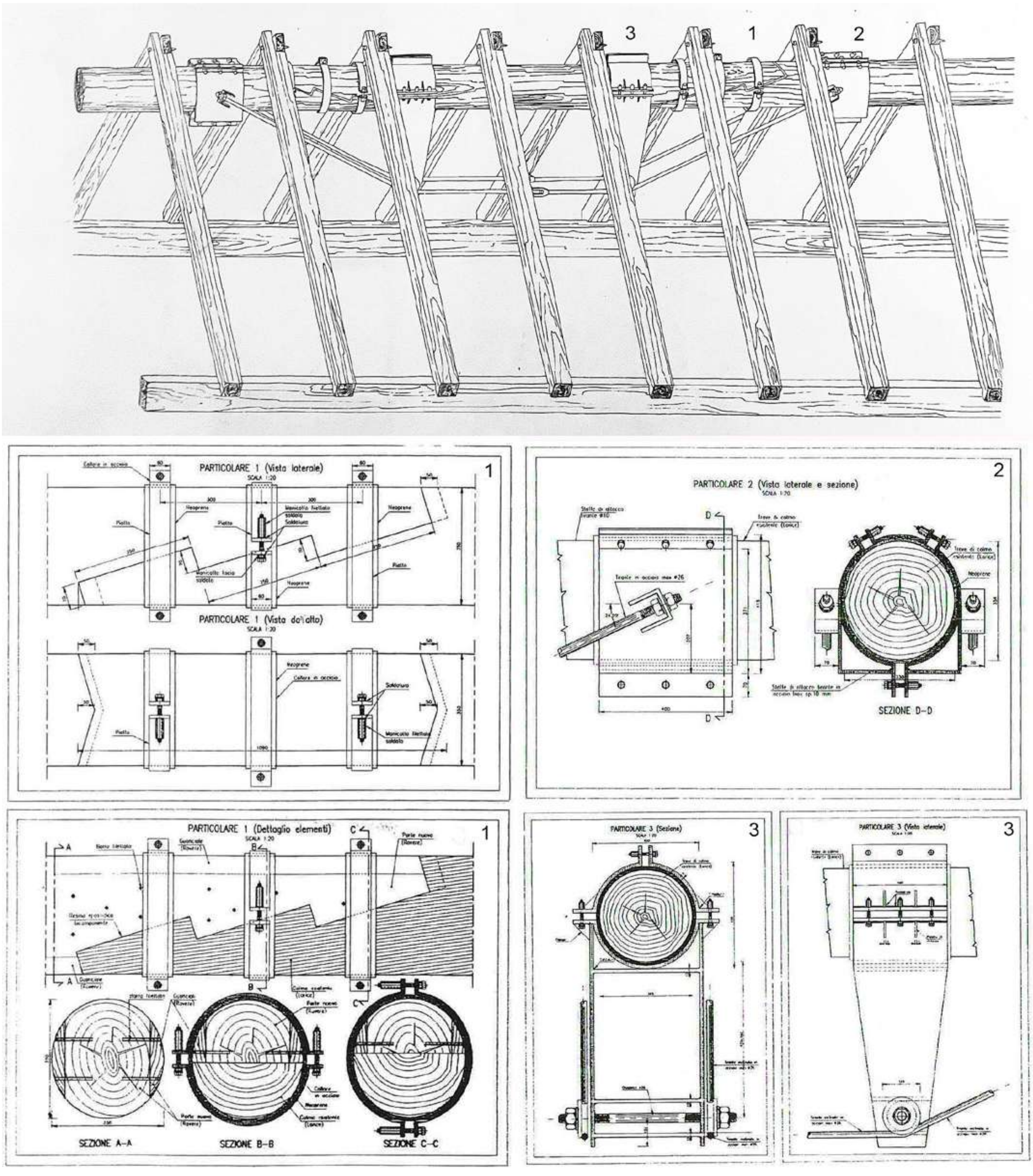

Fig. 9. *Church of Salbertrand. Reinforcement interventions on the ridge beam. General scheme of the interventions (on top); detail of the double "dart of Jupiter" joint, tightened by 3 bolted stainless-steel collars (1, on the right); and details of the stainless-steel tie rods and struts of the reinforced beam (2 and 3, on the left) system. (Image source: drawings courtesy of C. Bertolini-Cestari).*

Fig. 10. *Phases of the interventions carried out in 2000 with the integration of timber elements, the realization of the double "dart of Jupiter" joint, the transportation of the larch beam on the construction site, some details of the steel elements of the reinforced beam, and the scheme of the reinforcement intervention with Polonceau system. (Image source: drawings and photos courtesy of C. Bertolini-Cestari).*

[30, 31], that it was possible to obtain an evaluation of the mechanical resistances of the elements in a standardized manner. Over time, the diagnostic phase became increasingly accurate in defining the proposal of rehabilitation intervention. In the past few decades, technical and experimental research has developed survey methods involving various specialist areas, at times adapting knowledge from disciplines that do not belong to the construction sector (such as thermography, ultrasound, radiography, and tomography) and using the technological refinement of procedures inherited from research into architecture and technology of construction materials. In particular, non-destructive tests have become increasingly widespread, following new methodologies and diagnostic instruments, especially in recent years. Three-dimensional acquisition and modeling of structures, artifacts, and interior spaces can be based on LiDAR (Light detection and ranging) systems, photogram-

metry, and range cameras. Although, even if it was not yet so standardized and accurate, the diagnosis carried out in Salbertrand did already present the same primary objectives, and it was still possible to obtain important information that constituted an essential basis for the definition of the subsequent intervention phase.

The reinforcement interventions carried out in Salbertrand in the early 2000s as a reinterpretation of historical craftsmanship rules and traditions, which contemplated the use of steel devices, constitute an important reference given the assessment of the medium- and long-term durability of these interventions and the compatibility of different structural elements. Today, given the recurring requirements of conservation authorities in terms of the reversibility of interventions and compatibility between historical and new materials, an increase in the use of non-invasive reinforcement materials and reversible techniques was observed. Subsequently, engineers and researchers have increasingly employed stainless steel in retrofitting historic timber structures, also searching for innovative and advanced solutions [32–34]. Examples in which it is highlighted how steel (and in particular stainless steel due to its resistance to electrochemical corrosion facilitated by the absorption of the air humidity from timber members) is a suitable material to efficiently solve most of the static problems of timber structures, especially with the adoption of bar or cable systems placed in contact and forced to act in parallel with the existing structure, in order obtain several advantages: minimum intervention, the specificity of the solution with respect to the uniqueness of the object on which to operate, adaptability over time, low invasiveness, reversibility, recognizability, easy maintenance: a solution compatible and respectful of the existing structure and its original structural conception [35]. Steel allows interventions in line with the practice of conservative restoration, involving solutions placed side by side with the existing structures, without any subtraction, as a recognizable addition. The use of modern stainless steel and special tensioning makes solutions easy to adopt, light, removable, and cost-effective [36].

Visual inspections (according to the standard UNI 11119:2004 *Cultural heritage - Wooden artifacts - Load-bearing structures - On-site inspections for the diagnosis of timber members*) were recently carried out to assess the efficiency of the intervention and of the present condition of the joints (Fig. 11). The overall length of the ridge beam appears perfectly aligned. All the connections have also been checked and do not show any visible decay or alteration. Also, the connections realized with a steel tensioner (threaded coupling) are aligned and do not need additional settings or adjustment.

Wood moisture content was also recently determined through a portable resistance-type electrical hygrometer to assess the efficiency of the additional natural ventilation devices added during the restoration phase. An overall good condition was detected since all the wood moisture content measures were below 18%.

Regular visual inspections are planned, and in the future, it would be necessary also to carry out additional NDT investigations, numerical simulations, and monitoring to detect eventual movements in the structure and to assess the effectiveness of past interventions.

Fig. 11. *Church of Salbertrand. From left: Images of the roof structure in 2000 with the small portion of the roof that was opened during the intervention works, some details of the steel elements of the reinforced beam. Images of the roof cover with stone slates and the roof structure in 2021. (Image source: courtesy of N. Faure and R. Casse).*

7. CONCLUSIONS

Many structural renovations carried out in recent times have betrayed the idea of conservation, sometimes even involving the unnecessary demolition of centuries-old roofs. Such arbitrary or "excessive" interventions frequently stem from difficulties in assessing the state of conservation of the material and its actual load-bearing capacity, the incorrect evaluation of the structural behavior of these elements, or the adoption of superficial procedures guided by profit rather than by the real needs of the works in question. Despite the renewed interest in wood and the proliferation of studies on it, its load-bearing capacities continue to be questioned by operators. Their inadequate knowledge and lack of confidence in the material are revealed in their use of reinforcement techniques using supports made from innovative "new" materials deemed to solve all structural problems. Not only do such projects share a lack of confidence in traditional materials, construction techniques, and skills, but they frequently neglect one of the fundamental steps of structural renovation project management, the diagnostic phase. The historical knowledge of the construction is fundamental for any intervention.

This paper has presented some indications taken from the works of treatise writers. It illustrates how applying traditional rules of workmanship can attain the aims of preservation and structural efficiency with overall cost-effectiveness in recent restoration works. These intervention technologies could be defined as sustainable, with the use of simple steel elements, inspired by the construction rationality proposed in the middle of the 19th century by Polonceau: «[…] any construction system is required to meet the dual conditions of duration and economy, or, in other words, all materials used in a building system must be placed under conditions of resistance so that one can give them the smallest possible dimensions, and that their assembly should be of the greatest simplicity […]» [13].

Introducing these criteria in new projects can increase the quality of the interventions compared with those achieved using other technologies that still leave some doubts about their durability and reliability. In the final balance, the successful outcome of the intervention on the timber roof structures of the church of Salbertrand is the result of mindful design proposals that reflect the close link between the diagnostic phase before the final project and the investigations that preceded the executive project. The possibility of assessing the overall good state of conservation of these reinforcement interventions nowadays, some twenty years after their undertaking, could constitute an essential reference for future interventions. The durability and reliability of this and other reinforcement techniques presenting innovative features need to be evaluated through surveys conducted on the broadest possible statistical sample [37, 38]. These surveys allow us to better address and plan reinforcement and to study the actions required to prevent the medium- to long-term adverse effects of these interventions.

In conclusion, this research project confirms the usefulness of learning from tradition in the multi-disciplinary activity that involves a vast number of various professional figures, such as architects, civil engineers, wood technologists, and dendrochronologists, with the supervision of national boards responsible for the safeguarding of structural interventions on buildings belonging to our historical-environmental-cultural heritage.

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WIDE-SPAN TIMBER TRUSSES IN THE AREA OF BOLOGNA: A CASE STUDY ANALYSIS AND COMPARISON

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Abstract

This paper summarizes a larger research project that used an innovative method to study several timber roof structures in Bologna. The research was focused on developing a support tool for analyzing the geometry and the structural behavior of these structural systems, utilizing a Terrestrial Laser Scanner (TLS) and point cloud geometric information via visual programming generative algorithms. In addition, the method has been developed to collect and trace data on various types of timber trusses and function as an information system.

Specialized literature frequently oversimplifies the comprehension of these structural systems by basing its theories on structural analysis methods that originated in the nineteenth century. This approach typically needs a thorough understanding of material properties and structure deformations, which cannot be easily obtained.

Innovative methods of research, as well as typological construction investigations, can help gain a thorough understanding of these objects. Such knowledge is essential for the conscious conservation of these amazing construction systems. This paper compares five types of timber trusses from the roofs of noticeable buildings from the 17th and 18th centuries, associating their typological and construction characteristics with the geometric and deformative information from previous research. The study provides a deeper understanding of these objects in the Italian context by highlighting some critical issues.

Keywords

Historical timber trusses, Structural efficiency, Terrestrial Laser Scanning, Parametric modeling algorithms, Displacement analysis.

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1. INTRODUCTION

The difficulties in accessing and moving inside old wooden roofs and the lack of visibility of trusses, frequently hidden by masonry vaults, have reduced them to minor issues in the fields of Conservation and Restoration of Cultural Heritage. Roofing systems are often ignored until severe deterioration or damage threatens the building's safety. Few studies correctly understand the actual behavior of large-span trusses and their evolution over time. Furthermore, the specialized literature frequently oversimplifies their structural conception, basing their theories primarily on methods of investigation and calculation associated with the traditional idea of elastic structures. At the same time, the contemporary approach is overly specialized or dictated by the need to solve accidental or local problems. Interventions often overlook factors of primary importance, such as the transformations of these building systems over time and the actual behavior of joints, generally modeled as pure hinges. These factors complicate the numerical quantification of the actual behavior of these complex structures and put traditional calculation models at stake.

The configuration of the historical trusses belongs to the Art of Construction rather than the Science of Construction [1]. The trusses' strong intertwining of history, architectural technology, and material culture suggests that their conception is not easily referable to simple structural schemes unless at the cost of high approximations. Timber trusses are mostly statically indeterminate structures, and their structural safety is linked to good execution, junctions and connections implementation, material quality, and long-term preservation. Deeply understanding the current state of a truss means going back in time to see how this structure evolved throughout its service life.

Since the beginning, this multi-year research experience has proposed to overcome the principle of classification and schematization, which is typical of the 19th-century culture of Construction Science. Therefore, the "epistemological" approach has been favored over the "technicist" approach for knowledge, interpretation, and analysis of structural behavior [2]. It entails the use of an innovative and non-invasive method of investigation based on digital Terrestrial Laser Scanner (TLS) surveys, integrated with the collection of other data, such as historical and archival research, in-situ surveys, reverse engineering procedures, parametric modeling algorithms, and data science techniques.

An investigation method for deepening knowledge of these construction elements has been developed by approaching this issue differently. The analyses enabled the systematization of a procedure for highlighting some displacements that timber trusses commonly experience during their life cycle in terms of rigid body motions and deformations [3, 4]. This three-step method has been applied to various case studies and gradually updated and implemented to consider any type of timber truss. The first step is to acquire data on-site using TLS survey techniques. In the second step, the method enables a fast and reliable virtual representation of the trusses using 3D parametric modeling and Building Information Modeling (BIM) software [5, 6] due to the consistency of digital information. The third step allows different types of information to be associated with the 3D digital models. These models can be linked to archival documentation, monitoring data, and information on intervention hypotheses for the conservation and functional recovery of these examples of historical construction.

In this paper, five case studies are briefly introduced, adding references to more comprehensive studies that cannot fit this short essay. Then, the geometric characteristics of each truss type are compared to the theoretical schematizations of a popular manual from the second half of the nineteenth century [7, 8]. The goal is to link pre-nineteenth-century building practice and emerging nineteenth-century technical-scientific knowledge. Finally, the application of the method provides an interpretation of the displacements these systems have undergone over time, highlighting any recurrence or typical behavior. The objective is to prepare the ground for a more in-depth and structured understanding of the Bolognese trusses built between the seventeenth and eighteenth centuries and pave the way for the same method of investigation to be applied to other national and international case studies.

2. THE CASE STUDIES

In the mid-sixteenth century, the city of Bologna, which was at the apex of its political and cultural importance in the Pontifical State, had nothing to envy other important Italian and European cities. It was true for the splendor and uniqueness of its built heritage, as well as the dynamics of its economic and social structure's development. The constant presence of scholars granted by the University, which always drew prominent figures from all over Europe, and the close relationship with the Papacy acted as driving forces for the edification activity. The city's fabric was gradually enriched with significant buildings to meet the demands dictated by the social status of influential families and the powerful Curia. It is no coincidence that in 1575 Pope Gregory XIII, the Bolognese Ugo Boncompagni, commissioned a fresco of his hometown for his private apartments at the Vatican

Palaces, depicting all of the most important religious and theatrical buildings, in addition to the noble palaces.

The need to build roofs for these numerous large-scale halls has undoubtedly favored the development of trussed roof systems. Indeed, many advantages were provided by these roof structures, which differed significantly from North-European wooden roofing systems with steeper slopes since they enabled the construction of relatively light non-thrusting roofs while also overcoming large spans. Even though obvious variations of the base type emerged over the years, it is immediately possible to say that construction practices and production processes can be considered relatively stable during this period. However, it should be noted that, in many cases, existing trusses are in a different state than their original configuration, having been subjected to consolidation interventions or changes in the use of the spaces beneath [9].

The trussed roofs on which the method has been fully applied are listed below in descending order of complexity and span covered:

- the Teatro Comunale of Bologna (TEA) was built in the late 17th century. It was initially designed by the architect Antonio Galli Bibiena [10] and completed by the architect Lorenzo Capponi in 1763. The 5 timber trusses, with more than 25 meters span, were erected right after the construction of the great eighteenth-century hall [11]. The municipal architect Giuseppe Tubertini addressed severe deterioration issues that had arisen in the timber trusses in the early nineteenth century, despite the building's young age [12]. Other strengthening interventions on the roof structure occurred around 1980 [13] when the great hall's wooden ceiling was hung with tie rods to the reinforced trusses [14];
- the Cathedral of San Pietro (PIE) was rebuilt in the mid-12th century after being destroyed by fire [15]. Since then, the church has undergone several changes. In the early sixteenth century, architect Giovanni Ambrogio Magenta took on the construction site, which was later modified and continued by master carpenter Niccolò Donati. The construction was completed in 1748 by the architect Alfonso Torregiani who erected the façade [16]. Among the eighteen trusses of the central nave, which span more than 25 meters, sixteen of them date from the early 17th century, while the other two were built after the mid-18th century, at the same time as the new façade;
- the Basilica of San Petronio (PET) began construction in 1390 and was built intermittently over the following centuries. Due to a lack of funds, famine, and political changes, the church was built in one or more bays at a time until the mid-1600s [17], when the central nave was raised and the roof was made. Girolamo Rainaldi designed the timber trusses according to his expertise dating back to 1625, in which he proposed modifications to the original project of Francesco Terribilia. Today, the roof over the nave is made up of 35 original trusses that span just under 19 meters, built between 1646 and 1658, plus 7 trusses that were replaced in 1905;
- the Church of the Santissimo Salvatore (SAL), of late-medieval origin, was rebuilt in the fifteenth century and then, between 1600 and 1623, took on the current form of a baroque church. The design can be attributed to Giovanni Ambrogio Magenta, who was active in Bologna during that period. The project was inspired by the Basilica of Massenzio, according to Magenta's writings, and later modified by the architects Onorio Longhi and Carlo Maderno. The trusses of the central nave, with a span of about 18 meters, date from this period. In 1980, a restoration intervention involving the entire church was carried out, replacing the most degraded timber elements of the trusses and renovating the upper enclosure;
- the Basilica of San Domenico (DOM) is the patriarchal church of the Dominican order, and it was commissioned in the mid-13th century [18]. Between 1298 and 1654, the church was renovated several times. Then, the architect Carlo Francesco Dotti designed and coordinated various structural interventions around 1730, including the roof replacement, which was raised to match the new façade. The existing timber trusses, which span 17,5 meters, date from that period and are almost entirely original, particularly those above the choir. Other restoration work was done on the building in the 1900s.

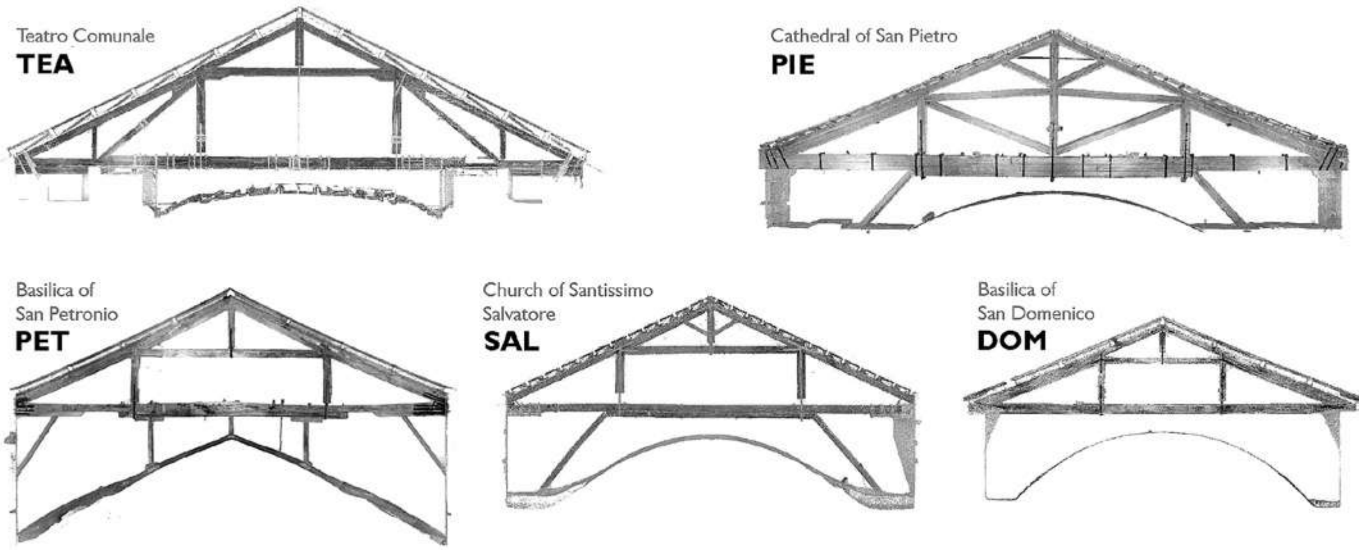

Fig. 1. *Orthophotos of the analyzed R-trusses extracted from the TLS digital survey.*

To make a transversal comparison between the case studies, it was decided to select a Reference-truss (hereafter named R-truss) for each building. The R-truss can be considered the one that best represents the series of trusses that form the entire roof of each building in terms of regularity, deformation state, and originality (Fig. 1). The use of an R-truss is acceptable because all the trusses in each building are very similar, differing only in a few elements due to construction inaccuracies or local substitutions. The 5 selected R-trusses were analyzed with the latest version of the developed method to compare the results [5].

Tab. 1 summarizes some of the major time evolutions of the analyzed trusses. PIE owns the oldest trusses, which date from the early 17th century. Following these are the PET (mid-17th century), DOM (early 18th century), and SAL trusses (early 18th century). The most recent trusses are from TEA and date from the mid-18th century. The comparison of the ages of each truss and the corresponding building reveals that the trusses were only built concurrently in the case of TEA. In all other cases, the truss framings in place today could have been made years, if not decades, after the walls beneath. After the realignment of the ridge to the new façade, built by Dotti in 1730, DOM, for example, had the entire roof rebuilt. Other interventions on PIE and PET have also been documented and can be considered somehow historicized. According to the information gathered, these roof configurations can be thus regarded as homogeneous.

On the other hand, the SAL and TEA trusses underwent significant consolidation work at the end of the twentieth century. In the first case, several eighteenth-century beams were replaced. In the second case, a metal tie rod was added to work alongside the wooden one, and metal straps were added at some joints. As a result, it is critical to recognize these changes to avoid a superficial interpretation of the numerical data obtained through the digital analysis method.

	Teatro Comunale (TEA)	Cathedral of San Pietro (PIE)	Basilica of San Petronio (PET)	Church of San Salvatore (SAL)	Basilica of San Domenico (DOM)
Construction phases					
Masonry structure	Mid-18th century	13th to late 18th century	Late 14th to mid-17th century.	Beginning of 17th century	From mid-13th to 18th century
Timber trusses	5 Mid-18th century	16 early 17th century; 2 mid-18th century	35 mid-17th century; 7 early 20th century	11 early 17th century	11 early 18th century
Deep renovations and strengthening	1818-1820; 1980-1981	-	-	1980	-

Tab. 1. *Historical background of the analyzed trussed roofs.*

3. IDENTIFICATION OF R-TRUSSES

The trusses are analyzed on two scales: a general scale related to the scheme of the R-trusses and a detailed one related to the joints. Accurate observation and an extensive photographic campaign in situ are essential to achieving this. Nonetheless, the TLS survey proved critical for extracting orthophotos and obtaining proper graphic restitution (Fig. 1).

Comparing different R-trusses allows for the assignment of specific typological characteristics to the deformation properties of the entire roof system, leading to a better understanding of each R-truss behavior and clarifying the role of each timber beam within the construction system.

Smaller span R-trusses (PET, SAL, and DOM), ranging from 17 to 19 meters, show only minor differences and can be traced back to the recurring Italian type known as Queen post truss or *palladiana*. This truss scheme has two principal rafters, two queen posts, one king post, a straining beam, and a tie beam [9]. When assembled with spliced or scarf joints in the longer members (typically tie beams), queen post trusses can cover a 20-meters span.

The two larger R-trusses (TEA and PIE) represent more complex schemes, almost unique, even though they can be traced back to the concept of compound trusses with straining beams [19]. The strutting arrangements are different. In PIE, the straining beam is divided into two inclined elements connected by a king post; in this case, the straining beam becomes a sort of discharging arch. In TEA, the straining beam is connected to the tie beam by compressed struts and two additional lateral posts. These latter are also compressed, working as props that transfer the load acting on the truss to the structures of the theatre hall.

Tab. 2 summarizes the typological and geometric properties of the analyzed R-trusses, which are discussed further in the following paragraphs.

3.1. FRAMING SCHEMES

In terms of framing, each analyzed case has unique characteristics due to the shape of the bearing structures beneath and the span covered. In general, all trusses are of the queen post truss type, with minor variations in the number and size of beams, supports, and joints. This geometric scheme consists of a lower trapezoidal shape and an upper triangular shape. These two parts seem to correspond to two successive stages of assembly: the tie beam, rafters, and straining beam are erected on-site first, followed by the completion of the triangular upper part. As partial evidence, it can be considered that principal rafters are always doubled in cross-section, except in SAL, by using a second timber element to strengthen the rafter/straining beam joint. This second overlaying rafter is usually continuous near the joint, while the first one, due to construction needs, is interrupted by the queen post.

The presence or absence of additional beams produces different alternatives due to how the roof structures are connected to the masonry elevation walls and the span length. This standard scheme is used in case studies with a shorter span (PET, SAL, DOM) (Fig. 2). While considering the tie beams, three variations can be found: the tie beam resting on the vault via masonry supports (PET), the tie beam loading the sidewalls via structural timber brackets (SAL), and the tie beam resting on masonry abutment supports (PET) (DOM). These alternatives suggest a process of refinement in construction over time and possibly reflect the availability of materials and the technical skills of the builders at that moment. For example, PET construction has suffered from significant financial difficulties, resulting in less care in the works.

The other two case studies have non-standard characteristics and do not fit into a typical typological scheme. The PIE truss has undoubtedly piqued the public's interest with the widest span of all the analyzed examples. This truss has an additional discharging arch system; the king post divides the straining beam into two parts, and the elements are slightly inclined towards the queen posts. This choice expresses the architect's intention to streamline the function of these compressed elements by resorting to a solution relatively uncommon. The TEA case is also unusual, with lower sub-struts that allow loads to be discharged on two intermediate supports that connect to the tie beam. As a result, the Bolognese case is rich with variations and a fertile field for investigation.

	Teatro Comunale (TEA)	Cathedral of San Pietro (PIE)	Basilica of San Petronio (PET)	Church of San Salvatore (SAL)	Basilica of San Domenico (DOM)
General strutting					
Classification	Queen post truss with additional posts resting on 4 external supports	Queen post truss with internal discharging arch and double struts	Queen post truss	Queen post truss with struts	Queen post truss
Beams number	13	14	9	11	7
Joints number	9	9	5	7	5
Beams assembly					
Tie beam	Assembled	Assembled	Assembled	Assembled	Continuous
Straining beam	Continuous	Continuous	Continuous	Continuous	Continuous
Principal rafters	Interrupted	Interrupted	Interrupted	Interrupted	Interrupted
Stiffening rafters	No	Yes	No	No	Yes
Joints and notches					
Ridge/rafters	Head of king post	Head of king post	Head of king post	Head of king post	Head of king post
Rafter/straining beam	Head of queen post with wood corbels	Head of queen post with struts and stiffening rafter	Head of queen post	Head of queen post	Head of queen post with stiffening rafter
Tie beam/rafter	Double step	Double step	Single step	Single step	Single step
Metalwork					
Ridge/rafters straps	No	No	No	Added in 1980	Added (undated)
Rafter/straining beam straps	Yes	No	No	Added in 1980	Added (undated)
Tie beam/rafter straps	Yes; Restored in 1980	Yes	Yes	Added in 1980	Added (undated)
Tie beam straps/ bolts	Yes; Restored in 1980	Yes	Yes	No	No
Queen post stirrups	Yes; Restored in 1980	Yes	Yes	Restored in 1980	Restored (undated)
External Supports					
Sidewalls support number	2	2	2	2	2
Other supports number	2	2	5	2	0
Total supports	4	4	7	4	2
Sidewalls support type	Masonry	Masonry	Masonry and timber brackets	Masonry and wood corbel	Masonry corbel
Other supports type	Masonry	Three-voussoirs timber arch	Wood props on the vault	Three-voussoirs timber arch	-
Roofing and vault					
Roofing type	Masonry tiles	Wood planking	Masonry tiles	Wood planking	Masonry tiles
Vault type	Hanging wooden ceiling	Barrel vault	Groin vault	Sail vault	Sail vault

Tab. 2. *Typological characteristics of the R-trusses analyzed.*

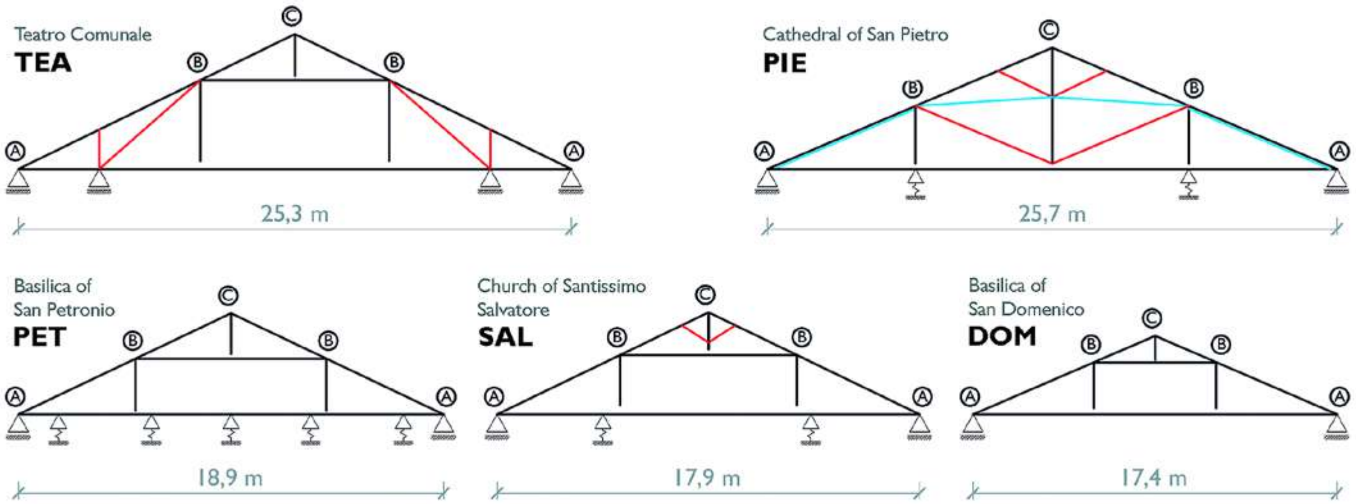

Fig. 2. *Framing schemes of the analyzed R-trusses. In black, the elements referable to the scheme of the standard queen post truss; in red, the additional struts; in cyan internal discharging arch elements. The line thicknesses do not represent the proportioned real cross-sections of the elements.*

3.2. ASSEMBLING PROCESS

Concerning the timber used to realize the truss elements, it is well known that one of the most critical points in their assembly has consistently been material availability. It was nearly impossible to find sufficiently long trunks to produce single continuous beams of eighteen meters or more, and the materials (spruce in almost all of these cases) had to be transported from long distances to Bologna. Besides, assembled beams of greater length also had thicker cross-sections. Assembled beams are made up of several timber elements connected by metal strap junctions. These joined elements become sufficiently rigid and flexural stress-resistant [19–23]. Tie beams are assembled by double-length or triple-length elements, being the longest members in a timber truss. The cross-section of the tie beams is nearly doubled at the midpoint (PIE, PET, SAL). The 26-meter-long PIE trusses have a tie beam cross-section of 30 cm x 90 cm formed by connecting three elements. In contrast, the DOM tie beam is the only example of a single-length element tie beam among the analyzed case studies.

The tie beams longer than 19 m in the examples analyzed are all assembled, except those of DOM, the most recent ones, which are made with a single timber piece. It should be noted that during an intervention in 1980, the SAL tie beams were replaced or consolidated. The solution for the assembled tie beam of the PET truss, made of three different pieces scattered but overlapping in the middle of the span, is quite fascinating: fearing the excessive bending of the assembled tie beam or directly observing a considerable displacement, the builders decided to support the elements with masonry pillars, loading the vault below and affecting the vault's thrust line. Conversely, the straining beams are all made in a single piece and have a maximum length of 11 m, as are the struts, posts, and other minor elements.

The rafters show significant construction variability because of the interference with the queen posts and the straining beam. Builders used a variety of solutions to create these junctions, which are subject to compression and flexural stresses. Except in the case of DOM, where the elements are continuous from the lateral support to the ridge, in each of the five case studies, the rafters are interrupted at the joint with the straining beam to allow the truss assembly in two stages. SAL is the simplest case, with rafters made up of only two beams separated by queen posts. In other cases, the rafters are surmounted by continuous beams, which can be considered as stiffening elements – stiffening rafters – installed at the end of construction.

3.3. JOINTS FEATURES

The analysis of joints is critical for understanding the system's behavior as well as tracing the structural evo-

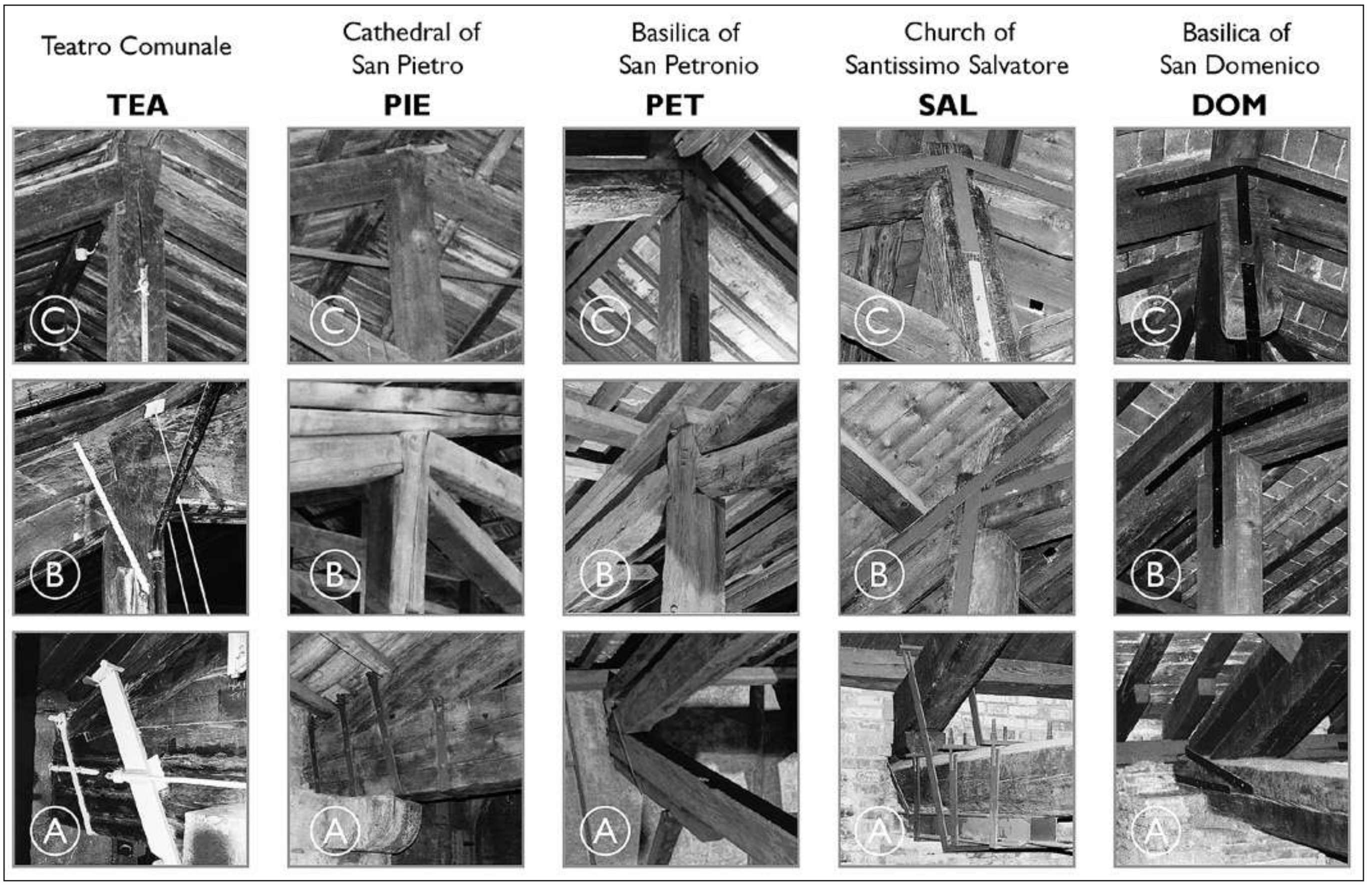

Fig. 3. *Pictures of the main structural joints of the analyzed R-trusses.*

lution of timber trusses over time (Fig. 3). Refinement and higher precision of the notches at the ends of the beams were achieved as practices and techniques continued to evolve during the period studied, particularly in DOM. In general, the complexity of junctions confirms that trusses belong to the Art of Workmanship rather than the Science of Construction. In the actual case studies, the joints, often reduced to mere internal hinges in modern structural schemes for simplicity, exhibit a difficult-to-describe behavior. It is directly understandable by evaluating the equilibrium between the elements convergent towards the joints themselves. On the other hand, the small rotations of the queen posts show a minimal transmission of the bending moment through the members.

For each of the five R-trusses, in Figure 3, the three more significant types of joints were collected: the ridge connection (C), the rafter-tie beam connection (A), and the joint between the rafter, the straining beam, and the queen post (B). This comparison is critical to understanding the different kinematics and activated deformations in each framing scheme.

All type C joints have very similar notch characteristics, and the timber elements are let into each other with minimal variation in all case studies. Consistent metal strapping is present in these joints, but in the case of TEA, SAL, and DOM, they have been clearly integrated or modified recently. On the other hand, joint B differs from case to case, and its configuration varies depending on whether the rafter is interrupted or in single-length. For example, in PIE and DOM, the presence of the stiffening rafter increases the rotational rigidity in joint B. The most straightforward B joint is found where the top rafter is interrupted, for example, in both PET and SAL; in these cases, the truss sizes are similar, and the strutting layouts are less rigid. The B joints in SAL and DOM cases have been stiffened recently with metal straps to counteract their rotation and the consequent kinematics of the whole structure. A unicum is the R-truss of TEA, in which the lower sub-rafters, more likely acting as struts, are let into the B joint of the queen posts and side blocked by short lateral posts that rest on secondary supports on the inside walls of the great hall underneath.

The notching between the principal rafter and the tie beam is fundamental to ensuring the whole system works. The outwards thrusts from the rafters need to be correctly transferred to the tie beam, thus avoiding horizontal push/pull actions on the walls below. In the case of R-trusses with single-length, principal rafters leaning onto bottom sub-rafters, the latter are the ones that are let into the notches on the tie beam. The variable shaping of the principal rafters affects the configurations in the tie beam scarfing, which can be “single step” or “double step” joints. PET presents the less complicated and refined A joint among those analyzed. More exemplary construction skill in wood carving is evident in the A joint of TEA, a symptom of a specific evolution of timber construction practices between the 17th and 18th centuries.

3.4. METALLIC CARPENTRY

Another critical piece of information for interpreting the behavior of the trusses is the analysis of the metallic carpentry (Fig. 3). Metallic straps allow a congruent deformation of the cross-section along the axis of the assembled elements, primarily the tie beams. Brackets, nails, wedges, and keys at the joints block the rotations of the beams in the truss plane, which could compromise the overall system’s stability. Bracing straps can also be used at the foot of the posts to ensure that the posts and tie beams stay on the same plane.

During the service life of a truss, metal components are probably the easiest to modify and integrate. The most recent strengthening interventions, especially when the truss timbers did not need to be substituted, have frequently focused on replacing or integrating metal plates in the joints to prevent movement. The interventions performed in the 1980s on TEA and SAL trusses and the undatable interventions performed on the DOM trusses clearly demonstrate this approach. On the contrary, the PET and PIE trusses have kept their original metallic carpentry, implying that the elements were more stable from the start and maintenance was accurate.

Metalwork is present in only two analyzed cases in correspondence with the C joint, and both are later additions. As a result, it can be assumed that truss construction in the seventeenth and eighteenth centuries did not generally foresee the presence of metallic elements in this junction. Original metallic parts are only found in the trusses of TEA for joint B, but they do not connect all the converging elements and are thus partially effective. Metallic plates of more recent origin are present in DOM and SAL but not in the others. In all cases, there are metallic straps at joint A, often dating back to the original construction of the trusses or added later.

Analyzing the assembled tie beams, the TEA R-truss contains a significant amount of metalwork, most of which dates to the 1980s intervention. Numerous metal straps on the tie beam of the PIE truss also appear to be original based on their craftsmanship. On the other hand, the “keyways” or connection bolts of the PET tie beam are relatively sparse and strikingly different from those of the other case studies. Tie beam-holding stirrups are present in all cases, mostly replacing the originals.

3.5. LATERAL SUPPORTS

The A joints, which serve as the lateral supports for the tie beams, are built directly into the brickwork elevation structures that are typical of the Bologna area. The span of the tie beams is reduced by supporting brick corbels or timber brackets. Steel hammerbeams were recently installed in the SAL trusses to reinforce the heads of the tie beam. Additional intermediary supports are used to limit deflection and improve its static effectiveness. These internal supports are implemented in various ways and are slightly yielding, depending on the technical solution used.

The R-trusses in PIE and SAL use a three-voussoir timber arch solution with rafters and a crossbeam. This contrivance suggests that the builders may have been similar – the construction period is comparable –; this timber arch under the tie beam was probably the most common practice to support the beams in naves with large spans and vaulted masonry ceilings. The TEA and PET intermediate supports, on the other hand, appear as situational solutions. In the first case, the intermediate supports are provided by small walls resting on the masonry structures of the bell-shaped plan of the theatre. In the second case, the tie beams are supported by various wooden props directly resting on the vault. There are no intermediate supports in the DOM truss, which has the shortest span.

4. GEOMETRIC FEATURES AND STRUCTURAL EFFICIENCY

The investigation continues by analyzing the geometric characteristics of the R-trusses. TLS surveying techniques are commonly used with reverse engineering methods to draw interpretations of the trusses' structural efficiency. The geometric information, organized in the form of an indexed point cloud, can be processed using parametric programming software to remodel the hypothetical original undeformed condition of the R-trusses at the time of construction [5]. The five R-trusses utilized as case studies were analyzed with the most recent version to ensure homogeneity in the results.

Tab. 3 illustrates the dimensions and geometric proportions of the analyzed trusses.

4.1. STRUTTING DIMENSIONS AND PROPORTIONS

The dimensional analysis of the timber elements provides interesting information when correlated to the geometric scheme of the trusses and the scantling of the members (Fig. 4).

In the analyzed buildings, the spans (L) vary from 17 m (DOM) to 26 m (PIE). The PIE trusses, still original, are believed to be the largest in Europe at the time of their construction. Table 3 and Figure 4 show that the schemes' complexity rises in the number of members and joints as the span increases. The slope of the pitches (P) is between 22° and 26°, typical values in the Bologna area and also very close to the graphical ones proposed by Valadier (≈24°) [8].

The distance between the axis of symmetry of the trusses and the queen posts (D) is the parameter that significantly differs from case to case. This length defines the proportions of the reference queen post truss scheme adopted. In TEA, D equals about 4 m, resulting in the top rafters being about half as long as the bottom ones. In PIE, having a similar span, D equals about 6 m, thus causing the rafters to be divided into two almost equal parts. The comparison reveals that

	Teatro Comunale (TEA)	Cathedral of San Pietro (PIE)	Basilica of San Petronio (PET)	Church of San Salvatore (SAL)	Basilica of San Domenico (DOM)
Layout dimensions					
Span (L) - m	25.30	25.70	18.90	17.90	17.40
Total Height (H) - m	6.30	5.60	4.40	4.70	3.60
Straining beam Height (D) - m	4.40	5.90	4.00	4.20	2.60
Queen post Distance (E) - m	4.00	3.00	2.40	2.50	2.20
Spacing (I) - m	4.10	3.20	2.50	2.90	2.90
Slope (P) - °	26.00	22.00	25.00	26.00	23.00
Layout Efficiency					
Truss Total Area - m^2	86.10	88.20	60.40	52.10	39.30
Void Area - m^2	58.40	47.20	33.80	33.70	20.80
Wood Area - m^2	27.70	41.00	26.60	18.40	18.50
Tributary Area - m^2	115.41	88.70	52.13	57.76	54.82
Layout Efficiency (Tributary Area/Wood Area)	4.17	2.16	1.96	3.14	2.96

Legend

min. … max.

Tab. 3. *Geometric data of the analyzed R-trusses. Higher values on each row are highlighted in red.*

Fig. 4. *Framework proportions of R-truss layouts. The red dotted line highlights Valadier's dimensioning procedure.*

in the two most recent R-trusses (TEA and DOM), the black shape in Figure 4 tends to be a square. This fact is ideally in line with the practical suggestions given by Valadier's manual [8, fig. 2, tav. LXVI]. By dividing half the tie beam into nine parts, the queen post should stand near the five- or six-ninths from the lateral support, that is, in other words, one-third from the centerline. In the case of TEA, there is a further lateral post and an additional strut element, which prefigures nineteenth-century truss schemes and hence the need to divide the truss into three parts. In the case of DOM, the greater length of the bottom rafter, two-thirds of the total, is compensated by increasing its flexural rigidity using two assembled timbers. In the reported examples of queen post trusses by Pizzagalli and Aluisetti [19], the queen posts divide the principal rafters into almost equal parts.

4.2. LAYOUT EFFICIENCY

The total amount of wood used in relation to the span and spacing of each truss can be a parameter to quickly assess these systems' degree of structural efficiency, keeping in mind that structural redundancy is, on the contrary, an indicator of robustness and durability. Starting from the layout of each R-truss in its vertical plane (Fig. 5), it is possible to calculate the frontal wood area (in gray) by subtracting the void area (in black) from the total area of the R-truss.

Calculating the ratio between the frontal wood area and the total area of the R-truss and not considering the spacing, TEA (0.32) seems to be the most efficient and PIE (0.46) and DOM (0.47) the most robust one. The other values range from 0.35 (SAL) to 0.44 (PET). The frontal area of trestles in PIE and SAL and the masonry intermediate supports in TEA and PET is not considered.

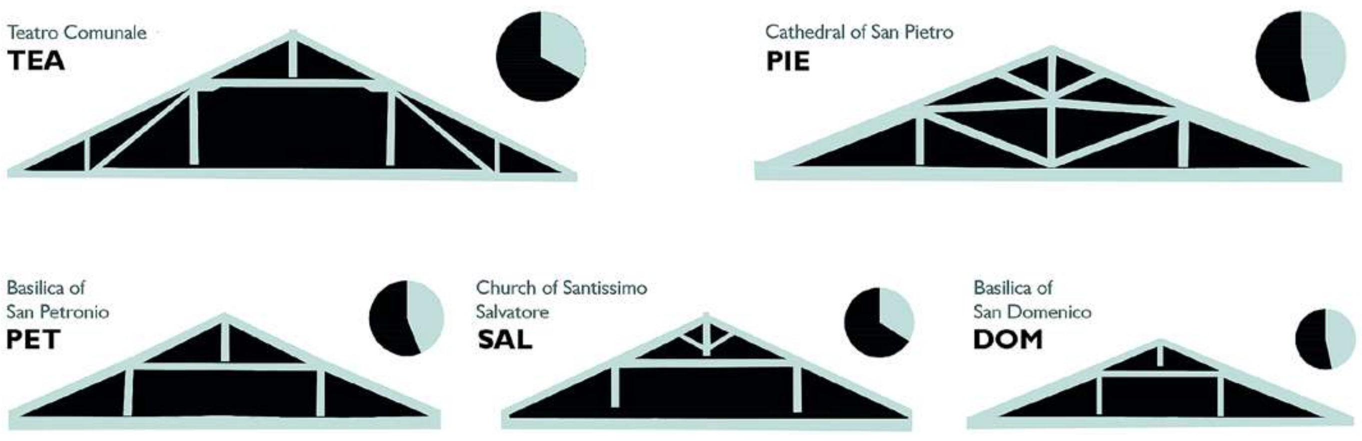

Fig. 5. *Wood and Void areas of the analyzed R-trusses.*

Fig. 6. *Layout efficiencies of the analyzed R-trusses.*

The layout efficiency is defined as the ratio between the tributary area and the wood area of each R-truss. The greater the value of this parameter, the lower the relative amount of material used in the whole roof and, as a result, the greater the level of structural optimization. Table 3 shows the layout efficiency values without considering the supporting gantries and the intermediate supports.

Despite the wide span, TEA has the most efficient framing system; DOM and SAL's classical queen post truss schemes are attested to intermediate values, while PIE and PET have low layout efficiency. The presence of masonry-made supports justifies TEA's efficiency. DOM has good results, regardless of the absence of wooden supports under the tie beam. Perhaps it represents the real optimal target at the end of the considered period. Its redundant static scheme against a remarkable span may justify PIE's low efficiency. PET's un-efficiency, instead, can be ascribed to the poor construction skills of the builders and its scattered construction process with poor materials, already mentioned during the presentation of case studies.

The graph in Figure 6 relates the wood area (x-axis) and the load area (y-axis). In this case, the layout efficiency is indicated by the slope of the line connecting the value plots of each R-truss with the origin. The greater the slope, the more efficient the strutting adopted.

4.3. COMPARISON WITH NINETEENTH-CENTURY MANUALS

The dimensional data of the beams refer to the main elements found in all the schemes examined: the tie beam, straining beam, and principal rafters.

Table 4 collects some dimensional data from point clouds surveyed using TLS techniques. The first three lines for each member show the total length (L), real base (Br), and real height (Hr). As expected, the measured values increase with the span covered by the R-trusses. Tie beams typically have a larger cross-section area than other members because they are assembled by multiple timbers.

A comparison was made between the values of the R-trusses and the theoretical dimensions proposed in nineteenth-century manuals. The goal was to match the results of the strutting practice of the seventeenth and eighteenth centuries and the outputs of the structural theories developed in the following century (Tab. 4).

As a reference text, the practical construction guide published in 1877 by Achille Lenti [7], a nineteenth-century scholar, was chosen. Lenti's observations on the scantling and cutting of timber elements highlighted a fixed relationship between cross-sections and the length of the truss members. These empirical formulas were deemed correct for designing the tie beams and the principal rafters of a classical truss with a king post and stiffening struts.

Lenti indicates the following practical ratios:

- Rafters: Hi/L = 0.048; Bi/L = 0.034; where (Hi) is the ideal height and (Bi) is the ideal base;
- Tie beams: Hi/L = 0.030; Bi/L = 0.020; where (Hi) is the ideal height and (Bi) is the ideal base.

Even though Lenti's formulas refer to the simple king post truss scheme, he seems to have considered in his writings the proportions suggested by Valadier in the volume *L'architettura pratica. Libro II* [8]. Analyzing Valadier's text (page 37), tables (page 16), and drawings (tav. LXVI), the proportions between cross-sections and length of the truss members are similar to those proposed by Lenti. The strutting for the queen post truss seems to be even more slender than the king post one. Furthermore, Lenti adds information about both the base and height of the cross-section, in this way helping to compare the dimensions of members and the span of R-trusses (Fig. 7). According to Lenti's proposal, the cross-sections of the tie beams, given the same lengths, are smaller than those of the rafters, which must withstand flexural and compression stresses.

	Teatro Comunale (TEA)	Cathedral of San Pietro (PIE)	Basilica of San Petronio (PET)	Church of San Salvatore (SAL)	Basilica of San Domenico (DOM)
Tie Beams					
Span (L) - cm	25,300	25,700	18,900	17,900	17,400
Real Base (Br) - cm	34	30	26	34	39
Real Height (Hr) - cm	55	62	53	37	44
Br/L	0.013	0.012	0.014	0.019	0.022
Hr/L	0.022	0.024	0.028	0.021	0.025
Straining Beams					
Span (L) - cm	8,800	11,600	7,900	8,400	5,400
Real Base (Br) - cm	30	30	26	30	24
Real Height (Hr) - cm	36	30	29	23	25
Br/L	0.034	0.026	0.033	0.036	0.044
Hr/L	0.041	0.026	0.037	0.027	0.046
Top Principal Rafter					
Span (L) - cm	4,900	6,400	4,400	4,700	2,900
Real Base (Br) - cm	31	30	26	27	21
Real Height (Hr) - cm	35	30	24	29	23
Br/L	0.063	0.047	0.059	0.057	0.072
Hr/L	0.071	0.047	0.055	0.062	0.079
Bottom Principal Rafter					
Span (L) - cm	9,200	7,800	6,100	5,900	6,500
Real Base (Br) - cm	30	30	26	28	25
Real Height (Hr) - cm	34	61	29	34	50
Br/L	0.033	0.038	0.043	0.047	0.038
Hr/L	0.037	0.078	0.048	0.058	0.077

Tab. 4. *Real dimensions of the main members of the analyzed R-trusses. Br/L and Hr/L values for each R-truss.*

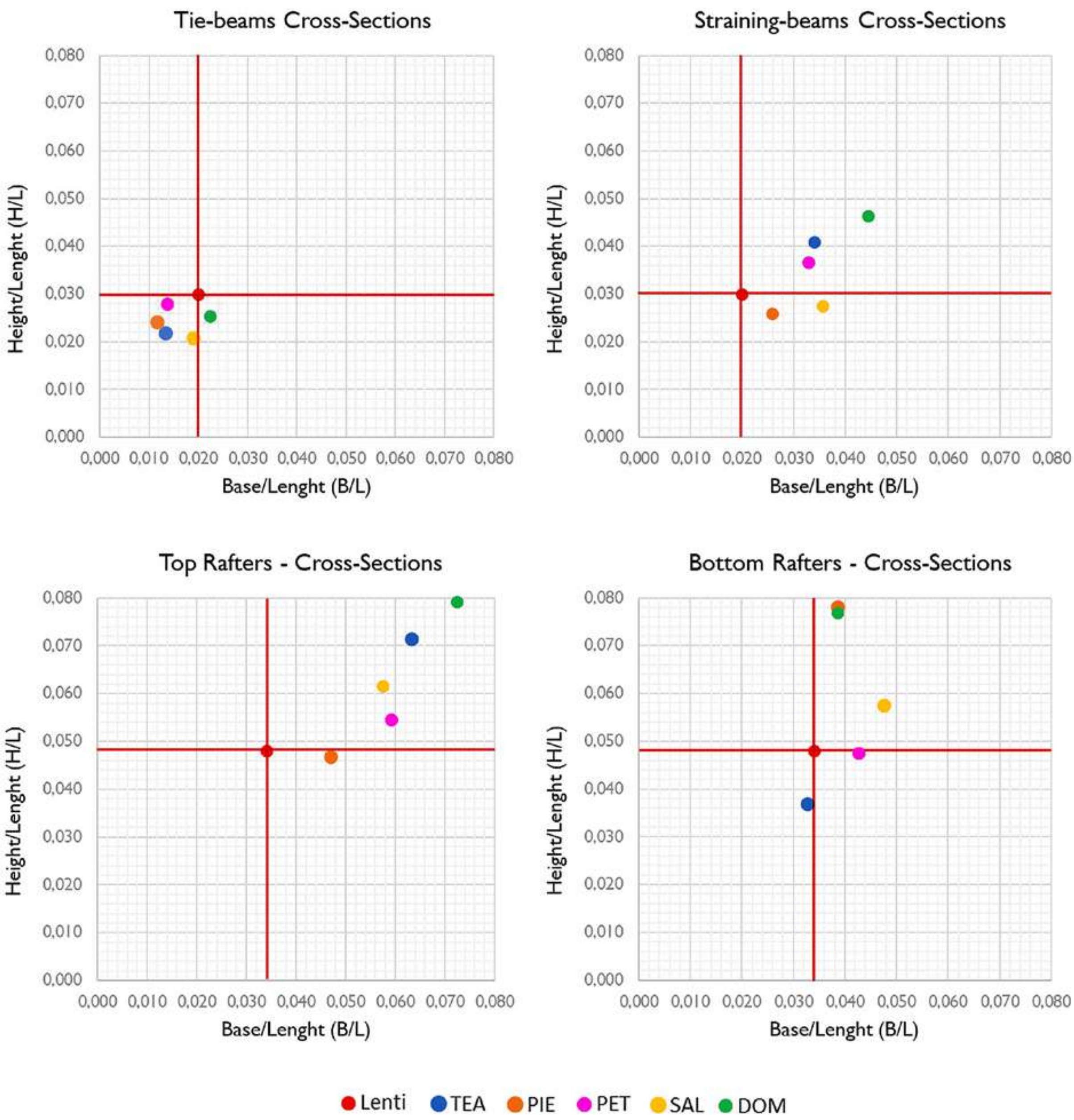

Fig. 7. *Measured cross-sections scantling with respect to Lenti's practical guide. Tie beam and rafter ratios for each R-truss.*

The graphs in Figure 7 show the Br/L and Hr/L ratios of the element sections belonging to the five trusses under study to compare them visually. The red points identify the dimensional ratios suggested by Lenti for the tie beams and rafters. In particular, the lengths and sizes of the top and bottom rafters were considered separately since all the truss layouts present interrupted rafters. The straining beams of the five R-trusses are also evaluated with the same formulas.

What emerges is that the tie beams are generally slightly undersized. Given the approximations used, this fact proves the function of the intermediate supports and the gantries in the case studies. The builders knew, even empirically, that the length of the tie beams, which were typically made in double or triple lengths, could be a weak point in the system. The straining beams are generally oversized, especially concerning the base dimension. A greater cross-section is probably due to construction opportunities, given the sizes of the other transported elements.

Since the top rafters are always shorter than the bottom ones, they are generally oversized, given the same considerations drawn for the straining beams. Even the bottom rafters are generally oversized, although closer to Lenti's proportions. TEA bottom rafter results as an anomaly in the values because they are significantly longer than in all the other case studies and are supported by detached sub-struts. This data comparison backs up what has already been said about the layout efficiency in the analyzed cases. The particular slenderness of the elements of the TEA R-truss is due to the introduction of new elements in the design, which helped meet the requirements of an economical construction [11], as well as to the presence of the intermediate props, which allowed for more freedom in strutting.

5. DISPLACEMENT ANALYSIS

TLS surveying techniques and truss modeling helped carry out a displacement analysis on a geometric basis. The analysis method is based on identifying the most likely kinematics and displacements to which the actual timber truss may be subjected under vertical loads.

Starting from the point cloud, the truss framing is transformed into a wireframe model of linear elements through an appropriate algorithm and then brought back to an ideal undeformed condition. The standpoint is always the truss's hypothetical initial undeformed condition compared to the actual state. In particular, the main displacements considered are those in the truss's vertical plane and the orthogonal direction (out of the plane). The displacements are measured in correspondence to the joints and at the center of the tie beams. In the truss plane, the basic assumptions are that the ridge joint (C) may sag, the queen posts joints (B) may lower and/or rotate towards the inside of the truss (following the rotation of the principal rafters and favoring the rotation of the queen posts), while the lateral wall supports (A) remain fixed. The downward tie beam deflection is also considered.

Figure 8 depicts the displacements and deformations in the vertical plane of the highlighted R-trusses on a magnified scale, with respect to the hypothetical perfect linearity of rafters and tie beams in the absence of any load, neither permanent nor transient (ideal undeformed situation). No dangerous situations have been encountered. The assumed kinematics and displacements are respected in all cases, with all R-trusses exhibiting approximately symmetrical behavior. It has always been possible to attribute some punctual differences or deviations to local factors, such as asymmetries, which have had a decisive influence on that non-compliant behavior. Furthermore, it can be stated that observing a difference frequently serves as an alert signal to highlight local problems.

A common outcome is that C joints have smaller displacements than B joints. The R-trusses that have undergone interventions during their life cycle (TEA and SAL) exhibit sagging values that are not as high as those left as originals. Along with minor sagging, there is evidence of a rotation inwards of the posts in TEA and DOM trusses, going in the opposite direction than the others, and this is not linked to the deformation of the straining beam but to the position of the queen posts at one-third of the rafter length.

Tab. 5 shows the sagging values (∂) of a few control points (joint C, joint B, and tie beam midpoint T), as well as the sagging/length ratio.

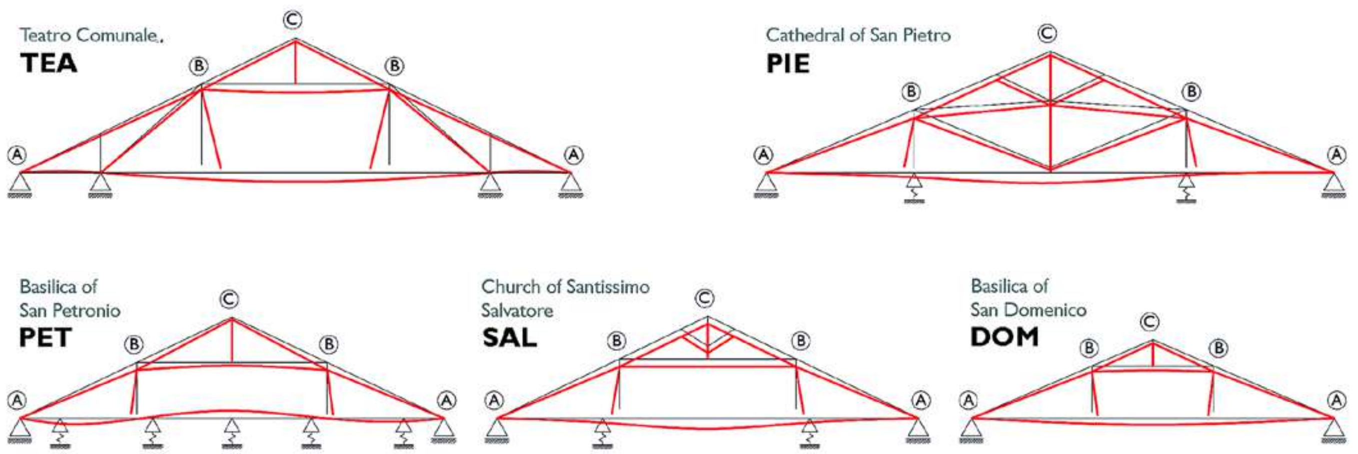

Fig. 8. *Qualitative deformation kinematics of the analyzed R-trusses.*

	Teatro Comunale (TEA)	Cathedral of San Pietro (PIE)	Basilica of San Petronio (PET)	Church of San Salvatore (SAL)	Basilica of San Domenico (DOM)
Control points displacements					
Vertical displacement joint C (∂_C) - cm	1 cm	1 cm	3 cm	2 cm	1 cm
Vertical displacement joint B (∂_B) - cm	2 cm	7 cm	6 cm	5 cm	5 cm
Span to vertical sagging ratio (∂_B/L) - %	0.07%	0.27%	0.47%	0.28%	0.29%
Queen post rotation (Ω_B) - °	0.57° (inward)	0.17° (outward)	1.48° (outward)	0.46° (outward)	0.47° (inward)
Tie Beams Deformations					
Midpoint vertical sagging (∂_T) - cm	4 cm	7 cm	-12 cm	11 cm	4 cm
Span to midpoint sagging ratio (∂_T/L) - %	0.16%	0.26%	0.61%	0.63%	0.23%

Tab. 5. *Deformations and main displacements of the analyzed R-trusses.*

In order to grasp more relevant information, some of the values in Tab. 5 have been correlated to the span of each R-truss. Figure 9 shows both the tie beam and the rafter sagging as a function of the span. The red dotted line highlights the average values, meaning that points above the line stand for higher sagging while points under the line stand for minor sagging. The rafters' behavior is somehow in line with the increase in the span. The wider the span, the higher the sagging, except for the case of the TEA R-truss, which was restored in 1981. In general, the tie beams' behavior seems to vary independently from the span.

The TEA R-truss is the one that supports the heaviest loads of all the case studies; it spans approximately 25 m and has a spacing of approximately 4 m. The TEA R-truss also has higher layout efficiency and members with high slenderness. According to the traditional approach, these factors would imply poor truss behavior in capacity/demand satisfaction in the strength and stiffness of the various members. However, its displacements are pretty small. In that regard, some determining factors must be considered. Firstly, there are more elements and external supports; in fact, the TEA R-truss is the most recent (mid-18th century), and the static layout has inevitably evolved compared to older R-trusses. Secondly, it should also be noted that the significant strengthening interventions increased its efficiency and reduced its deformations. The eighteenth-century truss of Milan's Scala Theater, which had a very similar configuration, was built in the same period and was deemed extremely slender and criticized at the time.

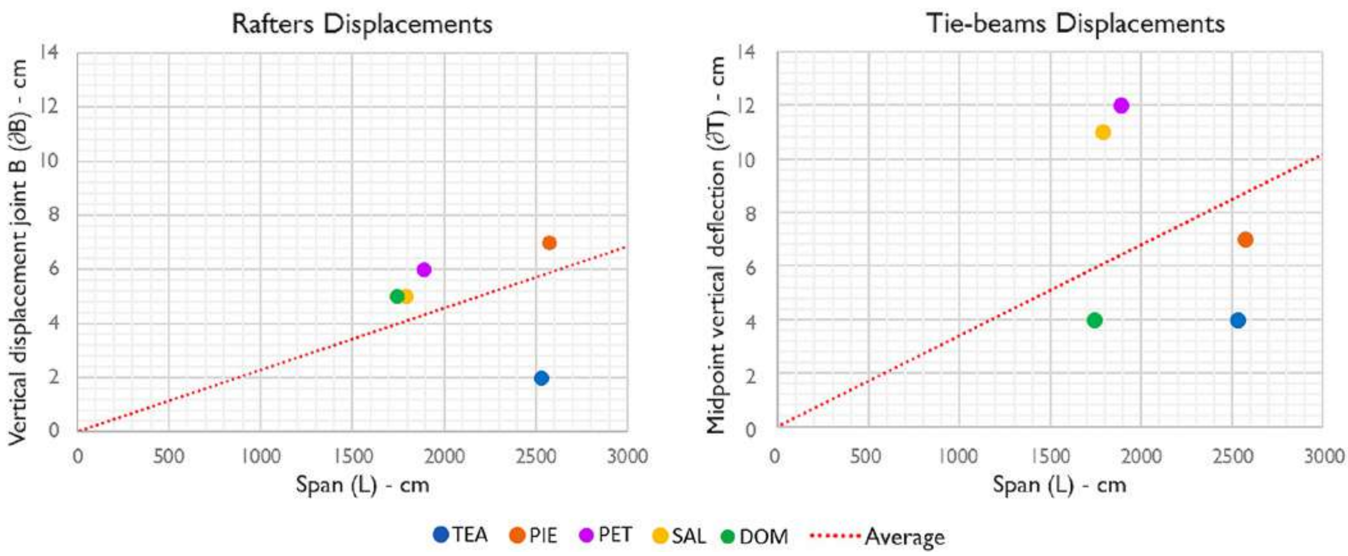

Fig. 9. *Displacements' analysis of joint B and tie beam midpoint for the R-trusses.*

The oldest and most complex PIE R-truss has demonstrated good behavior without significant interventions. Despite its large size, the displacements it has experienced are relatively lower than those of the other case studies, indicating particular robustness derived from the execution and design concept. The presence of an intermediate discharging arch, a continuous king post, and large diagonal struts demonstrate the overall rigidity of the static scheme. Even the construction methods identified in the historical documentation attest to a very high level of design care and significant financial commitment, which is supported by the accounting records [24].

The PET R-truss has a much smaller span than the TEA and PIE trusses and a smaller spacing and tributary area than the other case studies. As a result, it also has the lowest level of stress. It has an average void/solid ratio and the simplest pattern among the various R-trusses. Its origins are older than PIE and SAL, dating back to the mid-17th century. The builders' confidence appears to be smaller than in the other case studies. Using timber brackets to shorten the span and intermediate props on the vault to support the tie beam confirms that the builders relied primarily on masonry walls. Straight cuts are also used to notch and assemble the double-length tie beams without precise scarfing. This incorrect solution tends to absorb tensile stresses less effectively, resulting in excellent system deformability, mainly when dealing with horizontal loads.

The SAL and DOM R-trusses are substantially in line with expectations, presenting a standard framing typical of many other Italian wooden roofs and demonstrating a good stiffness of the principal rafters. Above all, the DOM R-truss is the only one with a single-length tie beam. As a result, its behavior is more understandable, with reduced flexures of the rafters and tie beams and a slight inward rotation of the queen posts.

The findings show that each case has construction quirks and an evolutionary process that must be thoroughly examined. In general, variations in the realization of the static scheme (presence of timber brackets or corbels, position and arrangement of the joints, intermediate supports such as masonry props on the vault or three-voussoirs timber arches, principal rafters length, and scantling) are thought to have a more significant effect than dimensional variations of the members. The use of rapid and non-destructive digital techniques provides a real opportunity to improve the understanding of timber trusses in a relatively short time.

6. CONCLUSION

The paper presents a few findings from a study of a few timber trusses in the Bologna area located in historically significant buildings between the 17th and 18th centuries. This study is based on a novel approach that employs TLS digital survey technologies and parametric modeling algorithms to obtain geometric and spatial data that would be impossible to achieve using traditional methods. Furthermore, this approach allows for the systematization and tracking of information gathered through other types of investigation (documentary, instrumental, and photographic), establishing itself as a sort of information system for analyzing and interpreting results.

When applied to the five case studies, the method identified shared and unique characteristics of the R-trusses used in each wooden roof under consideration. The archival research enabled the collection of important historical data from all case studies. The typological analysis confirmed how each case presents itself as a variation of the classical queen post truss, demonstrating that each construction reveals its individuality, which is primarily determined by the workers' knowledge and economic constraints. The geometric analysis enabled a sharp and precise assessment of the dimensional and proportional differences between the roofing systems used in the various study buildings. The comparison of the dimensions of the timbers surveyed and the theoretical ones proposed in the first engineering manuals of the nineteenth century revealed significant deviations, confirming how member sizing is more related to builders' experience than scientific theorization. Finally, displacement analysis has allowed estimating the significant deformations that these systems have experienced over time, which has aided in understanding their behavior. This final analysis, which extensively uses generative algorithms, is based on some hypotheses about truss deformation kinematics. These kinematic mechanisms were identified first from a theoretical standpoint and then from experience, and they were confirmed after the elaborations.

The cross-analysis of the data collected across the different case studies allowed for a deeper understanding of the construction culture of the timber trusses and proved to be especially significant because it refers to study samples from a relatively short period (150 years) and a well-defined context. Traditional structural modeling, which simplifies geometric specifications, cannot always account for the types of displacements experienced by the trusses over their lifetime. This paper demonstrates how seemingly similar cases have undergone more or less significant displacements due to multiple factors, such as the characteristics of the construction joints, the assembling of the members, and the scantling of the cross-sections.

The many transformations applied over time, in particular, frequently alter the original conception. The ability to structure the information embedded in the method enables a back-office analysis. Previously, this analysis could only be done roughly during the survey campaign. It becomes critical during the diagnostic phase and allows for critical considerations about the health of these structures in terms of conservation, maintenance, and enhancement.

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