SHATIS'2019

5th International Conference on Structural Health Assessment of Timber Structures

25-27 September 2019, Guimarães, Portugal

Editors
Jorge M. Branco *University of Minho*
Hélder M. Sousa *University of Minho*
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Universidade do Minho
Escola de Engenharia

Proceedings of the International Conference on Structural Health Assessment of Timber Structures, SHATiS'2019

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SHATiS'19 - 5TH INTERNATIONAL CONFERENCE ON **STRUCTURAL HEALTH ASSESSMENT OF TIMBER STRUCTURES**

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Jorge M. Branco, Elisa Poletti and Hélder S. Sousa

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ISBN: 978-989-54496-2-0

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Electronic version, 2019

PREFACE

Timber structures are an important part of the architectural and cultural heritage. In fact, any action concerning the conservation, repair, retrofitting and monitoring of the built heritage cannot avoid understanding how timber structures behave, from the material level to whole structures, including their joints, to assess their present condition and to promote different strategies for interventions. In this context, SHATiS International Conferences on Structural Health Assessment of Timber Structures aim to ensure that the past know-how is not lost, promoting the best practice and encouraging that new developments are brought to light.

SHATiS'19 is not different and aims to present an overview on the main steps involved on the Structural Assessment of Timber Structures, from the diagnosis and assessment to the retrofit of timber elements, joints and structures. Original contributions regarding experimental research and numerical analysis from the academic field, as well as from practice are collected and presented.

A special acknowledgement to the authors for their contributions and enrolment to the success of SHATiS'19, the fifth edition of this unique international meeting dedicate to the assessment of built heritage in timber.

One last word to acknowledge the support of all the sponsors and support institutions that made possible this SHATiS'19, in Guimarães, the cradle of Portugal.

Jorge M. Branco
On behalf of the SHATiS'19 Organizing Committee
September 2019

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THE ANALYSIS OF THE TIMBER ROOFS OF THE LORETA IN PRAGUE

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Keywords: health assessment, historic timber structure, loading jack, non-destructive tests, dendrochronology

Abstract

The Loreto of Prague is a remarkable Baroque historic monument, a place of pilgrimage with a captivating history. Behind the main façade, which is the marvelous work of the famous Dientzenhofers, there is a courtyard with a copy of the St. Mary´s Holy House of Nazareth. Since the placement of the first construction stone in 1626, the Loreto has faced many construction phases with many alterations. The majority of the works progressed between 1698 and 1748 under the supervision of Christoph and Kilian Ignaz Dientzenhofer. These alterations also generated some changes in the configuration of the roof structure and in some parts caused even damages. Some of interventions observed on timber frames are not documented and their purpose is not always obvious.

For a better understanding, a historical analysis of the Loreto complex was used recently refined by dendrochronological dating of timber roofs elements from different time periods, so the historical chronology of the building could be accomplished. Afterwards, diagnostic tests were conducted on site for the health evaluation of the members to assess their condition. The mechanical parameters are obtained with the new diagnostic tool which is the loading jack and correlates well to the compressive strength parallel to fibers. A structural analysis is performed to evaluate the causes of the deformations on the members.

Elements subjected to the diagnostic testing proved relatively good condition with small qualitative differences between individual members. With the structural modeling, it was possible to evaluate their safety and stability and also to assess the effectiveness of their strengthening. The possible causes of their actual deformations are assessed with a brief proposal of eliminating future problems.

1 INTRODUCTION

In the Hradčany district of Prague is located the Loreta, a remarkable Baroque historic monument. The Prague Loreta is a place of pilgrimage with a captivating history. The expansive decorative frontal façade is a work of the prominent architects of the Czech Republic, the Dientzehofers. The clock tower in front carries on top the authentic Loreta carillon. The building on the inside hides the famous Loreta treasure, the Prague Sun, a 6222 diamonds monstrance. Beside the St.Vitus Cathedral treasure, it is the most valuable treasure in the Czech Republic. The surrounding cloisters conceal on the inside a copy of the St. Mary's Holy House of Nazareth, which also represents the first building in the Loreta complex.

The building of the monument, since the placement of the first stone in 1626, has faced many phases of construction. The majority of the works have progressed between 1698 and 1748, where the majority of them were supervised by Cristoph and Kilian Dientzenhofer. The changes in the cloisters levels and the extension in the buildings affected the timber roofs. In many areas of the roofs, the internal configuration of the structure is altered and lead to many damages. The southern part of the west wing roof contains many of these alterations in terms of historical changes, interventions and damages. Some of the interventions are not documented and their purpose is not quite understood. One of the timber frames is significantly deformed and a strengthening column is used as reinforcement. The cause of deformation is not straightforward and the strengthening efficiency purpose is not documented.

The scope of this paper involves the historical analysis of the chronological construction of the monument. A historical background is developed with the available literature resources enriched by the dendrochronological dating of the timber roofs. The historical background is used to make the assumptions on the historical changes in the roof with the significant deformation. After the distinction of the interventions on that area, with the help of non-destructive tests a comparison between the original and new elements is accomplished. Mechanical parameters of the timber elements are obtained from the tests and are used in the structural analysis. Structural analysis of one frame suffering deformation is executed to demonstrate the historical assumptions. The stability of the current state of the frame is controlled to evaluate its safety. A strengthening proposal is given with the results obtained from the structural model.

2 ARCHITECTURAL FEATURES

The Loreta of Prague as an iconic place of pilgrimage in Prague is a monument which consists many buildings in the form of an integrated complex. The integration of this complex was achieved by the famous architects, son and father, the Dientzenhofers. The main features of this complex are namely the Santa Casa, the church, the cloisters together with the chapels, the front façade and the bell tower which consists of the famous carillon (Figure 1).

The Santa Casa or "The Holy House" is a copy of the Nazareth Santa Casa which is currently located in the Italian Loreto, whereas its copy is located in the center of the Loreta complex in Prague. Its significance comes from the legend that describes it as the building where Virgin Mary was announced to conceive the Son of God. The Church of Nativity of Our Lord is located in the eastern wing in the mid-cloister with an extension to the east. On the lateral sides of the church are located two polyhedral towers with truncated polyhedron roofs with spire finishes on top of an onion dome. The front entrance is featured with a marvelous façade with the bell-tower located in the center. The front façade is the work of the Dientzenhofers which features a Baroque style, symmetrical and divided by pilasters. The tower starts with a square base at the bottom to be followed by an octagonal one up to the roof. The truncated polyhedron roof finishes with a spire on top of an onion dome. In the tower inside lays the famous and one of the few authentic carillons that survived time. On the four corners and on the northern and southern mid-wings of the complex, are located six chapels. They are connected with two-story cloisters which enclose the entire complex and are open in the form of arcades in the ground level and are closed on the second level.

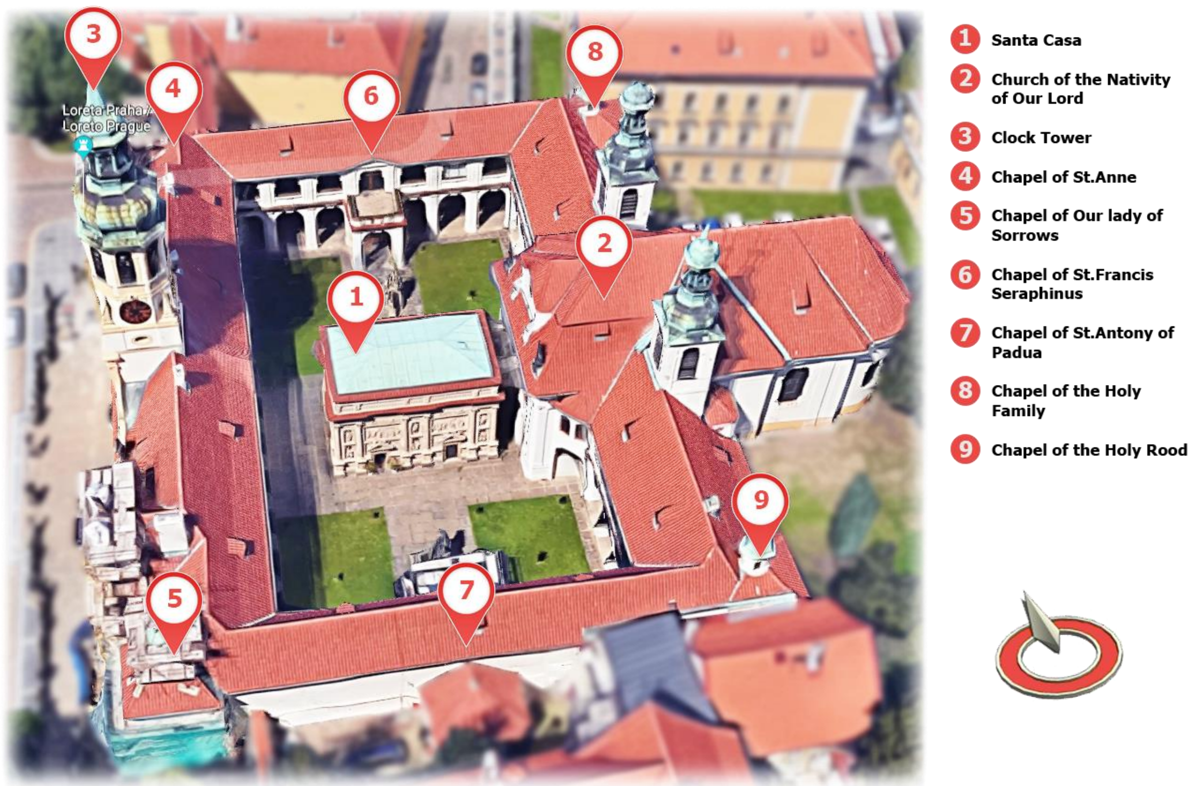

Figure 1 – Compound parts of the Loreta in Prague

3 HISTORICAL TIMELINE

The construction of Loreta in Prague started in 1626 and experienced many construction phases which were influenced by political issues and diverse architectural styles. The appearance of the Loreta as of today is achieved through many changes in its structure which inflicted damage and stress changes in its members. The historical timeline of Loreta is explained chronologically by **[1]** and is presented in the form of a table in the Table 1 below.

Table 1 – Historical timeline of the Loreta construction

Year		Event
1626	-	The foundation stone for Santa Casa was laid with the financing of the Lobkowicz family
1634	-	The cloisters construction in the form of a fortification, very typical for the Thirty Years' War. Works started with Giovanni Battista Orsi followed by Andrea Allio after his death
1646	-	Tower was finished and the works were continued with Silvestro Carlone after Allio's death
1664	-	A hypothetical model of Loreta for the year 1664 was built from these data and the archival reports presented in Figure 2 (left). It features the Santa Casa, one story cloister, no chapels, no church, the bell-tower and two towers on the eastern wing
1685	-	Two chapels were added on the south-west and north-west corners
1691	-	Two more chapels added on the north-east and south-east corners
1699	-	The upper part of the bell-tower was shifted from a rectangular base to an octagonal base to fit the famous carillon and can be seen in the hypothetical model of 1699 in Figure 2 (middle)
1712	-	The addition of the mid-cloister chapel in the southern wing and mark the beginning of Dientzenhofers works
1716	-	The addition of the mid-cloister chapel in the northern wing
1717	-	The church of Nativity is extended to the east and the elevation of the cloister to fit the treasury in the northern side of the front façade. The southern side cloister of the front façade is covered with an attic wall
1720	-	The hypothetical model of the year 1720 presents the updated changes and can be seen in Figure 2 (right)
1722	-	Cristoph Dientzenhofer is contracted for the enlargement of the Church of Nativity to the west but following his death, the works are continued by his son Kilian. He unified the front façade and divided it with pilasters
1747	-	The cloisters were elevated, and Kilian finally diminished the fortification character of the Loreta complex

Figure 2 – The hypothetical models of the year 1664 (left), 1699 (middle) and 1720 (right); **[1]**

4 ROOFS OF LORETA

Timber roofs during their lifecycle develop annual rings which provide valuable information about the history of the structure. Up until the felling date, the tree develops the annual growth rings which form the cross-section of each tree. This cross-section is divided into the heartwood and the sapwood part and each growth ring layer consists of the earlywood and the latewood. From these layers a tree can be assessed its growth conditions and can be further dated through the dendrochronological dating. A dendrochronological survey was performed in Loreta by [3] and the results are presented in Figure 3 (left). Comparing it to a stratigraphy model in Figure 3 (right) a correlation of the dates can be made with some differences. The date differences in the church are clearly seen to match in the two models where the dendrochronological model compared to the stratigraphy one provides the felling dates of the timber elements used. The central part of the church has an earlier construction date, nevertheless the dating shows a later date. This can be a result of the adjustment of the central part to the extensions on the western part followed later with the eastern extension. The cloisters, as previously mentioned, were elevated from one to two levels. The dates of construction match the felling dates of the cloisters with exceptions to the northern and western wing. The northern wing timber elements match the construction date of the chapel and this can justify it. On the other hand, the front façade as it was mentioned, faced many diverse changes. The dendrochronological dating of these changes matches the construction phases of the western part of the complex. The room made for the treasury can be clearly distinguished in the dating together with the clear differentiation of the cloisters on the southern part of the front façade.

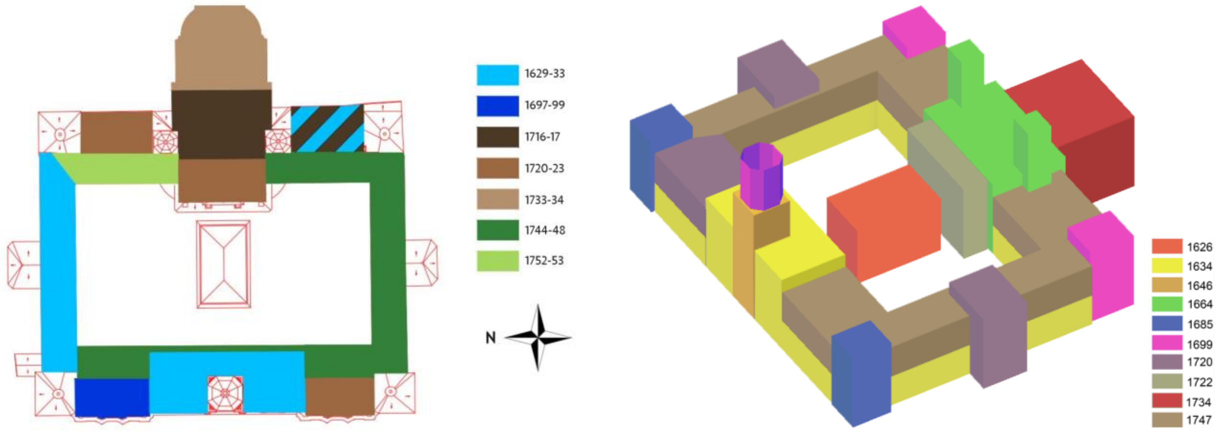

Figure 3 – Dendrochronological dating of the timber roofs (left) [2] and the stratigraphy model of Loreta (right)

5 DIAGNOSTIC TESTS

The area of interest and the focus of this paper are mainly the roof of the church and the roof on the south part of the western wing. This was chosen since there were some damages seen in these two areas, where the roof of the church was already under restoration at the time of inspection. Nevertheless, a huge deformation was found on a frame of the southern part of the west wing. The cause of deformation and its stability where unknown and are assessed and presented in this paper. Initially a visual inspection was carried out in the roof for assessment and for the choice of diagnostic tests. It was seen suitable and not invasive to use the resistance microdrilling, loading jack and the moisture content.

5.1 Diagnostic tests

Following the [4], a visual inspection was carried out in the southern part of the west wing. The structural timber elements were classified into classes, namely S1, S2 and S3 and according to [4] are obtained the mechanical parameters. The authentic joint connections of the structural timber elements are carpentry joints with some minor late modifications. There are not many archival reports on the past interventions carried out in the roof therefore many statements are made from observation only. Strengthening short columns are seen to reinforce the lower ends of the roof rafters, which are connected to an embedded timber beam on the concrete floor (Figure 4). Additionally, small diameter holes are seen in the majority of the timber elements which can be due to the injections as a manner of protection against insects (Figure 4). Furthermore, fireproofing is identified in some elements and metal connectors are detected in some other as measures of joint reinforcement (Figure 4). Afterwards, in order to perform the timber grading as per [4] the features examined were the wane, growth rate, knots, slope of grains, fissures and other geometric features. The final class found for the timber elements as per these norms was found to be S1 leaning towards S2. This classification gives a range of values to be further assessed and verified with other tests.

Figure 4 – Characteristic features on the elements from the visual inspection

5.2 Resistance microdrilling

Utilizing the Resistograph a small diameter hole is driven into the timber elements. The output provides the timber profile of the section in terms of energy required to keep the needle at constant speed. Therefore, the difference in the energy output shows a variation in material resistance and thus interprets decays and damage inside the timber. The microdrilling was performed in the frame which showed the extensive deformation and especially on the lower end support where it is resting on the wall. Furthermore, on that lower end a strengthening with a prosthesis can be seen and from the results the effectiveness is assessed. The microdrilling was carried out in 18 locations. Following the correlation between the resistance measure (RM) and the compressive strength of wood (Sc) provided by [6] with a correlation factor of R^2=0.807 from the equation (1), the results are given in Table 2. Additionally, equation (2) with a correlation factor of R^2=0.746 provides the correlation between RM and density (ρ). The average compressive strength is around 22.3 [MPa] and the average density is around 176 [kg/m³].

$$RM = (-343.83) + (22.296) * S_c \tag{1}$$

$$RM = (-137.67) + (1.665) * \rho \tag{2}$$

Table 2 – Correlation of RM to compressive strength and density of timber

Test no.	RM	Sc [MPa]	ρ [kg/m³]
R1	161.8	22.68	179.85
R2	146.1	21.97	170.41
R3	145.1	21.93	169.81
R4	150.7	22.18	173.21
R5	146.2	21.98	170.52
R6	133.9	21.43	163.12
R7	131.4	21.31	161.60
R8	161.9	22.68	179.94
R9	155.8	22.41	176.27
R10	190.2	23.95	196.92
R11	139.9	21.70	166.74
R12	180.3	23.51	190.96
R13	174.3	23.24	187.36
R14	152.4	22.26	174.24
R15	168.8	22.99	184.07
R16	173.2	23.19	186.72
R17	143.4	21.85	168.83
R18	142.1	21.79	168.01

From the output profile of the timber cross-section the damage and decays were able to be observed. Furthermore, following the peaks of the resistances in the results from the Resistograph the rings of the tree are constructed hypothetically and utilized for the qualitative categorization of the element. An example can be seen in Figure 5.

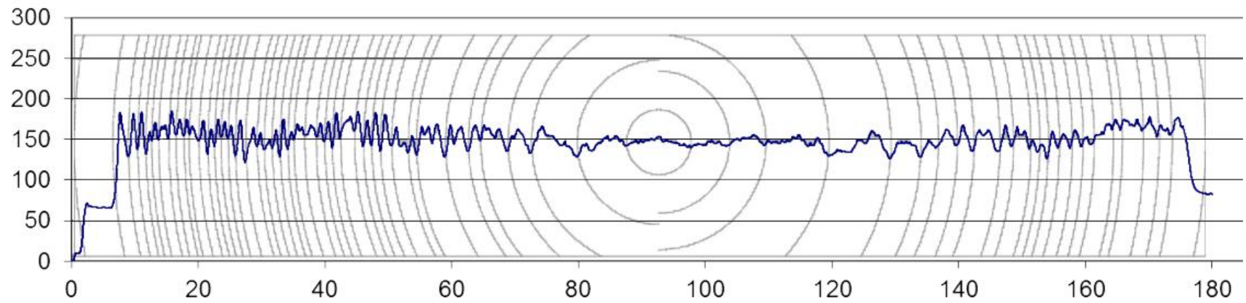

Figure 5 – Ring trees and possible damage obtained from resistance microdrilling

5.3 Loading Jack

One innovative diagnostic test with minimalistic invasion on the timber elements is the loading jack device. It is used to evaluate directly the compressive strength of the timber elements parallel to fibers. The jack is inserted into a predrilled hole of 12 mm wide and the jaws of the device are opened apart by pushing laterally up to 1.5 mm. A force-displacement curve is plotted following this procedure and from the maximum force achieved is obtained the maximum compressive strength. For further information on this intuitive and innovative device the reader can be directed to [5,8,10]. The loading jack test is carried out in the south part of the west wing (6 locations) and in the roof of the church in the east (5 locations). The results for the conventional strength and (CSc) and the modulus of deformability (MOD) are shown in the following charts in Figure 6.

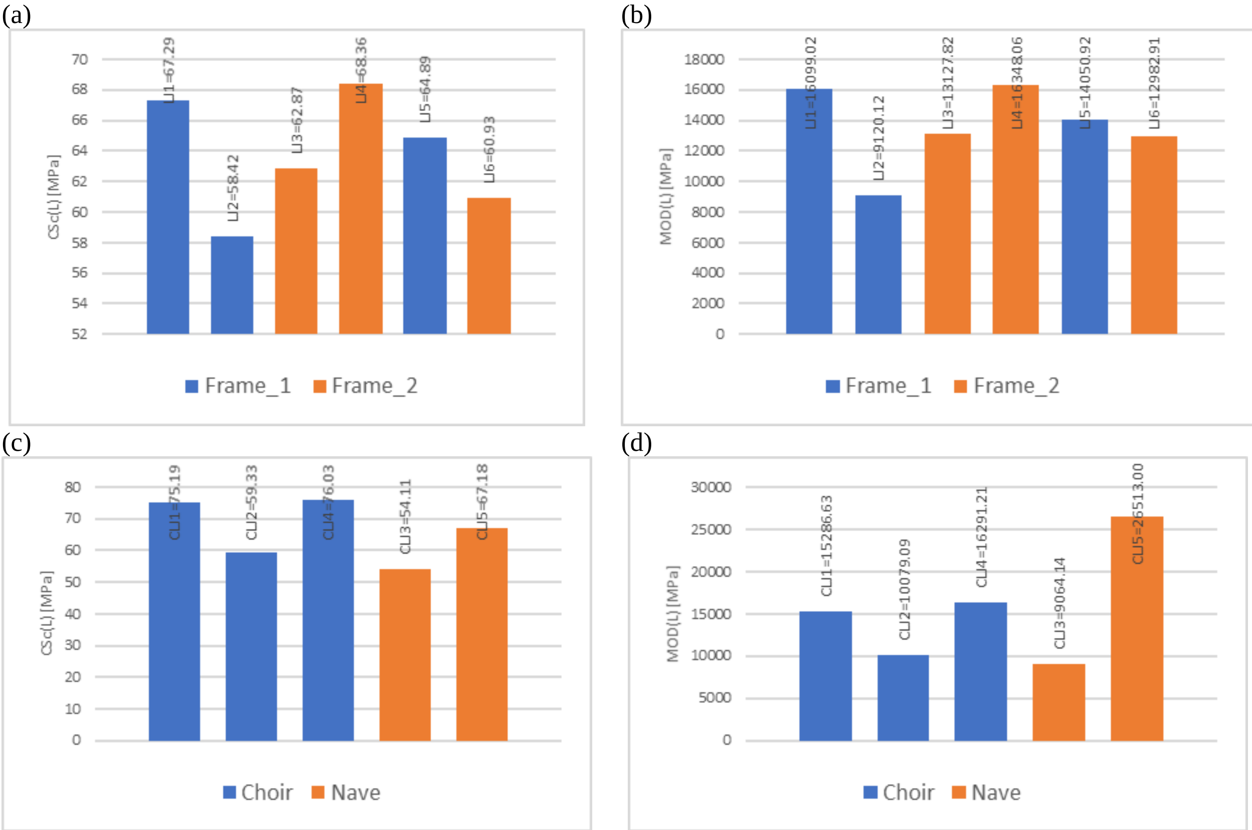

Figure 6 – Loading jack results for south part of the west wing CSc (a) and MOD (b) and the church CSc (c) and MOD (d)

[9] provides correlation between the loading jack results and the compressive strength, modulus of elasticity and the density. The correlation follows the results from specimens with 12, 18 and 24 % moisture content. However, only the correlation for 12 % is used since the moisture measured and shown in the subsequent section, in the frames does not exceed 12%. The correlation between conventional strength measured from loading jack (CSc) and the compressive strength (Sc) with a correlation factor of R^2=0.887 is shown in equation (3). The correlation between the modulus of deformability (MOD) from the loading jack and the modulus of elasticity (MOE) with a correlation factor of R^2=0.679 is shown in equation (4). Lastly, the correlation between the conventional strength (CSc) and the density (ρ) with a correlation factor of R^2=0.840 is shown in equation (5). The results are shown in Table 3. From the Czech annex of Eurocode [7] the elements are classified as C20 structural timber class.

$$S_{c\,(L)} = 0.5661 * CS_{c\,(L)} - 3.3584 \tag{3}$$
$$MOE_{(L)} = 0.4934 * MOD_{(L)} - 1901.7 \tag{4}$$
$$\rho = 5.1214 * CS_{c\,(L)} + 27.265 \tag{5}$$

Table 3 – Correlation between loading jack and mechanical parameters

		CSc [MPa]	MOD [MPa]	Sc [MPa]	MOE [MPa]	ρ [kg/m³]
South part of the west wing	**FR-LJ1**	67.29	16099.02	34.73	6041.56	371.86
	FR-LJ2	58.42	9120.12	29.71	2598.17	326.44
	FR-LJ3	62.87	13127.82	32.23	4575.56	349.26
	FR-LJ4	68.36	16348.06	35.34	6164.43	377.38
	FR-LJ5	64.89	14050.92	33.37	5031.02	359.58
	FR-LJ6	60.93	12982.91	31.13	4504.07	339.29
Church roof	**CH-LJ1**	75.19	15286.63	39.21	5640.72	412.34
	CH-LJ2	59.33	10079.09	30.23	3071.32	331.12
	CH-LJ3	76.03	16291.21	39.68	6136.38	416.65
	CH-LJ4	54.11	9064.14	27.27	2570.55	304.38
	CH-LJ5	67.18	26513.00	34.67	11179.81	371.32

5.4 Moisture content

The moisture content provides a significant qualitative measurement of the level of decay and strength of the timber elements. The durability of timber decreases with the increase of moisture above fiber saturation point. This due to the vulnerability of timber towards insects and fungi. Furthermore, the strength and elasticity are reduced, and the elements swell. The moisture content is measured on some elements of interest and is shown in Figure 7. The internal moisture content is measured on the 6 predrilled holes used for the loading jack. Therefore, the moisture is measured at four depth levels on 3 locations per frame, a total of 6 locations. Additionally, the superficial moisture is also measured with the help of a hygrometer, on both sides of the frame no.1 (Figure 7-left) and on the internal side of the frame no.3 (Figure 7-right) because the other side was embedded in the wall. The moisture levels on the timber elements do not present any threat to the health of wood.

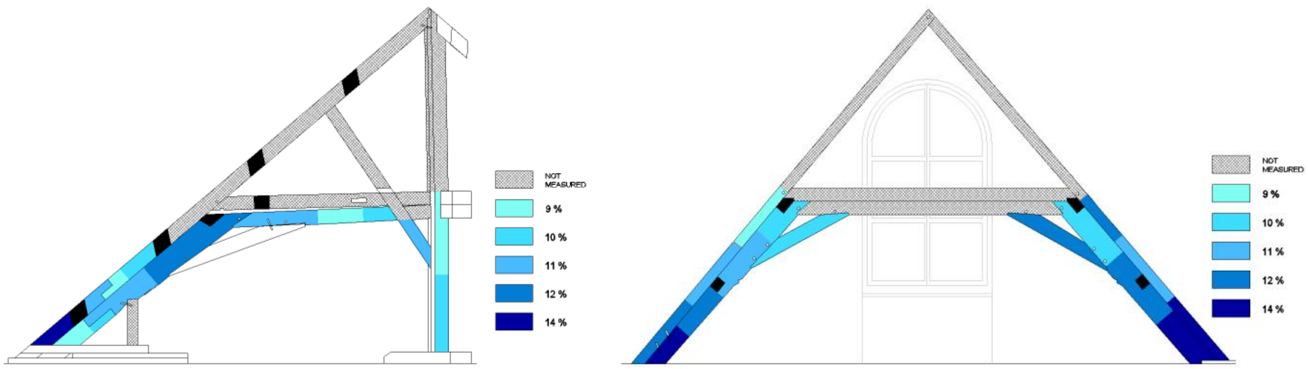

Figure 7 – Moisture content for the Frame no. 1 (left) and Frame no. 3 (right)

6 STRUCTURAL ANALYSIS

The different construction phases of Loreta present many challenges for the health assessment and structural stability of the structure. The cause of damage and decay may be from different reasons and the evaluation is not always accurate and many hypotheses are to be constructed to achieve the most probable cause. The main focus of this paper is the cause of deformation of the frame in the south part of the west wing (Figure 8) and the evaluation of its safety, stability and a possible strengthening measure. This deformed timber frame is the supporting element for both pitches of the southwestern roof and therefore attracts the majority of forces. The frame presents a rather complex element in means of numerical modeling since the forces are mainly distributed through deformations of the members and the connections are made of carpentry joints. The frame consists of a vertical column attached to a masonry wall and at the top attached to the principal rafter which forms the connecting edge of the two pitches of the roof. A supporting substructure is formed under the principal frame which reduces the span of the main structure. On the lower end of the principal rafter can be seen the strengthening means which were later added to the frame. Additionally, a metallic fastener can be seen on the diagonal truss connecting the secondary rafter and the horizontal beam, which is added later to avoid slipping of the joint.

Figure 8 – The deformed frame of interest in the south part of the west wing

The structural analysis is carried out only for this portion of the roof (southwestern corner) by simplifying its geometry to a simpler one. This enables a detailed computation of wind and snow forces acting on the roof, according to the present codes [11,12]. From the global analysis of this portion of the roof, are calculated the internal forces acting on each element. Afterwards, a two-dimensional model is built to assess the different phases of construction. The deformation is assumed to have been caused during the restoration works which were present on the roof. Three scenarios are proposed for the analysis and the determination of the deformation on the frame. The first scenario consists of the phase before the strengthening and it is used to justify the need for a reinforcement of that frame. The first scenario consists of the frame without the short column and the concrete slab which are existing on the roof. Typically, a tie beam is usually placed at the bottom of the frame and it is modelled as a support with a specific stiffness according to the assumed axial stiffness of the tie beam. The second scenario is assumed that during the process of strengthening the effect of the tie beam on the truss is removed. Finally, a third scenario is built to represent the current state of the frame containing the actual deformation and to evaluate its stability. Rigid links are created to transmit the deformation of the principal rafter to the secondary one. The rigid links work only under compression to depict the transmission of the deflection by contact. To reproduce the superficial friction between the elements a tangential axial force is computed and input in the model. The results of interest are shown in Figure 9.

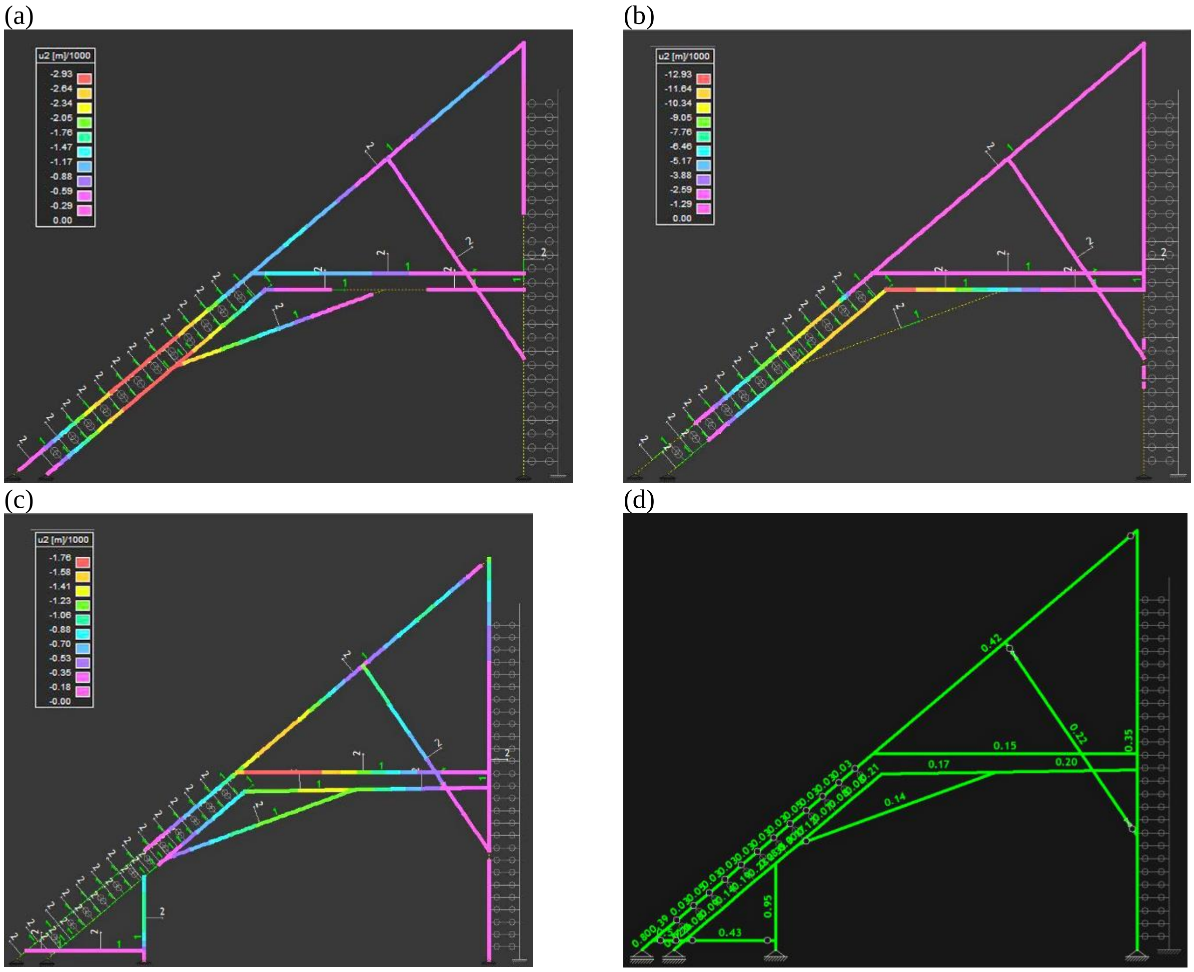

Figure 9 – Results for: Scenario 1 deflections (a), Scenario 2 deflections (b), Scenario 3 deflections (c) and structural stability evaluation (d)

The loading conditions at the time of Scenario 1 are unknown and therefore the results are only relative. A large deflection is seen in Figure 9 (a) in the middle of the principle rafter which can justify the need for a strengthening. In Scenario 2 after the removal of the tie beam the diagonal truss starts acting on tension and loses its function and the horizontal beam on the left side can be seen to suffer a large deflection Figure 9 (b), which is still present on the frame. Scenario 3 shows a strengthened frame and reduced deflections seen in Figure 9 (c) and the same scenario is evaluated to be stable for the actual level of forces acting on it presented in Figure 9 (d). This hypothesis presents a high level of reliability for the cause of deformation. Nevertheless, if more hypothesis can be formed a comparative analysis can be done to assess the most reliable one. After this assessment the frame does not require excessive strengthening measures. A possible strengthening proposal can be the placement of a tie beam at the bottom to take the horizontal thrust.

7 CONCLUSION

A chronological timeline and the understanding of construction phases can aid the assessment of any structure. From the historical construction phases, it was able to make an assessment of the roof.

Diagnostic tests performed on the roof help on the health assessment of the timber elements. The roof does not show any potential threat of extensive decay or damage. Furthermore, the mechanical parameters are obtained with satisfactory correlation factors from the tests. Utilizing these parameters, a more reliable assessment of the cause of deformation is carried out. The hypothetical scenario for the cause of deformation shows a high reliability and unless a more accurate one is presented, this can be the actual cause.

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