

Certificate of attendance and completion

This is to certify that

Valentin Atanasov

Attended the Training school on:
Assessment and reinforcement of timber elements
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Organized by
University of Mons, Faculty of Engineering, Belgium,

This training course focused on how to improve the assessment of timber members and connections to both estimate the weaknesses and the individual strengths as well as propose suitable reinforcement measures.

Valentin Atanasov has successfully completed a
scientific report entitled:

***Assessment of the roof structure of St. Michaels
church in Vienna.***

for: 7.5 ECTS

Grade: Passed

Assessment of the roof structure of the St. Michaels church in Vienna

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ABSTRACT

Most European cities, like Vienna, exhibit a large number of historic buildings. The roof structures of these buildings consist mainly of timber elements. These timber structures, which were generally built based on experience, have hardly been assessed by means of modern design principles. The assessment of the reliability of these structures is essential, since many of them are of public importance. Therefore, a project together with our colleagues from the Institute for History and Theory of Architecture and Historic Building Survey, Vienna University of Technology, has been started to assess the condition of the roof structure of some important buildings in Vienna. One of these buildings is the church of St. Michaels. A detailed building survey in combination with visual inspection of the structure has been carried out, to identify possible pathologies. In addition, a dendrochronological examination and determination of the timber moisture content have been conducted. In order to give a first statement on the reliability of the structure, a preliminary structural analysis based on the principles of the current versions of the design standards, has been executed. The presented results approve the timber roof structure to be in a good condition, especially taking into account the age of the roof structure. The general structural behavior, as well as the two detected main defects have been able to be qualitatively reproduced by the conducted structural analysis. A first positive statement related to the reliability of the structure can be given.

1 Introduction

The church of St. Michael's is a cultural monument and one of the oldest churches in the capital of Austria - Vienna. Over time, the church has undergone several disasters. In 1525 the entire roof of the building burnt down in a fire and was, subsequently new built. Later in the century, parts of the tower collapsed due to an earthquake, but in general, the roof structure was not affected. Figure 1 (left) shows the modern view of the church, the interior of the nave roof structure is shown in Figure 1 (right).

The roof comprises a main longitudinal part (nave) which is crossed in the middle with a transverse part (transept). The overall dimensions of the structure are: length 62 m, width 11.5 m and a height of 10.2 m. The



Fig. 1. View of the St. Michaels church (left), interior of the nave roof structure (right).

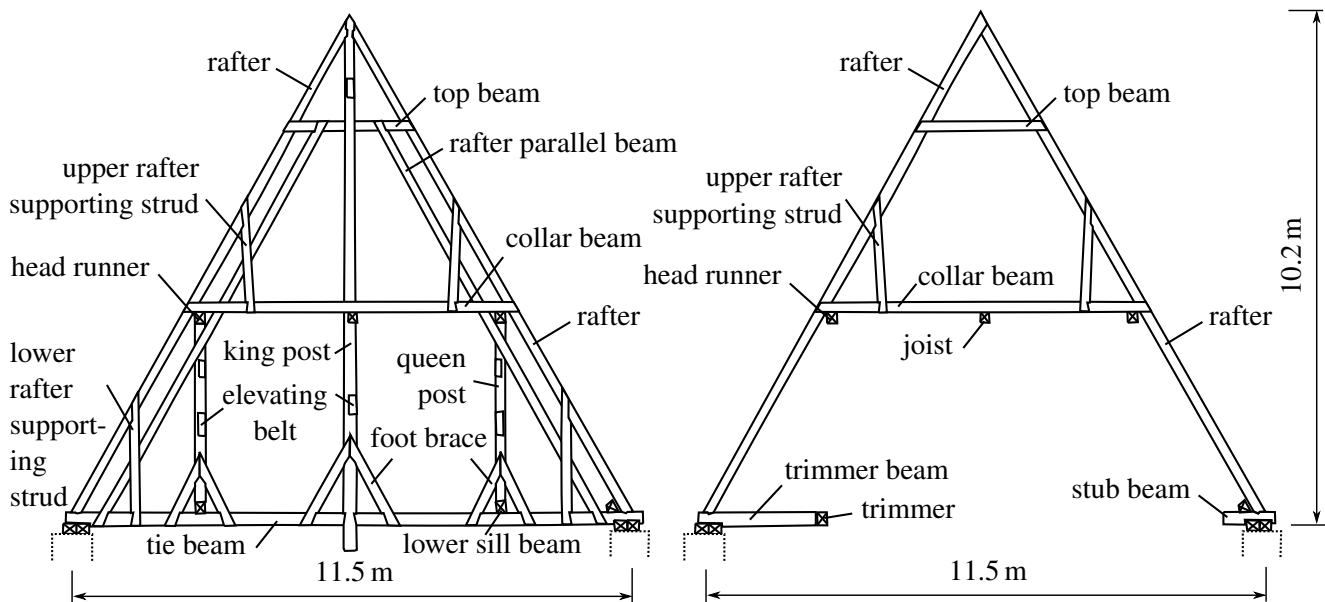


Fig. 2. Collar beam roof - elements of the main rafters (left) and of the intermediate rafters (right).

roof structure itself is a so-called collar beam roof, which is a special type of a rafter roof. This roof type has been used for larger spans and is characterized by the collar beam, which connects the two rafters in order to reduce bending deformations of these elements. In addition, the tie beam is supported by the so called king post. Furthermore, the rafters (collar beam) are supported by queen posts on both sides. Only the main rafters are constructed like described above (see Figure 2 - left). Between each of two main rafters three intermediate rafters are located. These intermediate rafters consist only of the rafters connected by the collar beam, supported by head runners, which transfer the loads in bending to the main rafters (see Figure 2 - right).

In general, the timber roof structure is in good condition, especially taking into account the age of the structure. The main purpose of this work is to understand the structural behavior of the timber roof, and to give a first statement on the reliability of the timber structure, including observed defects of the roof. The main pathology of this timber structure is addressed to the connection of the nave and transept roof part, where each of the four notched head runners of the transept exhibits massive cracks in longitudinal direction (see Figure 9 - left). Furthermore, large deflections of the transept apex can be seen (see Figure 9 - right). In addition, missing elements of the structure, which have been removed in the past to make the roof structure accessible, may affect the structural behavior. Furthermore, at some parts of the roof, moisture stains have been detected, which may indicate the possibility of water intrusion.

To get a better understanding of this timber structure, a project together with our colleagues from the Institute

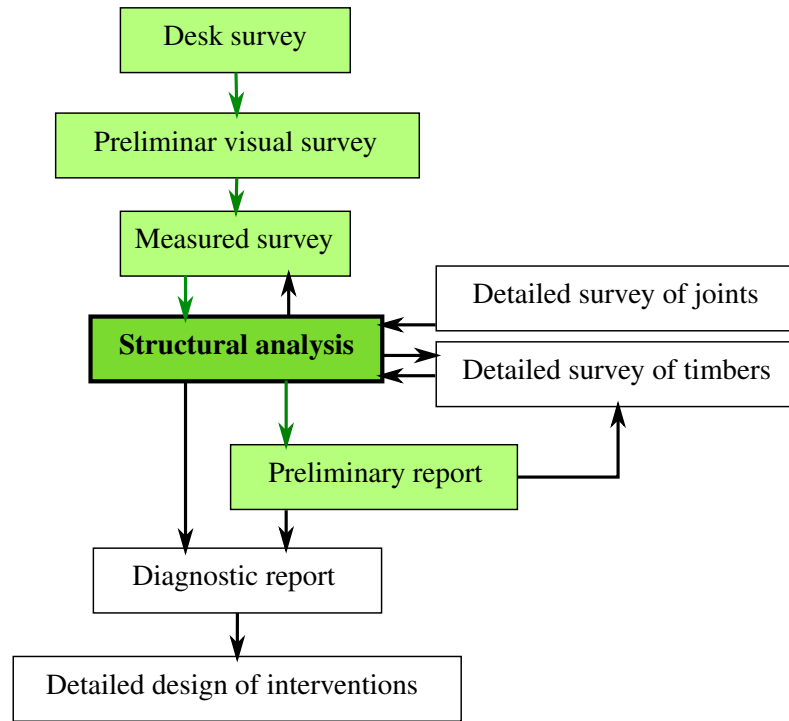


Fig. 3. Steps for the assessment and planning of interventions in historic timber structures according to [1].

for History and Theory of Architecture and Historic Building Survey, Vienna University of Technology has been started. The aim of our colleagues work was to record the geometry of the structure (see Section 2.1), as well as to get an idea of the assembling procedure of the roof. Therefore, the geometry of the real, deformed structure and of several joints has been documented. Based on the design of the joints, together with the carpenter marks, possible assembling procedures have been reconstructed. In addition, a dendrochronological examination has been carried out by Dr. Michael Grabner (University of Natural Resources and Life Sciences, Vienna) in order to determine age and wood species of the timber elements.

Based on the information, provided by our colleagues, our work was addressed to the assessment of the timber structure from the point of a structural analysis. Therefore, the recently proposed stepwise assessment procedure for the assessment of historic timber structures was followed herein [1] (see Figure 3). The work at hand covers the steps marked by green color in Figure 3, with a special emphasis on the structural analysis. Based on a visual inspection (see Section 2.2), we tried to get a first idea of the stiffness and strength properties of timber as input for our structural analysis. Different methods for visual grading, e.g. presented in [2], have been investigated. In addition, the moisture content, as important parameter of timber properties, has been measured on-site for different roof elements. Finally, the detected information has been used for the structural analysis and possible reinforcement strategies are proposed.

In general, the visual inspection indicated that the timber structure is in a good condition. Furthermore, the structural analysis has been able to show the structural behavior of the roof, as well as to reproduce the detected defects at the crossing nave - transept. A first positive statement addressed to the reliability of the structure can be given.

2 Methods

When carrying out an assessment of timber structures, two main steps should be taken into account [3]:

1. Assessment of the technical conditions of the building and review of assemblies;
2. Assessment of wood's health.

When reporting the technical condition of the building, cracks, large knots, distortion, deformation and other mechanical damages of wood are investigated. Attention is drawn to the entire structure and individual elements.

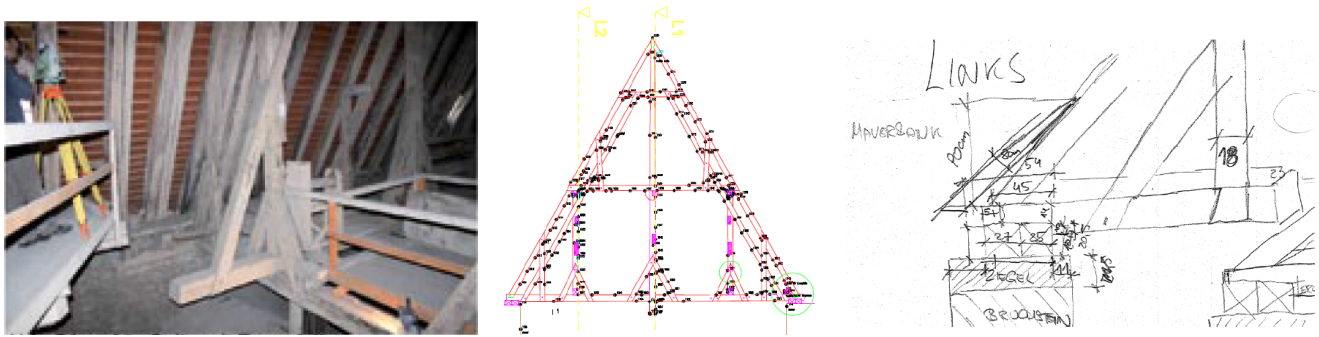


Fig. 4. Survey by means of a total station (left); recorded points (middle); sketch of a connection (right).

Assessing the health of the wood is performed on the basis of attack by fungi and insects. It could be done visually, by non-destructive method or by local destruction. Due to the fact, that the application of destructive methods in historic buildings is not desirable, non-destructive methods are commonly used.

Considering the age of the current wooden structure, its necessary to carry out an inspection for potential problems. Following assessment methods have been carried out to achieve the goals of our assessment program:

1. Recording of the geometric properties of the roof structure;
2. Dendrochronological examination to determine age and wood species of the elements;
3. Performing a visual inspection including the measurement of the timber moisture content;
4. Performing a structural analysis.

2.1 Geometry

To record the geometry of the timber roof structure, different survey methods have been applied. First, simple freehand sketches of the ground view, transverse section and longitudinal section have been conducted. Based on these sketches, the main survey program has been arranged. The ground view, transverse and longitudinal section of the roof structure have been surveyed at the current deformation state, by means of a total station. A total station is an electronic theodolite (measures angles) in combination with an electronic distance meter (measures the slope distance) - see Figure 4 (left). The geometry of the single elements is recorded by a large number of points on the surface of these elements (see Figure 4 - middle). The measured points have than been transferred automatically to the program TachyCAD (kubit GmbH). The surveyed points act as basis to generate the planes of the roof structure, which in turn serve as basis for the structural analysis model. Hidden points have been recorded by hand, using a laser distance meter and a tape. Details, like connections and special types of fasteners have been recorded by means of sketches (see Figure 4 - right).

2.2 Visual inspection

The visual inspection represents one of the most important parts in the assessment of our timber structure. When carrying out our visual inspection, defects of the timber as: moisture stains, missing or added elements, cracks, number and size of knots, quality of the connections,... have been taken into account.

Moisture stains indicate areas of timber elements with high moisture contents. This areas have a high risk to be attacked by fungi and insects, which in turn lead to a loss of stiffness and strength of the material. At our visual inspection special attention has been paid to these possible wet areas. To check the timber moisture content itself, additional equipment, by means of an electrical meter to determine the moisture content of wood elements, has been used (see Section 2.3).

The usage may change several times during the life time of such a structure. So, special attention has been paid on reconstruction actions, which usually include adding or removing elements of the structure, and consequently may changes the load-carrying behavior of the structure.

Related to the strength and stiffness properties of the timber elements, a large number of information can be extracted from the visual inspection. For example cracks may reduce mechanical properties, depending on the characteristics of the cracks. It has to be distinguished between the different reasons and positions of the

cracks. While for example a typical longitudinal crack on the element surface, in the center of the beam, caused by shrinkage, may be less important, cracks at connections may probably indicate an overloading of this joint, which may affect the load-carrying behavior of a part of the structure, or even of the entire structure. Detected cracks have been taken as motivation, in order to explain these pathologies by means of the conducted structural analysis.

Even the strength and stiffness properties can be roughly estimated by visual inspection. The number and size of the knots are a good indicator for the quality of the used wood. In addition, it is possible to conclude from the execution of the timber elements, especially from the quality of the connections, to the general quality of the used wood, and therefore, to the strength and stiffness properties. Since the work at hand mainly focus on the structural analysis of this historic structure, the visual estimation of the mechanical properties, has been probably the most important part of our visual inspection. In addition, the species and age of the wood has been identified by means of a dendrochronological examination (see Section 2.4, which consequently affects the input parameter of the structural analysis).

2.3 Timber moisture content

In order to confirm possible wet timber elements, detected by the visual inspection, the measurement of the timber moisture content at this areas as well as at randomly chosen points has been carried out. Furthermore, the measured moisture content serves as basis to determine the service class according to ÖNORM EN 1995-1-1 (EC5) [4], subsequently used in the structural analysis.

Primary, by a contact-less thermometer (pyrometer), the surface temperature of the timber was measured, and the wood species has been determined. Thereafter, the timber moisture content was determined by using an electrical meter (BES BOLLMANN COMBO 200) with a depth of penetration of approximately 3 cm, based on the primary determined surface temperature and wood species. The purpose of this operation is to determine the threat of fungal attack, rotting, . . . Furthermore, the moisture content remarkably influences the mechanical properties of the timber. Due to the fact that timber elements have different moisture contents in separate parts, the moisture content is determined at three points for each investigated element [2]. Then the average moisture content was calculated as the arithmetic mean value.

2.4 Dendrochronological examination

Dendrochronology is a scientific field which studies the anatomy of wood or rather - the tree rings (growth rings), in order to date the time at which the tree rings were formed. Since the characteristics of the growth rings depend on factors like, wood species, habitat conditions, density of plantation, local climate conditions, . . . the dendrochronological examination allows to determine origin, species and the time at which the tree was cut. Therefore, dendrochronology is a suitable method for dating of buildings, like it has been done for our project (St. Michaels church). Simple speaking, in a dendrochronological examination the growth ring characteristic of extracted timber core samples are compared to reference sequences. Our colleague, Dr. Michael Grabner (University of Natural Resources and Life Sciences, Vienna) carried out a comprehensive dendrochronological examination including more than 80 wood samples, extracted from different positions of the roof structure.

2.5 Structural analysis

The main purpose of the work at hand is to understand the structural behavior of this historic roof structure. The aim of the structural analysis is to identify realistic inner forces (e.g. normal forces in tension or compression, bending moments, . . .), which have to be carried by the different elements of the structure. Furthermore, the structural analysis will give us a first clue of the forces, which have to be carried by the connections. Therefore, the general nonlinear, compliant behavior of the connections have to be taken into account, in order to obtain a realistic force distribution within the structure.

Finally, the structural analysis shall predict the load-driven pathologies of the structure, which have been observed during the visual inspection see Section 3.2. These are mainly local defects, like cracks close to connections (see Figure 9 - left), and global problems of the structure, like the deflections of the crossing structure (transept) at the intersection to the main structure (nave).

To limit the effort for modeling the structure, as well as to minimize the computational time, only a part of the structure has been investigated within the structural analysis (see Figure 8). For this purpose, the intersection between the two crossing roof structures has been chosen, since all important elements of both structures are included in this area, and furthermore, the main defects of this roof structure can be found here.

2.5.1 Modeling

For the structural analysis, the commercial software package RSTAB 8 (Ing. Dlubal Software GmbH, Germany) has been used. This software allows to define a three dimensional beam model of the structure, including the non-linear compliant behavior of the different types of connections.

Geometry and material properties

Since this work presents the first circle of the assessment procedure shown in Figure 3, a number of simplifications have been done. One of this simplifications is related to the geometry of the model used for the structural analysis. For example, the width of the structure for the northern and southern part of the transept is assumed to be constant. Moreover, the structure is assumed to be symmetric with respect to the king post. In addition, the inclination between the nave and transept is assumed to be of a 90° angle.

The geometry of the cross sections for the single timber elements have been extracted from the survey of the roof structure carried out by our colleagues from the institute for History and Theory of Architecture and Historic Building Survey, Vienna University of Technology (see Table 1). The wood species used for the roof structure elements have been determined by a dendrochronological examination (see Section 3.4). As an first assumption the strength class C24 (according to ÖNORM EN 338 [5]) has been chosen for several timber elements based on the visual inspection described in Section 3.2.

2.5.2 Design concept

The structural analysis is based on the current version of the Eurocode series. For loadings, the standard ÖNORM EN 1990 (EC0) [6] in combination with ÖNORM EN 1991 (EC1) [7–9] has been used. The timber related regulations, like the service class or the material properties, have been extracted from EC5 [4].

This approach, based on the partial factor concept, allows us to assess the reliability of this historic roof structure by a state of the art design concept.

2.5.3 Loading

Several loadings have been defined according EC1 [7–9]. The following loads act on the structure:

1. **Dead loads** of the timber roof structure itself, as well as of the roof covering.
2. **Live loads** acting on the tie beam and collar beam caused by people for inspection of the structure, as well as by storage objects.
3. **Snow** has not to be taken into account according to EC1 [7–9] since the roof pitch (61°) is higher than 60°.
4. **Wind loads:** It has to be distinguished between wind loads parallel and perpendicular to the apex for the nave and for the transept.

Stability issues have been neglected in this project stage.

2.5.4 Connections and bearings

In order to finally obtain a realistic force distribution within the structure, several connections and bearings are defined to be compliant. The compliance of this connections are obtained by in general non-linear slip curves for the different force - relative deformation relationships. For a three dimensional model, like it has been used in this work, each connection exhibits six degrees of freedom, which are the axial relative displacement (Δu_x), the two transverse relative displacements (Δu_y and Δu_z), the two relative rotations around the transverse axis ($\Delta \phi_y$ and $\Delta \phi_z$) and the axial relative rotation $\Delta \phi_x$.

The compliant behavior for each of this relationships can be defined by means of user-defined slip curves. The interaction between different deformation states at the same time cannot be considered by the current version

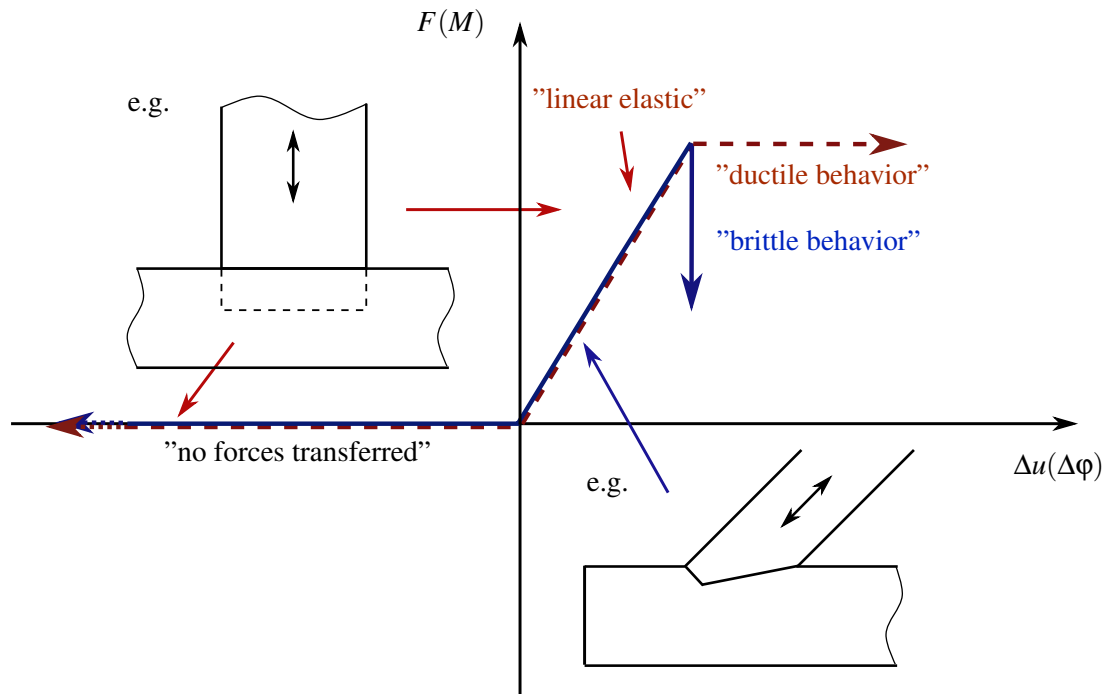


Fig. 5. General modeling approach for slip curves of connections.

of the used software. To keep the clearness of the compliant behavior of the connections, only the decisive compliant relations of the connections have been considered. At this preliminary stage of the structural analysis, the compliance, as well as the bearing capacities of the connections are mainly based on estimations.

In general, the force - relative displacement and moment relative rotation relationships are modeled by loading-direction dependent slip curves (e.g. loading in tension or compression, which means positive relative displacement and negative relative displacement, respectively). Moreover each linear elastic part of the slip curve, related to positive or negative relative displacement or rotation of the connection is followed by a ductile behavior (ideal plastic), or followed by a brittle failure. Also, it is possible that certain types of connections, cannot carry tensile loads (see Figure 5). The usual initial slip of such connection types has been neglected, since the attempt to include it within the structural analysis lead to instabilities in the calculation procedure.

Since carpentry connections go most of the time hand in hand with weakening of the cross-sections, only the net-sections have been modeled in the connection areas. Also the eccentricities, which come along with weakening of the cross-sections have been considered. This eccentricities may generate additional forces in the beams and in the connections.

In the following, the modeling procedures of two representative connections are illustrated (see Figure 6 and 7):

Dove-tail connection

This connection type is used for example as connection between king post and tie beam. Slip curves are defined for load transfer in axial direction (x) and transverse direction within the structure plane (z). Relative rotations within the plane, as well as deformations perpendicular to the structure plane have been neglected.

The deformation at the elastic limit has been estimated by experience from several experiments documented in different publications. The bearing capacity has been estimated by simple calculations mainly based on the strength of the material in combination with the contact area.

The general shape of the two slip curves of this connection are shown in Figure 6, where for axial positive relative displacements (loading in tension) a linear-elastic behavior, followed by a brittle failure can be seen (see Figure 6 - left). For negative axial relative displacements (loading in compression) a ductile behavior is assumed after the elastic limit is reached. Loading in transverse direction leads to ductile failure modes for both loading directions (see Figure 6 - right).

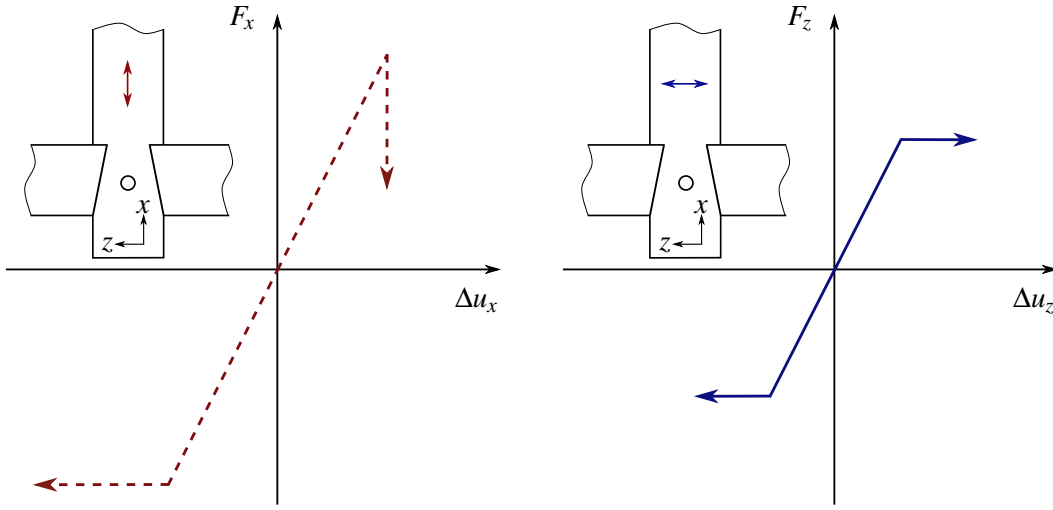


Fig. 6. Slip curves for dove-tail connections: Loading in axial direction x (left) and loading in transverse direction z (right).

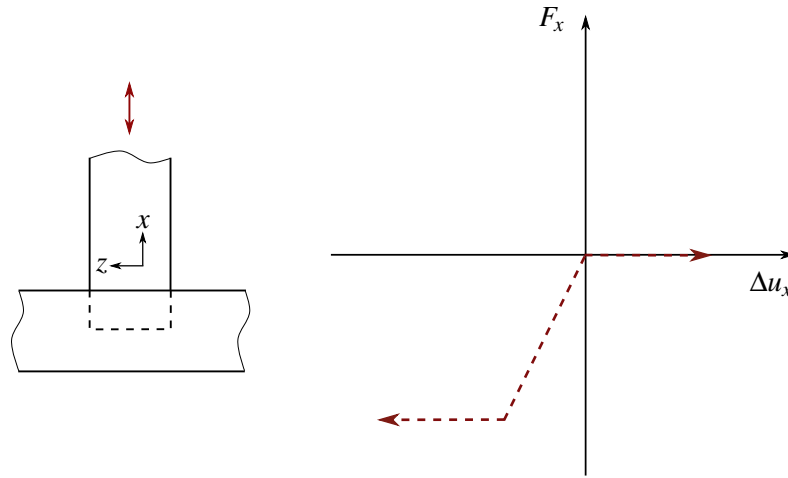


Fig. 7. Slip curve for tenon connections (Loading in axial direction x).

Tenon connection

In this structure, tenon connections can be found at the queen post and head runner intersections, as well as at the joint of the queen post and lower sill beam. Slip curves have only been defined for load-transfer in axial direction (x). The minor load-transfer in transverse direction (y and z) has been neglected, since no decisive influence on the global structural behavior is expected.

For this type of connection, only forces in compression can be transferred (see Figure 7), which means no forces can be carried for positive relative deformations. The behavior under compression is assumed as linear elastic - ideal plastic (ductile failure).

3 Results

In this section, the findings of our work are presented. First, the output of the building survey is shown. This is followed by the main results of the visual inspection, where only the results affecting the structural behavior are discussed. In addition, the data extracted from the timber moisture content measurements and the dendrochronological examination are presented. Finally, the results of the structural analysis are shown and reinforcement measures are proposed.

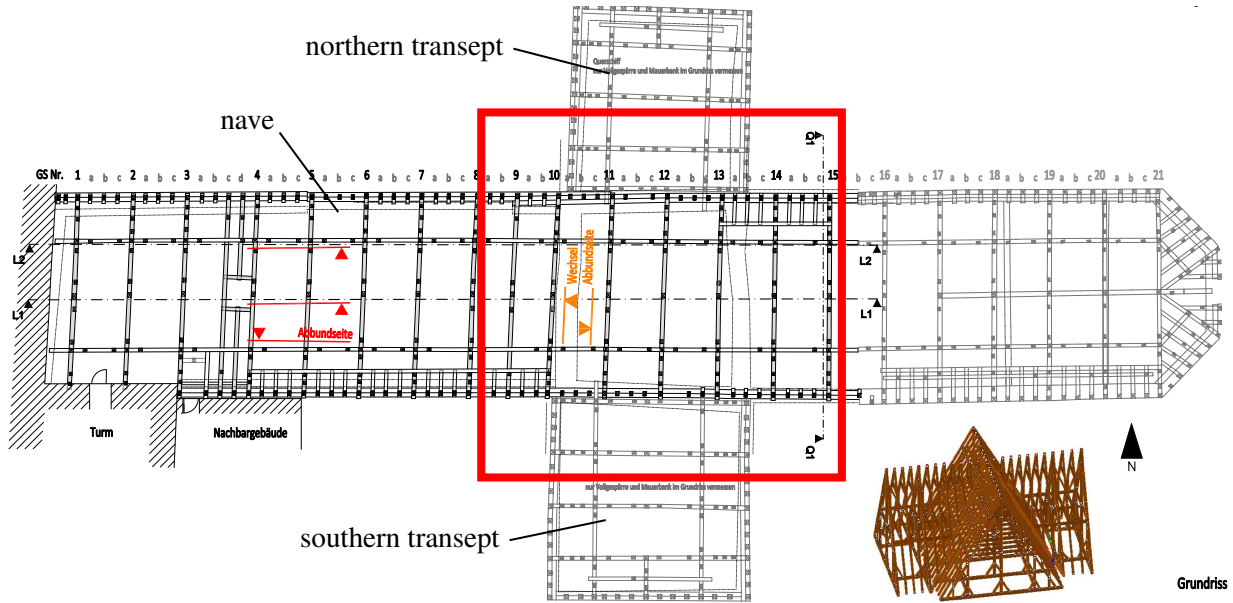


Fig. 8. Ground view of the St. Michaels church.

3.1 Geometry

The main output of the survey of the structure are accurate plans of the roof. In Figure 2, the geometry of the main and intermediate rafters, including the notation of the single elements, as well as the overall dimensions, are shown. Furthermore, the ground view of St. Michaels roof structure is shown in Figure 8.

The nave roof structure consist of 21 main rafters, each with three intermediate rafters in between. The same principle is used for the northern and southern transept parts, which cross the nave structure between the tenth and 14th main rafters. The area modeled in the structural analysis is highlighted by a red box (see Figure 8). In total, the nave structure exhibits a length of 62 *m*.

Beside the geometry of the structure, also the recorded cross-sections of the single elements, have been used as input for the numerical model. The most important cross-sections are given in Table 1.

Table 1. Dimensions of the most important elements of the roof structure.

element	dimension [mm]
tie beam	230/250
king post	200/250
queen post, upper rafter supporting strud	190/210
head runner, joist	200/240
load spreading valley rafter	200/200
rafter, rafter parallel beam, collar beam	190/190
foot brace, top beam, lower rafter supporting strud	190/190
top brace	160/220
elevating belt, wind brace	150/170

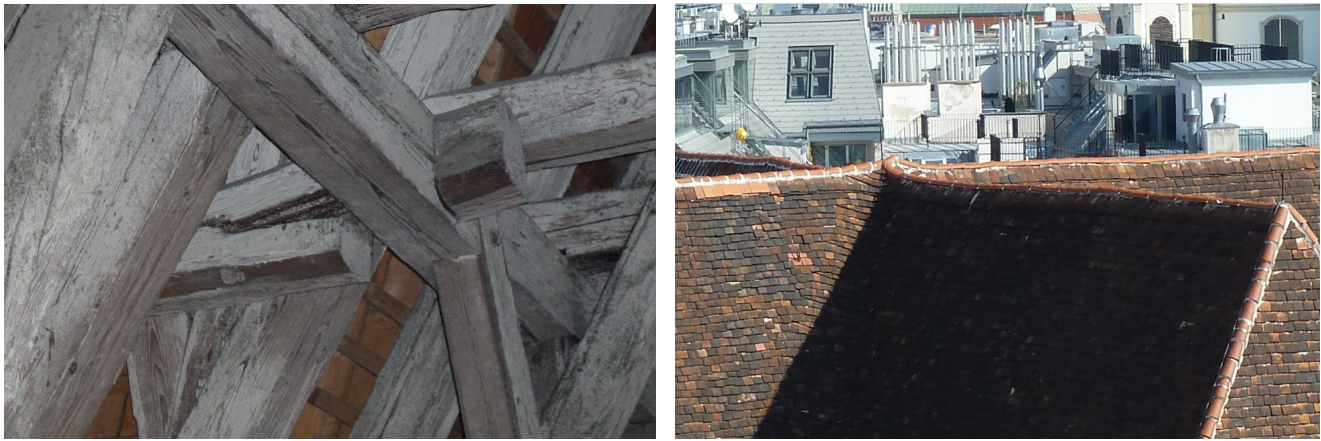


Fig. 9. Longitudinal crack in the notched head runner (left), and deflection of the transept apex (right) at the crossing nave - transept.

3.2 Visual inspection

Beside the results from the building survey and the information extracted from the timber moisture content measurements, the findings from the visual inspection are the only information used in the structural analysis.

During the visual inspection, two areas of moisture stains have been identified. After the timber moisture content has been checked (see Section 3.3), it was clear that this moisture stains are of older date (the timber was dry). It was difficult to identify the usually white colored wet areas, since most elements of the roof structure are covered by some kind of white lime slurry, added decades ago to increase fire resistance of these timber elements (e.g. Figure 9 - left).

A large number of missing elements has been identified, especially missing foot braces, since the roof structure has been made accessible for maintenance purposes in the past, without any compensation measures. These missing elements have not been considered at this preliminary stage of the structural analysis. However, some pathologies related to this missing elements, like twisted king posts have been observed.

The main two defects of this timber roof are related to the crossing of the nave and transept part of the structure. On the one hand, a massive crack in the notched head runner of the transept at the connection to the nave has been identified (see Figure 9 - left). This crack can be found at each of the four head runners, which indicates a structural problem. Close related to the first problem is the second one, which are high deflections of the transept apex in the crossing area nave - transept. The structural analysis attempts to explain these two defects (see Section 3.5).

Finally, the strength and stiffness parameter of the timber elements have been estimated by means of a simple visual grading method, since the methods presented in [2] seem not to be reliable for our purposes. Concluding, the timber quality has been derived from the quality of the used wood, indicated by the number and size of the knots, in combination with the execution quality of the connections. Both, wood and connections, are of very high quality. To be on the safe side, and to consider the age of the timber elements, the strength class C24 according to ÖNORM EN 338 [5] has been chosen.

3.3 Timber moisture content

The measured timber moisture contents are presented in Table 2. The timber surface temperature has been determined to 13.5°C. Spruce has been chosen as wood species for the moisture content measurements.

The wood-decay fungus are organisms which cannot synthesize their required nutrients independently. They prefer wood moisture content from 18 to 180% (optimally 28 – 130%). On the other hand, the optimal values for insects attack are from 20 to 50%. The generalized average value of the considered construction is around 15%. Therefore, the results about moisture content in this roof construction can be defined as dry. The inspected moisture stains are probably of older date. Based on the results from the timber moisture content measurements, the structure is assigned to service class 2 according to EC5 [4]. Service class 2 is characterized by a moisture content between 12% and 20% for softwoods related to a temperature of 20°C.

Table 2. Determined timber moisture contents u_i (St. Michaels church, 2014-03-12).

	Pos.1 south transept lower king post	Pos.2 south transept rafter	Pos.3 nave (east) rafter	Pos.4 nave (west) rafter	Pos.5 chancel area rafter
u_1	15.0%	14.9%	17.2%	15.4%	14.2%
u_2	15.4%	15.4%	15.7%	15.6%	14.5%
u_3	14.4%	14.8%	15.8%	15.1%	14.1%
u_m	14.9%	15.0%	16.2%	15.4%	14.3%

3.4 Dendrochronological examination

The results of the dendrochronological examination show that only softwood, mainly spruce (*Picea abies*), has been used. In addition, some fir (*Pinus sylvestris* L.) and pine (*Abies alba* Mill.) elements have been found. Based on these findings, a softwood strength class according to ÖNORM EN 338 [5] has been chosen for the structural analysis. The number of the sample growth rings ranges from 50 to 150. Most elements have been dated back to the year of 1525, which is the year where the roof structure was new built, after it burnt down in a fire. Also some probably recycled elements, dated back to 1350, as well as some elements from the 19th century have been found. These newer elements indicate later maintenance or adjustment measures of the roof structure.

3.5 Structural analysis

In this section, the overall structural behavior of the roof at the intersection between the nave and transept is presented. Therefore, the distribution of the decisive internal forces, as well as the deflections of the structure are qualitatively shown. Furthermore, the results of the structural analysis are used to explain the defects of the timber beams at the intersection between the nave and transept of the roof structure. These defects are remarkable deflections in the roof at the crossing and a massive crack in the notched head runner of the transept (see Figure 9).

3.5.1 General structural behavior

The structural analysis including compliant connections, carried out within this work, is able to predict the typical behavior of a collar beam roof. Since this structural analysis is a first approximation, only qualitative results are given.

In Figure 10 - left, the distribution of the normal forces within the main rafters is shown. The normal forces presented here are caused by symmetric loading (consisting of dead load and live load). The normal forces marked in red color are related to normal forces in compression, the blue one to tension, respectively. This definition is valid for all normal force distributions presented in this work. In historic roof structures the distribution of the normal forces is a good indicator for the load bearing behavior of the structure.

The external loading on the roof, as well as the dead load of the roof structure causes bending moments and normal forces in compression in the rafters. Since this rafters are connected compliant with the tie beam, a remarkable amount of the load is carried in compression by the beam element parallel to the rafter. The compression forces in the rafter, and in the rafter parallel beam, try to push the roof outwards, which is prevented by the tie beam. Consequently, the tie beam is loaded in tension. The king post acts as a middle support of the tie beam, in order to reduce the span of the latter element. This results in tensional loading of the king post. The load transfer between tie beam and king post is supported by the foot braces, and therefore these elements are also loaded in tension. Based on the same principle, the queen posts are loaded in tension. The collar beams act as support for the rafters, and therefore, are loaded in compression. Remarkable bending moments can be found in the rafters as well as in the tie beam close to the outer supports, where the tie beam is loaded by the rafter parallel beam.

Normal forces

Load combination:

Dead load + live load

tension

compression

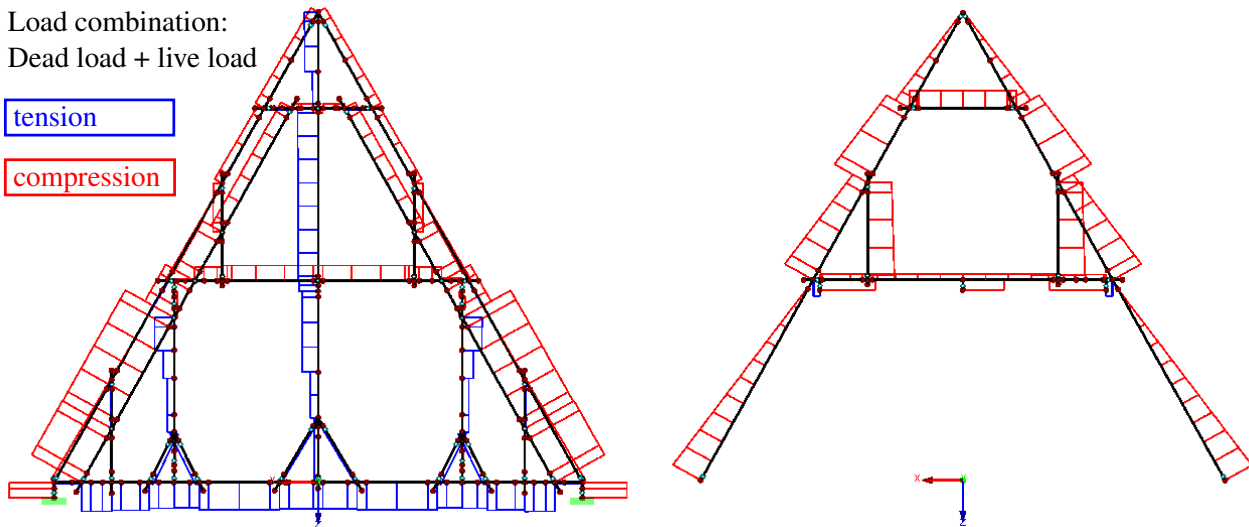


Fig. 10. Normal forces of the main rafters (left) and intermediate rafters (right) under the load combination dead load and live load.

The intermediate rafters (see Figure 10 - right), located between the main rafters, is mainly loaded in compression, since the elements to carry tensional forces are missing. In this case, the loads, which cause tension in the structure, are transferred by the elements in longitudinal direction of the roof structure to the main rafters. The elevating belts, responsible for the stability of the structure in longitudinal direction, located at the level of the queen posts and king posts are mainly loaded by normal forces in tension and compression.

The deflections of the entire structure are presented in Figure 11. It shows the deflections illustrated in the cross section of the nave roof structure, as well as the elevation of the transept part. This Figure supports the explanation of the defects seen at the crossing area. One specialty of this roof structure is related to the intersection of the nave and transept part. The structural system of the nave roof structure along the entire nave, is continuous, while the transept roof structure is set on the nave structure at the intersection between this two structures. Consequently, the valley rafter, which is fixed on the top of the nave rafters, has only a load spreading function. This means the forces of the transept rafters are transferred via the load spreading valley rafter to the rafters of the nave. Consequently, additional loads have to be carried by the elements of the nave structure. Since the cross section of the standard elements have not been strengthened, or additional elements have not been added in the area of the crossing, the extra load lead to additional deflections (see Figure 11). So, the nave structure is subjected to high vertical deflections between the two bearings located at the outer walls. Consequently, this high deflections occur also in the attached transept roof. In addition, each transept rafter is only fixed by one slender forged steel nail on the load spreading valley rafter. Due to this highly compliant connection, the forces are mainly transferred by friction in combination with large deformations. In the following, detailed results of the structural analysis for the crossing between nave and transept are given.

3.5.2 Local structural behavior

The results of the structural analysis for the two main defects of this roof structure are presented in the following. Both defects are related to the intersection between the nave and transept of the St. Michaels church. The root of the problem can be found in the simple design of the crossing area of the roof structure, where the nave part continuous unchanged through this area, while the transept part is set on the nave. The problem of this construction approach is described in the previous section. A more detailed description of this problem, separated for the north and south part of the transept is given in the following. In addition, the results of the structural analysis shall provide a basis for the explanation of the cracks detected at the head runners of the transept at the crossing.

Global deformations

Load combination: Dead load + live load

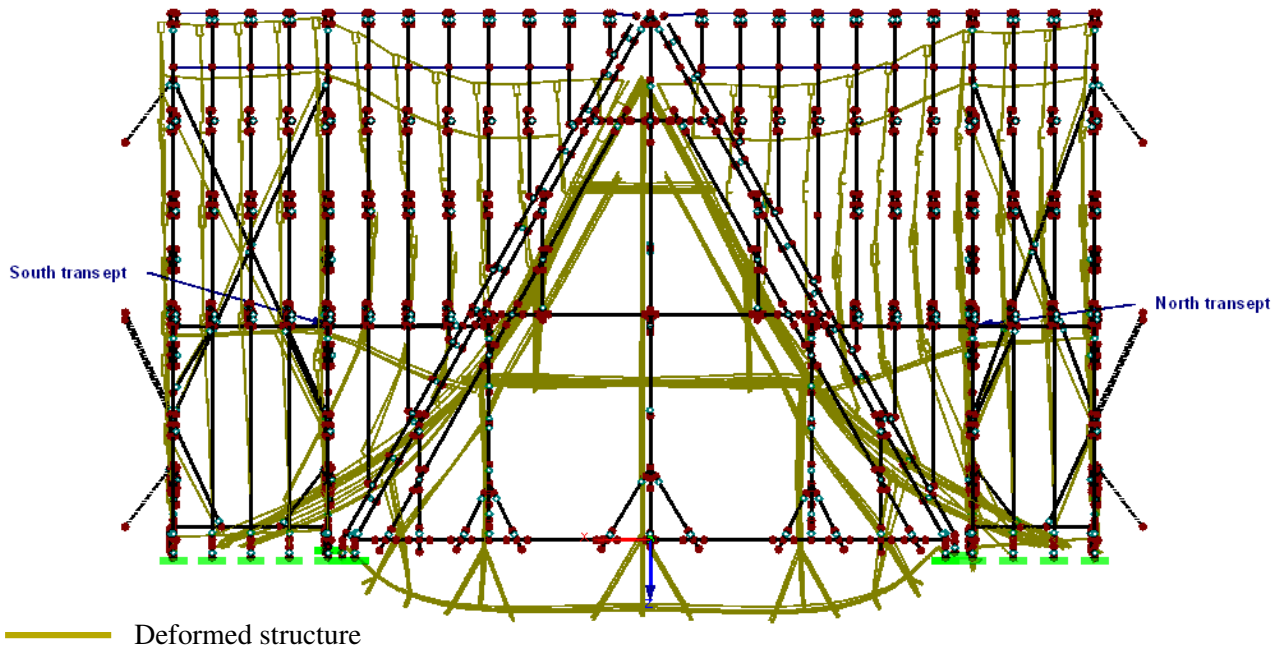


Fig. 11. Global deformation of the crossing nave-transept under the load combination dead load and live load.

First, the deflections in the transept are discussed, which have been detected by on-site visits. It is distinguished between the northern and southern part of the transept, since the execution of the structural system deviates from each other. For both, the northern and southern transept, compliance in axial direction (Δu_x) and transverse direction (Δu_z) for the connection between transept rafter and the load spreading valley rafter are defined. Forces between this two elements are exclusively transferred, by means of friction. The bearing capacity of the slender forged steel nail has been neglected. This kind of load transfer goes hand in hand with large deformation. In simple terms, the rafters slide downwards the load spreading valley rafter under vertical loading. In other words, relative displacements between the rafters of the transept and the valley rafter are generated. This compliance of the connection between the transept rafters and the valley rafter, in combination with the vertical displacement of the load spreading valley rafter, caused by deflection of the nave roof structure, causes the settlement of this “wedge” in the transept roof between apex and the first main rafters, which can be seen on-site (see Figure 9 - left) and also from the result of the structural analysis (see Figure 11).

The difference in the deformed system between the southern part of the transept (Figure 11 - left part) and northern part of the (Figure 11 - right part) can be found in the bearing situation of the tie beam. The tie beam of the first main rafters at the southern transept rests on the outer walls, which is not the case for the northern part. This circumstances results in even higher deformations for the northern part and a softer transition between the highly deflected crossing part and the less deflected main transept part.

Beside this serviceability issue of large deflections at the crossing transept-nave, this weak point in the design also causes problems in the load bearing behavior of the structure. All four head runners of the transept exhibit massive cracks in longitudinal direction of this beams at the connection to the nave roof structure. The deflections extracted from the structural analysis confirm, that the head runners exhibit remarkable relative displacements related to the head runners of the nave. The connection in the head runners of the transept have not been able to carry this large relative displacements, which leads to failure of the connection by means of longitudinal cracks at the notches of the timber beams. Verification of this connection, based on EC5 [4] including the shear-forces from the structural analysis, confirms the detected failure.

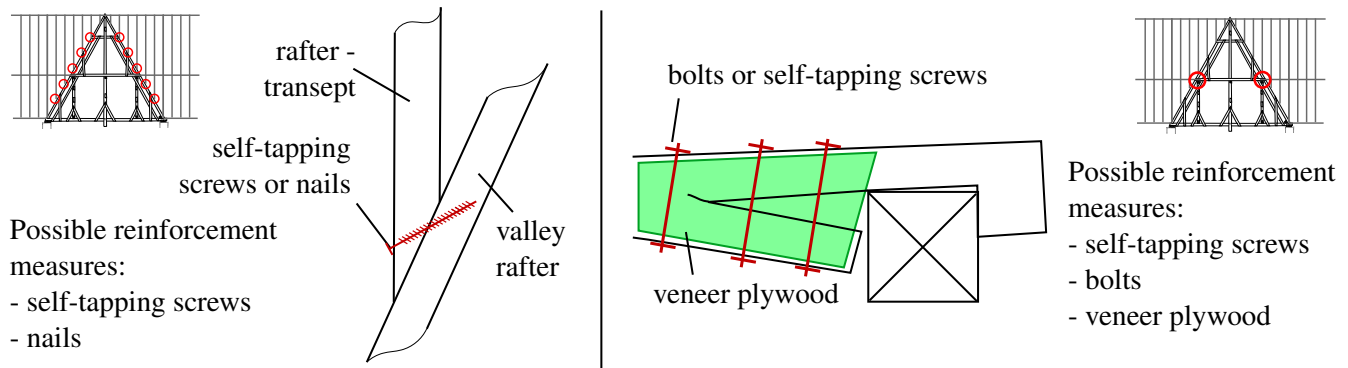


Fig. 12. Reinforcement measures for the connection transept rafter - valley rafter (left) and for the connection head runner transept - nave (right).

3.5.3 Reliability of the structure

In general, the roof structure fulfills the requirements according to the current standards (Eurocode series), based on the first simplified approach of the structural analysis.

However, the deflections in the transept roof structure at the crossing to the nave roof structure may cause problems related to the serviceability of the roof, addressed to the tightness of the roofing. In case of water intrusion, reconstruction of the roof covering may get necessary. Cracks in the head runners of the transept part exhibit no imminent risk for the global stability of the roof structure, based on the results from this first attempt of a structural analysis. However, this connection should be investigated in detail, which may lead to a proposal of reinforcement actions. Some possible reinforcement measures are presented in the following section.

3.6 Reinforcement measures

The only two defects, which may require reinforcement measures are related to the intersection nave - transept. This is on the one hand side the connection of the notched transept head runner to the nave head runner, which exhibits a massive longitudinal crack. On the other hand, high deflections in the transept part next to the intersection nave - transept may cause problems in the future. Since the root of both problems can be found in the weak connection between the nave and transept roof structure, strengthening the connection of these elements may avoid further damages in this area. The following reinforcement measures may be applicable:

Connection transept rafters - load spreading valley rafter

This reinforcement action shall prevent the transept rafter from further sliding down along the load spreading valley rafter, in order to avoid an increase of the deflections in the transept roof structure, which may cause serviceability problems due to water intrusion. Possible reinforcement measures are to connect this two elements by self-tapping screws or nails, in order to transfer shear forces between the transept rafter and the valley rafter (see Figure 12 - left). In addition, this reinforcement action increases the position stability of the rafter.

Connection head runner transept-nave

Reinforcement measures at the head runner are addressed to prevent further cracking of this connection, in order to avoid stability and serviceability problems. Additional elements to carry tensional forces perpendicular to the grain have to be added. This elements may be self-tapping screws or bolts in transverse direction of the connection, or glued on engineered timber plates, like veneer plywood (see Figure 12 - right).

4 Discussion/conclusions

The goal of this work is to improve the understanding of the structural behavior of the timber roof of St. Michaels church. Particularly, the structural analysis carried out within the assessment of this structure should serve as basis for the explanation of the observed defects of the timber roof.

The geometry of the structure as well as of several connections has been recorded to serve as input for the numerical model. Furthermore, a visual inspection including the measurement of the timber moisture content

provides a first approach of the stiffness and strength situation of the structure. Dry wood has been detected at several investigation points at the time of measurement. In addition, a dendrochronological examination has been carried out to provide information about wood species and age of the single elements, which again may affect the strength and stiffness properties.

All these information together act as input for a first structural analysis, including compliant connections. Although several simplifications have been done, the results of the structural analysis are of sufficient accuracy. The model is able to depict the real structural behavior observed on-site. Furthermore, the observed defects could be reproduced by the model. Concluding from the results of the structural analysis, reinforcements strategies have been suggested. At this point, it should be highlighted that no immediate danger for the structure is obvious from this preliminary assessment. The reinforcement should only prevent the structure from future damages. In summary, the structural analysis of this historic timber roof structure has been able to give a first statement on the reliability of the structure based on the current design standards.

However, further investigations should be carry out in order to give a comprehensive statement about the reliability of the structure. Particularly, the model for the compliance of the connections should be refined to better reflect the real behavior of the connections. This may result in an even more realistic deformation behavior of the timber structure. In addition, a more accurate determination of the strength and stiffness properties of the timber elements should be carried out in the future.

Acknowledgements

Thanks go to our colleagues form the Institute for History and Theory of Architecture and Historic Building Survey, Vienna University of Technology for providing us with the results from their building survey of the St. Michaels church. Furthermore, we would like to thank Dr. Michael Grabner (University of Natural Resources and Life Sciences, Vienna) for the data of the dendrochronological examination. Thanks go to the priest of the St. Michaels church, Dr. Peter van Meijl, who provided us access to the roof structure of his church.

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Monday 9th December?

8:20 – 9:00

Welcome

Pierre Dehombreux, Vice-Rector for International Relations, University of Mons.
Dina D'Ayala, Chairwoman of COST FP1101, University College of London.

Wood Structural Engineering in Belgium.

A. Rossignon, J-C Goffaux and S. Maréchal., Belgian Wood Engineers Association.

9:00 – 9:50

In situ assessment - Introduction and overview of methods. Part 1

Bo Kasal, TU Braunschweig and Fraunhofer Institut für Holzforschung, Germany.

9:50 – 10:10

Coffee break (ground floor) – registration for the dinner

10:10 – 11:00

In situ assessment - Introduction and overview of methods. Part 2

Bo Kasal, TU Braunschweig and Fraunhofer Institut für Holzforschung, Germany.

11:10 – 12:00

In situ assessment - Introduction and overview of methods. Part 3

Bo Kasal, TU Braunschweig and Fraunhofer Institut für Holzforschung, Germany.

12:00 – 13:10

Lunch (by your own)

13:10 – 14:00

Dendrochronology.

Patrick Hoffsummer, University of Liège, Belgium.

Old carpentry connections.

Thierry Descamps, University of Mons, Belgium.

14:10 – 15:00

15:20 – 17:00

***Demonstrations on site and discussions:
Town Hall of Mons***

- **Bo Kasal**, TU Braunschweig and Fraunhofer Institut für Holzforschung, Germany
- **Ron Anthony**, Wood Scientist for Anthony & Associates, Inc. in Colorado, U.S.A
- **Patrick Hoffsummer**, University of Liège, Belgium

Tuesday 10th December?

8:30 – 9:50

Keynote lecture: Ron Anthony. Wood Scientist for Anthony & Associates, Inc. in Colorado, U.S.A

Understanding the Link between Timber Behavior and Assessment.

- What is the wood telling me? Most people use NDT during an assessment, often without looking closely at the structure, the timber, and the connections. Moisture stains, distress and displacement, and an understanding of loads (current and prior) can often tell us more than any combination of NDT techniques.
- Wood mechanics and structural behavior. Many engineers and architects have a limited understanding of wood mechanics yet there is considerable interest in characterizing the in-situ behavior of timber.
- Physics, statistics, and nondestructive testing. What is the role of physics and statistics in nondestructive testing ?

9:50 – 10:10

Coffee break (ground floor) – registration for the dinner

10:10 – 11:00

FRP, introduction, basis of design, material, bonding

Annette Harte, Civil Engineering College of Engineering and Informatics, National University of Ireland, Galway, Ireland.

11:10 – 12:00

FRP, flexural design, prestressed FRP

Annette Harte, Civil Engineering College of Engineering and Informatics, National University of Ireland, Galway, Ireland.

12:00 – 13:10

Lunch (by your own)

13:10 – 14:00

Timber-concrete composites in reconstruction : overview and modeling

Kay-Uwe Schober, Department of Civil Engineering, Timber Engineering and Structural Design, University of Mainz, Germany.

14:10 – 15:00

Glued in rods : analytical investigations and developments

Kay-Uwe Schober, Department of Civil Engineering, Timber Engineering and Structural Design, University of Mainz, Germany.

15:10 – 16:00

Workshop I

Glued in Rods and Reinforcements.

D. Smedley, Rotafix UK.

Workshop II

Wood thermography.

O. Carpentier & E. Antczak, University of Artois, France.

Workshop III

New devices for in-situ assessment.

M. Kloiber; Centre of Excellence, Telč, Czech Republic.

16:00 – 16:20

Coffee break (ground floor)

16:20 – 17:10

Workshop I

Glued in Rods and Reinforcements.

D. Smedley, Rotafix UK.

Workshop II

Wood thermography.

O. Carpentier & E. Antczak, University of Artois, France.

Workshop III

New devices for in-situ assessment.

M. Kloiber; Centre of Excellence, Telč, Czech Republic.

Wednesday 11th December?

8:30 – 9:50

Keynote lecture : Thomas Tannert. Associate Chair in Wood Building Design and Construction at the University of British Columbia in Vancouver, Canada

Assessment and Reinforcement of Glulam Beams.

- Practical examples of the assessment of Glulam beams: problematic of using small samples and then make inference on the structural integrity.
- Analytical model developed by Dietsch that allows calculating reinforced Glulam beams.
- Practical experience

9:50 – 10:10

Coffee break (ground floor)

10:10 – 11:00

Wooden reinforcements.

Laurent Bléron, Université de Lorraine, ENSTIB / LERMAB, Epinal, France.

11:10 – 12:00

Reinforcement perpendicular to the grain with self tapping screws.

Damien Lathuillière, Université de Lorraine, ENSTIB / LERMAB, Epinal, France.

12:00 – 13:10

Lunch (by your own)

13:10 – 14:00

FRP, applications and examples

Annette Harte, Civil Engineering College of Engineering and Informatics, National University of Ireland, Galway, Ireland.

14:10 – 15:00

Modeling of historical connections

Kay-Uwe Schober, Department of Civil Engineering, Timber Engineering and Structural Design, University of Mainz, Germany.

15:00 – 15:20

Coffee break (ground floor)

15:20 – 17:00

Practitioner's view: Frank Rinn. Rinntech, Heidelberg, Germany

Proven concept of timber inspection and result documentation

How to use stress wave tomography and other methods to inspect timber and how to document the results for engineers, architects and carpenters ?

How wood anatomy influences the results of technical timber inspection ?

This aspect is often not understood. Even many experts do not know how strongly wood anatomy influences the results of localized measurements. And this often leads to misinterpretations results and wrong recommendations.

Diner in town (7 PM)

Thursday 12th December?

8:30 – 9:00

9:00 – 9:50

Stability of timber frames and slabs : weaknesses and reinforcement

Benoit Parmentier, Audrey Skowron. Centre Scientifique et Technique de la Construction, Brussels, Belgium.

9:50 – 10:10

Coffee break (ground floor)

10:10 – 11:00

Timber columns and walls.

Wen-Shao Chang, Department of Architecture and Civil Engineering, University of Bath, UK.

11:10 – 12:00

All wood connection.

Wen-Shao Chang, Department of Architecture and Civil Engineering, University of Bath, UK.

12:00 – 13:10

Lunch (by your own)

13:10 – 14:00

Enhancements using FRP profiles

Gary Raftery, Department of Civil and Environmental Engineering, University of Auckland, New Zealand.

14:10 – 15:00

Connection of sandwich panels to wooden joists

Lieven De Boever, Kurt de Prooft, Patrick Van Den Bossche, Belgian technical Centre of wood transformation and furniture, Brussels, Belgium.

15:00 – 15:20

Coffee break (ground floor)

15:20 – 17:00

Practitioner's view: Albino Angeli. Rothoblaas, Italy

Case of study 1: timber-timber floor.

Case of study: Castel Belasi and the tests made in collaboration with the University of Trento

Case of study 2: self tapping screws.

Reinforcement about the joints of an existent roof made with self tapping screws.

Friday 13th December?

8:30 – 9:50

Keynote lecture : Milan Vatrovec. Simpson Gumpertz & Heger Inc*, New York, U.S.A

Diagnosing and Addressing Performance Issues of Wood Structures in Service – An Engineer's Perspective.

- How engineers can benefit from wood science and technology knowledge?
- Real projects and examples on bad strengthening.

9:50 – 10:10

Coffee break (ground floor)

10:10 – 11:00

Bridges: static, reinforcing techniques. Part I

Roberto Crocetti, Department of Structural Engineering, Lund University, Sweden.

Bridges: static, reinforcing techniques. Part II

Roberto Crocetti, Department of Structural Engineering, Lund University, Sweden.

12:00 – 13:10

Lunch (by your own)

13:10 – 14:00

Dowel Type Connections. Part I

Jan-Willem Van de Kuillen, Chair of Wood Technology, TU Munich, Germany & Chair for Timber Structures and Wood Technology, Delft University of Technology, the Netherlands.

Dowel Type Connections. Part II

Jan-Willem Van de Kuillen, Chair of Wood Technology, TU Munich, Germany & Chair for Timber Structures and Wood Technology, Delft University of Technology, the Netherlands.

14:10 – 15:00

15:00 – 15:20

Coffee break (ground floor)

15:20 – 16:10

Reinforcement of delaminated glulam beams.

Robert Widmann, Structural Engineering Research Laboratory. EMPA, Materials Science and Technology, Switzerland

Design and reinforcement of notched beams.

Robert Widmann, Structural Engineering Research Laboratory. EMPA, Materials Science and Technology, Switzerland

16:20 – 17:10

Closing

Key note lecturers

Ron Anthony. Wood Scientist for Anthony & Associates, Inc. in Colorado, U.S.A



Ron Anthony, received a Master of Science degree in Wood Science and a Bachelor of Science degree in Wood Science and Technology from Colorado State University. Since 1985 Mr. Anthony has been active in research and development of nondestructive evaluation technologies for timber structures. His consulting activities focus on the application of innovative inspection technologies for the assessment and conservation of timber structures. He has participated in or led investigations of numerous historic structures and workshops in the U.S., Europe, and Asia.

He also conducts forensic investigations on wood-related failures and has been involved with the development of standards and specifications for a variety of structural applications of wood. Mr. Anthony has authored over 100 publications; participated in conferences and seminars; and consulted in North America, Europe, Asia, Africa, Australia, and the South Pacific. Mr. Anthony is the 2002 recipient of the James Marston Fitch Foundation Grant for his approach to evaluating timber in historic structures.

Thomas Tannert. Associate Chair in Wood Building Design and Construction at the University of British Columbia in Vancouver, Canada.



Dr. Thomas Tannert holds a joint appointment in the Departments of Civil Engineering and Wood Science at UBC. Dr. Tannert received degrees in from Germany, Chile and Canada, and from 2008 to 2011 worked at the Bern University of Applied Sciences in Switzerland. During that time he was secretary of the RILEM committee AST-215 and co-edited the Springer book "In-situ Assessment of Structural Timber". Furthermore, Dr. Tannert successfully proposed two international research networks: 1) the European COST Action FP1101 "Assessment and Monitoring of Timber Structures", and 2) the RILEM committee RTE-245 "Reinforcement of

Timber Elements" which he chaired from 2011 to 2013. Dr. Tannert's current research focuses on the development of design guidance for timber joints and structures considering seismic loading.

Milan Vatovec. Simpson Gumpertz & Heger Inc*, New York, U.S.A



Dr. Milan Vatovec is a Senior Principal at Simpson Gumpertz & Heger Inc. (SGH) and Head of Structural Engineering Practice at the firm's office in New York. In his seventeen years at SGH, he has worked on more than 400 different projects involving evaluation and structural design for repair or modification of various existing wood, concrete, masonry, and steel structures in the United States. Dr. Vatovec is a member of the Wood

Committee at ASCE/SEI and of the ACI Committee 440 on Fiber Reinforced Polymer Reinforcement. Dr. Vatovec has written and presented extensively in the United States, including an award-winning paper on the topic of structural evaluation and concrete strengthening of an existing garage. As an adjunct professor, Dr. Vatovec currently teaches two graduate-level classes at Manhattan College: a structural renovation, repair and rehabilitation class, and a wood-structures design class.

* Simpson Gumpertz & Heger (SGH) is an American engineering firm that designs, investigates, and rehabilitates structures and building enclosures. More than 400 employees in five offices: Boston, Los Angeles, New York, San Francisco, Washington, DC. Named a "Best Firm to Work For" by Structural Engineering & Design and CE News magazines for the past five consecutive years.

TRAINING SCHOOL

GRANT LETTER - version 1.1 - CGS

Mr Valentin Atanasov

Vasil Levski, bl. 35, ent. B, fl. 1 ap. 11

Kazanlak

Bulgaria

Subject Trainee Grant

Reference COST Action FP1101

Host institution: University of Mons

Period: 09/12/2013 to 13/12/2013

Reference code: COST-TS-ECOST-TRAINING_SCHOOL-FP1101-091213-034167

Amount up to: EUR 600

Dear Mr Valentin Atanasov,

With reference to your application for a Training School, I am pleased to inform you that the Management Committee of the COST Action **FP1101**, has awarded you a fixed grant amount of **EUR 600**, for the above mentioned Training School.

The funding will be spent according to the Guidelines for "Training School" that can be found on <http://www.cost.eu/participate/networking>. The training school support grant is intended to cover travel and subsistence expenses, but no lecture fee.

Please note that you should start your mission at the foreseen time and inform the Grant holder about any change in advance of the training school.

The payment will be carried out by the Grant Holder Institution to the bank coordinates provided by you in your e-cost profile.

Yours sincerely,

Dr Juergen HINRICHSEN

Grant Holder

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