SURVEY AND REALISTIC MODELLING OF ANCIENT AUSTRIAN ROOF STRUCTURES PART I

Andreas Meisel[1]

Thomas Moosbrugger[2]

Gerhard Schickhofer[3]

[1,2,3] Graz University of Technology, Graz, Austria

Abstract

Different traditional roof structures have been developed in various regions of Austria. Despite fires, wars, degeneration and demolition, a great number of these roof structures (some dating back as many as six hundred years) have stood the test of time. To perform a structural analysis, a detailed inspection of the as-built structure, incorporating geometry, joints, strength, support conditions and possible damages of the structure, is essential.

The structural analysis of ancient roof structures is full of difficulties and uncertainties. Consider, for example the reference object, a "Grazer Dachstuhl", which demonstrates that the representation of the spatial load-carrying behaviour and the determination of flexibilities and eccentricities of joints are both time-consuming and problematic. However, both effects highly influence the results due to the fact, that ancient roof structures represent highly statically indeterminate systems. A realistic reconstruction of the load-carrying behaviour of these hybrid structures is only possible when the mechanical behaviour of the joints is considered.

In general, ancient roof structures are redundant, and thereby robust, structures. As a result of load-redistributions, the beams and joints are able to keep up the capacity of the whole structure, despite of frequent and partial damage.

INTRODUCTION

Recently many attics of ancient roof structures are converted to residential space. At the same time many roofs are in need of rehabilitation caused by poor maintenance and incorrect structural modifications. Because of building standards as well as economical and ecological reasons, the demolition is often out of question. In addition to conversions, historically valuable roof structures should be preserved for future generations.

The knowledge of the mechanical behaviour of these roof structures is the basis for repair concepts. The representation of 'reality' in an uncomplex as possible model, is a great challenge.

At the moment, many repairs are accomplished by craftsmen, without a complete structural analysis. Hence the level of structural safety can hardly be quantified. In the case of repairs supported by an engineer analysis, the load-carrying capacity of the stock is often underestimated as a result of neglecting the behavior of joints and the spatial load-carrying behaviour. Repair measures based on such calculations are mostly contradictory to saving costs and preserving the historical structure.

PROBLEMS

The erection of ancient roof structurs was carried out by hands-on experiences, fine traditions and the courage of craftsmen. For this reason a lot of characteristics have to be considered while performing the structural analyses:

- Ancient roof structures frequently carry the loads distinctly three-dimensional. Hence a simplified analysis of these structures, considering only plane parts of the system (two-dimensional), is often hard to realise or completely impossible. The interaction of the substructures has to be taken into account.
- The consideration of the flexibilities and eccentricities of the carpentry joints for the structural analysis is required according to EN 1995-1-1 (see [12], [13]). The mechanical behaviour of the joints highly influences the distribution and magnitude of the internal forces. Concerning this matter, the specifications of the engineering standards and the literature are unsatisfactory.
- The definition of appropriate support conditions is difficult.
- The determination of appropriate material constants is time-consuming.
- The dimensions of the system and beams show wide variations caused by the manufacturing through manual labor.

AIM OF THIS STUDY

In this contribution it will be shown exemplarity, how the load bearing behaviour of an ancient roof can be modeled as realistic as possible, so that a structural analysis is at least possible on a low level of safety.

THE „GRAZER DACHSTUHL“

Introduction

Single surveys (see [10]) and interviews with an experienced carpenter have shown, that a lot of so–called „Grazer Dachstühle“ have been built in Graz in the 19th century. The roof structure of the house „Mandellstraße 9“ (Fig. 1, 2 und 3) was chosen as reference object for this paper. According to the archive of the city of Graz the house was erected in 1867 (see [8]).

The Structural System of the „Grazer Dachstuhl“

The structural system of the „Grazer Dachstuhl“ can be described as collar beam roof with braced, double upright principal frame and jamb wall. As it is explained in (see [10] chapter 1.2), the description „collar beam roof“ means, that the rafters are not supported by the purlins (or longitudinal beams). However the typical tie member, which is a necessary element of every collar beam roof, does not exist in the „Grazer Dachstuhl“. So it represents a hybrid system consisting of a rafter and a purlin roof, demonstrating the transition from the rafter to the purlin roof in the 19th century.

The principal frames of these structures have been built with braces, so almost all wind forces are picked by the principal frames. Finally the term „double upright“ indicates, that the frames are supported from two frame walls. Furthermore it is remarkable, that the tie-beams run parallel to the ridge (as members of the frame walls) and transfer the loads from the posts to the cross walls of the storey below the attic.

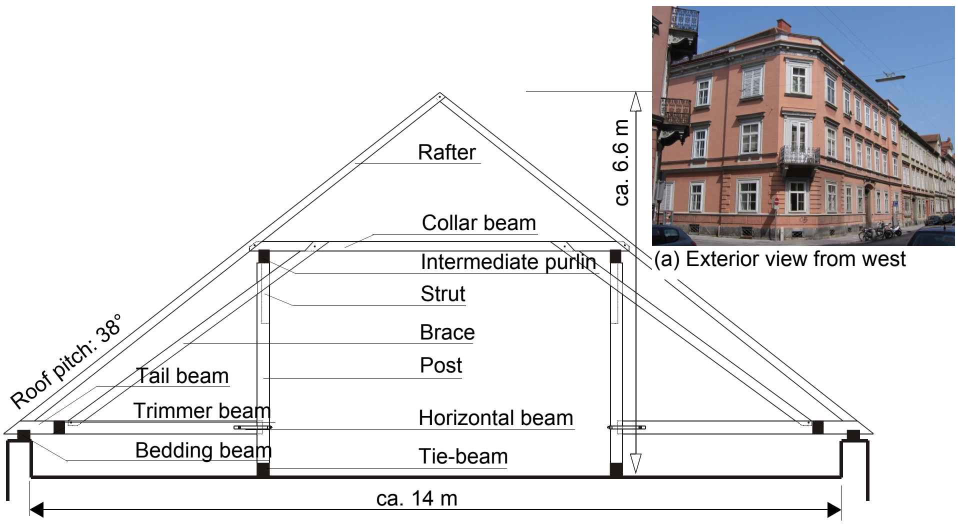


Fig. 1. Typical cross-section of a principal frame.

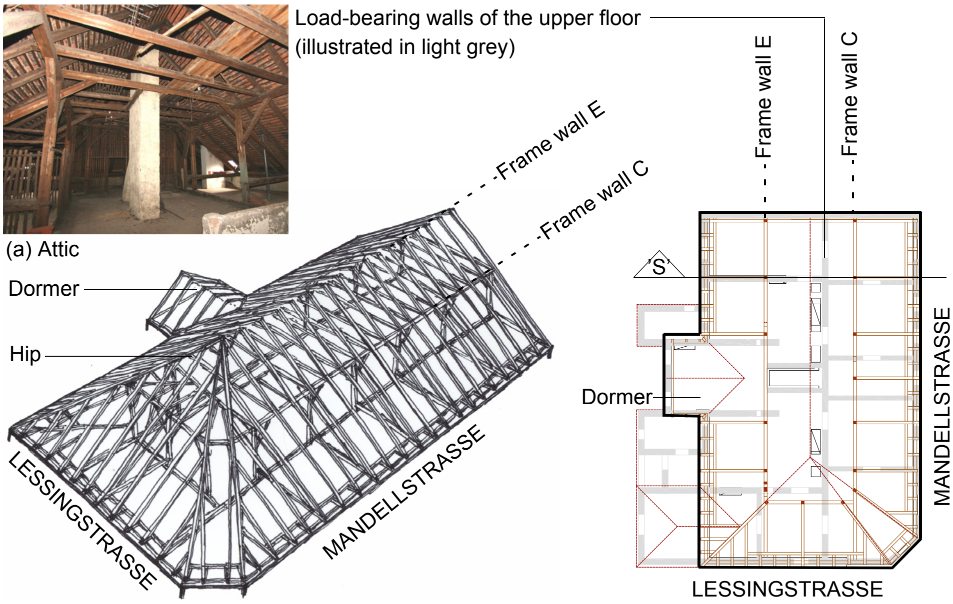


Fig. 2. Left: Perspective of the spatial structure (without chimneys), Right: Floor plan.

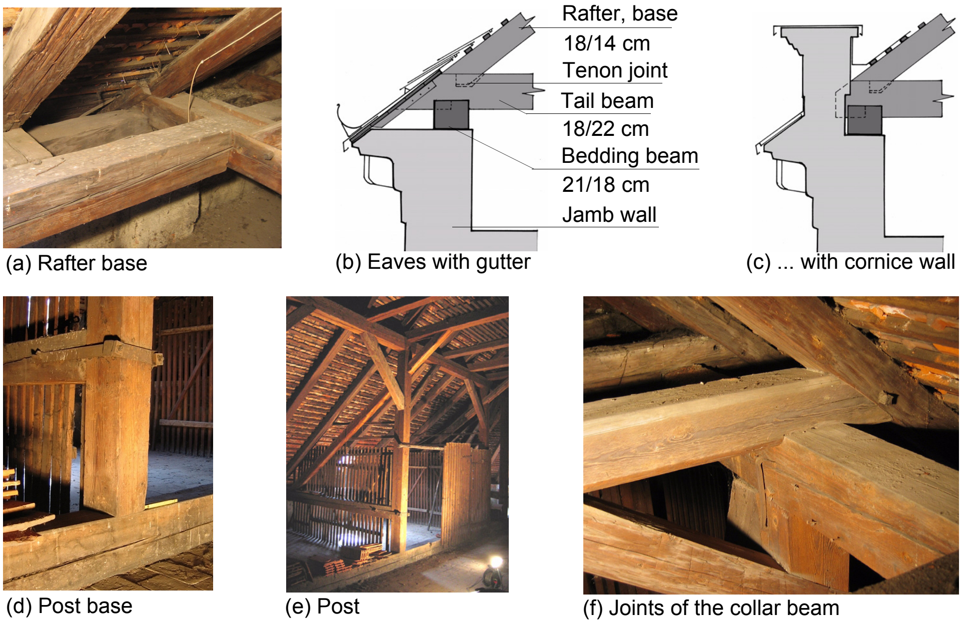


Fig. 3. Typical joint of the „Grazer Dachstuhl“.

Carpentry Joints

Carpentry joints were chosen based on the intended area of application and built according hands-on experiences, fine traditions and the courage of the craftsmen (see [3], [4], [16]). Due to this fact and the complex orthogonal-anisotropic mechanical behaviour of timber, the real load bearing capacity of historical joints can hardly be modeled by simple mechanical models.

Problems

According to EN 1995-1-1 the flexibilities of the structural parts and joints as well as the eccentricities of the joints have to be taken into account for the computation of the internal forces (see [12] 5.1 (4) and (5), [13] B.1). The analysis in (see [10]) have illustrated, that the derivation of the stiffness values, eccentricities and also design values of these joints according to EN 1995-1-1 (see [12], [13]) is often problematic or even impossible.

Additionally, the corresponding literature (for instance see [4], [5]) covers only special types or restricted geometries of these joints without the possibility of general conclusions. The main challenges during the derivation of the mechanical behaviour of carpentry joints are the following:

- Usually there is an interaction of connectors in carpentry joints. Relevant statements of the load-displacement behaviour of the whole connection can only be made with the knowledge of a combination coefficient that considers the interaction of different connection stiffnesses.
- EN 1995-1-1 (see [12], [13]) declares no rules for the determination of the material strengths under combined stresses. Nevertheless combined stresses (for instance lateral pressure or lateral tension and shear) often appear in carpentry joints.
- Carpentry joints transfers compression and tensile forces principally via contact pressure. Frequently, high local lateral pressure stresses – and even more problematic – lateral tension stresses result from the geometry. Those cannot be determined quantitatively without complex numerical simulations, because the interaction of stresses, reference faces and distribution of stresses are unknown.
- To a large extent the eccentricities, stiffness values and design values of numerous joints depend on the type of loading. For example, tensile forces in a skew tenon are only transferred by the treenail as the skew tenon itself is unable to bear tensile forces (geometrical non-linearity).

The flexibilities and eccentricities of joints using the example of a skew frontal tenon

Until the beginning of the 20th Century so called skew frontal tenons (see figure 4) were frequently used for joints of struts to posts and purlins. In the following, a method for estimating the magnitude of stiffness values and eccentricities of such joints loaded in compression is demonstrated.

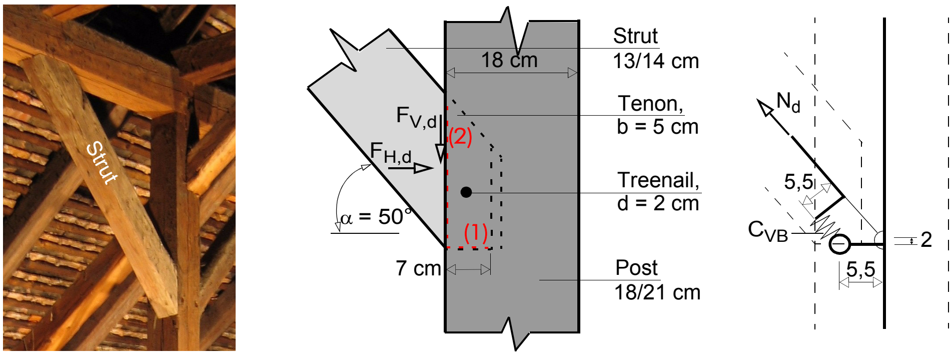


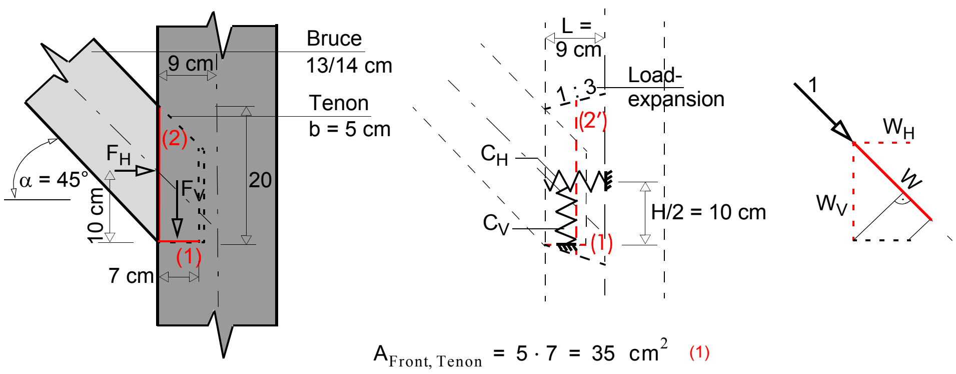
Fig. 4. Left: Strut, Middle: Construction of the skew frontal tenon,
Right: Structural model.

Structural model

According to [1], [4], [10] all carpentry joints are modelled as flexible joints of beams. The load transfer of $F_{V,d}$ und $F_{H,d}$ is carried out at different contact surfaces, so that the definition of the eccentricity is an approximation. The worst case szenario was chosen. This assumes that there is a gap in the contact surface (2) between the end grain of the strut and the post respectively the purlin, and so all forces are transferred due to friction in the head of the tenon (1).

The eccentricity of the strut in the wall frame and the existence of the treenail are not considered.

The derivation of the spring stiffness C_{VB} (without friction)



$$A_{Front,\,Tenon} = 5 \cdot 7 = 35\ \text{cm}^2 \quad (1)$$

$$A_m = \left(20 + 2 \cdot \frac{9}{2} \cdot \frac{1}{3}\right) \cdot (13 - 5) = 184\ \text{cm}^2 \quad (2')$$

Fig. 5. Reduced geometry and model for the derivation of the spring stiffness.

Assumptions

- The load F_V is only transferred in the contact surface (1), the load F_H is only transferred in contact surface (2). There is no interaction between these forces due to friction, etc.
- Both timber contact surfaces are plane, closed and stay like this during deformation of the structure.
- The force F_V has to be transferred into the tenon up to the centroidal axis of the strut. The force F_H has to be transferred up to the centroidal axis of the post.
- All eccentricities are neglected.
- $E_{\alpha,mean}$ is calculated on the basis of ((6.16) [12]).

Spring stiffnesses

$$E_{\alpha,\,mean} = 71,6 \ \ \mathrm{kN/cm^2} \qquad\qquad E_{90,\,mean} = 37 \ \ \mathrm{kN/cm^2}$$

$$C_V = \frac{E_{\alpha,\,mean} \cdot A_{Front,\,Tenon}}{H/2} = 251 \ \ \mathrm{kN/cm} \qquad\qquad C_H = \frac{E_{90,\,mean} \cdot A_m}{L} = 756 \ \ \mathrm{kN/cm}$$

$$C = \frac{1}{W} = \frac{\sqrt{2}}{W_V + W_H} = \frac{\sqrt{2}}{\frac{1/\sqrt{2}}{C_V} + \frac{1/\sqrt{2}}{C_H}} = 377 \ \ \mathrm{kN/cm}$$

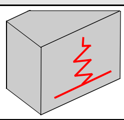
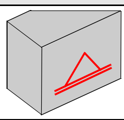
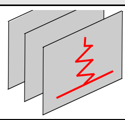
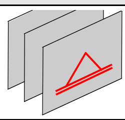
Caused by gaps in the contact surfaces, it can be assumed that the spring stiffness is lower than estimated above. Therefore, for further calculations spring stiffness is set to $C_{VB} = 300$ kN/cm.

MODELLING OF THE „GRAZER DACHSTUHL“

Models

In all models, the structure is represented as a beam system with perfectly straight longitudinal axis. Geometric and material imperfections are taken into account during the design with the model column method. The following different structural models of the roof structure of the house ’Mandellstraße 9’ are studied in RSTAB [2].

Table 1: Overview of the investigated mechanical models.

	3D		2D	
Models	M1	M2	M3	M4
Sketch				

As illustrated in Table 1, in the models M1and M2 the structure is represented by a spatial model. By contrast, in the models M3 and M4 the structure is decomposed to plane subsystems.

- Model M1 represents the distinctive spatial load-carrying behaviour as well as the flexibilities and eccentricities of the joints as realistic as possible. Nonlinear effects like failed beams and supports as well as nonlinear springs are taken into account. As the superposition of load cases is therefore no longer allowed, load groups have to be defined according to the theory of first order.
- The difference between model M1 and M2 is that for M2 a linear calculation neglecting failed beams and supports as well as the flexibilities and eccentricities of the joints is performed.
- Model M3 tries to describe the load-bearing of the structure by the use of plane subsystems by considering as far as possible the interaction of sybsystems. For the determination of the support springs, substituting the interaction of sybsystems, the stiffnesses of model M1 are used. The flexibilities and eccentricities of the joints are taken into account.
- In model M4 all interactions of subsystems and the mechanical behaviour of joints are neglected.

Global load bearing

Around 46 % of all vertical loads of the roof structure are transferred by the frame walls to the masonry cross walls of the floor under the attic, 54 % are carried by the jamb walls. As comparative calculations have shown, the frame walls of an analog purlin roof would transfer 56 % of all vertical loads. Consequently it appears that in the „Grazer Dachstuhl“ the loads are transferred both as a rafter roof and as a purlin roof. Thus the load bearing of the rafter roof involves redistributions of vertical loads from the braced, double upright principal frame to the jamb walls in the order of magnitude of 20 %.

The horizontal forces of the loadbearing behaviour of the rafter roof are transferred through the tail-, trimmer and bedding beams to the horizontal beams. The horizontal beams carry the tensile loads to the frame wall plane, where they can be introduced into the cross walls of the floor under the attic by friction, due of the high vertical loads of the posts (see Fig. 6).

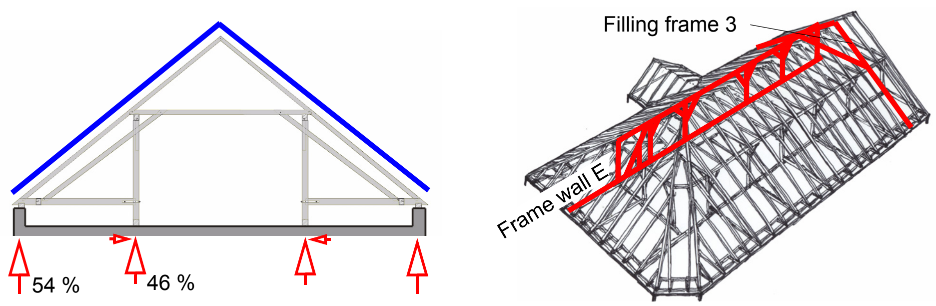
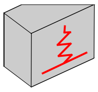
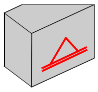
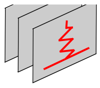
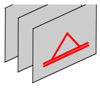


Fig. 6. Left: Load bearing, Right: Frame wall E and filling frame 3.

Results of the computation of the internal forces (ULS–design)

Table 2: Results [kN or kNm and percent] for filling frame 3.

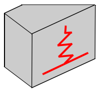
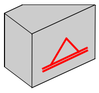
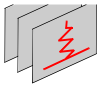
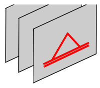
Models	*M1*		*M2*		*M3*		*M4*	
Vertical supporting force truss wall E	*16.24*	*100 %*	*16.64*	*102 %*	*14.75*	*91 %*	*21.49*	*132 %*
Horizontal supporting force truss wall E	*1.36*	*100 %*	*1.12*	*82 %*	*0.87*	*64 %*	*0.89*	*65 %*
Axial force of the rafter base	*-13.34*	*100 %*	*-11.80*	*88 %*	*-11.50*	*86 %*	*-3.66*	*27 %*
1. Field moment of the rafter	*4.75*	*100 %*	*4.70*	*99 %*	*4.75*	*100 %*	*4.64*	*98 %*
Min. moment of the rafter	*-5.73*	*100 %*	*-5.79*	*101 %*	*-5.72*	*100 %*	*-5.93*	*103 %*
1. Field moment of the rafter	*2.31*	*100 %*	*2.46*	*106 %*	*2.50*	*108 %*	*2.41*	*104 %*

Selected support and internal forces of filling frame three are illustrated in Table 2. The two-dimensional computations deviate from the results of the spatial analysis due to the simplifications of support conditions in the plane subsystems. As only fixed or free supports are used in model M4, the axial forces in the rafter base show large, deviations in comparison to the results of the other models. It has to be pointed out that the reaction forces and the internal forces in the area of the supports given by model M4 are wrong.

All investigated models showed approximately the same bending moments (and lateral forces), due to the relatively low bending stiffness of the rafters. Slight deviations result for example from the fact, that the eccentrics of the joints in model M1 changes the span lengths and support conditions of the rafters. This particularly influences the internal forces of the 2nd rafter field.

The direkt comparison of the results of model M1 and M2 to the results of model M3 und M4 is not allowed because in the latter models the loads transferred from the frames are applied as smeared along the purlin. Thereby in the area of the dormer, oversized vertical loads on the intermediate purlin are taken into account.

Table 3: Results [kN or kNm and percent] for the frame wall.

Models	*M1*		*M2*		*M3*		*M4*	
Max. moment intermediate purlin	*15.03*	*100 %*	*10.34*	*69 %*	*27.20*	*181 %*	*17.31*	*115 %*
Min. moment intermediate purlin	*-17.68*	*100 %*	*-13.84*	*78 %*	*-20.03*	*113 %*	*-19.14*	*108 %*
Max. axial force of the struts	*8.52*	*100 %*	*12.42*	*146 %*	*17.16*	*201 %*	*22.84*	*268 %*
Min. axial force of the struts	*-46.72*	*100 %*	*-78.20*	*167 %*	*-54.70*	*117 %*	*-73.24*	*157 %*
Moment of the struts	*2.80*	*100 %*	-	-	*3.28*	*117 %*	-	-

Plane parts versus spatial calculation

For the calculation of the internal forces of many filling frames, the decomposition to plane subsystems is possible and meaningful. However in the reference object only eleven uniform filling frames exist and additionally, they are not loaded equally because of the chimneys. The analysis as plane system is not possible for some frames (especially in the area of the dormer and hip).

Exept for the influence of the dormer and hip, the load-bearing capacity of the frame wall can be sufficiently modeled with planar systems. For the determination of the load-influence area of the frame wall, the knowlegde of the magnitude of the load transfer achieved by the rafter-roof load-bearing of the hole structure is required in the present case. For this, a spatial analysis is needed in advance. That should at least include the basic flexibilities of the joints of the frame walls, otherwise the purlin-roof load-bearing is significantly overestimated.

Results of the utilization ratios (Model M1 versus M2)

The utilization ratios of the joints in M1 tend to be lower as those in M2. In return, the overall utilization ratios of beams are lower in Model M2. That means, that due to the consideration of the flexibilities and eccentricities of joints, load restributions from the joints to the beams take place.

The load-carrying capacity of the roof structure of the reference building can be plausibly represented by model M1 except for those areas that are significantly affected by the load transfer of the hip and dormer. The structural safety level no longer meets that of engineering standards. Instead of a global safety factor of approximately 2.5 as it is required indirectly by the standards, only a structural safety level in the area of 1.5 can be stated for the reference object.

RECOMMENDED PROCEDURE FOR THE STRUCTURAL ANALYSIS

Based on the findings of this work and a literature research, some recommendations for the structural analysis of ancient roof structures can be given for the daily practice.

I. Material constants

- As part of the first surveys of the roof structure it has to be estimated, to which grade (and thus strength class), the obstructed, undamaged timber can be allocated. This estimation is based on the visual grading criteria according to ÖNORM DIN 4074-1 (see [15]). In most cases S10 (equivalent to C24 in accordance with [14]]) can be assumed as a minimum (see [4]).
- After the structural analysis it is investigated, whether and which joints and beams do not comply with the safety requirements of the standards.
- Finally it has to be checked, if a higher timber grade for those joints and beams can be justified. As this detailed investigations (further information see [4], [7]) only cover single elements of the system, time and effort reamin manageable.

II. Structural dimensions and deformations

- To perform the structural analysis, the knowledge of the geometry of the structure with an accuracy to decimeters is sufficient. Only in the area of eccentric connections of beams or load introduction points, a survey with an accuracy to centimeters is required.
- Deformations of the structure are not included in the mechanical model. Nevertheless, it is recommended to check the verticality of posts, which transfer high axial forces. If the entire structure contains large deformations in one direction, it is recommended to estimate the deviatoric forces and transfer them via additional bracing to the supports.

III. Supports

- Typically ancient roof structures are not connected for tensile forces to the masonry walls. Instead of that, they rest without anchoring (see [6], [9], [10]). Horizontal forces are usually transfered by friction.
- If the definition of correct support conditions is extremely uncertain, different extreme values shall be considered in the structural analysis (see [6]).

IV. Cross section values

- The measured values of the cross-sectional properties are rounded mathematically to centimeters.
- Wanes have to be considered according to the grade or a reduced cross section is taken into account.
- The average values of the cross-section of a cross-section category (for instance rafters) has to be determined for the calculation of the internal forces (see [11]).
- The characteristic values of the cross-sectional properties of a cross-section

category (for instance rafters) have to be determined for the design (see [11]). After the structural analysis it can be investigated, if beams with a high utilisation ratio have larger cross-sections than used in the structural analysis.
- Beams whose cross-sections decrease in longitudinal direction, can be represented by a piecewise constant cross-section in every field. For the design in the area of middle supports the existing cross-section at this point has to be used. (see [10])

V. Joints
- The internal forces and therefore the stresses and strains in ancient timber structures depend to a large extent on the consideration of the flexibilities and eccentricities of the joints.
- For simple carpentry joints like the frontal offset, numerical simulations have shown that the representation with an eccentric to the middle of the contact surface delivers good results (see [6]).
- The flexibilities and eccentricities of most carpentry joints are neither covered by standards (Eurocode 5 see [12], [13]) nor discussed in literature satisfactory.

VI. Modelling und computation of the internal forces
- If, as a result of the survey, a pronounced spatial structural behaviour is estimated, at first a simplified spatial model neglecting the flexibilities and eccentricities of joints should be established at first. This model provides the basis for the knowledge of the fundamental effects of global load transfer and the size of load-influence areas.
- Based on the findings of the simplified spatial model, a decomposition into plane subsystems can be undertaken. Because of the existence of mostly statically indeterminate systems, the interactions between the subsystems have to be considered. The boundary conditions of the supports and the eccentricities and flexibilities of the joints have to be taken into account in a second step. However, the decomposition into plane systems is not possible in all cases (see [10]).
- Structural changes can be executed in all cases in the context of repairs and structural strengthenings. Particularly in the area of arris and valleys the possibility to add braces exists which may lead to an easier calculation.

VII. Design
- If available, the design is based on the internal forces of the two-dimensional computations.
- The design values of the strength of most carpentry joints are currently neither defined in standards nor satisfactorily addressed in the literature.

SUMMARY

Up to the 19th century most roof structures were erected on the basis of hands-on experiences, fine traditions and the courage of the craftsmen and not according to a structural analysis. As illustrated in the literature (see [1], [4], [6], [10]) and exemplarily shown in this papaer the simulation of a realistic load bearing behaviour is difficult and time-consuming. In practice mainly hybrid structures exist that carry the loads simulaneousely with purlin and rafter roof. The computed internal forces depend on the simplifications used for modelling of the structure to a large extent, but the real load bearing behaviour will remain unknown.

The most realistic picture of the load bearing behaviour of many ancient hybrid roof structures is revealed by a three-dimensional structural analysis using nonlinear supports (for instance the missing tensile anchors) as well as the deformation characteristics and eccentricities of the carpentry joints. The analysis of highly statically indeterminate systems lacking the mechanical behaviour of joints leads to inappropriate distributions of stresses in beams and joints, as shown in this work and the literature (see [4], [6]). Redistributions of the internal loads are only possible if the flexibilities and eccentricities of the joints are taken into account. Thus the load-carrying capacity of numerous historical roof structures can be simulated in accordance to surveys, although mostly the required level of structural safety cannot be achieved. In general, ancient roofs are not over-dimensioned in terms of current standards (see [1], [4], [10]). The utilisation rates of the carpentry joints tend to be higher than the one of the beams. Due to the fact that carpentry joints transfer loads mainly by contact, stresses perpendicular to the grain result, which lead to a ductile behaviour, because of the mechanical properties of timber.

As pointed out, ancient roof structures are mainly redundant and therefore robust structures. As a result of load-redistributions and despite of frequent and partial damage, the beams and joints are able to keep up the capacity of the whole structure for a long time.

REFERENCES

[1] DEINHARD Martin: *Die Tragfähigkeit historischer Holzkonstruktionen* : Dissertation : Karlsruhe. In: bauen mit holz Bruderverlag (1963), Nr. 1–3, S. 1/13–1/27, 2/71–2/85, 3/113–3/129

[2] DLUBAL Georg ; DLUBAL Ingenieur-Software GmbH (Hrsg.): *RSTAB 6.03.* Tiefenbach, 2009. - Programm-Version 6.03.3331

[3] GERNER Manfred: *Handwerkliche Holzverbindungen der Zimmerer*. Stuttgrat : Deutsche Verlags-Anstalt, 1992. - ISBN 3-421-03027-8

[4] GÖRLACHER Rainer: *Hölzerne Tragwerke : Untersuchen und Beurteilen.* Reihe B. Karlsruhe : Universität Karlsruhe, 1996. - Sonderforschungsbericht 315

[5] HEIMESHOFF Bodo ; KÖHLER N. ; Deutsche Gesellschaft für Holzforschung (Hrsg.): *Untersuchung über das Tragverhalten von zimmermannsmäßigen Holzverbindungen : T 2189*. München : IRB Verlag, 1989. - Forschungsbericht

[6] KIRCHLER Markus: *Modellierung eines historischen Dachstuhls – Vergleich von Stab- und FE-Berechnungen.* Graz, Erzherzog-Johann-Universität Graz, Fakultät für Bauingenieurwissenschaften, Dipl.-Arb., 2009. – Institut für Holzbau und Holztechnologie

[7] KRAFT Udo ; PRIBBERNOW Doreen: *Handbuch der Holzprüfung : Anleitungen und Beispiele*. 1. Auflage : Vbt Verlag Bau U. Technik, 2006. - ISBN-10: 3764004592

[8] LAND STEIERMARK (Hrsg.): *Stadtarchiv*, Auskunft von Frau Hary, 26.01.2009

[9] MAJCENOVIC Herbert (Sachverständiger für historische Bauwerke): *Informatives Fachgespräch : historische Dachstühle/Meisel Andreas*. Graz, 19.01.2009

[10] MEISEL Andreas: *Historische Dachstühle : Tragsysteme, Bestandserfassung, statische Analyse und Sanierung mit flächenhaften Holzwerkstoffen.* Graz, Erzherzog-Johann-Universität Graz, Fakultät für Bauingenieurwissenschaften, Dipl.-Arb., 2009. – Institut für Holzbau und Holztechnologie

[11] ÖNORM EN 1990 *Eurocode 0: Grundlagen der Tragwerksplanung,* 01. März 2003

[12] ÖNORM EN 1995-1-1 *Eurocode 5: Bemessung und Konstruktion von Holzbauten: Teil 1-1: Allgemeines - Allgemeine Regeln und Regeln für den Hochbau,* 01. Jänner 2006

[13] ÖNORM EN 1995-1-1 *Eurocode 5: Bemessung und Konstruktion von Holzbauten: Teil 1-1: Allgemeines - Allgemeine Regeln und Regeln für den Hochbau, Nationale Festlegungen,* 15. Jänner 2009

[14] ÖNORM EN 338: *Bauholz für tragende Zwecke: Festigkeitsklassen,* 01. Juli 2003

[15] ÖNORM DIN 4074-1 *Sortierung von Holz nach der Tragfähigkeit: Teil 1: Nadelschnittholz,* 01. November 2004

[16] ZWERGER Klaus: *Das Holz und seine Verbindungen : Traditionelle Bautechniken in Europa und Japan.* Basel : Birkhäuser - Verlag für Architektur, 1997. - ISBN 3-7643-5482-8