Historic timber roof structures, assessment and analysis on example of roof structure "TU Graz Alte Technik"

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Historic Timber Roof Structures, Assessment and Analysis on example of Roof Structure "*TU Graz Alte Technik*"

MASTER THESIS

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Posebno se želim se zahvaliti mojoj obitelji na njihovoj bezuvjetnoj podršci, bez njih sve ovo ne bi bilo moguće.

“We are trying to prove ourselves wrong as quickly as possible, because only in that way can we find progress.”

— *Richard P. Feynman*

Abstract

The combination of increased global awareness of the protection of the cultural heritage buildings, and a rising need for living spaces in old city areas, resulted in the need for a standardized legislation and norms. The main goal is to maximize the success of preservation of the cultural heritage and to regulate vast conversion of the attic spaces in living spaces. This master thesis deals with an overall approach to a preservation project of the cultural heritage with an emphasis on the timber roof structures.

First, an overview of historic timber roof structures in Europe, with an emphasis on Central Europe, is given. In the following section, the need for cultural preservation of structures is described and explained in relation to global and local institutions.

The main part of the thesis involves assessment, survey, analysis and strengthening of the historic timber roof structures. Therefore, in the second part of the thesis, different assessment methods are described and compared, with an attempt to standardize the survey process. The impact of different approaches to the analysis of the timber roof structures is explained in more detail, taking into account different influences, and quantitatively compared on an example of the roof structure of TU Graz Alte Technik. Lastly, an approach to strengthening and an overview of different strengthening measures are presented. The second part of the thesis is dedicated to the implementation of derived approaches of assessment, analysis and strengthening on a real example of the roof structure on the south wing of TU Graz Alte Technik.

Keywords: historic timber roof structures, heritage, assessment, survey, analysis, strengthening timber

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CHAPTER 1: HISTORIC TIMBER ROOF STRUCTURES

This chapter aims to show the historical overview of the development of historic timber roof structures during the history of humankind, and parallel to that to introduce timber as a significant structural material. This chapter is mostly focused on timber roof structures in Central and Southern Europe during the 18th and 19th century. Furthermore, the typical classification of traditional roof structures on rafter and purlin roofs and their respective sub-classification in Central Europe is here illustrated and described.

1-1 HISTORY OF TIMBER STRUCTURES

Due to its great availability all over the world, especially during the ancient times and until the middle of the 17th century, timber was one of the most important building materials. Many structural parts of houses, buildings or bridges were built from timber. The extent usage of wood in the past can be related to fact that wood can be easily cut, carved, and shaped, with the help of tools that were available before the First Industrial Revolution. Before the 20th century when erection of steel straps and timber-steel connections became common, the development of the timber structures was mostly influenced by the development of timber-timber joints and tools. (cf. [1] [2]) Also, the development of timber structures was linked to the availability of timber in a particular region.

Around 10 000 years ago when people started to build more permanent structures, the history of timber structures began. At first, people built simple shelters from branches and logs that they found in forests. With the development of tools, the building boom started. The progression from the simple structures during the late Stone Age towards the more complex structures during the Bronze Age is illustrated in Figure 1.1.

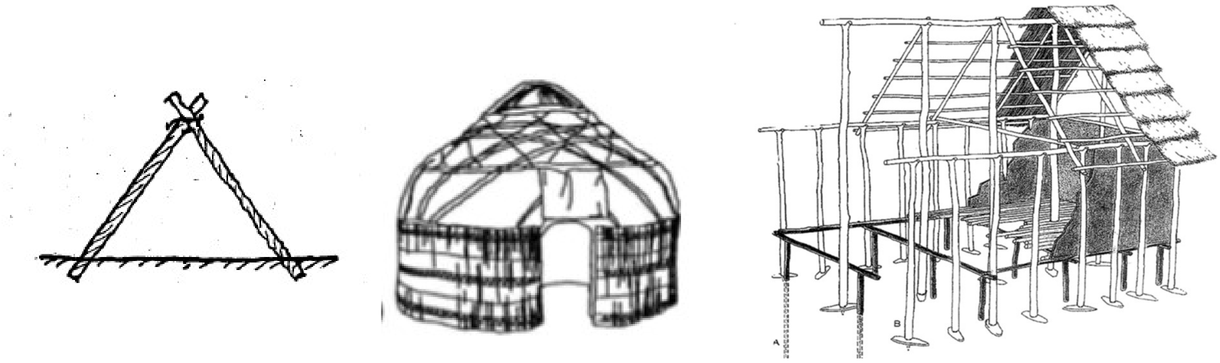


Figure 1.1. ***Development of early building (left: first variation of rafter roof "dachstute" [1]; middle:"yurt" [2] ; right: the house from young period of stone age) [3]***

The progression and development of connections in timber structures are structurally presented in [3] at Page 7. Also, for a detailed description of timber to timber connections and its development through time, see. [3]

In medieval period, timber was the dominant structural material. Timber was used for the frame structures of ordinary houses, while more prestigious buildings were made of stone and brick. Nevertheless, all roof structures during the medieval period were made of timber. During the medieval period, the erection of churches and cathedrals was very common, so complex and innovative roof structures could be found. In the medieval period, timber structures could be divided into vertical Frame structures for walls of houses, and roof structures. The more complex roof could be only associated with cathedral roofs because large spans and complex geometry needed to be satisfied, as shown in Figure 1.2 (cf. [4] [5])

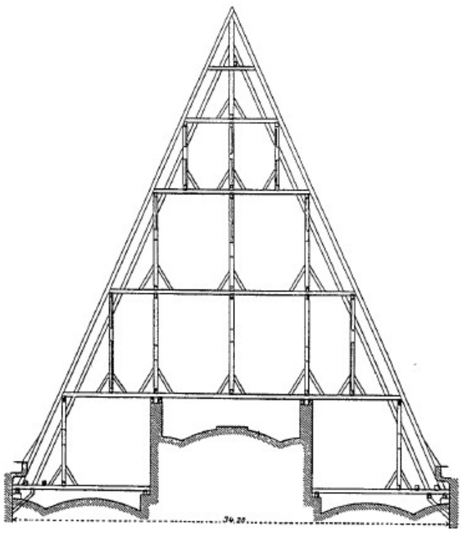


Figure 1.2. Example of timber roof structure used in medieval time – St. Stephan church [4]

From the Renaissance and continuously until the 21st century, wood was pushed out by other materials. At the beginning, brick replaced wood, while in the 18th and 19th century, the development of new materials (Steel and Reinforced concrete) became more prominent. The most important reasons for the reduced usage of timber were: flammability of timber, lack of durability in comparison to other materials and turnaround in architecture. For example, the introduction of brick over timber took a giant leap forward after "The Great Fire of London" took place in 1667 see [6]. Since then, timber was mainly used for roofs, barns, floor structures etc. (cf. [5])

Nonetheless, timber roof structures and timber bridge structures reached their development pinpoint during the 18th century especially after the Industrial Revolution in the late 18th century. During the industrial revolution, first sawmills were established where sawed timber is produced. This marked the opportunity for massive production of timber products. Furthermore, the introduction of iron, which preceded the steel and later mass production of steel bolts and nails contributed to the development of then existing statical systems through implementation of timber-steel connections. Introduction of the theory of statical analysis and theory of materials also made its contribution to the development. In the 18th century first idea of sustainable timber exploitation and construction was introduced in Germany. The idea was based on the equilibrium of seeded and cut trees in the forest. (cf. [7] [5]). Also, in the 20th century that idea prevailed and got its upgrade in the 20th century at the Brundtland commission see ([7], [8])

Today timber can be used for both small and large buildings, or in other words, its usage is not anymore restricted only for secondary objects and roof structures. With the development of technology, new timber products emerged on the market which marked the start of modern timber structures like houses and housing towers made from CLT panels. Because of breakthroughs in the field of durability, fire protection, the behaviour of timber products under fire, and new timber products, (e.g. CLT panels), whole houses, buildings or towers can be built from timber in contrast to the medieval era.

Today timber is considered as an ecological structural material and its usage is increasing. Concept of a sustainable and ecological building plays a major role today, and even more in the future. From that point of view, timber emerges like a rational solution to that requirement, as timber products and research in the area of developing timber towers take fast steps forward, timber will become a more and more significant part of everyday construction in the future. For more information about present use and future use of timber and developing of timber towers refer to [9] [10] [11].

There is more detailed information given about the history of timber structures in [12] [5] [2] [4] [13]

To sum it up, it could be concluded that timber is an important material of the past and even more important material for the future.

1-2 OVERVIEW OF DIFFERENT TYPES OF TIMBER ROOF STRUCTURES

The roof is the highest structural element on every building, often colloquially named 5th facade of the building (cf [14]) Roof beside load bearing function also must satisfy the protection function as roofs protect the inner volume of the building from sun insolation, wind, rain, etc. Roofs are always exposed to continues load amplitude fluctuations. Mostly due to wind and snow. Furthermore, roofs are subjected in some cases to extreme climate conditions from which most influential and important for roofs are: temperature changes and changes in relative humidity. Besides given functions, esthetical function greatly influences on the look of the building but even more on the overall landscape of the city, see Figure 1.3.

Today new function emerges for roof structures. Living areas in the attics in the present time has been revitalised, thanks to lack of living space and construction space in towns and the old town areas. Old attics which in past haven`t been used or have been used for storing purposes are converted through reconstruction and revitalisation to living spaces, see Figure 1.4. In past, those areas weren`t used mostly because of energy inefficiency, lack of useable space, etc. As modern insolation materials emerge on the market and in combination with lack of living space and overcrowded old town areas, attics become a point of interest for valorization in means of providing new living areas. These areas because of exquisite locations are becoming popular day by day. Each valorization needs to satisfy new building codes. As today codes are not adapted for existing structures often unnecessary and damaging alternations are made to the roof structures, greatly endangering cultural heritage. According to [15] most of the roof structures in Graz demand reparation to satisfy modern code regulations. Reconstruction in a mean of removal of an old structure, because of the law, the ecological and the economic view should be avoided. Furthermore, the old town of Graz is part of the UNESCO list and therefore one of the conditions is to preserve the look and structure of roofs as it is. These demands present a great challenge for all project participants.

Figure 1.3. View on the roofs in Graz, old town [16]

As a consequence, new research projects emerged with the topic on approach to perservation, assessment, modelling and strengthening of timber roof structures. Many master theses, articles, etc. can be found regarding this topic, in addition, they deal with real structures in the area of Graz. Refer to ([17] [16] [18] [15] [19] [20] [21] [22])

Figure 1.4. ***Reconstructed attic for living space [16]***

Because of the diversity of timber roof structures that can be found in these areas. Next subchapters try to expose those types and subtypes and classify them according to known literature.

For parts of the roof given in English, see Figure 1.6.

1-2.1 GENERAL DIVIDE

Traditional roof structures can be divided into several criteria:

a) According to the shape and number of slopes (i.e. roof types) as shown in Figure 1.7.
b) According to the roof pitch as shown in Figure 1.5.
c) Classification according to the statical system as shown in Figure 1.8.

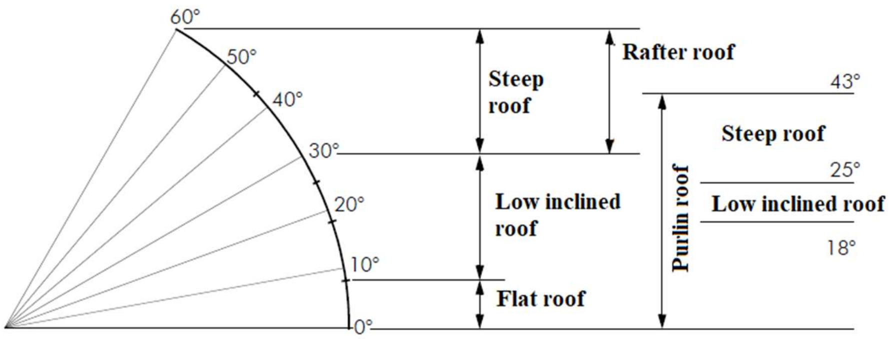


Figure 1.5. Classification of roofs according to the roof pitch [23]

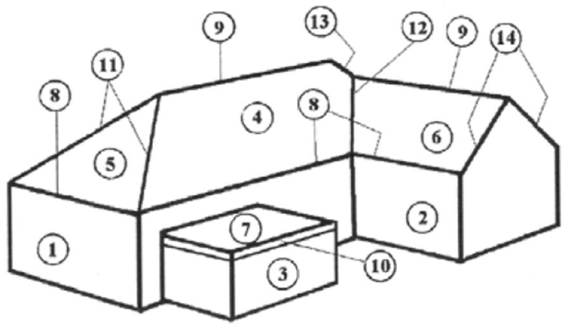


Figure 1.6**. **Parts of roof: 1: Gable wall, 8: Eave, 9: Ridge, 10: Facia board, 11: Hips, 12: Valley, 13: Hip, 14: Verges… [23] (English translation)

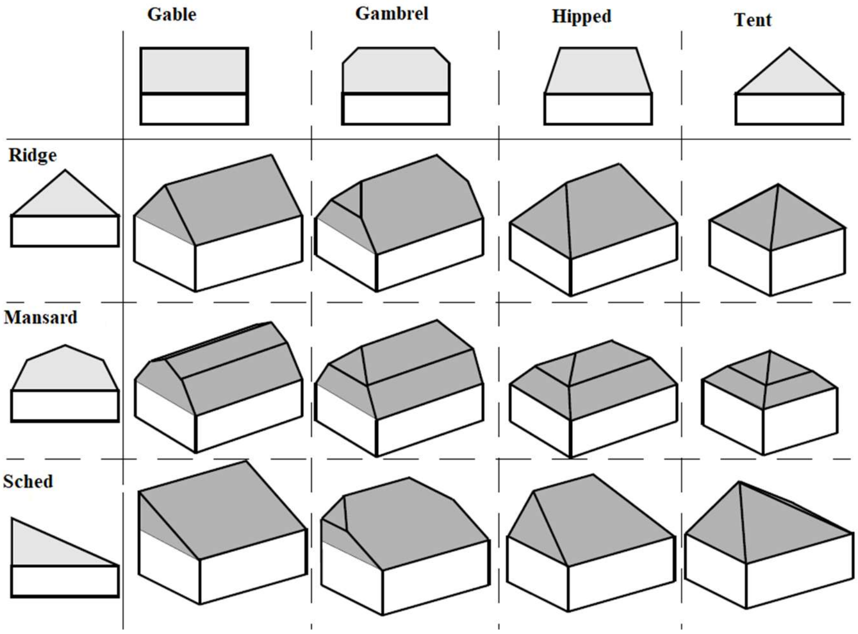


Figure 1.7. Classification of roofs according to the roof types [23]

In traditional timber roofs, transfer of loads from rafters can be achieved in two ways. Rafters can transfer forces directly to the supporting structure for example (Structural wall). On the other hand, rafters can be only part of roof cladding. In this case, rafters are transferring loads from the roof surface to the horizontal beams „purlins “. These two types present fundamentally different structure principles, which must be kept strictly apart for static reasons. Moreover, they differ by structural elements, and in range of suitable application, therefore must be divided. The divide is shown in Figure 1.8. and presents typical rafter and purlin roof structures.

In cases of complex structures with large spans combinations of these two principles is possible and optimal.

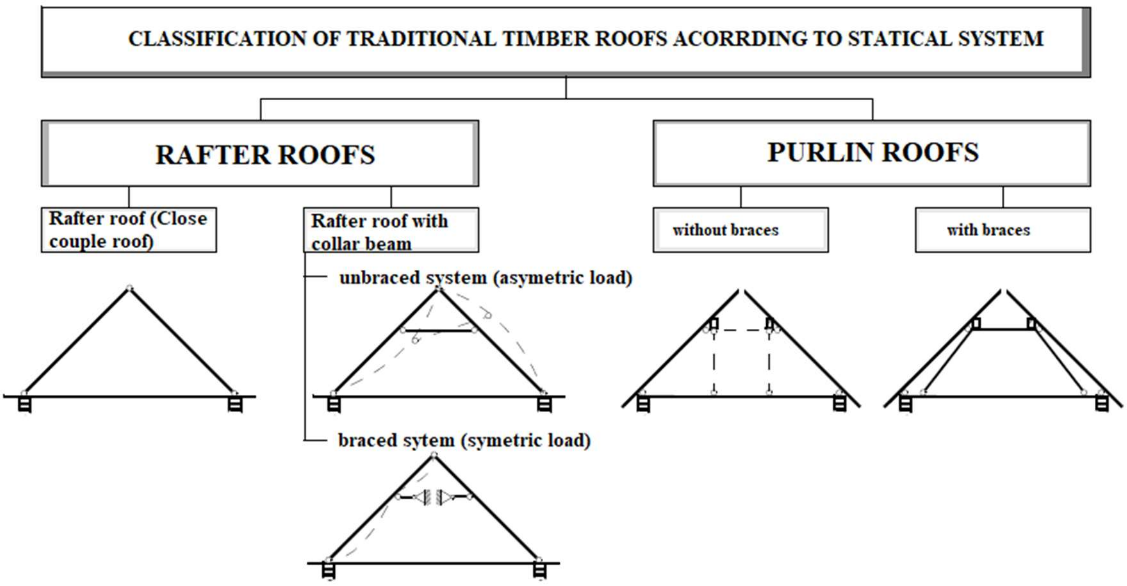


Figure 1.8 Classification of roofs according to their statical system [23]

More detailed classification is given in Meisel (cf. [17] [16])

Classification in Figure 1.5. Should be taken carefully when speaking about historic roof structures because purlin roofs have been used for steep pitches during the mediaeval period.

In the early Middle Ages, purlin roofs with suspended structures were used as roof structures which shows that even steep purlin roofs have a long building history. [16]

In Figure 1.10 and Figure 1.11 the vast overview of different types of roof structures is given according to Meisel [17]. For Figure 1.10 translation German-English translation is given according to the author. For Figure 1.11 translation is not given but it is possible to determine the type of the structures by the previous figure. In Figure 1.11 development of timber roof structures is shown with respect to a time reference. For a further description of types of roof structures that are given in Figure 1.10 refer to Meisel [17]. Moreover, an additional overview of historic roof structures in Central Europe is given in Figure 1.9 according to [24], where additional roof historic roof types in respect to time reference can be found.

For the sake of simplicity in this work, only the most important types of roofs will be described and discussed.

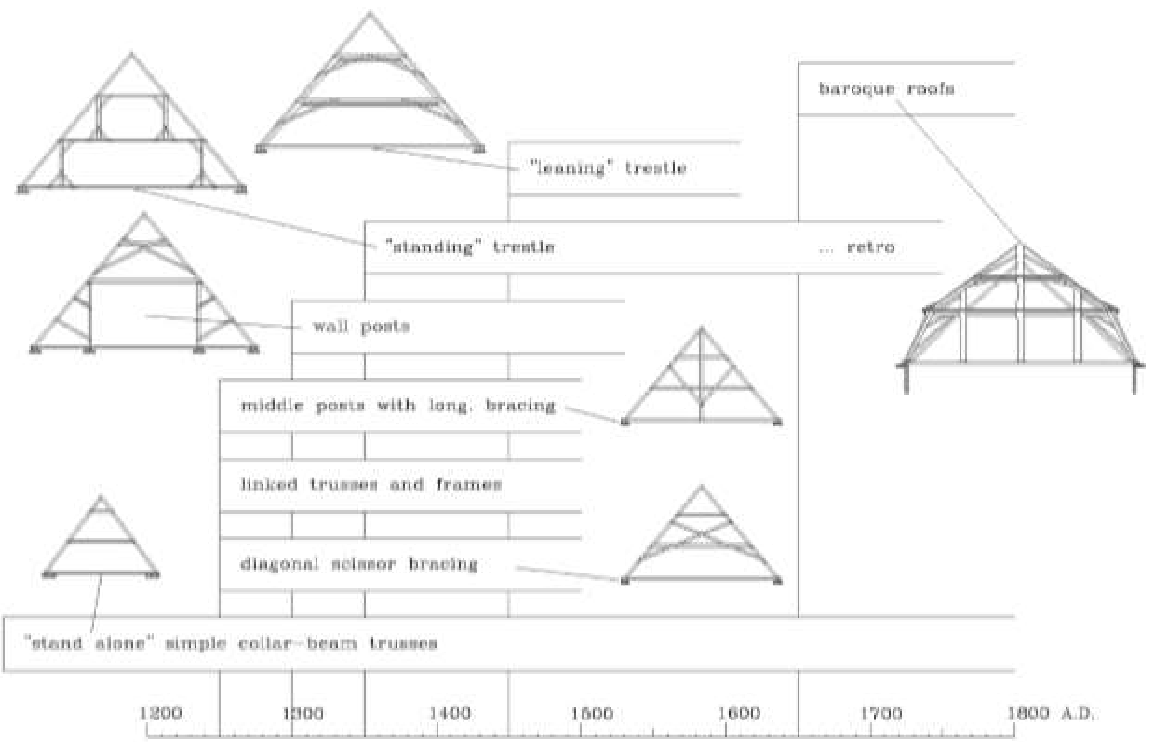


Figure 1.9. Development of the roof trusses in Middle Europe according to [24]

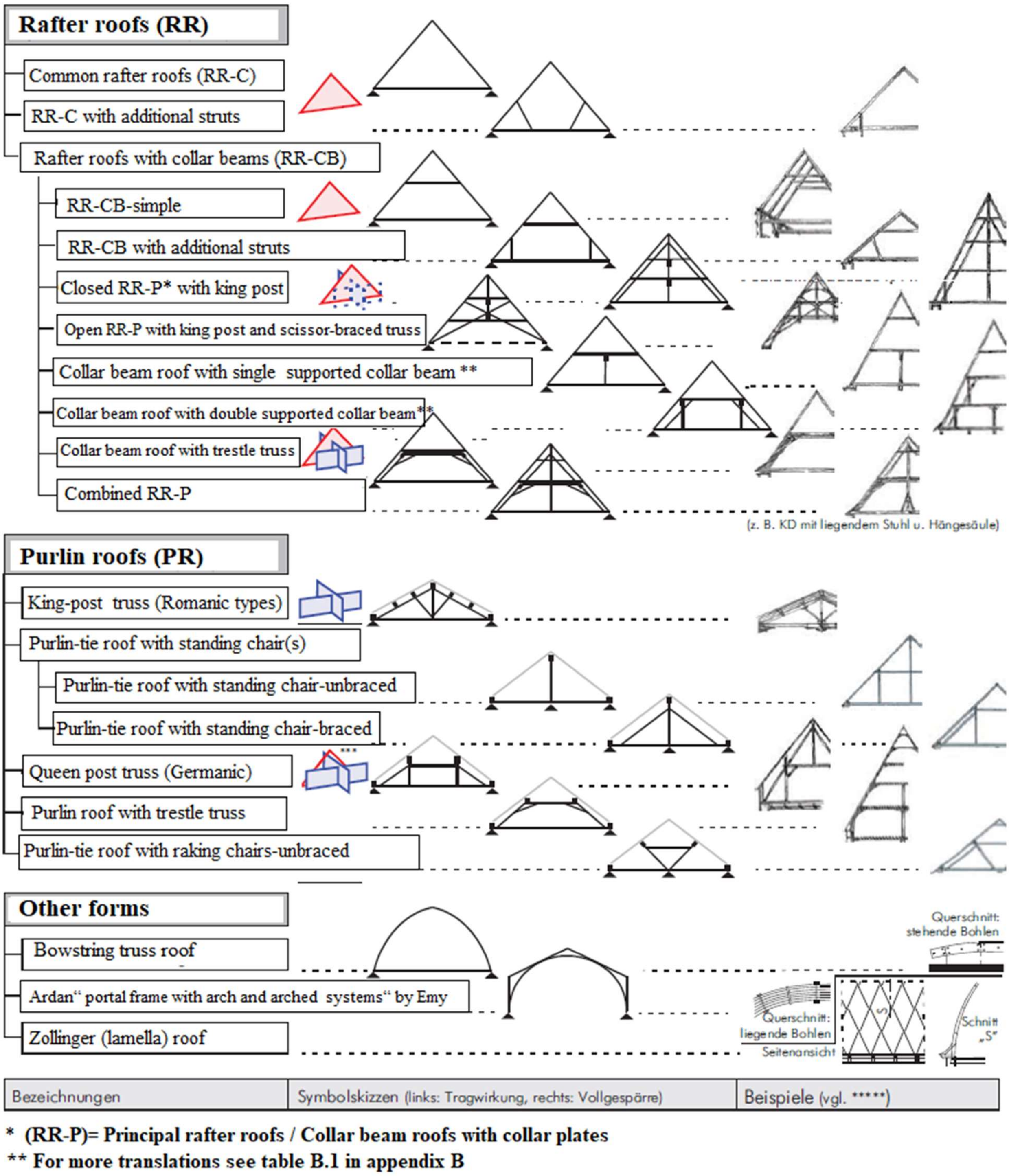


Figure 1.10. Classification of historic timber roof structures in Central Europe according to [17] English translation

NOTE: See also Annex B for more details.

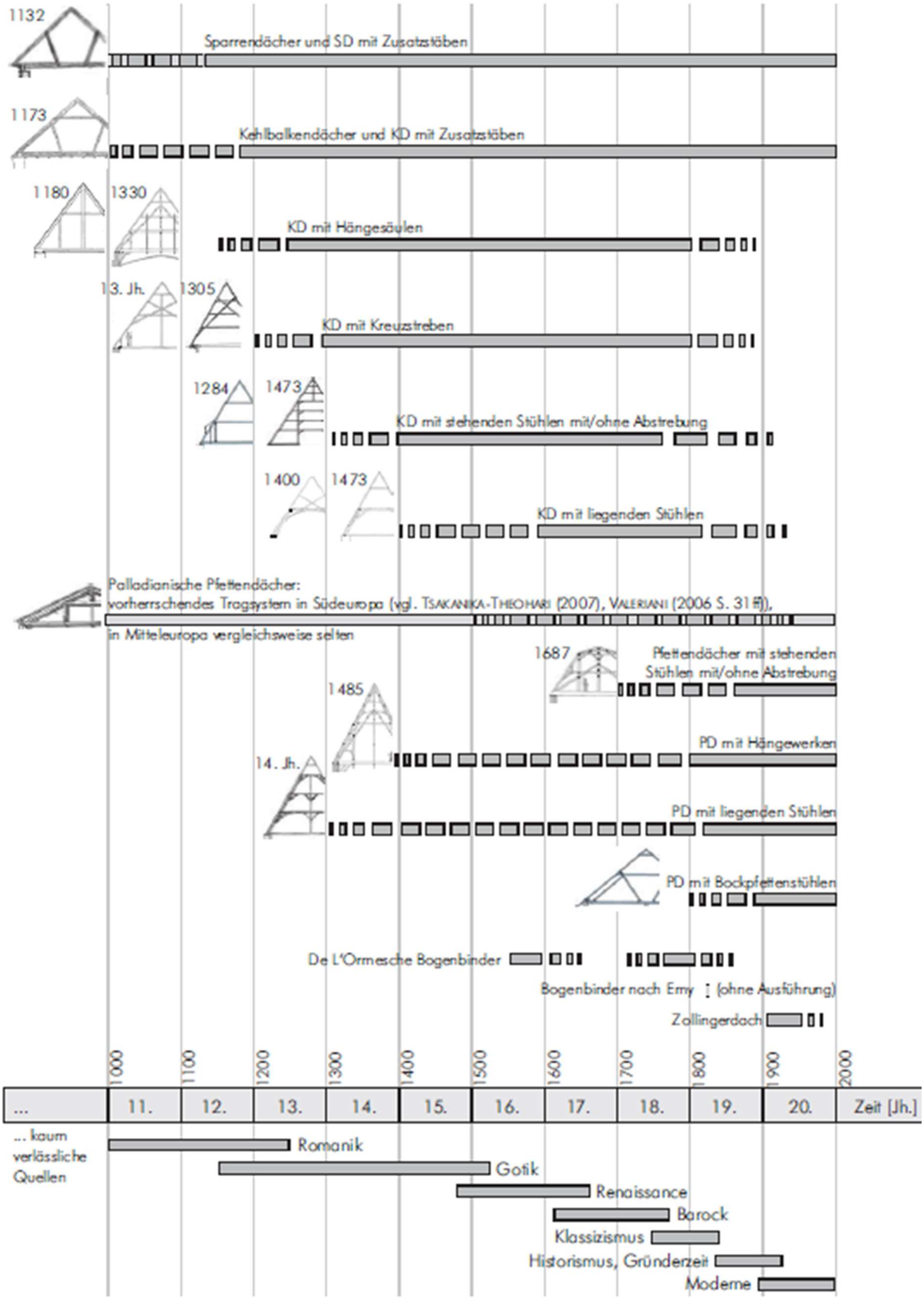


Figure 1.11. Chronology of the development of historic roof structures in Central Europe according to [17]

1-2.2 COMPARISON BETWEEN ROMANIC AND GERMANIC ROOFS

Before following subchapters on rafter and purlin roofs in Middle Europe, a short comparison of roof structures between different parts of Europe is given. The clear comparison of the diversity of roof structures in different parts of Europe is hard to discuss and explicitly define as literature regarding that area of research gives only implicit differences. However, according to the available literature, some main points will be given here. South Europe together with England will be compared to the area of Central Europe.

The translations of terms and expressions for roof structure types developed in German-speaking countries are not standardized for other languages, especially English. This makes the comparison of roof structures more complicated. For that reason, in Appendix B the German-English-Croatian translations of the structural systems and structural elements can be found, together with a description of the element and its function in the roof structure. By studying the literature author stumbled onto a large number of different English translations for the same types of Germanic roofs which emphasise the problem of lack of standardized translations, as different translations incite confusion to the reader. Unfortunately using the typology from similar roof structures from English speaking countries which adopted and customized Romanic type of roof structures from Italy is not always possible as differences are sometimes too large to be ignored.

In history in different parts of Europe different structural roof types have been developed through time, although these structures differ in their visual appearance and often in different types of structural elements the main load carrying principle is the same. For example (Queen post truss in Southern Europe is similar to doppelte Hängewerk in Central Europe area) as shown in Figure 1.12 from which can be concluded that load carrying mechanism is close to same, although elements are organised differently. This is also valid for example of King post truss, see Figure 1.13.

Reason for differences in roof structure types can be found in several factors such as: cultural diversity, traditional carpentry work, available tools, and roof geometry. The most obvious difference can be noticed in roof pitches between the Mediterranean and Central Europe. In Central Europe, pitches are much steeper than the Mediterranean roofs, mainly because of snow load, therefore rafter roofs were more common in Central Europe as they are more suitable for steep roofs while purlin roofs are more suitable for Mediterranean areas (cf. [25]). There are many more examples and reasons, which can be prescribed, to the diversity phenomenon that can be found in more detail in [26]. Best description on the allocation of alterations of roof structures types along European territory according to the author opinion gave *S.M. Holzer* "*Roof trusses developed along different tradition lines in different regions of Europe, generally in alignment with language borders.*" [25]

In the country itself, local variations of roof structures can be found. For example, a local variation of German type of "*Dachstuhl*" in the local area of Graz is referred to as ("*Grazer Dachstuhl*) see [16].

Often terms Romanic and Germanic roof can be encountered in literature. These terms are used to describe differences between different timber roof structures in Europe. In General, according to the literature [25] [26], it can be derived that Germanic roofs belong to the central Europe area (Germany, Austria, etc.). while Romanic type of roofs developed in Southern Europe and the Mediterranean, later spread to England [27].

Germanic type of roof precedes today known rafter roofs. High pitched rafters characterize these roof structures. Rafter roof will be in the next chapter more thoroughly described. Later from rafter roofs with the addition of support to the rafters, Germanic purlin roofs were developed which are also described in the following chapter. [25]

In contrast in Mediterranean areas, purlin roofs are more suitable, because of low roof pitches. Those kinds of purlin roofs are often called Palladian trusses Figure 1.12 left. The palladian truss is an alteration of a King post truss Figure 1.13 right, and it is developed in Italy by Andrea Palladio, who started to use this system massively (cf. [28]). These types of roofs have many variations which can be found for example in Italy, Portugal, England, etc.

Interesting phenomena can be found in the coastal part of Croatia which belongs to the Mediterranean territory. During history some of these areas were under Austrian-Hungary monarchy 19th and 20th century, as in that period most of the buildings were built, roof structures were mainly built as Germanic purlin roofs. The form of these roofs is copied from Austria and German-speaking territories. Moreover, in northern coastal parts which were under the influence of Austrian-Hungary and later Italy a mix of Germanic and Romanic roofs can be found. From Romanic type King post trusses were used usually for storehouses and buildings of industrial purpose. This phenomenon furthermore strengthens the fact that type of roof structure in different territory is defined by factors defined at the beginning of chapter.

As here is given only short and general overview of differences the author suggests the following literature for upgrading knowledge about this topic. [17] [28] [26] [24] [4] [5] [27].

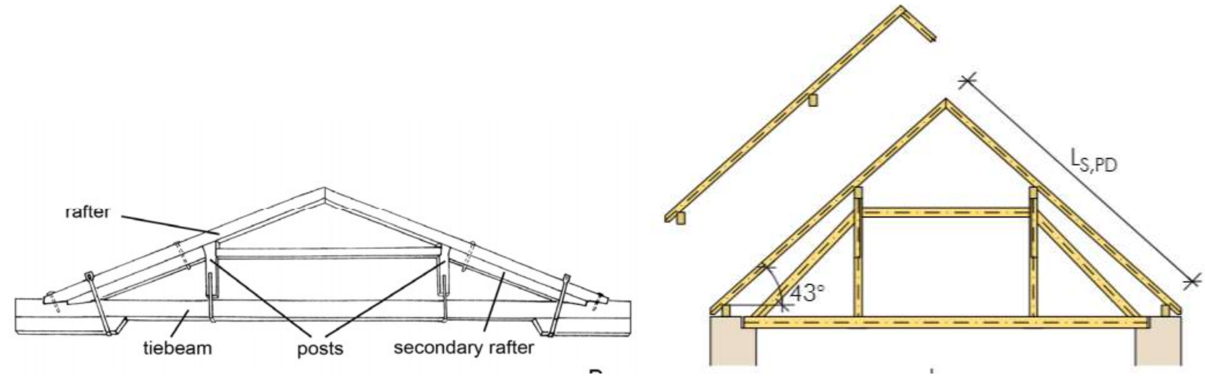


Figure 1.12. left: Queen post truss, Romanic ("Palladian post") [27]; right: Queen post truss, Germanic [19]

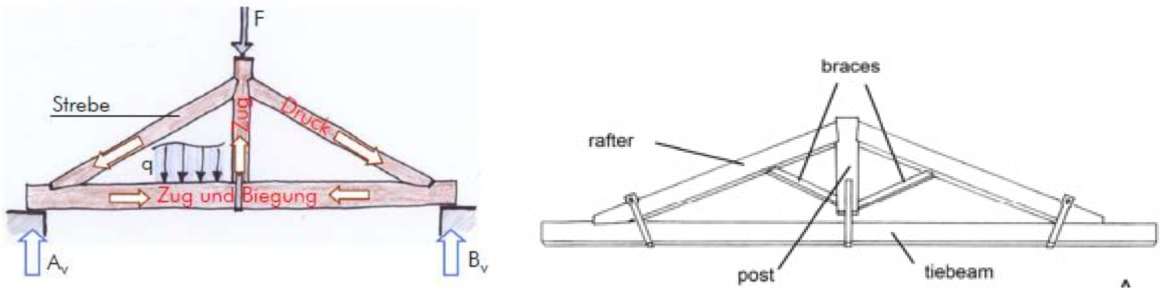


Figure 1.13. left: King post truss in the area of Central Europe Germanic [16]; right: King post truss in the area of Southern Europe Romanic [27]

1-2.3 RAFTER ROOFS

Rafter roofs as mentioned before are used for steep pitches from 30° (43°) up to 60° and therefore mainly used in continental territories with high snow load. Rafter roof as known today and in the medieval period have originally developed from "roof hut" (German-"Dachstute" Figure 1.1) which consist of two tree trunks leaned on each other and tied together to form a triangle. This type of construction is still in use today in the Alps as temporary shelters or tents, see [1].

From the standpoint of the statical system and load transfer according to Figure 1.8 rafter roofs are divided to

a) Rafter roofs
b) Rafter roofs with collar beam
    a. With collar beam – braced
    b. With collar beam – unbraced

Rafter roof with its structural elements is shown in Figure 1.15. Structural elements are defined in English and German respectively.

Rafter roof is a structurally determined system, consisting of a pair of rafters which rely on a tie-beam and support one another in the ridge. Rafters together with a tie beam form a triangle, which ensure stability and transfer of loads in the main plane. To provide stability out of the triangular plane, wind braces are inserted, diagonally across the rafters. Figure 1.15. Rafter frames are positioned in distance of 0,7m – 1,0m according to literature [28] [14] [1] [29].

Load transfer of a rafter roof is in principle simple. The load which acts on roof plane transfers to rafters which are loaded with bending, shear and compression. These three internal forces always exist in rafters. Load from rafters transfers to the tie beam and from the tie beam to supports. Illustration of the behaviour of rafter roof under load is shown in Figure 1.15 right, and under symmetric and asymmetric load with internal forces diagrams in Figure 1.19.

The rafters are the main part of a rafter roof as discussed in the previous subchapter they form the supporting structure of the whole roof and transfer forces to walls and tie beam. Rafters have a major impact on the total volume of the timber needed in rafter roof. Because of that, and the high intensity of bending and compression on rafters, to avoid uneconomical cross sections, various literature proposes limiting the unsupported length of the rafters from 4,5 m to 5,0 m. From that restriction and for given pitch interval, a span of the rafter roof usually is not bigger than 7,0m. In some literature expressions for optimal depth/span ratio for rafters are proposed, see [1]. The main function of the tie beam is to receive horizontal trust from rafters, because of the closed force polygon system of the rafters-tie beam, resulting horizontal trust on outer structural walls is neglected (see Figure 1.15). In case of massive concrete slab polygon of forces stays the same and slab is working as a tie. In some cases, tie beam can be loaded by deadweight of the ceiling or floor in the attic, also with live loads, if the attic is in use, therefore it can be subjected to a combination of bending and tension. In some cases when attic walls exist, and rafter support is not on a tie beam, measures must be taken to avoid horizontal trust on attic walls. That is accomplished with additional struts, furthermore, they reduce the unsupported span of rafters. Such a system with the redistribution of forces inside of system is shown in Figure 1.14.

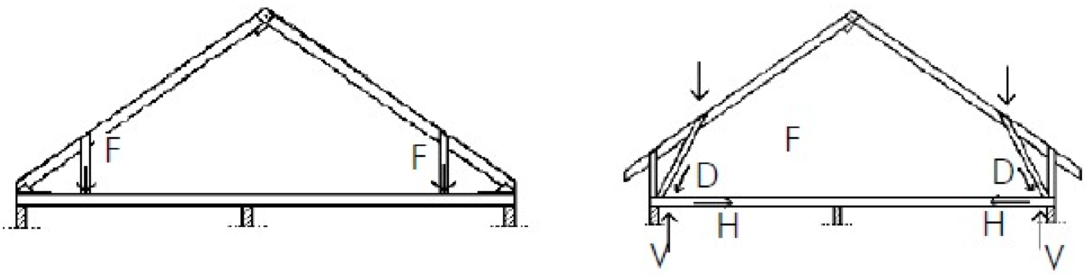


Figure 1.14 left: Rafter roof with vertical struts; right: Rafter roof with raking braces [23]

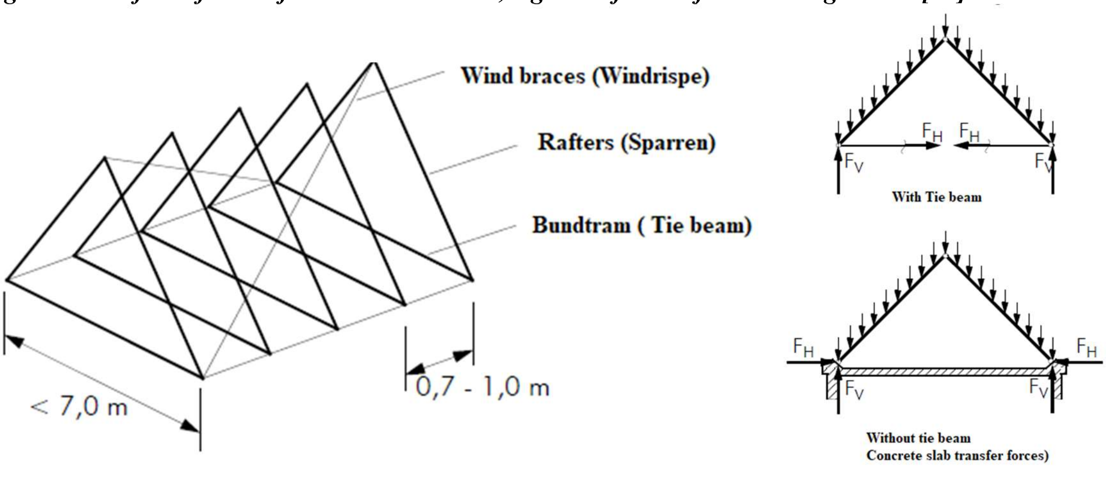


Figure 1.15 left: Rafter roof with structural elements [23]; right: Load transfer in rafter roof [23]

The structural system shown in Figure 1.16 might be considered as an alternative to a rafter roof. The structural system with collar ties shown in Figure 1.16 is principal rafter roof, where the principal trusses contain tie-beams, while the rafters within secondary trusses are supported on foot purlins which are placed on tie-beams. Tie beams can be positioned on every third to sixth rafter frame. The horizontal forces from secondary trusses are transferred to the foot purlin, and collar ties combined. The distribution between these two depends upon the bending stiffness of the wall plate. Collar ties must be fitted to each pair of rafters. With this alternative, less timber is used. (cf. [16])

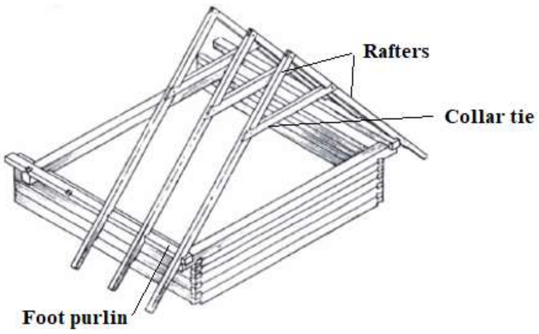


Figure 1.16 System of trusses with collar ties [16]

For spans larger than ca 7,0 m rafters are longer than 5,0 m, Collar beam then need to be placed in rafter roof. This result with a new type of rafter roof called Rafter roof with collar beam (or collar beam roof) Figure 1.17, Figure 1.18. The function of the collar beam is to support rafters, therefore reduce bending of rafters. With the introduction of the collar beam length of rafters can increase up to 8,0m. Rafter roofs with collar beams are used for spans no longer than 12,0m. Moreover, for rafter roofs with collar beam maximum pitch of 60° degrees is not optimal, recommended pitches for this type is from 30° to 40°. (cf [14] [1] [29])

For this collar beam roof, two subtypes exist where a collar beam can be braced or unbraced. Braced rafter roof with collar beam shown in Figure 1.18. In case of braced roof collar beam act as a support to rafters and reaction from support is transferred along bracing system to bracing system supports. Usually, supports are abutting walls. In other hand, rafter roof with unbraced collar beam Figure 1.17 shows different behaviour under different load cases. Under symmetric loads collar beam acts as support to the rafters, and in case of asymmetric loads collar beam doesn't act as a support. The behaviour of a rafter roof with braced and unbraced collar beam under load is shown in Figure 1.19.

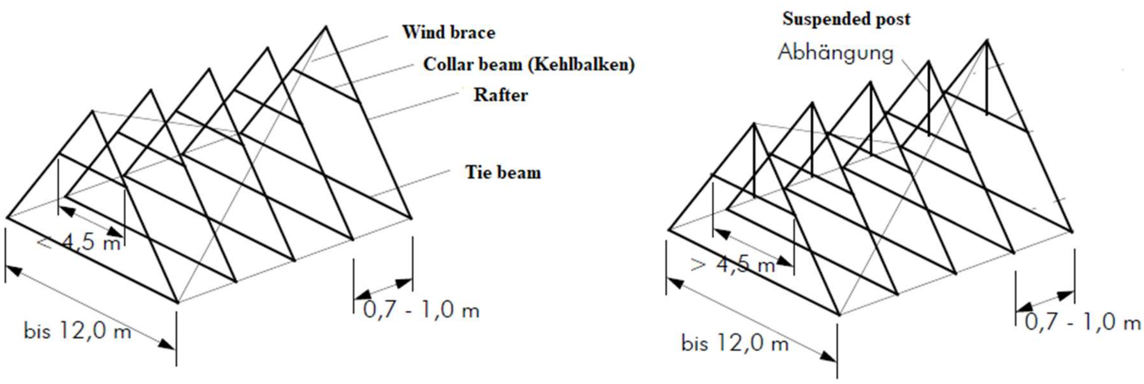


Figure 1.17 left: Rafter roof with collar beam, unbraced [23]; right: Unbraced, with collar beam supported by hanged post [23]

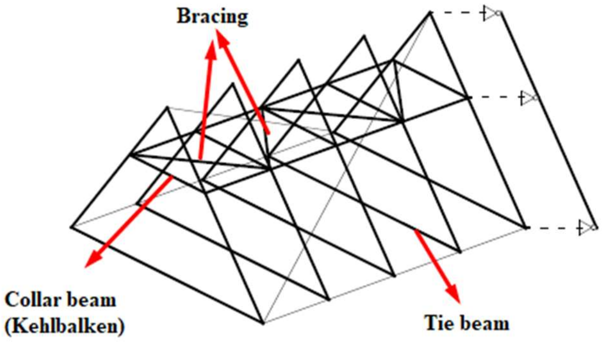


Figure 1.18 Rafter roof with collar beam, braced [23]

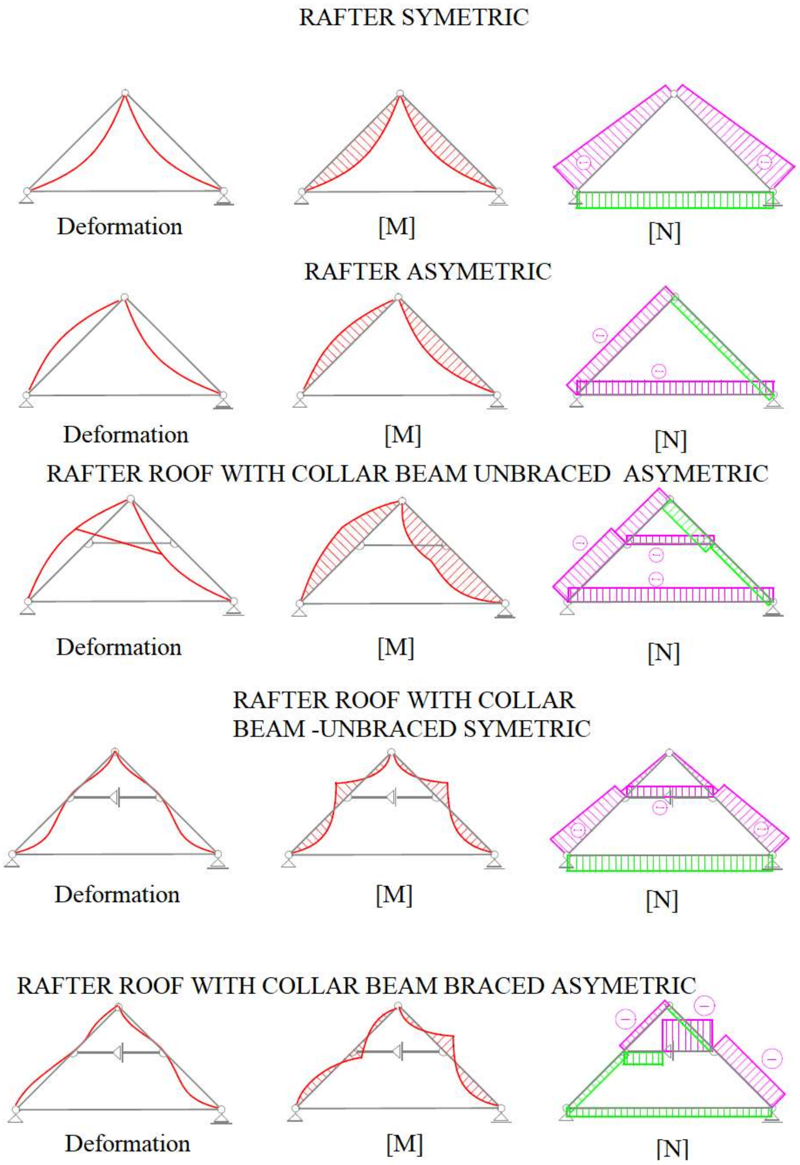


Figure 1.19 Behaviour of rafter roofs (common rafter / and collar beam roofs) under various loads [16]

Collar beam should not be longer than 4,5m according to the literature. In case the collar beam is longer than 4,50 m, the collar beam must be supported. Collar beam can be supported by suspended post as shown in Figure 1.17 right, but more often it is supported by one or more collar plates which rest on vertical collar struts forming the principal truss. In that manner, principal truss supports the collar beam in one or two points, transforming the collar beam roof (as essentially rafter roof type) into so-called principal rafter roof with collar plates (or also referred as collar beam roof with single/double supported collar beam). Figure 1.20. These supporting structures are described in [23] [19]. Some details (which are more related to terminology issues) are given in Table B1 (see Annex B), too.

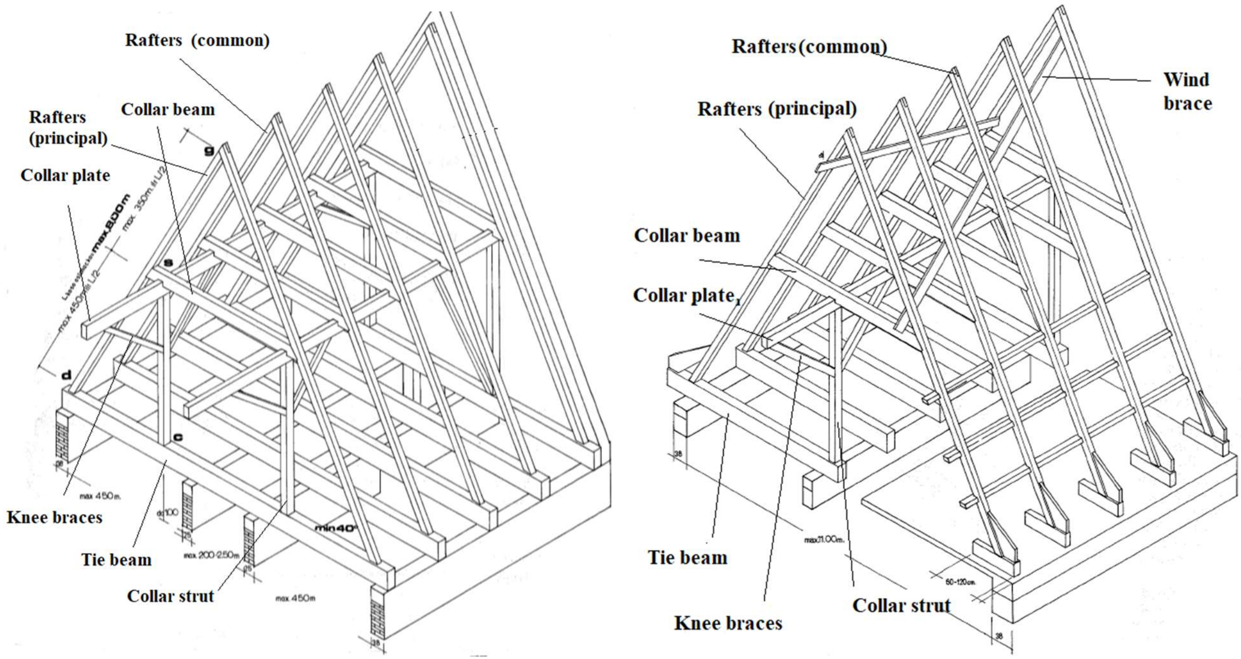


Figure 1.20. left: Principal rafter roof with two collar plates [29]; right: one collar plate [29]

Alternation, where a pair of sloped raking struts are used, is shown in Figure 1.21 right. These raking struts are parallel to rafters, and in conjunction with a straining beam and soulaces form a trapezoidal portal frame supporting a collar beam. These roofs are known as trestle truss, and they are very common structural roof type in Central Europe while in Southern Europe and England are rarely found, (or not at all). Therefore, it is not surprising that many English translations can be found in the literature. Among them, a "trestle truss" is chosen to be used in the followed text. See Annex B for more information about terminology.

The sloped raking struts in trestle roof leave large space in the middle area of the attic which can be used for living spaces in contrast to Collar beam roofs with collar plates. Large spans can be achieved with these supporting structures by stacking the raking struts one of top of another to form several levels. In medieval churches and cathedrals where big spans were needed this type of supporting structure was commonly used. Furthermore, for larger spans, king post which goes from ridge to the lowest tie beam is inserted to reduce deflection of the tie beams and eventually take the load from the ceiling. Spans larger than 20,0 m can be obtained with this supporting structure. A great example is building of Alte Universität in Graz, which was built at the beginning of the 17th century, Figure 1.22. The span of this structure is approximately 20 m. This roof structure is in detail analysed in the master thesis of Ortner [19]. Moreover, information can be found in [22].

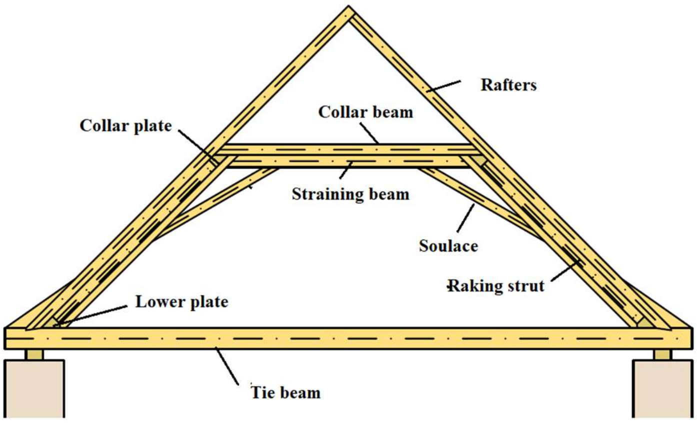


Figure 1.21. Collar beam roof with trestle truss [19] – see also Table B1 in Annex B

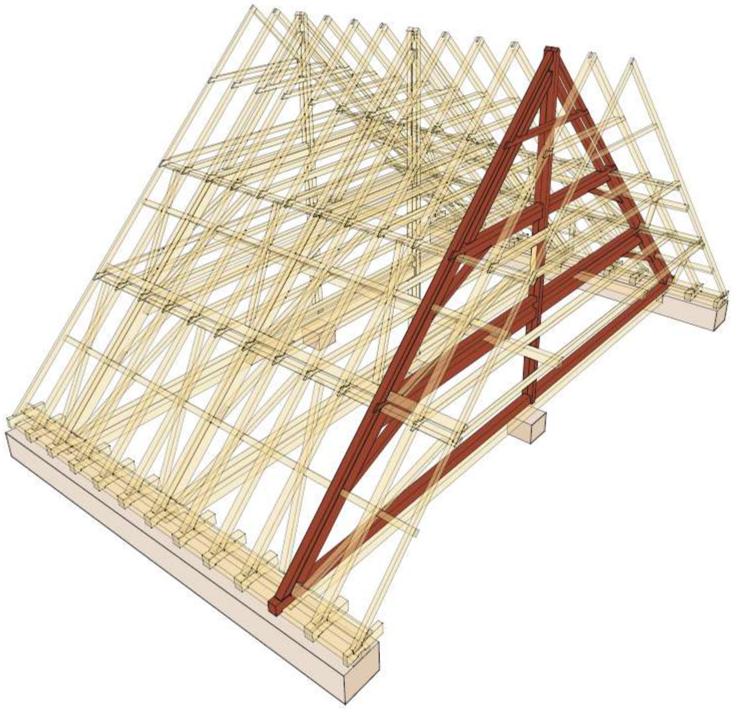


Figure 1.22. Collar beam roof with trestle truss with king post at Alte Universität in Graz [19]

In mentioned types of rafter roofs, usually, 3 characteristic types of connections can be found.

a) Rafter to rafter - Figure 1.23 (middle)
b) Rafter to collar beam -Figure 1.23 (right)
c) Rafter to tie beam - Figure 1.23 (left)

Some of these connections are described in Chapter 2.3, therefore only 3D details are shown here. In Figure 1.23 (right) is shown one of the possibilities for connection of collar beam to the rafter. In German literature this kind of connection is called "Schwalbenschwanzblätter" and it is one of the most used types of connections to join the horizontal to vertical elements. Today many articles describe geometry and behaviour of this connection under load, refer to. [17] [30]

In modern timber constructions instead of the tie beams, a massive concrete slab transfer tension forces. Alternative connection of rafter to tie beam is used then. In this work, modern types will not be discussed, but more modern types of this roofs and their respective connection details can be found in [14] [23] [29].

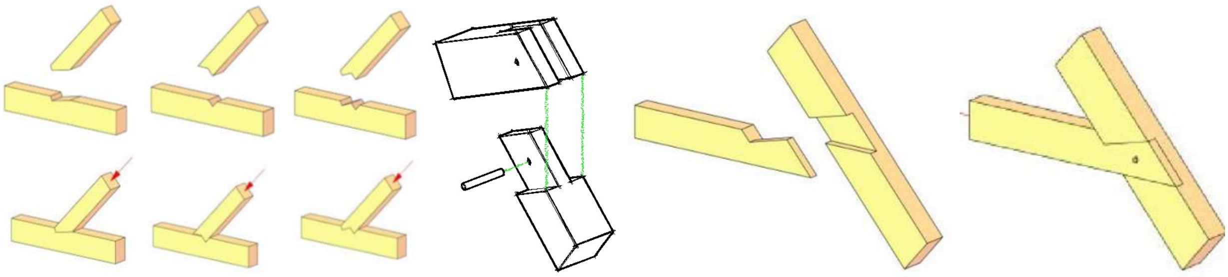


Figure 1.23 Common connections in rafter roofs [31]

1-2.4 PURLIN ROOFS

Purlin roof became popular in Central Europe around the 18-th century. Since then most of the roofs in that part of Europe but also in the other parts of Europe are made as purlin roofs. Purlin roofs are suitable for small to medium-large pitches up to 40°. Purlin roofs prevailed over rafter roofs, because of smaller wood consumption compared to the rafter roofs that can go up to 50%, easier calculation and easier construction. The main disadvantage of purlin roofs related to rafter roof is lack of space in the attic because of dominating posts.

In this thesis, only purlin roofs from Central Europe region are discussed. For Palladian purlin roofs and other purlin roof types from southern Europe refer to [5]. More detailed information, especially those related to terminology issues are given in Annex B.

In general, purlin roofs owe the name to the purlin, a longitudinal structural member which directly supports the rafter. This fact results with the two main differences between the purlin roofs and rafter roofs:

a) here discussed purlin roofs consist of two different transversal statical system – a principal truss and common / secondary truss, by default;
b) Longitudinal stability in purlin roofs is obtained by the system of purlin – knee brace – post / strut (this system denotes braced longitudinal wall), so wind braces are not needed (in contrast to rafter roofs).

Statical system of principal truss is shown in Figure 1.24 below, and statical system, known as common/ secondary truss in Figure 1.24 upper. The latter one is supported only by purlins and loads from it are transferred via purlin to the main structural system with principal rafters.

Existence of the two carrying systems (principal and secondary truss) affects the way of transferring the load, by combining performances of these two load-carrying systems. Besides the transfer of forces by purlins which is defined as the first system (employing principal truss), the rest of the forces is transferred by rafter load-carrying system (i.e. secondary truss). The distribution between load transferred by these two systems depends upon the pitch of the roof and stiffness of the purlins. For example, for higher pitches, the rafter load carrying system will transfer more load and opposite in case of lower pitches.

Accordingly, to the literature, further sub-classification of purlin roofs characteristic for Central Europe depends on the number of the points in which rafter is supported. In fact, it refers to the number of purlins other than foot purlin which matches the number of either post (as tensional, hanged members) or struts (as compressive members, “chairs”) carrying purlins. This result with the following sub-classifications:

a) principal trusses contain one or two posts (it relates to King / Queen post roofs),
b) the number of vertical struts can vary from one to three (which relates to purlin-tie roofs with standing chairs / vertical struts),
c) the number of sloped struts (as individual members in purlin-tie roofs with either raking or lying / leaning struts) does not exceed two, and
d) “lying chair truss / trestle truss always contains a pair of raking struts which run parallel to rafters, supporting a straining beam. It all goes for purlin roofs which are not multi-storey structures.

Introducing a distinction between post and strut, as well as those between vertical and sloped strut, leads to the division of purlin roofs by statically behaviour. Purlin roofs which contain post(s) within principal trusses do not require a tie-beam supported inside of the span (outer structural walls are sufficient). The same goes for purlin roofs with “lying chairs” whether it is about individual leaning struts or “chair/trestle truss”. In contrast to them, when it comes about principal trusses with vertical struts and raking struts, a tie-beam must be supported inside of span. It requires the interior structural walls positioned right below or close to the points in which the struts are supported on a tie-beam.

When a tie beam is not supported inside of span, new structural elements called braces must be used, to obtain different load transfer. Braces are used here as a simplified name for “principal/passing braces”, denoting braces within principal truss which are mainly parallel to rafters, but do not support them [71])

Purlin roofs are statically divided on braced and unbraced. Unbraced roofs don’t have an additional structural member called braces as shown in Figure 1.8. Purlin roofs which are shown in Figure 1.24 are referred to as purlin-tie roof with single/double standing chairs, in German literature is known as “Pfettendach mit einfach oder doppelte Stuhl“. The literal translation of the term from German would mean “single or double standing chair” can be also found in some construction dictionaries (translated on the Croatian language, these literal translations are commonly used there, too). See Table B1 / Annex B:

To sum up, braced and unbraced purlin roof structures are divided on:

a) UNBRACED
    a. Purlin-tie roofs with the standing chair(s) (“stehenden Stuhl”) Figure 1.24,
    b. Purlin-tie roof with raking chairs Figure 1.25
b) BRACED
    a. King post (“einfach Hängewerke”) Figure 1.28
    b. Queen post (“doppelte Hängewerke”) Figure 1.31
    c. Purlin roof with trestle truss (“liegenden Stuhl”) Figure 1.26

As it is the case in rafter roofs length of rafters is also restricted for purlin roofs. Unsupported length of rafters in all purlin roofs should not exceed 4,5m from foot purlin to intermediate purlin and 2,5 m from intermediate purlin to ridge. Recommended lengths for each roof type are given in Figure 1.32.

Purlin-tie roofs

It should be noted that purlin-tie roofs with struts contain those with vertical struts (“standing chairs / “stehenden Stuhl”), with raking struts (“bockptefften Stuhl”) and lying/leaning (“liegenden Stuhl”) struts. Collar-tie is essentially tensional member which connects strut(s) with principal rafters, running under middle purlins, parallel to a tie-beam. To avoid confusion about used terms, it should be noted that raking struts here refer to sloped members which run toward to rafters (at an angle to the rafters) while lying struts are parallel to the rafters. In both latter cases, struts are set in pairs (see also Appendix B). Taking account, the way of transferring the load to the walls, structural systems with lying struts do not require intermediate walls. Therefore, these systems will be here considered together with purlin roofs with trestle truss.

Purlin-tie roofs with the standing chair(s) (“stehenden Stuhl”)

As defined purlin-tie roof is unbraced type of purlin roof. Because posts are subjected to compression and they are direct support for rafters, large loads are transferred to the tie beam. Tie beam is therefore subjected mainly to bending in combination with tension. Because of such unfavourable type of stress, this kind of roofs seeks for intermediate supports inside of span. To avoid high bending stresses in the tie beam in most of the literature, the distance between struts and intermediate supports for this kind of roof is limited to 0,6-0,8m. If distance is larger, purlin-tie roof with raking struts should be considered.

Speaking about purlin-tie roofs with the standing chair(s), braces can be introduced without changes in load transfer for vertical loads but braces significantly stiffen structure subjected to horizontal loads and give transversal stability for steep pitches. Therefore, roofs with vertical struts can be constructed as braced, but usually, the braces are neglected as shown in Figure 1.24. Area of application is shown in Figure 1.32. (cf. [1])

In area of Graz an interesting alternation of the purlin-tie roof was developed through time known as “Grazer Dachstuhl”, for description and illustrations refer to [16] [17] [32].

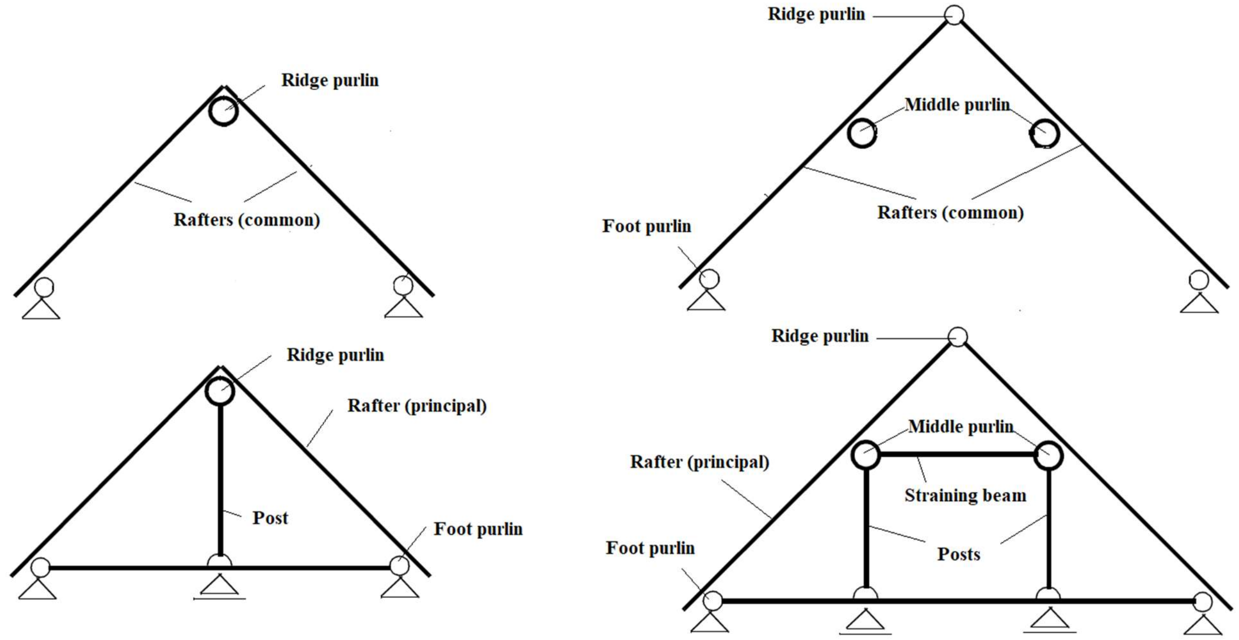


Figure 1.24. left: Purlin-tie roof with the single standing chair-unbraced (up-common truss, down-principal truss); right: Purlin-tie roof with the double standing chairs-unbraced (up-common truss, down-principal truss) [16]

Purlin-tie roofs with raking chairs (“Bockptefftendach Stuhl”)

More alterations of purlin-tie roofs with struts can be found in literature, and among them, an example of this one with raking chairs is shown in Figure 1.25. Raking strut roof is braced, nevertheless, loads are still transmitted in way characteristic for strut roofs. This roof is suitable for larger spans where only one intermediate wall is present. The solutions with two intermediate walls also exist.

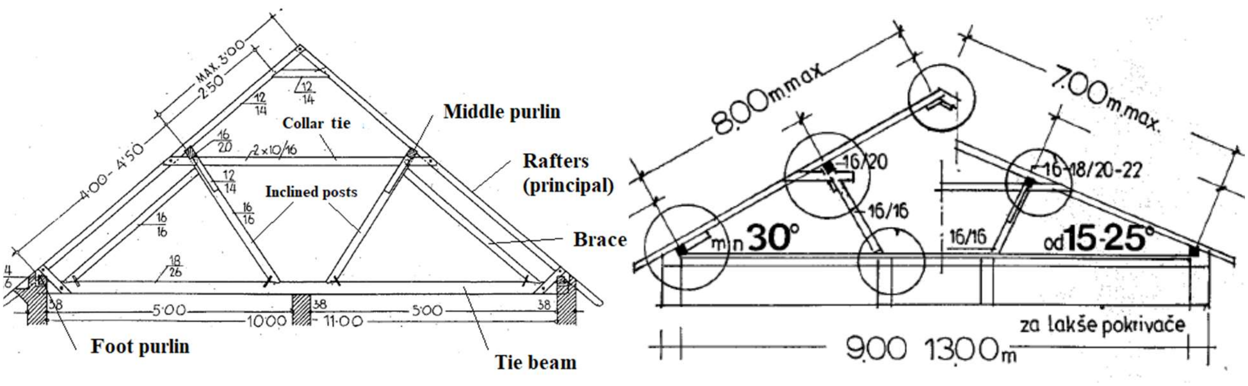


Figure 1.25. Purlin-tie roof wth raking chairs (one intermediate wall); right: two intermediate walls

Purlin-tie roof with lying struts and purlin roof with trestle truss (“liegenden Stuhl”)

Purlin roof with trestle truss presents a braced type of a purlin roof. In German literature can be found under the term “liegenden Stuhl”, in a sense of complex structure in which a pair of raking struts in conjunction with straining beam and soulaces acts as a portal frame. This system with its structural parts is shown in Figure 1.26 left, while on the Figure 1.26 right, a purlin-tie roof type with double lying struts is presented. This latter one has a ridge purlin, supported by struts themselves, while longitudinal stabilization is provided by knee braces. Both these structural systems leave attic space free (for their principal trusses, it means up to the level at which the horizontal members are set). Presented types, as well as the purlin-tie roof with raking chairs (as presented in Figure 1.25 left), can be used instead of the purlin-tie roof with standing chairs when the spacing of vertical struts from inner structural walls exceed 1,5 m. [1].

If the attic wall is erected, additional ties (so-called "short / little" collar-ties) similar to those on Figure 1.31 are introduced to reduce trust in the attic wall. (cf. [16] [1] [17])

Similar to a collar beam roof with trestle truss, a purlin roof with trestle truss is also suitable for large spans, therefore in the medieval period this type of roof can be mainly found in Churches and Cathedrals. Larger spans are obtained in the same way as in trestle roof by building up levels as previously explained. Furthermore, king post can be introduced for the same reason as in collar beam roof with trestle truss. Example with ornamental "crown post" applied (instead of king post) is given in Figure 1.27 (see also Annex B).

A ridge purlin is required if the rafter section from the middle purlin to the ridge is more than 2.50 m long. The ridge purlin is supported in a way which is shown in Figure 1.26 right (by lying struts extended to the ridge and overlaid with rafters). In this case, the horizontal element is not straining beam but collar beam which indicates that raking struts, and collar beam don't form the trapezoidal frame. Therefore, this roof is referred to as a purlin-tie roof with double lying struts. If larger roof space is required, the Mansard purlin roof can also be constructed with a knee pole. For Mansard roof type refer to [16] [19] [1].

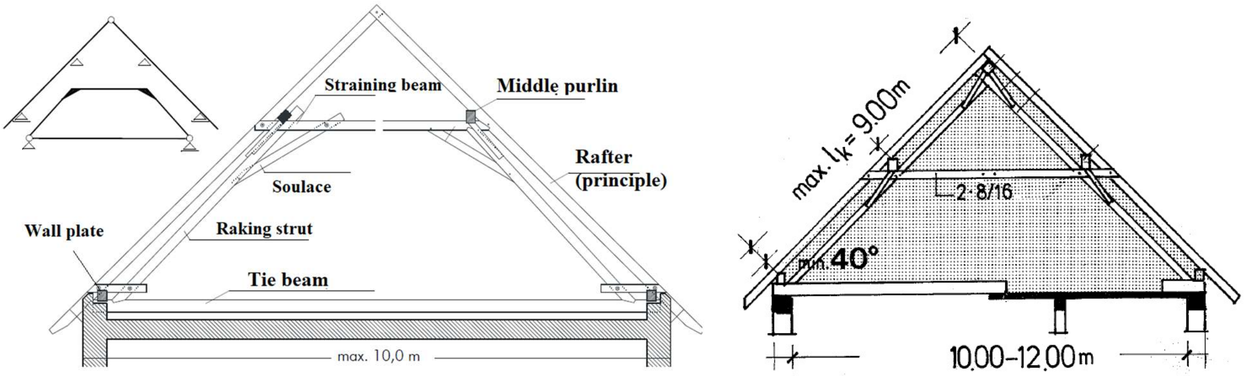


Figure 1.26. left: Purlin roof with trestle truss [16]; right: Purlin-tie roof with double lying struts [29]

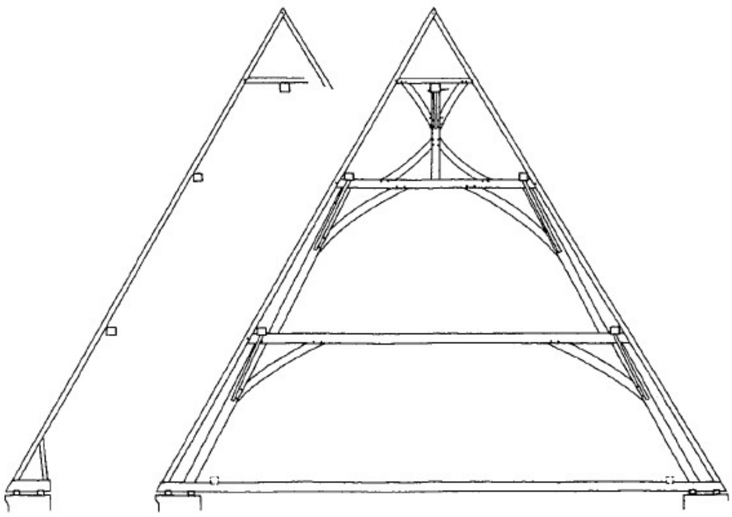


Figure 1.27. Multi-storey roof with two trestle trusses and collar beam with crown post in the upper portion oft he roof " Kreis Tecklenburg year 1500" [4]

King post ("einfach Hängewerk")

For spans up to 8,0 m without intermediate structural walls (supports) king post trusses are economical. Without intermediate supports, load transfer of king post truss significantly differs from the purlin-tie roof with the standing chair. King post denotes a central vertical member always subjected to tension (see also Annex B). King post truss with names of structural elements is shown in Figure 1.28. This type of structure is also a very popular statical system for timber bridges. King post truss is a one-degree indeterminate structure. King post has one purlin that is placed under a ridge in standard king post trusses, although in rare cases it can support intermediate purlins, as it is the case on roof structure at "*TU Graz Alte Technik*" see Chapter 2. Braces and a tie-beam are most stressed constitutive structural members. As it is shown in Figure 1.28 "short collar-tie" is introduced to reduce horizontal trust on the attic wall (which essentially has cantilever behaviour) and therefore reduce risk of tumbling. This structural member is a tensional member which is common when overhang rafters rely on foot purlin above the attic wall. In this case, it connects a principal rafter to the brace (principal/passing) above foot purlin which relies on a tie beam. For the symmetric loads King post truss acts as truss, loads from common and principal rafters are transferred to foot purlin and ridge purlin. 3D illustration of load transfer for symmetric load is shown in Figure 1.28. Load transfer can be furthermore presented by internal forces diagrams for symmetric and asymmetric loads in Figure 1.29. It confirms that a tie-beam receives tensional axial stresses, despite the character of the load.

This roof type can be found in the roof structure of "*TU GRAZ "Alte Technik*". Connections regarding this type of the roof structure are explained in Chapter 2-1. Area of application is shown in Figure 1.32.

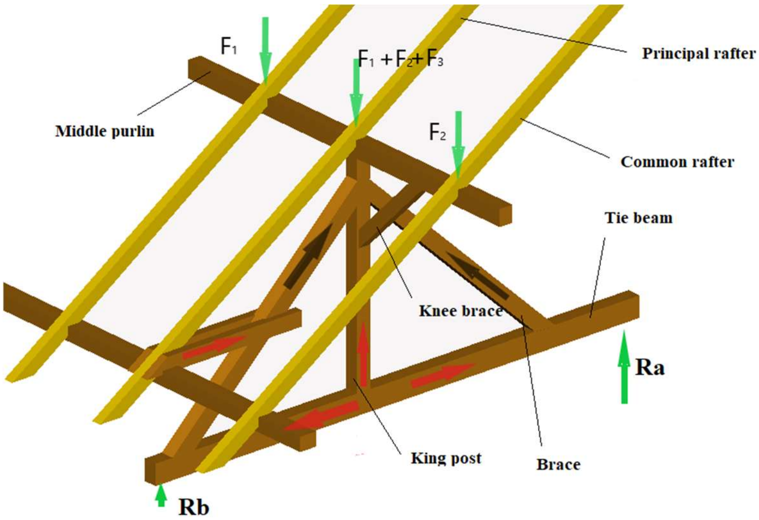


Figure 1.28 load transfer and elements of the King post truss, (King post truss in the structure of "Tu Graz Alte Technik")

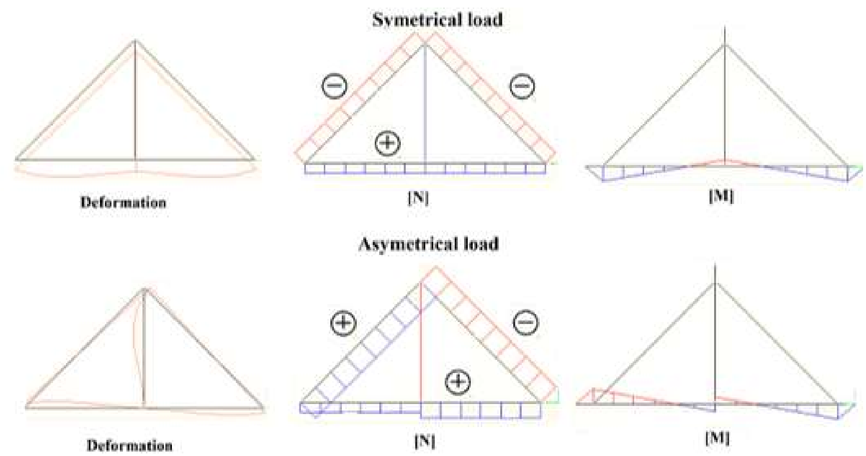


Figure 1.29. Behaviour of King post under various loads

Queen post ("doppelte Hängewerk")

For spans from 8 to 13,5 m without intermediate structural walls (supports) queen post structure is used. Queen post is a one-degree inner indeterminate structure with tensional posts which are set in pair raising from a tie-beam (see also Annex B). Subsequently, queen post is capable of load distribution up to some point. This is one of the main advantages for this type of roof, although in case of failure of one structural element new question arises about the overall capability of connections to receive this load redistribution in way of transferring tension forces that may occur. For that reason, a detailed analysis should be conducted refer to Chapter 5.

Queen post roof structure with names of structural elements is shown in Figure 1.31. Essential structural members (within Queen post principal truss) which transfer load are: braces, posts, tie beam and straining beam, as they are making a stiff trapezoidal shape. In contrast to a king post, load transfer in queen post differs in case of symmetric or asymmetric loading. For symmetric loading braces and straining beam are subjected to compression and tie beam to tension without major bending. In case of asymmetric loading queen post resists with bending of the tie beam see Figure 1.30. 3D illustration of load transfer for symmetric load case is shown in Figure 1.31. Because of possible tension and compression forces in different structural elements, connections need to be designed to transfer compression and tension forces. In case when the connection of braces to queen post does not transfer tension, alternative load distribution pathway is obtained through the post, although with a consequence of larger deformation. In cases where the roof rises above the attic wall, the „short collar-ties" obtain the same function as previously explained in Figure 1.31.

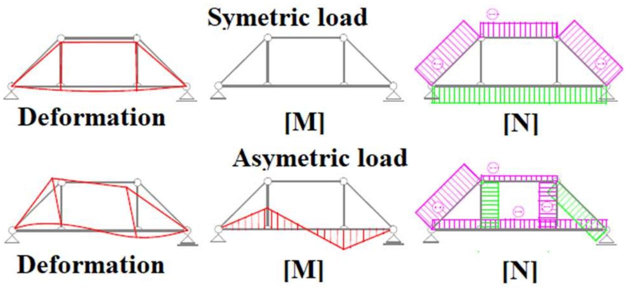


Figure 1.30. Behaviour of Queen post principal truss under various loads [16]

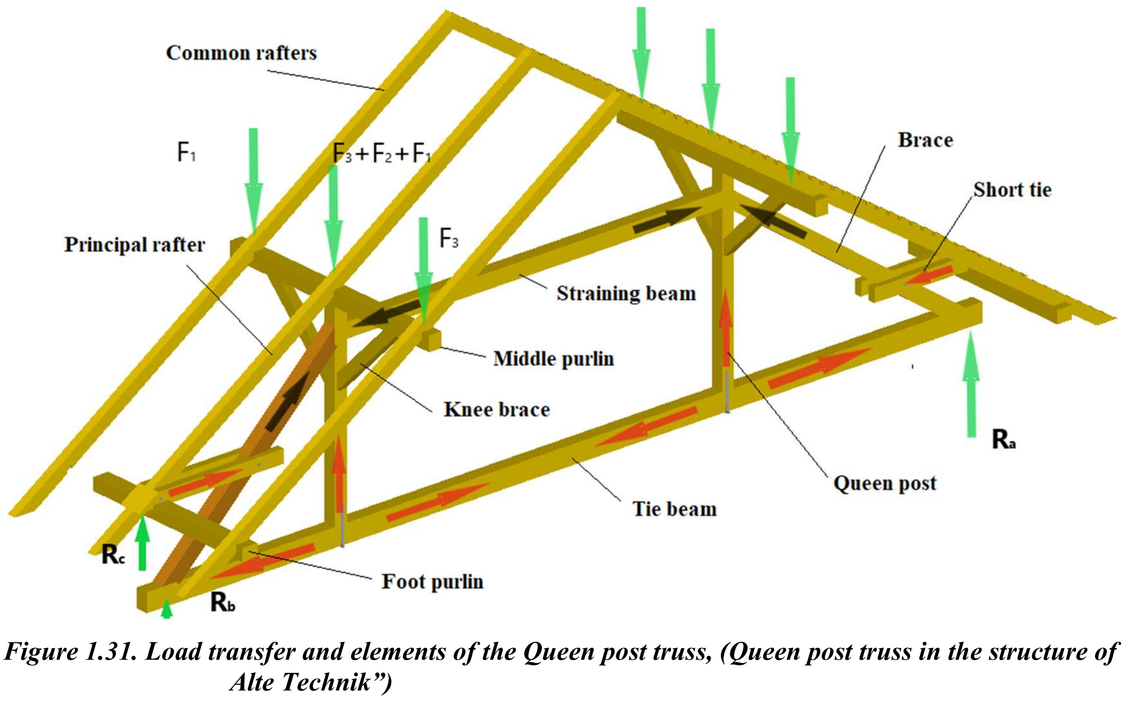


Figure 1.31. Load transfer and elements of the Queen post truss, (Queen post truss in the structure of "Tu Graz Alte Technik")

1-2.5 GENERAL AREA OF APPLICATION

For the conclusion table of general overview and area of application for various rafter roofs and purlin roofs from the region of Central Europe is given. In Figure 1.32 a table with roof structures from this chapter and also relevant roof structures found in the literature [14] [29] [1] are presented. For some additional explanations and terminology issues, see Tables B1 and B2 (Annex B).

This table aims to present general information for the reader and give a better view for comparing different roof structure. Also, it can be helpful in the first phases of assessment of the roof structure or design. In other words, to reveal if the roof structure corresponds to the generally accepted dimensions, spans, etc. which are defined by experience. This can point to critical points in the structure and to draw attention to them. Moreover, for each roof structure, structural weak points, and the critical areas from the standpoint of durability are given

For roof structures which are the result of a combination of different systems of rafter and purlin roofs, and roof structures for which are not in common use especially referring to roof structures of churches and cathedrals only provisional numbers are given. For better understating refer to [4] [17]

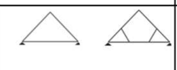
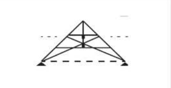
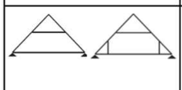
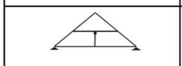
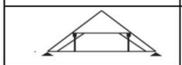
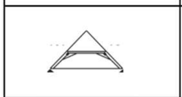
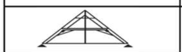
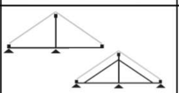
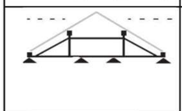
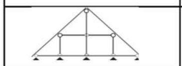
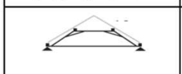
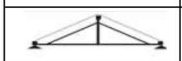
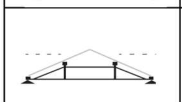
		Name	max.Span [m]	Pitch [°]	max Collar/ Straining beam length [m]	Purlins	Statical in determination	Out of plane raster [m] common (principal) trusses	max.Rafter length [m]	Critical areas from standpoint of durability, and structural disadvantages
RAFTER ROOFS		Common rafter roofs (RR-C) / RR-C with additional struts	7,0	40-60	-	NONE	0/2	0,7-1,0 (-)	4,5 - 5,0	Connection of raking strut to rafter, rafter foot
		Open RR-P with king post and scissor-braced truss	7,5-9,0	40-60,	6,0	NONE	more than 3	0,7-1,0 (-)	7	Connection of scissor braces to rafter= failure of wooden pegs, rafter foot
		Rafter roofs with collar beam (RR-CB) / RR-CB with additional struts	9,0	40-60	4,5	NONE	1	0,7-1,0 (-)	7,0-8,0	Rafter foot, to short shear length for connection of rafters to tie beam
		Collar beam roof with single supported collar beam-unbraced	9,0-12,0	35-60	5,0	NONE	2	0,7-1,0 (3,5-4,0)	7,0-8,0	High bending stresses and deformation of the tie beam
		Collar beam roof with double supported collar beam-braced	up to 14,0	35-60	5,0-6,0	NONE	1	0,7-1,0 (3,5-4,0)	8,0-12,0	High bending stresses and deformation of the tie beam
		Collar beam roof with trestle truss	8,0-12,0	35-60	4,5	NONE	1	0,7-1,0 (-)	7,0-7,5	Rafter foot, concetration of connections from rafters and raking struts causes large weakening of the tie beam
		Combined RR-P	<16,0	35-60	4,5	NONE	UNKNOWN	0,7-1,0 (-)	4,0-4,5 per level	Same as Trestle truss
PURLIN ROOFS		Purlin-tie roof with single standing chair-unraced / braced	6,0-8,0	30-45	-	1	1 (inner) 1	0,7-1,0 (3,5-4,0)	4,5	Leave enough air between the foreheads of the rafters., in order to avoid over loading in foot purlin, foot purlin if attic wall is present
		Purlin-tie roof with double standing chair-braced	8,0-12,0	30-45	5,0	2	1	0,7-1,0 (3,5-4,0)	7,0-7,5	Missing ridge purlin causes deformation of ridge for larger spans, decay of foot purlin due to moist and water
		Purlin-tie roof with triple standing chair-unraced	up to 16,0	30-45	-	3	3 2(inner)	0,7-1,0 (3,5-4,0)	8,0-10,0	Rafter foot, foot purlin
		Purlin roof with trestle truss	10,0-12,0	up to 55	-	2	UNKNOWN	0,7-1,0 (3,5-4,0)	7,0	Deformation of tie beam and straining beam for larger spans
		King post truss	5,0-7,5	30-45	-	1	1(inner)	0,7-1,0 (3,5-4,0)	4,0-4,5	Same as purlin-tie roof with single standing chair
		Queen post truss	9,0-13,0	30-45	5,5	2	1(inner)	0,7-1,0 (3,5-4,0)	7,0-7,5	decay of foot purlin due to moist and water, to short shear length for connection of braces to Tie beam

Figure 1.32. Table with an overview and general application area of historic roof structures

1-3 APPROACH TO THE PRESERVATION OF CULTURAL HERITAGE

Architectural heritage reflects the nations culture, tradition, and history. When spoken about architectural heritage in this work, it refers to buildings and its structures. It should be kept in mind that heritage is often not only presented from art or architectural point of view. In other words, visual identity and appearance of the historic building is not the only important part of its identity. This last sentence marks the problem when referring to the broader community because this alternative view which introduces less visible and known topics in building such as material and structural authenticity, construction technology are insufficiently presented to the broader community or investigated enough and supported by experts. Subsequently, this problem refers to some engineers which don`t have acceptable span of expertise in historic buildings, but somehow in some regions they are still enrolled to take part in reconstruction of these structures, which often results with incompetent design and solutions which lead to devaluation of these structures, which are part of cultural heritage. This can be also expanded to contractors, which often in spite competence of the engineers and conservation architects, decide to make rearrangements because of economical or time management reasons, again to the expense of cultural heritage. Nevertheless, global awareness about the importance of preservation of cultural heritage is rapidly increasing as can be recognised by work of following organisations or committees:

- Work of comities: CEN/TC 250/WG 2 Assessment and Retrofitting of Existing Structures [33] and CEN/TC 346, Conservation of Cultural Heritage [34] on developing the standards for a new generation of Eurocodes which will have an emphasis on the structural part of the cultural heritage.
- CEN/TC 346/WG 10 a part of the committee which focuses on the historic timber structures
- Vast numbers of Principles and recommendations given by ICOMOS and ISCARSAH, see following literature [35] [36] [37] [38]
- Developed ISO 13822 Bases for design of structures-Assessment of existing structures [39], see Chapter 3-1.1
- Guidelines for the On Site Assessment according to Cruz [40] used as a basis for a draft version of the norm (prEN 17121) work result of CEN/TC 346. See Chapter 3-1.2

For maintaining and upgrading global awareness on this topic global institutions are responsible. (cf. [41] [42] [38] [37])

Due to the increasing number of heritage buildings which require restoration works, it is not surprising that conservation interventions in existing buildings represent an important part of the contemporary construction industry. A heritage intervention projects are complex, demanding and interdisciplinary process with a lot of involved participants. Therefore, it is important to establish a system which provides better information exchange and managing the interventions to maximize the performance in all life cycles of the preservation project (see also Chapter 3) and at the end success of the project which mainly reflects on how successful the heritage building is protected or valorized. (cf. [38] [37])

During history, structures were mainly made from timber, brick, and stone, see Chapter 1-1. As the time needed for timber to deteriorate in comparison to stone or brick is significantly smaller it is easy to comprehend the vulnerable part of historic structures. In addition to that in historic structures timber was mainly used for the roof structures, where damped conditions pronounced deterioration of the timber. During past and today, a battle to preserve and keep timber roof structures in the original state presents a great challenge. Therefore, an effort has been made to develop the main principles for the preservation of timber structures. As mentioned, architecture and subsequently structures differ from region to region and they are tightly bonded to the culture and tradition. In compliance with that respective approaches to conservation, reconstruction, etc. also differ and they are bonded to the culture. Nevertheless, some fundamental ideas and principles on approach to the conservation of buildings and structures present to be the same in the world. As a consequence of these similarities and globalisation phenomena, the need for internationally recognised principles emerged, subsequently, international institutions and organisations developed. The most important ones are here given and described. (cf. [41])

a) UNESCO - (United Nations Educational, Scientific and Cultural Organization) established in Paris in 1946. From other purposes, we can distinguish one relevant for this topic, and that is the preservation of heritage buildings and structures. The buildings and structures enrolled in this list must satisfy numerous criteria. Those criteria can be found in [43] Buildings and structures on the list are legally protected by international treaties. Those need to preserve their condition during the time, and they need to be maintained and if needed reconstructed accordingly to principles given by UNESCO. (cf. [44])

There is no need to emphasise that only the most important sites (buildings and structures) are on this list. That doesn't mean that buildings and structures which are not on the UNESCO list should be not considered and protected. On the contrary, their valorization is necessary in order to classify them by their importance and significance, enabling to the competent institutions and authorities to make decisions about possible interventions (reuse, management, etc.), including those to either prevent rapid deterioration or emergency measures (eg. repair or collapse insurance, etc). These buildings and structures present majority and if not adequately preserved, during time irreversible damage to the culture and tradition can be made. For example, this can be foreseen in old town part of cities where if the strategical approach to the preservation of buildings and structures is not planed, because of fast urban development, and economic reasons there is a big probability that majority of these structures could be devaluated. Countermeasures, in this case, must be introduced trough several levels such as a presentation to the general population about the importance of cultural heritage preservation, introduction of legislation on the country level, etc.

b) ICOMOS – (The International Council on Monuments and Sites) *is an international non-governmental organisation of professionals, dedicated to the conservation of the world's historic monuments and sites* [41]. It was founded in 1965. ICOMOS main idea is to develop principles for preservation of historic buildings and structures which could be implemented globally. The work on this topic resulted with Venice Charter (1964) [35], but the complaints that charter is mainly suited for European countries and masonry structures, lead to the introduction of the Nara Document on Authenticity (1995). ICOMOS is a supporting institution to UNESCO.

1-3.1 PRINCIPLES OF PRESERVATION OF TIMBER STRUCTURES ACCORDING TO ICOMOS

Main principles for preservation of timber structures which should be closely followed by conservation architects, engineers, and contractors are given in [35] [38]. Updated version in [36]. Nevertheless, these principles only give general guidance on how the conservation process should be approached. Respecting the cultural diversity is in the pinpoint of conservation principles for timber structures developed by ICOMOS. Further points are given according to [35].

a) recognise the importance of timber structures from all periods as part of the cultural heritage of the world;

b) take into account the great diversity of historic timber structures;

c) take into account the various species and qualities of wood used to build them;

d) recognise the vulnerability of structures wholly or partially in timber due to material decay and degradation in varying environmental and climatic conditions, caused by humidity fluctuations, light, fungal and insect attacks, wear and tear, fire and other disasters;

e) recognise the increasing scarcity of historic timber structures due to vulnerability, *misuse and the loss of skills and knowledge of traditional design and construction technology;*

f) take into account the great variety of actions and treatments required for the preservation and conservation of these heritage resources;

g) note the Venice Charter, the Burra Charter and related UNESCO and ICOMOS doctrine, and seek to apply these general principles to the protection and preservation of historic timber structures;

1-3.2 CONCLUDING REMARKS

To preserve the cultural identity of a nation, the nation must work together on cementing the principles of building and structure preservation and protection, trough defining firm nation legislation which supports the preservation of buildings, in compliance with international principles which are previously derived. The consequences of putting the preservation of the structures aside, when dealing with the historic structures in a large extent, at the end can lead to the loosing of the building tradition, knowledge and in the end cultural heritage. In the other hand nurturing the preservation principles doesn't only protect the buildings, tradition, knowledge, culture, etc. but sometimes can have a positive influence on the economic way through tourism development. (cf. [42] [41])

Although preservation and protection are widely recognised when referring to a single monument and significant buildings, problems occur when referring to the historic old town areas in cities. The main reason is often the economic importance of city centre and urban development inside of the city, influenced by many parties which often have different goals and views. It is easy to comprehend how the preservation of the old city areas can be pushed into a secondary role. (cf. [42] [41])

A positive example of nurturing the identity of historic old town area is present in the city of Graz. The old town in Graz is currently on the UNESCO list, because of the extent of heritage preservation, and a great cultural value. Subsequently, an extent plan for the preservation of building inside of city centre is developed. For example, in Figure 1.33 protection zones for the city of Graz are shown. Zones are in connection with regarding legislation. For more information refer to [20] [45] [44].

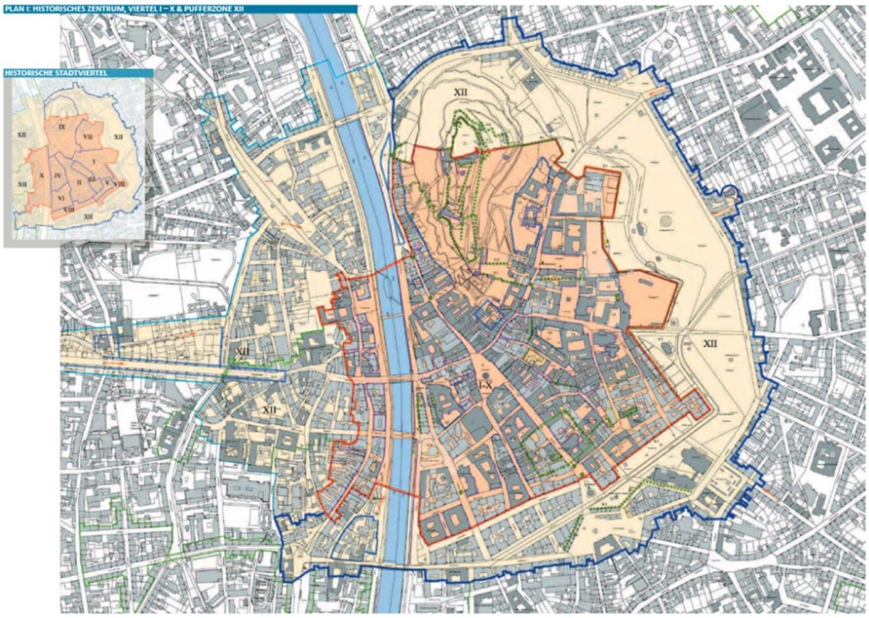


Figure 1.33 Protection zones in Graz city centre [45]

Here several main points concerning the approach to preservation of historic timber structures are given. These points are following international principles and they are internationally accepted. Each of these points is further explained in detail in [46]. The author encourages reading this literature to better understand the scope of principles and actions.

- Development of national legislation
- Education of engineer's, contractors, architects and especially public – for example in Austria several projects in this direction have been conducted [17] [22] [47]
- Securing adequate material (old forest, or forest of the same specie)
- Training program for carpenters
- Monitoring and maintenance of historic structure a close collaboration between landlords, engineers and conservation architects is needed.

From given points, monitoring and maintenance can be derived as most important. Nevertheless, to obtain an efficient and successful system for the preservation of the historic timber structures all given points need to be considered and blended in one system. For further explanation regarding given points, and definitions regarding the topic such as: Heritage, Authenticity, Conservation, Restoration, Preservation etc., refer to [46] [38]

The importance of the maintenance is especially emphasised in Krakow Charter:

Maintenance and repairs are a fundamental part of the process of heritage conservation.

These actions have to be organised with systematic research, inspection, control,

monitoring and testing. Possible decay has to be foreseen and reported on, and appropriate

preventive measures have to be taken. [46]

1-4 BIOLOGICAL DEGRADATION

Wood is a natural material, made from 40-50% of cellulose, 15-25% of hemicellulose, and 15-30% of lignin. [12]. This composition makes wood suitable for insects, termites (shorten pests) and fungi, which destroy wood by using it as a source of food. Even though this is a normal ecological process in nature, it is an unwanted process in timber structures because it can deteriorate wood in several years depending on the scope of the process. In contrast wood without any biological degradation can last for centuries. The most important aspect of deterioration is degradation of mechanical properties. For example, if a wooden beam attacked by various pests or/and fungi lose from 5% to 10% of its weight, mechanical properties can be reduced up to 80%. Wood can be attacked by various pests and fungi in various conditions Figure 1.34. In this paper only, pests which can survive and develop under conditions which are similar to conditions in roof areas are discussed. More information about pests that attack timber in conditions that are not discussed here (e.g. fresh-cut wood) refer to [48] [49].

Wood-damaging insect species	Galleries[a]	Environmental conditions in wood: Moisture (%)		Temperature (°C)	
		w_{opt}	$w_{min–max}$	t_{opt}	$t_{min–max}$
House longhorn beetle (*Hylotrupes bajulus*)	Interior (C)	30–35	9–65	28–30	12–38
Giant horned beetle (*Ergates faber*)	Exterior (C)	60		30	
Common furniture beetle (*Anobium punctatum*)	Interior (C, B)	28–30	10–50	21–24	12–29
House borer beetle (*Hadrobregmus pertinax*)	Interior (C – rotten)	30	19–55	25–26	
Brown powder post beetle (*Lyctus brunneus*)	Interior (B)	14–16	7–23	26–27	18–30

Figure 1.34. Activity of wood attacking insects depending on conditions [49]

1-4.1 INSECT ATTACK

Various species of insect that can develop in timber structures such as wood boring beetles which deteriorate wood, and others which use timber only as shelter (e.g. ants). To understand the process of wood deterioration basic knowledge of behaviour and life cycle of wood boring insects is needed. One generation of insects are represented by several life stages as shown in Figure 1.35 left. Damage to the timber by insects is made during the "Larva" period. The Larva is developed from eggs. Afterwards, Larva bore into the wood and trough the wood where they stay for almost their whole life span when comparing to other life cycles. Duration of this cycle is between 3 to 10 years depending on species. After Larva period, transformation period called Pupa follows, in which Larva metamorphoses to Adult. Adults are breeding and laying eggs in cracks or holes in the wood from which new generations develop. Because of this process trough time, insect infestation can significantly propagate trough elements and structure in a short period. Propagation and existence of insects depend upon temperature, moisture content and wood nutrients. Because such variety of species which can operate under various conditions possibility for infection is high. (cf. [48])

House Longhorn Beetle

House Longhorn Beetle (Hylotrupes bajulus L.). or in German literature known as Hausbock is the most present insect which makes most of the economic damage in roof structures in the area of Austria and Germany. The boundary and optimal living conditions are shown in Figure 1.34. The long range of suitable living conditions makes the House Longhorn Beetle so dangerous. The larvae reach a length of up to 30 mm. The beetles grow up to 25 mm long. Depending on the conditions the life cycle can last from 3 to 10 years. In softwoods such as spruce they can attack the whole cross section, while in hardwoods only sapwood. Beside the exit points which are 1-2 mm in diameter the surface of the infested timber is intact, therefore infestation by this insect is hard to recognize. When boring they usually bore trough earlywood as it has more nutrients. Only after several years of active infestation, the cracks in timber can be observed Figure 1.35 right.

Attack by this insect is also found in the roof structure of TU Graz Alte Technik. The infestation is found during survey on one member Figure 4.5.

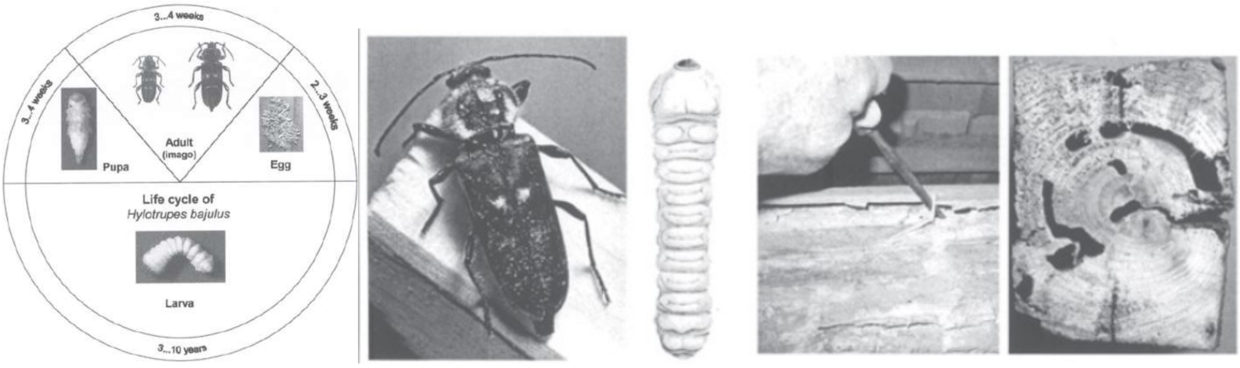


Figure 1.35 left: the life cycle of House Longhorn Beetle; right: House Longhorn Beetle and damage to the timber beam [48]

Common furniture beetle

This beetle can be found in Central Europe and it attacks usually softwood (e.g. Spruce). Adults are few millimetres in size and brown Figure 1.36 left. Larves are a few millimetres in size and yellow. Gallery holes have 1,0-4,0mm in diameter and exit holes are circular with 1,5-2,0mm in diameter, Figure 1.36 right. While boring it produces recognisable Cream-coloured, lemon-shaped pellets. The boundary and optimal living conditions are shown in Figure 1.34. Metamorphosis to adult from larva usually takes form 2-4 years depending on conditions, and adults are emerging between May and August. It can cause damage to the wood depending on the size of infestation. Mechanical degradation is depending on the remaining cross section. (cf. [48] [49] [42] [50])

Figure 1.36 left: Common furniture beetle [48], right: Entry and exit holes from common furniture beetle [42]

1-4.2 FUNGAL ATTACK

Spores of various fungi species are always present in the atmosphere. In conditions of high moist these spores swell, germinate and glue to the wood. After germination, a single, tube-shaped fungal cell called mycelium is established which cannot extract carbon through photosynthesis, but for life, they need to obtain carbon from wood, therefore fungi use cellulose, lignin and hemicellulose from wood. The process of carbon extraction and deterioration goes until fruit body on the wood surface is established. (cf. [42] [48])

When the moisture content of wood exceeds 20% risk from fungal attack is high. Furthermore, temperature plays a big role in fungi development, for most species optimal temperature is in range of 20°C to 35°C. The first sign of deterioration by fungi is: change in the colour of wood, bleached, green, red or blue, brown areas can point to areas infected by fungi. Depending on the colours and visual appearance of the damage, the type of fungal infection can be approximately determined, although for verification laboratory analysis is often needed. There are several groups of fungi depending on their field of attack see [49] [48]. Fungi are usually classified by effects that they have on timber, in other words, if mechanical properties are influenced and which timber components are influenced. This classification recognises two groups: Destroying fungi and staining fungi, subclassification is given in Figure 1.37. (cf. [42] [48] [51])

Destroying fungi			Staining fungi		Surface molds
Brown rotters	White rotters	Soft rotters	Blue-stain fungi	Other staining fungi	
Basidiomycota	Basidiomycota	Ascomycota, Deuteromycota	Ascomycota, Deuteromycota	Ascomycota, Deuteromycota	Ascomycota, Deuteromycota

Figure 1.37 Classification of fungi [48]

Destroying fungi use components of wood for producing needed carbon. In that way by digesting the components wood deteriorates and mechanical properties are heavily influenced. Forth sub classification of destroying fungi is shown with its respective characteristic. We can distinguish three main types: brown, white and soft rot fungi. (cf. [48])

Brown rot

Brown rot attacks mostly softwood in building, it can be found in Central Europe. The optimal temperature for development is around 23°C and moisture content of 50,0%-60,0%. Therefore, only in timber exposed to a direct source of water (e.g. roof leakage), this kind of rot can be expected. After moisture content drops below 20% spreading is terminated. Brown rot attack only cellulose and hemicellulose, while deterioration of lignin is limited depending on species of brown rot fungi. Wood infected by brown rot fungi in the last stage of deterioration is brown or yellow and fragile. Although at the beginning of deterioration colour can be greyish. At the last stage of deterioration, the timber is separated in grain direction on small shiny cubes smaller than 5,0 cm. (cf. [42] [48] [49] [52])

Figure 1.38 Brown rot [42]

Fungus (Serpula lacrymans) is part of brown rot fungi family and it is only true dry rot in brown rot group. It is considered as the most dangerous kind of fungal infection in Europe. Wood gets infected by these species in conditions where moisture content is in region 20,0% – 30,0%. In the last stage of deterioration, timber is deteriorated to big timber chunks larger than 5cm. The timber mainly separates in a direction perpendicular to the grain Figure 1.39. The main characteristic of this species is that when once infected it can survive, although inactive even for several years under dry conditions in places with low ventilation. Furthermore, dry rot fungi can migrate from infected wood to other structural elements with higher moisture content and develop there. In that process even, masonry doesn't present an obstacle for this kind of fungi. This makes them very dangerous as they are hard to detect and once when detected is hard to determine the full range of infection. Subsequently this result with large expenses in repairing. DIN norm in case of dry rot infestation requires that timber in a radius of 1,5m of the recognized area of infestation is removed. Luckily this fungus is not often found in historic timber roof structures (cf. [42] [48] [49] [52] [53])

Figure 1.39. Dry rot [54]

White rot

This kind of fungi attacks cellulose, hemicellulose and lignin. These results with characteristic white or bleached colour of deteriorated timber. In the first stages, colour can be grey, although the red colour is also prescribed to the white rot as a consequence of lignin decomposition products. White rot optimally develops at temperatures around 27°C and moisture content of 40%-60%. It is a common type of fungi attack in roofs where roof leakage is present. Compared to brown rot for the same mass loss strength reduction by white rot is smaller, this is mainly prescribed to smaller degradation of cellulose and hemicellulose. In the last stage of deterioration by white rot timber decomposes along its fibres parallel to the growth rings, and under the finger’s fibres separate and crumble into dust Figure 1.40. When this rot attacks softwood surface is often intact, while the interior is decomposed, which presents a problem in detection of damage. (cf. [48] [49] [52])

Figure 1.40. White rot [54]

Soft rot

Soft rot is another type of fungi attack which attacks usually hardwood but also softwood. Strength reduction can be severe. Infected wood turns to a black colour and fibre structure is decomposed, although the decomposition of the cellulose is slower than in brown rot fungi. Damage and visual appearance of the attack by soft fungi are shown in Figure 1.41. Beside black colour, deteriorated wood by soft fungi is moist, sticky and soft on touch, which differs from characteristics of other species. (cf. [55])

Figure 1.41 Soft rot [55]

1-4.3 MECHANICAL DAMAGES

In contrast to biological degradation mechanical damages presents a group of damages often caused by a human factor. These damages can cause lack of stability or load bearing capacity of damaged elements or whole structure. Although the historic timber roof structures usually have spatial load transfer and subsequently poses sufficient redundancy to withstand these mechanical damages, accumulation of them, can lead to failure of one part or whole structure and therefore mechanical damages must be considered cautiously during the inspection in the same manner as biological degradation.

These damages can be a result of: structure overload in the past, bad construction, bad initial design if design ever existed, and badly undertaken strengthening's or reconstructions.

To understand if some types of mechanical damages influence bearing capacity and stability of the structure, or they don't have significant influence (e.g. longitudinal cracking in members due exceeded tensile strength perpendicular to grains due to the MC changes/shrinkage) it is important to understand mechanical properties and behaviour of the timber under various loads as well as understand load transfer principle in the observed structure.

There are many types of mechanical damage which can occur to the structure or its elements, it is impossible to present them all in an organised way. Here only the most important types of mechanical damages and some critical areas will be expressed. For a more detailed presentation of mechanical damages and its causes refer to the various literature such as [56] [57].

Damages due to the overload of structural elements are usually represented by extensive deformations followed by cracks. A most dangerous type of failure is a brittle failure by bending and tension as shown in Figure 1.42 right.

Cracks in old timber structures are impossible to avoid due to the dimensional instability of the timber. This is furthermore contributed with the usage of cross sections with included heartwood and pith. These cracks are longitudinal and beside reduction in flexural stiffness, torsional stiffness and torsional load carrying capacity (ULS verifications should be carried out on remaining cross section), no critical reductions are remarked, Figure 1.44 left. However, if these cracks occur in areas where tension perpendicular to the grain in combination with shear exist these kinds of cracks can become critical for load bearing capacity and strengthening measures must be undertaken. This points out the importance of understanding the principle of the load transfer trough structure.

Moreover, cracks in connections present usually a serious problem and they reduce bearing capacity of connection and structure because of the complex state of stress in this area and reduced load transfer area. This also affects the load bearing capacity of bolts situated in these connections or cracked areas. Therefore, cracks in connections are often critical and they need to be assessed during inspection. Figure 1.44 left (cf. [57])

Biological degradation directly influences the mechanical properties of the wood and the strength of elements, therefore they are a direct cause of mechanical failures as presented in Figure 1.42 left.

Figure 1.42. left: Decayed beam – result of the fungal attack; right: Cracked beam – indicates a brittle fracture (bending failure) due to the overload [57]

During the exploitation of the structure damages produced by inadequate: usage, reconstructions, and strengthening are common. In some cases, essential members are removed from the structure to obtain larger living space (e.g. removal of collar beams in rafter with collar beam roof) Figure 1.44 right. These actions usually result with extensive deformations of the roof structure and serious decrease in load bearing capacity. Strengthening measures should improve the behaviour of the structure, but badly designed or undertaken measures can induce even larger damage to the existing structure. More described in Chapter 6.

As traditional timber to timber connections transfer load by contact, changes in moisture can cause a disconnection between connected elements. Disconnection of connections and discontinuity in connection which interferes a transfer of the forces can severely reduce load bearing capacity and stability of structure Figure 1.43 right (cf. [57])

To sum up every part of a structure which shows alternation in shape, large deflection or severe critical cracking which is not only consequence of moisture changes should be furthermore inspected, documented and analysed. In the end, the answer needs to be provided upon the need for strengthening of the element to preserve structural stability and bearing capacity upon satisfaction safety factor.

Mechanical damages found in roof structure which is the topic of this thesis are described in Chapter 4.

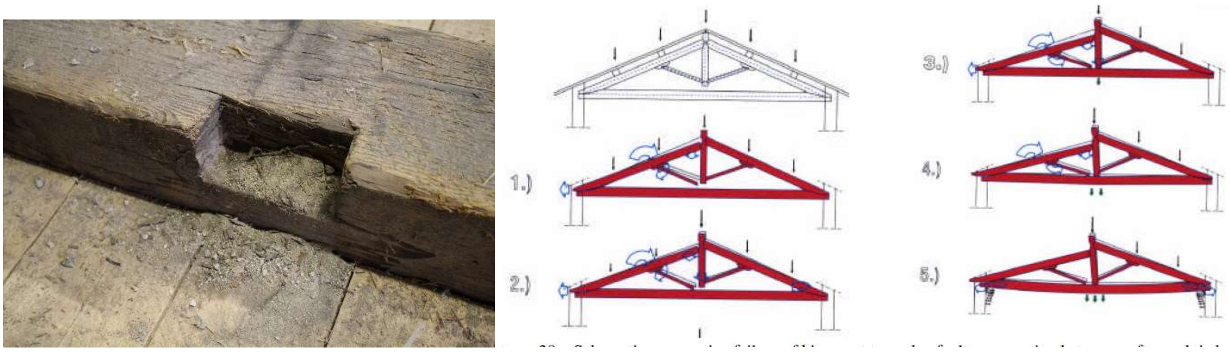


Figure 1.43. left: Damaged beam “TU Graz Alte Technik”; right: Failure of the truss due to the disconnection of the connection [57]

Figure 1.44. left: Cracks in brace at "TU Graz Alte Technik" (green uncritical, red critical) right: Removed collar beam in order to provide room for chimneys "TU Graz Alte Technik"

1-5 APPROACH TO PRESERVATION AGAINST BIOLOGICAL ATTACK

As historic timber structures present big cultural value. Great attention should be introduced in the suppression of biological attacks by insects or fungi as they can irreversibly damage the structure in a short period. Today there is a vast number of biocide chemical products which protect the timber from these kinds of effects, but their usage is questionable in regard of ecology, furthermore, it can chemically affect timber and reduce its strength. All of these effects are unwanted in the historic timber roof structures, therefore more classical ways of preservation should be nourished. (cf. [41])

In history, there are several examples where old civilizations like Greeks and Romans, protected wood from biological attacks. More techniques have developed during the medieval period. The techniques usually come from experience and products which are available in vicinity. [34]

These preservation techniques are based mainly on usage of: oils, tars and resins to form more durable wood, with impregnation. Timber was often treated with white chalk against the pesticide, and in some examples, the timber was submerged in a seawater for some time, from which the timber obtained better durability.

The Greeks and Romans used oil, tars and resins, extracted from resistant timbers, to preserve structures, for wood preservation purposes they used various animal, vegetable and mineral oils. For example, Roman statues were treated with olive and cedar-oil. [41]

Effectiveness of some of the techniques which are here given are proven by modern science, therefore should be used as a more acceptable alternative to biocide chemicals. (cf. [41])

Besides treatment of wood, it is even more essential and most effective for the structure durability to keep the climate conditions inside of the roof structure area unhospitable for fungal and insect attacks in combination with regular inspections.

1-6 TYPICAL ROOF DAMAGES

As a result of previous subchapters regarding biological and mechanical damage, the summary of the typical damages in the roof structures are presented in Figure 1.45.

Damages to roof structure				
Damages related to high moisture			Damages not related to high mositure	
Possible causes for high moisture content	Biological	Mechanical	Mechanical	Chemical
Roof leakages Lakages aaround metal scheeting (chimney areas, eaves, etc.) Using attick for drying Condensation insufficient ventilation	Insect attack various insects various termites Fungal attack - brown rot -dry rot -white rot -soft rot	Longitidunal cracks due to schrinkage Extent deformation Disconecction of connections	Corosion of iron joinery Disconecction of connections (due to loading) Tension craks in members (due to bending or perpendicular tension) Cracks from perpendicular tension in connections Removed or missing elements Badly undetraken retrofittings Poor original design	Corosion of timber Microstructure - destruction by chemically aggressive media

Figure 1.45 Typical damages in the roof structures [16] [19] [57]

CHAPTER 2: THE ROOF STRUCTURE AT “*TU GRAZ ALTE TECHNIK*”

2-1 GENERAL INFORMATION

The building of Tu Graz Alte Technik where roof structure discussed in this thesis is situated was built in the period from 1884 to 1887. Finally, the official opening of the building took place on 12 December 1888. (cf. [18]) The building itself consists of four side wings (North, South, East and West) and one middle wing which divides inner court on half Figure 2.1. The roof structure discussed in this thesis is situated on the southern wing of the building.

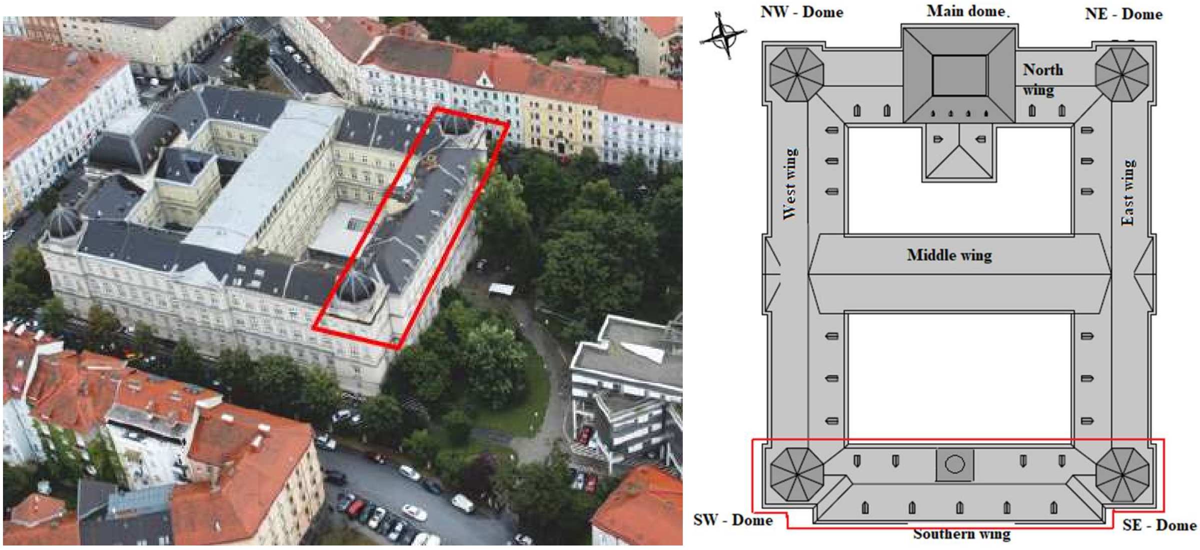


Figure 2.1. left: Sky view to the building of “TU Graz Alte Technik”; right: Schematic top view of “TU Graz Alte Technik", with marked wings, translation from [18]

The southern wing of the building consists of a hip and valley roof bordered with the domes on the East and West. Southern wing together with domes is 65,67 m long and 13,85 m wide including the masonry walls. By excluding the domes dimension of hip and valley roof is 49,63 m long and 13,85m wide.

This thesis deals with the roof structure on the southern wing, which will be analysed in detail including structural analysis, where for domes: plans, sections, and assessment of structural condition together with a short description of the structural system will be given.

When referring to the roof structures in this thesis both domes are excluded, as they will be separately accessed. This divide is mainly made to keep the scope of work on a reasonable scale. Furthermore, structurally domes are independent from the roof structure, therefore, it makes logical sense to analyse them separately.

In Figure 2.2 and Figure 2.3 original blueprints of the roof structure and observatory are shown. Comparing Figure 2.2 to Figure 2.10 several differences can be observed.

The heights on the northern and southern attic wall are in original blueprints the same, but in the observed condition, the south side is higher than the north side. Moreover, iron cramps which have been foreseen by original design on all principal trusses in the connections of a brace to a tie beam and a brace to a queen post are missing on most of the principal trusses.

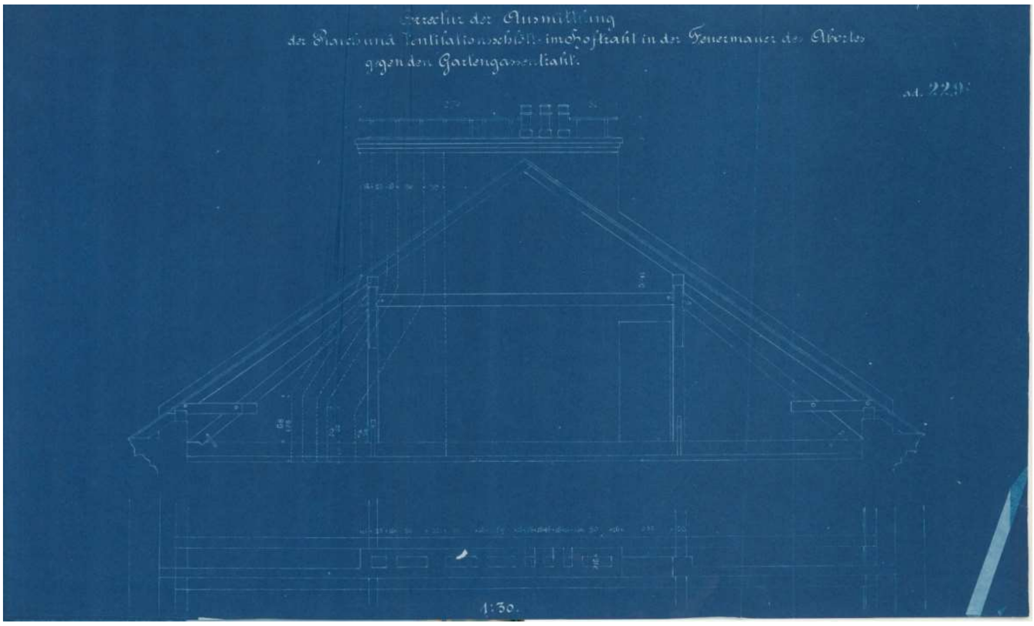


Figure 2.2. Cross section of the Queen post truss at "TU Graz Alte Technik" (Original blueprint)

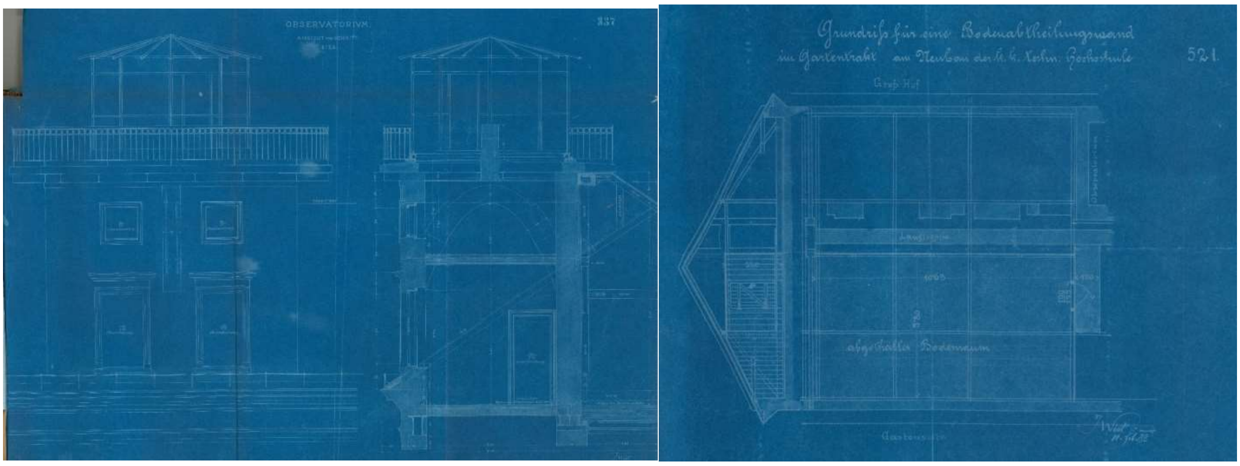


Figure 2.3. Original blueprints of the observatory at "TU Graz Alte Technik"

Roof structure at the southern wing is divided into 3 rooms by firewalls. In room 1 an entrance from the hallway to the roof structure is marked in Figure 2.5. The roof structure is divided longitudinally in 18 axes and transverse in 5 axes Figure 2.5, Figure 2.6. Across the structure, several structural systems alter, therefore the analysis of the roof structure is complex and time consuming. The best way for the reader to comprehend the structure is to refer to a 3D view of the structure Figure 2.11. Structural overview and alternations will be explained and divided by rooms.

Axes 1 and 18 are indicated on plans but they will not be assessed as they only represent ends of the roof structure where rafters are supported on the foot purlin with a birdsmouth joint.

In the room 1, on the area from axes 15 to 17, where the attic space is split into a separate compartment by an introduced corridor/hallway, roof structural system is a purlin-tie roof with double standing chairs (triple if we include axis A1-A), as it is shown in Figure 2.8. The similar intervention has been made for principal truss in axis 14 (which is a borderline of room 1), too (see Figure 2.9).

In room 2, the latter one is also applied in axe 13, while modifications from axes 12 to 10 were made similarly, as previously presented for axes 17 to 15. In the same room, the principal truss from axes 9 to 6 is a queen post truss. Cross section of the queen post truss is shown in Figure 2.10, view of the roof structure in room 2 is shown in Figure 2.4.

Figure 2.4. View on the roof structure in room 2

Room 3 is similar to room 1, only mirrored, therefore axis 2 is same as axis 17. From axis 5 to 3 supporting structure is a queen post truss.

The characteristic cross sections, ground plans, and longitudinal sections of the roof structure and the domes are presented in scale in Appendix D.

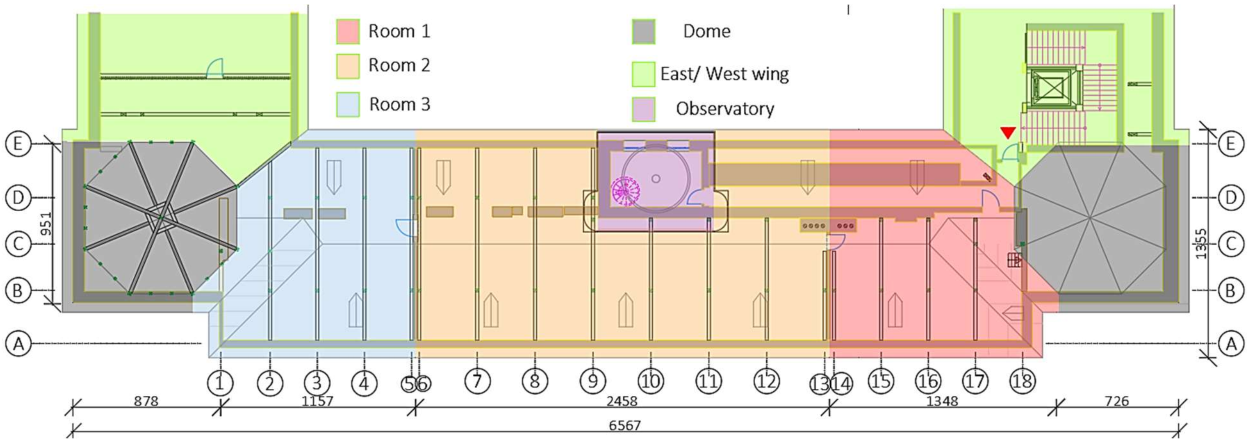


Figure 2.5. Top view roof structure "TU GRAZ Alte Technik"

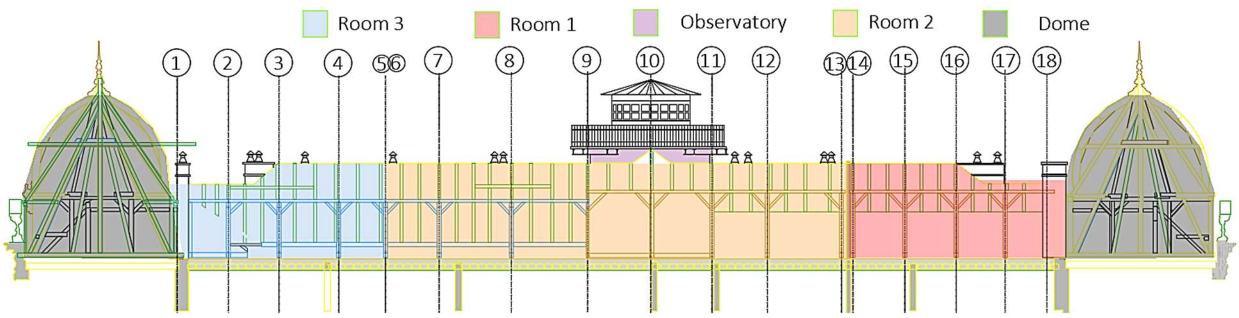


Figure 2.6. Longitudinal view B-B roof structure "TU GRAZ Alte Technik"

From the entrance to the roof structure at the east wing up to the observatory, attic room is divided in a transverse direction into two segments by hallway. This hallway provides communication between the observatory and the rest of the building. Due to the hallway, characteristic (symmetric) purlin-tie roof with the double standing chair is modified to accommodate the hallway. To support foot purlins and rafters on the northern side, in space between the hallway and northern eave a purlin-tie roof with the single chair is stationed. As the structure can be regarded as one system (neglecting discontinued tie beam) for the sake of simplicity in this work the structural system on these axes will be referred as the purlin-tie roof with the triple standing chair, see Figure 2.8. Furthermore, it is believed that in original design, the principal trusses at axes 14 and 13 were designed same as axes 15 to 17 and from 11 to 12, but to be later introduced with additional braces which lead to development of the king truss in addition with purlin tie roof with single standing chair (on axis A1-A), see Figure 2.9. This theory can be supported by a different type of connection used in the connection of the braces to the post and the tie beam found on the king post truss Figure 2.7.

Figure 2.7. Connection brace - post in axis 13 and 14

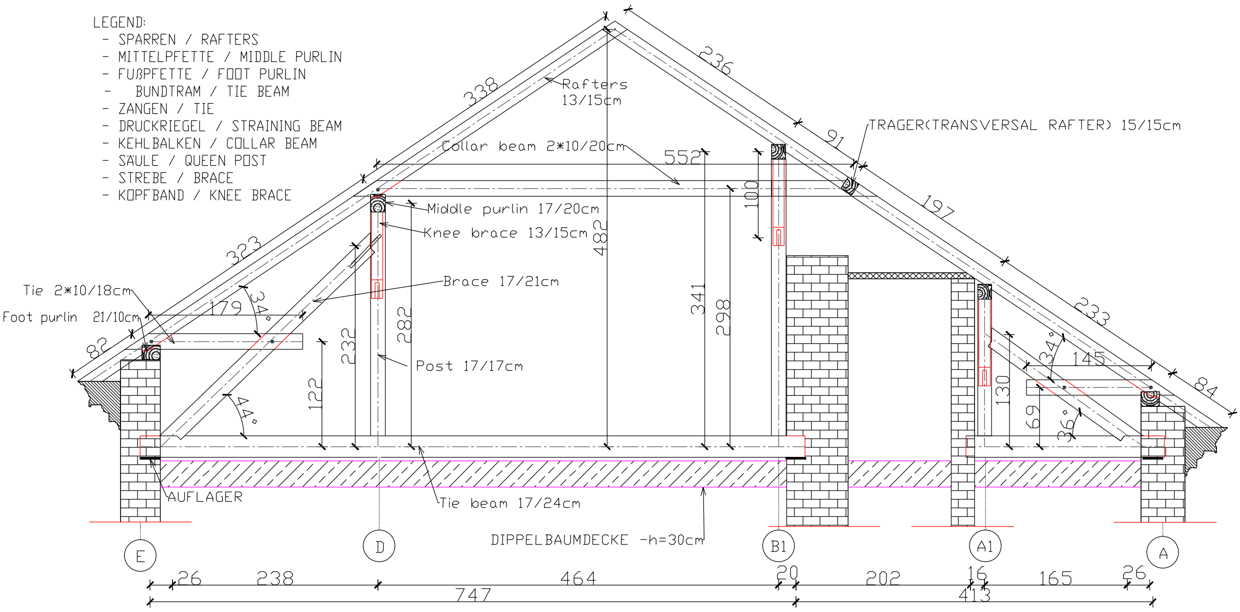


Figure 2.8. E-D-B1: Cross section of the purlin-tie with double standing chairs; / A1-A: purlin-tie with single standing chair (principal truss)

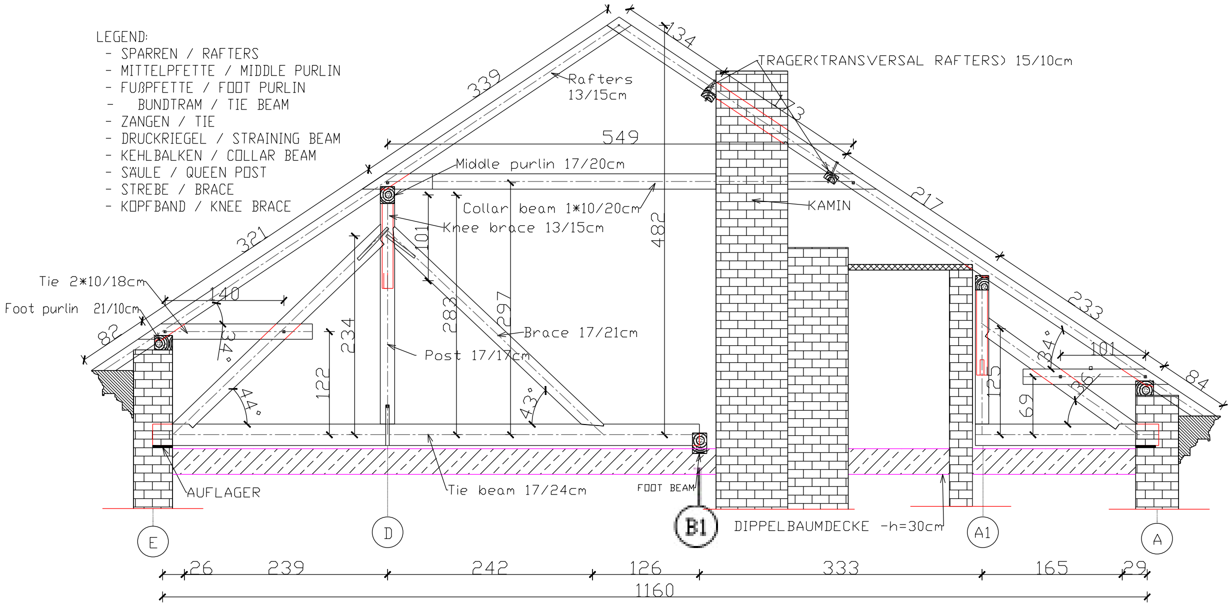


Figure 2.9. E-D-B1: Cross section of the King post truss (Germanic), / A1-A: purlin-tie with single standing chair (principal truss)

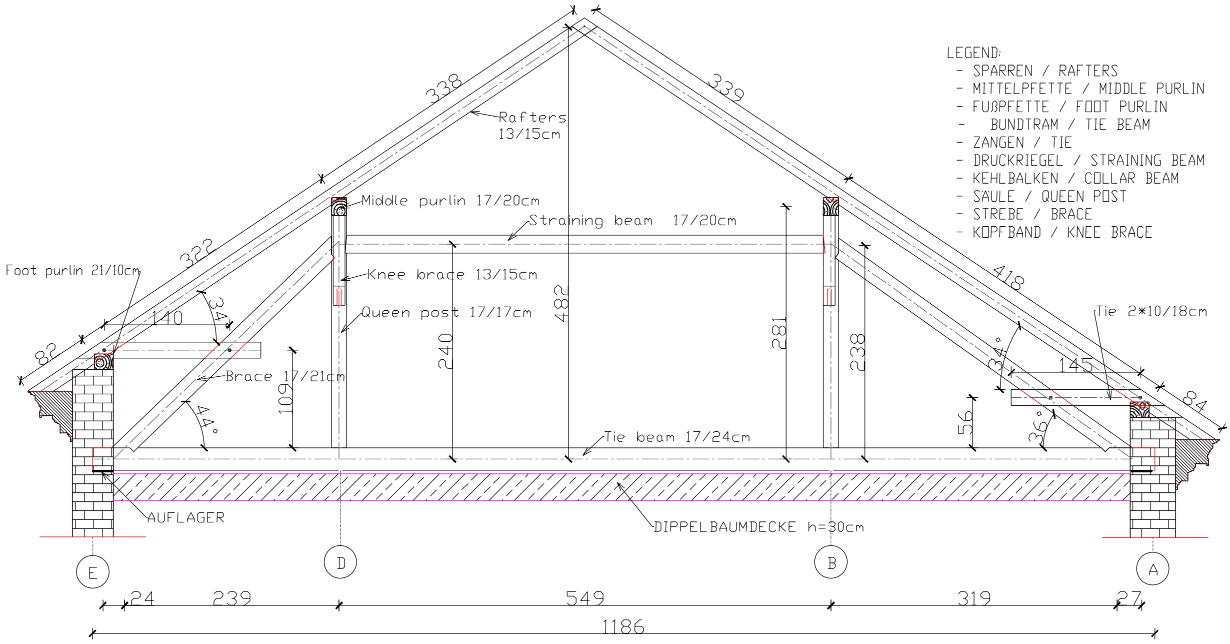


Figure 2.10. Cross section of the Queen post truss (principal truss)

Axes 2 and 17 are abutting areas of the roof structure. These two axes are accepting load from the valley and hip rafters which follow the lines of hips and valleys. These rafters have a larger cross section than other rafters as they accept bigger loads over a larger span. The rafters are supported at foot purlin, middle purlin and they are connected with other common rafters at the point where line of ridge intersects the line of valley or hip. The abutting areas need to be assessed because of the spatial load transfer. The span between the valley and hip rafters are covered with the jack rafters. The abutting areas with valley and hip rafters together with the jack rafters can be seen on 3D view, Figure 2.11.

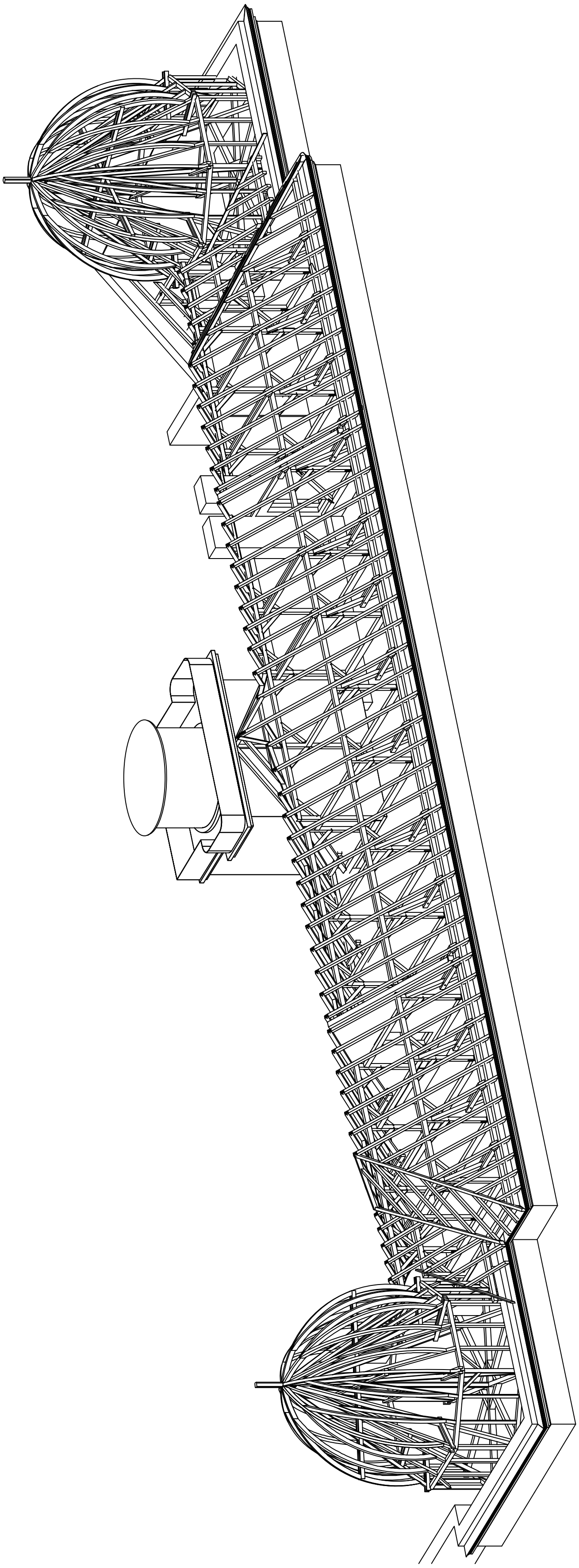


Figure 2.11. The 3D view of the roof structure and the domes at the south wing of “TU Graz Alte Technik”

In room 2 from axis 11 to axis 9 the dormer is stationed, see Figure 2.12. The northern side of the dormer is abutted by masonry wall of the observatory. This wall plays a crucial role in restricting movements of the dormer. This part of the structure because of its spatial load transfer, influences on the global behaviour of the roof structure in room 2 which will be shown and explained in Chapter 5.

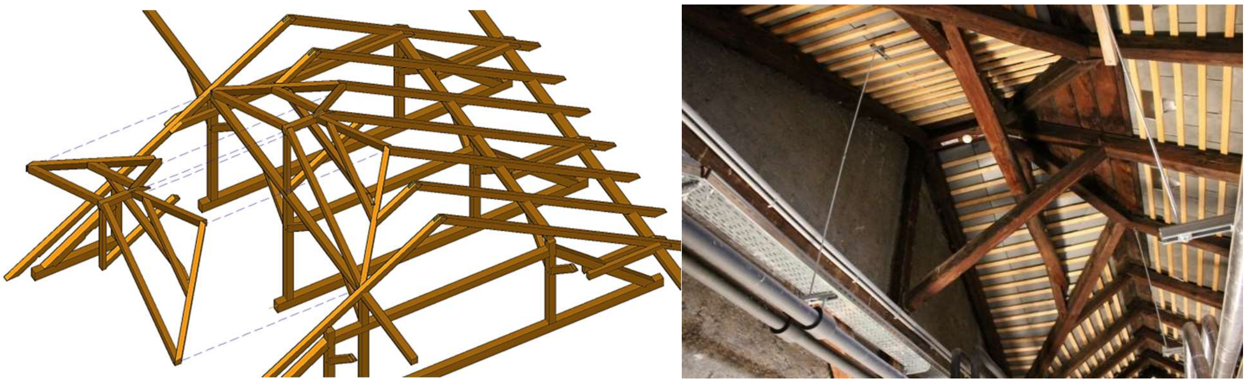


Figure 2.12. Dormer in room 2 up against the Observatory

2-1.1 DOMES

For names of the structural elements refer to Figure 2.14 right. The 2D plans are given in Appendix D, while the 3D model is shown in Figure 2.14 left.

The domes are designed as a simple eight-sided tent roof construction with a centrally located post. The post as a central element is connected to the tie beams. There are 8 tie beams altogether. Main tie beams consist of one continue tie beam which stretches under the central post, and two discontinued tie beams which are connected to the continue tie beam. Other 4 tie beams are connected to the main tie beams by secondary beams Figure 2.13 centre. The main tie beams are connected to the post with U- shaped iron strap.

Rafters are assembled from curved member and straight member to form bow shaped unit Figure 2.13 right. The main and secondary rafters can be distinguished. On Figure 2.13 right, the main rafter (in the middle) and secondary rafters (on margins) can be seen. The two secondary rafters are connecting in the upper part of the dome to the main rafter. Bottom supports of the main and secondary rafters rest on middle purlin which is supported in a plane by two intermediate posts in each field. Posts rest on foot purlin which is supported by tie beams. Horizontal trust on the support of main rafters to the middle purlin is accepted by ties which are connected to braces. On upper supports reactions from the main rafters are transferred to the central post. The load is transferred from the central post along the braces to the tie beams. Braces are connected to tie beams near the support of the tie beams, so the bending moment is neglected. (cf. [18])

When decoupled in the planar section along the tie beams the supporting structure is similar to king post truss as load transfer is similar.

Figure 2.13. left: View on the dome spike; centre: Tie beams, secondary tie beams and central post; right: Main rafter unit

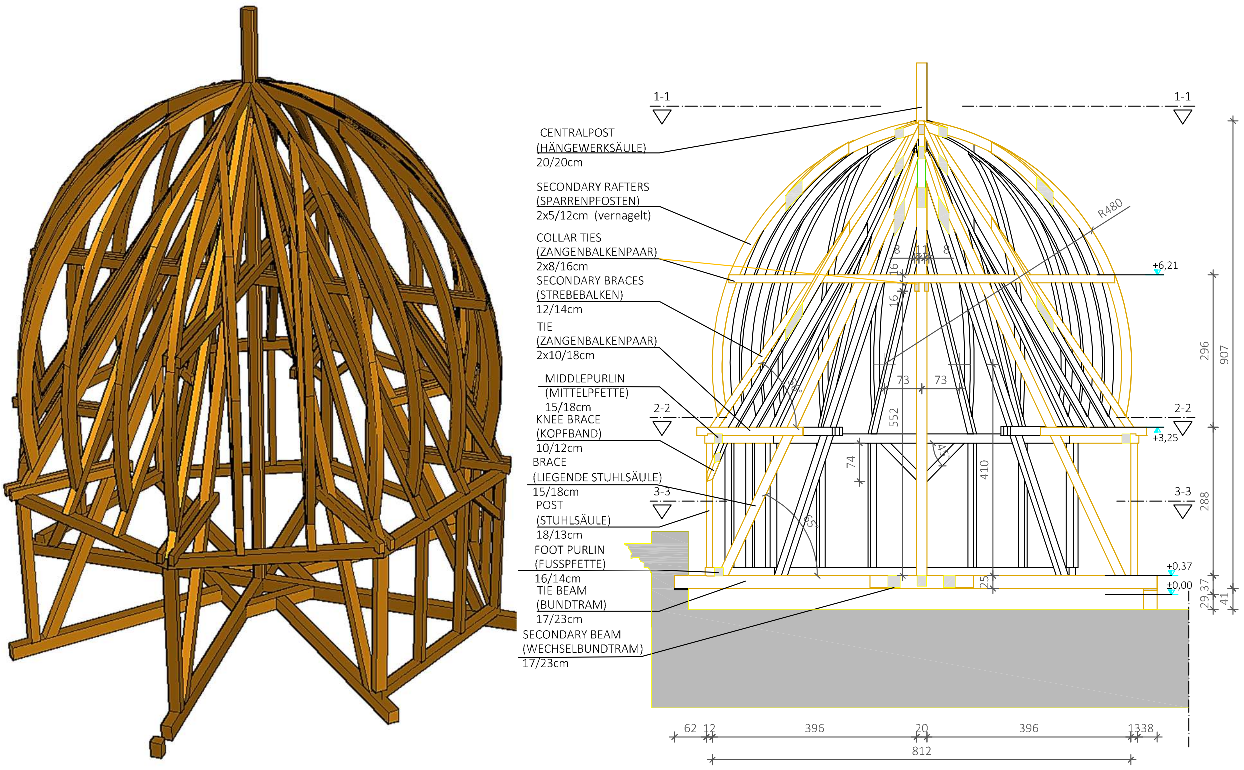


Figure 2.14. left: The 3D model of the structure of the domes; right: The 2D cross section, axis D-D

2-2 BUILDING PROCESS

Assembly of the Queen post truss will be shown and described in 5 steps. Assembly process of King post truss and purlin-tie roof will not be shown here, as building principles doesn't differ much from the Queen post truss.

1. **STEP:**
   The slab structure known as "Dipplebaumdecke" is used as a working platform for building up the roof structure. The erection process begins with laying of the tie beams in the prepared pockets (supports) in masonry. In the same time foot purlins are laid on top of the attic walls Figure 2.15.

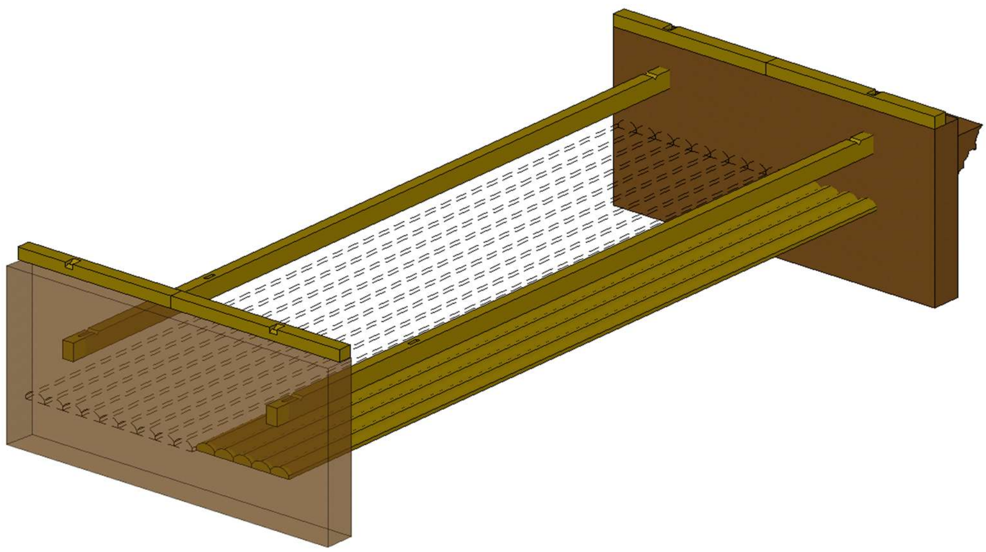


Figure 2.15. Building process, step 1

2. **STEP:**

Individual structural elements are lifted to the working platform. Posts, straining beam and braces are then connected on the working platform to form the unit. The unit is them lifted from horizontal position to vertical position by the help of ropes. During the rotation, the tie beam is providing the fulcrum for lift. Once in vertical position unit is fitted in tie beam on 4 places where connections are established (queen post to tie beam, brace to tie beam). Furthermore, iron U straps are introduced to the connections which during construction provides out of plane stability for the unit until knee braces are implemented. Figure 2.16

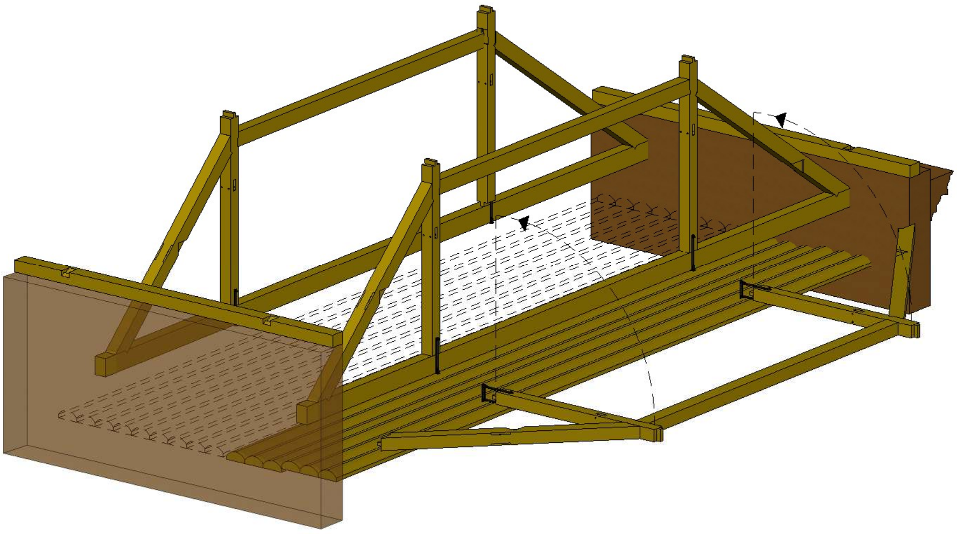


Figure 2.16 Building process, step 2

3. **STEP:**

In the third step, middle purlins are laid on top of the queen post and secured by wooden pegs for accepting sporadic negative vertical forces or torsional moments. After placing the middle purlin, the knee braces are connected. Connection of the knee brace to the middle purlin and queen post is also established by tenon and mortise connection secured by a wooden peg. Ties are then inserted between braces and foot purlin and secured on both ends by bolts. With the introduction of middle purlin and knee braces, the roof structure is now stable out of the plane and it can transfer forces in a plane and out of the plane. Figure 2.17.

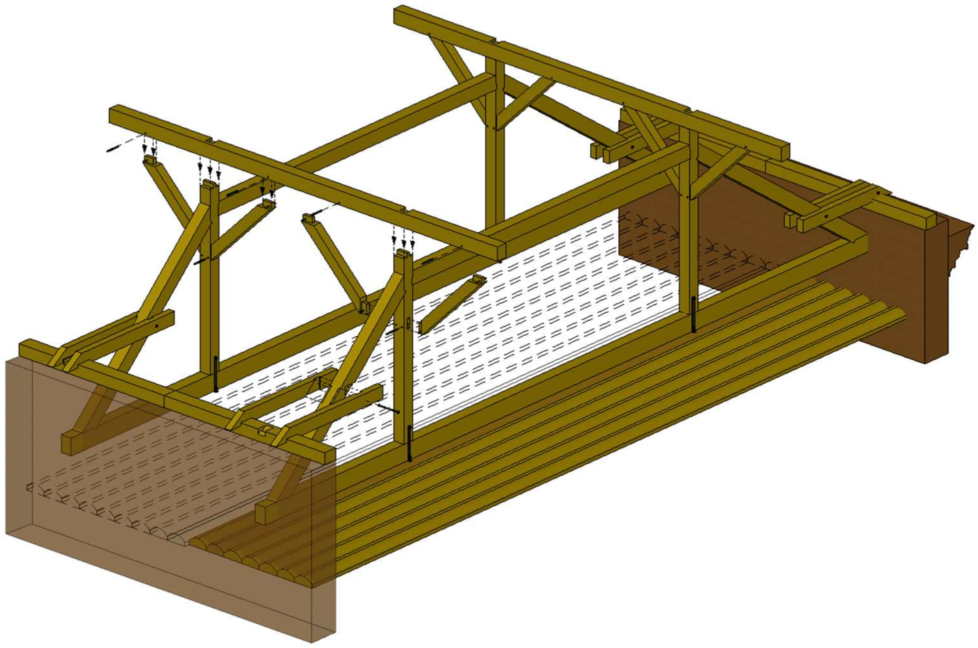


Figure 2.17. Building process, step 3

4. **STEP:**
   In the fourth step, the rafters are placed on previously shaped and prepared connections on the foot purlins and the middle purlins. Furthermore, on the top rafters are connected with a fork joint and secured with a wooden peg Figure 2.18.

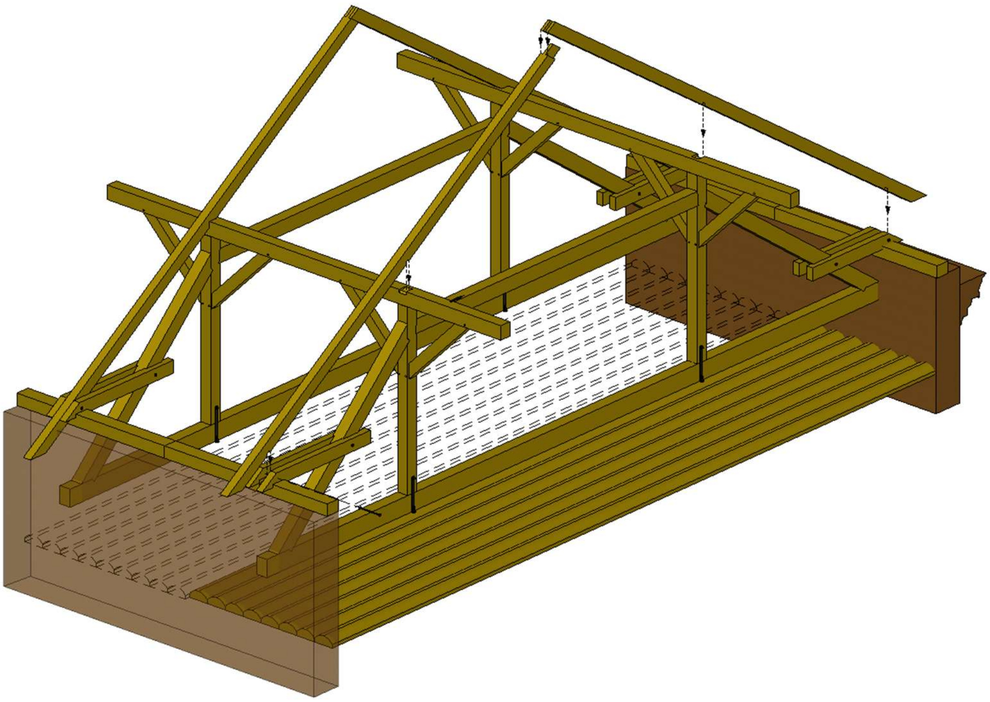


Figure 2.18. Building process, step 4

5. **STEP:**
   In the last step of the construction, wooden battens for carrying of roof covering are placed. Distance between battens is 16 cm. Figure 2.19.

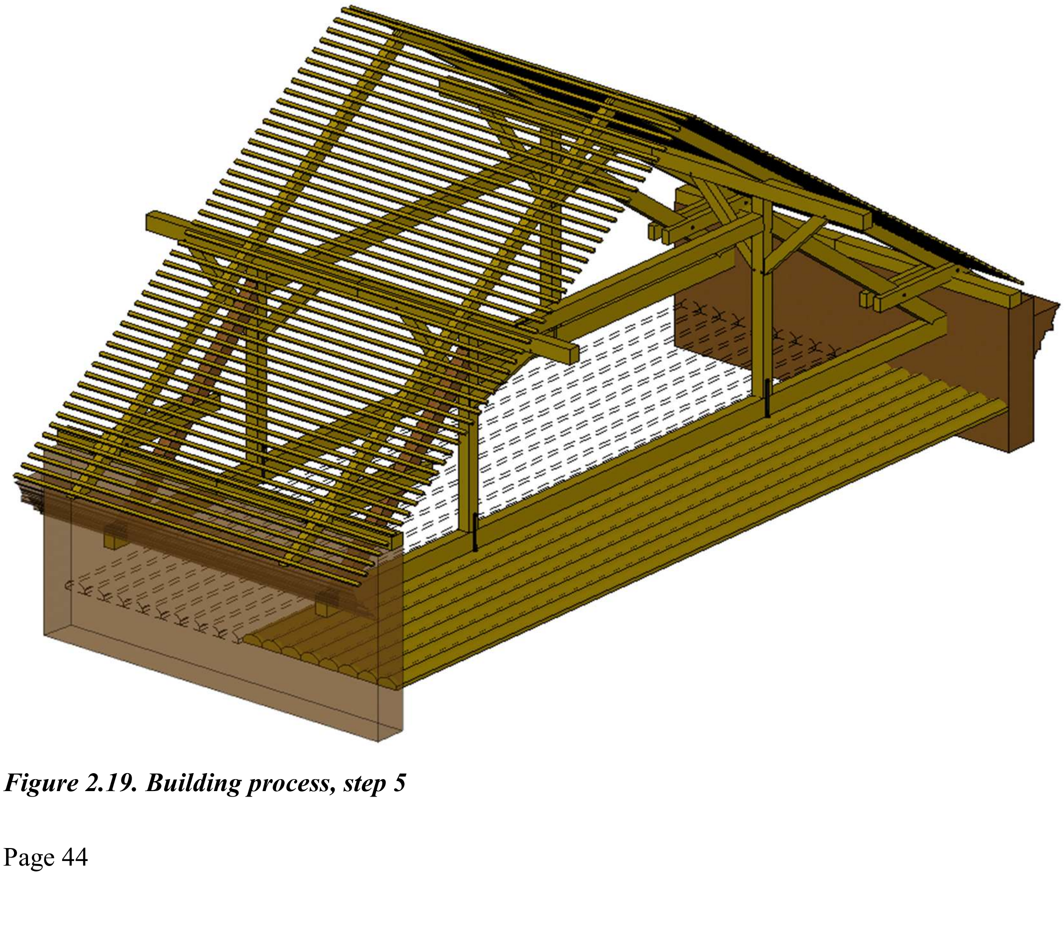


Figure 2.19. Building process, step 5

2-3 CONNECTIONS

This subchapter aims to describe and classify timber to timber connections which are found in the roof structure "*TU Graz Alte Technik"*. The modelling of the timber to timber connections and their respective impact on calculations will be derived in Chapter 5. The damages registered during inspections will be shown in Chapter 4. For all connections mentioned in this subchapter 3D and 2D drawings in scale with dimensions and named parts of each connection are given in appendix C.

First, the general classification of timber to timber connections according to Meisel [17] is given in Figure 2.20. This classification is in general also recognized in the other parts of Europe besides Germany and Austria, although alternations are possible because of differences in building tradition. (cf. [30] [17])

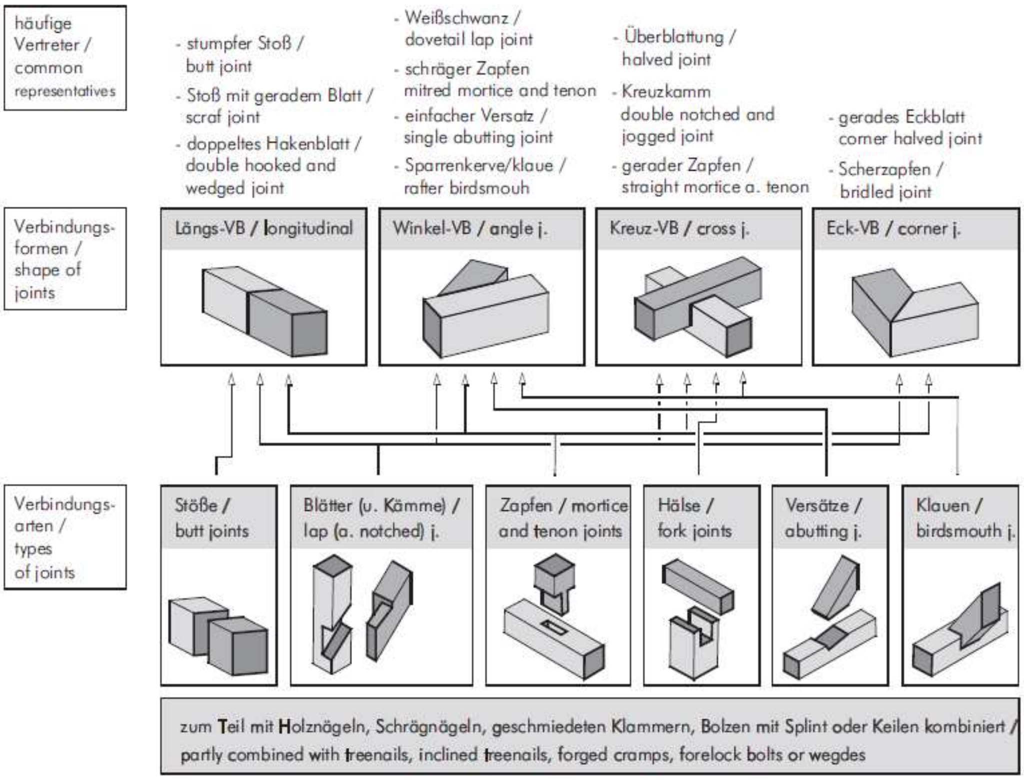


Figure 2.20. Classification of joints [17]

Historic fasteners

In many historic timber to timber connections, additional fasteners were used. Their function is to transfer tensile forces by shear, (e.g. wooden pegs) or by axially loaded iron straps and cramps. Many types of fasteners were developed. In Figure 2.21 a good example of a variety of fasteners is shown.

Wooden pegs are most used fasteners in the historic roof structures made from softwood or hardwood. Cross section can be a circle or prismatic, see Figure 2.21 right. Often one end is sharp shaped for easier installation. Wood pegs are driven in the holes with the hammer, as a diameter of the hole is smaller than the peg to account for the dimensional change. Iron fasteners are used for tensile connections. Before the introduction of iron fasteners, the tensile connections in timber structures have been avoided as much as possible, as traditional timber to timber connections have small load bearing capacity and big deformability under tensile loads furthermore pronounced with dimensional changes problems. The most used type of timber to timber connection for tensile loads before the introduction of iron were lapped connections. Nevertheless, the introduction of the iron fasteners in Middle Europe wasn't popular until 18th century, when global production started, in comparison to South Europe where iron fasteners were used since 15th century. (cf. [19] [58])

In the roof structure of “*TU Graz Alte Technik”* wooden pegs are made from European larch. Iron U straps 50 cm long 3 cm wide and 0,7cm thick are used for connection of a tie beam to a queen post. Iron cramps Figure 2.21 right are used for connection of a brace to a tie beam. In addition, nails are used for connection of the jack rafters to the hip and valley rafters.

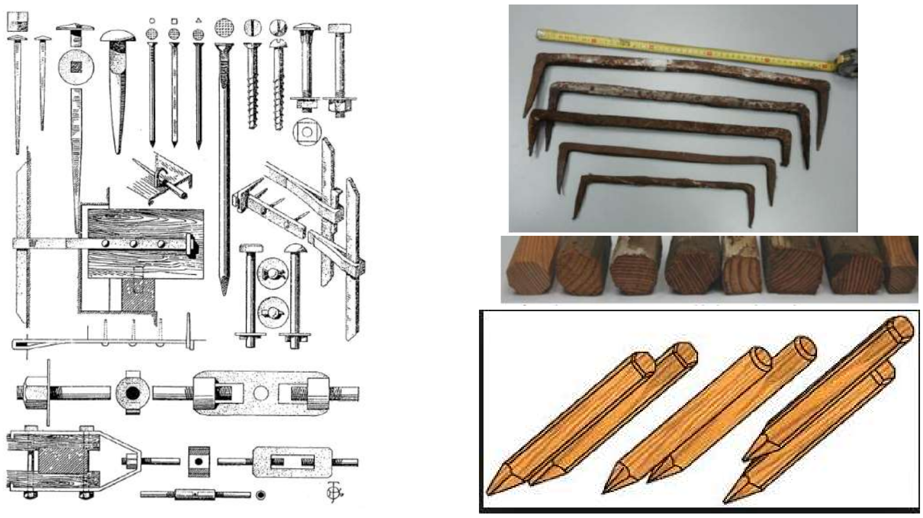


Figure 2.21. left: Iron fasteners used in historic timber structures [19]; right: Iron cramps and wooden pegs [19]

Brace to tie beam

Braces are connected to the tie beam by a notched connection in German literature also referred to as “Fersenversatz” Figure 2.22.

Notched joints have many alternations depending on the geometry of the notch and number of notches. Notched connections are usually combined with mortise and tenon joints (eg. for steep inclinations). If introduced, tenon in the middle of the contact plane provides out of the plane stability.

The notched connection in this thesis is classified as a single notch joint with tenon and mortise. The notched connection transfer compression forces trough the contact planes of the notch marked red in Figure 2.22 left. Tenon and mortise are excluded from this load transfer because contact is not established due to imperfections and dimensional changes. Upper contact marked with blue colour is not in contact with the tie beam, therefore loads are not transferred, see Figure 2.22. This gap is essential for avoiding longitudinal cracks in the brace, in the literature 1,0-2,0 mm gap is recommended (cf. [12]) Small tension forces can be transferred through friction, but this should be neglected during the calculation. Therefore, the transfer of tension force is only possible with the introduction of iron cramp. The notched connection behaves as semi rigid and depending on the geometry, bending moments can be transferred. The transfer of the moments is largely improved by using tenon and mortise, especially for moments in a direction out of the plane and torsional moments.

Figure 2.22 left: Rear /heeled notch with marked load transfer planes; right: The gap in notched connection

Straining beam to queen post

Straining beam is connected to the queen post with tenon and mortise Figure 2.23. Tenon in this joint is only introduced for constructive reasons to provide stability in case of loads perpendicular to the straining beam axis. The queen post is cut in the shape of a triangle. The cut is 2cm deep. The straining beam is cut in the same manner. This cut provides support for straining beam and transfer of vertical loads. Horizontal loads are transferred by contact plane on face of the straining beam. This connection can only transfer compression forces. Transfer of tension forces is only possible with iron cramps. In Figure 2.23 connection with iron cramp is shown, however in most connections, iron cramps are not present.

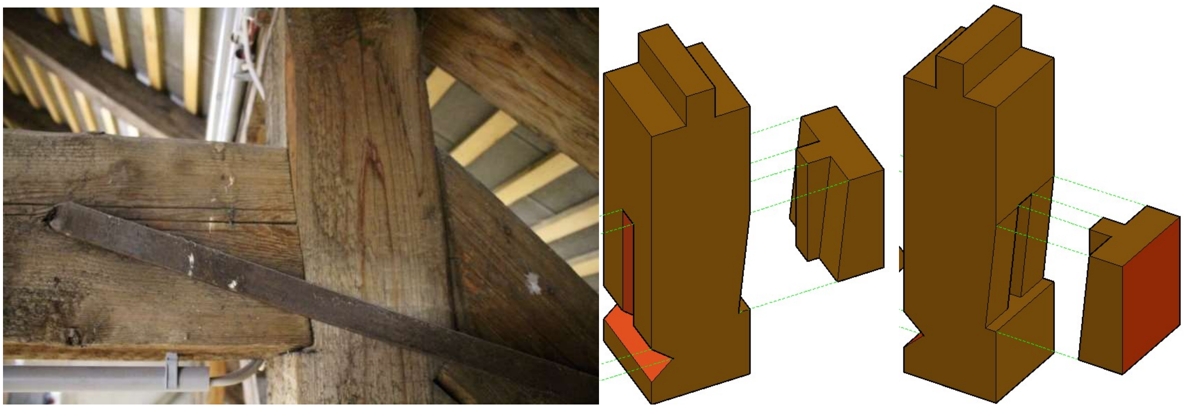


Figure 2.23. Connection straining beam to queen post

Connection of middle purlins on axes 2 and 18

The connection showed in Figure 2.24 is referred to as a half lapped connection. Besides the majority of the traditional connections, this connection is still popular today. A critical part of this joint presents the notch at the support which vastly reduces cross section area. Moreover, in the notched corner, stress concentration and tension perpendicular to the grain is developed. As a consequence, if the connection is badly designed cracks from the notched corner along the fibres are possible as shown in Figure 2.24 left. In Eurocode 1995-1-1 [59] Section 6.5.2 expression for verification for this kind of connection is given. In this connection, the beam can be notched either at the upper edge, see Figure 2.24 left, or at the bottom edge, see Figure 2.24 right. Critical is cut on the bottom edge of the beam as the beam is subjected to tension perpendicular to the grain. Small teeth are shaped on the end of the notch on one beam for transfer of horizontal loads. In middle purlins horizontal forces can be neglected though, therefore teeth part is only for constructive reasons.

Figure 2.24. left: Half lapped connection with a notch on the upper edge; right: Half lapped connection with a notch on the bottom edge

Brace to queen post

Connection of the braces to the queen post is the same as a connection of the braces to the tie beams.

Figure 2.25. Connection Brace to Queen post

Connection of knee braces to middle purlin and queen post

Knee braces are connected to the middle purlin and queen post with a tenon and mortise, in German literature also referred as "Zapfen" Figure 2.26. Wooden pegs are an indispensable part of this connection. In this case, wooden pegs 2cm in diameter made from European larch are used Figure 2.27. Their function is to receive the tension forces which may occur in the knee braces. For compressive forces, resultant force in the knee braces is divided into a horizontal and vertical component. These forces are then transferred by two contact planes. For vertical loads via tenon and for horizontal via contact plane between the queen post and knee braces.

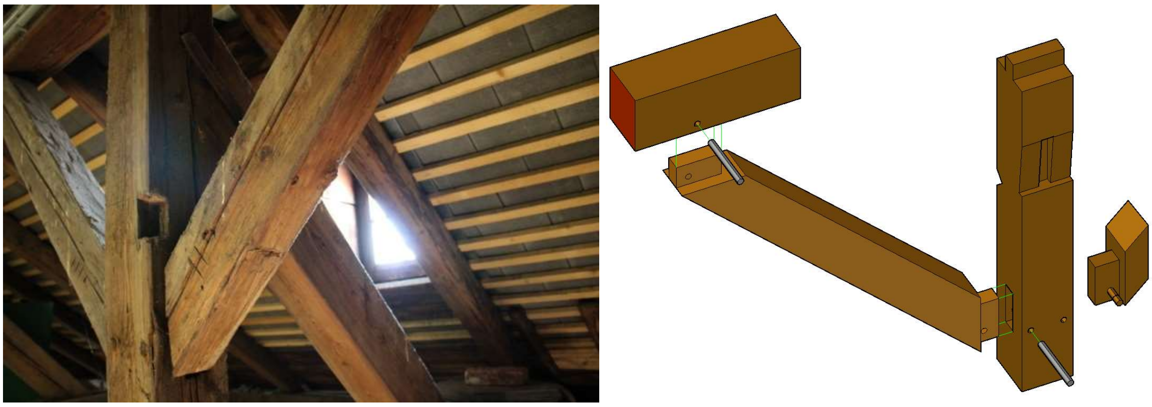


Figure 2.26. Knee braces

Figure 2.27. Detailed view on tenon and wooden peg

Connection of the short tie and the rafter to the foot purlin

In this joint, the short tie and the rafter are connected to the foot purlin. Connection of the short ties to the foot purlin is classified as a cross lap joint. Foot purlin supports a short tie and the rafter. Short tie receives a horizontal component of the resultant force in the rafter, preventing overturning of the attic wall. This horizontal component is transferred via contact plane which is marked with red lines in Figure 2.28. Connection of the rafters to the foot purlin is categorised as Birdsmouth connection. Besides to the foot purlin, rafters are also connected with the short ties with a steel bolt, 15mm in diameter Figure 2.28. The same Birdsmouth connection can be found in connection of the rafters to the middle purlin Figure 2.29.

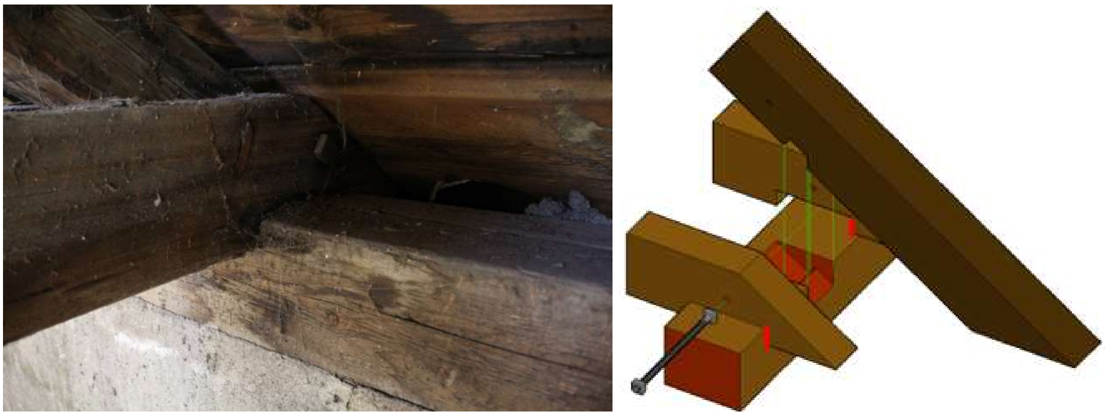


Figure 2.28. Connection of the rafter and the short ties to the foot purlin

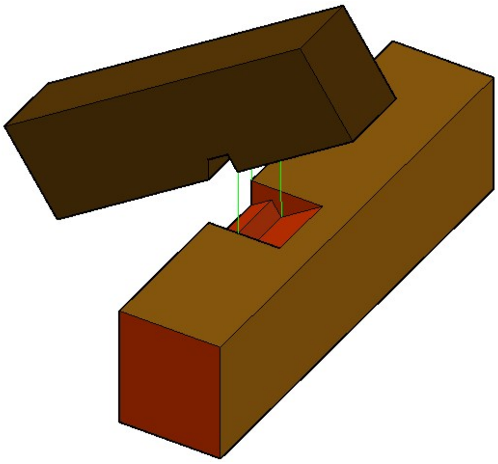


Figure 2.29. Connection of the rafter to the middle purlin

Connection of middle purlin to the queen post

Middle purlin is connected to the queen post by tenon and mortise Figure 2.30. The supporting reactions from middle purlin to the queen post is transferred by contact surface. Area of the tenon is excluded from the contact surface. Tenon is used as a constructive part of the connection for transferring horizontal load in and out of the plane, and torsional moments from the middle purlin. To provide transfer of the horizontal forces and torsional moments from middle purlin to the tenon, a wooden peg, 2,0 cm in diameter is used.

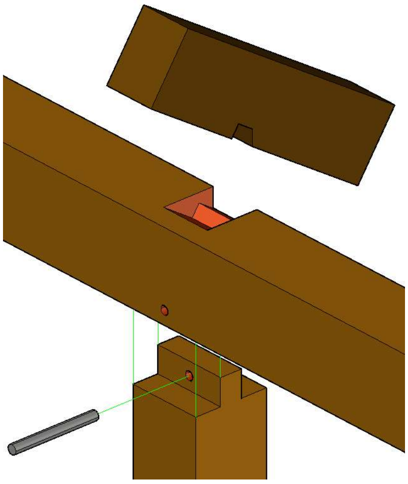


Figure 2.30. Connection of the middle purlin to the queen post

Connection of rafters

Rafters are connected by traditional fork joint Figure 2.31. This is the most used type of connection for connecting rafters. The connection is secured with the wooden peg, 2 cm in diameter, for loads which tend to separate rafters.

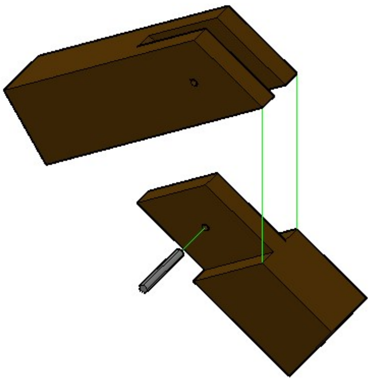


Figure 2.31. Connection of the Rafters

Connection of the short tie to the brace

The short ties are connected to the brace with a dovetail halving joint. Tension forces from the short ties to the brace are transmitted via the vertical contact surface. Moreover, the short ties are secured to the brace and mutually connected with a steel bolt 15mm in diameter. This bolt also transfers sporadic compression forces that may occur in the short ties. The connection is shown in Figure 2.32.

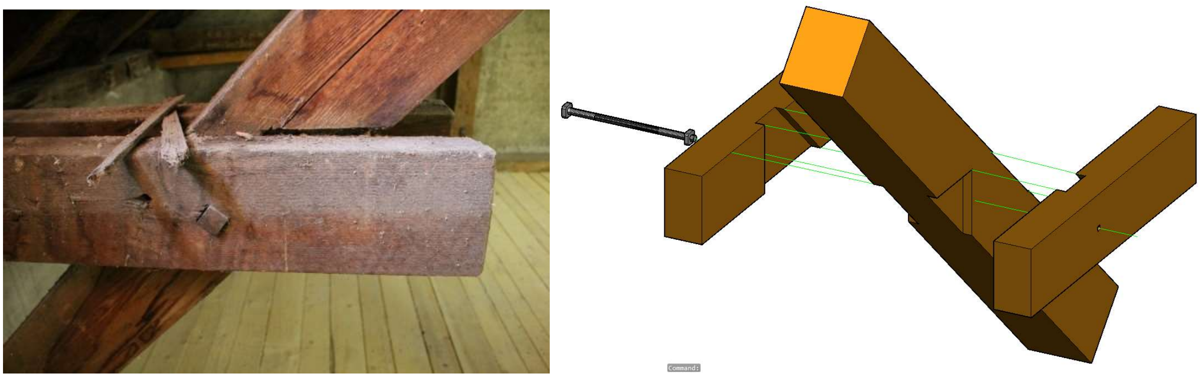


Figure 2.32. Connection of the short tie to the brace

Queen post to the tie beam

As wooden flooring in the attic hid the lower half of the tie beam, the complete view of this connection couldn't be established. Nevertheless, as roof supporting structure in west and east wing of the building doesn't differ from the south wing, information on this connection from the master thesis of Roman Popatnig are used [20].

Queen post is connected to the tie beam with a tenon and mortise and with an Iron U- strap Figure 2.33. Tenon and mortise transfer horizontal forces in the plane and out of the plane. According to the literature, mortise must be deeper than the tenon at least 0,5 cm so the vertical forces from the column are not transferred through tenon to the tie beam [14]. This is also confirmed by the resistance test which accidentally passed through this gap. Compression forces are transmitted on the contact plane of the queen post and the tie beam.

Tension forces are transmitted with the Iron U strap, which is made from 2 parts. The upper part consists of two iron rods, which are at the top folded into the timber and secured with 16mm diameter bolt. The lower part consists of the iron rod, which is situated under the tie beam. The rod is connected to the upper rods with two bolts. Exploded view of iron U strap is provided in Figure 2.33 right.

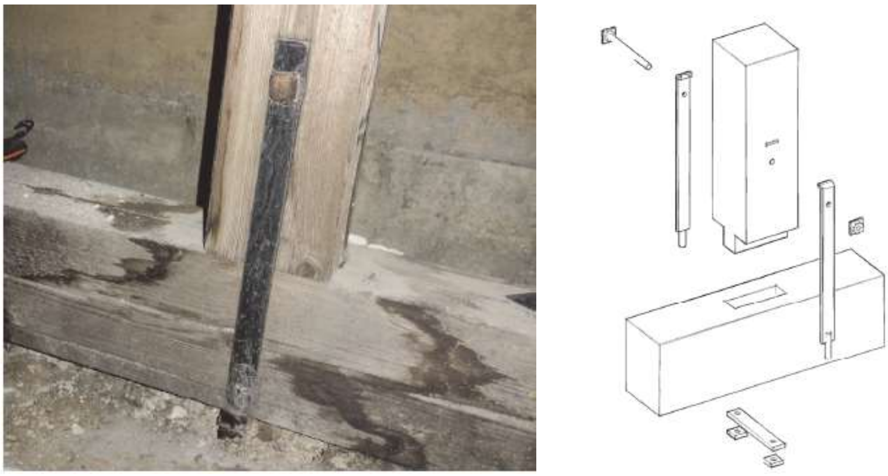


Figure 2.33. left: Connection queen post-tie beam [20]; right: Exploded view of the connection queen post-tie beam [20]

Transversal beams

The roof structure is crowded with chimneys in all 3 rooms. Today the chimneys are no longer in use, so they are removed and sealed. Nevertheless, transversal beams are still structural part of the roof. These transversal beams carry the load of rafters which are interrupted because of chimneys and transfer the load to the continue rafters. Some transverse beams are connected to the continue rafters with bolts. In that case, transverse rafters are lying beneath continuous rafters Figure 2.34 left. Other transverse beams are connected by tenon and mortise or a nail. In that case, they are levelled with continue rafters Figure 2.34 right.

Figure 2.34. left: Transverse beam connected with bolts; right: Transverse beams connected with the tenon and mortise

Support of the Tie beams to the masonry

During the assessment of the roof structure, special attention was focused on the supports of the principal trusses on the masonry. The supporting length is measured on several supports on the south and north side, as well with geometry of the wall pockets. The supporting length of all measured supports is around 26 cm. The wall pockets are 18cm wide and 28-30 cm deep. As a width of the tie beam is 17 cm, it can be concluded that enough space for ventilation is provided. Some supports are completely visible Figure 2.35 left. However, the majority of supports are invisible in the wall pockets Figure 2.35 right.

Figure 2.35. left: Visible support of the tie beam to the masonry; right: Support of the tie beam to the masonry hidden in the wall pocket

CHAPTER 3: APPROACH TO ASSESSMENT OF THE HISTORIC ROOF STRUCTURES

In this chapter first, short introduction and comparison between different approaches to the assessment of the historic timber structures is presented. Special attention is given to on site methodology of evaluation, including surveying and diagnosis. At the end overview of tools which should or could be used is presented.

3-1 APPROACH TO THE ASSESSMENT

Historic and heritage structures don't need to apply to the today valid codes and laws. This is valid until a change in function is planned or when damage which directly threatens the load bearing capacity of the structure is noticed. During years of exploitation load bearing capacity of the timber roof structures inevitably decrease. On Figure 3.1 general diagram resistance-action is shown according to Meisel [17]. Period III and IV marks the periods where damage in the roof threatens structure stability. Therefore, the assessment of the existing structure should be conducted, and subsequently structural verification according to the valid construction codes and laws. The need for standardization in this area is recognized and defined through the mandate of the Working Group of the CEN Technical Committee 250 (Structural Eurocodes), CEN/TC 250/WG2: Assessment and Retrofitting of Existing Structures. [33]

The assessment at the end needs to answer the question if the strengthening measures are needed. To preserve cultural heritage, those strengthening measures should be kept to the minimum. This emphasises the need for a detailed assessment of the structure to provide the realistic condition of the structure and subsequently produce suitable preservation procedure without damaging the value of heritage structure. This kind of approach corresponds to the internationally accepted principles such as ICOMOS. (cf. [36] [17] [18] [40])

Historical structures, especially those representing the architectural heritage require a specific approach. ICOMOS documents (principles and guidelines) [35] [38] [37]are valid for built heritage in general. These documents, as well as recent documents which are specific for historic timber structures [36] point out that „the condition of the structure and its parts/elements should be assessed and recorded before considering any action". In a broader sense, it refers on the following: an "anamnesis" represents the first step prior to a "diagnosis"; an accurate "diagnosis" (based on inspection, survey, research, analysis and evaluation) must precede any "therapy" – intervention, and control measures should be planned and continuously implemented. More detailed explanations can be found in [35] [38] [37] [40]

Assessment of the historic structures is a multidisciplinary task where a structural engineer is working in collaboration with numerous other specialists such as Conservators, Architects, Contractors, Owner of the building, Authorities, etc. A mix of different fields, which sometimes can differ in view and objectives is making the process of assessment and preservation of the building more complicated. (cf. [18] [16])

There is no unified European code for assessment of the historic structures. Only international standard ISO 13822 is developed on the base of internationally developed principles for preservation. Moreover, some countries have developed national codes for assessment of historical structures such as: Switzerland (SIA-269-2011), and Italy (UNI). However, most countries don't have developed national codes, therefore this represents risk to the preservation of historic structures. (cf. [40])

In this thesis, three approaches are presented.

- The approach according to ISO 13822 [39]
- The approach developed by Cruz & others [40]
- Approach to on-site evaluation developed by Meisel. [17] [16]

Approach to Meisel is used in the process of assessment and surveying of roof structure at TU Graz Alte Technik. Therefore, more attention is given on this approach.

Before revising these approaches, it should be considered that direct comparison between ISO 13822 and other two approaches is not possible as approach according to ISO 13822 is made for existing structures in general, while other two are made for assessment of timber structures.

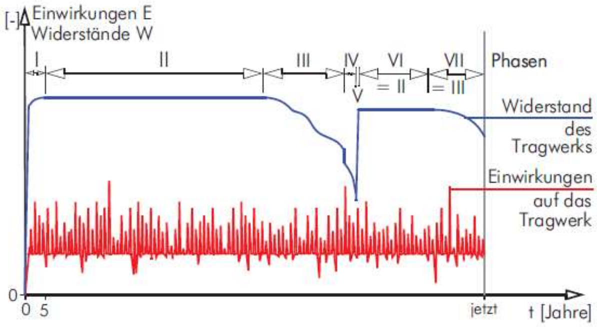


Figure 3.1. Diagram resistance-action with respect to time [17]

3-1.1 APPROACH ACCORDING TO ISO 13822

ISO norm (ISO 13822:2001) is developed for any type of existing structures. Therefore, details which would take into consideration the behaviour of timber and its characteristics are not provided. This norm provides only guidelines for a methodology of the approach to the assessment of existing structures. For specific characteristic guidelines for preservation of the timber structures ICOMOS should be revised. Nevertheless ISO 13822 provides a good approach to the assessment of historic structure, for following prescribed procedures defined in a flow chart during the assessment period, see Figure 3.2. Moreover, in Annex I of the ISO 13822 norm the approach to assessment of heritage structures is described. [39]

Before the beginning of the assessment, objectives of the assessment should be specified in consultation with the owner, architect, structural engineer and other parties. On top of that some scenarios should be established. Scenarios present specific structural conditions or actions which can occur during the lifespan of the building. Earthquake scenario is here especially highlighted. The identification of different scenarios presents the basis for the assessment and possible strengthening. The result of the assessment process and possible future conflicts will depend upon the successful development of these objectives at the beginning of the project. To sum up, the procedure of the assessment depend upon the assessment objectives. (cf. [39] [18] [60] [61])

Afterwards, a preliminary assessment should be taken. A preliminary assessment is concentrated on inspection, recording and documentation of the structure. This include site visit. The goal is to determine if immediate actions are needed. Moreover, the result should suggest if more detailed assessment is needed and if yes in which scope. [39]

In detailed inspection, a detailed overview of the structure should be undertaken. The material properties should be determined with help of the existing documentation if it is reliable. In contrary, material testing should be conducted to determine material properties by non-destructive tests. The destructive test should not be considered for historic structures. When referring to timber structures this task becomes complicated as no reliable non-destructive test can be obtained to determine material properties. [39]

Structural analysis can be established according to the ISO 2694 or according to the valid Eurocodes, depending on applicable law. Usually, actions and verification process need to be conducted according to the valid Eurocodes. The deterioration should be taken into consideration. The uncertainty of model accuracy should be taken into consideration with partial factors or uncertainty variables. (cf. [39])

The standard also gives in Annex H the format of the assessment report which gives the list of information which final report should present.

When assessing the heritage/historic structure Annex I should be considered together with other provisions given in the standard. Structural behaviour of the building and its cultural value should be analysed by the specialists from several fields and on that basis any decision regarding the structural strengthening should be carried out in a group. To keep the heritage value of the structure, structural interventions should be limited and avoided if possible, especially on the character defining elements. According to the Annex I too conservative structural modelling, verification, and overall assessment should be avoided in way to preserve heritage. (cf. [39])

For a more detailed explanation of the procedure refer to the standard itself [39], and to the following theses [60] [61]

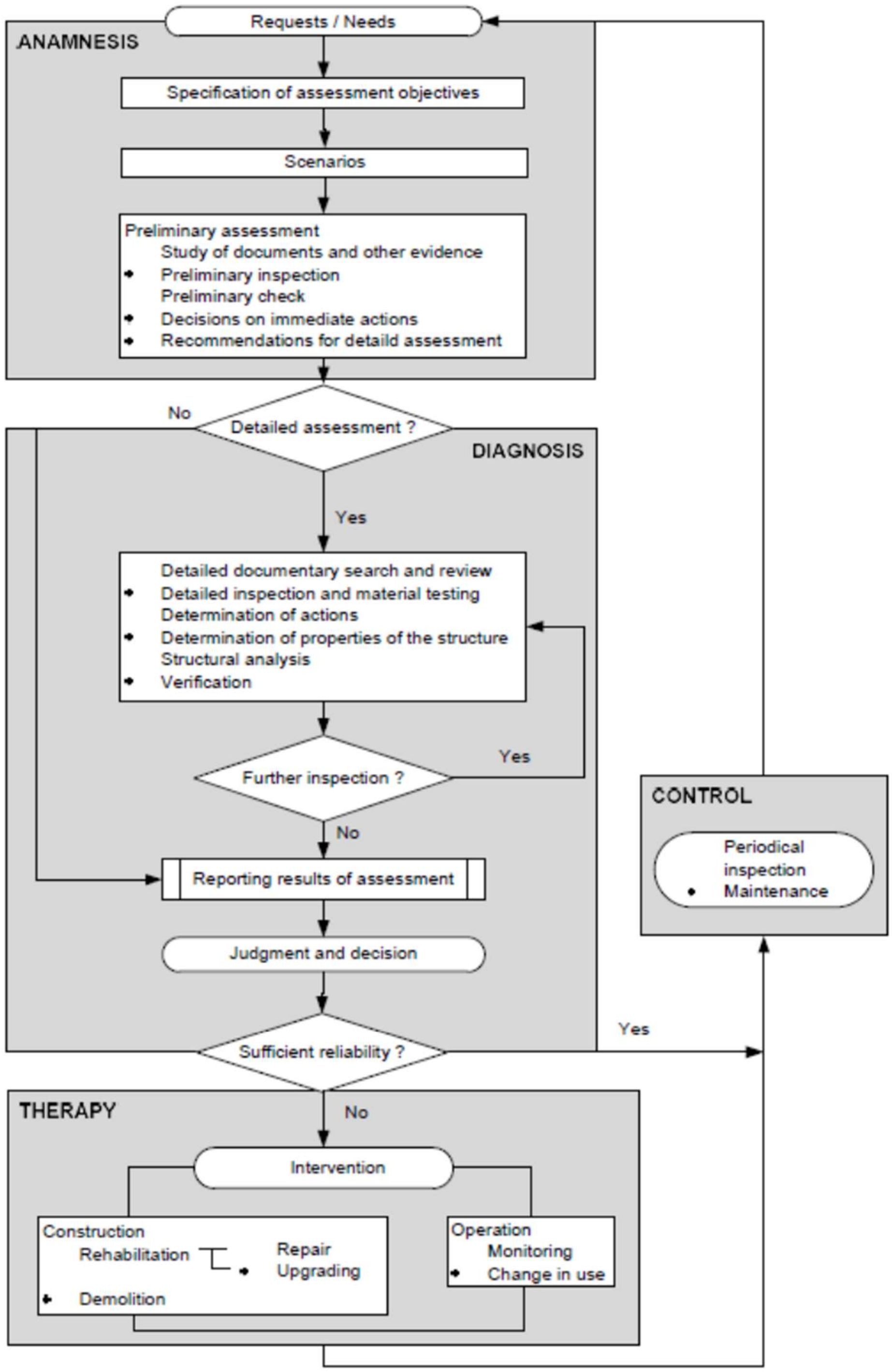


Figure 3.2. Flow chart for assessment of existing structure according to ISO 13822-2010 [39]

3-1.2 APPROACH ACCORDING TO CRUZ & OTHERS

It should be noted that prEN 1712:2019 (CEN), which results from the work of CEN/TC 346 (Conservation of Cultural Heritage) is based on the paper of Cruz. [40]

Paper [40] targets at all those concerned with the preservation of heritage buildings and covers the principles and possible approaches for the safety assessment of the old timber structures of the historical relevance. In Figure 3.3 the decision and process flow chart are shown. (cf. [40])

According to this approach, a distinction needs to be made between heritage/historic structures and existing structures. This distinction needs to be made in order to justify the greater expense for surveying of the historic structures. The detailed and best possible assessment should be conducted. Subsequently, this reduces the risk of the need for extent strengthening. Strengthening should be kept to a minimum. In case of need for strengthening the less invasive methods must be used even though this could result in greater expenses. (cf. [40])

Proposals for remedial measures need to take into account the owner's intentions as indicated in item 1 but also to point out any areas of conflict between

a) the need to preserve its heritage value

b) the owner's intentions and

c) the need for public safety.

A holistic approach is always required, considering and assessing the structure as a whole, rather than just the individual members and joints. [40]

The steps in flow process are explained in detail in [40] and therefore here will not be further explained. The process itself is an iterative process. This happens often as flaws are encountered during preliminary report or during the making of the documentation which results with additional surveying or investigation work. This also applies to the closed process of the modelling and the strengthening, where two are in a strong connection. (cf. [40])

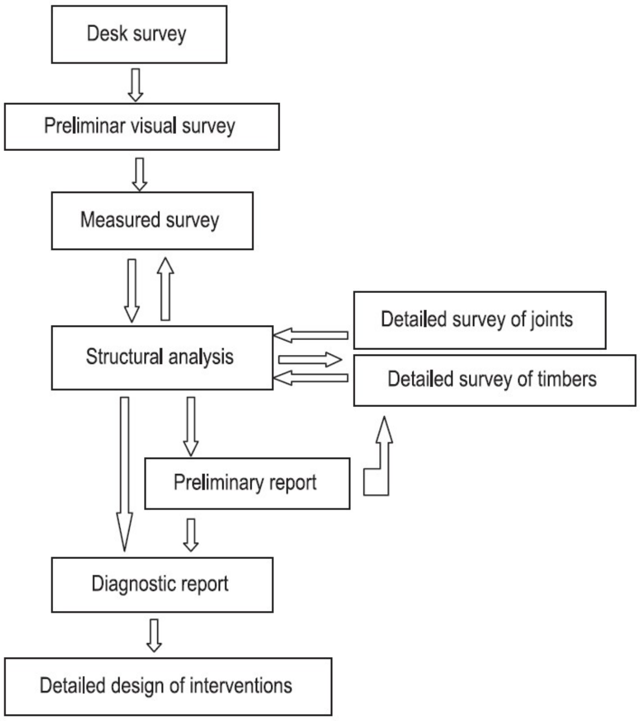


Figure 3.3. Flow chart for assessment of existing structure according to Cruz & others [40]

3-1.3 APPROACH ACCORDING TO MEISEL

The project flow of the assessment process for the historic structures according to Meisel is shown in Figure 3.4. On this project flow, typical tasks and operations regarding each step of the project are given. This can be compared with other project flows discussed in this thesis. In this chapter main attention is given to steps 2 and 3, while step 4 is presented in Chapter 5 and Chapter 6.

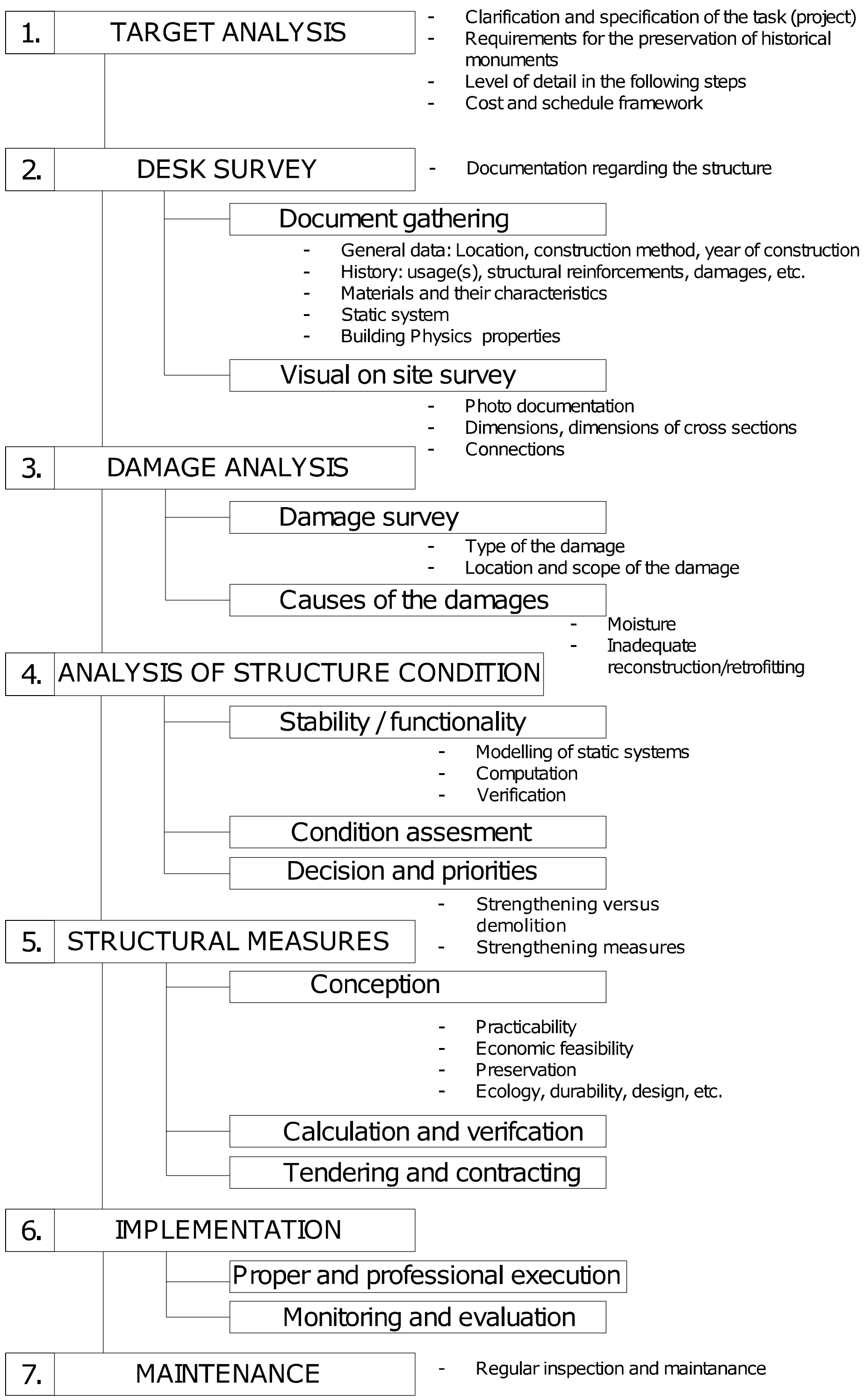


Figure 3.4. Flow chart for assessment of historic structures according to Meisel [21]

The step 2 and 3 in project flow are furthermore elaborated in flow chart Figure 3.5. The subsequent actions are depending upon first evaluation of the structure condition after the first visual inspection. This step would correspond to Preliminary visual inspection according to the other two approaches. The main objective is to determine if the structure is safe enough, and accessible for further investigations and survey. For cases of positive or negative answer to this question, necessary actions are given in flow chart.

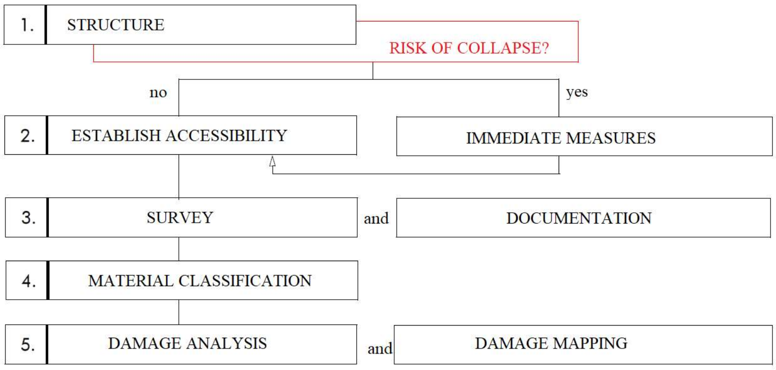
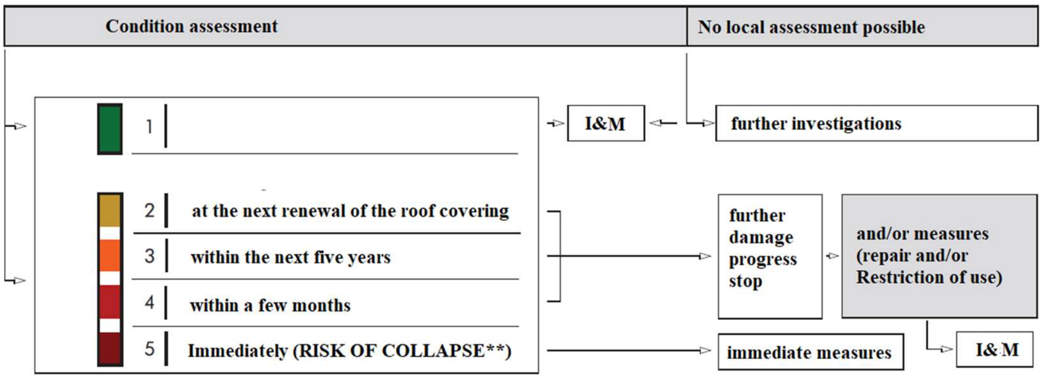


__Figure 3.5. Flow chart for surveying of structure according to Meisel [16]__

3-1.3.1 Grade system

Meisel developed a grading system where according to the 9 points the condition of the structure is evaluated. For each category according to the prescribed conditions points are awarded. This approach is highly sensitive upon the experience of the engineer. Nevertheless, a comparison has been made where several students and engineers conducted the assessment of the same structure and result differed by 0,5 points. This could be prescribed to the clear explanations of the representative conditions. Depending on the sum of points the structure condition is divided into 5 categories. Maximums sum of points that can be given is 5 points. When structure is not negatively affected, 0 points should be assigned. An engineer can also give any value of point inside of interval from zero to the maximal allowable point for the particular category. Depending on the result, further approaches are proposed Figure 3.6. Moreover, estimated time in which actions or repairs should be conducted is given. It is important to emphasise that this grading system applies to the whole structure in general, rather than on single element.



*** ... I & M: regular inspection and maintenance**
**** ... Definition: A building structure can be defined with adequate Probability of endangerment of people in the foreseeable future.**
***** ... e.g. evacuation**

__Figure 3.6. Condition assessment according to Meisel [17]__

In Figure 3.7 is given the practical grade system with conditions and span of the point that can be prescribed for each point. This format of the assessment can be directly used during the assessment in the same format. A detailed description of conditions and some explanations regarding point allocation within it is given in [17].

Condition assessment of a historical timber roof structure.	
Consequence class	(maximal 1 point) 0,5... for ordinary buildings 1... for buildings with crowds of people
Structural safety	
Degree of static indetermination	0,5... for statically determined or almost statically determined structures
Conversions and repairs	0,5 ... for structures that have not been professionally modified in the last decades (about 50 years). 1 ... for load-bearing structures, which in the last decades have been subjected to changes, particularly statically important members and/or joints have not been professionally modified.
Biological degradation	0,5 ... for structures showing moderate damage without proper repair.These damages go beyond superficial defects. and also concern statically essential components. 1 ... for load-bearing structures that show severe damage without proper repair 1,5 ... for load-bearing structures, which can cause severe damage to statically particularly important members or connections
Connection and/or member failure	0.5 ... if individual, statically essential connections and/or members are formed as a result of overloading or timber destruction. 1 ... if several statically essential compounds and/or members formed as a result of overloading or destruction of the wood have failed and this has particular consequences on the overall load-bearing behavior.
Constructive shortages	0.5 ... when the dimensions of the statically essential components are insufficient. 1 ... if the supporting structure is obviously a faulty construction and/or statically absolutely necessary members or connections are missing
Large deformations and/or gaps	0.5 ... if statically essential members and/or the entire supporting structure are clearly deformed and/or numerous connections are gaping. The deformations (or gaps) can no longer be caused by the anatomy of the wood.
Deterioration trend	0.5 ... if the roof covering, roof connections and roof drainage are defective and/or the adjacent masonry is soaked and/or the construction timber is heavily contaminated and/or for other reasons a deterioration of the condition of the roof elements is to be expected.
In-situ load conditions	-0,5 ... if an exceptionally large load has occurred in the last three years and as a result, no damage occurred. -1 ... as before, but for specifically applied test loads.
Total sum of the points	
Further criteria and experience	The sum of the points serves as an initial basis for the assessment. Within the framework of the "further criteria", a critical evaluation of the sum available so far is carried out. the points as well as the consideration of all criteria or aspects that have not yet been considered. The performance of regular inspections, for example, could be taken into account here. For the allocation of these points (positive/negative) the experience of the expert is required.

Figure 3.7. Grade system according to Meisel [17]

3-1.4 CONCLUSION AND COMPARISON

By comparing the approaches which are given in this thesis several conclusions can be derived.

a) The project structure is, in general, the same for all three assessment approaches
b) All approaches emphasise the importance of preserving heritage structures and the need for developing objectives of the assessment at the beginning of the project
c) The approach according to Cruz & others is developed for all types of timber structures, while approach according to Meisel is concentrated on the roof structures in the area of Graz.

To sum up, it can be concluded that approach according to ISO 13822 is not applicable for the assessment of the timber structures as material properties of timber are not taken into consideration, therefore during the assessment, approaches according to Meisel and/or Cruz & others should be used.

3-2 APPROACH TO THE SURVEY AND THE INSPECTION

It is important to develop a standardized approach to the survey and inspection of the roof structure, therefore recommended tools for surveying, allowable tolerances, the inspection and survey process will be discussed here. The recommendations regarding the surveying and inspections from papers [18] [62] [40] together with author experience during the survey are implemented to develop the recommended approach to survey and inspection process.

First **general tips** are given:

- Examiners need to take enough time for the survey. Rushing with surveying can only lead to mistakes or incomplete survey. Which will lead to more site visits and even more time consumption.
- Step by step survey should be planed. That means that members, cross sections, connections, damage assessment should be approached separately. In the case of mixing the steps, mistakes and confusion are probable.
- If original plans of the roof structure exist, they should be copied to serve as a basis. In other cases, hand drawings should be drawn on site.
- Plans or drawings should be made in large scale, as often measurements take a lot of space and in case of too small drawings, the drawing itself can become unreadable. A good and a bad example is shown in Figure 3.8.
- Reference axes in horizontal and longitudinal direction should be developed. For example, in this work, 18 transversal and 5 longitudinal axes are developed, see Figure 2.5
- The survey should be conducted in pair

The process itself should be separated into several parts, therefore several site visit should be conducted. The process can be divided on:

- **Existing documentation review**

Existing plans should be collected and reviewed. If existing, they should provide first drawing basis during a preliminary visual survey. In that way, drawing on site during the first survey is avoided and time is saved. However, even if original plans exist, they should be considered cautiously as built version usually differs from original plans. These differences need to be recognised and documented during the survey. (cf. [40])

- **Preliminary visual survey**

The preliminary visual survey serves as a basis for a detailed survey, and its main objective is to determine if the structure is safe enough for further survey. If the structure presents immediate danger Figure 3.5, temporary measures (e.g. propping) must be conducted to obtain safe condition. Furthermore, accessibility to all parts of the roof structure needs to be revised, and in case a part of the structure is not accessible, measures to obtain accessibility need to be taken (e.g. scaffolding). (cf. [40] [18])

- **Detailed visual survey**

The structure should be measured in detail as these measurements present the foundation for all further actions (e.g. modelling). Measurements according to the work of Harrauer can be conducted in 4 levels where level 1 is the simplest and level 4 is the most detailed (e.g. Level 4 measures consider the deformation of members). It is advisable for the sake of time and money consumption to keep the complexity of measurements in the middle of these two extremes. This approach was also used in this thesis and also in similar theses [18] [19] [16]. Nevertheless, for cases of invaluable heritage structures more complex measurements should be conducted. Allowable measurement tolerances for level 2 of the approach are given in Table 3.1.

Further steps which should be included in this part of the survey together with the needed tools are presented.

Establish sufficient lighting and clean surfaces

In attics, it is common that lighting is very weak to none. For that reason, sufficient lighting needs to be available. Regular pocket lamps are desirable but most of the time insufficient, therefore reflectors and several LED lights should be strategically distributed.

Surfaces of the timber members and connections should be clean in an extent that geometry can be determined and measured. Clean surfaces are also important for determining the timber species and visual grading of the timber.

Protective equipment

Examiners should wear protective gear given in Table 3.2.

Structure dimensions /disposition

- Disposition of roof structure should be determined. Roof structure needs to be fully measured. This includes the length of the members, the vertical and horizontal distance between members, etc.
- The vertical distance from slab structure to the lowest member of the roof structure should be determined in several places. Distance from members to the wall in order to place timber structure in respect to the rest of the structure should be determined on several places.
- Distance between axes of supporting structures needs to be determined at each side and in the middle.
- It is advisable that all dimensions are determined between member faces, instead of axial distances, because measuring the axial distances between members would provide larger space for mistakes.
- The verticality of posts should be examined with laser or simple construction string.
- When supporting structure is same in all axes there is no need for measuring the supporting structure in each axis. This is valid only when by visual inspection no obvious difference in dimension of supporting structure.
- It is recommended to make control measurements by determining the diagonal measures in combination with orthogonal measurements to form a triangle. This should be done in several places. This type of survey although time consuming is a reliable way how to obtain a correct survey of the structure or find a mistake in measurements. Examiners should decide upon their experience how many control measures are needed and where.
- Angles should be determined from triangles by using trigonometry relations. Measuring the angle of the member directly (e.g. Electric angle meter) should be avoided, because of the member imperfections.

Cross section dimension

- Dimensions of a member cross section should be determined in two different positions. The difference in measures between two positions should be taken into consideration only if the difference is larger than the given tolerance in Table 3.1. The measurement itself should be taken in position where the member isn't cracked. Cracks provide false results which can raise up to 2 cm. If the whole length of the member is cracked measurements should be taken in position where width of the crack is smallest. Alternatively, width of the crack can be subtracted from the measure which includes crack.

- For rectangular sections, dimensions can be noted in format width/height. In the case of irregular shapes, the cross section of the member should be drawn with its respective measures.
- Tree edges should be considered and documented according to sorting class see ÖNORM DIN 4074-1
- Cross sections should be documented on the same paper with structure dimensions, in that way axial dimensions can be determined in no time if necessary.
- In the case of repeating members, there is no need to measure them all if no clear deviation from usual dimension is observed. However, if every member is measured this can provide a good basis for statistical determination of the dimensions used in the structural model.

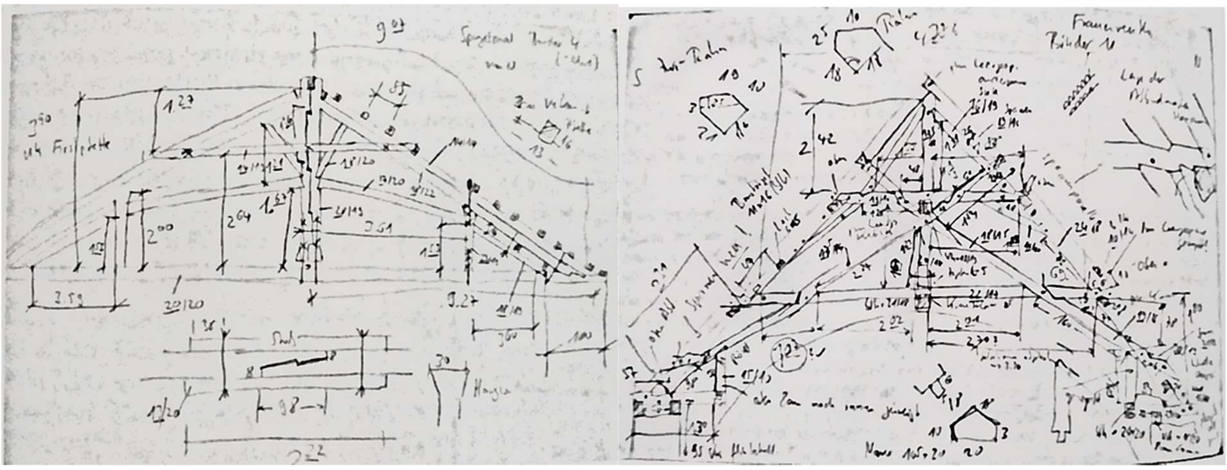


Figure 3.8. left: Good example of presenting the measurements; right: Bad example of presenting the measurements [62]

Table 3.1. Allowed tolerances during measurements

Tolerance values			
Nr,	Tolerance		Comment
1	Structure measurements	±5,0 cm	-
2	Angle measurements	±1,0°	-
3	Cross section measurements	±0,5 cm	-
4	Member verticality	±1cm/100cm	If the imperfection is smaller than tolerance then it can be neglected, otherwise, the member needs to be presented with the imperfection.
5	Deviation of a cross section along member	+-2,0cm	If not satisfied modelling of the member with a mean dimension of the cross section should be conducted. Also, the modelling of tapered elements is possible.

Measuring secondary construction

Secondary construction refers to any parts of the building structure which are influencing the position, disposition or it is structurally influencing roof structure in some way. These elements should be also measured and shown in plans.

Measuring connections

- Al connection types should be properly measured and drawn. The hand sketches should be made on site in large scale, with measurements which later provide a basis for detailed 2D/3D CAD drawings.
- Only several connections (2-3) of the same type should be measured, in order to check if dimension alters.

Photo documentation

- Photos of the roof structure should be documented, in order to get a better overview of the whole structure.
- It is recommended to take the pictures of the connections where scale meter is visible. This can be used for later control of the measurements. Photo documentation of the connections is of vital importance for later drawings of the connections. Pictures from several angels are advisable.
- All observed biological, mechanical damages must be documented
- For important images, the position and direction in which photo was taken should be documented in drawings.

Visual inspection of damage

In this inspection areas affected by biological degradation or mechanical damages should be documented.

- Any kind of damage should be photo-documented. The hand sketch and description of the damage should be also given in the last report. (e.g. geometry of the cracks, extent deformation of the member, disconnection of the connection, etc.)
- Roof cladding should be inspected for leakages
- Damage survey should be conducted by a structural engineer with sufficient experience in timber structures, alternatively for biological degradation by a wood technologist.
- A simple and effective way to determine the condition of the timber member is by a method of hammer sounding.

Conducting Non Destructive Tests

NDT should be conducted on places where visual survey recognised traces of timber deterioration. These tests provide invaluable data for later repair actions and for structural analysis by means of determining the effective cross section. The most important moisture and resistance test should be always conducted during the assessment, as they provide valuable data for further actions. More information about types of NDT test and their characteristic see subchapter 3.4

Damage documentation

- All damages should be documented in assessment report including description, scope and photos of the damages, in a way that is easy to read and understand, therefore more colour based sketches over long text descriptions are preferred.
- Schematic drawing of the structure in the top view and/or with some cross section should be presented. All acknowledgements from the visual inspection or NDT test should be marked according to symbols shown in Figure 3.9 left. An example is given in Figure 3.9 right.

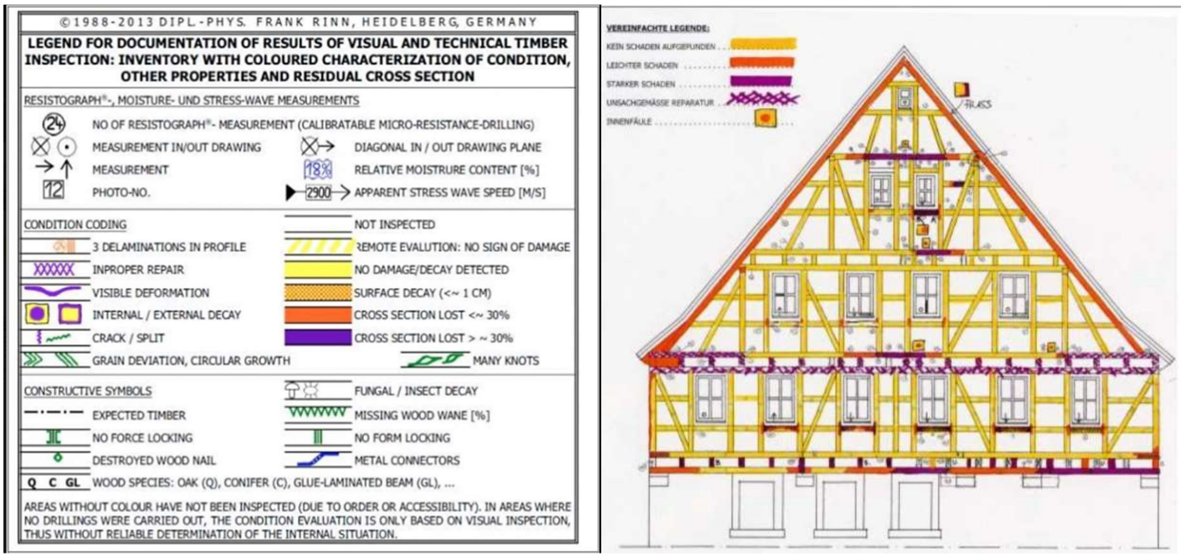


Figure 3.9. left: legend for the documentation of the results of the structural assessment; right: coloured map of the condition

Table 3.2. Needed equipment during the survey process

Overview of needed tools		
	Tools	Comment
Working environment	Ladders	-
	Broom	-
	Sandpaper	-
	Iron wire brush	-
	Crowbar	For removing floor or roof boards (accessibility)
	Lighting	LED lights, reflectors and hand lights
Protective gear	Hand gloves	-
	Protective shoes	-
	Protective glasses	In case of cutting or drilling
	Safety belt	-
	Helmet	
Documentation	Camera	Analogue camera or digital cameras with an additional Flash, tripod and wide-angle lens.
	Drawing papers	A3 / A4, and millimetre paper
	Chalk	White or in various colours
	Drawing plate	-
	various office tools	Pens, water-resistant markers, etc.
Measurements	Laser distancer	can be used for any kind of measurements
	Roll meter	length: min 5,0 m, 10,0m recommended
	Folding meter	length: 2,0 m for vertical measurements
	Laser	For checking verticality
	String	For checking verticality
	Wire	Depth of cracks, depth of supports in the masonry
	Electronic angle meter	-
	Feeler gauge	Width of cracks
Inspection	Nails	-
	Hammer	With face and claw
	Moisture	GANN M4050 with electrode M18
	Resistograph	ML Resi F300

3-3 ND TESTS

In most cases after visual inspection of the structure, complete picture of the structure condition can't be obtained, as visual inspection provides insight on the external degradation and location of degraded areas. Therefore, more powerful and effective actions should be taken. These actions include conducting various non-destructive test whose main objective is to determine the extent of degradation or condition of the inspected area. Semi destructive test can be used, however, they are not advisable for historic structures. The destructive test should be avoided. In Figure 3.10 list with various NDT methods which can be conducted on historic timber structures are shown. Conditions on site often prevent use of some methods. Some of the methods are more suitable for site and laboratory test while others are suitable only for Laboratory test, the last ones will not be discussed here.

Method	Information	ND/SD/D
Visual inspection	Geometry, rate of growth, natural and mechanical/structural defects, ...	ND
Stress and acoustic waves	Dynamic modulus of elasticity, internal defects	ND
Electric resistivity	Moisture content measurement	ND
Radiography – X-ray	Visualization of internal elements (screws, connection pieces), knots, voids, structural defects...	ND
Infrared thermography	Imaging of local moisture concentration, internal knots and voids, structural defects, decayed wood...	ND
Species determination	Determination of specious	ND
dendrochronology	Determination of age	ND
Endoscopy - videoscopy	Internal inspection of knots and voids, structural effects, decay of wood	SD
Resistance drilling (Resistograph ®)	Density and defects	SD
Core drilling (Zaphenschneider)	Core diameter is limited: 5-10 mm Used for compressive, tensile strength and Young's modulus	SD
Pin penetration resistance	Density and surface damage	SD
Specimen extraction	Extraction of cores or prismatic specimens for laboratory tests	D
Full-member tests	Mechanical properties of full-members	D
Standard tests of mechanical properties	Bending tests, compressive or tensile tests until specimen failure to obtain experimental data on ultimate strength	D

Figure 3.10. List of ND/SD/D test [63]

The methods and techniques presented above can be put in another context, too. It means that classification can be based on the other two possible criterions, defined as the expected result of the application and/or applicability which corresponds to certain level of the inspection. Given the first criterion, it refers to determination of general condition/information, the timber related properties and/or inhomogeneities in timber elements. With respect to the second criterion, it refers on suitability of certain method to be applied within any of the three following phases: preliminary inspection (first site visit), continuative investigation (detailed on-site inspection) and, finally, limit state analysis (including possible laboratory test).

The presented approach is out of the scope of this master thesis.

3-3.1 MOISTURE CONTENT MEASUREMENT – ELECTRIC RESISTANCE

Moisture content has the largest influence on timber properties and durability, therefore an effort must be made to determine these values.

Measurement of the moisture content is conducted with a device called hygrometer Figure 3.11. Hygrometer works on the principle of electrical resistivity. Weak current is discharged into the timber trough metal pins and resistance is then measured. The electrical conductivity of timber is in good correlation with moisture content. [14]

The results are highly dependent upon temperature, wood species and position of the pins in respect to the grain direction. The two pines must be positioned perpendicular to the grain direction otherwise results are incorrect. Before conducting the measurement, the temperature during the measurement and the wood species must be entered in the hygrometer.

The method is reliable in following ranges (slight differences are possible due to the device capabilities) [14]

- Temperature: 5 °C – 60 °C
- Moisture content: 7,0%-30,0%

Figure 3.11. Hygrometer Gann Hydrometers M4050 [18]

3-3.2 RESISTANCE TEST

Resistance test is one of the most popular NDT methods for use on the site. The test is conducted with Resistograph® Figure 3.12 left. The method is based on drilling the small diameter borer trough timber. During drilling the penetration resistance on the bore head is recorded and plotted as a function of drilling depth Figure 3.12 right. This method is used to determine sound or deteriorated zones and fissures inside the cross section, which can't be observed during visual inspection. After reading the test results, the residual cross section can be assumed. (cf. [14] [64])

Although it is possible to determine the density of the drilled specimen by this method, the obtained correlations are insufficient for a reliable assessment of the properties. The reason is that penetration resistance dependent upon the borer head sharpness, which can't be controlled. (cf. [64])

When drilling Reistograph must be positioned perpendicular to the member and held with both hands. This can present a problem for drilling in areas with limited space. Measures should be conducted in two perpendicular directions if possible, in order to obtain a complete picture of the cross section. Areas with nails or metal fasteners should be avoided. In case of hitting the knot deviations of the borer from a strait path are possible, this induces additional stress and results become ambiguous. In works [65] [62] the approach to the reading of the results from the resistance test is explained in detail. (cf. [64])

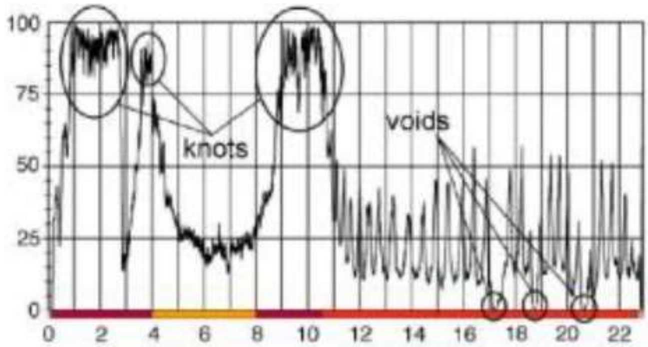


Figure 3.12. left: Resistograph Resi F 300 [18]; right: Resistance profile [14]

3-3.3 PIN PENETRATION RESISTANCE

Pin penetration resistance is determined with Piloydin Figure 3.13. A steel pin is driven into the wood by force released by spring. Depth of penetration is correlated to the density. The steel pin penetrates only few millimetres into the wood, therefore downside is that only superficial hardness is inspected. Nevertheless, depth of penetration and density is in good correlation independent upon timber age. The obtained correlation factors are in range 0,74-0,92 according to [64]. Correlation of the density and penetration depth is highly dependent upon species, moisture content, and a number of measurements. To obtain a good correlation lot of measurements are needed for each specie, and also to avoid the influence of local defects such as knots. Correlation between penetration depth and module of elasticity can also be obtained. (cf. [64])

Figure 3.13. Piloydin [61]

3-3.4 HAMMER SOUND TEST

By examining, the return sound after hammer hit the timber surface it is possible to approximately determine the condition of the member. For sounding method face, part of the hammer should be used when hitting. If return sound is sharp and sound and hammer handle vibrates timber is in good condition. If the sound is dull and silent, deterioration is present. Afterwards, claw part of the hammer should be used to determine if timber crumbles under the impact. If not, timber is in good condition. With using a claw part, it should be kept in mind not to provide extensive damage to the member if no deterioration is present.

CHAPTER 4: ASSESSMENT OF THE ROOF STRUCTURE "*TU GRAZ ALTE TECHNIK*" (SOUTH WING)

This chapter presents the evaluation of the structural condition of the roof structure at TU Graz Alte Technik. The approach according to Meisel is used. Measuring process and visual survey are conducted in compliance with Chapter 3-2.

4-1 DAMAGE ASSESSMENT

During a visual survey at the roof structure TU Graz Alte Technik, several damages to the members and connections are observed. These damages are presented together with photo documentation and assessment of the danger to the reliability of the structure.

4-1.1 MEMBER DAMAGES

Right brace on King post on axis 14 is marked as damaged Figure 4.1. As explained in Chapter 2-1 it may be assumed that the right brace is not part of the original structure, but that is later added. The brace itself is torsion rotated for approximately 10 degrees, accompanied by characteristic torsional cracks. It can be assumed that this deformation is caused by shrinking. This kind of deformation caused partly disconnection of the brace form the post and from the tie beam, see Figure 4.1 left. Iron cramps are installed only on one side of the brace, therefore they couldn't stop the disconnection. See Chapter 6-2 for further actions.

Figure 4.1. left: Connection Brace-Post (Photo:6500); right: Brace on axis 14 (Photo:6644)

On axis 14 and 13 the damage to the collar beams is detected Figure 4.2 left. On both axes one of two collar beam is cut in order to provide space for chimney and ventilation pipes. Although cross section of collar beam at these two axes is reduced by 50% in comparison to other axes, this damage is not marked as priority or critical as these axes are abutting and the collar beams, in general, are carrying a relatively small load.

In room 1 on axis 17 damage to Knee brace is detected Figure 4.2 right. Moreover, similar damages are found on Tie beam on axis 16 and 15. These damages are the result of human influence. As loss of cross section is not bigger than 10% these damages don't present an issue.

Figure 4.2. left: Cut Collar beam (Photo 6525); right: Damaged Knee brace (Photo: 6624)

In room 3 on axis 3 on the south side, an example of poorly undertaken repair is shown in Figure 4.3. This kind of an unprofessional repair can result in even lesser load bearing capacity than before the repair.

The critical part of this repair is the area where the new tie connects to the remains of the old tie. The ties are joined with self-taping wood screws. The load bearing capacity of the fastener group is not calculated and screwed according to profession doctrine. The bearing capacity is unknown and the repair itself doesn't represent the spirit of heritage preservation. See Chapter 6-2 for further actions.

Figure 4.3. left: Changed short tie axis 3 (Photo6604); right: Connection of the new short tie to the old short tie (Photo:6468)

- **Biological damage**

On the most roof structure members near the roofing, eaves and masonry the change in colours is observed. The colours observed are black, white, and darker brown. The inspection with hammer survey didn't show any damages to the inner timber. Only surface changes are present. With this, it may be concluded that change in colour is due to the sap fungi. The discolourations are shown in Figure 4.4.

The areas around former chimneys have distinctive white colouration. By inspection, it is determined that this is not the result of the fungal attack but rather lime which was present around chimney walls.

In room 3 on the tie beam, the damage due to insect attack is found Figure 4.5. The member is attacked by House longhorn beetle, see Chapter 1-4.2. The moisture content of the beam and environment temperature suggest that infestation may still be active, although no evidence which could support that is found during the survey. Further actions should be taken to determine if the insects are still active.

Figure 4.4. left: The white colour of the transversal beams and dark stripes on the knee brace; right: Sap fungi and wood oxidation

Figure 4.5. Damage to the tie beam due to the House longhorn beetle

4-1.2 CONNECTION DAMAGES

Overall most of the connections show moderate disconnections due to the shrinkage of members and connections.

The connection of the ties to brace on axis 10 south shows severe cracking in area of connection. The cracks severely reduce the area of the shear plane. On Figure 4.6 right, with the red line, the remaining height of the shear plane is shown. After the numerical calculation, the verification of this damaged connection hasn't pointed to failure under ULS. Therefore, no actions are needed. In addition, bolt acts as an alternative load pathway.

Figure 4.6. Cracks in connection brace-short tie

On a number of connections of a brace to the tie beam and a brace to the queen post partly disconnections are found. The examples are shown in Figure 4.7.

It is hard to define the exact reason why on some places such big gaps and near disconnections are present one of the reasons are: shrinkage, the force from short ties which tend to pull the brace out, mistakes during construction and possible tension forces in the braces which caused partly disconnection.

The single notched joint as shown in Figure 4.7 is liable to cracking in corner of the notch due to the concentration of the tension stress perpendicular to the grain. This crack occurs if no gap is left between the horizontal contact plane of the tie beam and brace. In literature in order to avoid this effect, the gap of minimal 0,5 – 1,0cm is needed. [12] By comparing the left and right side of Figure 4.7 this effect can be clearly seen. See Chapter 6-2 for further actions.

Figure 4.7. left: Partly disconnection of the connection brace-tie beam; right: Cracks due to the load transfer

On the north column of axis 10, both iron cramps have failed in tension. The cramps could approximately carry around 20kN of force. This force magnitude is not realistic in this structure, therefore it is probable that iron cramps where faulty at the beginning and during minor tension force the failure occurred, furthermore the column is rotated around its longitudinal axis which could also cause the failure. The failure itself is not critical.

Figure 4.8. Failure of the iron cramp on axis 11 (north post)

4-2 MOISTURE RESULTS

Moisture tests are conducted with Gann Hydrometers M4050 with the associated ram electrode type M 18. Air temperature during the time of inspection was 15 °C. All inspected elements are visually graded as spruce. In total 30 test points were conducted including the domes. The results are in the range of 9,0%-11,5%. Results are filtered to maximum values for each roof element. Those results are shown in Table 4.1.

Table 4.1. Moisture test results

Moisture level of timber members			
Nr.	Element	Element position	Moisture level [%]
1.	Tie beam	axis 16	9,6
2.	Brace	axis 8 - North	10,4
3.	Tie	axis 6 - North	10
4.	Straining beam	axis 6	9,2
5.	Queen post	axis 17 - North	9,8
6.	Foot purlin	North between axis 7 and 8	10,5
7.	Middle purlin	North between axis 7 and 8	11,4
8.	Rafter	(rafter axis) 40	13
9.	Collar beam	axis 15	9,1

4-3 RESISTANCE RESULTS

The resistance tests are conducted with Resi F300, mounted on the cordless drill Figure 3.12. The borer head is 3 mm while borer is 1,5mm in diameter. The borer head is shaped in a way that timber dust stay in the hole, in that way damage to the timber is smallest possible and risk from insect attack is reduced.

The tests are conducted in places where moisture content is high or where by visual inspections discolourations and damage to the members are observed. The tests are also conducted on positions where the geometry of connections couldn't be established by visual inspection (e.g. dimension of tenon and mortise). On the following figures, the results of the conducted resistance test are shown together with short description.

Tie beam axis 1-2 Nr. 17

The top surface of the tie beam is damaged from insect attack as shown in Figure 4.5. On Figure 4.9 the resistance profile of the tie beam is shown. In the first 3,0 cm, the smaller relative density is observed. This implies damage from insect attack as observed by visual inspection, however, the timber is not completely deteriorated as some resistance is still present. For calculation purposes, the first two centimetres are considered as deteriorated.

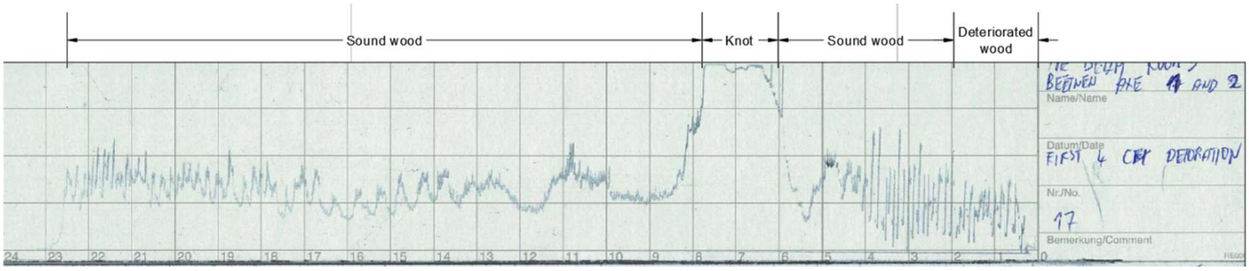


Figure 4.9. Resistance profile on the damaged Tie beam axis 1-2

Connection geometry and dimensions

By examining resistance profiles given in Figure 4.10, Figure 4.11, Figure 4.12 the geometry of the connections are established. By observing steep inclines in resistance graphs followed by areas of no resistance, gaps are discovered and subsequently geometry and dimensions of the connections. On all figures, the area of the gaps is presented, also with all parts of the connection. The dimensions can be determined from the ruler at the bottom of each figure. For better understanding refer to pictures of these connections in Chapter 2-3 or to the 3D/2D drawings in Appendix C.

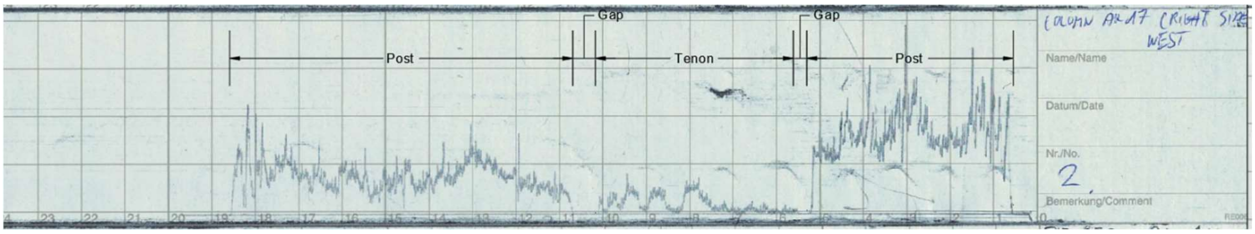


Figure 4.10. Resistance profile of connection knee brace-queen post

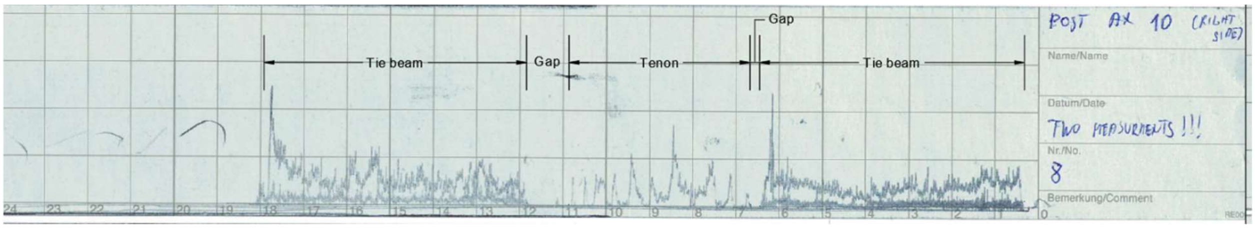


Figure 4.11. Resistance profile of connection tie beam-queen post

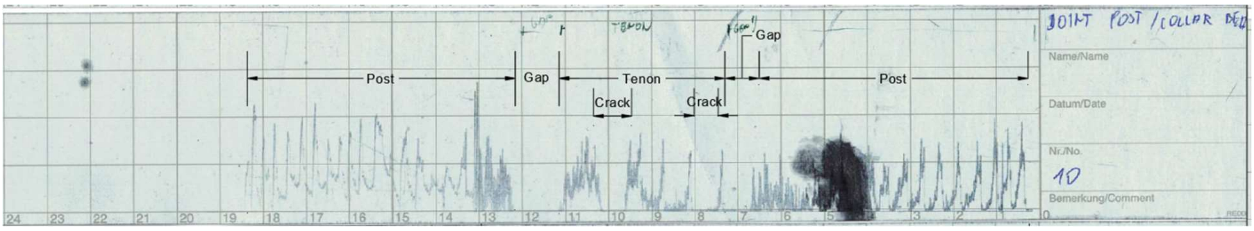


Figure 4.12. Resistance profile of connection straining beam-queen post

Rafter axis Nr. 11 and 12

In Figure 4.13 the view on the resistance profiles of the rafter axis 7.1 is shown. During the visual inspection, this rafter showed black/white discolouration accompanied by cracks. The resistance tests are therefore conducted in two perpendicular directions. By examining both resistance profiles no damages to inner timber are detected.

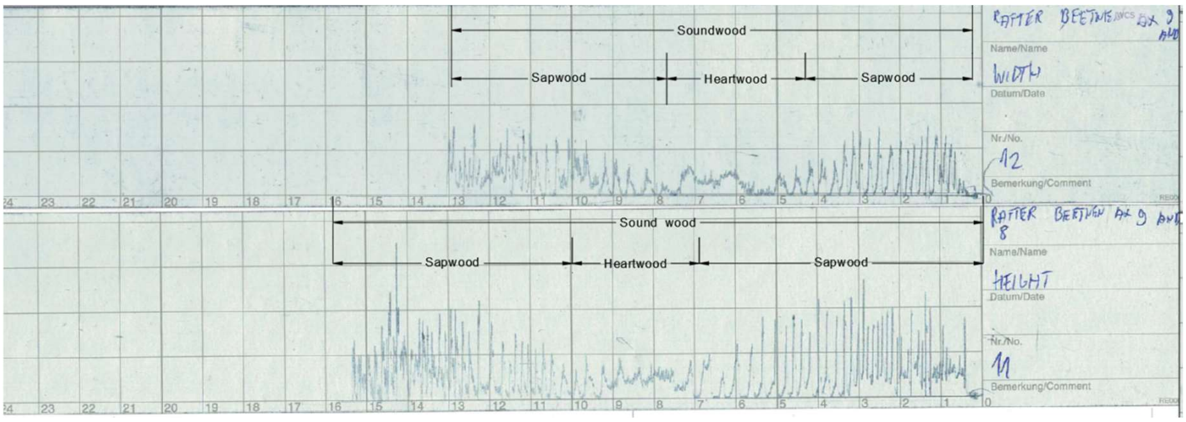


Figure 4.13. Resistance profile of rafter on axis 7.1 (up, profile along width); (down, profile along height)

Connection Tie-Rafter – Nr. 19

On axis 12 due to the severe discolourations resistance test is conducted. By detecting the gaps, the boundaries between ties and rafter are detected. In resistance profile shown in Figure 4.14 no damage or deterioration is detected.

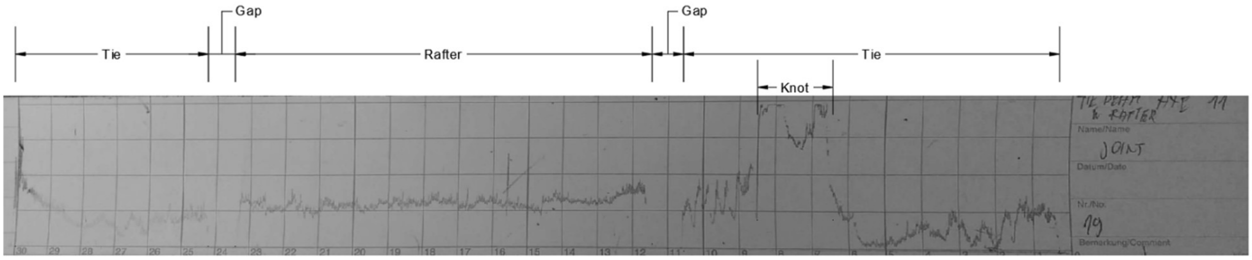


Figure 4.14. Resistance profile tie-rafter

4-4 MATERIAL GRADING

In comparison to new structures where the quality of the material is prescribed by an engineer during the design process and quality of the material is controlled and verified by various certificates, the material properties in existing timber historic structures are unknown.

There are numerous ways to determine properties of the timber such as:

- Visual evaluation according to the sorting criteria given in ÖNORM DIN 4074-1. [66]
- Ultrasonic pulse transit time measurements (modulus of elasticity)
- Tests on core drillings (bulk density, transverse compressive strength)
- Removal of wooden components and testing

By using one of these methods or combining them it is possible to obtain material properties of the timber. However, combinations usually result with high consumption of time and money. In addition, measures and geometry of the cross sections during surveying are usually not precise and for this reasons determination of correct material, properties are not economically feasible. In most of the works, visual gradation by an experienced surveyor is sufficient. (cf. [17] [16]) Only for heritage structures of the great value more precise approach by some of given methods should be considered. The last two approaches include damaging the existing structure up to some extent or complete removal of the elements for testing, the appropriateness of these methods should be reconsidered and evaluated with conservators before execution.

In numerous papers, the degradation of strength in timber is not confirmed. Visual grading is, therefore, suitable for the determination of the material properties. The visual grading according to ÖNORM DIN 4074-1 and ÖNORM DIN 4074-5 (2009) was developed for grading of sawed timber before its implementation in structure, which means that the whole element is accessible for inspection and the timber is not influenced by stress or time component. The approach by these norms is not suitable for historic structures. Therefore, a modification must be made for grading of historic timber structures on site, these modifications are presented by Meisel.

According to an approach by Meisel, the timber elements are divided into two groups

1. "Good construction timber" all timber that show no damage (such as destruction by fungi or insects), are graded as C24 (corresponds to grading class S10) according to ÖNORM DIN 4074-1. [66] It can be assumed that in the case of a detailed examination of all timbers, some proportion of the timber would have been classifiable as S13.
2. All woods that show statically relevant damage. The repair actions should be taken.

Moreover, in ÖNORM EN 388 (2009) [67] experience has shown that, as a rule, at least the strength class C 24 can be assumed. After carrying out of the verification process those members and connections, which doesn't satisfy the safety margin, should be checked by some of the methods given at the beginning of the subchapter or NDT methods, in order to justify the improvement of the strength class to C 30.

Further criteria according to ÖNORM DIN 4074-1 [66] for each grading category C 18/C 24/C 30 (S 7/S 10/S 13) are presented in Table 2 and Table 5.3 in ÖNORM DIN 4074-1 [18].

4-4.1 MATERIAL GRADING IN THE ROOF STRUCTURE "*TU GRAZ ALTE TECHNIK*"

For roof structure considered in this thesis, all elements are graded as C24. In case of lack of reliability shown by the verification process, the further visual inspection will be conducted on these members to determine if upgrading the quality of the member to C30 is justified.

4-5 RELIABILITY GRADING SYSTEM

For inspected roof structure and domes on the south wing, reliability grading system according to Meisel is undertaken. The grading system is elaborated before in Chapter 3-1.3.1 therefore only results and comments of the assessment are shown.

Condition assessment of the domes TU Graz Alte Technik (South wing)

Consequence class	**1**... Faculty - crowds of people
Structural safety	
Degree of static indetermination	**0,4 Axis A, D failure vertical column -> collapse of the part of the roof**
Conversions and repairs	**0** ... no repairs
Biological degradation	**0.1**.. minor fungal attack on secondary structure - wooden boards
Connection and/or member failure	**0** ...
Constructive shortages	**0** ...
Large deformations and/or gaps	**0** ... no gaps present
Deterioration trend	**0** ... the roof covering, is new and it is not leaking, for areas around chimneys and eaves same applies
In-situ load conditions	**-0 -** no large load has occurred in last three years
Total sum of the points	**1,5**
Further criteria and experience	**The structure of the dome is in very good shape, structural elements are not damaged or infected by any kind of biological attack.** **Recommended actions according to the result:** **-Regular maintenance**

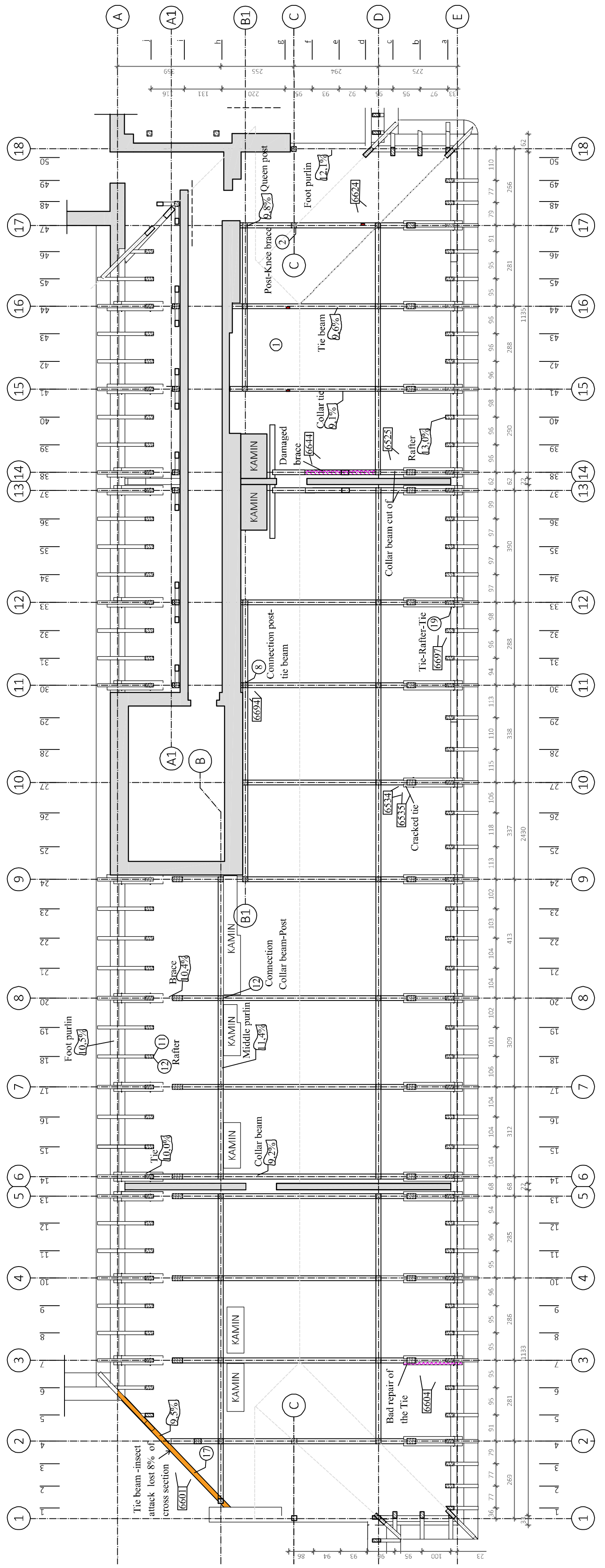
Condition assessment of the roof structure TU Graz Alte Technik (South wing)

Consequence class	**1**... Faculty - crowds of people
Structural safety	
Degree of static indetermination	**0,3**... Queen post = 1 degree of internal determination
Conversions and repairs	**0,2** ... repair on one location is not professionally undertaken
Biological degradation	**0,2** ... little damage from previous fungal attack, during assessment inactive
Connection and/or member failure	**0,1...**
Constructive shortages	**0** ... all dimension are sufficient
Large deformations and/or gaps	**0,3** ...gaps in connections are present
Deterioration trend	**0** ... the roof covering, is new and it is not leaking, for areas around chimneys and eaves same applies
In-situ load conditions	**-0 -** no large load has occurred in last three years
Total sum of the points	**2,1**
Further criteria and experience	**In general roof structure is in good shape and load bearing capacity and functionality of the roof structure are not endangered, although some damages are observed.** **- Minor damages from biological attack are observed** **- Badly undertaken retrofitting is conducted on one structural element** **- At two positions structural elements are cut , and removed** **Recommended actions according to the result:** **- Regular maintenance** **- During next repair of roof covering the retrofitting of the marked as damaged should be reconsidered**

4-6 CONCLUSION AND STRENGTHENING PROPOSALS

4-6.1 MAP OF LOCAL DAMAGES

On this map, the summation of all damages discovered during the survey is presented. The locations of conducted ND tests are presented, moreover, all relevant photos which were presented in text above are shown with an angle of view. The map and its symbols correspond to symbols given in Figure 3.9.


Queen post
9,8%
Foot purlin
2,1%
6624
Post-Knee brace
Tie beam
5,6%
Collar tie
9,1%
Damaged brace
6644
6525
Rafter
3,0%
KAMIN
Collar beam cut of
Tie-Rafter-Tie
6697
Connection post-tie beam
6694
6534
6535
Cracked tie
Brace
10,4%
Connection Collar beam-Post
Middle purlin
1,4%
Foot purlin
10,5%
Rafter
Collar beam
9,2%
Tie
10,0%
Bad repair of the Tie
6604
9,5%
Tie beam -insect attack lost 8% of cross section
6601

CHAPTER 5: MODELLING OF THE ROOF STRUCTURE “*TU GRAZ ALTE TECHNIK* “

In this chapter, the roof structure of TU Graz Alte Technik is structurally assessed. The load analysis and material properties are determined according to the valid norms. The approach to the modelling of the roof structures and connections is explained. Also, the results of numerical analysis of the roof structure *TU Graz Alte Technik* together with a comparison between different approaches to modelling (models) is presented.

5-1 LOAD ANALYSIS

5-1.1 SELF AND DEAD WEIGHT LOAD

Self-weight of the roof structure is taken into consideration with command “self-weight”. Densities which software uses are automatically embedded into it, according to ÖNORM EN 1991-1-1- 2009. The additional deadweight of roof layers which are not included automatically in models is shown in Table 5.1.

Table 5.1. Additional dead weight

Layer	Density [kN/m^3]	Distance [m]	Load per square meter [kN/m^2]
Roof tiles (Eternit)		-	0.3
Timber battens	420	0.16	0.05
∑ SUM			**0.35**

Timber battens are graded as class C24 therefore according to ÖNORM EN 1991-1-1 -2009 [68] Table A.3 mean density is 420 [kg/m3]. According to national annex ÖNORM B 1991-1-1 – 2011 [69], Table A.4 mean density for C24 is given as 550 [kg/m3]. Both densities can be used, but as the software uses density according to [68] same density is used for calculation of additional deadweight. For the density of Eternit roof tiles which are not included in norms, the density provided by manufactures is used.

During the inspection of the roof structure, it is noticed that pipes from air ventilation are supported on several places on Straining beams. As the weight of these pipes is unknown their load is not considered. It is believed that load from the pipes have a minor influence on model results with respect to the other loads.

5-1.2 SNOW LOADS

Snow load on roof structure is calculated according to [70] [71]. Snow load is calculated for pitch roof case. Three load cases are established: one symmetric and two asymmetrical see Figure 5.1 left. Characteristic snow load is given according to Table A.1 given in [71]. Characteristic snow load for location Graz is presented in Table 5.2.

The geometry of the valley and hip roof is suitable for the development of snowdrifts in valleys. Drifts in the valley areas are neglected as no realistic load model is developed by valid codes for this roof geometry (cf. [70]).

Snow load shape coefficients are calculated according to expressions in ÖNORM EN 1991-1-3 -2016 Table 5.2 [70], as calculated values shouldn’t be applied if point 5.3.3. (2) is not satisfied, the recommended value is used (cf. [70]) Calculated value by an expression given in Table 5.2 in [64] is given in brackets.

Calculation of characteristic snow load on the roof plane is following. Furthermore, a schematic illustration of loads on the roof for each load case is given in Figure 5.1 right.

Table 5.2 Characteristic snow load for location Graz according to [71]

Location	Load zone	Meters above sea level [m]	Characteristic Load s_k [kN /m^2]
Graz	II.	369	1,65

$C_e = 1{,}0$

$C_s = 1{,}0$

$\mu_1 = \mu_2$

$$\mu_1 = \mu_1(0^\circ)\cdot\frac{(60^\circ\text{-}\alpha)}{30^\circ} = 0{,}8\cdot\frac{(60^\circ\text{-}34^\circ)}{30^\circ} = (\cancel{0{,}69}) = 0{,}8$$

$$S_1 = \mu_1 \cdot C_e \cdot C_s \cdot s_k = 0{,}8\cdot1\cdot1\cdot1{,}65 = 1{,}32\ \frac{kN}{m^2}$$

$$S_2 = 0{,}5 * \mu_1 \cdot C_e \cdot C_s \cdot s_k = 0{,}8\cdot1\cdot1\cdot1{,}65 = 0{,}66\ \frac{kN}{m^2}$$

$$S_3 = 0{,}5 * \mu_1 \cdot C_e \cdot C_s \cdot s_k = 0{,}8\cdot1\cdot1\cdot1{,}65 = 0{,}66\ \frac{kN}{m^2}$$

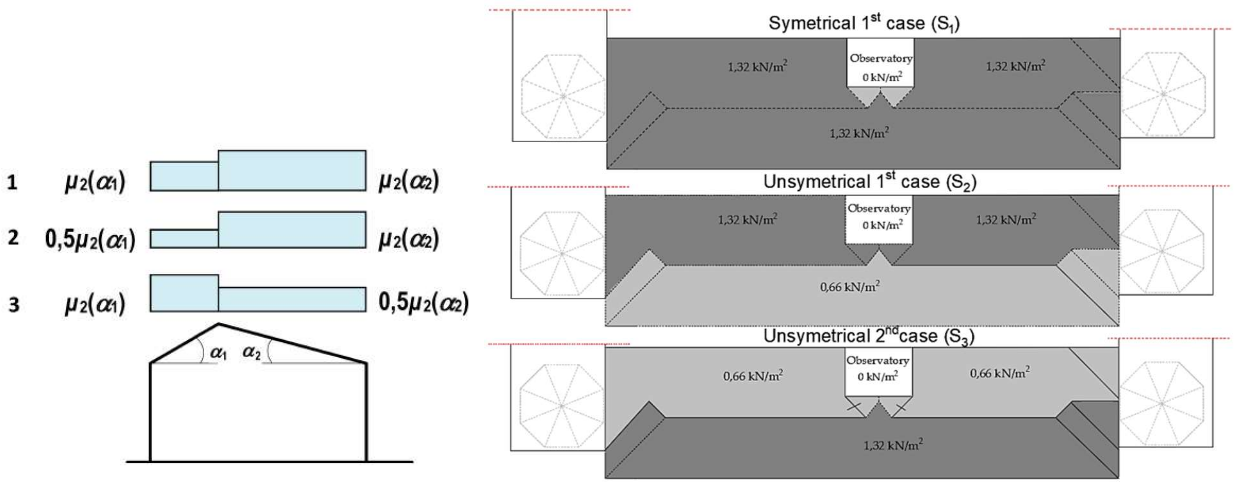


Figure 5.1. left: shape coefficients and 3 load cases according to ÖNORM EN 1991-1-3 [70]; right: characteristic snow load on roof structure for 3 load cases

5-1.3 WIND LOADS

Wind loads on roof structure are calculated according to ÖNORM EN 1991-1-4-2011 [72] and ÖNORM B 1991-1-4–2013 [73]. In these norms load model for the wind load is considered in two independent and perpendicular directions North/South and East/West. In mentioned norms load models only for simple roof geometry are given. Roof geometry of TU Graz Alte Technik differs from characteristic gable or hipped roofs, therefore usage of load models given in mentioned norms for obtaining pressure coefficients is not completely reliable as local influences around hipped and valley areas are unknown. Furthermore, literature in this area is slim and doesn't give any methods for approach to such geometry and influences. In lack of better models, available load models for simple roofs in mentioned norms should be used.

For modelling of wind loads on this roof structure author decided to choose an approach in which combination of two different load models are used. Different models are used in a way to approach real geometry of the roof in observed direction as much as possible. Load models for duopitched roofs and hipped roofs are used. For a schematic illustration of wind zones their sizes and shape refer to ÖNORM EN 1991-1-4-2011 [72] (Figure 7.8 for duopitch roof and Figure 7.9 for hipped roof). In Table 5.3 considered wind load cases and load models are shown.

Table 5.3. Wind load cases and used models

Number	Load case	Used model
1.	Wind – North (+) – pressure = Wind 1	Duopitch roof
2.	Wind – North (-) – suction = Wind 2	Duopitch roof
3.	Wind – South (+) – pressure = Wind 3	Hipped roof
4.	Wind – South (-) – suction = Wind 4	Hipped roof
5.	Wind – East/West (-) – suction = Wind 5	Duopitch roof

Combination of load models has questionable reliability and their usage makes calculations more complex and twice as slow. Moreover, pressure coefficients for these models do not differ much when we exclude hip and valley areas on east and west side of the roof structure. In this tone it`s vastly recommend not to combine load models and proceed with just one of the models which engineer find more suitable.

The input of wind loads in the structural model for more complex geometries can become very demanding and time consuming. Therefore, it is advisable to merge several wind zones into one big zone by means of calculating geometric mean value. This type of approach vastly simplifies load input. Influences of simplification are minor to none. Similar approach can be found in (cf. [18] [16] [20])

Calculation of characteristic peak pressure is following. Important input variables are given in Table 5.4 Furthermore schematic illustration of calculated loads on the roof for each load case is given in Figure 5.2. In appendix D table with results of interpolated pressure coefficients c_{pi}, c_{pe} are given. The loads are calculated according to expression (5.1)

Load cases presented in Table 5.3 present most unfavourable load case in each direction for cases of pressure and suction. Internal pressures are taken as recommended values given in [72].

a) Cpi +0,2 pressure
b) Cpi -0,3 suction

Friction force acting on the roof surface is neglected.

Table 5.4 Characteristic input values for calculation of wind load according to ÖNORM EN 1991-1-4-2011 [72] and ÖNORM B 1991-1-4 [73]

Location	Basic wind velocity $v_{b,0}$	Building height z	Terrain category	Minimum height z_{min}	Basic pressure $q_{b,0}$
Graz	20,4 m/s	20m	IV.	15m	0,45 [kN/m^2]

$$q_p\,(z)=\left[1{,}2\cdot\left(\frac{z}{10}\right)^{0.38}\right]\cdot q_{b,0}=\left[1{,}2\cdot\left(\frac{20}{10}\right)^{0.38}\right]=0{,}41\,\frac{\text{kN}}{\text{m}^2}$$ Peak pressure - ÖNORM B 1991-1-4 [46]

$w_i=c_{pi}\cdot q_p(z)$ load acting on internal surfaces- ÖNORM EN 1991-1-4-2011 [45]

$w_e=c_{pe}\cdot q_p(z)$ load acting on external surfaces- ÖNORM EN 1991-1-4-2011 [45]

$w=w_i+w_e$ (5.1) Resulting load on roof surface - ÖNORM EN 1991-1-4-2011 [45]

As for historic timber structures which have great historical value and in same time complex roof geometry, for example, pitched roof towers of cathedrals, more attention and time to the calculation of wind loads should be invested. As norms don't give answers to these kinds of geometry, some experimental approaches such as wind tunnel tests or implementation of pressure sensors on existing structure can provide an answer to such problem.

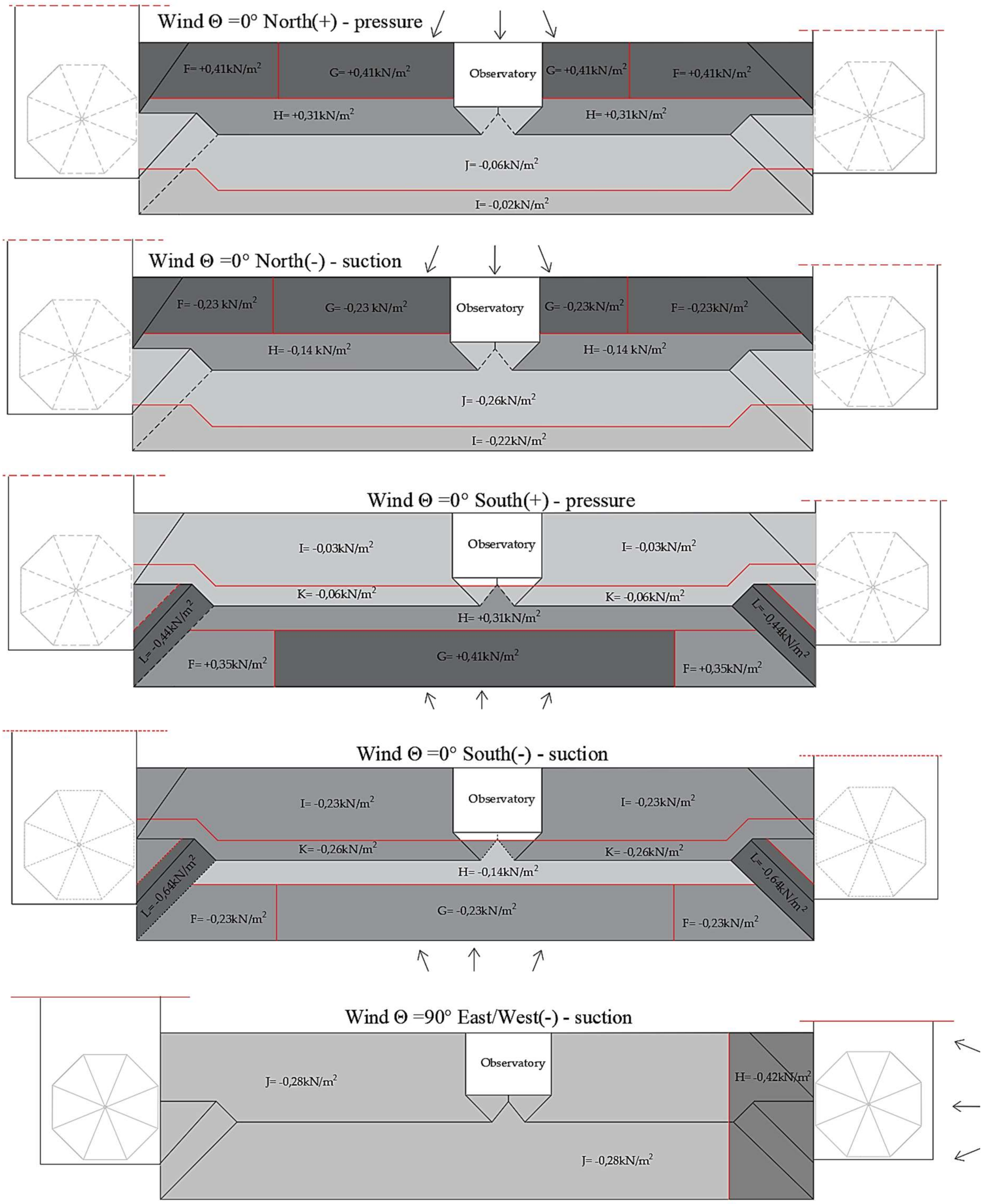


Figure 5.2. Illustration of wind loads on roof structure TU Graz Alte Technik

5-1.4 LOAD COMBINATIONS

Load combinations are determined according to ÖNORM EN 1990-2013 [74] [75] Load combinations for Ultimate limit states, farther referred as ULS, and Serviceability limit states farther referred as SLS are presented in Table 5.6. Partial factors and combination factors are given in Table 5.5. ULS are calculated according to [74] expression (5.2).

In combination where snow is combined with wind, only wind cases which produce pressure are combined with snow as they produce critical results see Table 5.6.

$$\sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} \text{ "+" } \gamma_P \cdot P \text{ "+" } \gamma_{Q,1} \cdot Q_{k,1} \text{ "+" } \sum_{i\geq 1} \gamma_{Q,i} \cdot \psi_{Q,i} \cdot Q_{k,i} \tag{5.2}$$

SLS states for Timber structures need to account the effect of elastic deflection and effects of the long-lasting load. Therefore, two SLS verifications need to be undertaken.

a) Characteristic combination for elastic deflection according to Section 7.2 [59]

$$w_{inst} = \sum_{j\geq 1} w_{inst,G,j} + w_{inst,Q,1} + \sum_{j\geq 1} \psi_{0,i} \cdot w_{inst,Q,i}$$

b) a quasi static expression for accounting creep according to Section 7.2 [59]

$$w_{inst} = \left[\sum_{j\geq 1} w_{inst,G,j} + \sum_{j\geq 1} \psi_{2,i} \cdot w_{inst,Q,i} \right] *(1+k_{def})$$

Table 5.5. Partial factors and combination factors according to [74]

Load	γ_{SUP}	γ_{INF}	Ψ_0	Ψ_1	Ψ_2
Self and dead weight	1,35	1,0	-	-	-
Wind	1,5	0	0,6	0,2	0
Snow	1,5	0	0,5	0,2	0

Table 5.6. Load combinations for ULS and SLS

Verification	Nr	1st load	2nd load	3rd load
ULS	1.	1,35 * Self weight and additional dead weight	-	-
ULS	2.-4.	1,35 * Self weight and additional dead weight	1,5*Snow1...+…Snow3	-
ULS	5-9.	1,00 * Self weight and additional dead weight	1,5*Wind1…+…Wind5	-
ULS	10-15.	1,35 * Self weight and additional dead weight	1,5*Wind 1 or Wind 3	0,75*Snow1...+…Snow3
ULS	16-21.	1,35 * Self weight and additional dead weight	1,5*Snow1...+…Snow3	0,9*Wind 1 or Wind 3
SLS	22	1,00 * Self weight and additional dead weight	-	-
SLS	23.-25	1,00 * Self weight and additional dead weight	1,0*Snow1...+…Snow3	-
SLS	26-30.	1,00 * Self weight and additional dead weight	1,0*Wind1…+…Wind5	-
SLS	31-36.	1,00 * Self weight and additional dead weight	1,0*Wind 1 or Wind 3	0,5*Snow1...+…Snow3
SLS	37-42.	1,00 * Self weight and additional dead weight	1,0*Snow1...+…Snow3	0,6*Wind 1 or Wind 3
SLS	43.	1,60* Self weight and additional dead weight	-	-

5-2 MODELLING OF THE HISTORIC TIMBER STRUCTURES

5-2.1 APPROACH TO THE MODELING OF THE HISTORIC TIMBER ROOF STRUCTURES

Structural analysis after assessment and surveying of the roof structure is the next step in the preservation project of the historic timber structures. The structural analysis output is direct input for any needed strengthening's or repair actions. As previously emphasised repair and strengthening's should be kept on the minimum to preserve the cultural identity of historic timber roofs, therefore the more realistic approach to the structural modelling of the roof structures should be nourished by an engineer.

The numerical model of the structure presents the mathematical interpretation of the real structure, therefore simplifications during the conversion of the real structure to the numerical model are unavoidable. In order for the engineer to produce a realistic model in an acceptable amount of time, the specialised knowledge in the behaviour of timber and historic timber roof structures is mandatory. In numerous literature step by step, modelling is advisable. In other words, the structural analysis should be approached at the beginning with the making of the simpler models and then proceeding to the more complex models which simulate state close to reality. The idea behind this approach is that simpler models provide answers on the global behaviour of the roof structure and its critical points, where more complex models provide the basis for determination of structural safety and give answers on needed strengthening or repairs. (cf. [62] [17] [32])

In simple and complex models simplifications are necessary, therefore the impact of those simplifications on later results must be analysed and derived in order to decide which simplifications are suitable for which complexity level of a numerical model. The decision is dependent upon several variables [17]:

- Value of the structure,
- Condition of the structure (biological degradation, damages)
- Time consumption (time spent in the making of the model)
- The regularity of the roof structure
- Quality of the assessment process and surveying

Another challenge during structural analysis is a validation of output results from the numerical model. In this regard following criteria can be established:

- The Similarity of deformations in model with the deformations actually present
- Connection deformations or gaps (only measurable to a limited extent)
- Qualitatively comprehensible cutting forces and deformations
- Unrealistically high degrees of utilisation are not present

In spite of all analyses and measurements, the actual mechanical behaviour and thus the existing safety margin of historical roof structures frequently only with considerable uncertainties can be determined. [16]

5-2.1.1 Simplifications and their influence on models

Next, most common simplifications made by an engineer during the making of models will be introduced, together with their impact upon calculation results.

- **Spatial load transfer- Large influence on results**

Frequently, historical roof structures carry the loads spatially (like principal rafter roof with collar plate, and purlin roofs). If these structures are regular along their length the spatial influence can be neglected and structure can be easily decoupled in 2D subsystems (principal/common truss). For example, in German stehender Stuhl, longitudinal Stuhl frame for symmetric load cases doesn't carry any load. In other words, for the case of symmetric loading, the spatial load transfer is not developed. For more explanation about this phenomenon refer to Holzer [25], [62].

In other cases, irregular structures have pronounced spatial transfer of loads around irregularities (e.g. dormers, chimneys changes in roof geometry) and decoupling is hard to impossible as boundary conditions are unknown and hard to obtain. The decoupling, in this case, should be avoided because obtained results are ambiguous and global behaviour is usually incorrect. The 3D model is necessary. (cf. [16] [32])

This point emphasises the need for specialized knowledge in timber roof structure behaviour in order to recognise global force path system to decouple spatial model into plane models, as the decoupling of spatial structure to plane subsystems can significantly reduce compute and modelling time.

- **Connection stiffness- Large influence on results**

  In numerous literature joints and supports are presented as a critical part of the structure where any oversimplifications regarding stiffnesses and eccentricities of joints result with significantly different internal forces in numerical models. The traditional approach where joints are modelled as hinged produces too conservative results, significantly reducing safety factors and marking the structure as critical when in fact the structure is safe. To avoid this Holzer [25] suggest modelling the joints with spring elements. In this attempt it is not always necessary to calculate the stiffness according to proposed methods such as component method see Chapter 5-2.2.1, but the approximation of the stiffnesses according to the Figure 5.6 will prove sufficient in most cases even for more complex structures. By avoiding the calculation of stiffnesses time consumption is greatly reduced while obtaining better results in comparison to traditional approach.

Approach to modelling by importing linear, bilinear and trilinear load-deflection diagrams for specific joints is also possible if such kind of diagrams supported by research exists. The connections are usually subjected to large compression stresses where bilinear diagrams or simple ideal plastic model of the connection could significantly improve reality of the numerical model, as the plastic behaviour of wood under compression at an angle is present. (cf. [25])

Moreover, for connections which prove to be critical from the standpoint of load bearing capacity more complex calculations of stress distributions are desirable. Volume FEM models prove to be a good asset for this kind of analysis, refer to [17] and master thesis of Kirchler [21].

- **Member nonlinearities-(unknown) influence on results**

  Member nonlinearities include: failure of a member to tension or compression, gaps between members, failed connections and members. Usually refers to the members that have the connections which can't transfer tension forces, therefore these members must be recognised during assessment and survey. For first models, these nonlinearities can be ignored if members during calculation result with tension forces evidence of these failures should be present on the structure, as it can be assumed that peak loads already occurred in the historic structures over a long period of its exploitation. In complex models failures should be considered as an alternative loading path is obtained in structure with high robustness, often causing changes in global behaviour and force redistribution. [17]

- **Battens- Small influence on results**

In the doctoral thesis of Andreas Meisel [17], the influence of timber battens on load distribution and member utilization is discussed. Research has shown that considering battens and their bending stiffness can lead to a decrease in member utilization and to the improvement of the global behaviour of the structure and rafter plane. Moreover, nails in connection of battens to rafters give torsional rigidity to the rafter plane. By modelling these torsional stiffnesses in connection of battens to rafters the battens improve stability of the structure, (similar influence as wind bracing). For more information refer to. [17]

- **Material characteristics- Medium influence on results**

A reliable way of determining exact material properties in historic timber structures without destructive test is not possible, therefore visual grading is usually sufficient. In case when members show insufficient safety factor but no damage or signs of overloading to the element is observed during the survey, usually problem lies in wrongly estimated material properties, therefore model should be updated. [17]

- **System and cross section measures- Medium influence on results**

Tolerances given in Chapter 3-1 are considered to restrict models mistakes roughly up to 10% therefore by following these rough tolerances, influences on results are kept at a reasonable scale. Example of the influence of the cross section measures on the cross section properties is shown in Table 5.7.

Table 5.7 Influence of tolerance of cross section measures on cross section properties

Nr.	Structural element	Width b [cm]	Height h [cm]	Cross section area A [cm^2]		Iy [cm^4]		Wy	
1	Tie beam	17	23,5	400	**97,92%**	18385	**93,88%**	1565	**95,88%**
1	Tie beam	17	24,0	408	**100,00%**	19584	**100,00%**	1632	**100,00%**
1	Tie beam	17	24,5	417	**102,08%**	20834	**106,38%**	1701	**104,21%**

The maximal difference for this example is 6,38%, but the difference can raise up to approximately 11% depending on cross section height. Following these differences, the difference in calculation approximately raises to a margin of 10%.

Moreover, cracks in timber cross section can have an impact upon cross section properties and results as emphasised in Chapter1-4.3. Although impact upon bending stiffens can be neglected, impact on torsional stiffness must be considered as cracks can reduce torsional stiffness up to 60%. the expressions for calculation of torsional stiffness for a cracked and uncracked cross section are shown in Figure 5.3 down. As differences are large the reduction of torsional stiffness should be considered for purlins which are subjected to high intensity of torsion from rafters. [17]

Cracks influence the shear distribution in a way that shear plane is reduced, see Figure 5.3 up. This effect is already well known and taken into account in Eurocode 1995-1-1-2015 [59] with factor k_{cr}=0,67

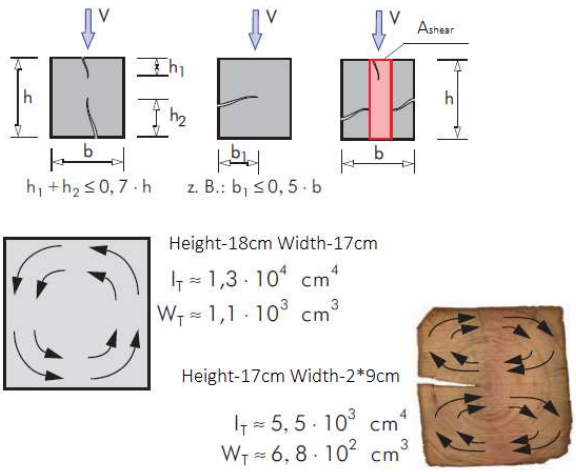


Figure 5.3 Influence of cracks on shear distribution and torsional stiffness [17]

- **Support conditions- Large influence on results**

Support present places where the principal and common trusses rest on structural walls. The traditional modelling of supports as hinged, sliding or rigid should be avoided at all cost, except for the first simple models, as they cause unrealistic behaviour of the analysed roof structure and subsequently incorrect force distribution. Because of that, it is advisable to model supports with springs which partially constrain support movement. Therefore, boundary conditions of supports must be carefully determined to obtain a realistic numerical model. Boundary conditions should be determined during the surveying process and then implemented in the model. Supports in roof structures usually can't receive tension support reactions. Only in the case when supports are stationed in wall pockets with enough wall height above the pocket the tension forces in supports can be transferred, and support can be regarded as rigid in that direction. (cf. [20])

Horizontal forces are transferred by friction. As given in several papers friction coefficient of 0,4 can be used [76], [20], [47]. Nonetheless modelling horizontal restraint in supports with friction often causes large horizontal forces on structural walls and especially attic walls. It often happens that masonry walls don't have enough load bearing capacity for the forces out of the plane, therefore masonry walls crack, and horizontal trust forces decline, as consequence load in roof structure redistribute in a way that roof carries the loads more as a purlin roof than a rafter roof. Therefore, using friction as a horizontal restraint is suitable if obtained horizontal forces on the walls don't exceed the calculated load bearing capacity of the wall. (cf. [20])

Because of the problems with the load bearing capacity of the masonry walls, in research Holz-Holz Verbindungen [47] it is recommended to model the supports as spring supports, where for horizontal restrains spring is introduced. The calculation of spring stiffness is based on the iteration process for the load combination of self-weight + additional deadweight. In this approach, the spring stiffness is altered until force in the tie beam drops to 97% of the tension force when spring stiffness is 0. In that way, walls take around 3% of the tension force in the tie beam. This approach usually results with spring stiffnesses in range of 250-500kN/m. This approach proves to be more suitable as the load bearing capacity of masonry walls is rarely compromised with this approach. (cf. [20] [21] [47])

To approve calculated spring stiffness, the resulting horizontal movement of the supports must be smaller than the value obtained with expression (5.3), developed in [47]. Also, shouldn't be larger than the value obtained with expression given by author (5.4)

$$u_{x,support}=\frac{b_{support}}{2}-4\ [cm] \quad (5.3)$$

Where:
$u_{x,support}$ – permisable horizontal drift of the support
$b_{support}$ – width of the support contact plane

$$u_{x,support}=b_{wall,pocket}-b_{support}\ [cm] \quad (5.4)$$

Where:
$b_{wall,pocket}$ – debth of the wall pocket

5-2.1.2 Numerical models

After introducing the influences of simplifications, models usually used or ones that can be used in modelling of the historic timber roof structures will be given together with their respective simplifications, advantages and disadvantages. It should be noted that with combining and neglecting different influences a lot of different models can be developed. In work of Meisel [17] several alternations of numerical models are shown, and graphically and numerically compared on examples of different roof structure systems. These models are only explained here, for better quantitative comprehension of differences between given models please refer to Chapter 4 in [17], and for some models also in Chapter 5-4.2 of this thesis.

- **Rod model with eccentricities, connection stiffness and nonlinearities 3D/2D**

  This model considers all influences which have a big impact on results. Advantage of these models is obvious as larger input information such as connection eccentricities and stiffnesses provide more realistic deformation and force distribution. Disadvantages lay, in more complex modelling which can cause secondary mistakes and errors in the model due to the small stiffnesses, and in overall spending more time on the model making. The time consumption can be reduced with decoupling of the model in 2D subsystems if assumptions given in the previous subchapter are fulfilled. [17]

- **Traditional rod model 3D/2D**

The traditional model represents simple rod model which simplifies the modelling of connections, (neglecting stiffnesses and eccentricities), supports, nonlinearities, etc., therefore it is only suitable for the first simple model which can help the engineer to determine global behaviour of the roof structure. As the model is oversimplified the obtained safety factor is too conservative and at the end can cause a devaluation of the historic roof structure. On a good side calculation time and time invested in the making of the model is small and these models can give a good insight into global behaviour of the roof structure. Decoupling from the spatial model to plane subsystems is possible for regular structures. Which makes the reading of the results easier.

- **Combined rod and volume model**

This model is a mix of rod model and volume FEM model. The critical points of roof structure where stress concentrations are present are modelled as volume elements. This mix eventually forms a whole truss where connections are modelled as volume elements and members are modelled as rods. In this approach computational time is reduced in contrast to volume models, although still large enough to prevent this approach to be used in common practice. The improvement can be made by decoupling of structure to planar subsystems and by implementing symmetry. An example of such analysis can be found in the work of Kirchler [21]. (cf. [17])

- **Volume model**

This approach implies solid FEM modelling of the structure. As modelling of the whole structure in this way is impossible due to the high computational time, needed resources and input, the usage is restricted to the small critical parts of the roof structure (usually connections). The advantages are numerous as nonlinearities can be implemented in the model such as anisotropy of wood, modelling of contact surface, friction, realistic behaviour of wood under compression with implementing load displacement diagrams, etc. (cf. [17])

5-2.2 MODELLING OF CONNECTIONS

Connections have a great influence on the behaviour of the structure and force distribution in the structure. The load transfer in traditional connections is obtained by contact surfaces where the load is transferred mainly by compression and friction. As a value of modulus of elasticity perpendicular to the grain is 30 times smaller than along the grain, the stiffnesses in connections differ from adjacent members as loads are usually transferred at an angle to the grain. Furthermore, the local influences caused by geometry are present and they induce additional stresses upon members (eccentricities of contact areas).

For these reasons, the connections can be denoted as a critical part of the structure and special care should be taken during their modelling. In Chapter 5.4 the influence of connections on results is derived by comparing different models in which connections are modelled differently.

To get closer to the realistic influence of the connections the following influences should be assessed during the making of the model.

- **Stiffnesses of the connection (axial and rotational)**

As emphasised the small modulus of elasticity in a direction perpendicular to the grain, results with high deformability of connection in comparison to members. Beside axial stiffness connections have rotational stiffness as they can transfer bending moment up to some extent, depending on the geometry of the connection, contact forces, friction, and gaps.

- **Gaps in connection**

In connections, gaps are often present. They greatly influence structural behaviour. In work of [77] the differences in a horizontal drift of the structure obtained by considering the gap of 1 mm raise up to 466%, in contrast to the case without any gaps, see Figure 5.4. However, to determine gaps in connection is often time consuming and not reliable as we often speak about small gaps that are not visible. Moreover, some software on the market maybe doesn't support modelling of a slippage. For these reasons, author opinion is that gaps shouldn't be considered during modelling. This can be further strengthened by acknowledgement made by Meisel where gaps influenced on deformability of the structure, but no significant internal force distribution was observed. (cf. [17] page 85)

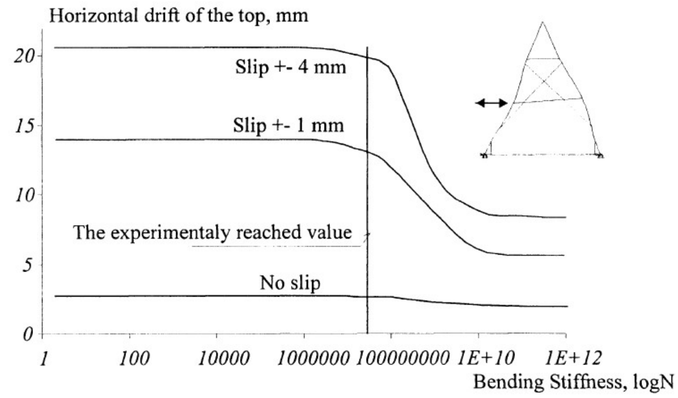


Figure 5.4. Influence of the slip on the structure deformation [77]

- **Connection eccentricities**

The eccentricities in the connections according to Meisel [17] can be a result of various consequences:

- *the geometry of the connections, as the position of the pressure contact surfaces often deviates from the rod axes*
- *of system deformations and resulting angle changes between the construction members*
- *of manufacturing or material-related shrinkage,*

The eccentricities result with a redistribution of internal forces and additional stresses on members.

5-2.2.1 CALCULATION OF THE AXIAL/ROTATIONAL STIFFNESS

The axial and rotational stiffnesses can be calculated according to the component methodology, see Figure 5.5. This approach is also introduced by Meisel [17] [16] and it is also used in this paper.

The spring stiffness is calculated by expression 5.5 and 5.6 This approach considers several simplifications such as:

- Load dispersion at slope of 1:3
- The module of elasticity calculated according to expression (5.7) or (5.8) may be used
- Imperfections neglected

$$\boldsymbol{C_{axial}} = \frac{E_{\propto} * A_m}{H} \tag{5.5}$$

Where:
$E(\propto)$ ....Modul of elasticity at angle α [kN/m^2]
A_m ....Area of middle plane [m^2]
H ....Height of load dispersion zone[m]
$\mathbf{C_{axial}}$........Axial spring stiffness[kN/m]

$$C_{rot}=\frac{M}{\varphi}=\sum_i (C_i*z_i^2)= \tag{5.6}$$

Where:

M .... Bending moment [kNm]
φ .... Rotation angle [rad]
z_i .... Distance from the centre of rotation [m]
C_i .... Axial spring stiffness of the i-th element[kN/m]
$\mathbf{C_{rot}}$ .... Rotational stiffness [kNm/rad]

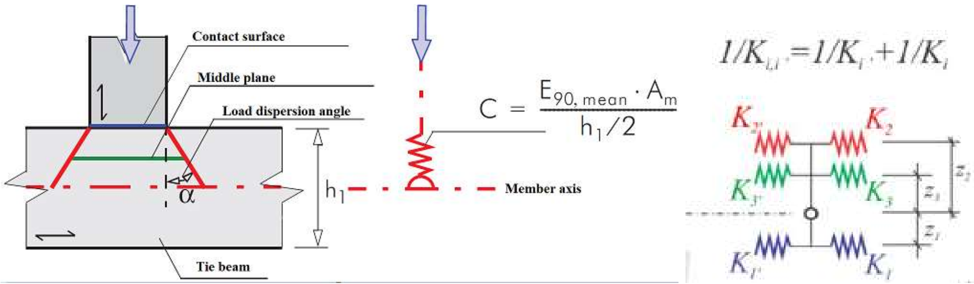


Figure 5.5. left: Simplified load expansion model [17]; right; Example of spring model for calculation of rotational stiffness [30]

$$E_{c,\alpha,mean}=\frac{E_{0,mean}}{\frac{E_{0,mean}}{E_{90,mean}}*\sin\alpha^2+\cos\alpha^2} \tag{5.7}$$

$$E_\alpha=\frac{E_{0,mean}*E_{90}}{E_{0,mean}*\sin\alpha^3+E_{90}*\cos\alpha^3} \tag{5.8}$$

The problems of determination of axial stiffnesses are covered well by literature as numerous papers deal with this kind of problem on different types of connections. However, for the case of rotational stiffness the literature gives less information as the determination of these values is more complex. To obtain more correct models for the assumption of semi rigid behaviour of traditional connections, more researches in this domain should be conducted. Furthermore, the load tests should be conducted where results can be statistically approached.

The valid ONORM DIN 1995-1-1 [78] doesn't provide the approach or expressions for verification of traditional joints, this represents a problem as no standardized approach is recognized. An exception to the rule is DIN-1052-2004 [79] norm where some expressions are given. Moreover, there is no expression for determination of material strength for the stress combinations which can be usually found in connections.

To answer these problems, the connection can be modelled as volume FEM model, where besides the geometry, the contact planes with its respective properties are modelled and timber characteristics such as anisotropy can be taken into consideration.

For purposes of fast calculation, it is possible to use the stiffnesses given by Meisel in Figure 5.6. The stiffnesses given for these connections are a result of an overview of numerous literature regarding this topic. [17]

More literature about modelling of connections gave Meisel in the PhD dissertation on page 16. [17]

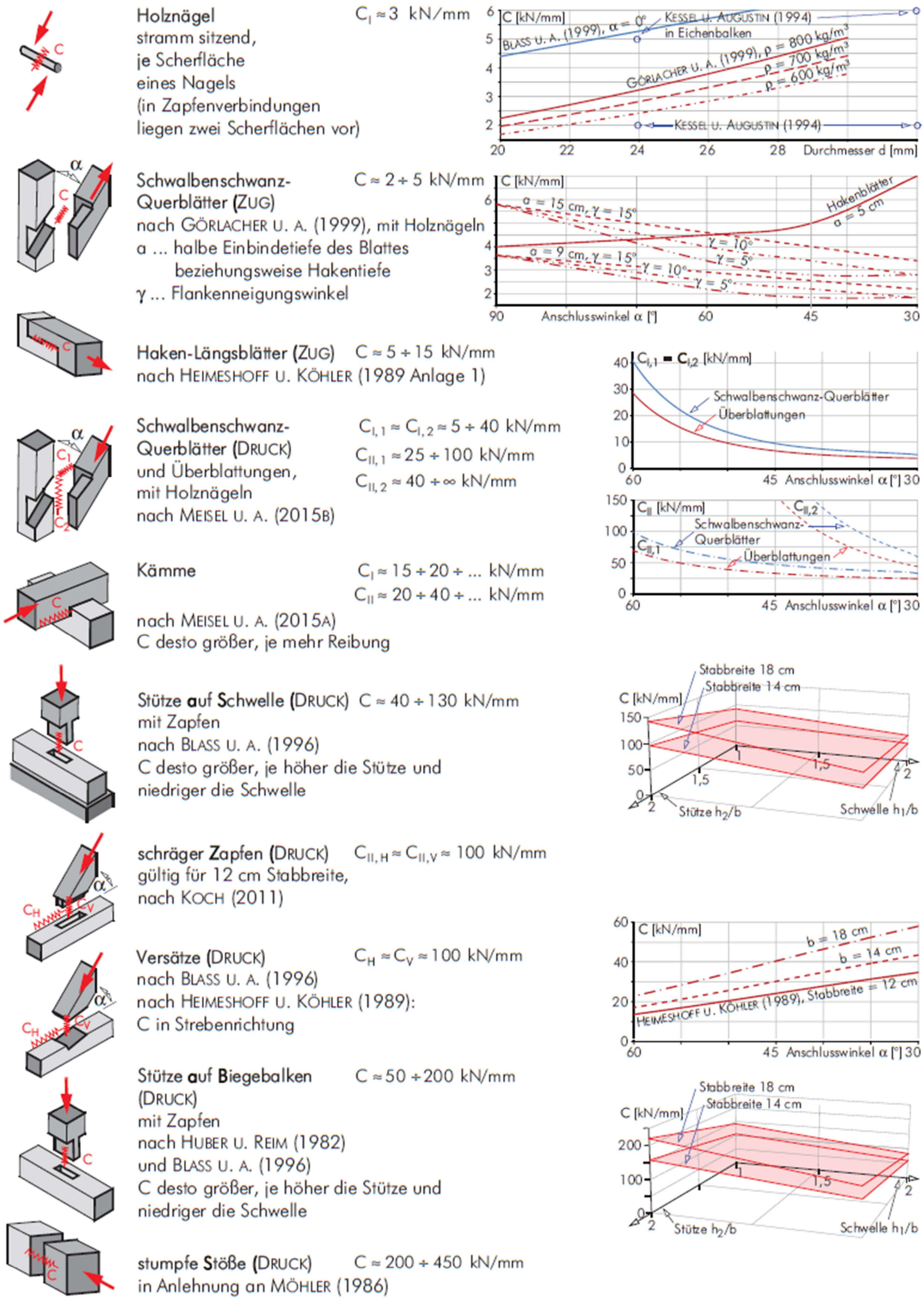


Figure 5.6. Magnitudes of the spring stiffnesses of carpentry connections [17]

5-3 NUMERICAL MODEL

5-3.1 MODELLING OF THE ROOF STRUCTURE "*TU GRAZ ALTE TECHNIK"*

The roof structure is decoupled in 3 models. Each model represents one room. The divide can be seen in Figure 2.5. This divide is realistic as roof structure is discontinued by firewalls. The models are made in software *Dlubal RFEM 5.14.03*. All members are modelled as rod elements and calculation is made according to the first order theory.

In order to present the influences of different approaches to numerical modelling 4 models in room 2 are developed. In that way, the influence of boundary conditions is clearly presented. The developed models are schematically presented in Figure 5.7.

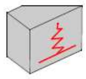
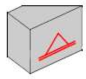
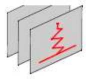
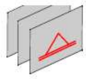
Models	3D		2D	
	M1	M2	M3	M4
Symbol				

Figure 5.7. Models developed for Room 2

Model M1

The Model M1 Figure 5.8, represents the most realistic approach to the modelling from all other considered models. In this spatial model special care is oriented to the modelling of connections and supports. In connections eccentricities and stiffnesses are considered. The eccentricities are modelled as rigid members. The contact surfaces are simulated as hinges on rigid elements with axial and rotational stiffnesses. The calculation of these stiffnesses is presented in Chapter 5-3-4. All supports are modelled as spring supports according with recommendations given in Chapter 5-2.1.1. As a result of the calculations, horizontal spring stiffness of 500kN/m is adopted for support of a tie beam on a structural masonry wall, and 350kN/m for support of a foot purlin on the attic wall. Supports of foot purlins are placed on places where rafters are connected with a foot purlin. Furthermore, geometrical nonlinearities in members are considered (e.g. failure of braces and straining beams to the tension forces).

Model M2

Model M2 is also a spatial model. However, this model represents the "stiffer" alternative to model M1 as no connection eccentricities, stiffnesses, and nonlinearities are considered. The tie beam is supported on masonry as a simple supported beam, while sliding supports are used to simulate contact between the foot purlin and masonry. This oversimplification in modelling of supports plays great role in the load distribution, and on overall results.

Model M3

Model M3 is a 2D model obtained by decoupling of model M1 in subsystems. The decoupling is made on the place where no influences from spatial load transfer of the dormer stationed in room 2 are present. Again, the stiffnesses and eccentricities of the connections and nonlinearities of members are considered. The decoupling is made for principle truss and also for common truss. For system of the secondary truss, the influence of middle and foot purlin is simulated with spring supports. Decoupled principal and secondary truss are shown in Annex C-2.4.

Model M4

This model represents the most conservative and simplest approach to the modelling of the roof structure. The 2D model is obtained by decoupling of Model M2. In that process, all nonlinearities, eccentricities or stiffnesses are neglected. Decoupled principal and secondary truss are shown in Annex C-2.5.

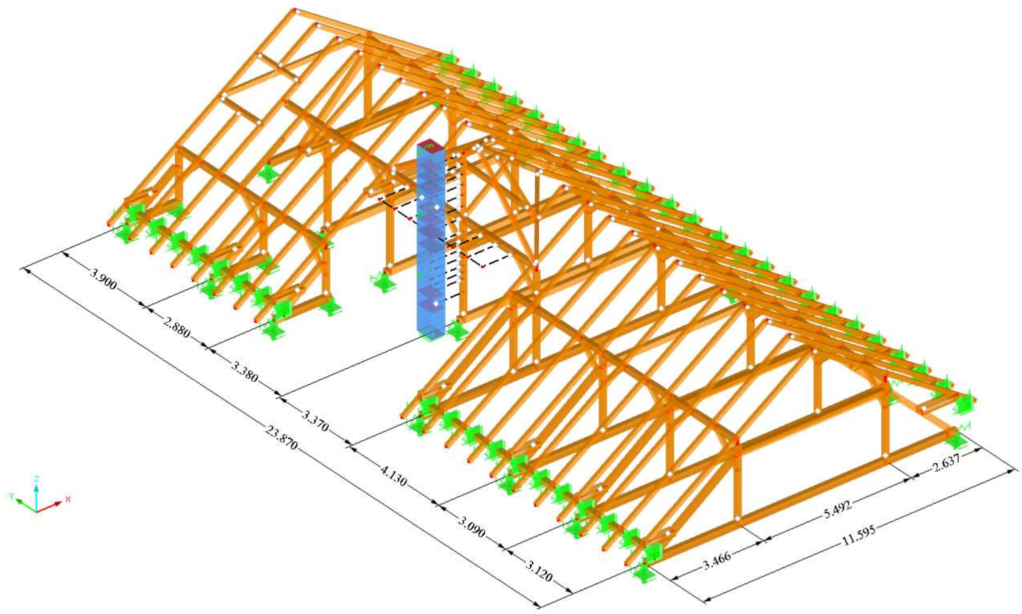


Figure 5.8. Spatial view with axial dimensions on Model M1 in room 2

In room 1 and room 3 only Model M2 is developed to keep the extent of the work in reasonable boundaries. The spatial view on the models in room 1 and room 3 is shown in Figure 5.9 and Figure 5.10.

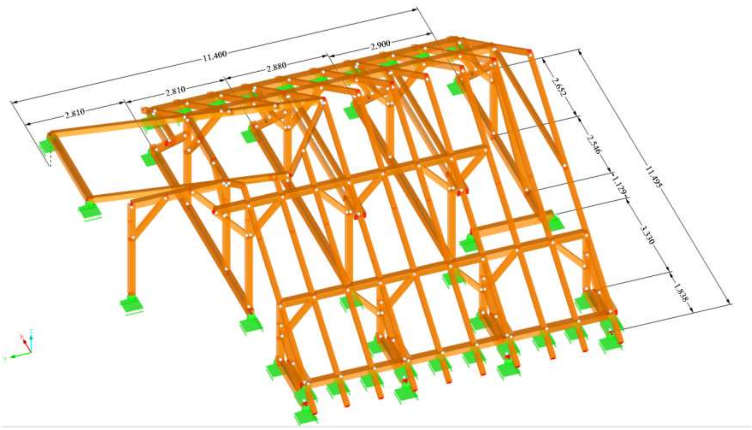


Figure 5.9. Spatial view with axial dimensions on Model M2 in room 1

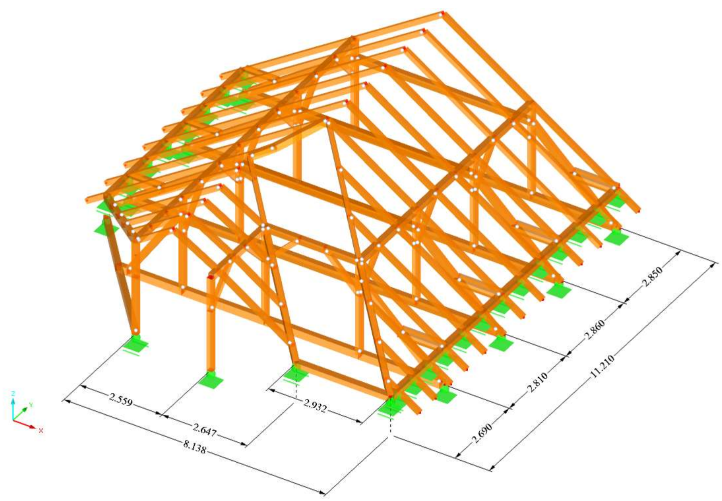


Figure 5.10. Spatial view with axial dimensions on Model M2 in room 3

5-3.2 CROSS SECTION PROPERTIES

The dimensions of the cross sections for modelling are determined on the base of statistical calculations. The dimensions from elements on each axis are documented. For collected data statistical "Mod" value is calculated. The resulting dimensions are shown in Table 5.8 and Table 5.9.

Table 5.8 Cross section properties of Queen post truss

	Queen post- Dimensions and cross section properties									
Nr.	Structural element	Width b [cm]	Height h [cm]	Cross section area A [cm^2]	I_y [cm^4]	W_y [cm^3]	i_y [cm]	I_x [cm^4]	W_x [cm^3]	i_x [cm]
1	Tie beam	17	24	408	19584	1632	6,93	9826	1156	4,91
2	Queen post	17	17	289	6960	819	4,91	6960	819	4,91
3	Brace	17	21	357	13120	1250	6,06	8598	1012	4,91
4	Tie	10	18	180	4860	540	5,20	1500	300	2,89
5	Straining beam	17	20	340	11333	1133	5,77	8188	963	4,91
6	Foot purlin	21	17	357	8598	1012	4,91	13120	1250	6,06
7	Middle purlin	17	20	340	11333	1133	5,77	8188	963	4,91
8	Knee brace	13	15	195	3656	488	4,33	2746	423	3,75
9	Rafter	13	15	195	3656	488	4,33	2746	423	3,75

Table 5.9 Cross section properties of King post truss

	King post- Dimensions and cross section properties									
Nr.	Structural element	Width b [cm]	Height h [cm]	Cross section area A [cm^2]	I_y [cm^4]	W_y [cm^3]	i_y [cm]	I_x[cm^4]	W_x [cm^3]	i_x [cm]
1	Tie beam	17	24	408	19584	1632	6,93	9826	1156	4,91
2	King post	16	18	288	7776	864	5,20	6144	768	4,62
3	Brace	16	18	288	7776	864	5,20	6144	768	4,62
4	Tie	10	18	180	4860	540	5,20	1500	300	2,89
5	Collar tie	10	18	180	4860	540	5,20	1500	300	2,89

5-3.3 MATERIAL PROPERTIES

The material properties are given in Table 5.10 according to the norm EN 338 [67], for strength classes which are determined by visual grading.

The design value of a strength property shall be calculated according to expression (5.9). The expression is referred in ONORM 1995-1-1-2009 [59] as expression (2.14):

$$X_d = k_{mod} \cdot \frac{X_k}{\gamma_M} \qquad (5.9)$$

$\gamma_M = 1{,}3$

Table 5.10 Material properties

Material properties			
Strength Properties [N/mm2]		Strength class	
		C24	C30
Bending strength	$f_{m,k}$	24	30
Tension // to grain	$f_{t,0,k}$	14,5	19
Tension ⊥ to grain	$f_{t,90,k}$	0,4	0,4
Compression // to grain	$f_{c,0,k}$	21	24
Compression ⊥ to grain	$f_{c,90,k}$	2,5	2,7
Shear	$f_{v,k}$	2,3	2,3
Stiffness properties [N/mm2]			
Mean modulus of elasticity // to grain	$E_{0,mean}$	11000	12000
Characteristic modulus of elasticity // to grain	$E_{0,05}$	7400	8000
Mean modulus of elasticity ⊥ to grain	$E_{90,mean}$	370	400
Mean shear modulus	G_{mean}	690	750
Density [kg/m3]			
Characteristic density	ρ_k	350	380
Mean density	ρ_{mean}	420	460

Coefficients k_{mod} and k_{def} are used for modification of strength and stiffness to account for changing properties of timber during the time. It is influenced by the duration of a specific load case. In combination where two load cases of different duration are presented, k_{mod} is used for a load of shorter duration. Coefficient k_{def} is depending on exposure class as given in [59]. Roof structure at TU Graz is classified as 1st. exposure class. Coefficients k_{mod} and k_{def} used in roof structure TU Graz are shown in Table 5.11

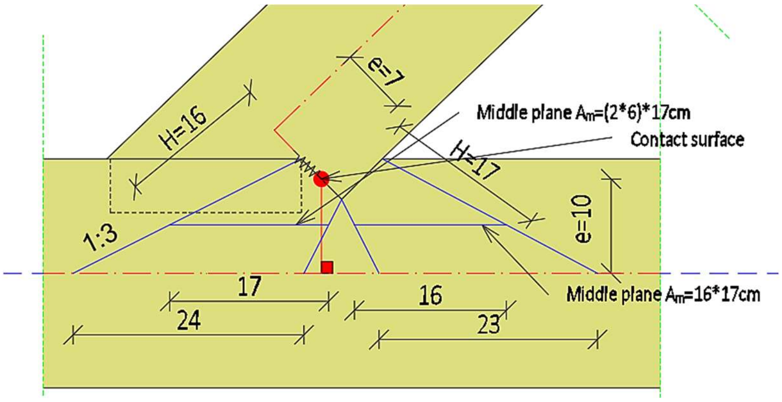
Table 5.11. Modification coefficients for timber k_{mod} and k_{def}

Load	Duration	k_{mod}	k_{def}
Self and dead weight	Permanent	0,6	0,6
Wind	Very short	1,0	0,6
Snow	Short	0,9	0,6

5-3.4 CALCULATION OF CONNECTION STIFFNESSES

Calculation of the node stiffnesses according to mentioned expressions is given. At the end in Figure 5.11 overview of adopted stiffness used in model M1 are given.

Brace to tie beam



E_0=1100,0 kN/cm^2

E_{90}=37,0 kN/cm^2

φ=45°

E_φ=101,0 kN/cm^2

H_x=17,0 cm

H_z=16,0 cm

h_x=16,0 cm

b_x=17,0 cm

h_z=16,0 cm

b_z=2*6,0 cm

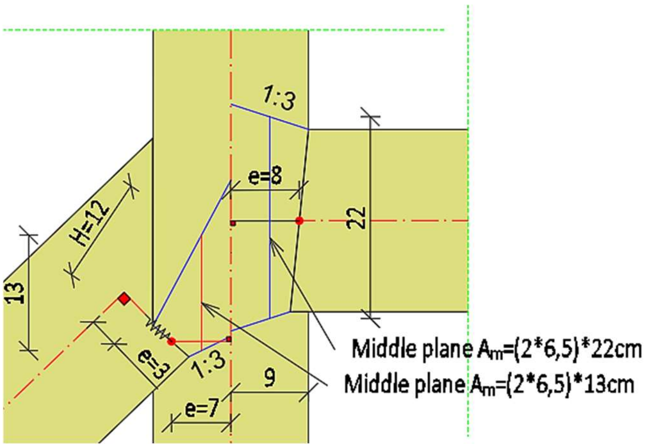
$A_x=b_x \cdot h_x=17{,}0 \cdot 16{,}0=272{,}0$ cm^2

$A_z=b_z \cdot h_z=2*6{,}0 \cdot 16{,}0=192{,}0$cm^2

$C_x=\frac{E_\varphi \cdot A_x}{H_x}=162000$ kN/m

$C_z=\frac{E_\varphi \cdot A_z}{H_z}=121500$ kN/m

Brace to Queen post



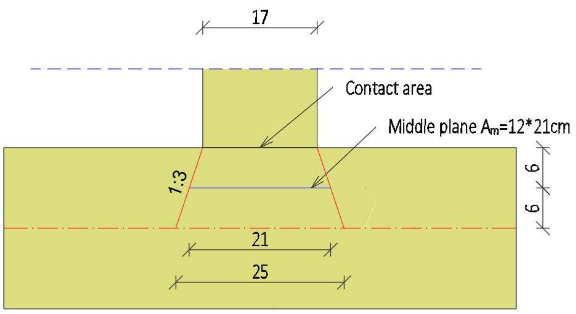
E_0=1100,0 kN/cm^2
E_{90}=37,0 kN/cm^2
φ=45°
E_φ=101,0 kN/cm^2
H_z=12,0 cm
h_z=13,0 cm
b_z=2*6,5=13,0 cm
$A_z=b_z \cdot h_z$=13,0*13,0=169,0cm^2
$C_z=\frac{E_\varphi \cdot A_z}{H_z}$=131300 kN/m
$C_x=\frac{E_\varphi \cdot A_x}{H_x}$=162000 kN/m … same as Brace to Tie beam

Straining beam to Queen post

E_0=1100,0 kN/cm^2
E_{90}=37,0 kN/cm^2
φ=90°
E_φ=37,0 kN/cm^2
H_x=8,5 cm
H_z=18,0 cm
h_x=22,0 cm
b_x=2*6,5=13,0 cm
h_z=17,0 cm
b_z=2,0 cm
$A_x=b_x \cdot h_x$=13,0*22,0=286,0 cm^2
$A_z=b_z \cdot h_z$=2,0*17,0=34,0cm^2
$C_x=\frac{E_\varphi \cdot A_x}{H_x}$=125000 kN/m
$C_z=\frac{E_\varphi \cdot A_z}{H_z}$=7000 kN/m

Queen post to Tie beam

- **Compression**



E_0=1100 kN/cm^2
E_{90}=37 kN/cm^2
φ=90°

E_φ=37 kN/cm^2

H_x=12 cm

H_z=8,5 cm

h_x=21 cm

b_x=12 cm

h_z=5 cm

b_z=4 cm

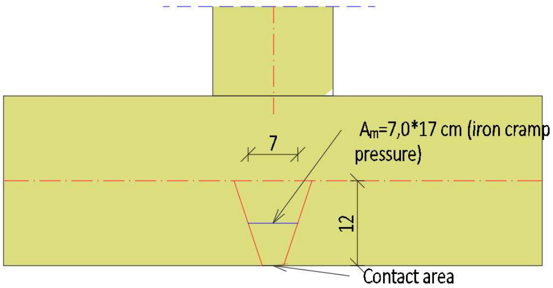
$A_x=b_x \cdot h_x=12 \cdot 21=252$ cm^2

$A_z=b_z \cdot h_z=4 \cdot 5=20$cm^2

$C_x=\frac{E_\varphi \cdot A_x}{H_x}=77700$ kN/m

$C_z=\frac{E_\varphi \cdot A_z}{H_z}=8705$ kN/m

- **Tension (timber in compression from U strap)**



E_0=1100 kN/cm^2

E_{90}=37 kN/cm^2

φ=90°

E_φ=37 kN/cm^2

H_x=12 cm

h_x=7cm

b_x=17 cm

$A_x=b_x \cdot h_x=7 \cdot 17=119$ cm^2

$C_x=\frac{E_\varphi \cdot A_x}{H_x}=36691$ kN/m

- **Iron cramp**

L= 50 cm

$A_{cramp}=3 \cdot 0,7=2,1$ cm^2

E_{steel}=21000 kN/cm^2

$C_{cramps}=\frac{2 \cdot E \cdot A}{L}=176400$ kN/m

- **Bolt**

ρ_m=420 kg/m^3

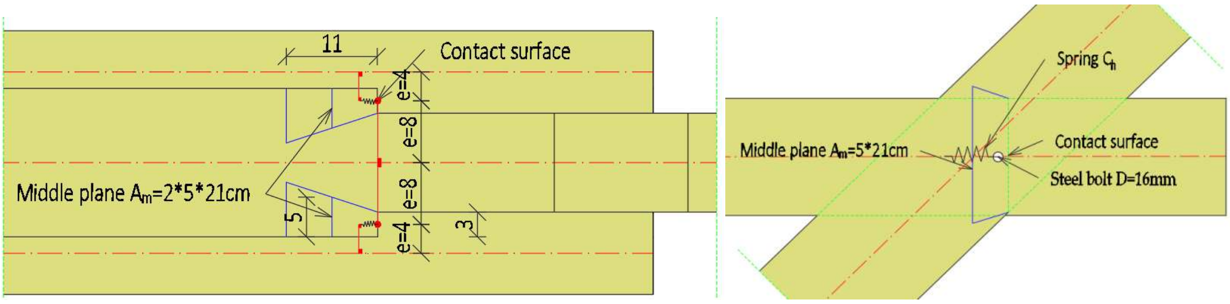
d=16 mm

$C_{bolt}=\frac{2\cdot \rho_m^{1.5}\cdot d}{23}=12000\ kN/\ m$

Total stiffness of joint in tension (serial connection)

$$\frac{1}{C_{total}}=\frac{1}{C_{strap}}+\frac{I}{C_{bolt}}+\frac{1}{C_{pressure}}=8000\ kN/\ m$$

Ties to brace



E_0=1100,0 kN/cm^2

E_{90}=37,0 kN/cm^2

φ=45°

E_φ=101,0 kN/cm^2

H_x=11,0 cm

h_x=21,0 cm

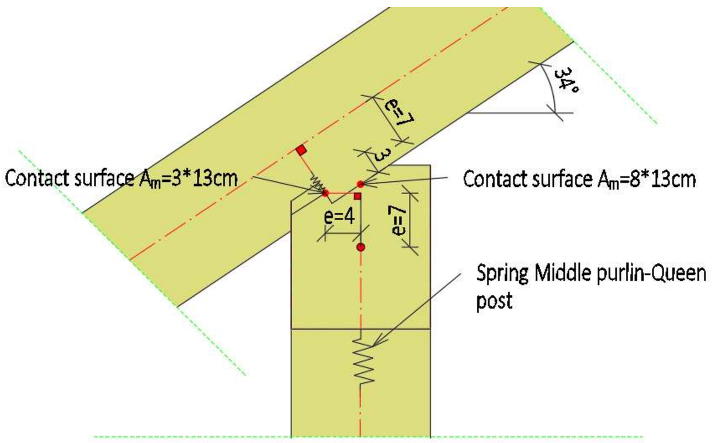
b_x= 2*5,0 cm

$A_x=b_x\cdot h_x$=2*5,0*21,0=210,0 cm^2

$C_x=\frac{E_\varphi\cdot A_x}{H_x}$=193000 kN/m

Bolt is neglected

Rafter to middle purlin



E_0=1100,0 kN/cm^2

E_{90}=37,0 kN/cm^2

φ=90°

E_{φ}=37,0 kN/cm^2

H_x=7,0 cm

H_z=5,0 cm

h_x=13,0 cm

b_x= 8,0 cm

h_z=13,0 cm

b_z= 3,0 cm

$A_x = b_x \cdot h_x$=8,0*13,0*=104,0 cm^2

$A_z = b_z \cdot h_z$=3,0*13,0=39,0cm^2

$C_x = \frac{E_{\varphi} \cdot A_x}{H_x}$=55000 kN/m

$C_z = \frac{E_{\varphi} \cdot A_z}{H_z}$=28900 kN/m

Middle purlin Queen post

E_{90}=37,0 kN/cm^2

$h_x = 17{,}0 + (2 * \frac{1}{3} * 10) = 24$ cm

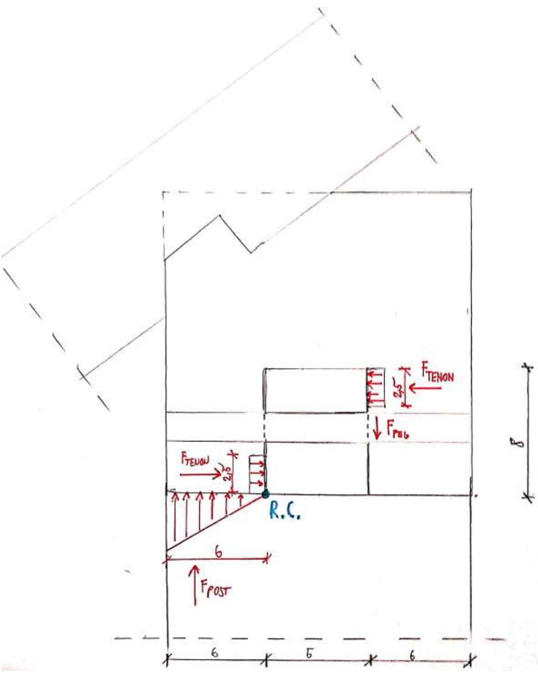
b_x= 2*6,0 cm

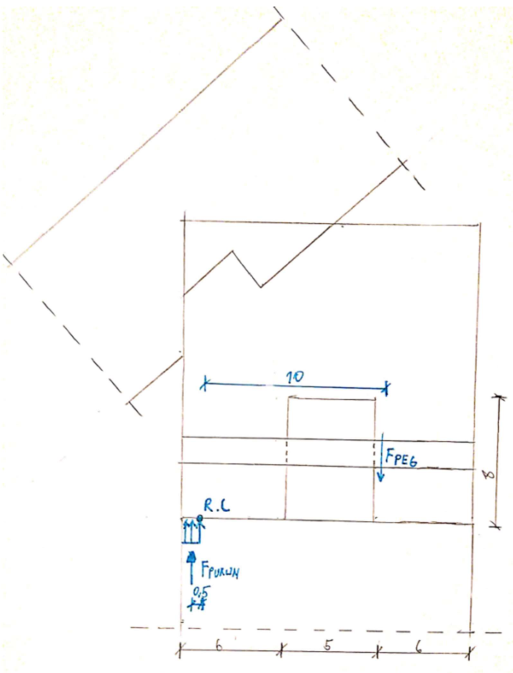
H_x=10,0 cm

$A_x = b_x \cdot h_x$=12,0*24,0=288,0 cm^2

$C_x = \frac{E_{\varphi} \cdot A_x}{H_x}$=106500 kN/m

Middle purlin to Queen post (rotational stiffness)





E_0=1100,0 kN/cm^2
E_{90}=37,0 kN/cm^2
φ=90°
E_φ=37,0 kN/cm^2

H_x=10,0 cm
H_z=2,5 cm
H_y=3,0 cm
$h_x=17{,}0+(2*\frac{1}{3}*5)\approx 19{,}0$ cm
b_x= 6,0 cm
h_y=17,0 cm
b_y=2,5 cm
h_z=17,0 cm
b_z=2,5 cm

$A_x=b_x\cdot h_x=6{,}0*19{,}0=114{,}0$ cm^2
$A_y=b_y\cdot h_y=2{,}5*17{,}0=42{,}5$ cm^2
$A_z=b_z\cdot h_z=2{,}5*17{,}0=42{,}5$cm^2

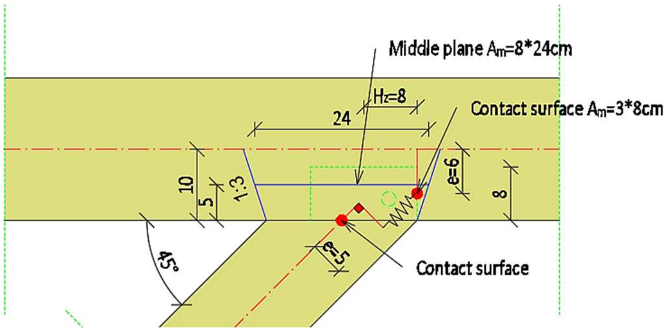
z_{purlin}=4,0 cm
$z_{tenon,bottom}$=1,25 cm
$z_{tenon,upper}$=6,75 cm
z_{peg}=5,0 cm
$C_{tenon,purlin}=\frac{E_\varphi\cdot A_y}{H_y}=52500$ kN/m
$C_{tenon}=\frac{E_\varphi\cdot A_z}{H_z}$=62900 kN/m
$C_{tenon,total}=\frac{1}{C_{tenon}}+\frac{1}{C_{purlin,tenon}}\approx 29000$ kN/m
$C_{purlin}=\frac{E_\varphi\cdot A_x}{H_x}\approx 42180$ kN/m
$C_{peg}\approx 3000$kN/m
$\varphi=\frac{\delta_1}{z_1}$
$\delta_1=z_1*\varphi$
$F_i=C_i*\delta_i$
$M=\sum_i(F_i*z_i^2)=\sum_i(C_i*\delta_i*z_i^2)$
$C_{rot}=\frac{M}{\varphi}=\sum_i(C_i*z_i^2)=[29000*(0{,}0125^2+0{,}0675^2)]+(42200*0{,}04^2)+(3000*0{,}05^2)\approx 211$ kNm/rad

Alternative – only wooden peg is active

$C_{rot}=\frac{M}{\varphi}=\sum_i(C_i*z_i^2)=(42180*0{,}005^2)+(3000*0{,}1^2)\approx 31{,}0$ kNm/rad

According to Meisel [17] torsional stiffness of the tenon and mortise connections shouldn't be larger than circa15,0 kNm/rad, therefore the value of 31,0 kNm/rad is taken in this paper. Reason for approval of 100% bigger rotational stiffness than value given by Meisel lays in bigger stiffness of wooden pegs, and geometry.

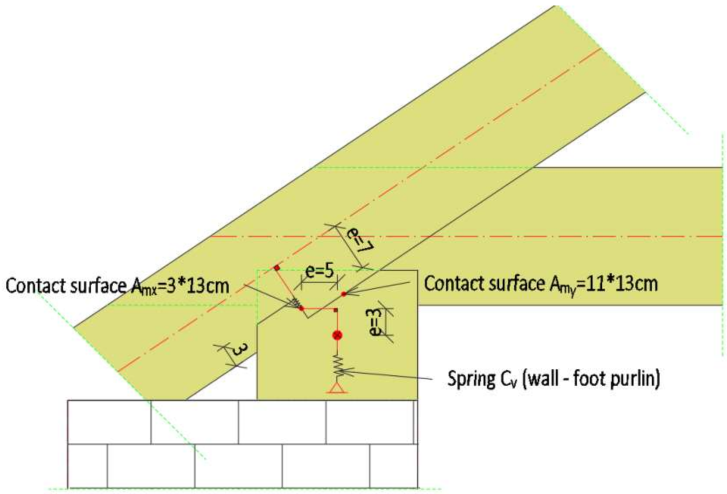
Knee brace to Middle purlin and Queen post



E_0=1100,0 kN/cm^2
E_{90}=37,0 kN/cm^2
φ=90°
E_φ=37,0 kN/cm^2
H_x=5,0 cm
H_z=8,0 cm
h_x=24,0 cm
b_x= 8,0 cm
h_z=8,0 cm
b_z= 3,0 cm

$A_x=b_x \cdot h_x$=8*24,0=192,0 cm^2
$A_z=b_z \cdot h_z$=3,0*8,0= 24,0 cm^2
$C_x=\frac{E_\varphi \cdot A_x}{H_x}$=71000 kN/m
$C_z=\frac{E_\varphi \cdot A_z}{H_z}$=11100 kN/m

Rafters to Foot purlin



E_0=1100,0 kN/cm^2
E_{90}=37,0 kN/cm^2
φ=90°

E_{φ}=37,0 kN/cm^2
H_x=5,0 cm
H_z=5,0 cm
h_x=11,0 cm
b_x= 13,0 cm
h_z=3,0 cm
b_z=13,0 cm

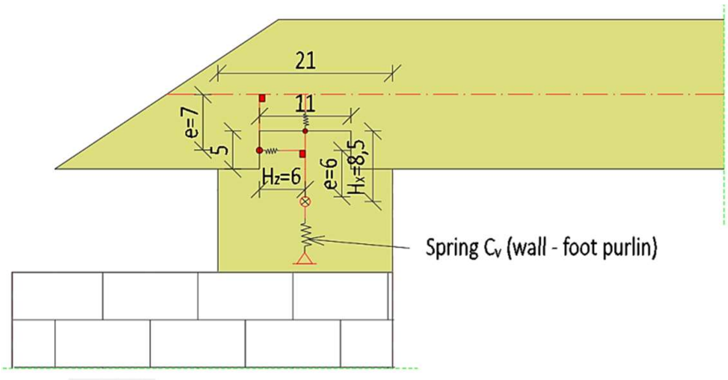
$A_x=b_x \cdot h_x$=11,0*13,0=143,0 cm^2
$A_z=b_z \cdot h_z$=3,0*13,0=39,0cm^2
$C_x=\frac{E_{\varphi} \cdot A_x}{H_x}$=105800 kN/m
$C_z=\frac{E_{\varphi} \cdot A_z}{H_z}$=28900 kN/m

Rafter to Ties

- **Bolt**

ρ_m=420 kg/m^3
d=1,6cm
$C_{bolt}=\frac{2 \cdot \rho_m{}^{1.5} \cdot d}{23}$=12000 kN/ m

Ties to Foot purlin

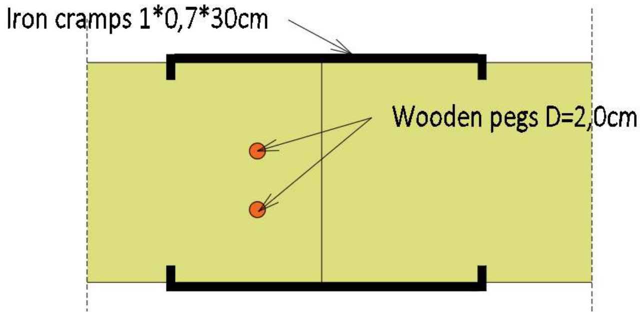


E_0=1100,0 kN/cm^2
E_{90}=37,0 kN/cm^2
φ=90°
E_{φ}=37,0 kN/cm^2
H_x=8,5 cm
H_z=6,0 cm
h_x=21,0 cm
b_x= 2*10,0=20,0 cm
h_z=5,0 cm
b_z=2*10,0=20,0 cm
$A_x=b_x \cdot h_x$=20,0*21=420,0 cm^2
$A_z=b_z \cdot h_z$=20,0*5,0=100,0cm^2

$C_x = \frac{E_\varphi \cdot A_x}{H_x} = 182000$ kN/m

$C_z = \frac{E_\varphi \cdot A_z}{H_z} = 62000$ kN/m

Foot purlin to Foot purlin



- **Wooden pegs**

$C_{peg} \approx 3000$kN/m

- **Iron cramps**

$A_{strap} = 3 \cdot 0{,}7 = 2{,}1$ cm^2

$E_{steel} = 21000$ kN/cm^2

$b = 1{,}0cm$

$h = 0{,}7cm$

$L = 20{,}0(30{,}0)cm$ ….. The cramp bends along 2/3 of the complete length

$I = \frac{b*h^3}{12} = 0{,}03cm^4$

$C_{strap} = \frac{12 \cdot E \cdot I}{L^3} \approx 100$ kN/m

$C_{total} = 2*(C_{peg} + C_{strap}) = 6200$kN/m

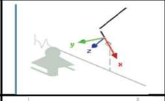
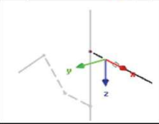
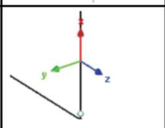
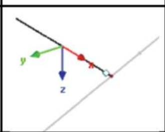
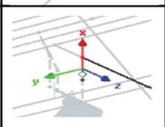
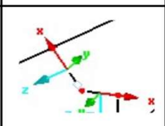
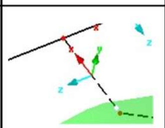
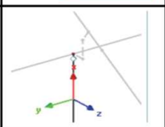
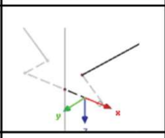
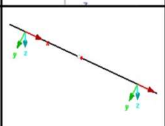
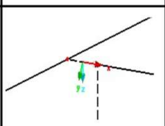
Spring stifness							
Conecction		Cx [kN/m]	Cy[kN/m]	Cz[kN/m]	φx[kNm/m]	φy[kNm/m]	φz[kNm/m]
Brace to Tie beam		162000	Fixed	121500	Free	Free	Fixed
Brace to Queen post		162000	Fixed	131000	Free	Free	Fixed
Straining beam to Queen post		125000	Fixed	7000	Fixed	Free	Free
Queen post to Tie beam		C=77700 T=8000	Fixed	8700	50	Free	Free
Tie to Brace		193000	Fixed	Fixed	Fixed	Free	Free
Tie to Foot purlin		182000	Fixed	62000	Free	Free	Free
Rafters to Middle purlin		55000	Fixed	29000	Fixed	Free	Free
Rafters to Foot purlin		106000	Fixed	29000	Fixed	Free	Free
Middle purlin to Queen post		107000	Fixed	Fixed	31	31	Free
Knee brace to Middle purlin / Queen post		C=71000 T=3000	Fixed	11100	Free	Free	15
Foot purlin to Foot purlin		Fixed	6200	Fixed	Fixed	Free	Free
Rafters to Tie		12000	Fixed	Fixed	Fixed	Free	Free

Figure 5.11. Overview of axial and rotational stiffnesses in connections

5-4 RESULTS COMPARISON

5-4.1 GLOBAL BEHAVIOUR OF THE ROOF STRUCTURE

This subchapter aims to exploit differences between the global behaviour of the roof structure in different rooms and models. The results of deformations are shown for the critical combination of SLS.

In Figure 5.12 the global deformation of the room 2 is shown. As expected, the biggest deformations are developed around the dormer, which because of the spatial load transfer produces large deformations. The dormer is stationed up against the masonry wall. The initial calculation without masonry wall produced the horizontal drift of 35mm. The gap between the structure and the wall is not present, therefore the wall acts as a horizontal restraint to the roof structure. After modelling the masonry wall the horizontal drift in dormer area dropped to 0,6mm.

However, the introduction of wall produced larger horizontal drift of the structure on south eaves around the area of the dormer. (on Figure 5.12 down, red zone) This influence can be further supported by horizontal deformations of the tie beams Figure 5.13 left.

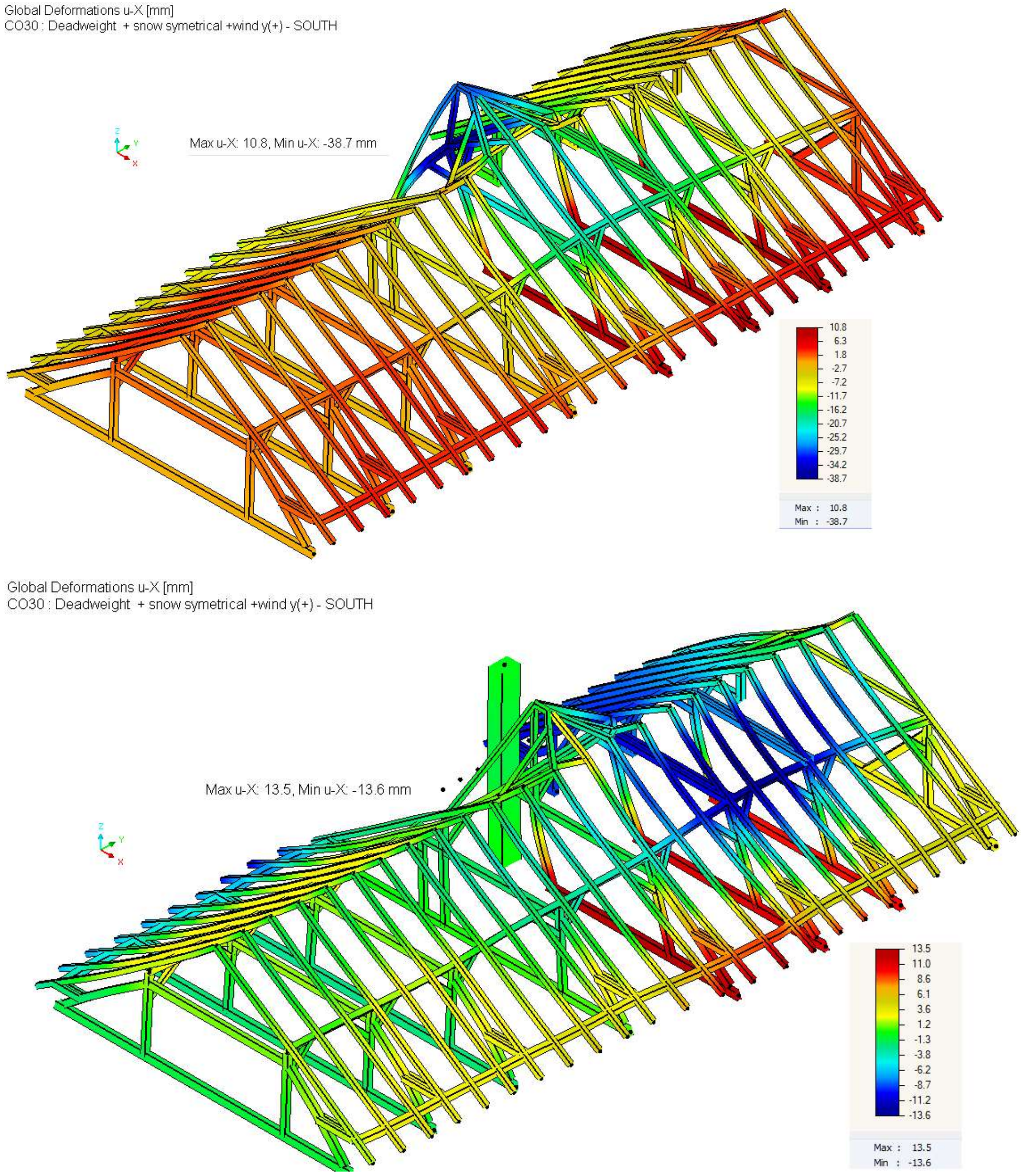


Figure 5.12. down: Global deformation of the roof structure with masonry wall M1; up: Without the masonry wall M1

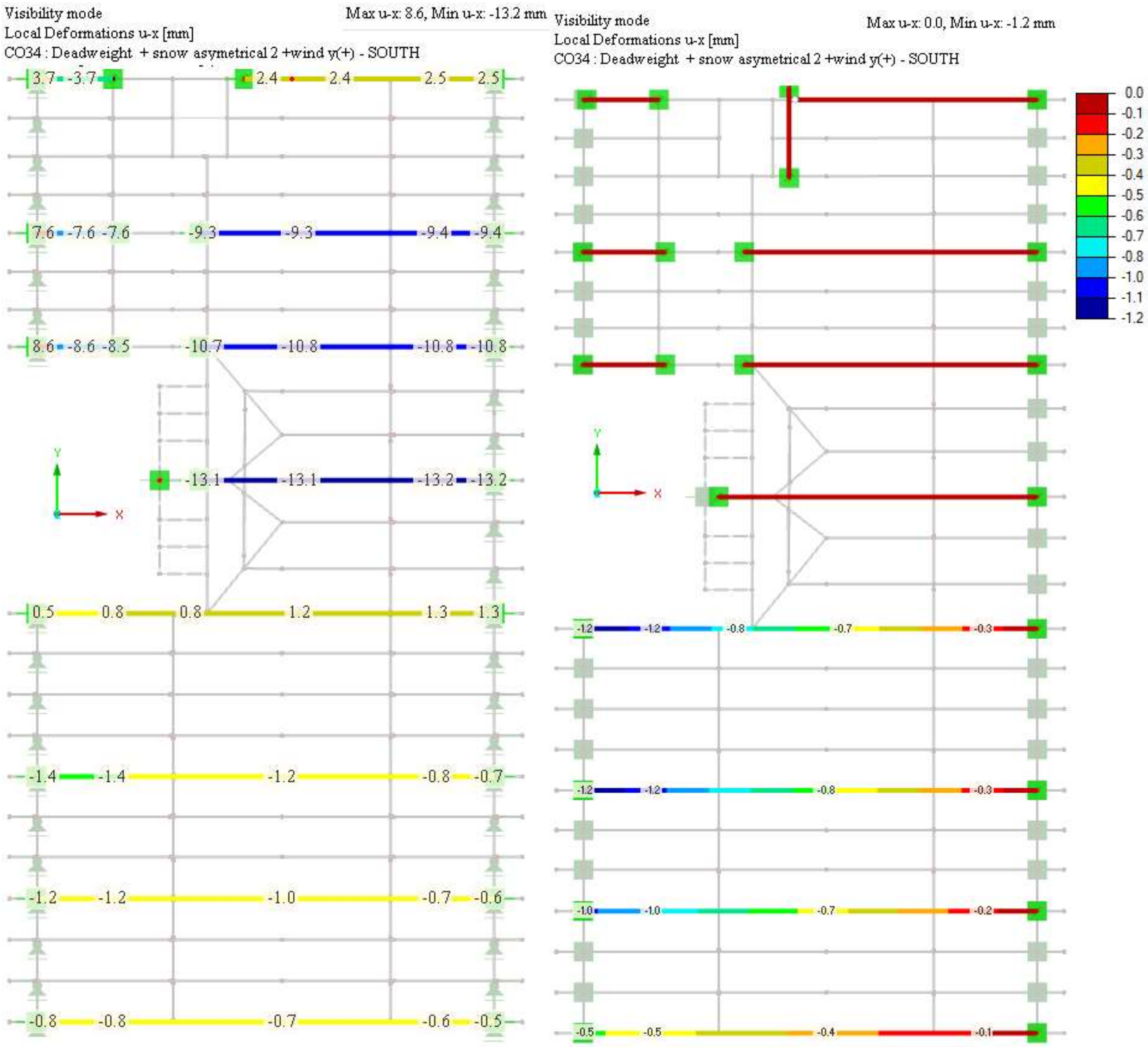


Figure 5.13. left: Horizontal deformation of the tie beams model M1; right: Model M2

On Figure 5.13 the influence of different supports on a tie beam is shown. Model M2 present unrealistic behaviour as the horizontal drift of the tie beams is restricted, especially on axes 11,12,13 where tie beams are discontinued.

The larger roof plane on the northern side results with asymmetric loading on the principal and common trusses for symmetric and asymmetric loading. This can be observed by bigger deflection of rafters on the northern side Figure 5.12, as well as bigger horizontal deformations of foot and middle purlin on the south side Figure 5.20. As a consequence, the loads on principal trusses are always asymmetric and bigger bending moments are utilized.

On Figure 5.14 vertical deformation of the roof structure in room 2 is shown. The biggest deformations are observed on axes (10,11,12). On these axes principal truss is a purlin-tie roof with double (triple) standing chair, therefore this behaviour is expected because struts in purlin tie roof are transferring loads to tie beam, which is then subjected to the big bending moments and deformation. On the other hand in queen post principal trusses smaller deformations are observed as post act as a support to a tie beam. However, bigger bending stresses were observed for queen post trusses. For explanation refer to Table 5.20 and explanation above the table. Deformations are still smaller due to the asymmetrical deformation of the tie beam, see also Figure 1.30.

In room 3 extensive deformations from the spatial influence of the valley and hip rafters are not present. For queen post trusses similar behaviour as in room 2 is observed. The bigger roof plane on norther side causes in plane rotation of principal truss and uplift of the middle purlin on the south side, as a consequence rafters on the south side have uplift deflection 12,4 mm (rafters in red zone), see Figure 5.15.

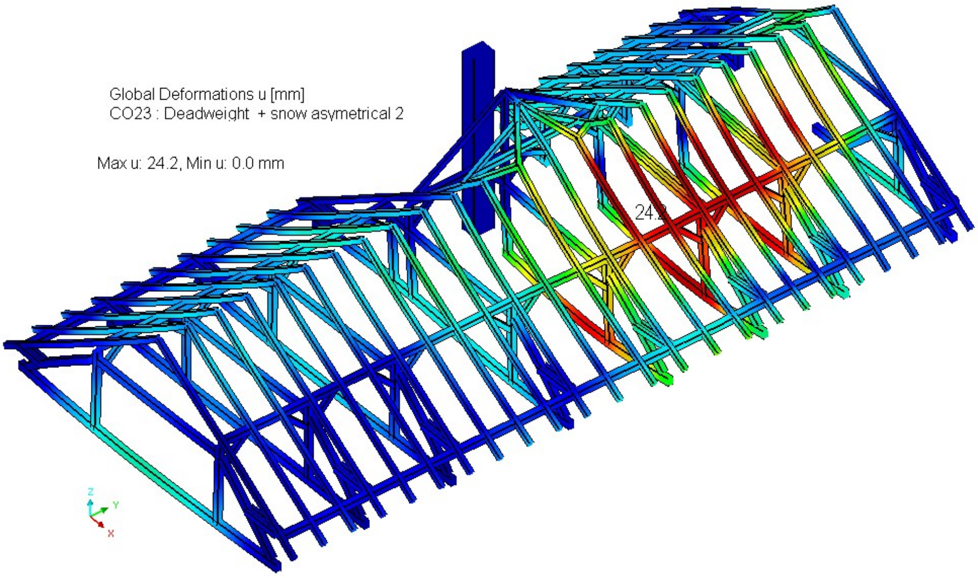


Figure 5.14 Vertical deformation of the roof structure in room 2 M1

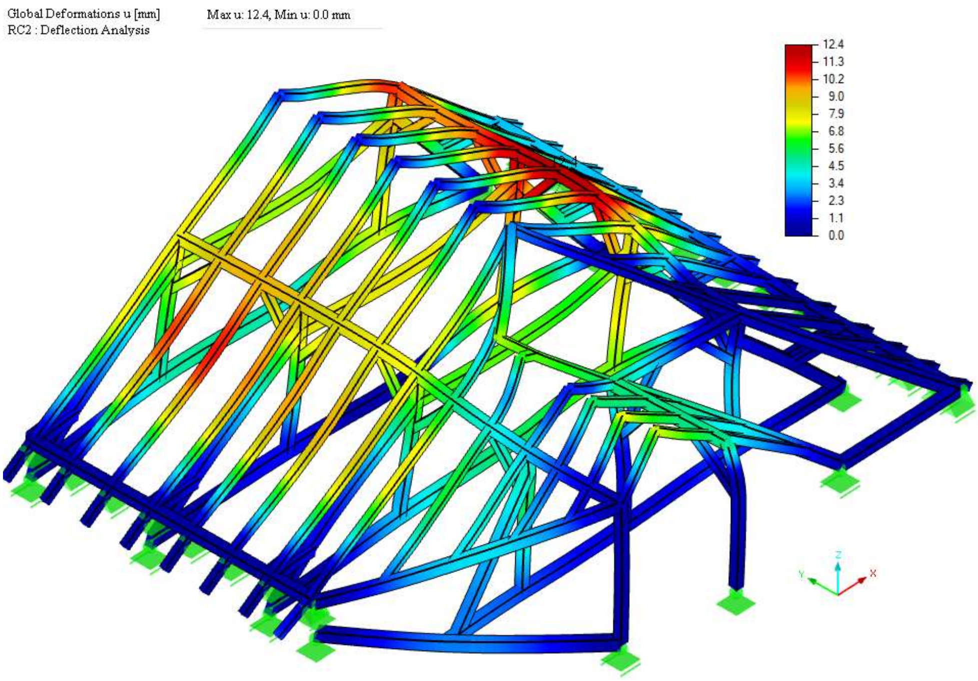


Figure 5.15. Global deformation of the roof structure in room 3, Model M2

On Figure 5.17 the global deformation of the model M2 in room 1 is shown. It can be clearly seen that critical areas are the valley and hip rafters, which because of their length, and loads from jack rafters receive the biggest loads and therefore largest deformations. Moreover, the horizontal trust from hip and valley rafters induces out of the plane deformation of the axis 17 for about 12,00mm. The rest of the structure doesn't show big deformations in any direction. This doesn't represent the realistic state as the structure is too stiff. The reason for such behaviour lays in supports which are modelled as hinged and sliding. The principal trusses in room 1 have discontinued tie beams, for each tie beam the hinged support is introduced.

This restricts the whole structure from moving. In other hand placing the hinged support on only one tie beam results with excessive deformation up to 25mm of foot purlin on the northern side and deformation of hip rafter in global direction of 42 mm. Therefore, modelling of spring supports is needed to obtain more realistic results.

On Figure 5.18 the global deformation of the model M1 in room 1 is shown. Modelling of spring supports represent the more realistic state in this case as roof structure has smaller stiffness and overall deformations are bigger. The deformation of the hip rafter is 29mm, while deformation of foot purlin dropped to 12mm. However, the supports on foot purlin are still modelled as sliding, with the introduction of spring support the deformation drops to 7,4mm, see Figure 5.16. In this case the spring supports on foot purlins give more stiffness, therefore, the deformation of hip rafter is also reduced to 20mm.

The changes observed between model M1 and M2 emphasise the need for realistic modelling of supports. However, the differences in global behaviour between models M 1 and M2 in room 3 don’t prove to be significant as in room 1. This can be prescribed to smaller spatial influence in room 3 as hip and valley rafters are shorter and there is no discontinuity (continues tie beams).

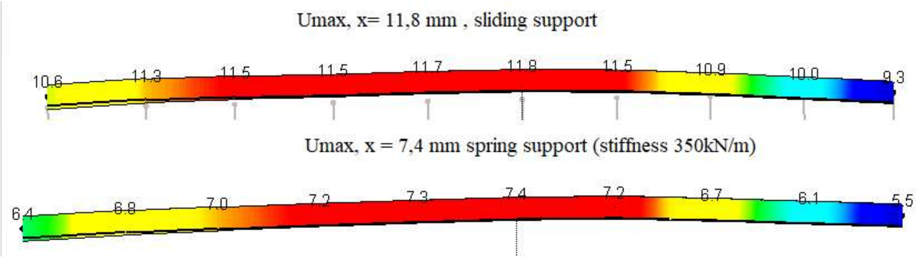


Figure 5.16 Comparison of different support stiffness on foot purlin, Model M1

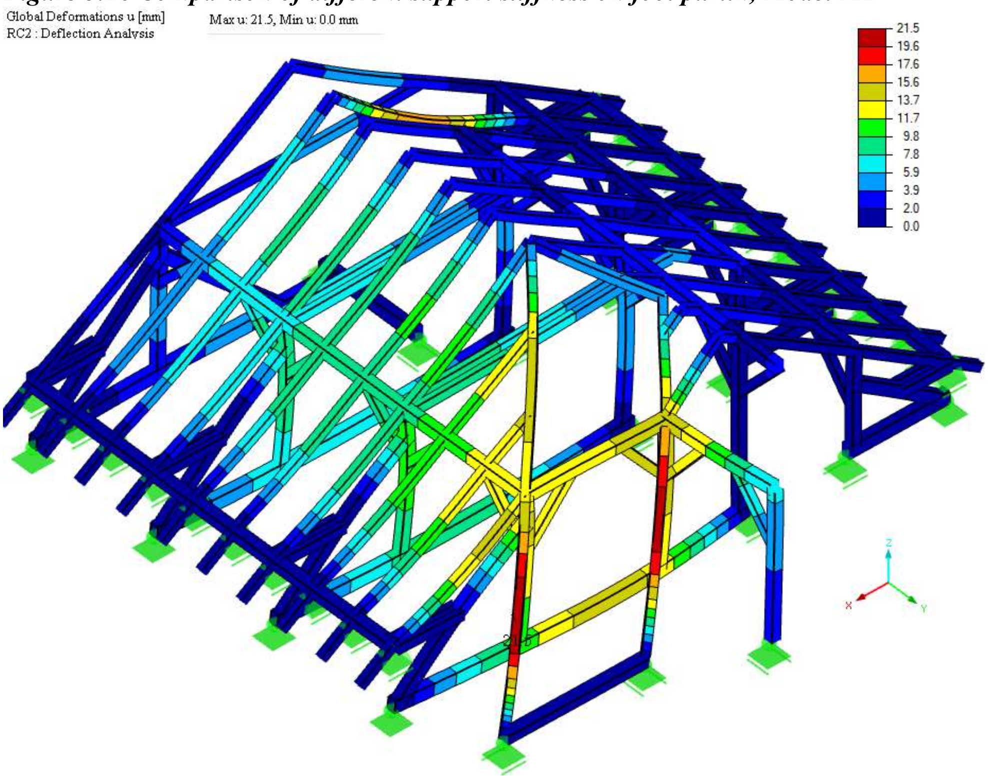


Figure 5.17. Global deformation of the roof structure in room 1, Model M2

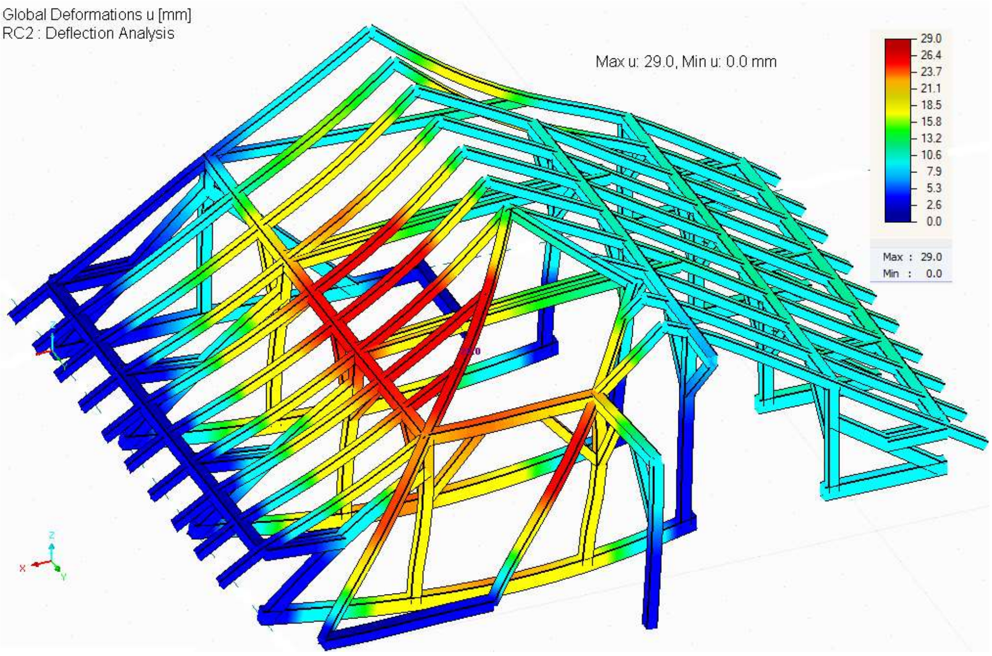


Figure 5.18 Global deformation of the roof structure in room 1, model M1

5-4.2 COMPARISON OF DIFFERENT MODELS IN ROOM 2

Next, the comparison of deformations and internal forces for developed models in room 2 are shown. Deformations are shown according to the SLS resulting envelope, while internal forces are shown for the ULS resulting envelope.

Comparison is conducted on critical principal truss in room 2. That is principal truss on axis 8 (queen post truss). Comparison for purlin-tie truss and king post truss is left out to keep the scope of the work in reasonable scale.

It should be noted during the reading of the results that by comparing the envelopes in models (M1, M2, M3, M4) some values from different models may not belong to the same load combination. This influence obstructs a clean comparison between models. However, by comparing enveloping diagrams and diagrams for particular critical load combination only minor differences in some elements are observed therefore, for sake of simplicity differences are neglected. Moreover, a clean comparison between 3D and 2D models is hard to obtain because of simplifications in load determination during modelling of models M3&4.

Characteristic of queen post trusses is load transfer as a combination of purlin roof and rafter roof. The relation between these load transfer systems depends upon several factors such as:

- Stiffness of middle purlin and foot purlin (torsional and bending)
- Support stiffness of foot purlin on places of principal trusses (i.e. short ties)
- Friction between attic wall and foot purlin
- Load bearing capacity and stiffness of masonry wall to out of the plane forces
- Torsional stiffness of the connection middle purlin – queen post
- Roof pitch (higher pitch = roof behaves more like a rafter roof, and other way around)

In queen post truss rafters are supported in places of purlins. The differences between stiffnesses of these purlins are responsible for redistribution of load transfer in queen post truss (rafter + purlin roof). The supports of rafters in rafter foot (foot purlin) are responsible for the behaviour of the roof as rafter roof, while intermediate supports (middle purlin) for purlin roof load transfer. In that manner, if more load from rafters is discharged trough intermediate purlin to principal truss the roof carries more as purlin roof rather than rafter roof. Alternatively, if foot purlin is very stiff and attic walls can withstand large thrust forces, then roof tend to transfer loads more as rafter roof. It can be concluded as torsional moment and vertical load on middle purlins are bigger the principal truss is more loaded and roof carries as purlin roof. This characteristic behaviour is also illustrated in Figure 5.19. For more explanation regarding this effect refer to master thesis from Kirchler [21]. (cf. [21]. [17])

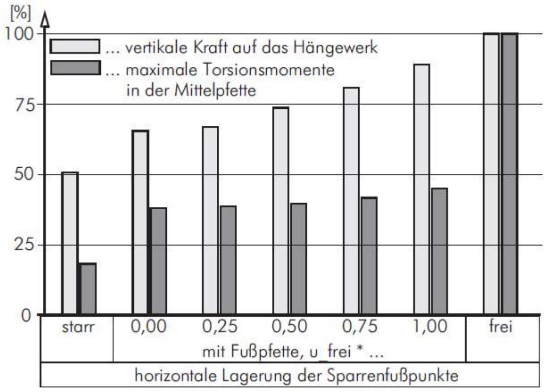


Figure 5.19. Effects of deformability of the foot purlin on torsion moments in middle purlin and support reactions on middle purlin [17]

This relation between load transfer is also present in analysed queen post truss structure, whereby comparing different models clear differences in values of internal forces and deformations were observed which point to change in global load distribution between compared models.

In Table 5.12 vertical reactions on middle purlin for different models in room 2 are shown. The vertical reaction in Model M2 is 26,70% bigger than in model M1. As connections in model M2 are modelled as hinged without realistic stiffnesses, principal queen post truss is stiffer than in model M1. Moreover, in model M2 supports of rafters on foot purlin have smaller horizontal stiffness compared to the same supports in model M1. As a consequence of these two effects model M2 has larger vertical loads upon principal truss and therefore has prevailing purlin load carrying behaviour, while in model M1 due to the spring supports on foot purlin, and connection stiffnesses the roof behaves more like a rafter roof. This effect is in accordance with Figure 5.19.

On Table 5.13 horizontal reactions on middle purlin are shown, the values are also following described behaviour, but now observed differences are larger as a horizontal force in model M2 is 75,85% larger than in model M1.

Table 5.12. Vertical reactions from middle purlin to queen post

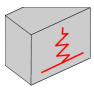
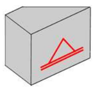
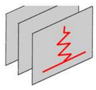
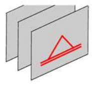
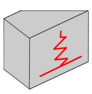
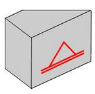
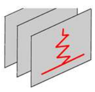
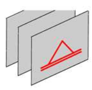
VERTICAL REACTIONS ON MIDDLE PURLIN				
Model	M1 	M2 	M3 	M4 
My	35.2 100%	44.6 126.70%	36 102.27%	38 107.95%

Table 5.13. Horizontal reactions from middle purlin to queen post

HORIZONTAL REACTIONS ON MIDDLE PURLIN				
Model	M1 	M2 	M3 	M4 
My	2.36 100%	4.15 175.85%	5.07 214.83%	5.42 229.66%

Next from Table 5.14 to Table 5.21 the values of internal forces in structural elements for queen post truss are given. Values are compared in room 2 for 4 developed models (M1-M4).

From Table 5.14 it can be seen that in Model 2 the normal force in the brace is higher. This is due to the higher stiffness of the queen post truss, and bigger loads from rafters on the principal truss. The moments are smaller due to the neglected eccentricities in connections and also due to the smaller normal force in the short tie, which is smaller for the Model M2 as the main load path is along braces, see Table 5.15.

In model M3 bending moment in the brace is bigger than the moment in model M4 although force in short tie in model M3 is smaller. This is due to the influence of eccentricities in connections, which proves how big influence they have on results.

In Table 5.15, interesting results are observed as a force in short tie is smaller for model M2 than model M1. In first place one would expect results to be contrary than observed as model M2 has sliding supports in place where rafters rest on foot purlin, which means that short ties which support foot purlin on places of principal trusses should take bigger loads than in model M1, because foot purlin has smaller bending stiffness in model M2. This represents not to be the case as in model M2 the majority of load from rafters is discharged in the middle purlin and therefore loads on foot purlins are smaller than needed to overcome the effect of sliding supports.

Table 5.14 Internal forces in brace

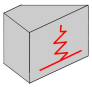
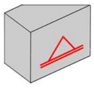
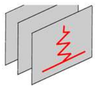
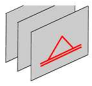
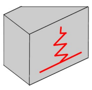
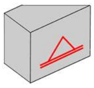
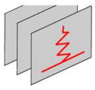
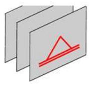
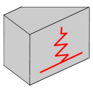
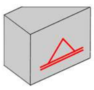
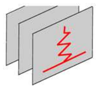
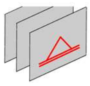
Internal forces - BRACE				
Model	M1 	M2 	M3 	M4 
My	11.6 100%	5.66 48.79%	9.03 77.84%	5.87 50.60%
N	74.7 100%	87.63 117.31%	73.5 98.39%	79.44 106.35%

Table 5.15. Internal forces in short tie

Internal forces – SHORT TIE				
Model	M1 	M2 	M3 	M4 
N(+)	13.32 100%	9.94 74.62%	9.47 71.10%	10.31 77.40%

In Table 5.16 bending moments in foot purlin are shown, as expected moment M_z is smaller for model M2.

Table 5.16. Internal forces in foot purlin

Internal forces - FOOT PURLIN				
Model	M1 	M2 	M3 	M4 
Mz	3.03 100%	2 66.01%	N/A N/A	N/A N/A

In Table 5.17 comparison of internal forces in middle purlin is shown. On these results, major influence has modelling of connection (middle purlin-queen post) where torsional stiffness of connection has a major impact upon torsional moments in middle purlin and on overall load transfer behaviour. In Figure 5.22 diagram of torsional moments for models, M1&M2 are shown. These diagrams point to that considering realistic torsional stiffness of connection in model M1 greatly reduces torsional moments in middle purlins, in contrast to traditional modelling (model M2). In model M2 two cases were considered regarding torsional stiffness. First full restrain was introduced where the connection is regarded as fully rigid for accepting torsional moments. In that case middle purlin in accepted up to 10 times bigger torsional moments comparing to model M1. In second attempt the connection is regarded as fully hinged, the torsional moments in that case almost decreased to 0. It can be concluded that both cases produce extreme results, and both don't present realistic results, although differences observed when connection in model M2 was considered as hinged were smaller. This emphasises the need for realistic modelling of connection of middle purlin to queen post. Moreover, this has a big influence on load transfer principle as bigger torsional moments in purlins are characteristic for purlin roofs.

Due to severe shrinkage cracking in middle purlins, the load bearing capacity for torsional moments is reduced, therefore the question arises, how much torsional moments can middle purlin actually receive? In that manner, it is advisable to limit torsional stiffness of connection in a range of 15-30 kNm/rad.

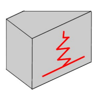
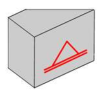
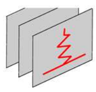
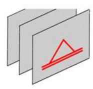
In Figure 5.20 the deformation of foot and middle purlin are shown. Due to the asymmetrical roof structure, where northern rafters are longer and due to the asymmetrical loading, the asymmetric response is developed. Due to asymmetric loading bending moments M_y, M_z are in general bigger on the northern side, see Figure 5.21 (M_z), Figure 5.23. (M_y).

For models, M3&M4 longitudinal bracing wall on south side was modelled separately. For model M3 influence of the tie beam is substituted with spring supports, where spring stiffness is calculated by dividing deflection of the tie beam with force which caused the same deflection. For horizontal deformations spring produce good alternative as observed differences are smaller than 20%. However, when referring to vertical deformations, realistic results couldn't be established, mainly due to the different load distribution between models M1&3, which caused completely different behaviour of the bracing wall (i.e. lifting of middle purlin due to asymmetrical load, which can`t be simulated in 2D, model M3)., see Figure 5.24.

In a simplified model, M4 influence of tie beams is neglected and all supports are modelled as hinged or sliding.

The comparison of internal forces in middle purlins for all models can be seen in Table 5.17, bending diagrams on Figure 5.23., and global behaviour of the longitudinal bracing wall in Figure 5.24.

Table 5.17. Internal forces in middle purlin

Internal forces - MIDDLE PURLIN				
Model	M1 	M2 	M3 	M4 
My	5.49 100%	5.11 93.08%	4.26 77.60%	3.98 72.50%
Mz	2 100%	1.01 50.50%	0 N/A	0 N/A
Mt	0.33 100%	0.93 281.82%	0 N/A	0 N/A
N(+)	10.5 100%	17.5 166.67%	8.21 N/A	9.76 92.95%
N(-)	0.85 100%	1.43 168.24%	0.27 N/A	0.27 31.76%

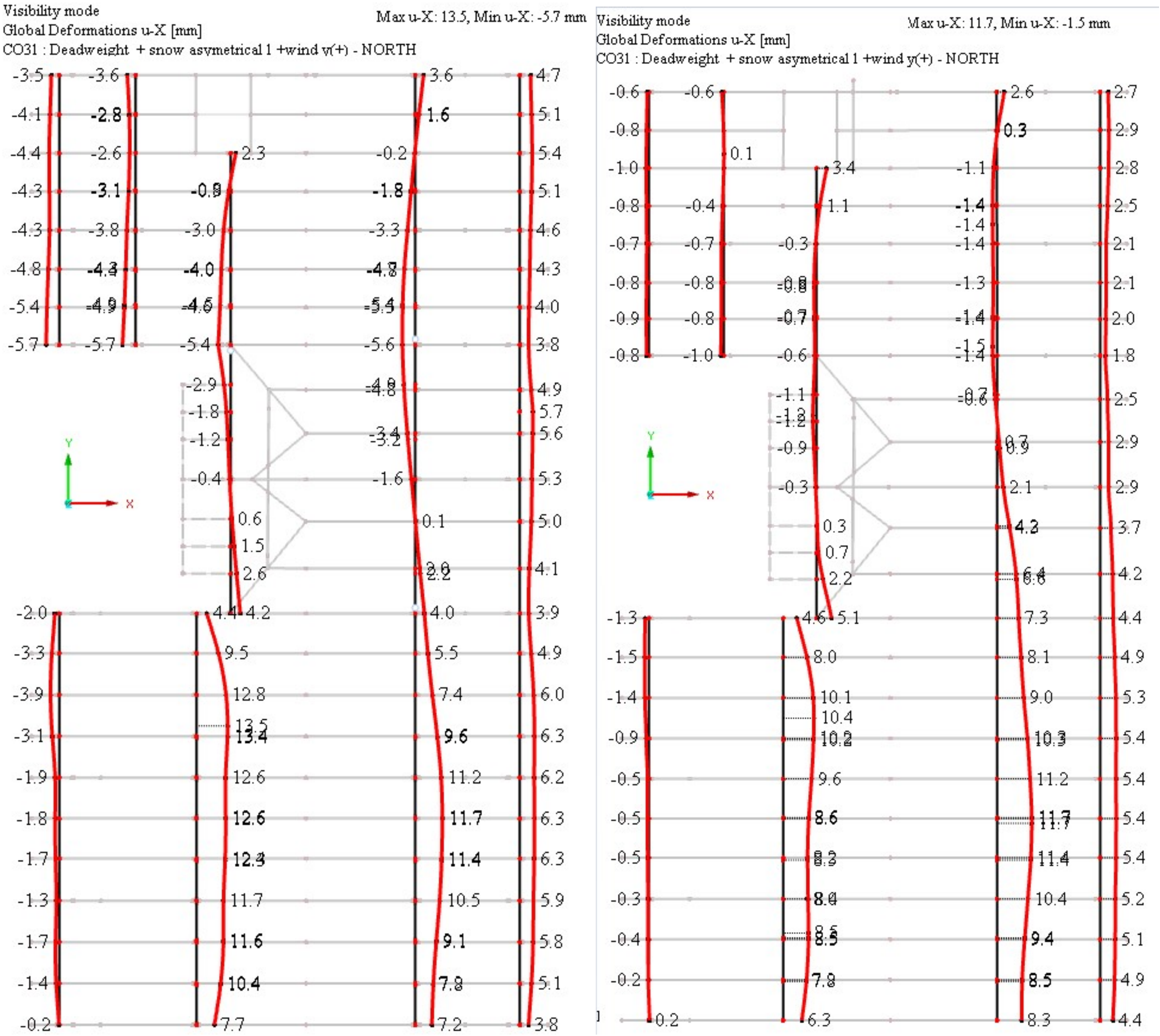


Figure 5.20. left: Deformation of the middle and foot purlin M1; right: M2

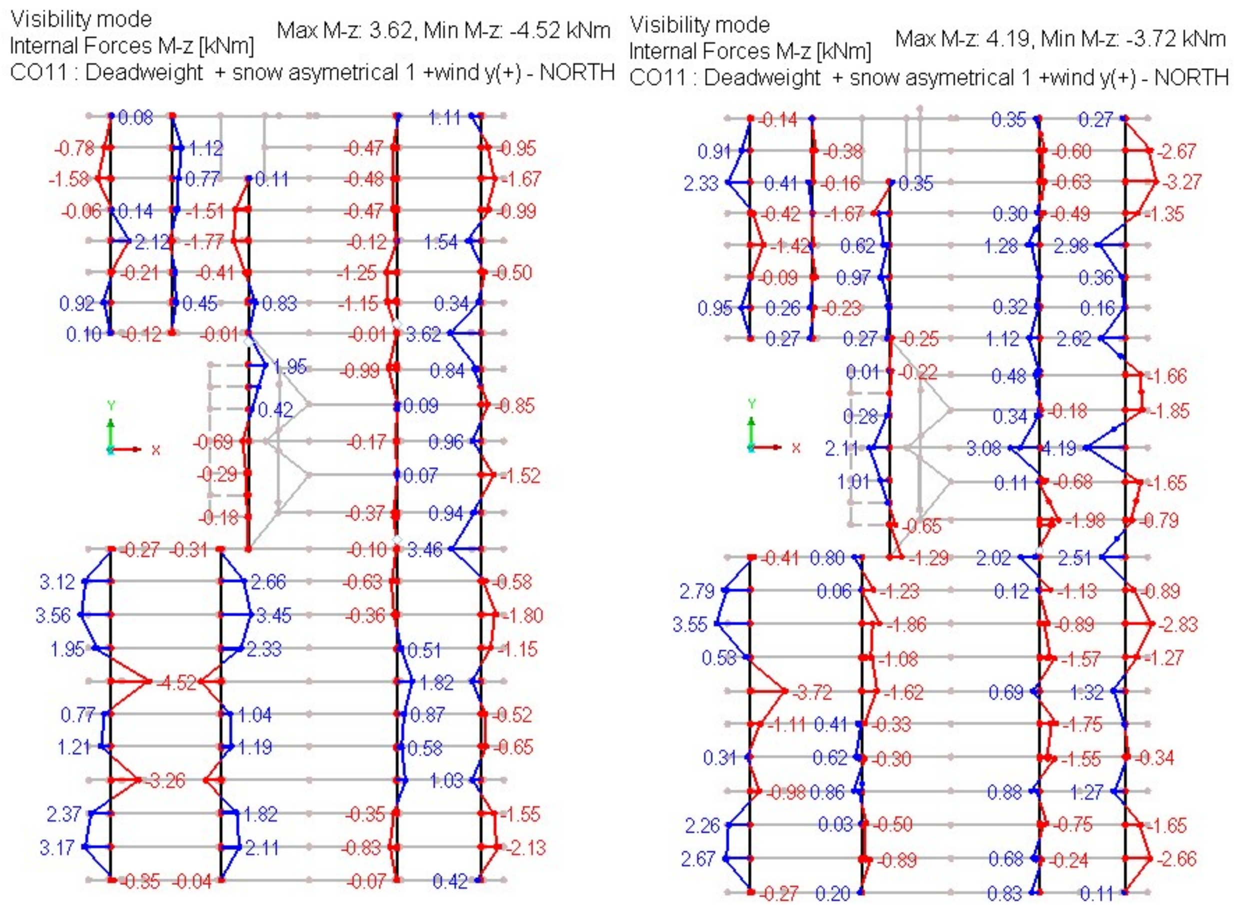


Figure 5.21 left: Mz bending moments in middle and foot purlins M1; right: Model M2

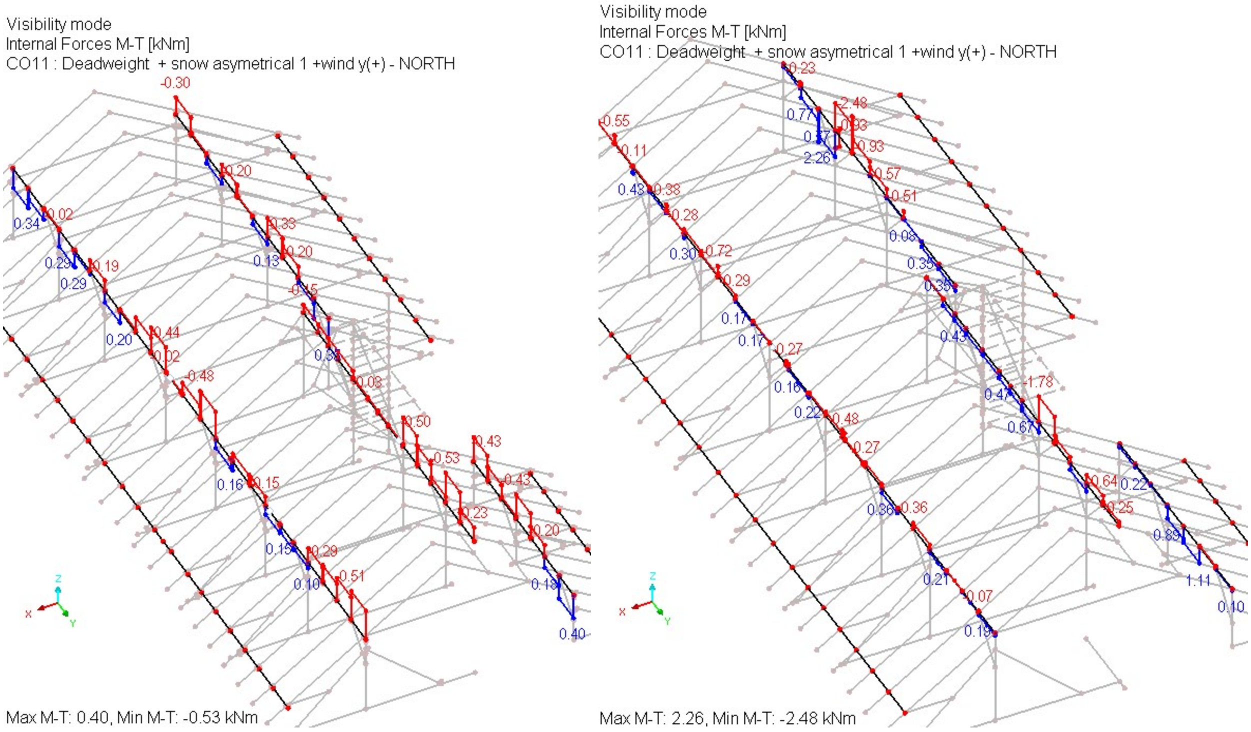


Figure 5.22 left: Torsion moments in middle purlins, model M1; right: Model M2

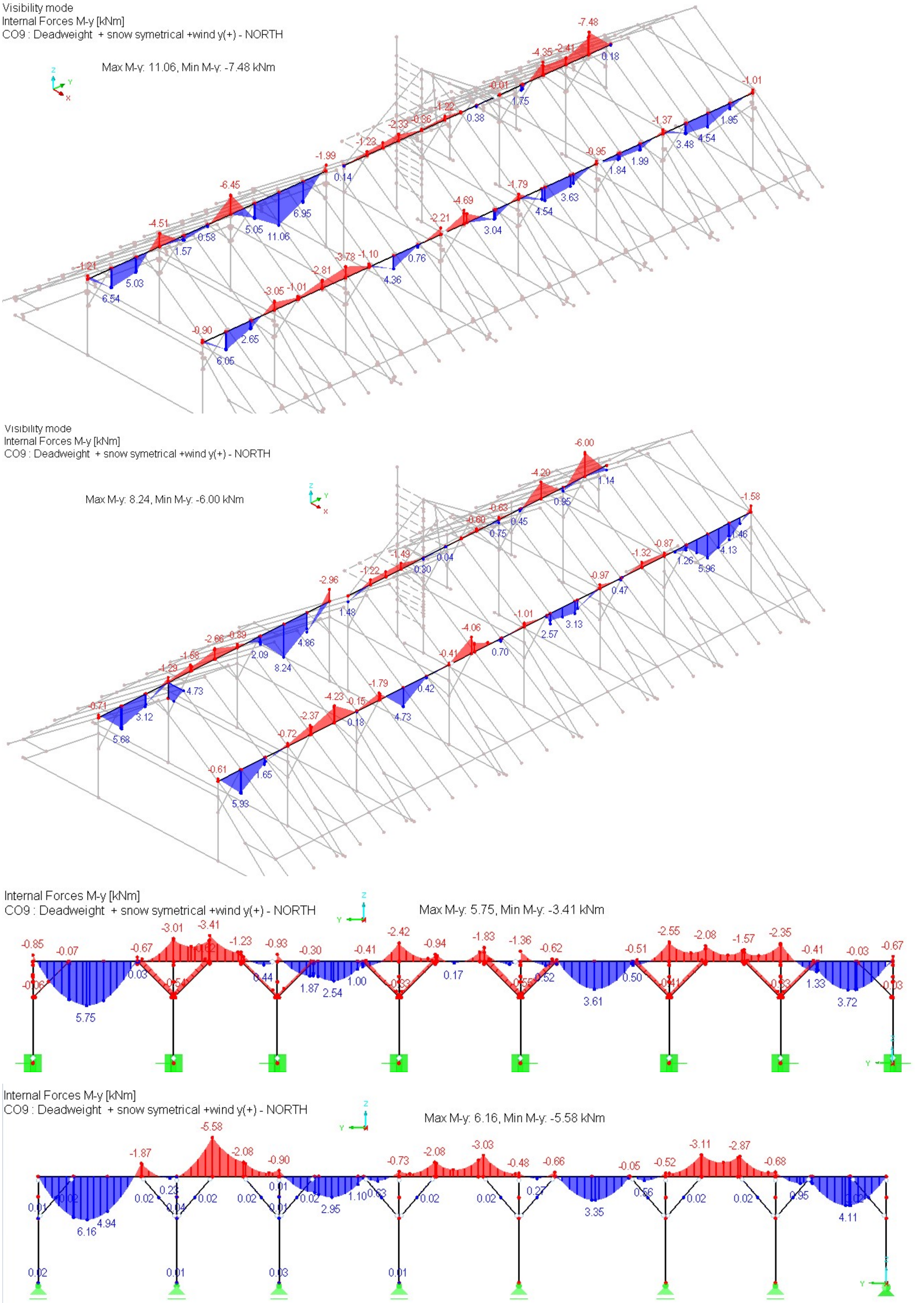


Figure 5.23 Bending moments My (from up model: M1, M2, M3, M4)

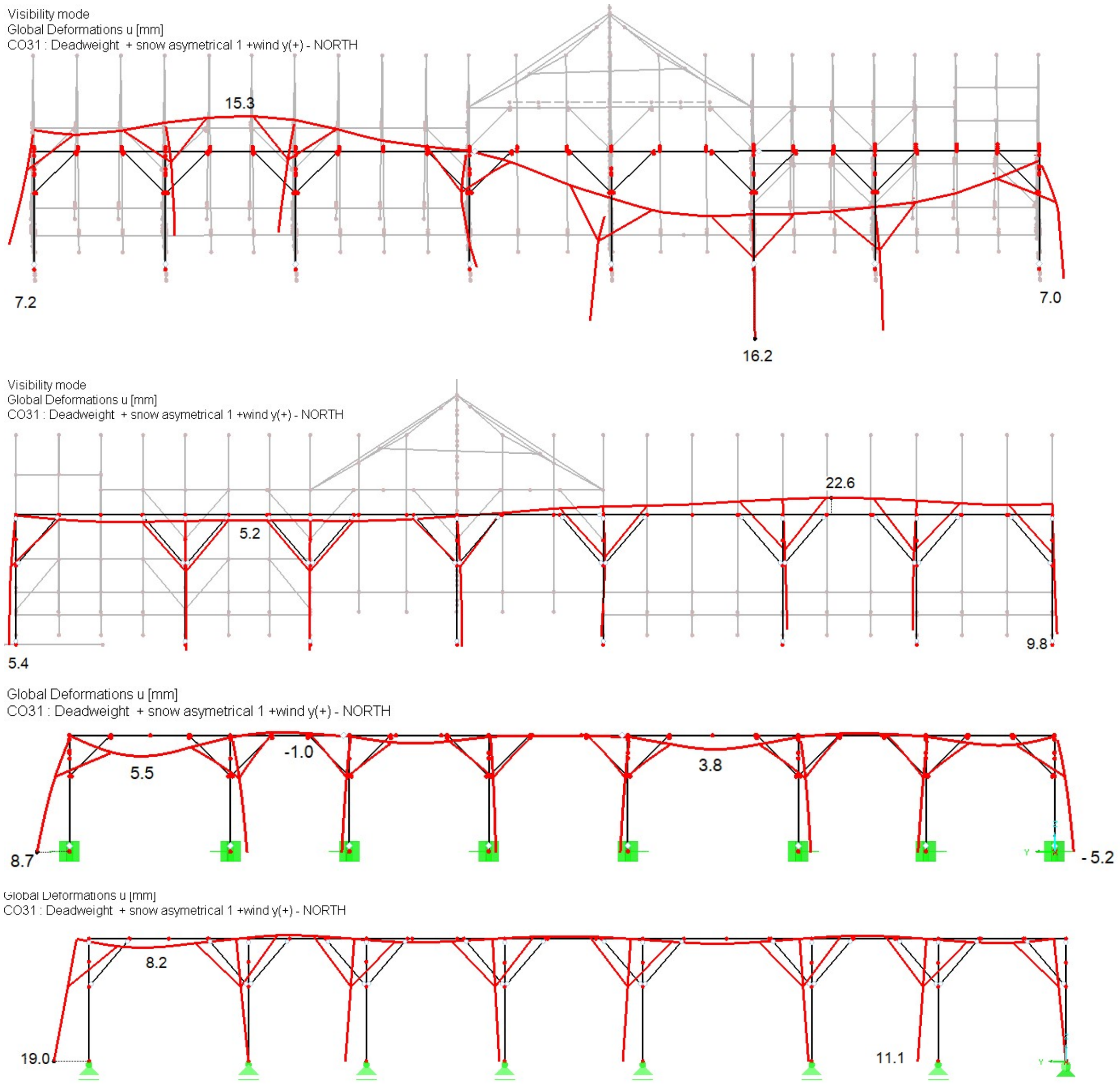
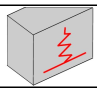
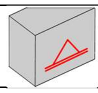
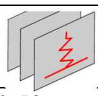
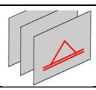


Figure 5.24 Deformation of a longitudinal bracing wall (from up M1, M2, M3, M4)

In Table 5.18 comparison of normal forces in knee braces is shown. Although longitudinal bracing walls in models M3&M4 present oversimplification of realistic state, the observed differences are smaller than the difference between models M1&M2. Reasons for such results are hard to explain as there are many influences which can produce this kind of results, in one way, they can be also regarded as random correspondence.

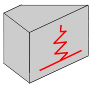
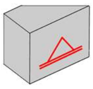
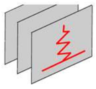
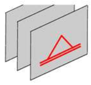
Table 5.18 Internal forces in knee braces

Internal forces - KNEE BRACES				
Model	M1 	M2 	M3 	M4 
N(+)	0.66 100%	0.76 115.15%	0.52 78.79%	0.47 71.21%
N(-)	15.26 100%	26.75 175.29%	12.27 80.41%	15.22 99.74%

By comparing rafters in models M1&M2 it can be clearly seen how lager portion of the load is discharged from rafters to middle purlin in model M2 due to the modelling of connections and supports. This strengthens the previous acknowledgements regarding rafter and purlin roof load transfer.

In model M1 it can be seen how rafters start to lose their support from middle purlins as negative bending moments in rafter decrease while bending moments in field increase. This has to do with modelling of supports and connections as well as asymmetrical behaviour of the principal truss. Similar behaviour of rafters is also observed in master thesis of Kirchler [21], where internal forces in rafter are compared to several different modes (i.e. Volume, combined volume and rod model, rod models), for more information refer to. [17] [21]

Table 5.19. Internal forces in Rafter

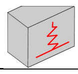
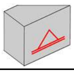
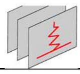
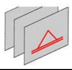
Internal forces - RAFTER				
Model	M1 	M2 	M3 	M4 
My FIELD 1	2.36 100%	1.94 82.20%	1.87 79.24%	1.82 77.12%
My MID PURLIN	3.68 100%	4.32 117.39%	3.6 97.83%	2.97 80.71%
My FIELD 2	1.19 100%	1.71 143.70%	1.18 99.16%	1.2 100.84%
N(-) FIELD 1	8.98 100%	8.12 90.42%	8.22 91.54%	7.71 85.86%
N(-) MID PURLIN	7.55 100%	3.06 40.53%	2.11 27.95%	1.15 15.23%
N(-) FOOT PURLIN	11.56 100%	7.04 60.90%	5.87 50.78%	4.91 42.47%

Different results in tie beam emphasise the influence of different approaches to modelling of supports. The biggest difference can be observed in tension force in the tie beam where tension force in model M1 is 22,5% smaller than model M2, as some portion of the force is discharged into structural walls. The tie beam in model M2 behaves like a simple supported beam, therefore all tension force is situated in the tie beam. The bigger bending moments in model M2 are a consequence of larger loads on queen post principal truss (asymmetric loading).

In room 2 bending moment in queen post truss (axis 8) is actually 10% bigger than moments in the purlin-tie roof with a double (triple) standing chair (axes 10-12) which at first can produce confusion. This irregularity developed as asymmetric loading on queen post trusses is highly pronounced, therefore queen post truss resists this effect with bending of tie beam, see Figure 5.25

On the other hand on axes (10-13) asymmetrical loading due is not so pronounced as observatory provide support to the roof structure and additional chair on northern side axes (11-13) also provide additional support.

Table 5.20. Internal forces in Tie beam

Internal forces - TIE BEAM				
Model	M1 	M2 	M3 	M4 
My	16.35 100%	17.19 105.14%	18.44 112.78%	23.79 145.50%
Mz	0.77 100%	0.89 115.58%	0 N/A	0 N/A
N(+)	57.02 100%	69.81 122.43%	53.74 94.25%	59.12 103.68%

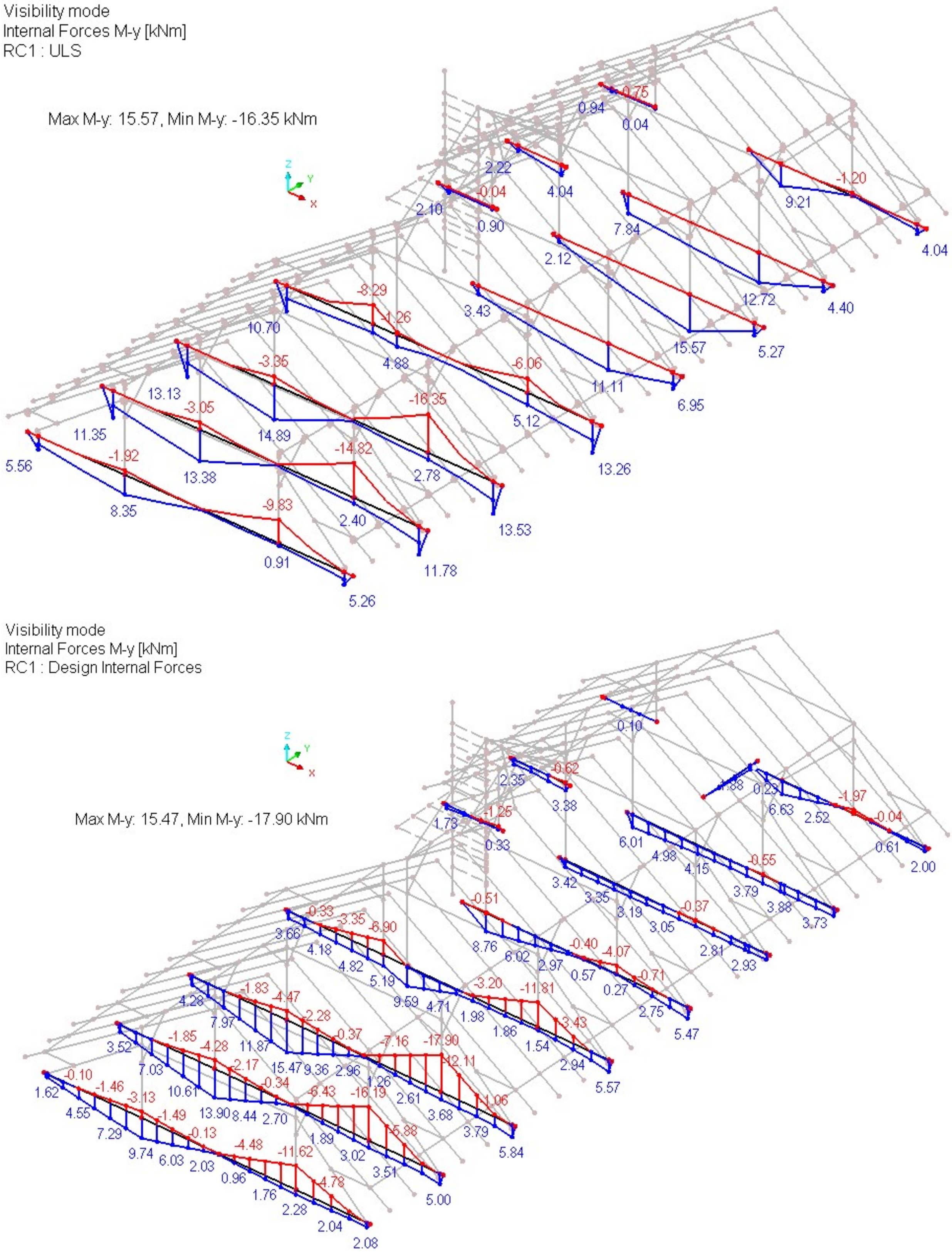


Figure 5.25 up: Bending moments My in tie beams in room 2 model M1; down: Model M2

In Table 5.21, Table 5.22, comparison of the results in the post and straining beam are given.

Table 5.21. Internal forces in Post

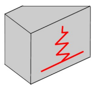
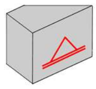
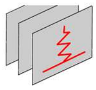
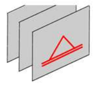
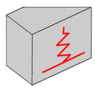
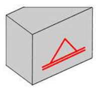
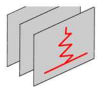
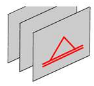
Internal forces - POST				
Model	M1 	M2 	M3 	M4 
My	4.33 100%	2.94 67.90%	4.08 94.23%	3 69.28%
Mz	0.76 100%	0.67 88.16%	0.57 75.00%	0.16 21.05%
N(+)	35 100%	52.8 150.86%	18.14 51.83%	20.39 58.26%
N(-)	15.63 100%	7.68 49.14%	35.45 226.81%	38.2 244.40%

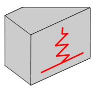
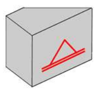
Table 5.22. Internal forces in Straining beam

Internal forces - STRAINING BEAM				
Model	M1 	M2 	M3 	M4 
N(-)	50.89 100%	50.26 98.76%	43.08 84.65%	43.29 85.07%

In Table 5.23 comparison of the results in the collar beam is given

Note: Collar beam is part of the purlin-tie principal truss.

Table 5.23 Internal forces in collar beam

Internal forces - COLLAR BEAM		
Model	M1 	M2 
N(-)	13.5 100%	27 200%

5-5 FINAL CONCLUSION

At last some final conclusions and acknowledgements concluded by the author during the process of modelling of the roof structure are given.

- Analysed models provided different load transfer, model M1 showed behaviour characteristic to a rafter roof, while model M2 showed behaviour characteristic to a purlin roof.
- Stiffness of connections and supports significantly changes load distribution and utilization factors in members and connections.
- Overall utilization factors for members tend to be higher in model M1 than M2, see Appendix C-1
- Overall utilization factors in connections tend to be lower in model M1 than M2, see Appendix C-1.1
- For connections, big utilization factors are obtained. These results don't correspond to the real state, as during inspection, damages and signs of overload in connections have not been found. This implies that verifications by current standards are too conservative. Moreover, the current standards don't give much (if any) information upon calculation of traditional timber connections.
- Material properties are usually better than predicted by default (C24), high utilization factors, without any observed damage in roof point to that possibility. Improving material properties in calculations should be considered then.
- In general load analysis and verification of structural members and connections according to the valid Eurocodes is too conservative for existing structures and especially for the heritage structures. Therefore, the author thinks that new norms with considerably lower partial factors should be developed, the preparations and discussions are already taking place in that direction.
- Obtained global behaviour for model M1&2 corresponds to deformations found on site.
- Decoupling can produce satisfactory results if the structure is decoupled at the right area, in other cases should be avoided by any means (i.e. pronounced spatial influence).
- Combination of extremely high and low stiffnesses in model M1 can produce random ambiguities in software which leads to errors and mistakes in the model.
- Time consumption for the making of the model M1 is significantly bigger than for the model M2.
- Automatic verifications provided by software should be avoided as remaining cross section area of the cross section is not considered.
- Purlin roofs carry loads as a combination of a rafter and purlin roof. Similar behaviour of purlin roofs is concluded in works such as [17] [32] [21]

At last author, opinion is that historic timber roof structures should be modelled according to the described approach by starting with a model without eccentricities and then modelling more complex model such as model M1. With this approach behaviour of the structure close to reality should be obtained. Calculations provided by software should be verified by simple hand calculations (e.g. hand calculation of vertical and horizontal reactions should match reactions given by the software.)

In Appendix C-2 diagrams of internal forces, and deformations for critical principal and secondary trusses are given (for all models).

In appendix C, the summary of the member and connection verifications are given.

CHAPTER 6:STRENGTHENING

6-1 OVERVIEW

The decision to proceed with the strengthening of historic timber structure should be carefully decided only after the detailed and extensive process of survey and assessment followed by appropriate structural analysis. In the preservation project of historic roof structure all steps before strengthening influence on the final decision if the strengthening of the roof structure is needed. In order to protect cultural and structural value emphasise should be put on the quality of these steps, as they have a big influence on the extent of the strengthening. Nonetheless, often is the case when strengthening measures are still needed after the detailed assessment. (cf. [80] [81] [82]

Today strict conditions and requirements regarding allowed strengthening measures are given in the very beginning of the preservation project by a Conservator (i.e.Conservation office). Although today new strengthening measures are quickly emerging, the strict conditions often leave a structural engineer with little manoeuvre space regarding finding and implementing the optimal strengthening solutions. Especially in this case the need from a structural engineer to be highly educated and experienced with dealing of historic timber roof structures, but also innovative and critic minded during the design process is crucial in order to provide the best strengthening measure and in the same time obtain the needed level of structural safety. This is the case as sometimes requirements and priorities given by conservation office are contradicting the structural safety of the roof structure. [81] [82]

The strict requirements imposed by conservation office are special for each country, or for the region in the country itself. Nonetheless, the majority of these requirements are based on internationally recognised principles presented in ICOMOS („ principles for the conservation of wooden built heritage “) [36] such as

- *Interventions should follow the criteria of the minimal intervention capable of ensuring the survival of the construction, saving as much as possible of its authenticity and integrity, and allowing it to continue to perform its function safely.*
- *The first stage in the process of intervention should be to devise a general strategy for the conservation of the building. This needs to be discussed and agreed by all parties involved.*
- *The intervention strategy must take into account the prevailing cultural values.*
- *Interventions should preferably: a be the minimum necessary to ensure the physical and structural stability and the long-term survival of the structure or site as well as its cultural significance;*
- *Interventions should preferably: follow traditional practices*
- *Interventions should preferably: be reversible, if technically possible; d not prejudice or impede future conservation work should this become necessary*

The interventions in the roof structure can be distinguished in two ways: rehabilitation or substitution. Rehabilitation refers to repairs and strengthening (strengthening actions), the desirable approach in which repairs or strengthening is done usually only to achieve original load bearing capacity of the structure, although improvement of the load bearing capacity is possible, especially when referring to the improvement of seismic behaviour (i.e retrofitting measures). Substitution refers to the removal and substitution of the old and damaged element with a new one. This approach should be used only when damages to the member or part of the structure are so extent that the damaged member is unrepairable and strengthening measures are inefficient or impossible to conduct. In Figure 6.1 approaches to repair in the roof structures according to Meisel [17] are shown, where besides rehabilitation and substitution additional simple but effective approach is introduced where performance can be improved by reducing deadweight or live loads. [81]

In the spirit of recommendations given by ICOMOS in literature often terms: compatibility and reversibility of strengthening measures are mentioned. Compatibility emphasises using the traditional techniques (i.e. traditional carpentry) and materials for strengthening which correspond to the spirit of historic roof structure and materials which are in compliance with the behaviour of the timber (e.g usage of steel and timber over FRP). reversibility is often emphasised by conservation offices during preservation projects. This is due to the vast development of strengthening measures and techniques which during the time may replace existing strengthening measure with one more appropriate and effective strengthening measure, therefore the existing strengthening measures should be reversible so they can be demounted from the roof structure. Reversibility of a strengthening measure also leaves place for its removal if it degrades structural behaviour of the roof structure. Nonetheless, strengthening measure should not degrade structural behaviour of the roof structure by any means, this can only be avoided with „step by step" iteration design in which impact of planned strengthening measures on the structural behaviour is analysed with suitable numerical models.

It should be noted that for the historic timber structures of great value, the approach where a global structural system is modified (Addition) should be avoided. This is already recognised by the conservation offices as the requirements are strict regarding this topic.

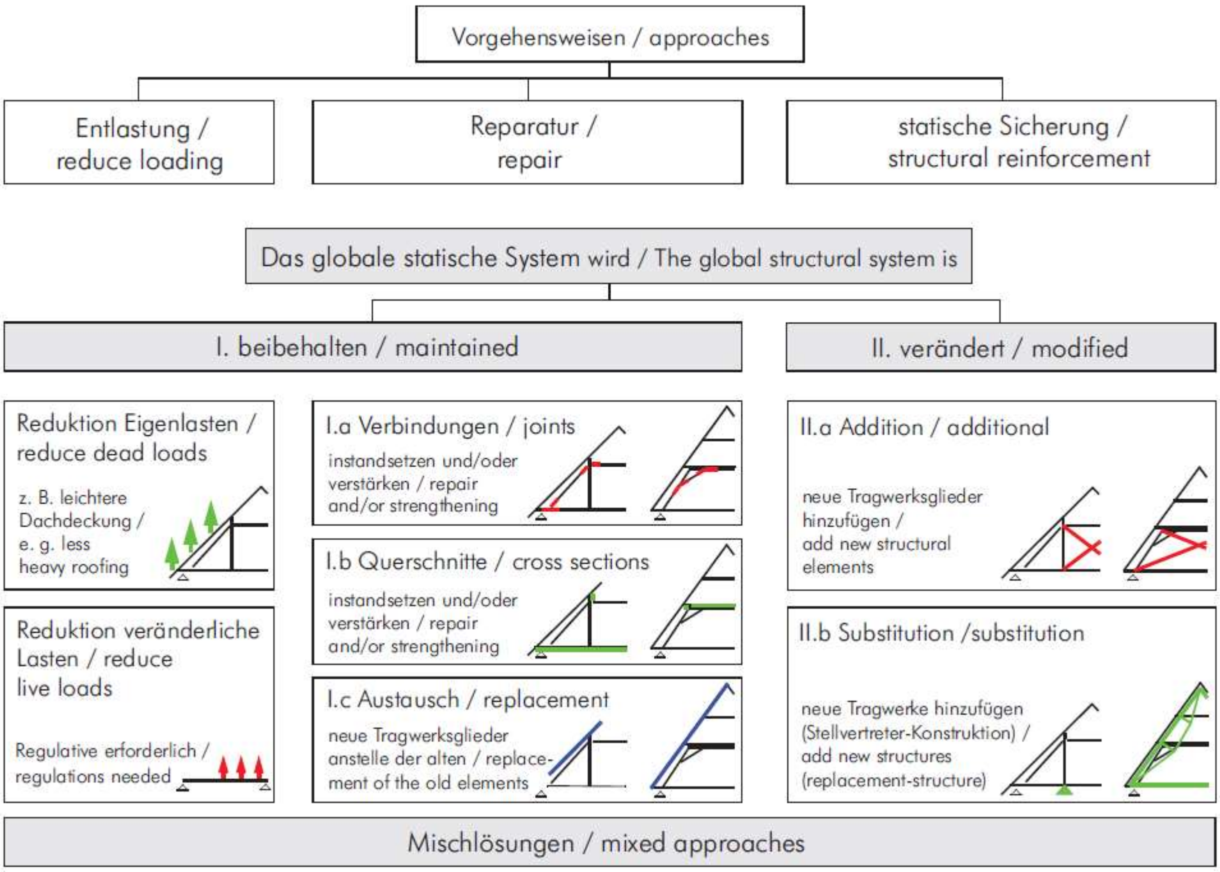


Figure 6.1 Approach to interventions in the timber roof structures according to Meisel [17]

Next, some common techniques used for strengthening of connections and structural elements in the historic timber roof structures are shown. As emphasised in ICOMOS and in other literature [80] [81] [82] [42], the less intrusive measures are often better as they can be substituted at any time. For strengthening and repair of the historic timber roof structures, the following materials are usually used.

Timber

Timber is the most used material in strengthening measures. The effectiveness of the timber in the strengthening measures is proven numerous times throughout history. Timber corresponds to the mentioned compatibility requirements and strengthening measures usually contain traditional approaches. The timber used in repair or strengthening should be compatible with timber used in original structural to avoid any incompatibilities following points should be respected:

- Used timber is dried out and have the same or similar moisture content to the timber stationed in the original roof structure.
- Timber from same specie to one used in the original roof structure should be used, or wood with similar density, the module of elasticity, and other mechanical and physical properties.
- For the historic structures of a great value using of timber from an old forest which date from time similar tot he erection of the structure, should be discussed, see ICOMOS

Steel, and stainless steel

First iron, later steel and stainless steel have been used in the timber roof structures for a long period of time, therefore usage of steel in strengthening measures is approved as long as steel doesn't become dominant constituent in the roof structure. Many traditional strengthening measures can be distinguished by using steel and stainless steel, as later will be shown. Moreover in strengthening measures steel is usually implemented together with timber as they are compatible, therefore compatibility requirement is satisfied. From the standpoint of reversibility strengthening measures from steel are highly reversible as they are usually fastened with removable fasteners (i.e. bolts, dowels). Downsides are lack of fire resistance in comparison to timber, although this problem can be partially solved by embedding the steel in timber so timber provides protection. Steel is also susceptible to corrosion, especially in contact with timber, this can be solved with using stainless steel over steel, although with a consequence of a bigger price. Moreover, the condensation can degrade the surrounding timber. To suppress mentioned downsides an engineer must account and solve these problems during the design process usually by shaping the detail to avoid problems of fire ability and condensation.

Good mechanical properties make the steel suitable for strengthening measures as less intrusive strengthening measures which can be hidden in the roof structure are enough to regain original load bearing capacity of the roof structure.

FRP

FRP is a modern type material with big tensional strength and pronounced linear behaviour resulting with brittle failure mode similar to wood in tension. The FRP is popular among engineers as they are easily applied, and with their characteristics are suited for the strengthening of connections and members in bending.

Nevertheless using of FRP is questionable on many levels such as durability, poor compatibility and reversibility, additionally, FRP doesn't represent the spirit of traditional techniques. Because of that many conservation offices prohibited the use of FRP in strengthening measures.

Epoxy resin

Epoxy resins are commonly used for repair of longitudinal and transversal cracks in timber. Principle of injection is shown in

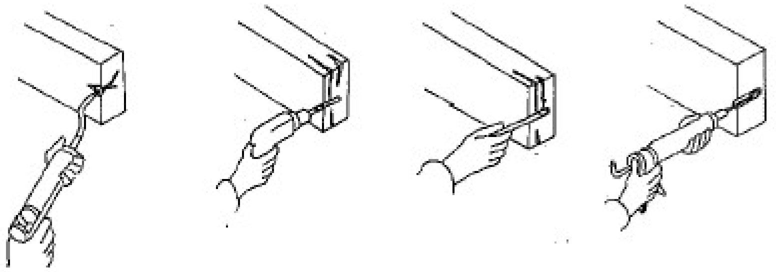


Figure 6.2 Injection of epoxy resin in the timber member [42]

6-1.1 STRENGTHENING MEASURES

6-1.1.1 Bad examples

Often poorly undertaken strengthening measures can be encountered during the assessment of the existing timber roof structures. They are the result of several shortcomings such as

- The bad initial design of the strengthening measures or poor execution on site
- Poorly defined goals of the preservation project, and poorly defined restrictions and requirements regarding the strengthening measures by the conservation office.
- Neglecting behaviour of the material, and material compatibilities
  During the assessment of the roof structure at TU Graz Alte Technik two major poorly undertaken strengthening measures are found. One of them is already described in Chapter 4-1.1, see Figure 4.3.
  In Figure 6.3 poor strengthening measure of the connection is shown. The connection of the brace to the tie beam is strengthened with a large steel plate and taped-screws. This design is faulty for several reasons:
- A big group of screws are screwed in a vastly cracked beam, inducing even more cracking and reducing strength.
- Steel plates are poorly shaped and may lead to condensation due to temperature differences and subsequently to degradation of the surrounding timber.
- The large still plate with a big group of screws restrains the rotation of the brace up to some extent, in that way changing the structural behaviour. This can lead to the redistribution of internal forces, resulting in even more loads on the connection.

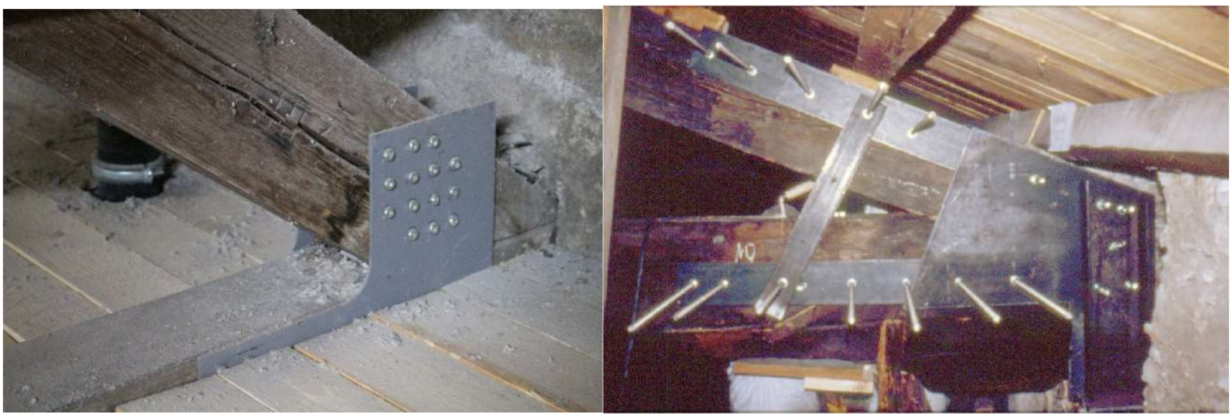


Figure 6.3 left: Poorly designed strengthening measure in connection of the brace to the tie beam; right: Large steel cage restrain rotation in connection-bad [82]

Although this kind of unprofessional repair is hardly impossible to encounter in heritage structures. This kind of approach should be avoided in the repair of any timber roof structure. One of the ways to overcome this problem is more control from local authorities (i.e. conservation office).

6-1.1.2 Good examples

Method	Appropriate for Historic Structures	Appropriate for Repair of Beam Ends	Appropriate for Repair/ Reinforcement of Truss Joints
Fasteners	Yes	No	Yes
Press-bended sheets	Yes	Yes	No
H-, T-, L-, I-shaped Profiles	Yes	Yes	Yes
Rods and prostheses	Yes/No [1]	Yes	Yes
Nail-plates	No	No	Yes

[1] This often depends on the used approach of a local conservation body.

Figure 6.4 Overview of appropriate strengthening measures

Prostheses

Prostheses are a quite common way of repair and strengthening of the historic timber structures. The method consists of removing the damaged part and replacing it with a carved and shaped prosthesis. The prosthesis is usually made from timber, although prosthesis from epoxy resigns are also possible for the small prosthesis (e.g.beam ends). Because timber is used in addition with metal fasteners the measure satisfies compatibility and reversibility requirements. Embedded steel plates with metal fasteners (bolts, dowels) are often used as a connecting element between the prosthesis and original structure, however traditional timber to timber connections secured with bolts are also possible as it is the case in Figure 6.5. The critical part of this method is a connection of prosthesis to the original structure as discontinuity is present there. The new connection should simulate the original state where the member was whole, therefore, the connection must be properly designed to obtain this kind of behaviour (i.e. usually rigid behaviour).

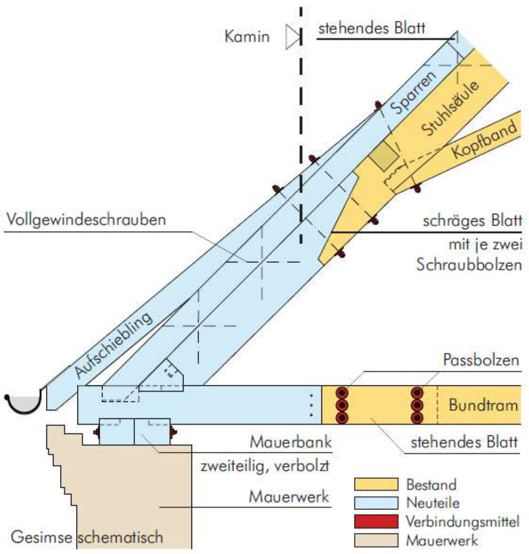


Figure 6.5 Strengthening measure „prosthesis“ [17]

Another alternative way of connecting the new and the old member is with glued-in rods. In connecting elements holes are bored. Holes are filled up with epoxy resin and the steel or FRP rods are inserted in the holes in adjacent members in that way members are connected. Glued in rods are acting are ensuring continuity between the old and the new member. Two members ready for connection with glued-in rods are shown in Figure 6.6.

Figure 6.6 Glued in rods [82]

Steel/stainless steel profiles and plates

The strengthening of the damaged or highly deformed member with steel or stainless steel strips, rods and profiles is the most common approach in the historic roof structures. The strips or rods can be either fastened with bolts or screws to faces of the member or embedded in member if embedded strips or rods are then bonded with epoxy resin, see Figure 6.7 up. The steel profiles can be placed in tension or compression, on one or both sides of the timber member. Usually U, L or I profiles without flanges are used as shown in Figure 6.7 down. The connection is established with bolts or screws. The effectiveness of newly created composite timber-steel member is highly dependant upon connection between timber and steel member. Also the problem of condensations on steel faces shouldn't be forgotten during the design (cf. [42] [80])

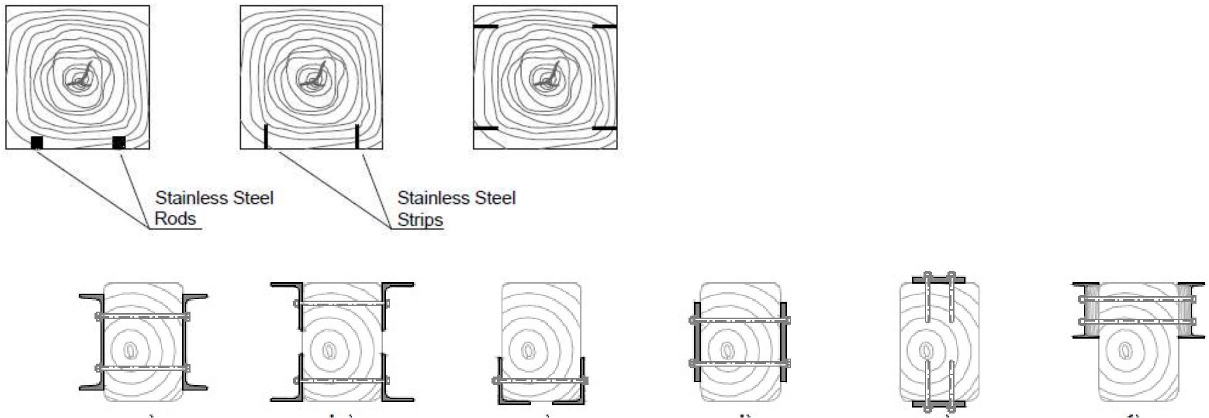


Figure 6.7 up: Examples of flexural reinforcement using bonded-in stainless steel rods or strips [80]; down: Examples of flexural reinforcement using steel / stainless steel profiles [42]

FRP strips

For strengthening with FRP usually, strips or plates glued with epoxy adhesives on the member faces or inserted in the timber members. FRP is usually used to strengthen the members subjected to bending. In order to provoke ductile failure of the members under bending, FRP is usually glued on the tension side of the member, therefore, failure of timber in compression occurs first. Moreover, FRP plates or stripes can be used for strengthening of the connections. If correctly implemented and glued the FPR strips and plates can improve strength and behaviour, but in other cases, they can induce serious damage to members due to large stiffness of glued contact where under moisture changes extensive cracking of the timber is possible in the member. Furthermore, local delaminations are possible especially if FRP is not glued correctly. Because of these downsides and poor reversibility, strengthening with using FRP stripes should be carefully accessed. (cf. [42] [80])

Figure 6.8 Strengthening measure with FRP strips [42]

Strengthening by Converting member into a supported member

With steel rods, it is possible to construct tensioned cable system. By tensioning the steel rods with turnbuckle it is possible to adapt the force discharged in places where cables support the members. This strengthening measure is especially efficient and suitable when extent deformations of the members or the whole roof structure are present. By adapting the force it is possible to reduce deformations as much is needed and also do the same in the future to counteract the influences of moisture changes. The examples of such strengthening are shown in Figure 6.9. When system of tensioned cables is implemented in structure it should be noted that whole structural behaviour is altered. (cf. [14] [80])

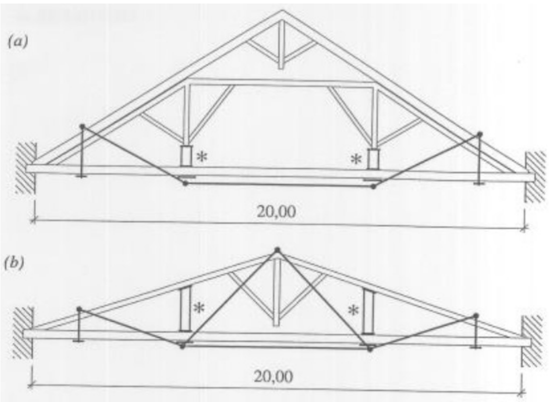


Figure 6.9. Strengthening by post tensioning [14]

6-2 STRENGTHENING ON THE STRUCTURE OF "*TU GRAZ ALTE TECHNIK*"

In Chapter 4 the damages observed during the survey of the roof structure TU Graz Alte Technik are shown and explained. Based on these damages and also later structural analysis the proposals for strengthening or repair measures are given here.

When assessing the state of the roof structure regarding observed damages but also results of structural analysis and verification of members and connections it can be concluded that the roof structure in every room is in great condition. This can be concluded by the fact that no damages from the overloading of members and connections or extensive deformations to the members were observed. The observed partial disconnections of connections are the result of dimensional changes and tension forces in short ties, as in original structure iron cramps were omitted the disconnections are expected consequence. Having in mind that the roof structure at TU Graz Alte Technik is almost 140 years old it can be assumed that peak loads which correspond to ULS already happened, although verification process in members and connections pointed to some critical points the author believes material properties in the structure are at least equal to characteristic values of timber (C24) or bigger. In that spirit and to preserve the value of the structure only minor repairs are proposed by the author.

1. **Implementing iron cramps on connections: brace-tie beam, brace-queen post and straining beam- queen post**

The cramps are actually planed in initial design as can be seen on original blueprints, however somehow on a majority of axes, these cramps are missing. Although iron crams are only constructive measure as tension forces in braces and straining beams were not observed during the analysis. Iron cramps can significantly improve behaviour of connections especially during earthquakes by transferring the tension forces and their ductility. This is observed in research done by Piazza&others obvious improvement in behaviour of the connection with iron cramps is shown in Figure 6.10. For more information about this kind of behaviour refer to the article itself [83]. Moreover iron cramps will restrict disconnection of notched connections which are currently present in the roof structure.

(Note: in Figure 6.10 the most similar to behaviour to iron cramps would produce binding stirrups.)

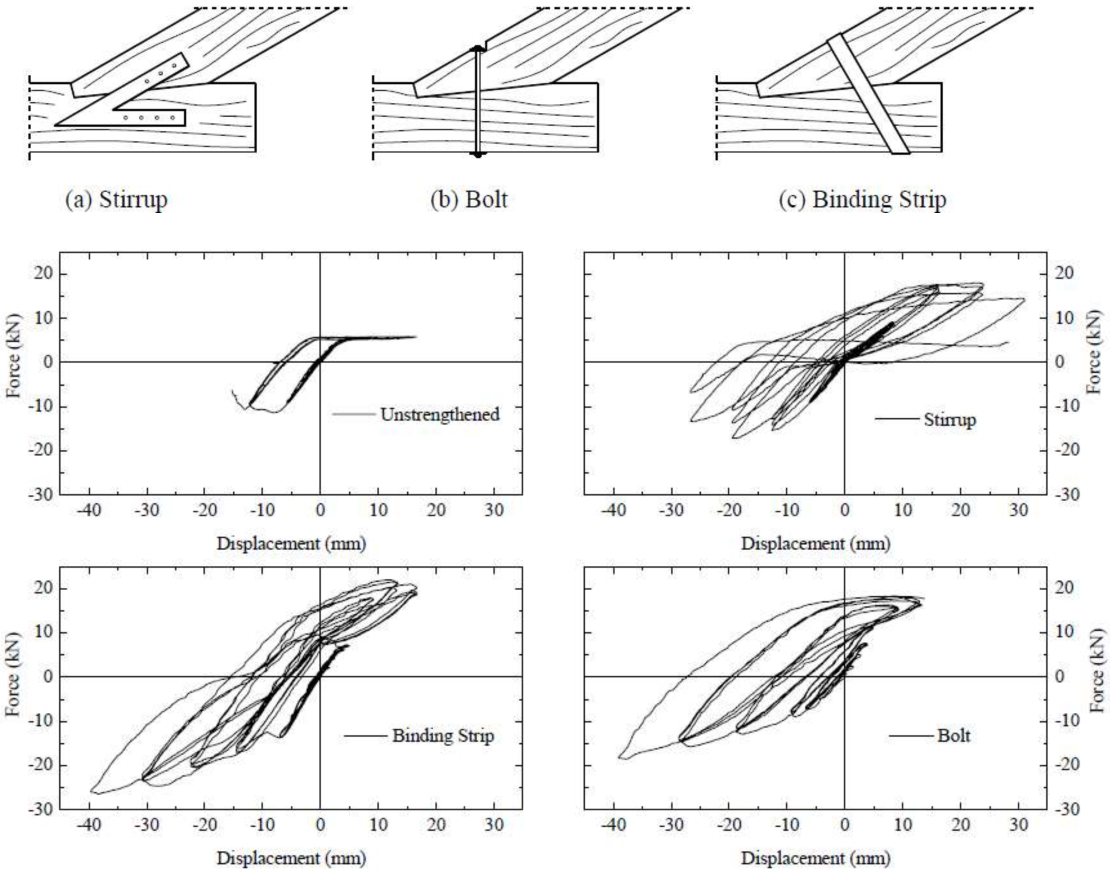


Figure 6.10. up: Sketches of the strengthened connections [83]; down: Force-displacement response for the cyclic loading [83]

2. **Cleaning the disconnected connections and restoring their initial geometry**

The disconnection in notched connections is presented in Figure 4.7. The gaps in connections should be cleaned so connections can fit in original positions. After cleaning the connections, the initial geometry should be restored by returning the braces in original position until full contact in notch is established. At the end gaps of 5 mm should be secured on horizontal plane if needed cutting of the timber is allowed. Detail of repaired notched connection is showed in Figure 6.11

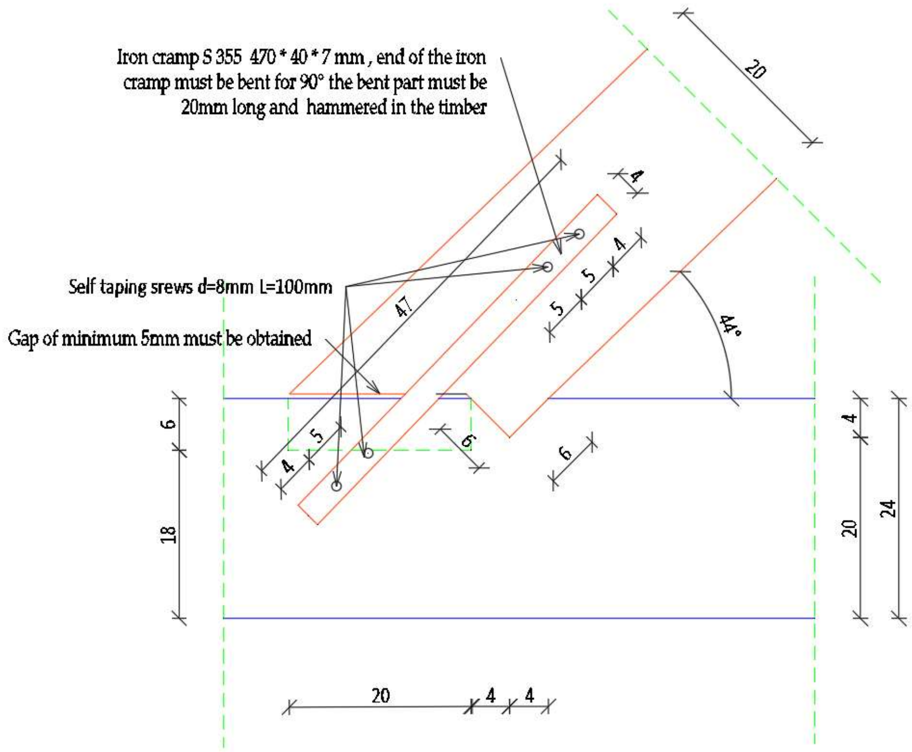


Figure 6.11 Detail of the repaired connection brace-tie beam

3. **Removal of the short tie on axis 3**

The existing short tie presents a bad example of the previous repair on the roof structure, the connection of the new short tie on the old short tie is poorly undertaken and the load bearing capacity of group of self tapped screws is unknown. Therefore the short tie should be completely removed and new short tie should be inserted. The geometry of the new short tie must be the same as other short ties in the roof. See Figure 4.3.

4. **Removal of the brace on axis 13**

The damaged and deformed brace on axis 13, Figure 4.1 should be removed and substituted with the brace of the same geometry as the remaining brace on that axis. The geometry of the notch must be accommodated to look the same as in Figure 6.11.

CHAPTER 7: CONCLUSION

By reviewing and comparing the roof structures from different parts of Europe, it could be concluded that the general division on rafter and purlin roofs was the same, but the crucial differences were observed in later subdivisions of structural roof types. The differences were usually distributed along with the speaking territories, although specific cases were observed. As a direct consequence of such differences, there was a lack of standardized terminology and translations regarding structural roof types and elements developed, which led to misunderstanding and confusion in the reader. As a result, the efforts, similar to the one given in Appendix B of this thesis, should be developed on a bigger scale where the differences would be systematically and chronologically presented with respect to time and territory.

Although global awareness of the protection of the historic timber roof structures developed through almost 60 years, some areas still face devaluations due to incompetence and a lack of legislation at the country and the European level regarding the preservation and conservation of the historic roof structures. This is especially prominent today when the need for living spaces significantly increased in already overcrowded cities. As a consequence, an enormous interest for the valorisation of attic spaces into living spaces emerged in the community. This threatens to devalue old town areas due to incompetent renovations where the original identity of the roof structures is not respected. On a bigger scale, this leads to a complete devaluation of the whole old town area. Although the outer appearance of the roofs is prescribed by conservators, in most cases the structural level is neglected. Therefore, the author suggests crucial changes and restrictions where this part of the roof structure is preserved, especially for roof structures in the old city areas, which are marked as cultural heritage. A good example of managing such problems was developed by the city of Graz.

When comparing different approaches of assessment, a pattern emerges, since each approach involves a holistic approach to the preservation project. However, it should be noted that not all approaches are suited for timber roof structures. Therefore, an approach specially made for timber roof structures is needed for future standards, similar to the approaches made by Cruz & others and Mesiel. Furthermore, during the assessment process, the author noticed a lack of a standardized approach to the survey of the structures, especially regarding tolerance. Thus, an attempt to develop a step by step process of measuring, inspection and surveying of the structure while obeying given tolerances is presented.

In Chapter 5, different models have shown how different simplifications and boundary conditions influence the distribution of force and behaviour of the roof structure. The connections present a critical part of the timber roof structure, therefore, most of the effort should be invested in the modelling of connections. In order to obtain realistic load redistribution, eccentricities and stiffnesses in connections must be calculated and implemented. Likewise, different approaches to modelling of supports showed that spring supports should be preferred over sliding or hinged supports for the same reason. The calculation of spring constants, in that case, depends directly upon stiffness and overturning load bearing the capacity of structural masonry walls. The author thinks that an effort to calculate load bearing capacity shouldn't be avoided, as the impact upon the results is huge, even if spring constants are determined in a more conservative way. According to Holz-Holz Verbindungen's research, the author still recommends a calculation of load bearing capacity to confirm spring constant values.

Furthermore, it can be concluded that purlin roofs carry loads as a combination of purlin and rafter roofs. This can be directly observed in analysed models, as different models produced different load carrying principle due to the considered simplifications.

Further research of connections should be conducted, as they prove to be critical to the overall behaviour. The author thinks that further research is needed for standardization of axial and rotational stiffnesses in the most common connections found in the roof structures (e.g notched, birdsmouth, dovetail joints).

Fasteners, such as wooden pegs or iron straps, also have an enormous influence. As a result, an approach suitable for implementation in future Eurocode norms can be developed. Moreover, the same effort should be made in verifying models of connections, which are not implemented in current Eurocode, and other norms which have some calculation models that seem to be too conservative for existing structures.

The results of the study can lead to new standards in the near future, create a basis for more realistic modelling and calculation of the historic timber roof structures and reduce invasive strengthening measures.

As for strengthening measures are concerned, an engineer should nourish traditional and less invasive methods which are in compliance with compatibility and reversibility requirements. Conservation offices should produce clear requirements in order to reach the best result. Also, further studies should develop standardized strengthening measures and their calculation. However, when referring to strengthening measures, this could not be completely possible as every repair represents a specific problem for itself.

APPENDIX A LITERATURE

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APPENDIX B TERMINOLOGY

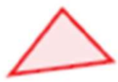
In order to provide easier comprehension of terms used in this paper, the German-Croatian-English translations for names of roof statical systems and structural elements mentioned in this work are given, together with short description of structural element and its function.

The translations are given by author opinion after reviewing different translations in following literature [17] [84] [14] [29] [85]. It should be noted that given translations still doesn't provide the standardised expressions and differences in expression between various literature are possible.

German-English translations of most common connections found in historic timber constructions are presented in Figure 2.20. The translations of roof statical systems are given in Table B.1., translations of structural roof elements found in historic timber roofs are presented in Table B.2.

For more terms translated on several languages refer to [84].

Table B.1. A review of historic roof structures characteristic for Central Europe (based on Figure 1.10) – typological and terminological specifics comparable with other European zones [17, 71, 14, 72, [86]]

English	Equivalent terms	Explanation	German	Croatian	Example
I. Rafter roofs (RR)		Closed roof based on sequence of independent trusses with horizontal tie-beam at the foot of (common) rafters. These main load-carrying trusses are arranged in the transverse direction of the roof. Wind braces ensure stability in longitudinal direction. In narrow sense, a term "rafter roofs" refers to common rafter roofs and collar beam roofs. In more broadly meaning, it covers the principal rafter roofs, as well.		Roženička krovišta / Krovišta roženičkog tipa (s pajantom i bez pajante)	

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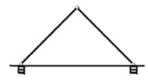
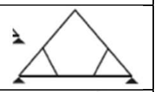
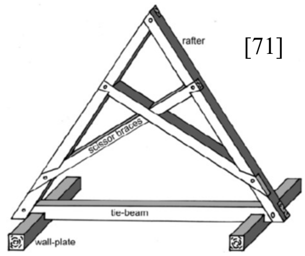
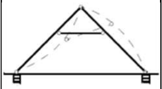
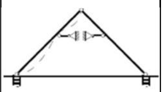
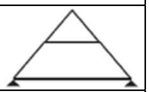
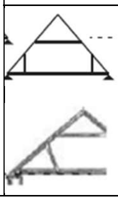
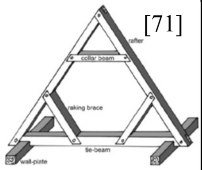
1. Common rafter roofs (RR-C)		The simplest structural type in which each of the transversal trusses represent a closed couple of rafters (a pair of rafters in conjunction with a tie-beam). Subtypes may have additional pair of members.	Sparrendach	Rozenički krov (bez pajante) – RK	
1.1 RR-C (simple)	Common rafter roof / Closed couple roof	Rafters rely on a tie-beam, supporting one another in the ridge and carrying the roof cover.		RK / jednostavna	
1.2 RR-C with additional struts	RR-C with raking braces	The additional transverse (inclined or vertical) compressive members rising from a tie-beam to support the rafters.		RK s kosnicima	
	RR-C with vertical struts			RK s vert. potporama	
1.3 RR-C with scissor braces * It could be considered as transitional form to the collar beam roofs.	Closed scissor roofs	X-shaped arrangement of braces connecting a pair of rafters. Scissor braces can reinforce or substitute collar beams. The tie-beam is horizontal member at the foot of the rafters, which differs this roof type from (open) scissor-braced truss with interrupted tie-beam, allowing vaulted (raised) ceiling area.		Škarasta RK krovišta  [71]	
2. Rafter roofs with collar beam RR-CB / Collar beam roofs		A RR with transversal member connecting two rafters on opposite sides of a roof (between the level of a tie-beam and the ridge). Collar beam is typically positioned in the upper third of the roof rise, receiving compressive stress. For subtypes, see also 2.2.		RK s pajantom / Pajantna krovišta (PK) Horizontalno pomični i ne-pomični PK	 Unbraced system  Braced system
2.1 RR-CB (simple)		Collar beam is fitted on each pair of rafters.		Pajantni krov (PK)	
2.2 RR-CB with additional struts	RR-CB with raking braces or vertical struts	Subtypes of collar beam roofs alike to those described in 1.2.		PK s kosnicima / vertikalnim potporama  [71]	

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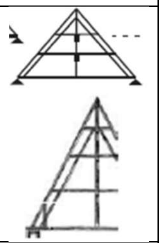
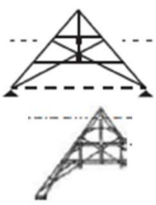
<table>
<tr><td colspan="6">3. Principal rafter roofs (RR-P) / Collar beam roofs with collar plates</td></tr>
<tr><td colspan="3" rowspan="2">A rafter roof (RR) in which the individual transversal trusses of the roof are not entirely independent, but connected and supported by longitudinal members such as collar plates.

Supporting collar beams, the collar plates never support rafters directly (unlike the purlins), but only indirectly, via collar beam.

The collar plates are supported by specially reinforced trusses – the "principal trusses“ (PT). “Secondary trusses" (ST), arranged as bays between two adjacent PT, do not contain elements such as collar struts (i.e. “chairs”, meaning an individual vertical members carrying collar plates), “chair structure / trestle truss” or king posts (as tension members). Typically, there is a regular sequence of PT and intermediate ST [71].

The additional members within PT can be employed to provide transverse stability to the roof whose stability in opposite direction is ensured by wind braces in conjunction with longitudinal truss system.</td><td></td><td>Pajantna krovišta s visuljama</td><td>
PT with suspended / hanged posts</td></tr>
<tr><td></td><td>Pajantna krovišta sa stolicama</td><td>
PT with either collar struts or “lying chair truss / trestle truss“</td></tr>
<tr><td colspan="2">3.1 RR-P with suspended / hanged posts

* Historic roofs of this type were mainly constructed as multi-storey structures, a common solution for church / cathedral roofs or high-status buildings.</td><td>A principal truss contains king posts, providing an additional support at the centre of a tie-beam or collar beam. Such subtype represents “closed” roof with a continuous tie-beam, while king posts in conjunction with scissor-braces are specific for “open” truss subtype with dis-continuous, interrupted tie-beam, creating a vaulted ceiling area.</td><td></td><td>Pajantna krovišta s visuljama</td><td>
PT with king posts (“closed roofs”) and those with additional X-braces / “open” truss</td></tr>
<tr><td>3.1.1 RR-P with king posts</td><td>Closed RR-P with king posts</td><td>The top end of the king post is typically joined to the ridge of the roof. Passing braces provide a stable triangular form of the PT.

In steep high roofs, the additional vertical struts can be found supporting rafters.</td><td></td><td>Pajantna zatvorena krovišta s visuljama</td><td></td></tr>
<tr><td>3.1.2 RR-P with king posts and scissor braces</td><td>Open RR-P with king post and scissor-braced truss</td><td>Specific structures, characteristic for aisled construction. They have “rafter foot” – a triangulated configuration at the base of rafter consisting of ashlar piece, sole piece and rafter. Acting as the “open truss”, this type of roof which do not have a tie-beam is much more sensitive to the modelling assumptions than are the roofs with rafters – a tie-beam closed triangle [73] 1).</td><td></td><td>Pajantna otvorena škarasta krovišta</td><td></td></tr>
<tr><td colspan="6">1) The loads of a scissor-braced truss may either be carried by a simple triangulated, statically determinate truss constituted by the scissor braces and the rafters, with the essential contribution of the king-post, or, alternatively, by some kind of a polygonal arch. The transition between these two distinct and essentially incompatible load-carrying structures highly depends on modelling assumptions and stiffness ratios [73].</td></tr>
</table>

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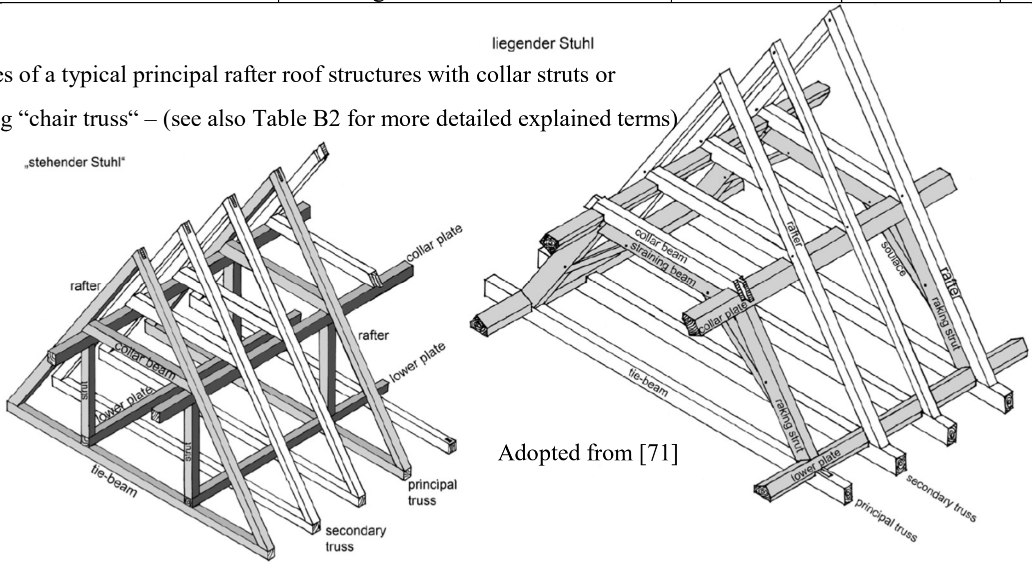
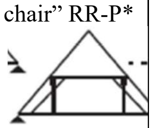
<table>
<tr>
<td colspan="2">3.2. RR-P with collar struts or “chair / trestle trusses”

*A typical Central European roof truss arrangement, especially characteristic for German-speaking areas [2].</td>
<td>Also known as either “standing chair” RR-P (with single or double vertical struts and the same number of collar plates) or “lying chair (trestle truss)” RR-P (as a structure with raking struts which form trapezoidal portal frame in conjunction with soulaces and a straining beam [2].</td>
<td></td>
<td>Pajantna krovišta sa stolicama</td>
<td></td>
</tr>
<tr>
<td colspan="6">[2] Examples of a typical principal rafter roof structures with collar struts or supporting “chair truss“ – (see also Table B2 for more detailed explained terms)


Adopted from [71]

[3] “Liegender stühl” is a truss of German origin, so the original German name is frequently used. The literal translation is “lying chair”, where “lying” refers on angled or leaning struts, and “chair” is used in sense of a support structure (in this case a truss). The opposite being a “Stehender stühl”, which is common roof truss type where simple vertical struts replace the more complex support structure of “liegender stühl” [71].</td>
</tr>
<tr>
<td rowspan="2">3.2. 1 RR-P with “standing chairs“ [4]</td>
<td>Collar beam roof with standing chairs

Collar beam roof with single / double / triple supported collar beam

RR-P with Stehender Stühl</td>
<td>PT is constituted by closed rafter couple. The tie-beam supports vertical struts (i.e. “collar struts“, sometimes resting on lower plates) which in turn support longitudinal, horizontal members ("collar plates") on which the collar beams rest [71].
Subtypes with double-chairs can have added bracing members (i.e. “passing braces”, parallel to rafters).
Subtypes with multi-collar beams may contain double standing chair with braces and triple ones at lower portions of high roofs.</td>
<td rowspan="2">Kehlbalken-dach mit einfach / doppelten stehendem stühl</td>
<td rowspan="2">Pajantna krovišta s vertikalnim stolicama</td>
<td>
Single “standing chair” RR-P*

Double ”standing chairs“ RR-P (braced example)</td>
</tr>
<tr>
<td colspan="2">[4] ”Crown-post roofs“ with a central vertical strut has got certain similarities to a ”single standing chair“ RR-P. As typical medieval structures, they were constructed with braces between the collar beam and common rafters and from the crown post to these braces. In this type of RR-P, braces have not only structural (racking prevention), but also ornamental function [72].</td>
<td>
Subtypes for multi-storey (steep) roofs</td>
</tr>
</table>

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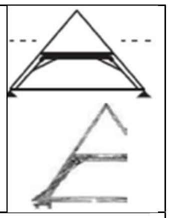
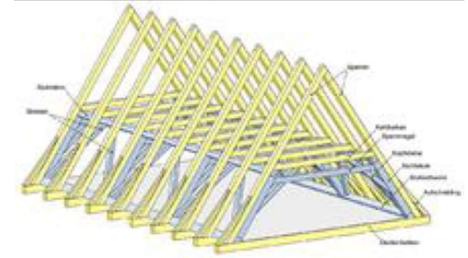
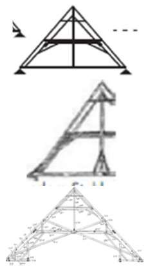
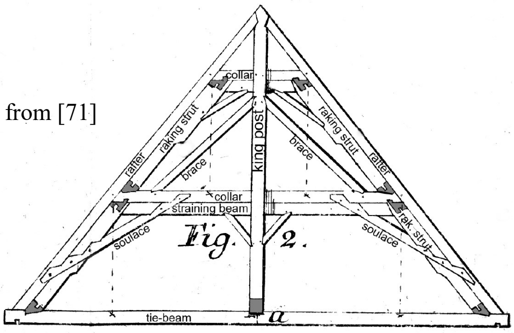
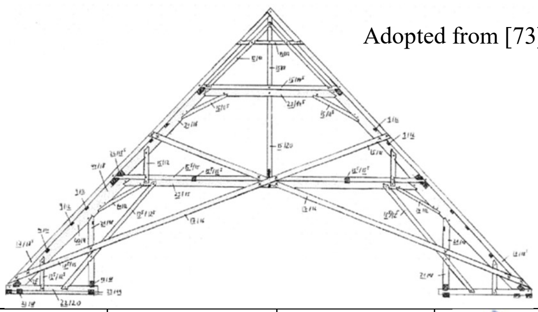
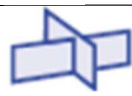
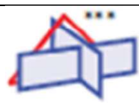
<table>
<tr>
<td rowspan="2">3.2.2 RR-P with “lying chairs“</td>
<td rowspan="2">Collar beam roof with trestle truss

RR-P with liegender Stühl</td>
<td rowspan="2">A principal truss is constituted by a tie-beam, two raking struts following the slope of the roofs (these leaning struts are supported on a tie-beam via lower plates), the horizontal member (”straining beam”) connecting the raking struts and soulaces (i.e. corner braces) [71].
This kind of configuration makes a “chair structure / trestle truss” acting as a portal frame which supports longitudinal members ("collar plates") on which the collar beams rest [71].</td>
<td>Kehlbalken-dach mit liegendem stühl</td>
<td>Pajantna krovišta s ležećim stolicama</td>
<td></td>
</tr>
<tr>
<td colspan="3"></td>
</tr>
<tr>
<td colspan="2">3.3 Combined RR-P</td>
<td>Multi-storey structures which are constructed combining the structural systems with king post and “liegendem stühl“ (closed roof). Combination of “liegendem stühl“ and scissor-braced roofs can be found in many sacral buildings (i.e. churches or cathedrals) [4].</td>
<td></td>
<td>Kombinira-na pajantna krovišta s ležećim stolicama</td>
<td></td>
</tr>
<tr>
<td colspan="6">[4] Examples of combined RR-P structural systems with closed and open principal trusses [71, 73]
Adopted from [71]


Adopted from [73]
</td>
</tr>
<tr>
<td colspan="2" rowspan="2">II. Purlin roofs (PR)</td>
<td rowspan="2">Roof whose roof cover is supported by horizontal members which run parallel to the ridge direction (i.e. longitudinal roof direction). These members are called purlins [71].
In traditional timber roofs, purlins are longitudinal members which directly support rafters, connecting two opposite gables. Typically, they must be supported at regular intervals by either masonry diaphragms (structural walls) or by (main) transversal trusses, the principal trusses (PT). Secondary trusses (ST) represent an intermediate bays, a transverse trusses resting on purlins [71].</td>
<td></td>
<td></td>
<td>
King-post & Palladiana / Queen post truss: Romanic types (II.1)</td>
</tr>
<tr>
<td>Pfettendach</td>
<td>Podrože-nička krovišta</td>
<td>
King / Queen post trusses: Germanic types (II.2)
Purlin-tie roofs and purlin trestle roofs (II.3)</td>
</tr>
</table>

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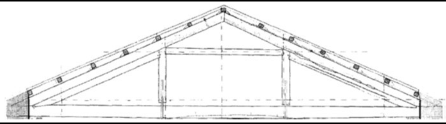
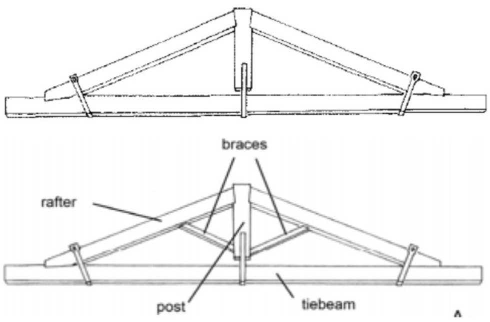
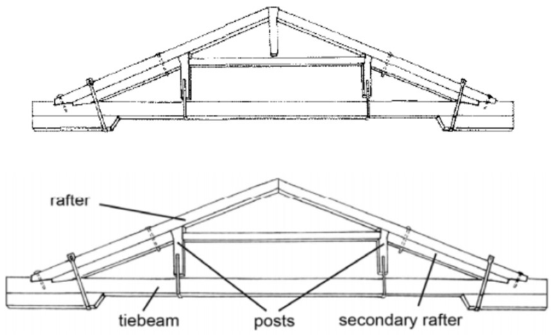
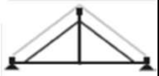
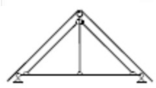
<table>
<tr><td colspan="6">1. King-post truss & Queen post truss / “Palladiana” truss (Romanic types)</td></tr>
<tr><td colspan="2" rowspan="2">* Representatives of the historic PR which are characteristic for Mediterranean region 5).</td><td rowspan="2">Typologies are of the Roman origin and have been improved during Medieval age. Both structural systems differ significantly from the homonymous roof trusses, characteristic for Central Europe.</td><td>Hängesäule</td><td>Rešetkaste jednostruke i dvostruke visulje</td><td></td></tr>
<tr><td colspan="3"></td></tr>
<tr><td colspan="6">5) Traditional “King post and Queen post / Palladiana trusses


Trusses with king post only / king post plus braces [28]


(Simple) “Palladiana“ truss [28]
King post truss – the simplest of trusses which has two principal rafters, a tie-beam and a central vertical king post. It is commonly used in conjunction with two angled / inclined struts (braces) which provide stability, supporting the rafters, as well. King post – vertical tension member providing additional support at the centre of a tie-beam. The top end of the king post is typically joined to the ridge of the roof [28, 71].
Queen post truss (socalled “Palladiana”) has many variations. The simple “Palladiana” has two principal rafters, two secondary (under-)rafters which run parallel to the principal rafters, a tie-beam, two queen posts and straining beam between the posts. Queen post – vertical tension member employed to provide intermediate support for a tie-beam. Queen posts do not reach the ridge, but join the rafters at some intermediate point between the ridge and the eaves. Straining beam is transverse member in compression which braces secondary rafters of opposite sides of the roof against each other [28, 71].</td></tr>
<tr><td colspan="6">2. PR with hanged / suspended posts: King post & Queen post PR (Germanic types)</td></tr>
<tr><td colspan="2">* Representatives of the historic PRs which are characteristic for Central Europe. They were developed in 18th century, having the German origin.</td><td>King post or queen post PR here refers on single or double hanged posts and the same number of the purlins6) other than foot purlins, as well, supporting the rafters. Foot purlins rely on a tie-beam, enabling the rafters to overhang.</td><td></td><td>Jednostruka i dvostruka visulja</td><td></td></tr>
<tr><td colspan="6">6) A pair of passing braces (each is more or less parallel to rafters) in conjunction with a tie-beam provides transverse stability to the principal trusses, so they act as braced system. Thanks to the system of “purlin – knee brace – post” (longitudinal bracing truss / wall), the longitudinal stability of the roof is ensured, and wind braces may be omitted (in contrast to rafter roofs).</td></tr>
<tr><td>2.1 King post PR / (Germanic)</td><td>King-post truss (Germanic)</td><td>Ridge purlin supports the rafters and transfers roof loads into king post within PT. In conjunction to a tie-beam, passing braces provide a stable triangular form.</td><td>Einfach Hangewerke</td><td>Jednostruka (trokutasta) visulja</td><td>
</td></tr>
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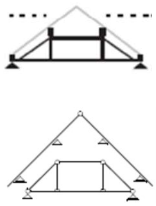
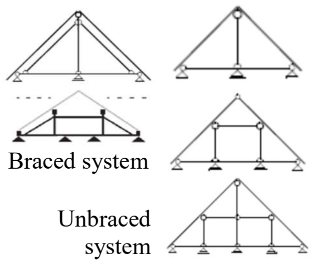
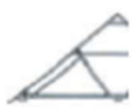
2.2 Queen post PR (Germanic)	Queen-post truss (Germanic)	Middle purlins are inter-mediate supports of the rafters transferring the roof load into queen post within PT. Passing braces and a tie-beam in conjunction with a straining beam provide a stable trapezoidal form.	Doppelte Hangewerke	Dvostruka (trapezna) visulja	
3. Purlin trestle and purlin-tie roofs – ”Standing chairs” PR; PR with sloped struts: ” Lying” and “Raking chairs”					
* Representatives of the historic purlin roofs which are characteristic for Central Europe since 18th century. The “chair” is a commonly used literal translation of the original German term “stühl”. Due to the relatively limited presence in other European regions, the original German names of these roofs are frequently used, as well.		In narrow sense, the “chairs” here indicate either vertical (“standing / stehender”) struts (compression posts) or sloped ones (“lying / “liegender” struts and/or “raking” struts – these two types mutually differ according to the way of transferring the load to the walls). The number of “chairs” coincides with number of purlins, other than foot ones. A collar-tie is transverse member which connect “struts” with rafters.[7] In a wider sense, “lying chair” indicates a “purlin trestle roof” with complex “chair structure”.	Pfettendach mit stehendem oder liegendem und Bock Stühl	Krovišta “stolica” – uspravne / stajaće stolice, kose stolice, kose ležeće i ležeće stolice	
[7] From the viewpoint of the statics performances and transverse stability, “standing chairs” may be classified either as braced or unbraced systems, depending of the existence of passing braces. The principal trusses with “raking / laying strut” and “lying chairs”, as well, are braced systems, by default. Truss system of “purlin – knee brace – “strut” provide the longitudinal stability to the roof, making wind braces unnecessary.					
3.1 PR with “standing chair”	Purlin-tie roof with standing chair(s)	Number of the “chairs” varies from one to three (e.g. single, double or triple chairs, depending of the roof span). Internal structural wall must support a tie-beam right bellow the “chair” (or close to that point) reducing the bending (deflection) of a tie-beam on which the “chair” relies.	Pfettendach mit stehendem Stühl	Jednostruke / dvostruke / trostruke (uspravne) stolice	 Braced system Unbraced system
3.2 PR with “raking chair”	Purlin-tie roof with raking chairs	This roof type with pair of raking struts which are supported toward the central part of a tie-beam. It is applicable for the floor plans with one or two inner wall(s), although the usability of the attic space is somewhat limited. Static systems can have braces parallel to rafters. They offload the raking struts enabling extra stability, too.	Pfettendach mit Bockptefften stülen (Bockpfetten - dach stühl)	Kose stolice	 Unbraced system  Braced system

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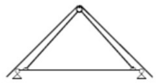
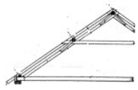
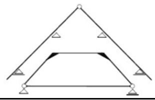
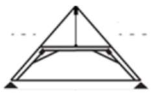
3.3 PR with "lying chair": purlin – tie roofs and purlin trestle roofs					
3.3.1 PR with "lying struts"	Purlin-tie roof with lying chairs	The "chairs" here indicate a pair of sloped struts which run parallel to rafters, transferring the load only to the outer structural walls. Presented system of double lying struts which run toward to the ridge having a collar-tie bellow the middle purlins differs from the original "liegendem stühl" structure.	Pfettendach mit (einfach / zweifach) liegendem stühl	Kose ležeće stolice	 Single lying struts  Double lying struts
3.3.2 PR with "lying chair"	Purlin roof with trestle truss	Purlin roof with complex "lying chair structure" acting as a trapezoidal portal frame.	Pfettendach mit Liegen-dem Stühl	Ležeća stolica	
4. Combined PR		Purlin roofs in which king post (above collar-tie) is combined with either "liegendem stühle" (trestle truss) or braced raking chairs (mostly when there are two inner walls in the floor plan). Such combined systems can be found in roofs of bigger spans and with long rafters.	Pfettendach mit – - Liegendem Stühl und Hängesäule -Bockpteftten stülen und Hängesäule	Kombinira-na krovišta ležećih stolica s visuljama i kosih stolica s visuljama	 

Table B.7.1. Terminology relating to structural elements [71, 72]

German	Croatian	English	Explanation
Kniestock	Nadozid	Attic wall	Short wall which rests on abutting structural walls of construction. Used to increase height of the attic.
Dachlattung	Krovne letve	Battens (laths)	Wooden laths used to form framework for mounting the roof tiles. Usually small cross section 3*5cm,2*3cm. Placed on small distance depending on type of covering.
Strebe	Kosnik	Brace	Inclined member usually around 35°-45°, provides bigger lateral stability and stiffness in roof structures such as: braced purlin-tie roofs with standing / raking chairs. In Queen and King post roof it is constitutive member of the principal truss.

Stuhlwand	Uzdužni stabilizacijski sustav (zidni)	Braced longitudinal wall (Longitudinal bracing frame/truss)	Longitudinal truss system made from Purlin / Collar plate, posts, and knee braces. Provides longitudinal stability of roof structure which mandatory exists in, principal rafter roofs and purlin roofs (see Table B1)
Kehlbalken	Pajanta	Collar beam	Horizontal member in each transversal truss within rafter roofs which provide support for rafters, always loaded in compression. Where it is part of bracing system which is usually positioned in the level of collar beams themselves (as additional to common wind bracing), provides stiffer supports to rafters.
Rähm (Stuhlrähm)	Podvlaka	Collar plate	Longitudinal member supporting collar beams. Supported in places of principal trusses, together with knee braces forms braced longitudinal wall.
Zangen	Kliješta	Collar tie	Member loaded in tension- prevents rafters from separating during uplift. In purlin-tie roofs it is positioned under the middle purlins. As a essential part of principle truss, it connects the “chairs“ / “chair truss“ with the rafters (see Table B1) .
Leergespärre	Sekundarni nosivi krovni sustav	Common truss (Secondary truss)	Common roof truss usually placed in distance of 0,7-1,0m supported by purlins or collar plates, in rafter roofs rest on tie beam, while in purlin roofs tie beams in secondary truss are omitted. Expression common rafter marks the rafters which belong to the common trusses.
Stuhlsäule	Stup	Strut “chair” (literal translation)	Compressive member supporting collar plate or purlin (in purlin-tie roofs). Depending of the type of principal truss, strut represents a main member (either vertical or sloped / inclined) main member, but also can be constitutive part of more complex system (see Table B1).
Kopfband Steigstrebe	Ruke	Knee brace Soulace (Arm brace) (Angle brace)	**Knee brace** = Short member usually under 45°angle connects (purlins/collar plate/) with (posts/struts/) in order to provide truss system knee-brace-purlin and provide longitudinal stability by forming longitudinal braced wall. **Steigstrebe** (Found in Kehlbalken-dach mit liegendem Stuhl). Long diagonal brace which connects lower plate and collar plate and passes by post, forming a braced longitudinal wall. It lays in rafter plane.

Schwelle	Podvlaka	Lower plate	Longitudinal structural member which rests on a tie-beam and supports a collar strut, preventing the concentrate loading of a tie-beam.
Vollgespärre	Glavni nosivi krovni sustav	Principal truss (Main truss)	Transversal truss of a roof carrying purlins, supporting collar plates, or containing "Stuhl" structures (in sense of „lying chair“ / trestle truss) [71]. Speaking of „closed truss“, it includes a tie-beam, by default. Expression principal rafter marks the rafters which belong to the principal trusses.
Pfette	Podrožnica	Purlin	Longitudinal member which is supported by posts/struts in principal trusses. It supports the common rafters. Depending upon place in the roof following purlins can be distinguished: (foot, middle, ridge) purlin
Hängesäule	Stup	Queen post/King post	Vertical member subjected to tension, provides support to tie-beam or (sometimes) collar beams in structural systems characteristic for Queen post and King post roofs. King post can be found in combined roof types, too (see Table B1).
Sparren	Rog	Rafter	*“Sloped beam carrying the roof cover. Typically, two rafters form a closed couple (triangular truss) together with the tie-beam. The rafter typically receives bending and compressive stress.” [84]*
Sparrenstreben	Kosa potpora	Raking strut	Used in rafter roofs (which are raised on attic wall) to support rafters. It removes trust from attic walls and reduces span of rafters.
Kreuzstrebe	“škarasti / križni kosnici”	Scissor braces	X shaped braces in common rafter roofs with scissor braces which provide support to rafters. They reinforce of substitute collar beam in collar beam roofs, too. Also, they represent a constitutional members in more complex, combined roof types (see also Table B1)
Spannriegel	Razupora	Straining beam	Horizontal member placed either between the queen posts or the raking struts, as constitutional members of “lying chair truss” (see also Table B1). In both cases, it is subjected to compression stress.

Zange	“Mala kliješta“	Short collar tie	Mainly horizontal element subjected to tension. They are commoly used to connect passing braces and the rafters when they are supported over wall plates on attic walls. Receiving horizontal trusts from the rafters, the “short collar ties“prevent their unforable effect on attic walls (eg. spreading / overturning).
Bundtram	Vezna greda	Tie beam	Horizontal structural element subjected to combination of tension and bending, receives loads from principal truss and transfers it to the supports of the roof structure. In rafter roofs tie beam provides support for rafters.
Mauerbank	Nazidnica	Wall plate	Longitudinal member placed on which tie beam rests. It acts as a support to tie beams.
Windrispe	Vjetrovni vez	Wind brace	Diagonal oriented laths or boards which lay in rafter plane. Used only in rafter roofs. Wind braces provide longitudinal stability for the roofs structures and contribute to their stifness.

APPENDIX C VERIFICATION PROCCES AND MODELS

C-1 Verification process

Verifications are conducted in room 2 according to model M1&2. Verifications are done for remaining cross section area.

ULS calculation-verification summary for model M1

MODEL M1																	
Structural element	Axis	Internal force						Area of cross section		Verification					Combination		
		N(+) [kN]	N(-) [kN]	Vy [kN]	Vz [kN]	My [kNm]	Mz [kNm]	Bruto area	Neto area	N(-) 0	N(+) 0	M y-y	M z-z	Shear	M+N(+)	M+N-(yy)	M+N-(zz)
Tie beam	8		57	42		13.56	1	408	378/328		0.16	0.84	0.06	1.30	1.06		
Brace	8		74.7	6.78		11.6		357	279	0.17		0.72		0.24		0.97	1.27
Short tie	8	19.26						360	252		0.07						
Queen post	8	7	34.9	44.29		4.2	0.8	289	197	0.02	0.16	0.52	0.07	1.91	0.75	0.59	0.46
Strut (purlin-tie roof)	12		38.62	5		5.5	3	289	197	0.12		0.68	0.26	0.22	0.94	1.11	0.90
Straining beam	8		50.89					340	340	0.09						0.31	0.39
Rafter	8 and 12		10.43	4.75		3.93		195	156	0.03		0.44			0.44	0.54	0.43
Knee brace	12	2	29			1		195	195	0.09	0.01	0.11			0.12	0.21	0.17
Middle purlin	8N-9N			14.5		11.06	3.5	340	267			0.68	0.22	0.63	0.90		
Foot purlin	8			5.48			5.31	357	292				0.28	0.240			
Collar beam	11		15					360	360	0.03						0.09	0.12
N(-)-Compression force , N(+)-Tension force		M+N(-) = Bending My-y + Mz-z + Tension 0						M+N-(yy) bukling around axis y-y + bending					M+N-(zz) bukling around axis z-z + bending				

ULS calculation-verification summary for model M2

MODEL M2																	
Structural element	Axis	Internal force						Area of cross section		Verification					Combination		
		N(+) [kN]	N(-) [kN]	Vy [kN]	Vz [kN]	My [kNm]	Mz [kNm]	Bruto area	Neto area	N(-) 0	N(+) 0	M y-y	M z-z	Shear	M+N(+)	M+N-(yy)	M+N-(zz)
Tie beam	8		69.81	43.12		17.19	1	408	378/328		0.17	0.66	0.05	1.34	0.88		
Brace	8		87.63	5.75		6.68		357	279	0.19		0.42		0.20		0.70	1.18
Short tie	8	14.37						360	252		0.05						
Queen post	8	8.08	52.8	4.84		2.04	0.8	289	197	0.03	0.24	0.25	0.07	0.21	0.56	0.33	0.28
Strut (purlin-tie roof)	12		30.26	4		3.77	1.92	289	197	0.10		0.46	0.17	0.17	0.63	0.78	0.62
Straining beam	8		50.26					340	340	0.09						0.31	0.38
Rafter	8 and 12		8.45	5.15		4		195	156	0.03		0.44			0.44	0.52	0.41
Knee brace	12	0.2	30			0		195	195	0.10	0.00	0.00				0.10	0.10
Middle purlin	8N-9N			18.94		8.24	1.86	340	267			0.51	0.12	0.83	0.62		
Foot purlin	8			4			3.2	357	292				0.17	0.175			
Collar beam	11		27.12					360	360	0.05						0.16	0.22
N(-)-Compression force , N(+)-Tension force		M+N(-) = Bending My-y + Mz-z + Tension 0						M+N-(yy) bukling around axis y-y + bending					M+N-(zz) bukling around axis z-z + bending				

C-1.1 Verification of connections

Next verification of ULS for connections are shown. Design forces are taken from model M1. Overall utilization factors are shown in … Verifications are made according to EN-1995-1-1 [78] DIN-1052-1-2004 [79].

Following some expressions which are valid for all calculations are shown, therefore same will not be shown in calculation steps.

$kc_{90} = 1.5$

$kc_{r} = 0.67$

$\rho_k = 420\ kg/m3$

$f_{c,0d} = 14.54\ MPa$ Compression along the grain (design)

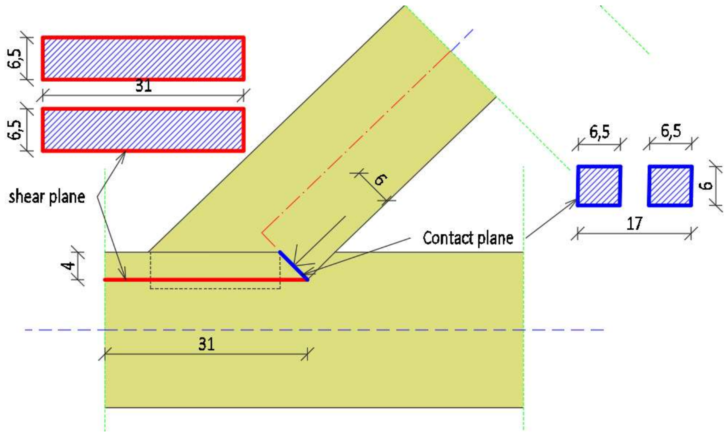
$f_{c,90d} = 1.73\ MPa$ Compression perpendicular to the grain (design)

$f_{v,d} = 1.59\ MPa$ Shear strength (design)

$f_{v,roling,d} = 2 * f_{t,90d} = 0.55\ MPa$ Rolling shear strength (design)

$f_{c,\propto} = \frac{f_{c,0d}}{\frac{f_{c,0d}}{kc_{90} * f_{c,90d}} * \sin(\propto)^2 + \cos(\propto)^2}$ Compression strength at an angle to the grain according to EN-1995-1-1 [78]

Brace-tie beam



Compression on a contact plane

$\propto = 44°$ Angle to the grain

$f_{c,\propto} = 4.5\ MPa$

$A_{eff} = 2 * (60 * 65) = 7800mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 7800 * 4.5 = \mathbf{35,2\ kN}$ Connection capacity at a contact plane

Shear

$t_v = 40\ mm$ Depth of the notch

$l_{v,max} = 8 * t_v = 8 * 40 = 320\ mm$ Length of shear plane (maximum allowed)

$\boldsymbol{l_v = 310\ mm}$ Length of shear plane

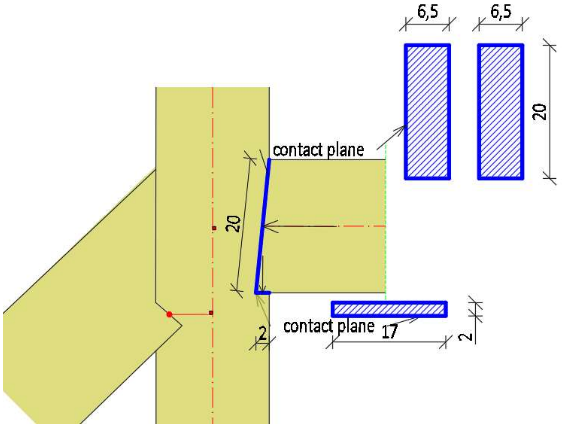
$A_{shear} = 2 * (65 * 310) * kc_r = 27000 mm^2$

$F_{shear,d} = A_{shear} * f_{v,d} = 27000 * 1.59 = \mathbf{42,93\ kN}$

Queen post-brace

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 7800 * 4.5 = \mathbf{35,2\ kN}$ **Same as brace-tie beam**

Straining beam-queen post



Compression on a contact plane – horizontal force (vertical contact plane)

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 2 * (65 * 200) = 26000\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 26000 * 2.6 = \mathbf{67,47\ kN}$ Connection capacity on a contact plane

Compression on a contact plane – vertical force (horizontal contact plane)

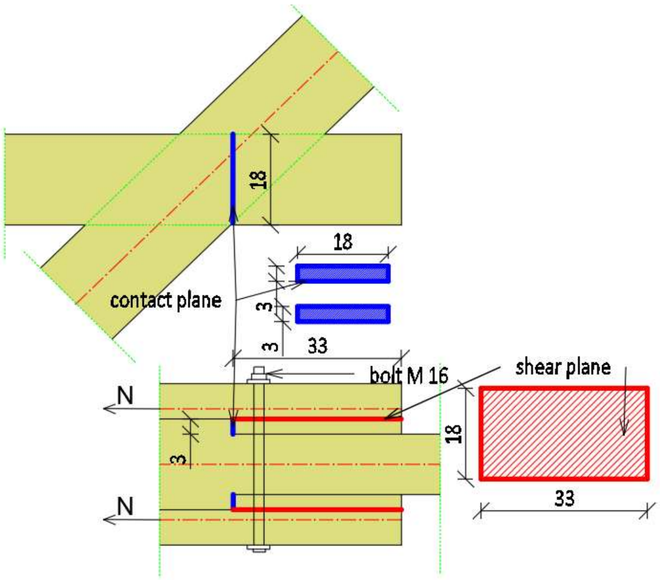
$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 20 * 170 = 3400\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 3400 * 2.6 = \mathbf{8,82\ kN}$ Connection capacity on a contact plane

Short tie-brace



Compression on a contact plane

$\propto = 44°$ Angle to the grain

$f_{c,\propto} = 4.5\ MPa$

$A_{eff} = 2 * (30 * 180) = 10800\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 10800 * 4.5 = \mathbf{48,75\ kN}$ Connection capacity on a contact plane

Shear

$t_v = 30\ mm$ Depth of the notch

$l_{v,max} = 8 * t_v = 8 * 30 = 240\ mm$ Length of shear plane (maximum allowed)

$\mathbf{l_v = 240\ mm}$ Length of shear plane *(maximum permissible)*

$\mathrm{A_{shear}} = 2 * (180 * 240) * kc_r = 57888\mathrm{mm}^2$

$F_{shear,d} = A_{shear} * f_{v,d} = 86400 * 1.59 = \mathbf{92\ kN}$

During surveying and assessment extensive cracking in one short tie is found. By analysing the crack residual cross section is determined. Residual cross section has 20% of initial cross section, therefore shear capacity at that location is equally reduced. Refer to Figure 4.6

$F_{shear,reduced} = F_{shear,d} * (0.67 - 0.2) = \mathbf{43,24\ kN}$ Shear capacity

Bolt

$d = 16\ mm$ Bolt diameter

$\propto = 44°$ Angle to the grain

$M_{yr,k} = 0.3 * 300 * d^{2.6} = 121605\ Nmm$ Characteristic yield moment of fastener

$t_1 = 100\ mm$ Thickness of element 1

$t_2 = 170\ mm$ Thickness of element 2

$f_{h0,k} = 0.082 * (1 - 0.01 * d) * \rho_k = 28,9\ MPa$ Characteristic embedment strength

$f_{h,k} = \frac{f_{h0,k}}{kc_{90} * \sin(\propto)^2 + \cos(\propto)^2} = 23.1\ MPa$ Characteristic embedment strength

From following expressions valid charachteristic strength is obtained according to EN-1995-1-1 [59]

– For fasteners in double shear:

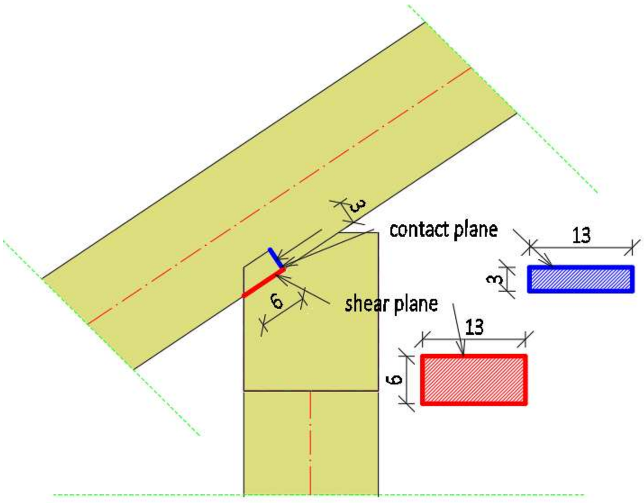
$$F_{v,Rk} = \min \begin{cases} f_{h,1,k} t_1 d & \text{(g)} \\ 0{,}5 f_{h,2,k} t_2 d & \text{(h)} \\ 1{,}05 \frac{f_{h,1,k} t_1 d}{2+\beta} \left[\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta) M_{y,Rk}}{f_{h,1,k} d\ t_1^2}} - \beta \right] + \frac{F_{ax,Rk}}{4} & \text{(j)} \\ 1{,}15 \sqrt{\frac{2\beta}{1+\beta}} \sqrt{2 M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4} & \text{(k)} \end{cases}$$

$\boldsymbol{F_{vr,k} = 11,5\ kN}$ Valid failure is ductile failure in bolt (failure (k)

$\boldsymbol{F_{vr,d} = \frac{n_{shear} * F_{vr,k}}{\gamma_s} = \frac{2*11,5}{1.1} = 20.9\ kN}$ **Bolt load carrying capacity**

NOTE: Bolt is neglected during verification process, verification is based only on load bearing capacity of short tie to shear and brace to compression stresses. Decision on safe side.

Rafter-middle purlin



Compression on a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 30 * 130 = 3900\ mm^2$

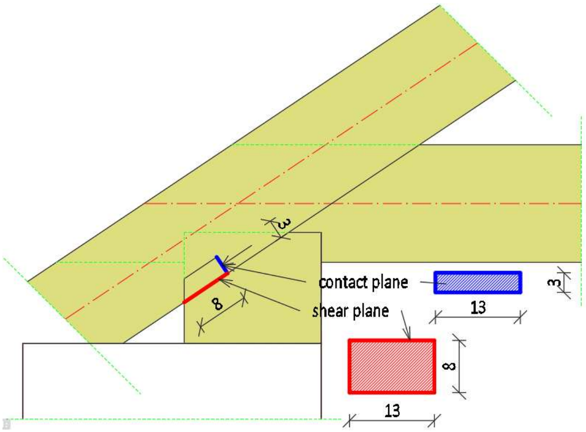
$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 3900 * 2.6 = \mathbf{10.12\ kN}$ Connection capacity on a contact plane

Rollin shear

$A_{shear} = 60 * 130 = 7800\ mm^2$

$F_{shear,d} = A_{shear} * f_{v,roling,d} = 7800 * 0.55 = \mathbf{4,29\ kN}$

Rafter-foot purlin



Compression on a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 30 * 130 = 3900\ mm^2$

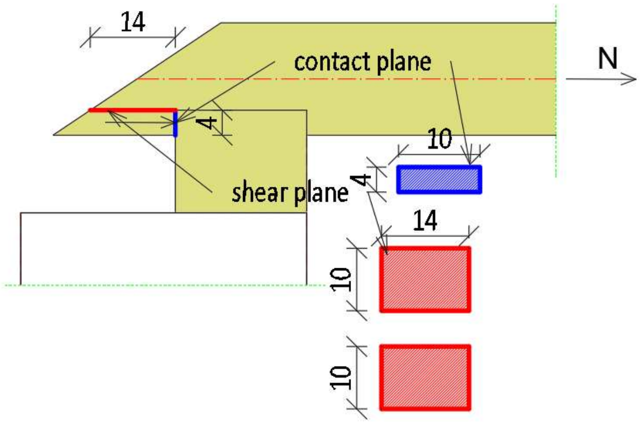
$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 3900 * 2.6 = \mathbf{10.12\ kN}$ Connection capacity on a contact plane

Rollin shear

$\mathrm{A_{shear}} = 80 * 130 = 10400\ \mathrm{mm}^2$

$F_{shear,d} = A_{shear} * f_{v,roling,d} = 10400 * 0.55 = \mathbf{5,7\ kN}$

Foot purlin-short tie



Compression on a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 2 * (40 * 100) = 8000\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 8000 * 2.6 = \mathbf{20,7\ kN}$ Connection capacity on a contact plane

Shear

$t_v = 40\ mm$

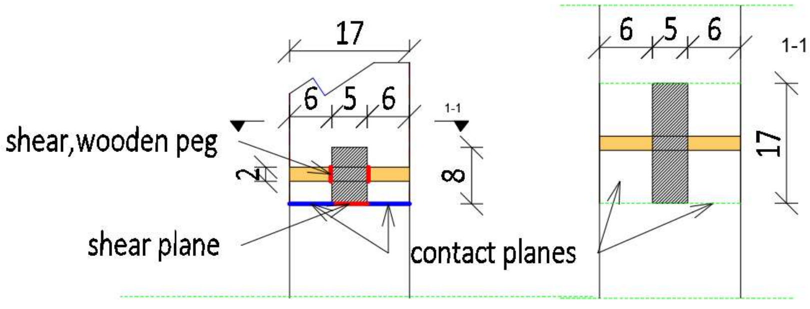
$l_{v,max} = 8 * t_v = 8 * 40 = 320\ mm$ Length of shear plane *(maximum permissible)*

$\boldsymbol{l_v = 140\ mm}$ Length of shear plane

$\mathrm{A_{shear}} = 2 * (100 * 140) * kc_r = 28000\ \mathrm{mm}^2$

$F_{shear,d} = A_{she} \quad * f_{v,d} = 28000 * 1.59 = \mathbf{44,5\ kN}$ Shear capacity

Middle pulin-queen post



Compression on a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 2 * (60 * 170) = 20400\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 8000 * 2.6 = \mathbf{52,93\ kN}$ Connection capacity on a contact plane

Notch

Compression on a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 50 * 50 = 2500\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 2500 * 2.6 = \mathbf{6,5\ kN}$ Connection capacity on a contact plane

Shear

$\boldsymbol{l_v = 170\ mm}$ Length of shear plane

$A_{shear} = 50 * \mathbf{170} = 8500\ mm^2$

$F_{she\quad ,d} = A_{shear} * f_{v,d} = 8500 * 1.59 = \mathbf{13,5\ kN}$

Shear-longitudinal to notch

$N_{tr,d} = min\left[F_{c\propto,d},\ \ F_{shear,d}\ \right] = [6.5\quad 13.5] = \mathbf{6.5\ kN}$

Notched members – shear (shear perpendicular to notch)

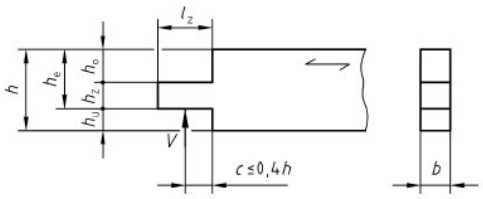


Figure D.1 Geometry of notch and symbols According to DIN 1052-1 [79]

$k_n = 5$ According to EN-1995-1-1 [59]

$h_{eff} = 110\ mm$ Effective depth, see EN-1995-1-1 [59]

$h = 170\ mm$ Height of member, see EN-1995-1-1 [59]

$\propto = \frac{h_{eff}}{h} = 0.647$ According to EN-1995-1-1 [59]

$b = 170\ mm$ Width of member

$h_e = 110\ mm$ See Figure D.1

$k_v = \frac{k_n}{\sqrt{h}\left[\sqrt{\propto*(1-\propto)}+0.8*\frac{x}{h}*\sqrt{\frac{1}{\propto}-\propto^2}\right]} = 0.61$ Reduction factor, according to EN-1995-1-1 [59]

$V_d = \frac{2}{3} * k_v * f_{v,d} * b * h_e = \mathbf{12,1\ kN}$ According to DIN 1052-1 [79]

Wooden peg

Shear

Load bearing capacity of wooden peg subjected to shear is calculated according to expression (286) given in DIN 1051-1 [79]. The expression is valid for wooden pegs with charachteristic density $\rho_k < 350 kg/m^3$ and $d < 30mm$.

Wooden peg in connection of middle purlin to queen post transfers torsional moments from middle purlin to queen post.

$R_k = 9.5 * d^2 = 9.5 * 20^2 = 3.8\ kN$ According to DIN 1051-1 [79]

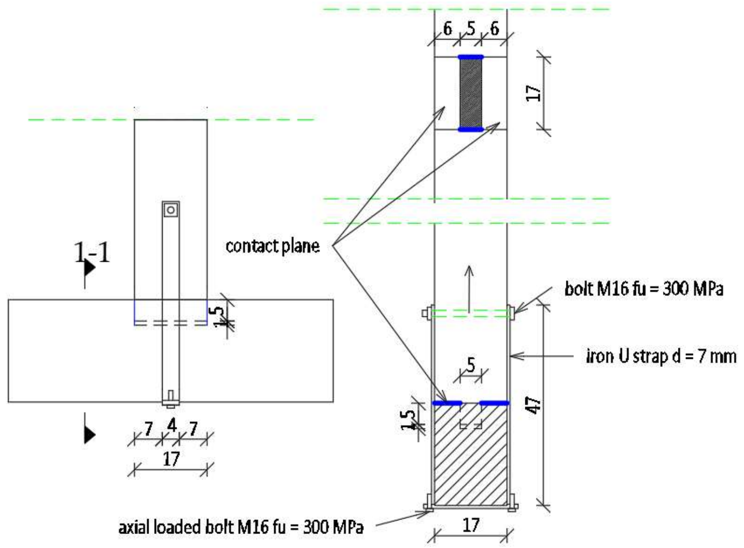
$M_t = 0.53\ kNm$ Maximum torsional moment in M1

$l = 100mm$ Moment arm (distance from shear plane of peg to the end of member)

$F_{t,d} = \frac{M_t}{l} = \frac{0.53}{0.1} = 5.3\ kN$

NOTE: Load bearing capacity of wooden pegs is insufficient. However, no damage to wooden pegs or signs of overload on connection of middle purlin- queen post were observed. Therefore, it can be assumed that wooden pegs have bigger load bearing capacity which can't be determined without destructive tests.

Queen post-tie beam



Compression a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 2 * (60 * 170) = 20400\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 20400 * 2.6 = \mathbf{53\ kN}$ Connection capacity at contact plane

Iron strap-tension failure

$f_u = 300\ MPa$ Tensile strength of iron

$\mathrm{b} = 30\ \mathrm{mm}$ Width of iron strap

$\mathrm{t} = 7\ \mathrm{mm}$ Tchicness of ion strap

$\mathrm{d_0} = 18\ \mathrm{mm}$ Hole diameter from bolt M16

$\gamma_{\mathrm{m,2}} = 1.25$ Partial factor EN-1993-1-1

$A_{neto} = (30 - d_0) * 7 = 84\ mm^2$ Remaining cross section area area

$F_{dt,k} = \frac{0.9 * A_{neto} * f_u}{\gamma_{\mathrm{m,2}}} = 33.91\ kN$ Design tensile strength of iron strap (1)

$F_{dt,total} = 2 * F_d = 67.84\ kN$ Tensile bearing capacity of iron strap (2)

Compression on a contact plane – (contact of iron U strap on tie beam-tension force)

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = b * 170 = 5100\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 5100 * 2.6 = \mathbf{13,23\ kN}$

Bolt

$d = 16mm$

$\propto = 0°$

$M_{yr,k} = 0.3 * 300 * d^{2.6} = 121605\ Nmm$

$t_1 = 170\ mm$

$f_{h0,k} = 0.082 * (1 - 0.01 * d) * \rho_k = 28{,}9\ MPa$

From following expressions valid charachteristic strength is obtained

– For thin steel plates as the outer members of a double shear connection:

$$F_{v,Rk} = \min \begin{cases} 0{,}5\, f_{h,2,k}\, t_2\, d & \text{(j)} \\ 1{,}15\sqrt{2 M_{y,Rk}\, f_{h,2,k}\, d} + \dfrac{F_{ax,Rk}}{4} & \text{(k)} \end{cases} \qquad (8.12)$$

$\boldsymbol{F_{vr,k} = 12,2\ kN}$ Valid failure is ductile failure in bolt (failure (k))

$\boldsymbol{F_{vr,d} = \frac{n_{shear} * F_{vr,k}}{\gamma_s} = \frac{2*12,2}{1.1} = 22.18\ kN}$ **Bolt load carrying capacity**

Notch

Compression on a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.6\ MPa$

$A_{eff} = 50 * 50 = 2500\ mm^2$

$F_{c\propto,d} = A_{eff} * f_{c,\propto} = 2500 * 2.6 = \mathbf{6,5\ kN}$

Shear

$\boldsymbol{l_v = 170\ mm}$ Length of shear plane *(maximum permissible)*

$\mathrm{A_{shear}} = 50 * \mathbf{170} = 8500\ \mathrm{mm}^2$

$F_{shear,d} = A_{shear} * f_{v,d} = 8500 * 1.59 = \mathbf{13,5\ kN}$

Notched members – shear (shear perpendicular to notch)

$k_n = 5$ According to EN-1995-1-1 [59]

$\propto = 0.5$ According to EN-1995-1-1 [59]

$b = 170\ mm$ Width of member

$h_e = 110\ mm$ See Figure D.1

$k_v = \frac{k_n}{\sqrt{h}\left[\sqrt{\propto*(1-\propto)}+0.8*\frac{x}{h}*\sqrt{\frac{1}{\propto}-\propto^2}\right]} = 0.6$ Reduction factor, according to EN-1995-1-1 [59]

$V_d = \frac{2}{3} * k_v * f_{v,d} * b * h_e = \mathbf{11\ kN}$ According to DIN 1052-1 [79]

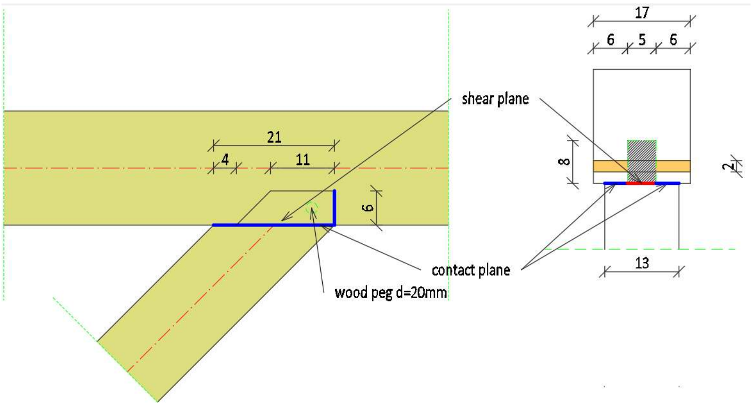
Load bearing capacity- Tension

$N_{tr,d} = min[F_{c\propto,d},\ F_{dt,total}, F_{vr,d}\ ] = [13.23\quad 67.84\quad 22.8] = 13.23\ kN$

Shear-longitudinal to notch

$N_{tr,d} = min[F_{c\propto,d},\ F_{shear,d}\ ] = [6.5\quad 13.5] = 6.5\ kN$

Knee brace-middle purlin



Compression on a contact plane

$\propto = 90°$ Angle to the grain

$f_{c,\propto} = 2.26\ MPa$

$A_{eff} = 2 * (50 * 210) = 21000\ mm^2$

$F_d = \frac{A_{eff}*f_{c,\propto}}{\cos\propto} = \frac{(21000*2.26)}{0.707} = \mathbf{67.18\ kN}$

Notch

Compression on a contact plane

$\propto = 45°$ Angle to the grain

$f_{c,\propto} = 4.41\ MPa$

$A_{eff} = 60 * 50 = 3000\ mm^2$

$F_{c\propto,d} = \frac{A_{eff} * f_{c,\propto}}{\cos\propto} = \frac{(3000*4.41)}{0.707} = \mathbf{18.71\ kN}$

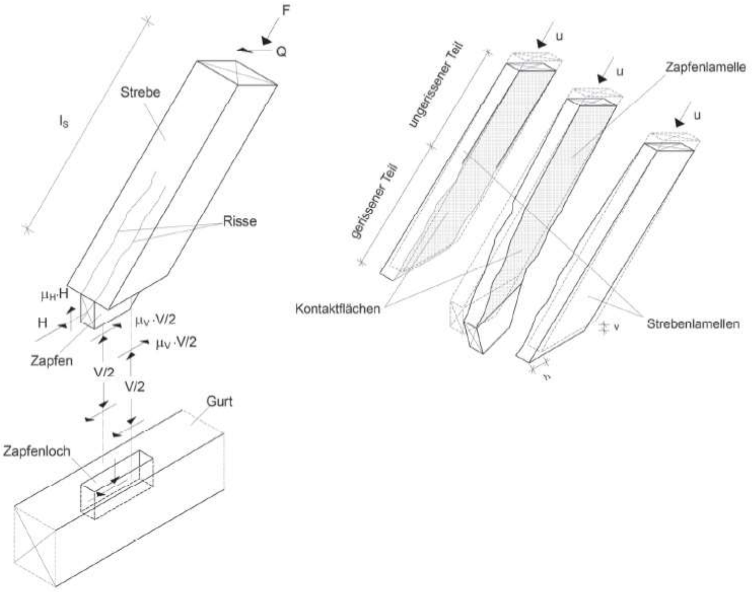


Figure D.2 Failure mode for member with tenon and mortise, see [20]

For failure of tenon and mortise connection by shearing, an empirical expression is given in [31]. This exspression gives good prediction in comparison to more detailed calculation given in master thesis of Roman Popatnig [20]. The emerical approach overestimates load bearing capacity for 9 % in comparison to more detailed calculation. For sake of simplicity emperical expression is used in this thesis.

$A_{tenon} = 50 * 60 = 3000\ mm^2$

$F_{vr,d} \approx \frac{A_{tenon}}{150} \approx 20$ kN

Shear-longitudinal to notch

$N_{tr,d} = min[F_{c\propto,d},\ F_{vr,d}\ ] = [18.71\quad 20] = 18.71\ kN$

Wooden peg

Tension

$R_k = 9.5 * d^2 = 9.5 * 20^2 = 3.8\ kN$ According to DIN 1051-1 [79]

$R_d = n_{shear} * R_k = 2 * 3.8 = \mathbf{7.6 kN}$ Load bearing capacity of wooden peg to tension

ULS calculation-verification summary for model M1 (connections)

MODEL M1																	
Connection	Axis	Internal force				Load bearing capacity [kN]				Shear 90°	Shear 0°	peg	Verification [%]*100				
		N(-)[kN]	N(+)[kN]	V0 [kN]	V90 [kN]	Compression	Tension	Shear	Roll. shear	notch	notch	Shear	N(-) 0	N(+) 0	shear/rolling	90° notch	0° notch
Brace-tie beam	8	74.97				35.2		42.93					2.13		1.24		
Brace-queen post	8	65.53				35.2		42.93					1.86		1.09		
Straining beam-queen post	8	50.89				67.47							0.75				
Short tie-brace	8	19.26				48.57		92					0.40		0.21		
Rafter-middle purlin	9.2	3.75				10.12			4.29				0.37		0.87		
Rafter-foot purlin	8	13.41				10.12			5.7				1.33		2.35		
Foot purlin-short tie	8	19.26				20.7		44.5					0.93		0.43		
Middle purlin-queen post	8	25		6.17	7.54	52.93				12.1	6.5	3.8	0.47			0.62	0.95
Queen post-tie beam	12	38.62	16.55	2.31	3	53	13.23			11	6.5		0.73	1.25		0.27	0.36
Knee brace-middle purlin	8	23.25	2			67.18	7.6				18.71	7.6	0.35	0.26			

ULS calculation-verification summary for model M2 (connections)

MODEL M2																		
Connection	Axis	Internal force				Load bearing capacity [kN]				Shear 90°	Shear 0°	peg	Verification [%]*100				
		N(-)[kN]	N(+)[kN]	V0 [kN]	V90 [kN]	Compression	Tension	Shear	Roll. shear	notch	notch	Shear	N(-) 0	N(+) 0	shear/rolling	90° notch	0° notch
Brace-tie beam	8	87.89				35.2		42.93					2.50		1.46		
Brace-queen post	8	80				35.2		42.93					2.27		1.33		
Straining beam-queen post	8	50.26				67.47							0.74				
Short tie-brace	8	14.37				48.57		92					0.30		0.16		
Rafter-middle purlin	9.1	10.06				10.12			4.29				0.99		2.34		
Rafter-foot purlin	9.2	9.34				10.12			5.7				0.92		1.64		
Foot purlin-short tie	8	14.37				20.7		44.5					0.69		0.32		
Middle purlin-queen post	10	11.56		4.86	6.36	52.93				12.1	6.5	3.8	0.22			0.53	0.75
Queen post-tie beam	12	30.77	15.41	1.54	2.19	53	13.23			11	6.5		0.58	1.16		0.20	0.24
Knee brace-middle purlin	8	27.06	4			67.18	7.6				18.71	7.6	0.40	0.53			

C-2 Models

C-2.1 Tie beam Model M1&2

Here diagrams of bending moments Mz and deformation of tie beam in respective direction are given for models M1&2

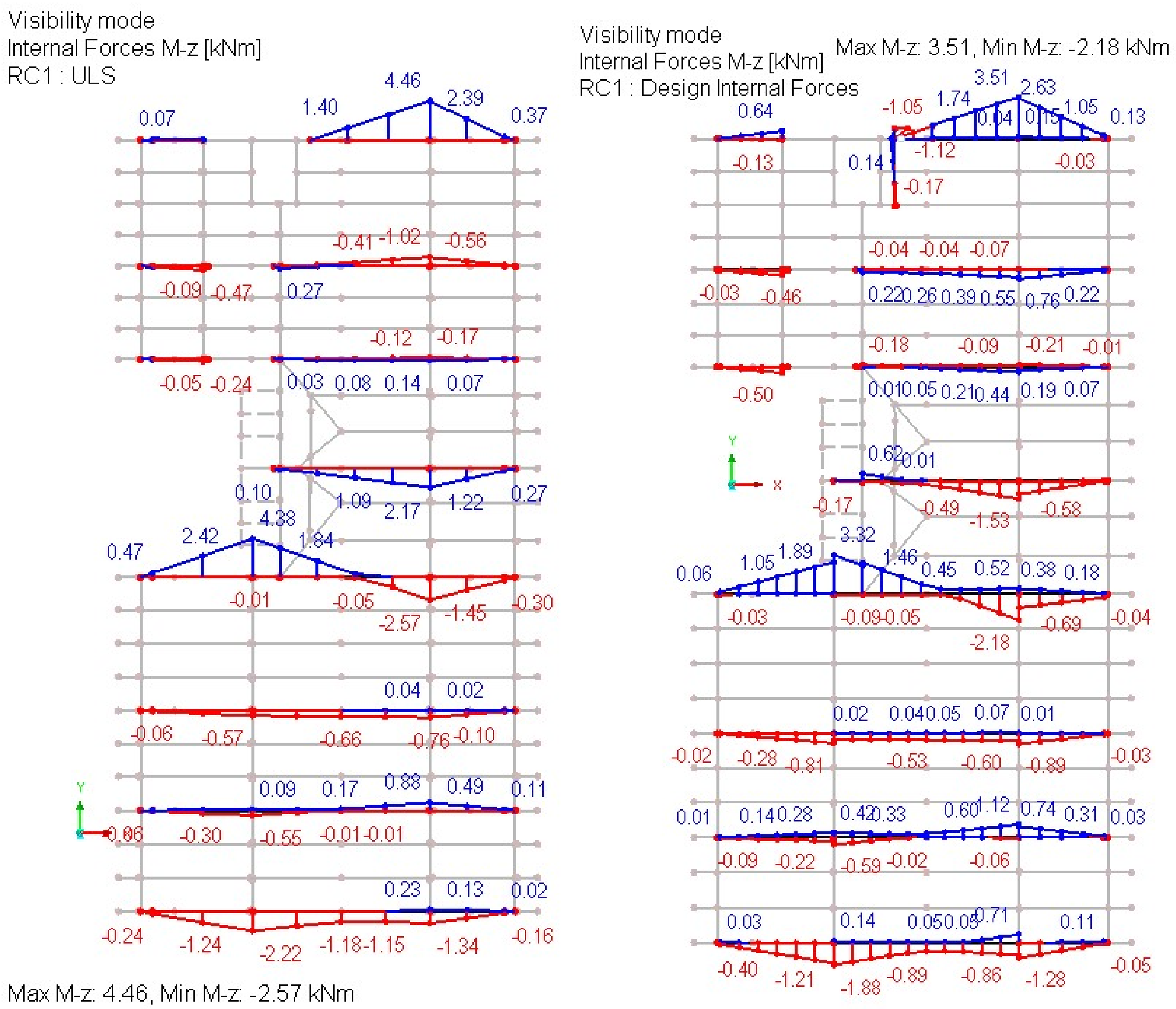
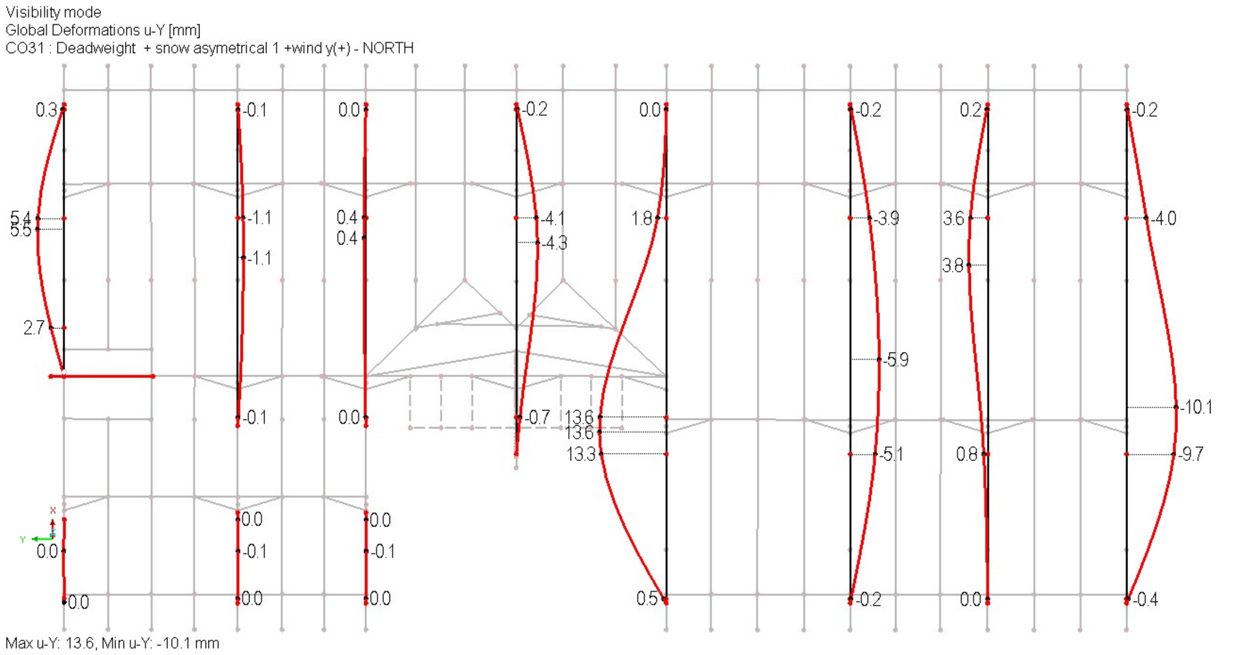


Figure C2.1 left: bending moments Mz in tie beams model M1; right: M2



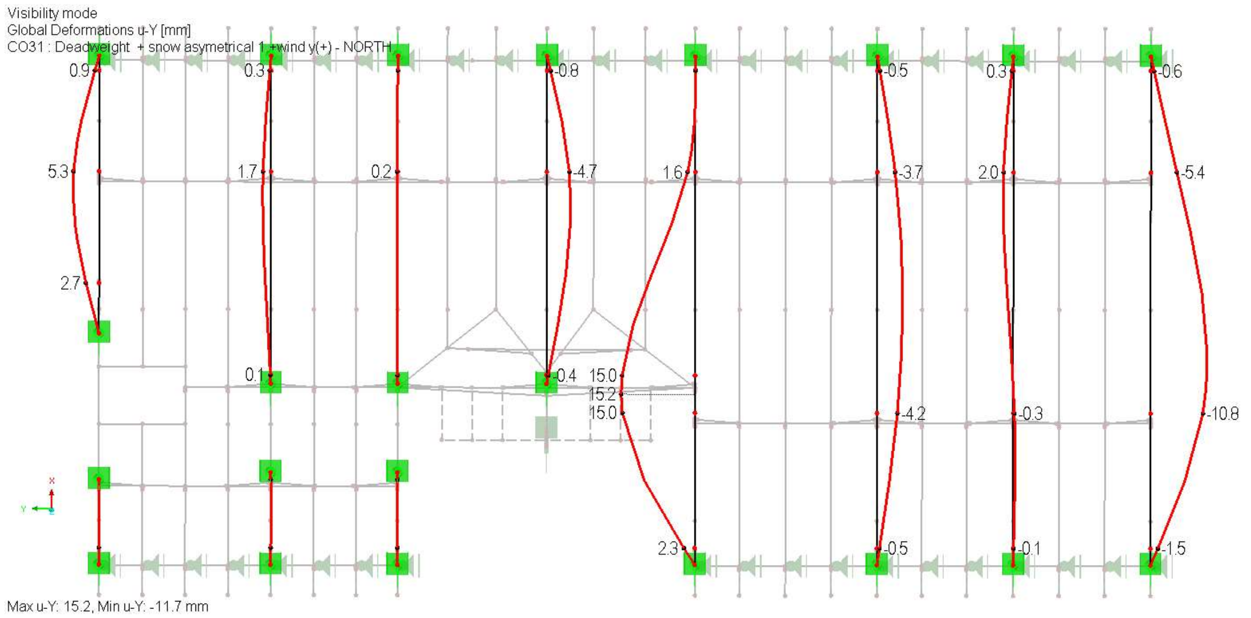
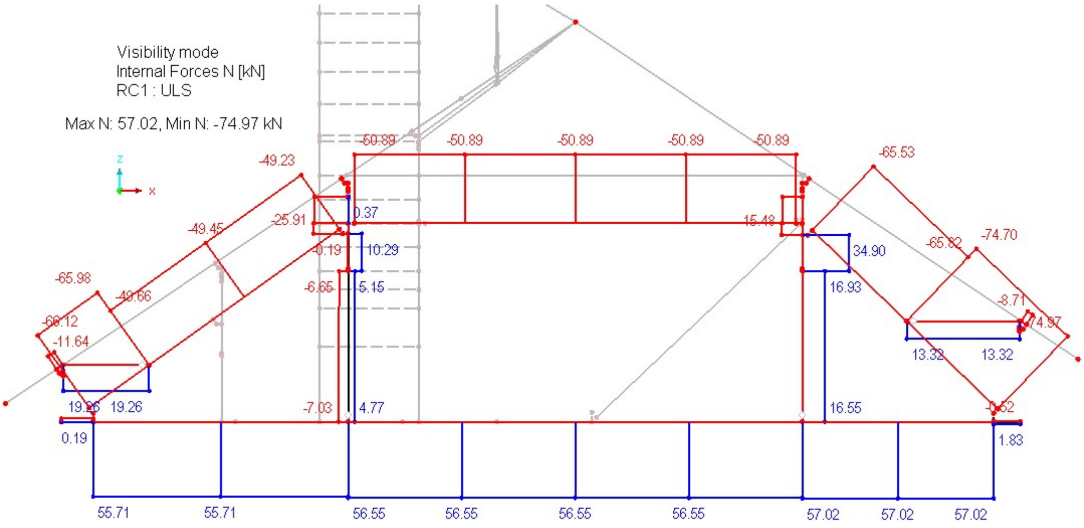
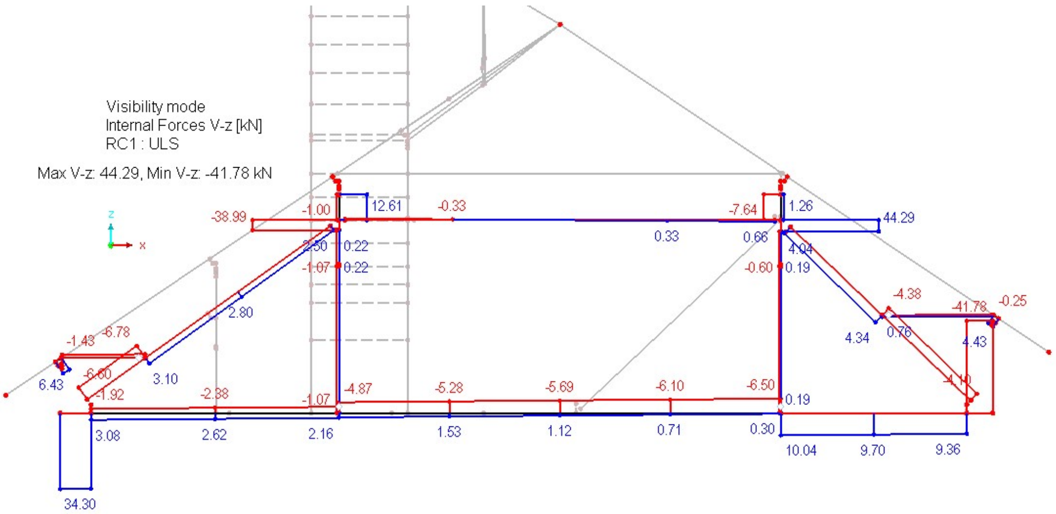


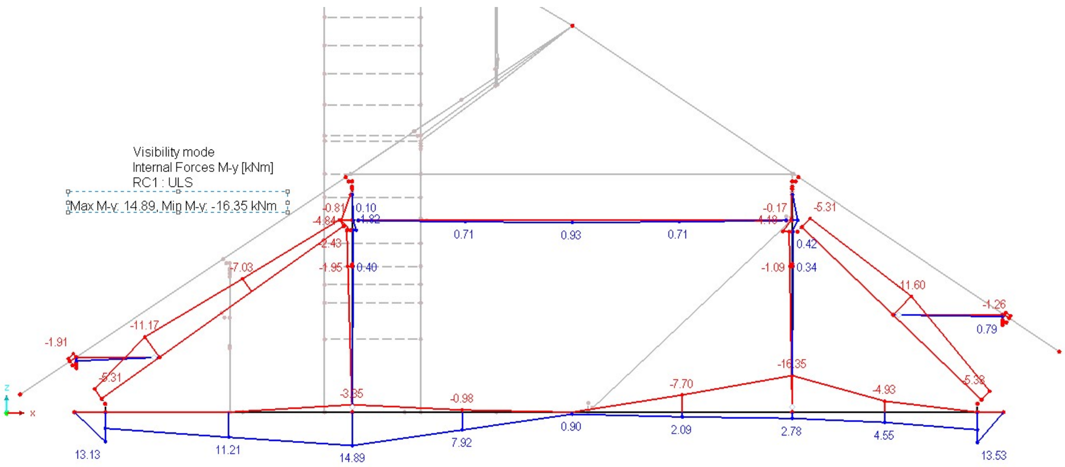
Figure C2.2 up: deformation of tie beams in global Y-direction, model M1; down: M2

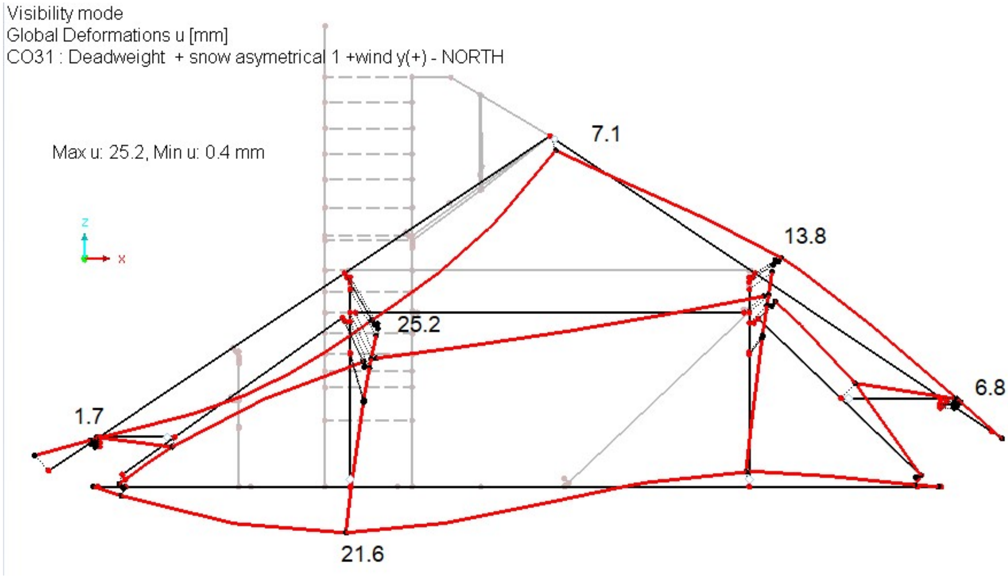
Next, internal forces (N,My,Vz) and deformation for axis 8 for all 4 models are given. Principal and secondary trusses are shown. Moreover, for models M3&4 view on model and supports is given.

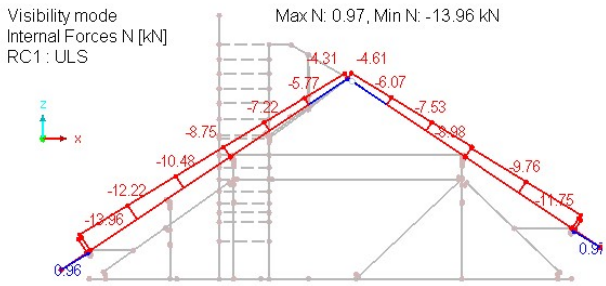
C-2.2 Model M1

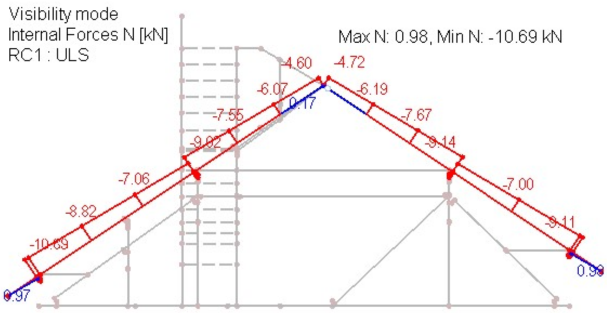

Visibility mode
Internal Forces N [kN]
RC1 : ULS
Max N: 57.02, Min N: -74.97 kN

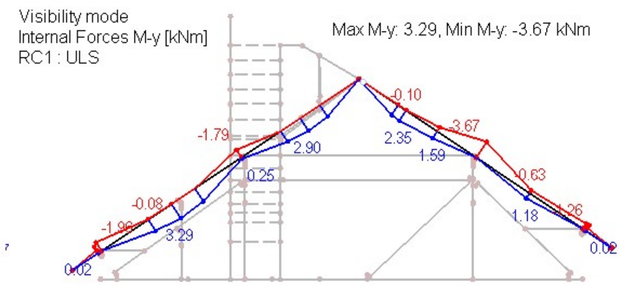

Visibility mode
Internal Forces V-z [kN]
RC1 : ULS
Max V-z: 44.29, Min V-z: -41.78 kN

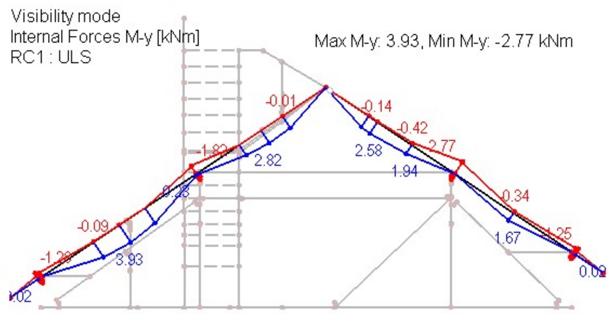

Visibility mode
Internal Forces M-y [kNm]
RC1 : ULS
Max M-y: 14.89, Min M-y: -16.35 kNm

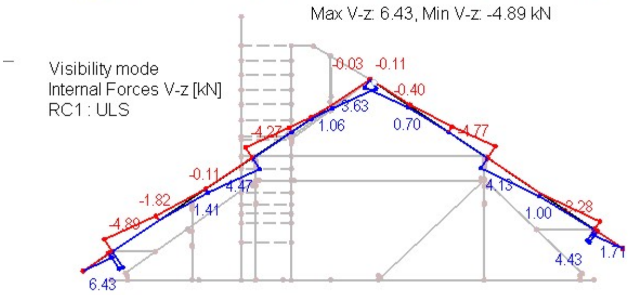

Visibility mode
Global Deformations u [mm]
CO31 : Deadweight + snow asymetrical 1 +wind y(+) - NORTH
Max u: 25.2, Min u: 0.4 mm
7.1
13.8
25.2
6.8
1.7
21.6

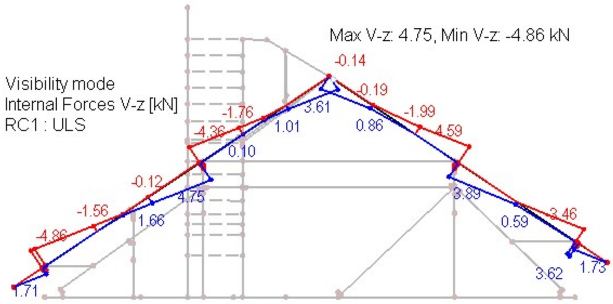

Visibility mode
Internal Forces N [kN]
RC1 : ULS
Max N: 0.97, Min N: -13.96 kN


Visibility mode
Internal Forces N [kN]
RC1 : ULS
Max N: 0.98, Min N: -10.69 kN

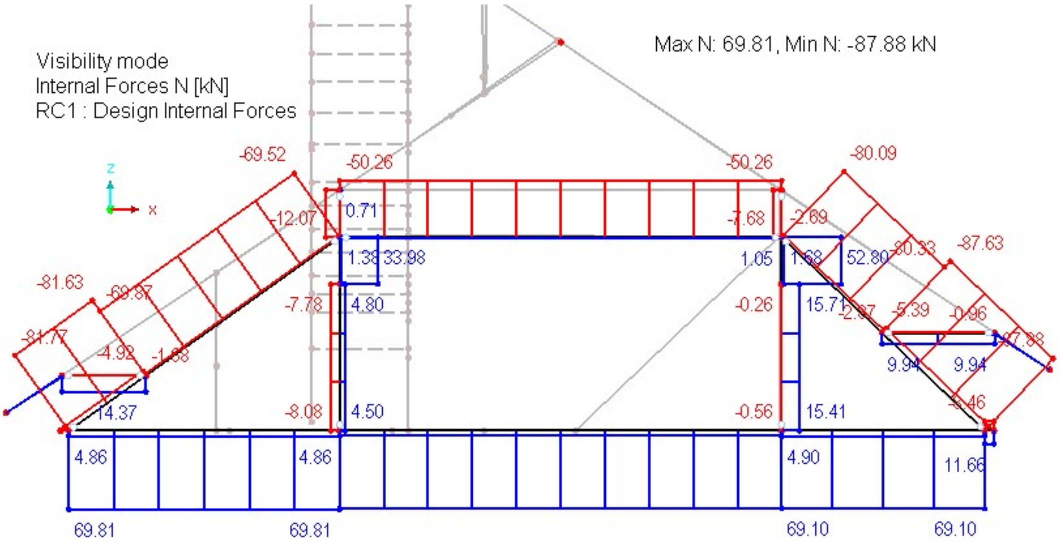

Visibility mode
Internal Forces M-y [kNm]
RC1 : ULS
Max M-y: 3.29, Min M-y: -3.67 kNm

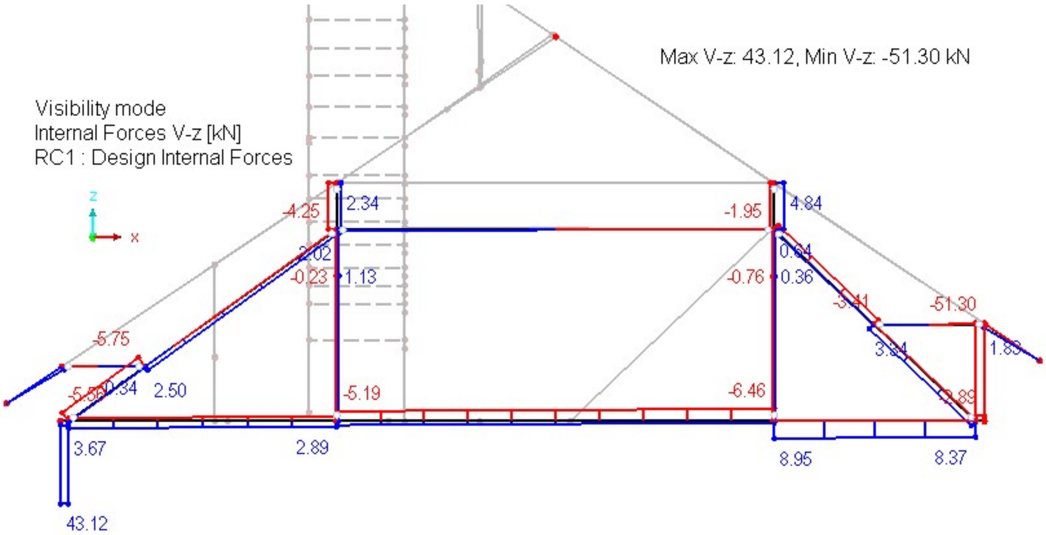

Visibility mode
Internal Forces M-y [kNm]
RC1 : ULS
Max M-y: 3.93, Min M-y: -2.77 kNm

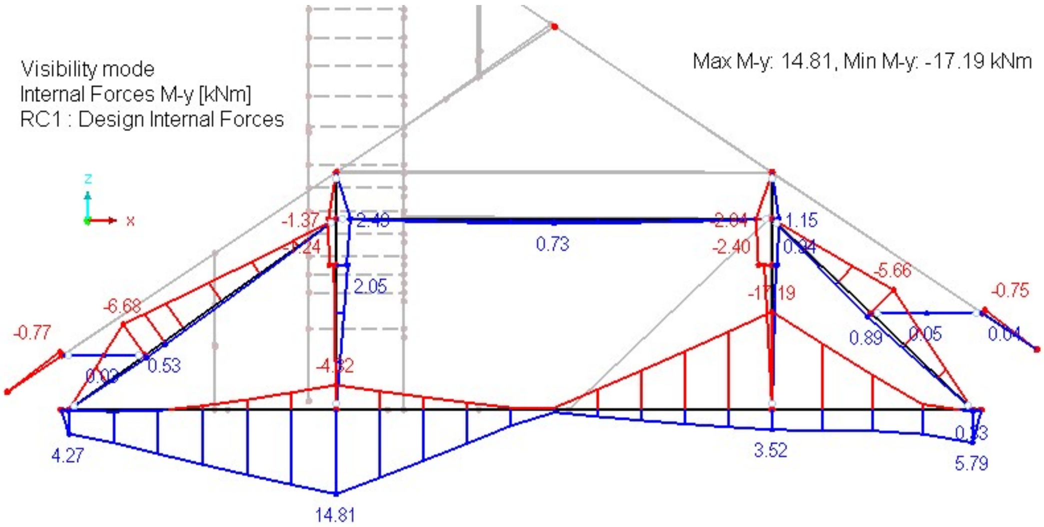

Max V-z: 6.43, Min V-z: -4.89 kN
Visibility mode
Internal Forces V-z [kN]
RC1 : ULS

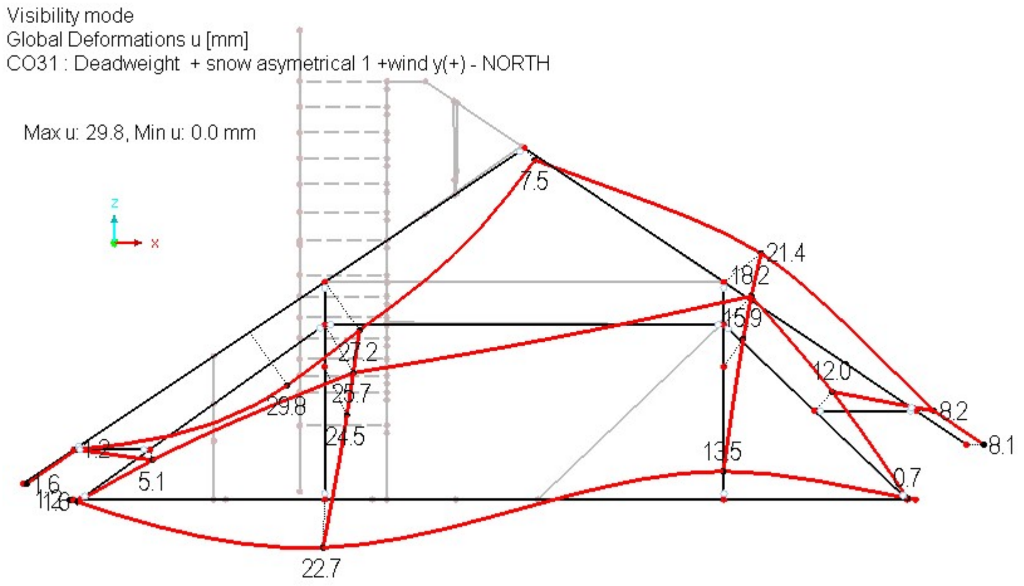

Max V-z: 4.75, Min V-z: -4.86 kN
Visibility mode
Internal Forces V-z [kN]
RC1 : ULS

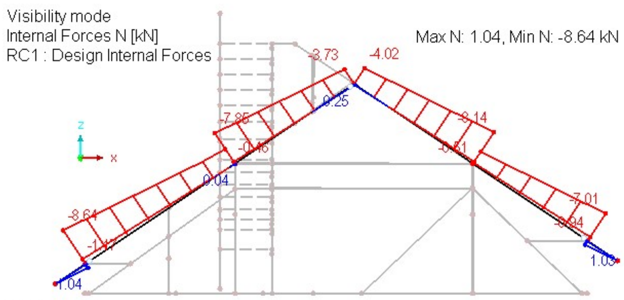
C-2.3 Model M2

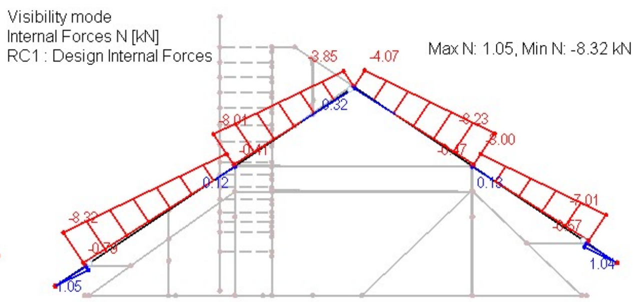

Visibility mode
Internal Forces N [kN]
RC1 : Design Internal Forces
Max N: 69.81, Min N: -87.88 kN
-69.52
-50.26
-50.26
-80.09
-12.07
0.71
-7.68
-2.69
-81.63
-69.87
1.38
33.98
1.05
1.68
52.80
-80.33
-87.63
-81.77
-7.78
4.80
-0.26
15.71
-5.39
-0.96
-4.92
-1.68
9.94
9.94
14.37
-8.08
4.50
-0.56
15.41
4.86
4.86
4.90
11.66
69.81
69.81
69.10
69.10

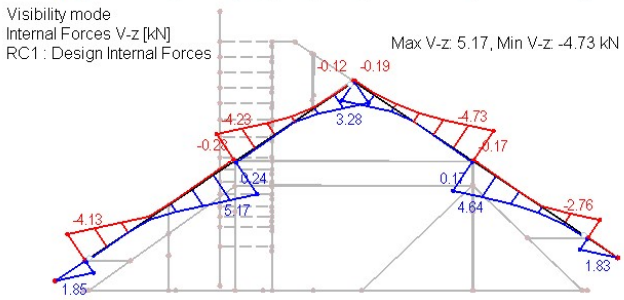

Max V-z: 43.12, Min V-z: -51.30 kN
Visibility mode
Internal Forces V-z [kN]
RC1 : Design Internal Forces
-4.25
2.34
-1.95
4.84
-0.23
1.13
-0.76
0.36
-51.30
-5.75
1.83
2.50
-5.19
-6.46
3.67
2.89
8.95
8.37
43.12

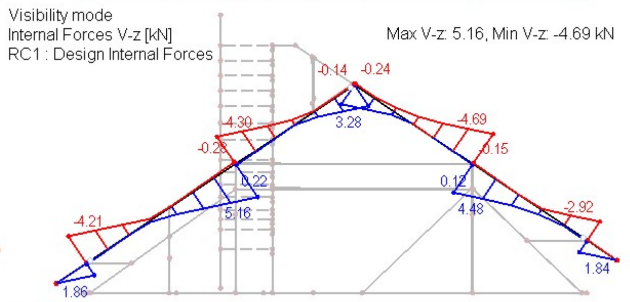

Visibility mode
Internal Forces M-y [kNm]
RC1 : Design Internal Forces
Max M-y: 14.81, Min M-y: -17.19 kNm
-1.37
-2.04
1.15
0.73
-2.40
2.05
-5.66
-0.75
-6.68
-0.77
0.89
0.05
0.53
4.27
3.52
5.79
14.81

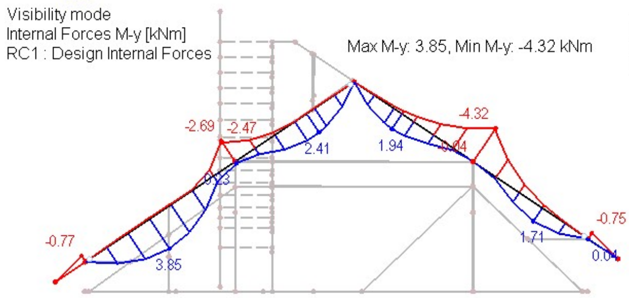

Visibility mode
Global Deformations u [mm]
CO31 : Deadweight + snow asymetrical 1 +wind y(+) - NORTH
Max u: 29.8, Min u: 0.0 mm
7.5
21.4
18.2
15.9
27.2
12.0
25.7
29.8
8.2
8.1
24.5
13.5
5.1
0.7
22.7

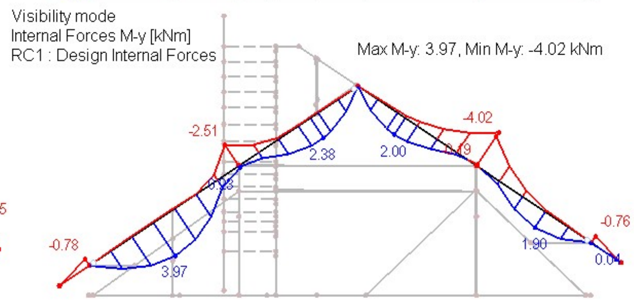

Visibility mode
Internal Forces N [kN]
RC1 : Design Internal Forces
Max N: 1.04, Min N: -8.64 kN


Visibility mode
Internal Forces N [kN]
RC1 : Design Internal Forces
Max N: 1.05, Min N: -8.32 kN

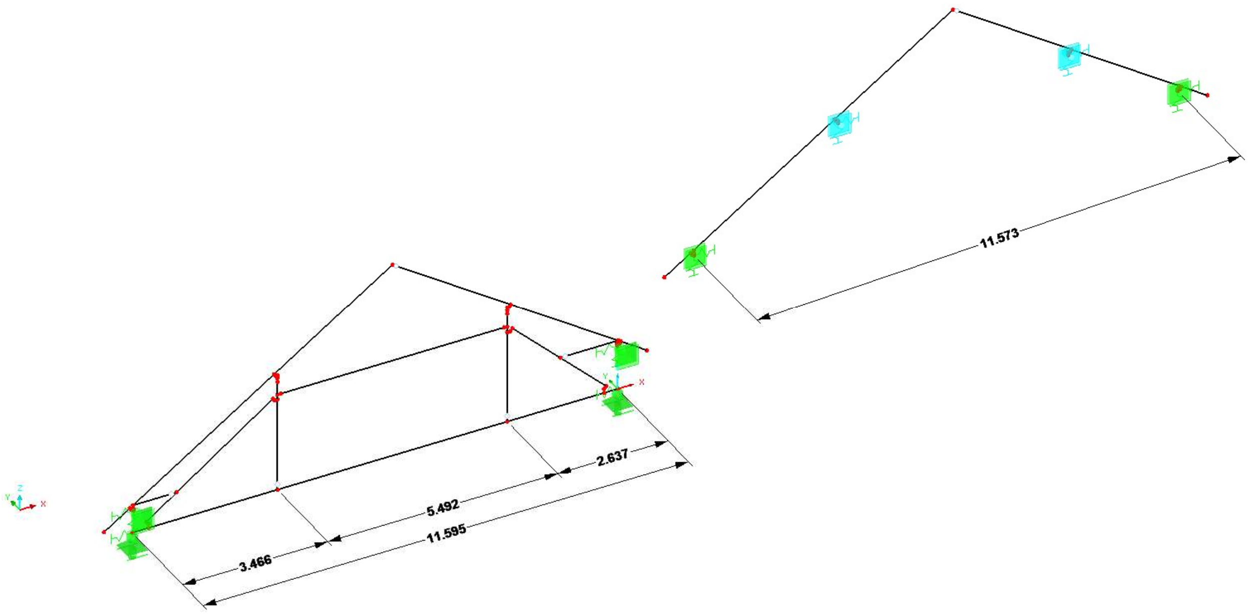

Visibility mode
Internal Forces V-z [kN]
RC1 : Design Internal Forces
Max V-z: 5.17, Min V-z: -4.73 kN

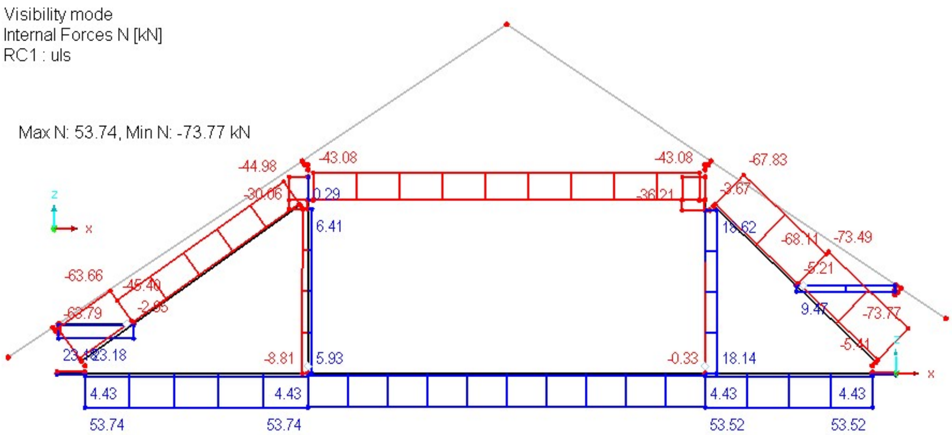

Visibility mode
Internal Forces V-z [kN]
RC1 : Design Internal Forces
Max V-z: 5.16, Min V-z: -4.69 kN

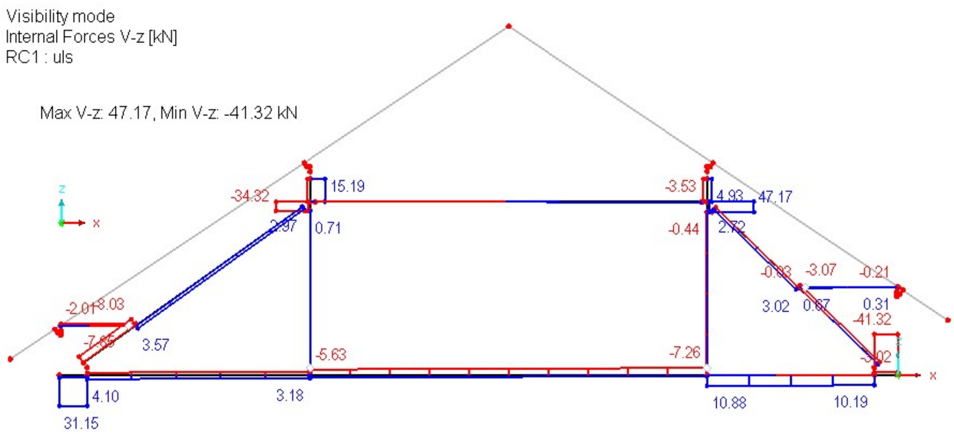

Visibility mode
Internal Forces M-y [kNm]
RC1 : Design Internal Forces
Max M-y: 3.85, Min M-y: -4.32 kNm

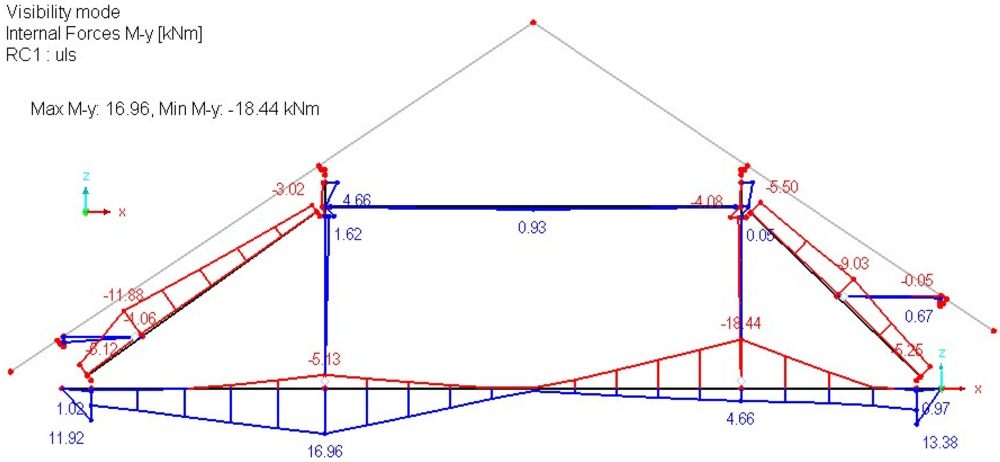

Visibility mode
Internal Forces M-y [kNm]
RC1 : Design Internal Forces
Max M-y: 3.97, Min M-y: -4.02 kNm

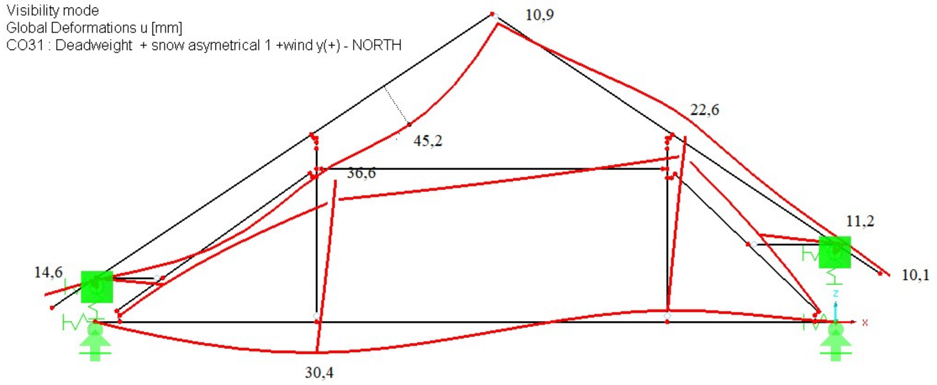
C-2.4 Model M3

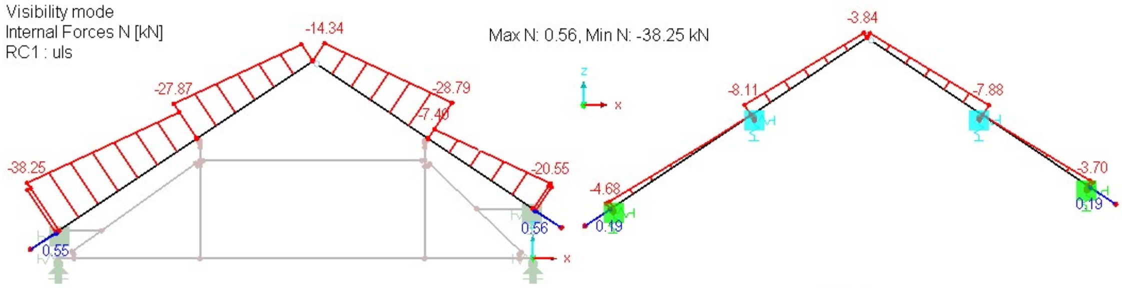

11.573
2.637
5.492
11.595
3.466

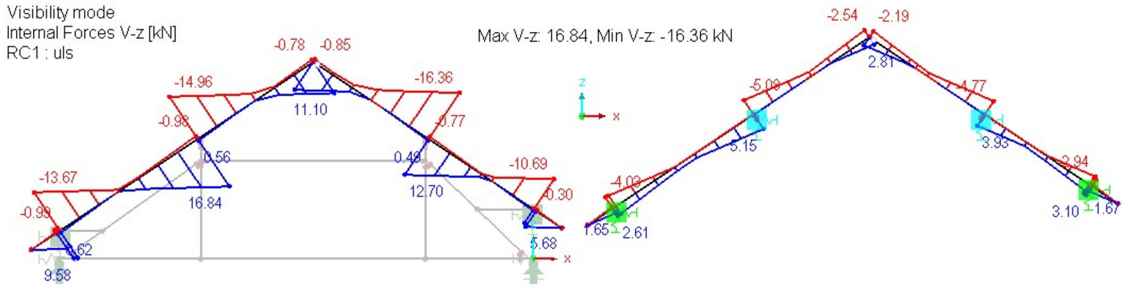

Visibility mode
Internal Forces N [kN]
RC1 : uls
Max N: 53.74, Min N: -73.77 kN
-44.98
-43.08
-43.08
-67.83
-30.06
0.29
-36.21
6.41
-68.11
-73.49
-63.66
-45.40
-73.77
9.47
-8.81
5.93
-0.33
18.14
4.43
4.43
4.43
4.43
53.74
53.74
53.52
53.52

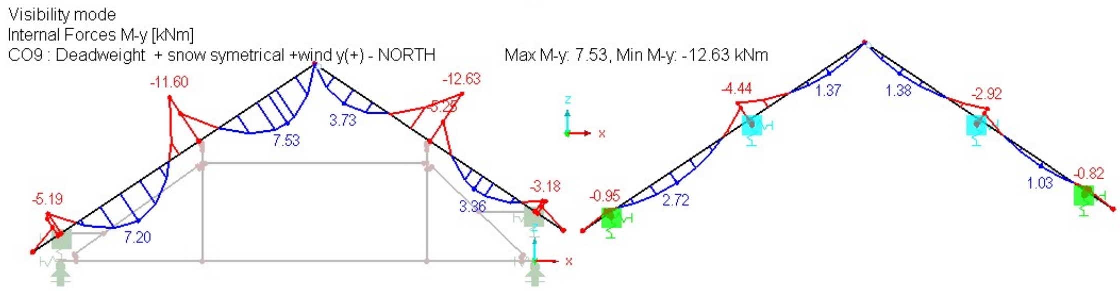

Visibility mode
Internal Forces V-z [kN]
RC1 : uls
Max V-z: 47.17, Min V-z: -41.32 kN
15.19
-34.32
-3.53
4.93
47.17
0.71
-0.44
2.72
-3.07
-0.21
3.02
0.31
-41.32
3.57
-5.63
-7.26
4.10
3.18
10.88
10.19
31.15


Visibility mode
Internal Forces M-y [kNm]
RC1 : uls
Max M-y: 16.96, Min M-y: -18.44 kNm
-3.02
4.66
1.62
0.93
-4.08
-5.50
0.05
-9.03
-0.05
0.67
-11.88
-1.06
-3.12
-18.44
-5.13
-5.25
1.02
11.92
16.96
4.66
0.97
13.38

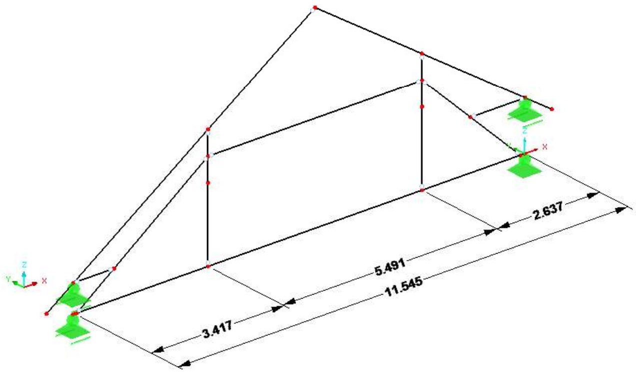

Visibility mode
Global Deformations u [mm]
CO31 : Deadweight + snow asymetrical 1 +wind y(+) - NORTH
10,9
22,6
45,2
36,6
11,2
14,6
10,1
30,4

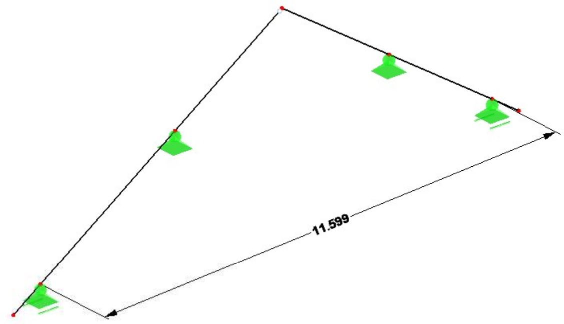

Visibility mode
Internal Forces N [kN]
RC1 : uls
-14.34
Max N: 0.56, Min N: -38.25 kN
-3.84
-27.87
-28.79
-7.40
-8.11
-7.88
-38.25
-20.55
-3.70
-4.68
0.19
0.19
0.55
0.56


Visibility mode
Internal Forces V-z [kN]
RC1 : uls
-0.78
-0.85
Max V-z: 16.84, Min V-z: -16.36 kN
-2.54
-2.19
2.81
-14.96
-16.36
11.10
-0.98
-0.77
-5.09
-4.77
5.15
3.93
0.56
0.49
-13.67
16.84
12.70
-10.69
-0.30
-4.09
-2.94
-0.99
1.65
2.61
3.10
1.67
5.68
9.58

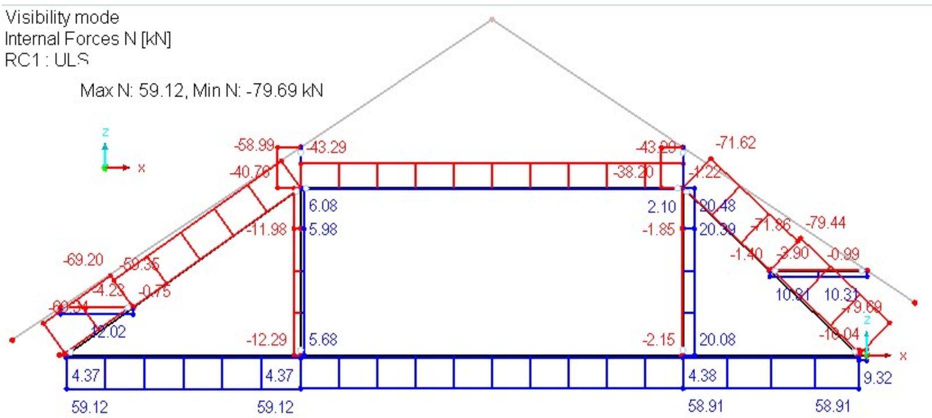

Visibility mode
Internal Forces M-y [kNm]
CO9 : Deadweight + snow symetrical +wind y(+) - NORTH
Max M-y: 7.53, Min M-y: -12.63 kNm
-11.60
-12.63
-5.23
3.73
7.53
-4.44
1.37
1.38
-2.92
-5.19
7.20
3.36
-3.18
-0.95
2.72
1.03
-0.82

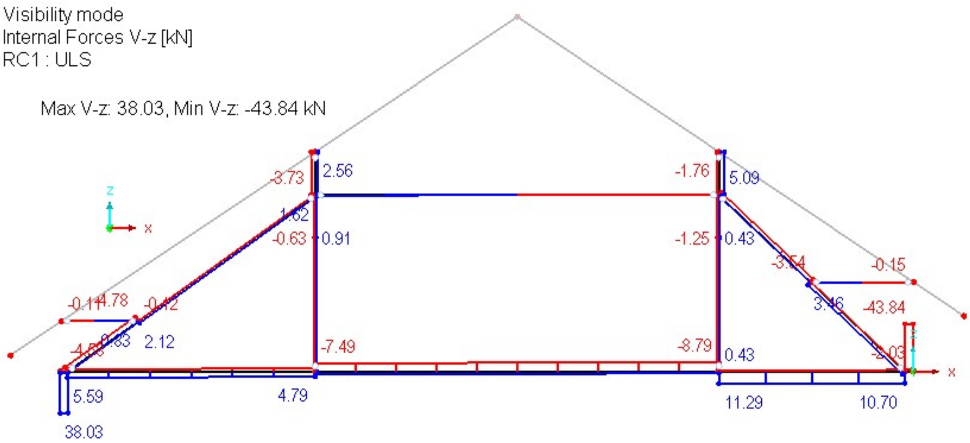
C-2.5 Model M4

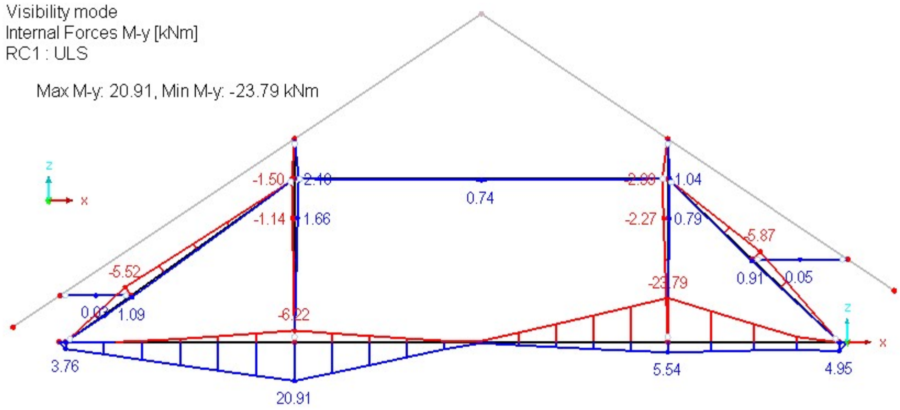

3.417
5.491
11.545
2.637

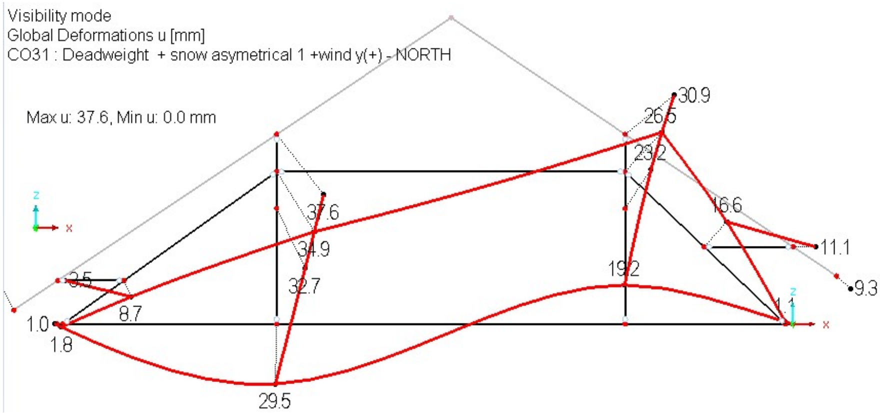

11.599

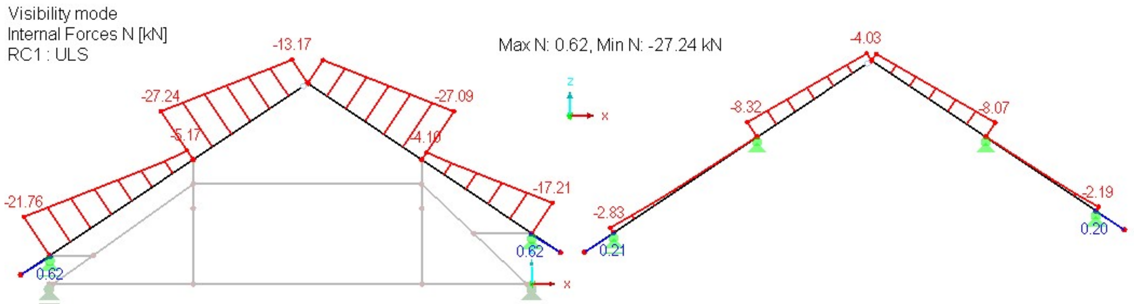
,

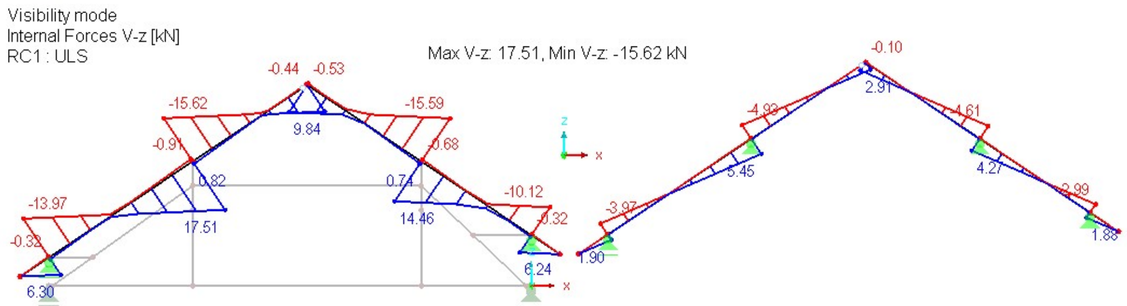

Visibility mode
Internal Forces N [kN]
RC1 : ULS
Max N: 59.12, Min N: -79.69 kN
-58.99
-43.29
-40.76
-43.99
-38.20
-71.62
6.08
5.98
-11.98
2.10
-1.85
20.48
20.39
-71.86
-79.44
-69.20
-1.40
-3.90
-0.99
-4.23
-0.75
10.31
12.02
-12.29
5.68
-2.15
20.08
4.37
4.37
4.38
9.32
59.12
59.12
58.91
58.91

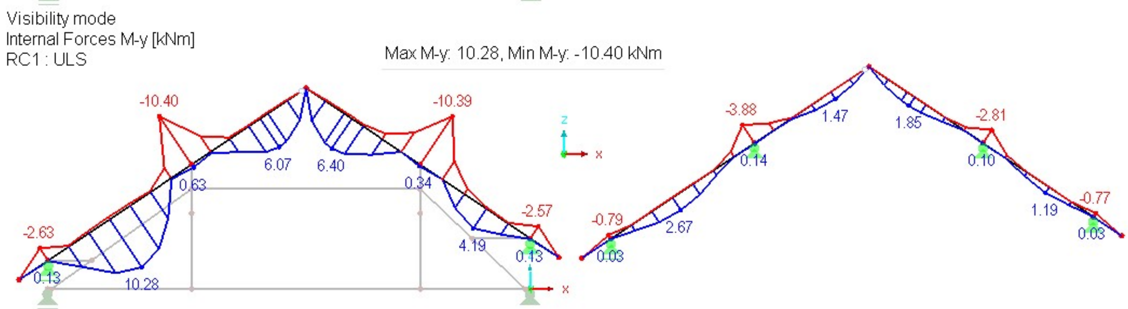

Visibility mode
Internal Forces V-z [kN]
RC1 : ULS
Max V-z: 38.03, Min V-z: -43.84 kN
-3.73
2.56
-1.76
5.09
1.62
-0.63
0.91
-1.25
0.43
-3.54
-0.15
-0.42
2.12
3.46
-43.84
-7.49
-8.79
0.43
-2.03
5.59
4.79
11.29
10.70
38.03


Visibility mode
Internal Forces M-y [kNm]
RC1 : ULS
Max M-y: 20.91, Min M-y: -23.79 kNm
-1.50
2.40
0.74
1.04
-1.14
1.66
-2.27
0.79
-5.87
-5.52
-23.79
0.91
0.05
1.09
-6.22
3.76
20.91
5.54
4.95


Visibility mode
Global Deformations u [mm]
CO31 : Deadweight + snow asymetrical 1 +wind y(+) - NORTH
Max u: 37.6, Min u: 0.0 mm
30.9
26.5
23.2
37.6
16.6
34.9
11.1
19.2
32.7
9.3
8.7
1.0
1.8
29.5


Visibility mode
Internal Forces N [kN]
RC1 : ULS
Max N: 0.62, Min N: -27.24 kN
-13.17
-27.24
-27.09
-21.76
-17.21
0.62
0.62
-4.03
-8.32
-8.07
-2.83
0.21
-2.19
0.20


Visibility mode
Internal Forces V-z [kN]
RC1 : ULS
Max V-z: 17.51, Min V-z: -15.62 kN
-0.44
-0.53
-15.62
-15.59
9.84
-13.97
17.51
14.46
-10.12
6.30
6.24
-0.10
2.91
5.45
4.27
1.90
1.88


Visibility mode
Internal Forces M-y [kNm]
RC1 : ULS
Max M-y: 10.28, Min M-y: -10.40 kNm
-10.40
-10.39
6.07
6.40
0.63
0.34
-2.63
-2.57
4.19
0.13
0.13
10.28
-3.88
1.47
1.85
-2.81
0.14
0.10
-0.79
2.67
1.19
-0.77
0.03
0.03

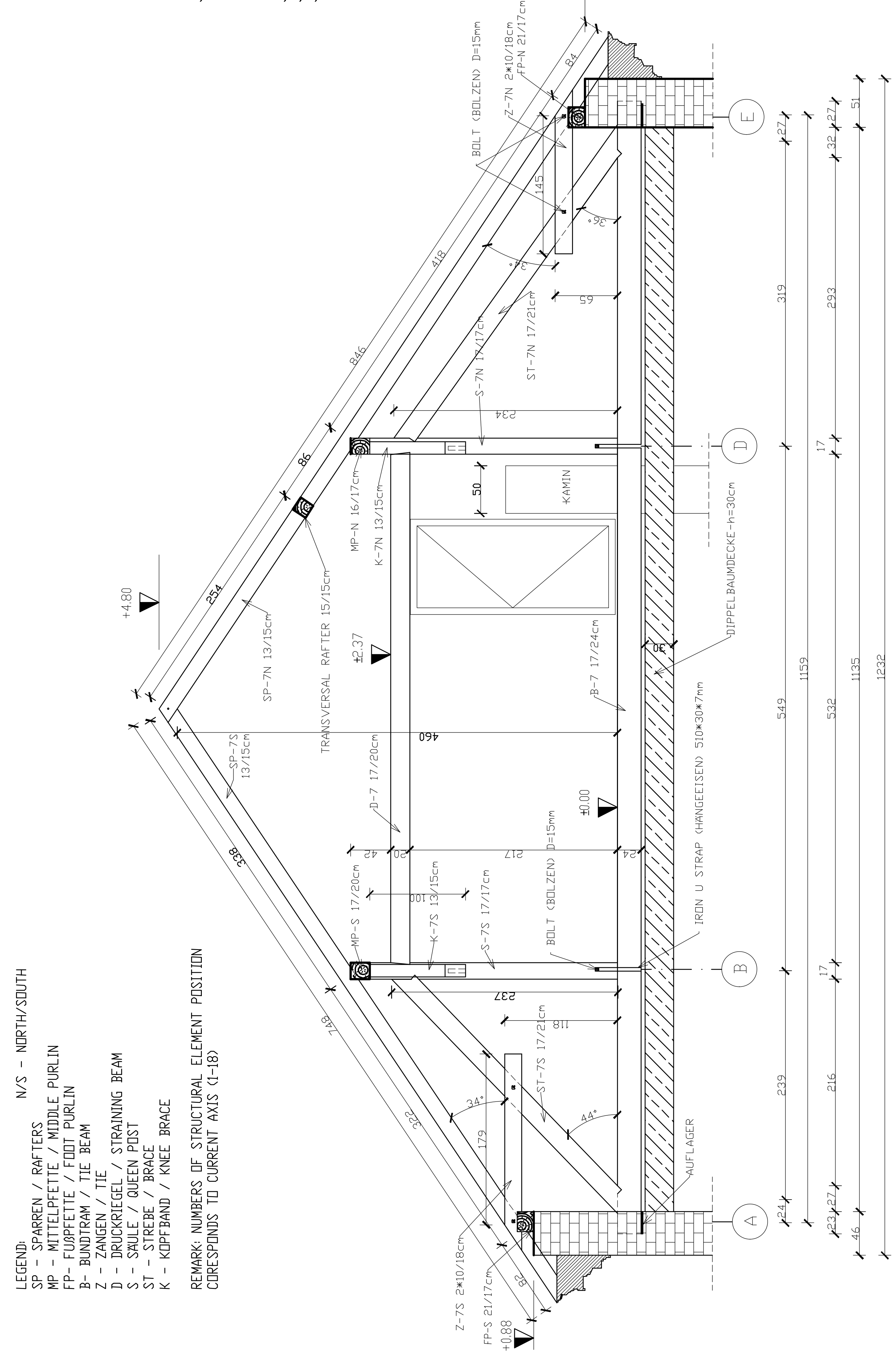
APPENDIX D DRAWING PLANS

In this Appendix drawing plans of roof structure TU Graz Alte Technik are given. All plans are printed on A4 paper too meet publishing requirements therefore scale values don't represent the standard values.

List of plans:

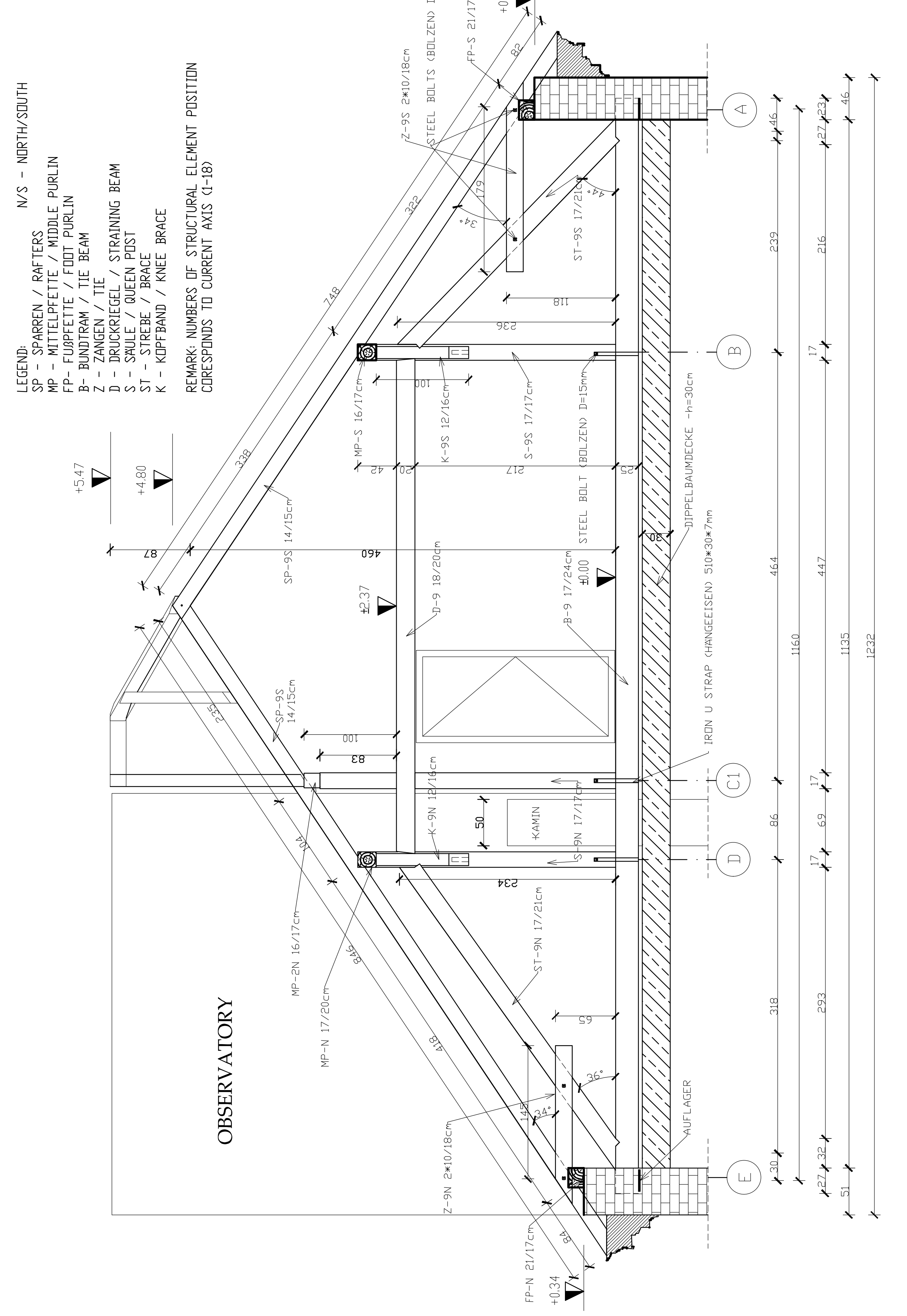
Plan number	Plane name	Scale
1.	Cross section 8-8, axes: 3,6,7,8	1:50
2.	Cross section 9-9 axis: 9	1:50
3.	Cross section 10-10 axis: 10	1:50
4.	Cross section 12-12 axes: 11,12,15,16	1:50
5.	Cross section 14-14 axes: 13,14	1:50
6.	Cross section 2-2 axis: 2	1:50
7.	Cross section 17-17 axis: 17	1:50
8.	Longitudinal sections C-C on axis 2 and 17	1:50
9.	Ground plan of the attic (South wing)	1:220
10.	Ground plan of the attic with domes (South wing)	1:300
11.	Longitudinal sections D-D, B-B, roof structure	1:220
12.	Top views on Domes sections 1-1, 2-2, 3-3	1:100
13.	Cross section of the Dome section K-D - K-D	1:100
14.	Detail: Tie beam to Queen post	1:10
15.	Detail: Knee brace to Queen post/Middle purlin	1:10
16.	Detail: Brace to Tie beam	1:10
17.	Detail: Brace/Straining beam to Queen post	1:10
18.	Detail: Rafter connection and connection of Middle purlins	1:10
19.	Detail: Rafter to Middle purlin	1:10
20.	Detail: Brace to Ties	1:10
21.	Detail: Rafter to Ties and Foot purlin	1:10
22.	Detail: Middle purlin to Queen post	1:10

CROSS SECTION 8-8, AXES: 3,6,7,8 M-1:50



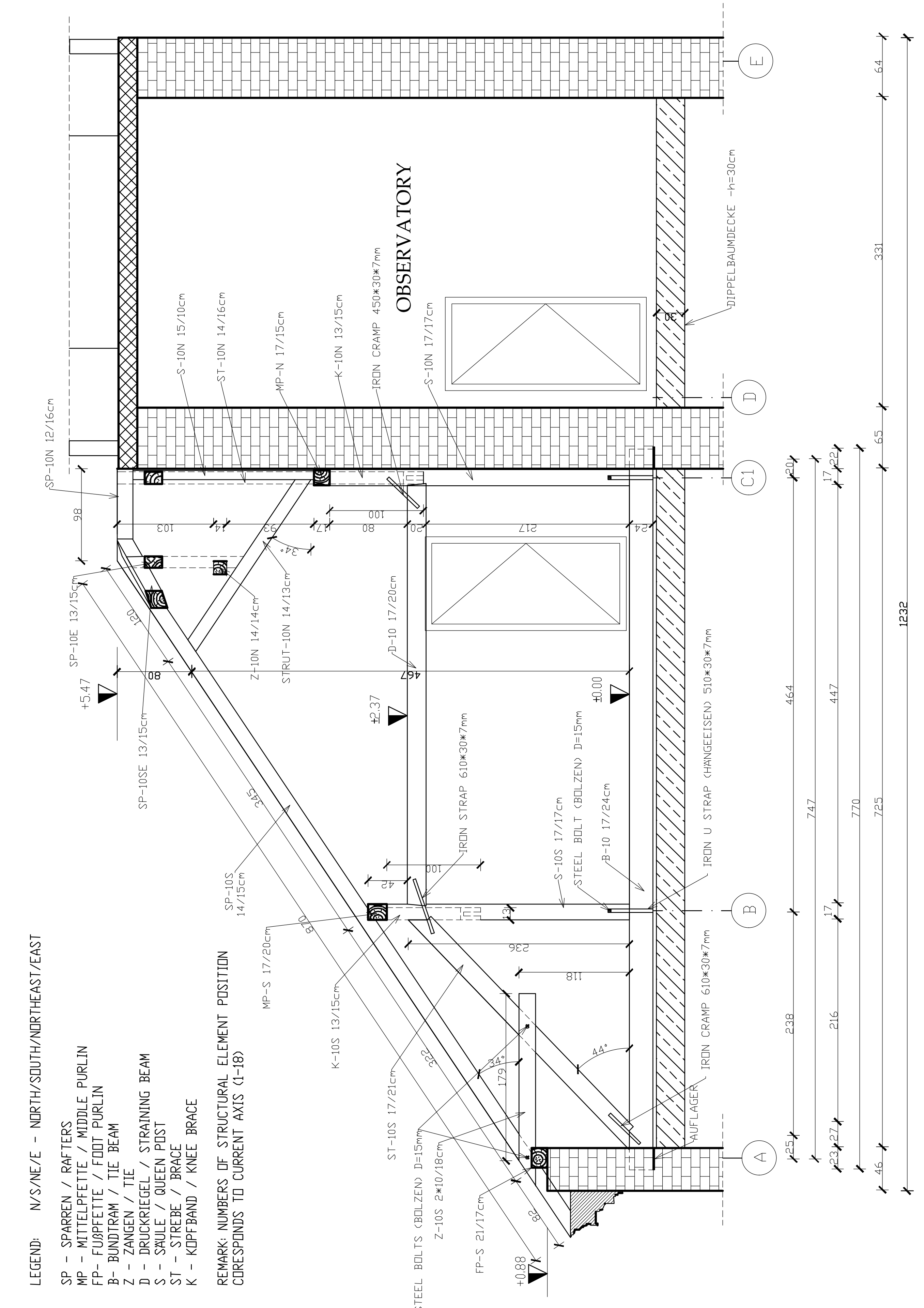
Plan number	Project:	Drawing:		Scale:
1.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Cross section 8-8, axes: 3,6,7,8	lignum study research engineering	1:50
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19
				Author: Alen Malagić
				Format: 210/297mm

CROSS SECTION 9-9 AXIS : 9 M-1:50



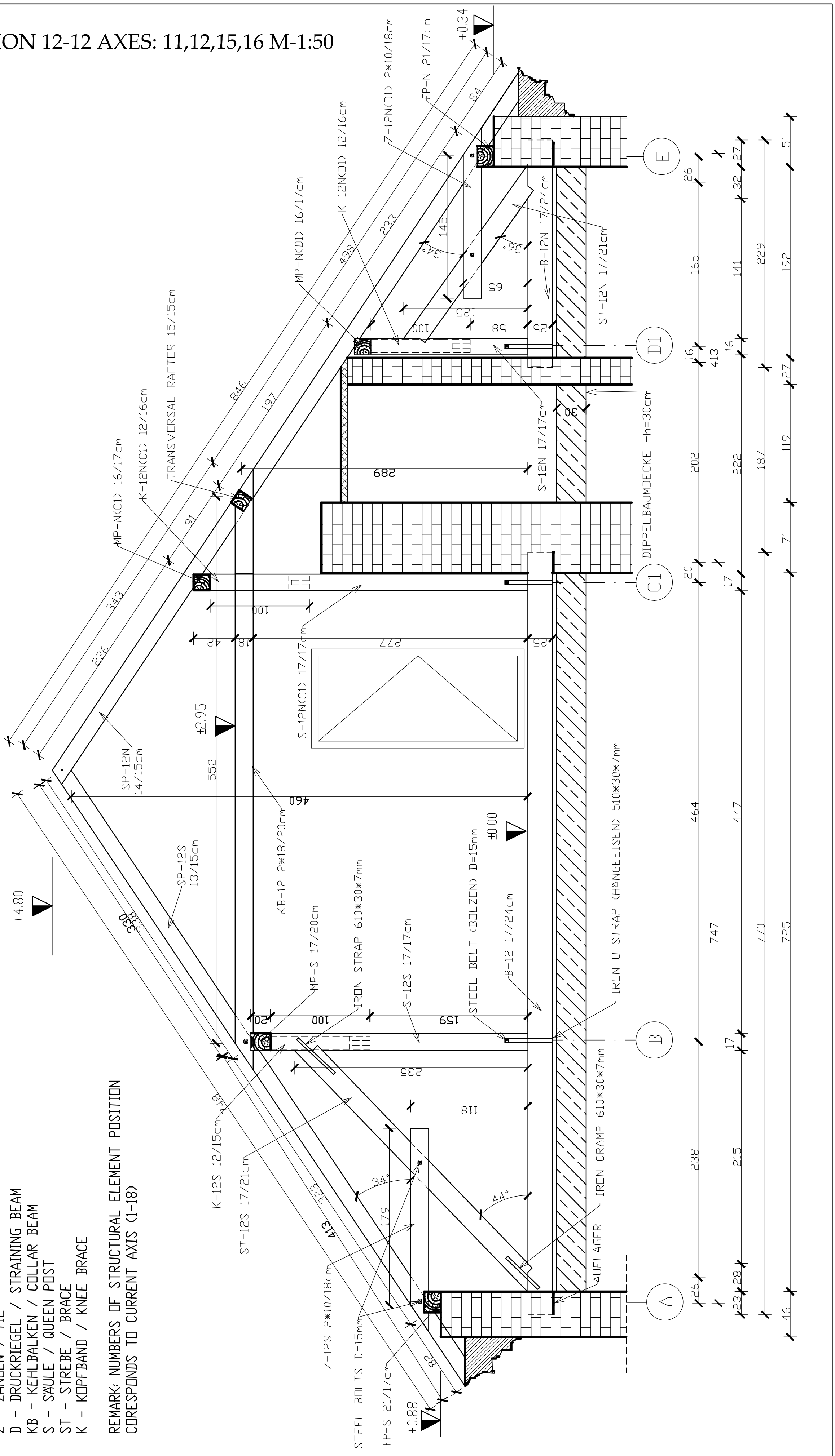
Plan number	Project:	Drawing:		Scale:
2.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Cross section 9-9 axis : 9	lignum study research engineering	1:50
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19
				Author: Alen Malagić
				Format: 210/297mm

CROSS SECTION 10-10 AXIS: 10 M-1:50



Plan number	Project:	Drawing:		Scale: 1:50
3.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Cross section 10-10 axis: 10	lignum study research engineering / TU Graz	Date: 26/03/19
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at		Author: Alen Malagić
				Format: 210/297mm

CROSS SECTION 12-12 AXES: 11,12,15,16 M-1:50



LEGEND:

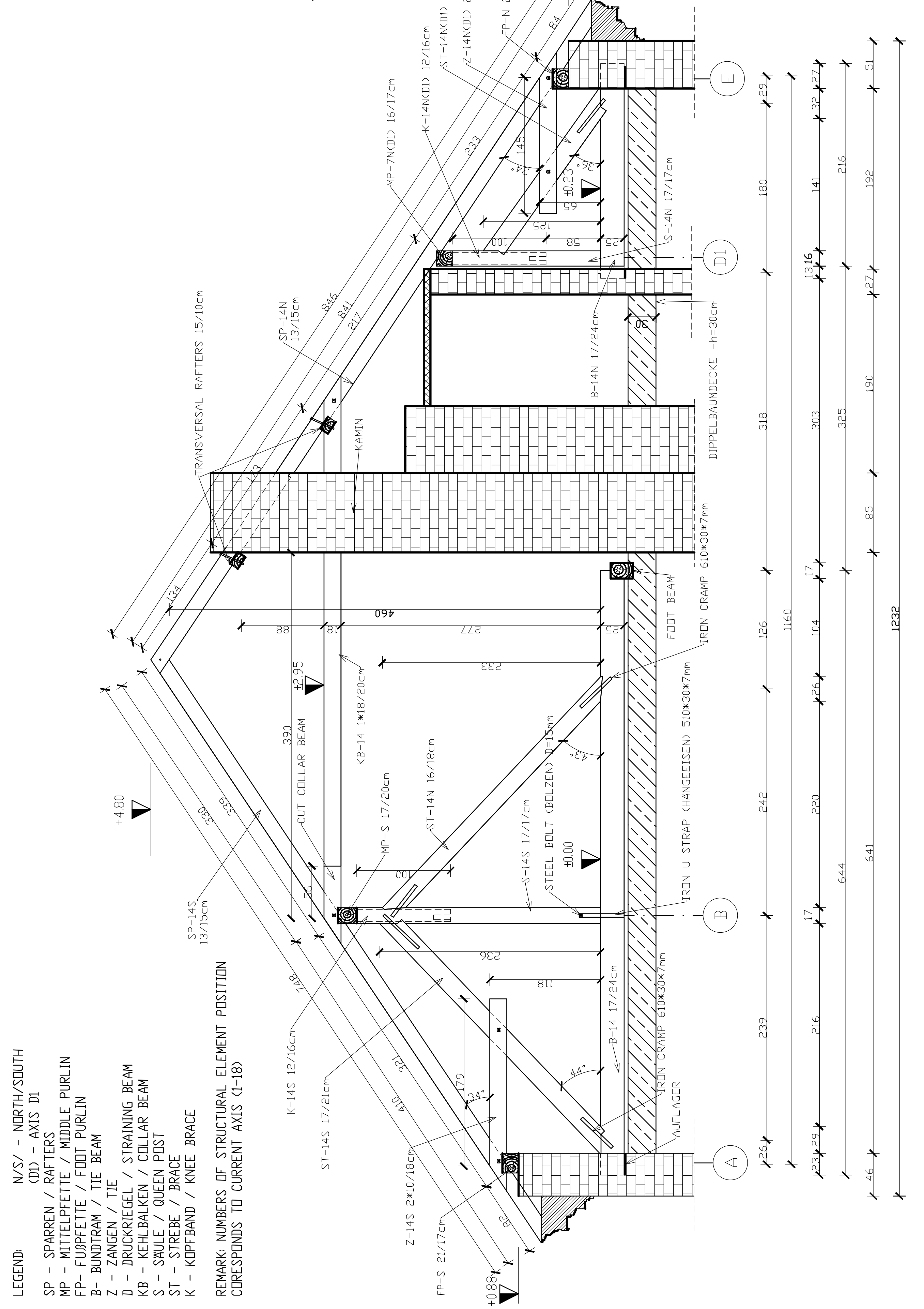
N/S/ - NORTH/SOUTH
(C1)/(D1) - AXIS C1/D1

SP - SPARREN / RAFTERS
MP - MITTELPFETTE / MIDDLE PURLIN
FP- FUßPFETTE / FOOT PURLIN
B- BUNDTRAM / TIE BEAM
Z - ZANGEN / TIE
D - DRUCKRIEGEL / STRAINING BEAM
KB - KEHLBALKEN / COLLAR BEAM
S - SÄULE / QUEEN POST
ST - STREBE / BRACE
K - KOPFBAND / KNEE BRACE

REMARK: NUMBERS OF STRUCTURAL ELEMENT POSITION CORESPONDS TO CURRENT AXIS (1-18)

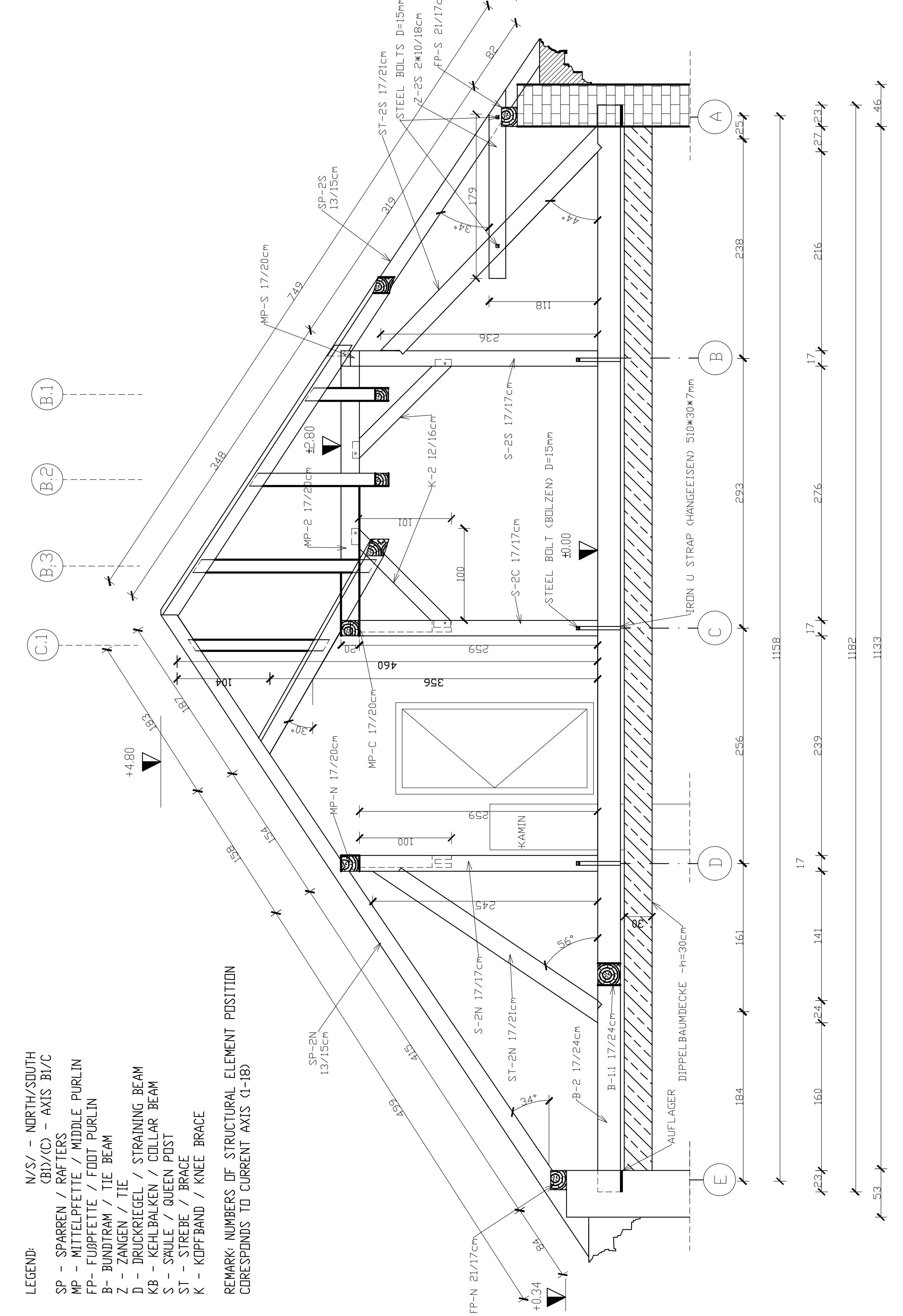
Plan number	Project:	Drawing:		Scale:
4.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Cross section 12-12 axes: 11,12,15,16	lignum study research engineering / TU Graz	1:50
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at		Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

CROSS SECTION 14-14 AXES: 13,14 M-1:50



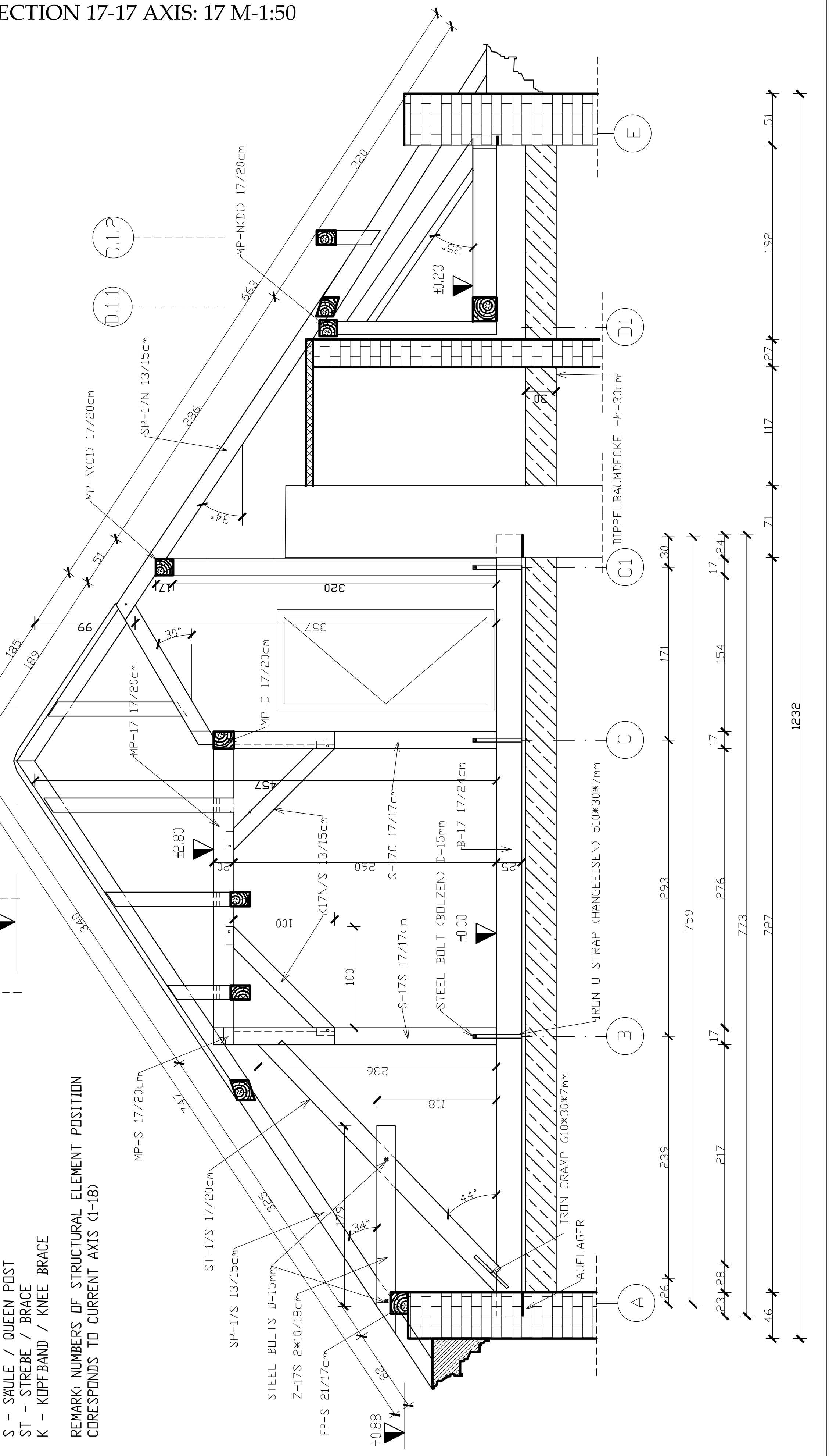
Plan number	Project:	Drawing:		Scale:
5.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Cross section 14-14 axes: 13,14	lignum study research engineering	1:50
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19
				Author: Alen Malagić
				Format: 210/297mm

CROSS SECTION 2-2 AXIS: 2 M-1:50



Plan number	Project:	Drawing:		Scale:
6.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Cross section 2-2 axis: 2	lignum study research engineering	1:50
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19
				Author: Alen Malagić
				Format: 210/297mm

CROSS SECTION 17-17 AXIS: 17 M-1:50



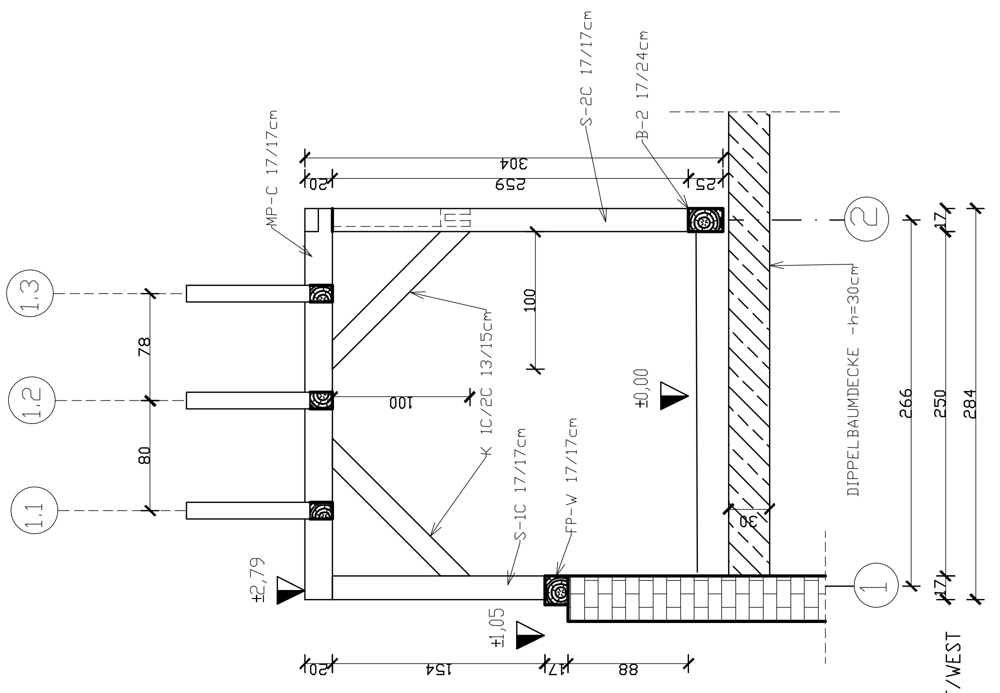
LEGEND:

- SP - SPARREN / RAFTERS
- MP - MITTELPFETTE / MIDDLE PURLIN
- FP- FUßPFETTE / FOOT PURLIN
- B- BUNDTRAM / TIE BEAM
- Z - ZANGEN / TIE
- D - DRUCKRIEGEL / STRAINING BEAM
- KB - KEHLBALKEN / COLLAR BEAM
- S - SÄULE / QUEEN POST
- ST - STREBE / BRACE
- K - KOPFBAND / KNEE BRACE

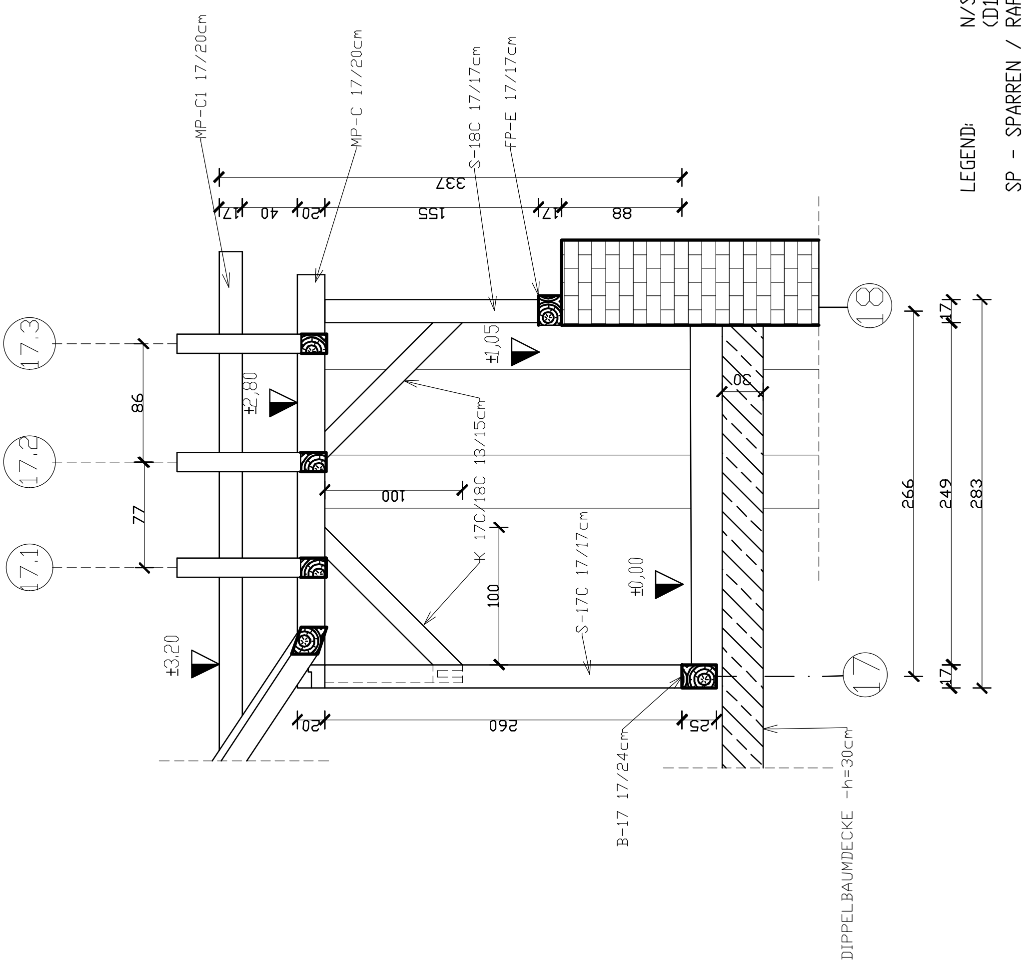
REMARK: NUMBERS OF STRUCTURAL ELEMENT POSITION CORESPONDS TO CURRENT AXIS (1-18)

Plan number	Project:	Drawing:		Scale:
7.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Cross section 17-17 axis: 17 Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	Scale: 1:50 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

LONGITUDINAL SECTION C-C AX: 1-2 M-1:50



LONGITUDINAL SECTION C-C AX: 17-18 M-1:50



LEGEND: N/S/E/W - NORTH/SOUTH/EAST/WEST
(D1)(C1) - AXIS D1/C1

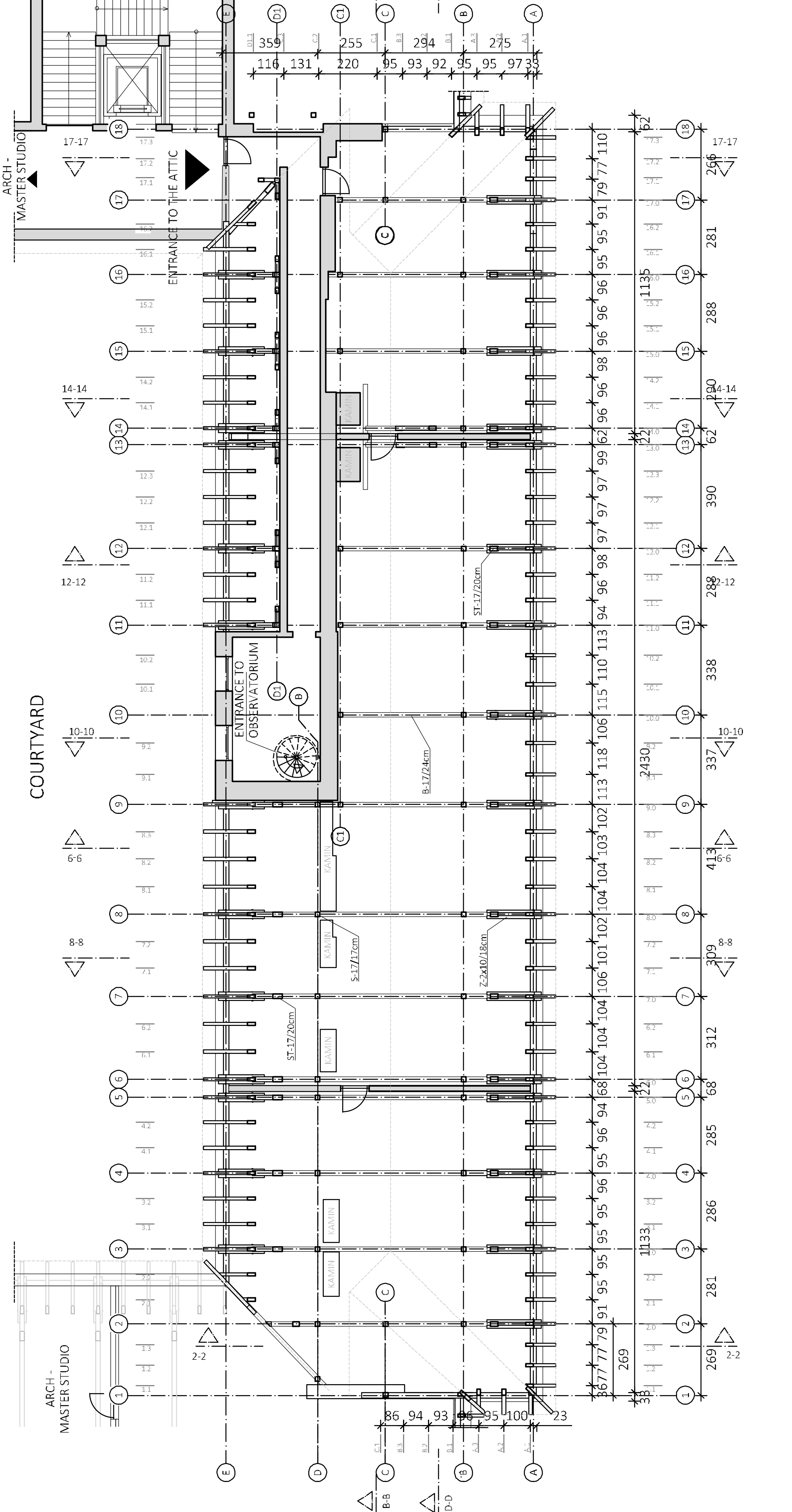
SP - SPARREN / RAFTERS
MP - MITTELPFETTE / MIDDLE PURLIN
FP- FUßPFETTE / FOOT PURLIN
B- BUNDTRAM / TIE BEAM
Z - ZANGEN / TIE
D - DRUCKRIEGEL / STRAINING BEAM
KB - KEHLBALKEN / COLLAR BEAM
S - SÄULE / QUEEN POST
ST - STREBE / BRACE
K - KOPFBAND / KNEE BRACE

REMARK: NUMBERS OF STRUCTURAL ELEMENT POSITION CORESPONDS TO CURRENT AXIS (1-18)

Plan number	Project:	Drawing:		Scale:
8.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Longitudinal sections C-C on axis 2 and 17	lignum study research engineering	1:50
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

GROUND PLAN - ATTIC M 1:220

N

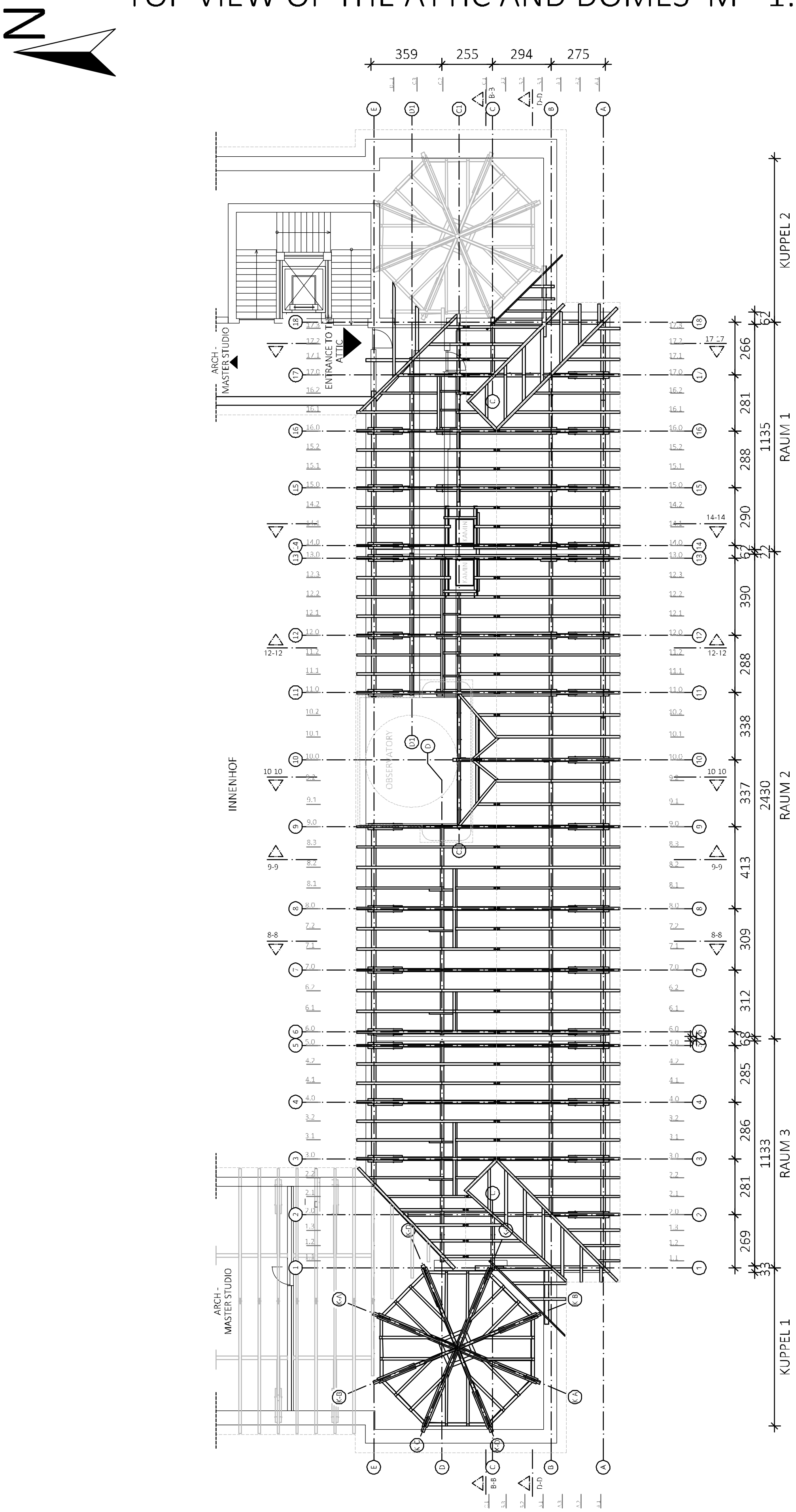


LEGEND:
SP - SPARREN / RAFTERS
MP - MITTELPFETTE / MIDDLE PURLIN
FP- FUßPFETTE / FOOT PURLIN
B- BUNDTRAM / TIE BEAM
Z - ZANGEN / TIE
D - DRUCKRIEGEL / STRAINING BEAM
S - SÄULE / QUEEN POST
ST - STREBE / BRACE
K - KOPFBAND / KNEE BRACE

REMARK: NUMBERS OF STRUCTURAL ELEMENT POSITION CORESPONDS TO CURRENT AXIS (1-18)

Plan number	Project:	Drawing:		Scale:
9.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Ground plan of the attic (South wing) Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	1:220 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

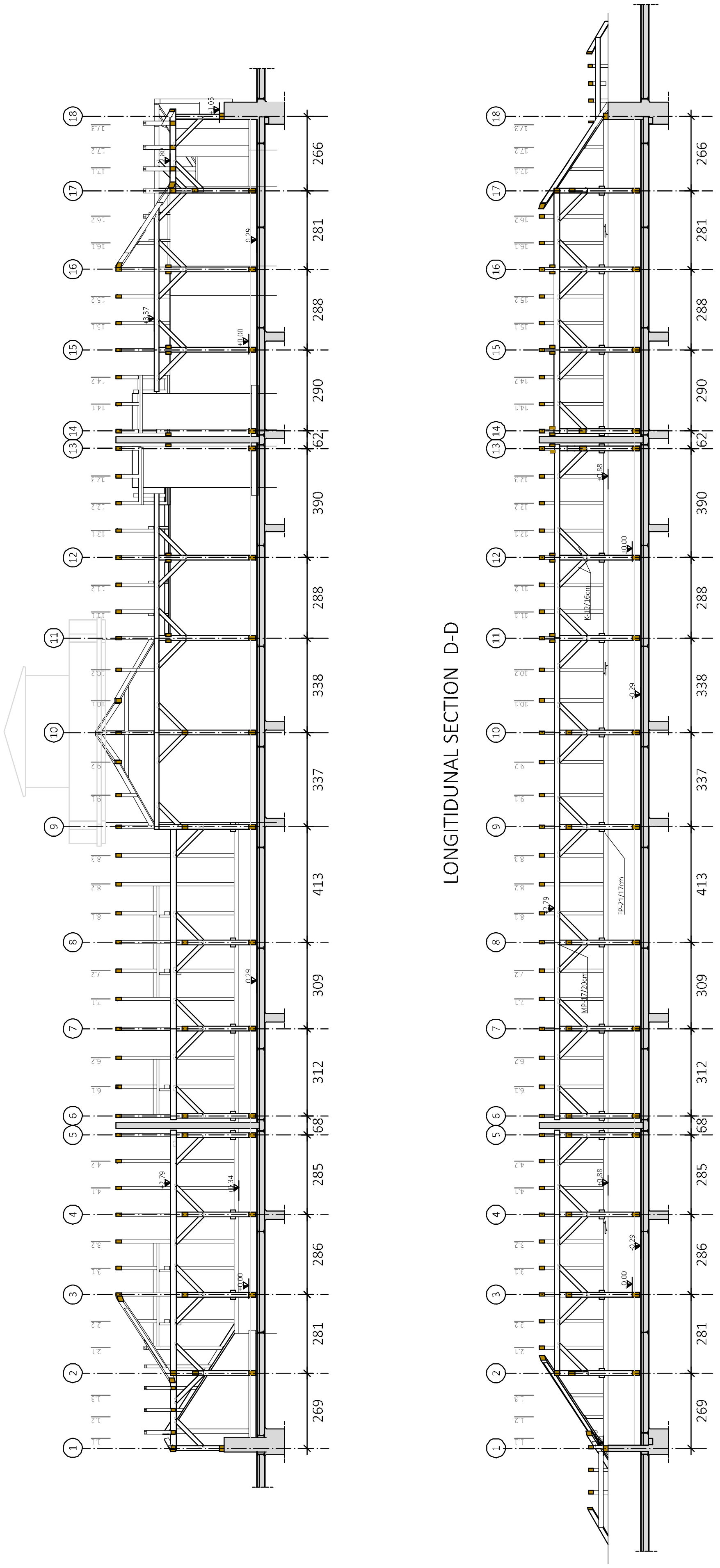
TOP VIEW OF THE ATTIC AND DOMES M - 1:300



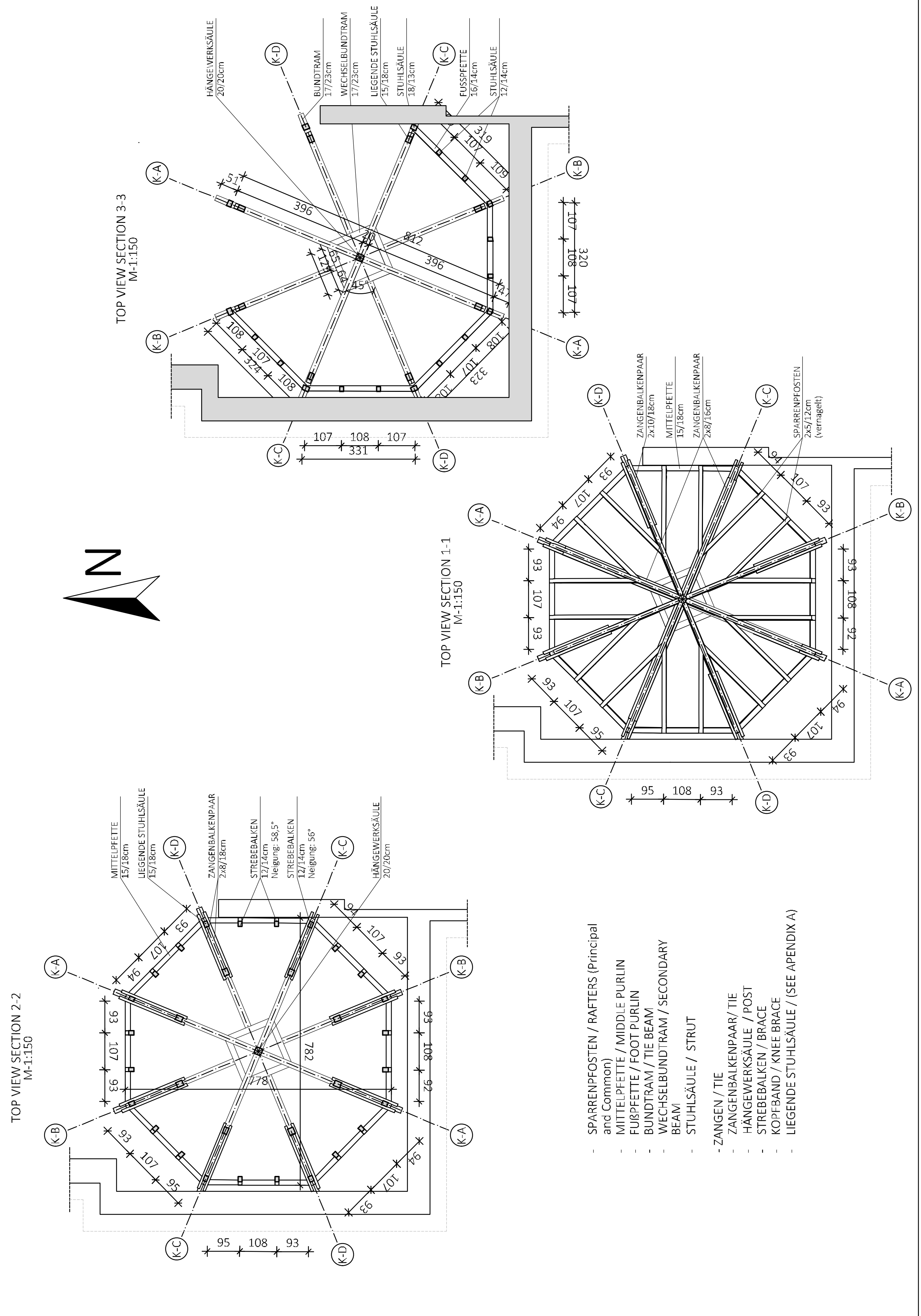
Plan number	Project:	Drawing:		Scale: 1:220
10.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Ground plan of the attic with domes (South wing) Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

LEGEND:
SP - SPARREN / RAFTERS
MP - MITTELPFETTE / MIDDLE PURLIN
FP- FUẞPFETTE / FOOT PURLIN
B- BUNDTRAM / TIE BEAM
Z - ZANGEN / TIE
D - DRUCKRIEGEL / STRAINING BEAM
S - SÄULE / QUEEN POST
ST - STREBE / BRACE
K - KOPFBAND / KNEE BRACE

REMARK: NUMBERS OF STRUCTURAL ELEMENT POSITION CORESPONDS TO CURRENT AXIS (1-18)

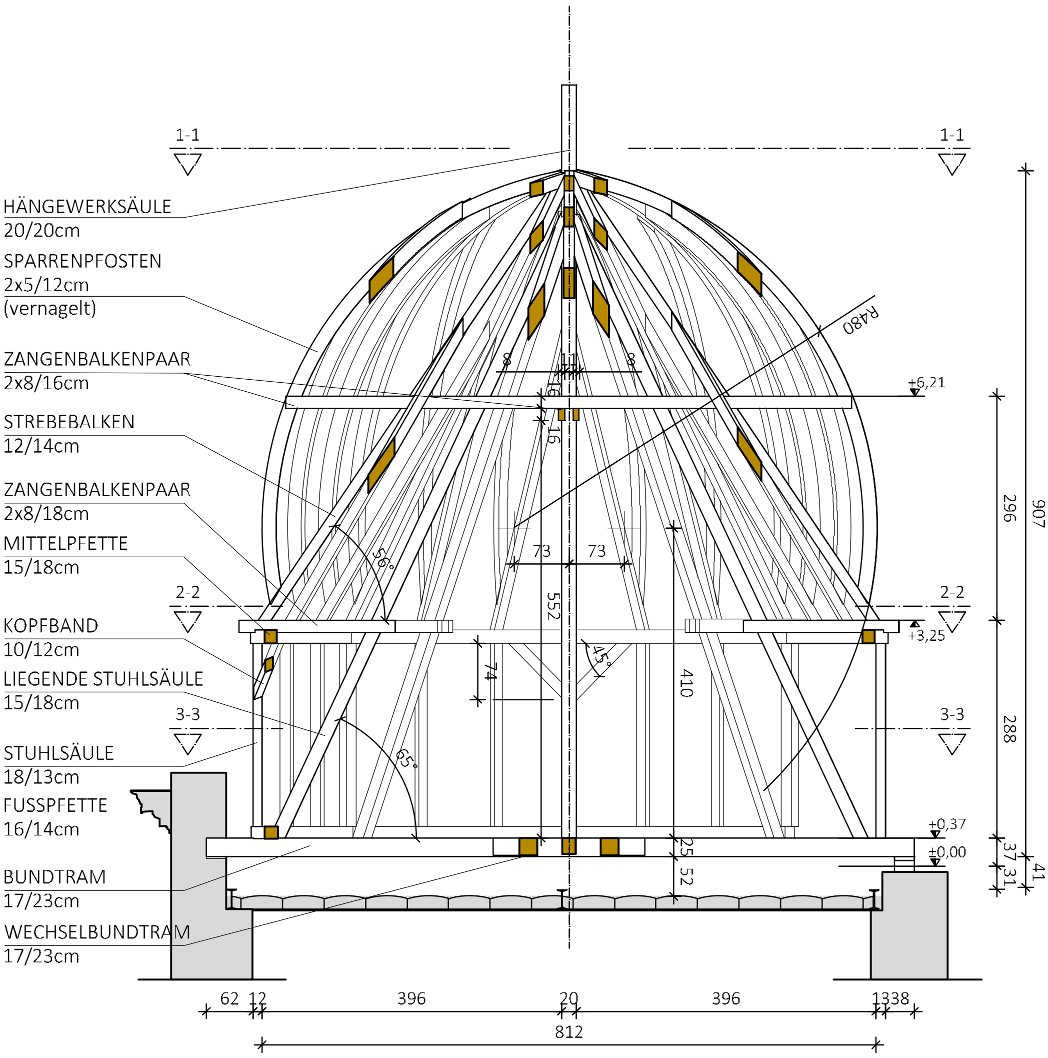


Plan number	Project:	Drawing:		Scale:
11.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Longitidunal sections D-D, B-B, roof structure	lignum study research engineering	1:220
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19 Author: Alen Malagić Format: 210/297mm



Plan number	Project:	Drawing:		Scale:
12.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Top views on Domes sections 1-1, 2-2, 3-3 Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	Scale: 1:100 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

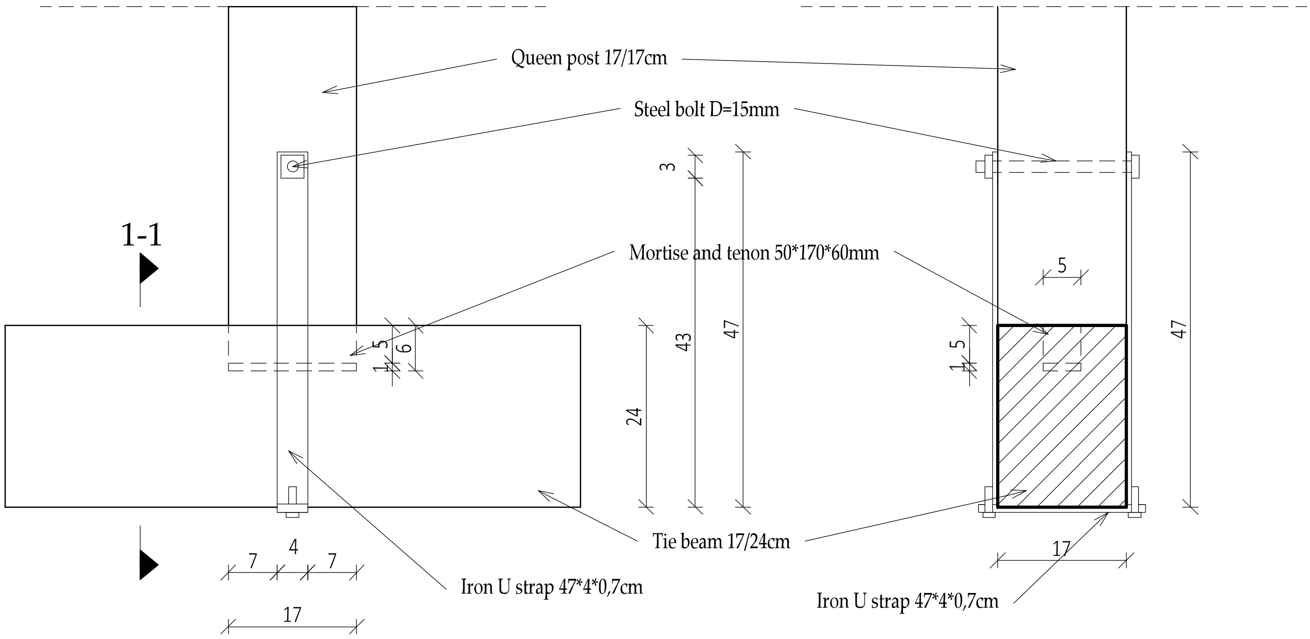
CROSS SECTION K-D - K-D
M-1:100

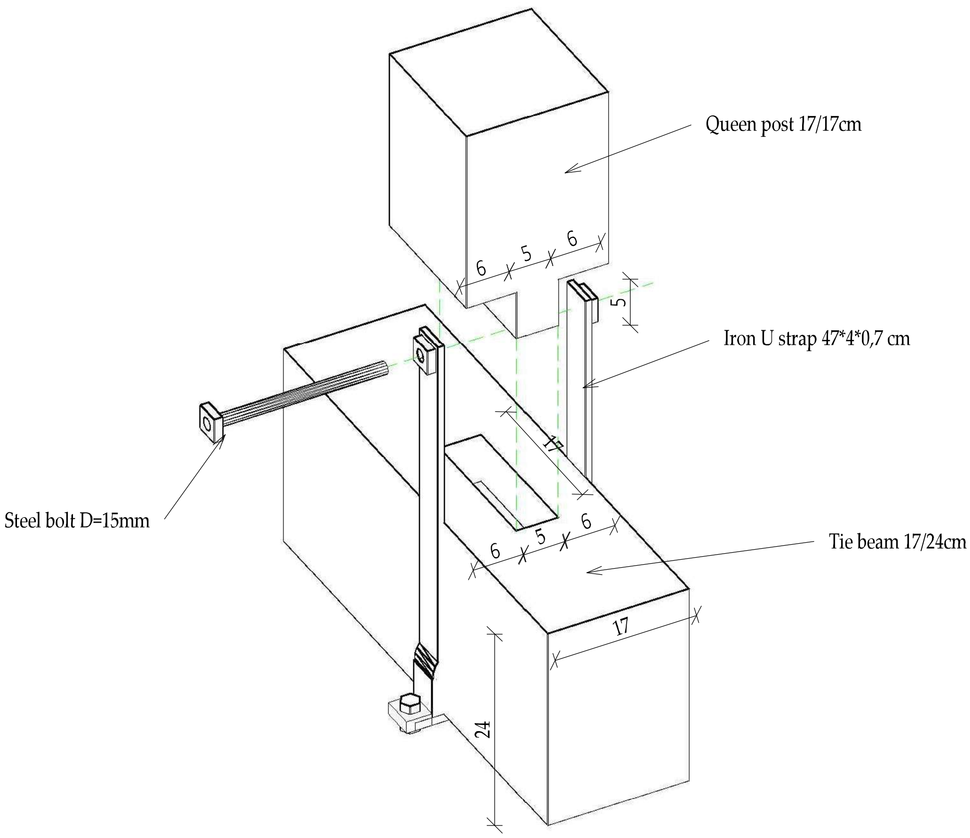


- SPARRENPFOSTEN / RAFTERS (Principal and Common)
- MITTELPFETTE / MIDDLE PURLIN
- FUßPFETTE / FOOT PURLIN
- BUNDTRAM / TIE BEAM
- WECHSELBUNDTRAM / SECONDARY BEAM
- STUHLSÄULE / STRUT
- ZANGEN / TIE
- ZANGENBALKENPAAR/ TIE
- HÄNGEWERKSÄULE / POST
- STREBEBALKEN / BRACE
- KOPFBAND / KNEE BRACE
- LIEGENDE STUHLSÄULE / (SEE APENDIX A)

13.	Project: Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Drawing: Cross section of the Dome section K-D - K-D Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	Scale: 1:100 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

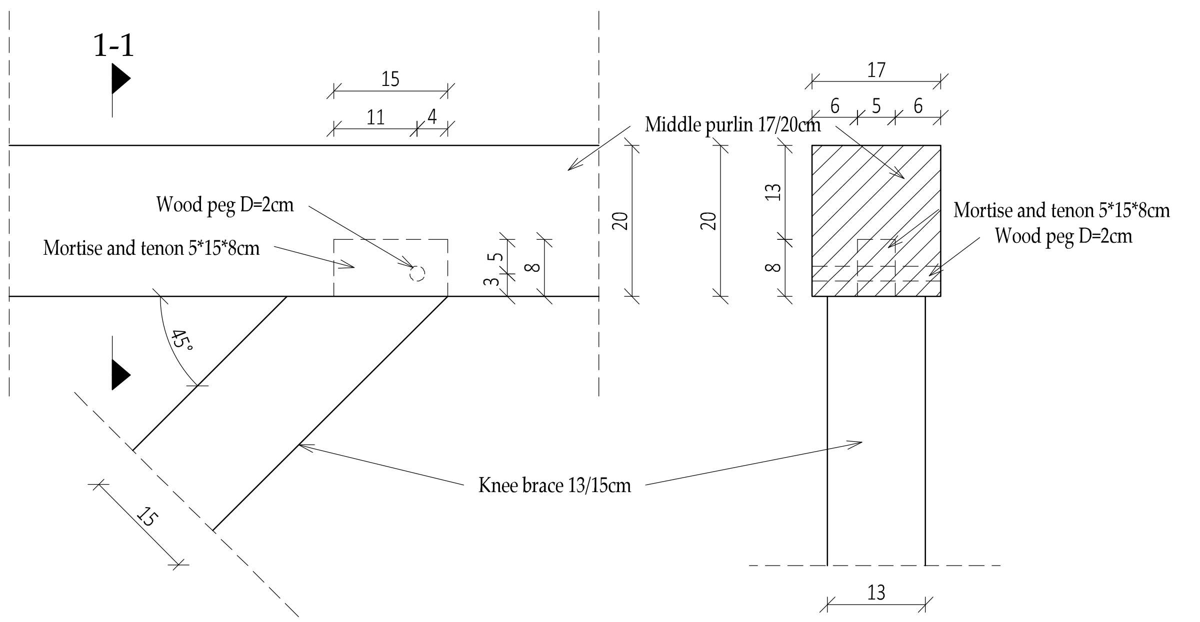
Detail: Tie beam to Queen post M - 1:10

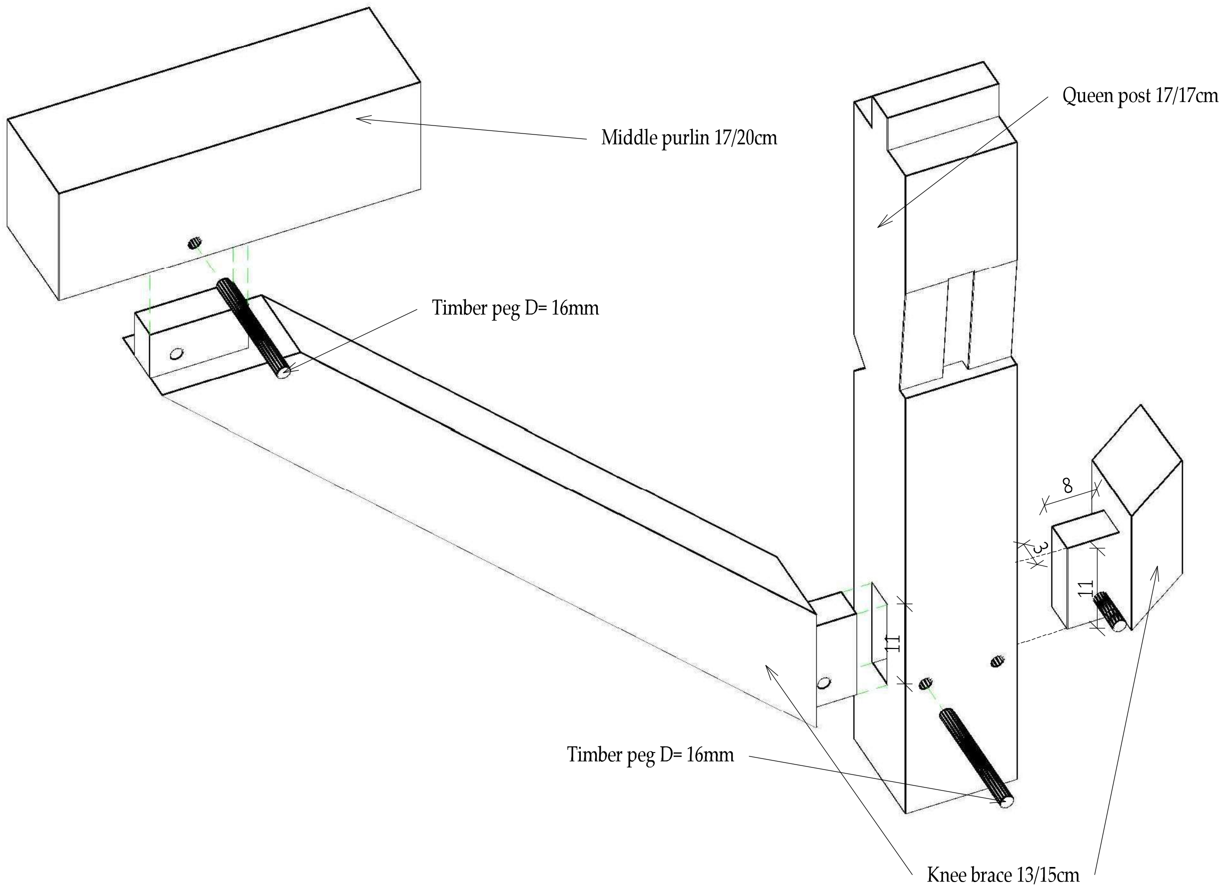




Plan number	Project:	Drawing:		Scale:
14.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Tie beam to Queen post	lignum study research engineering	1:10
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19
				Author: Alen Malagić
				Format: 210/297mm

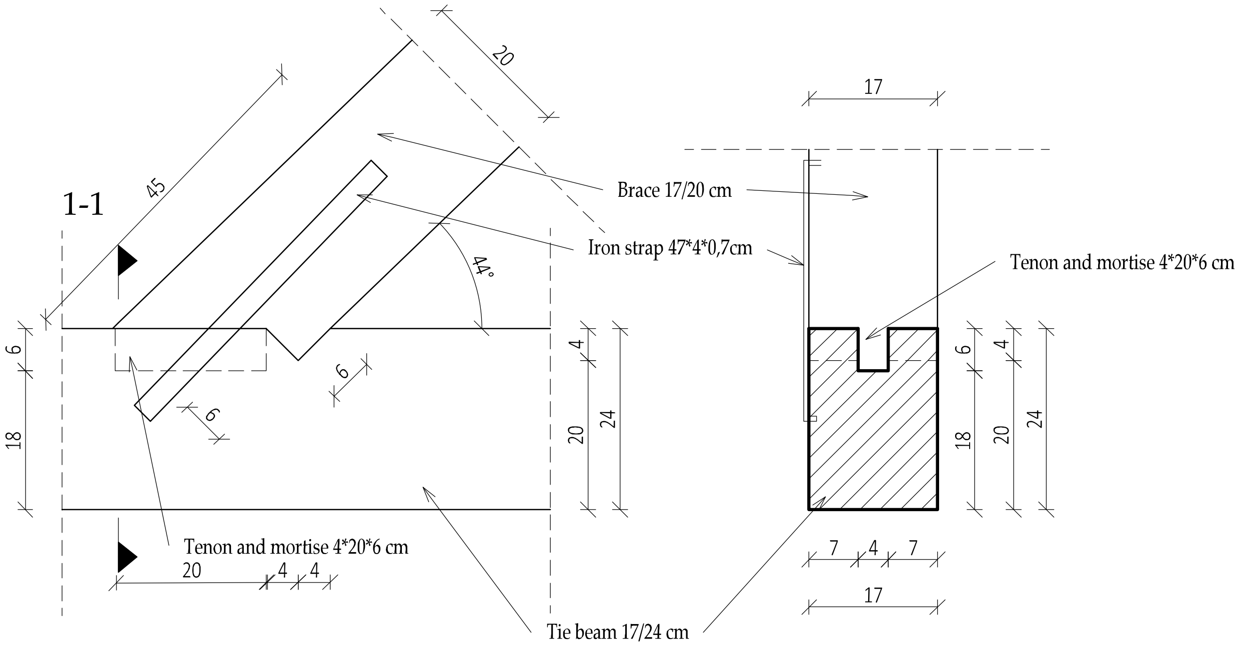
Detail: Knee brace to Queen post/Middle purlin M - 1:10

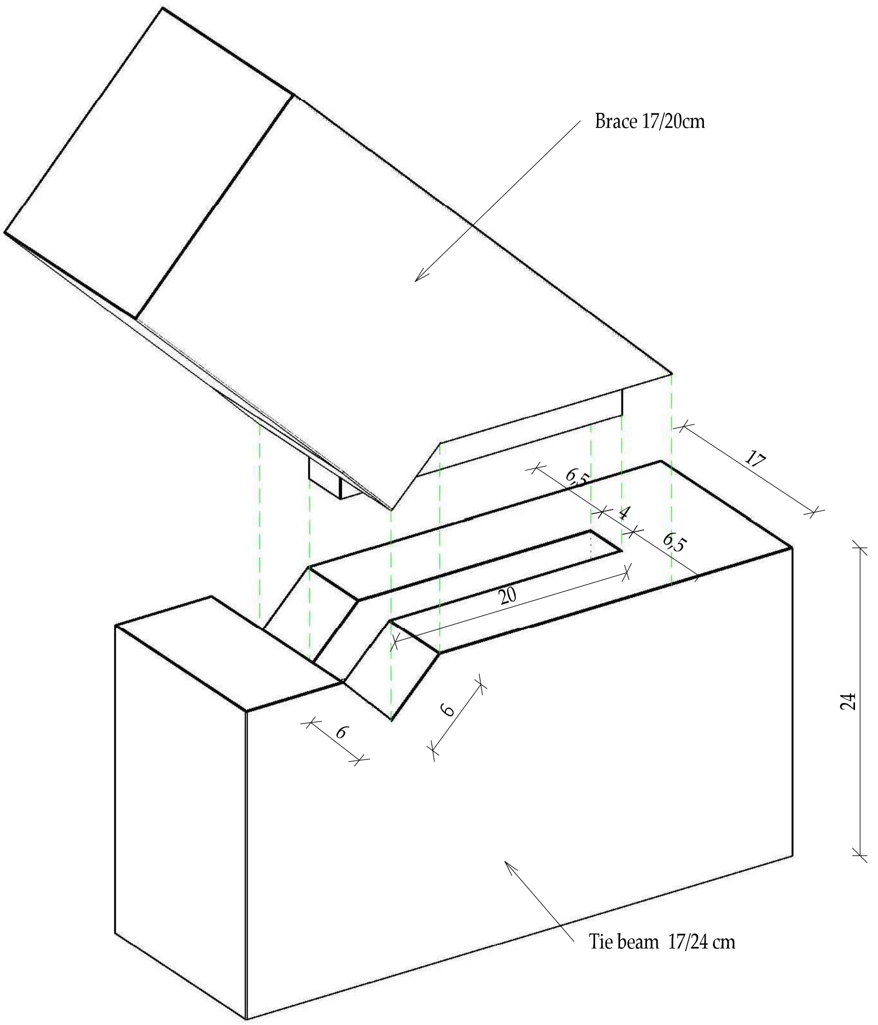




Plan number	Project:	Drawing:		Scale:
15.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Knee brace to Queen post/Middle purlin Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	1:10 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

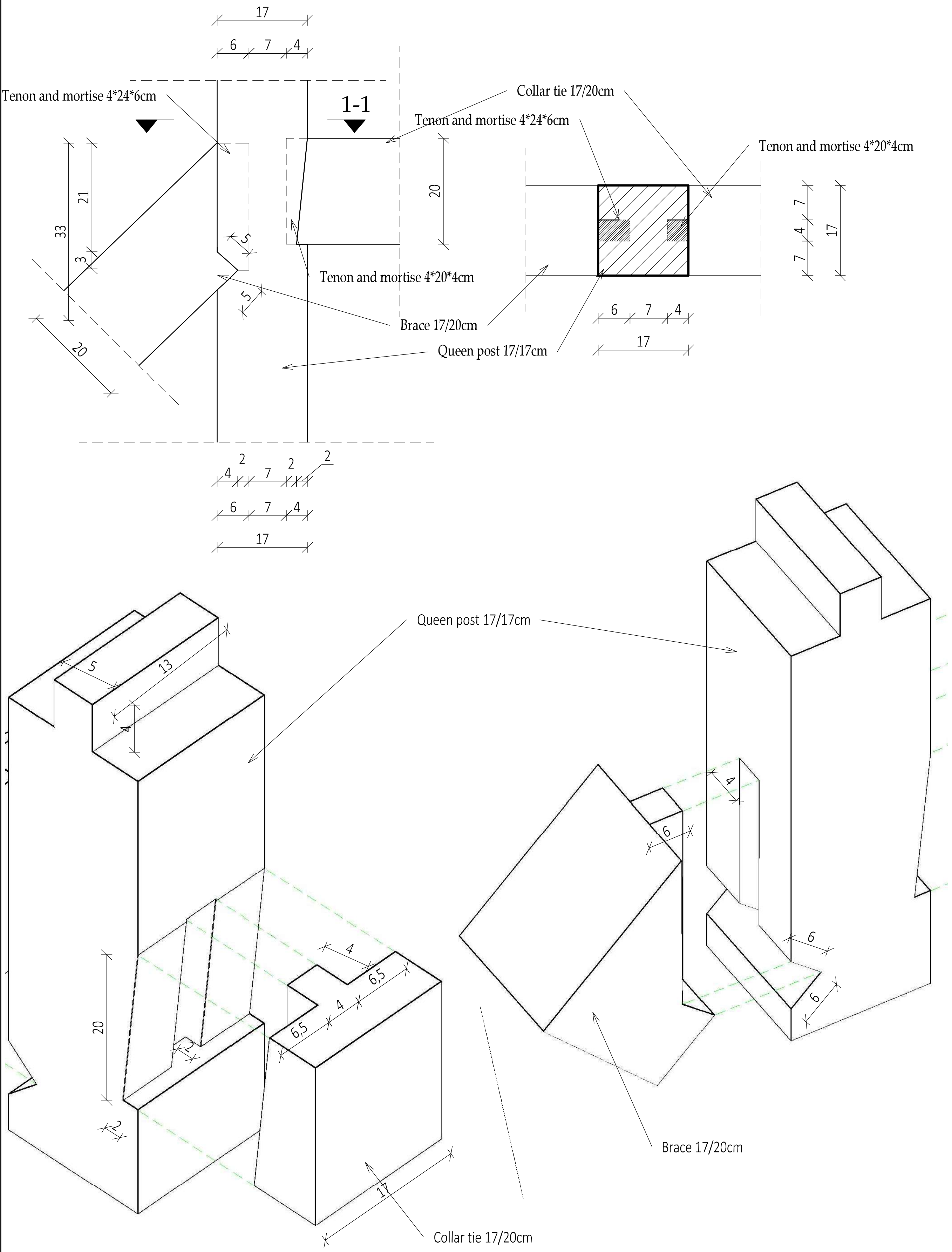
Detail: Brace to Tie beam M - 1:10





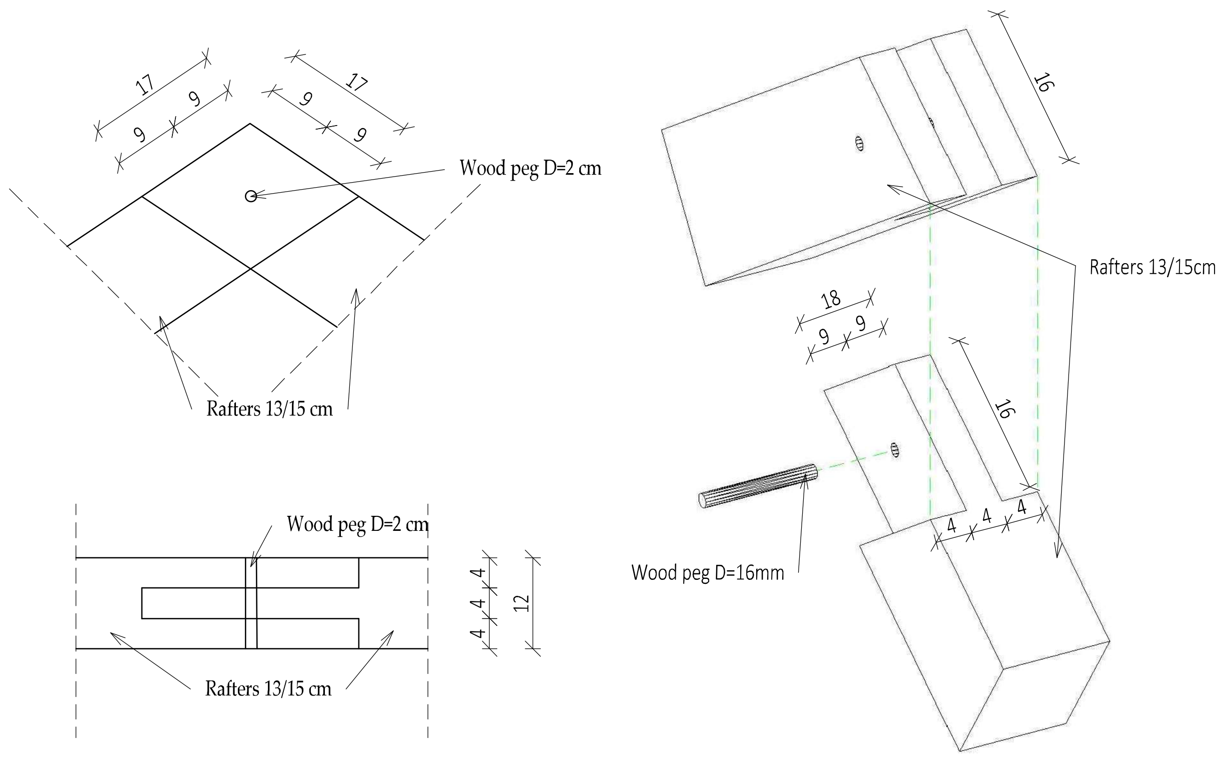
Plan number	Project:	Drawing:		Scale:
16.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Brace to Tie beam	lignum study research engineering	1:10
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

Detail: Brace/Straining beam to Queen post M - 1:10

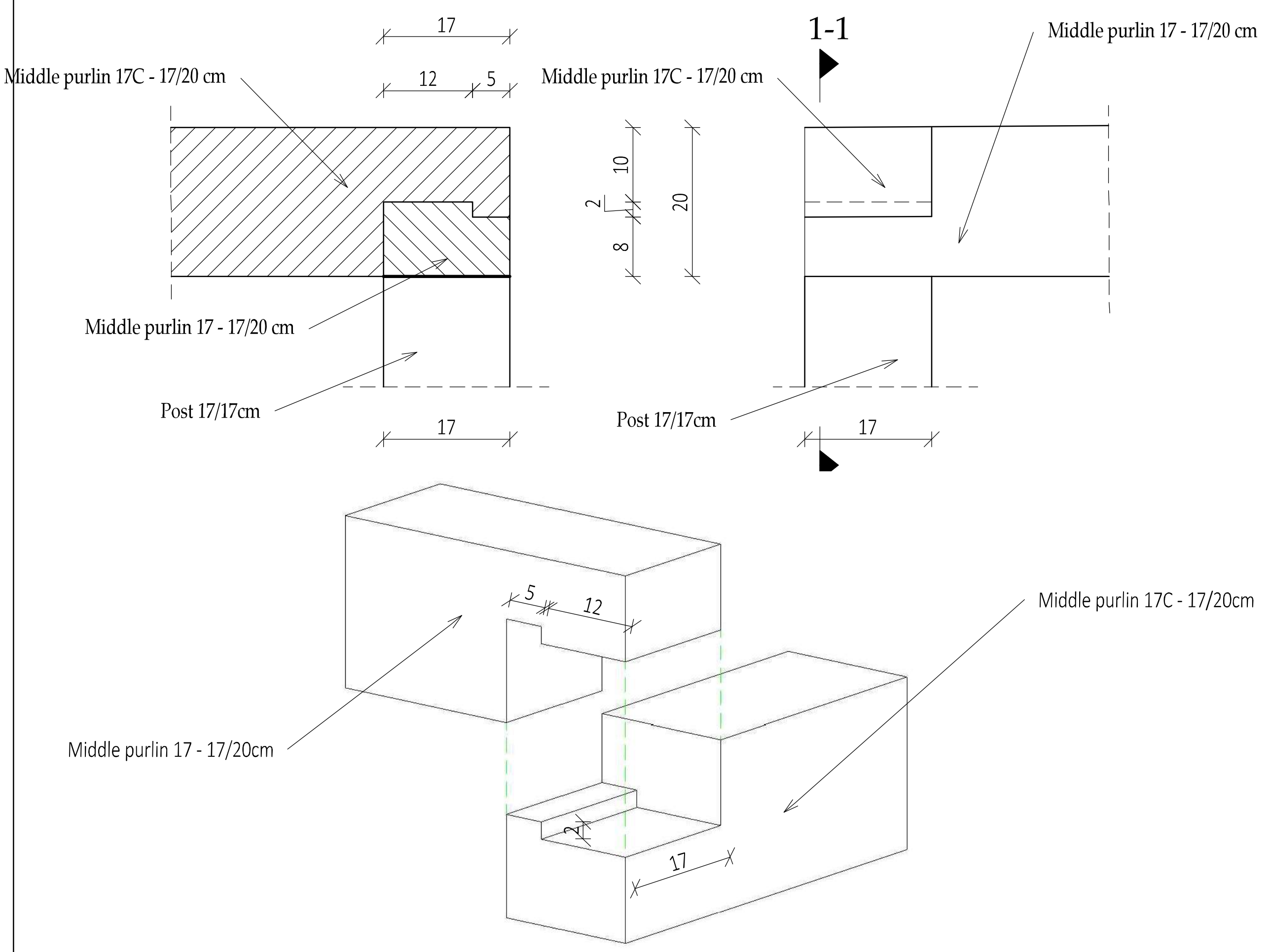


Plan number	Project:	Drawing:		
17.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Brace/Straining beam to Queen post Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	Scale: 1:10 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

Detail: Rafter connection (Ridge) M - 1:10

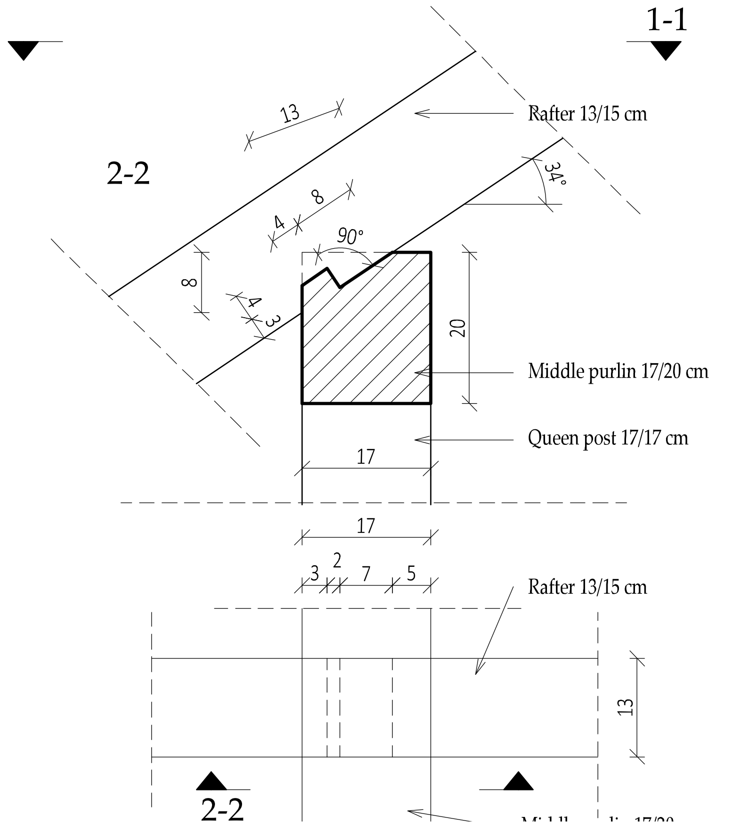


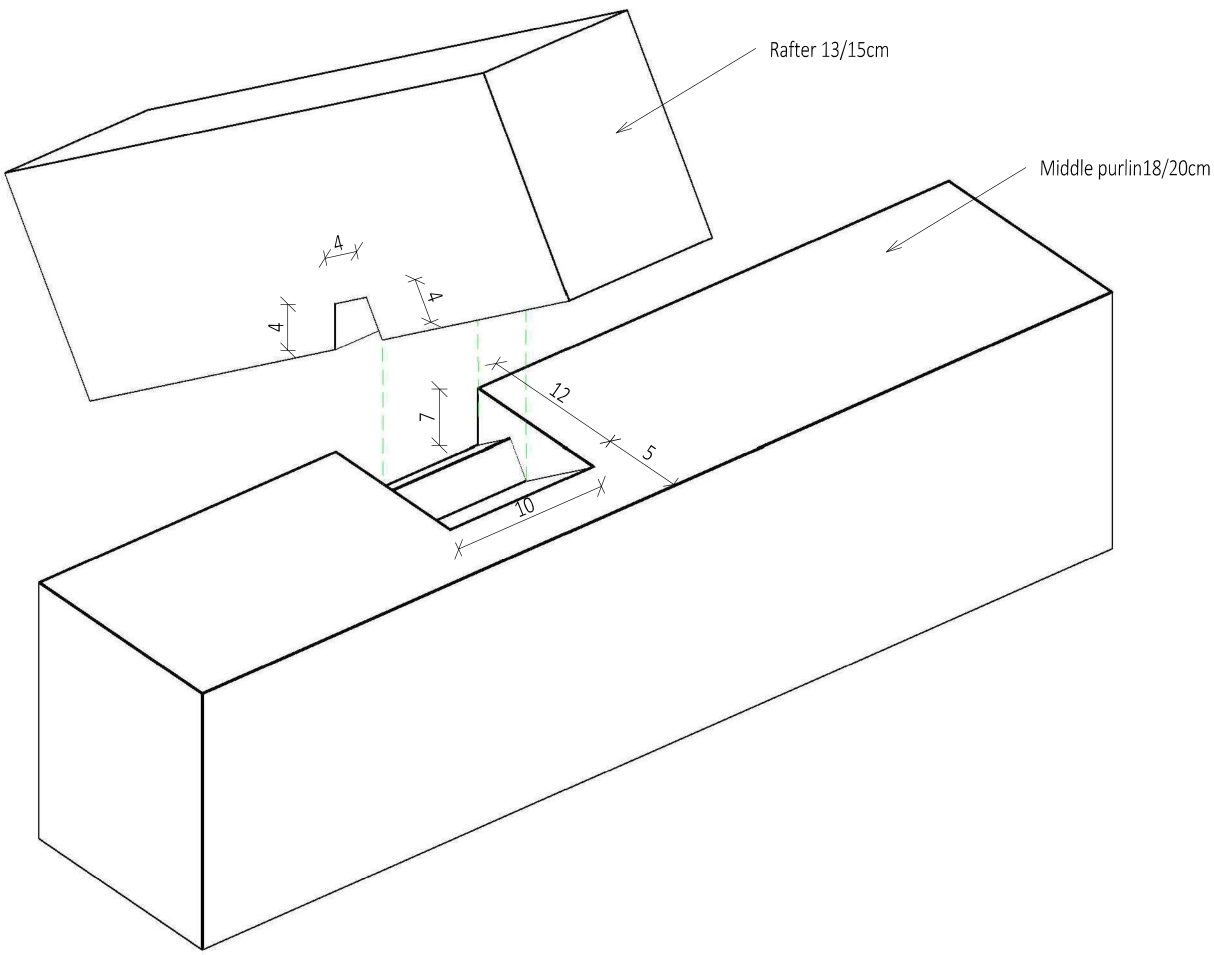
Detail: Middle purlin to Middle purlin M - 1:10



Plan number	Project:	Drawing:		Scale:
18.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Rafter connection and connection of Middle purlins Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	1:10 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

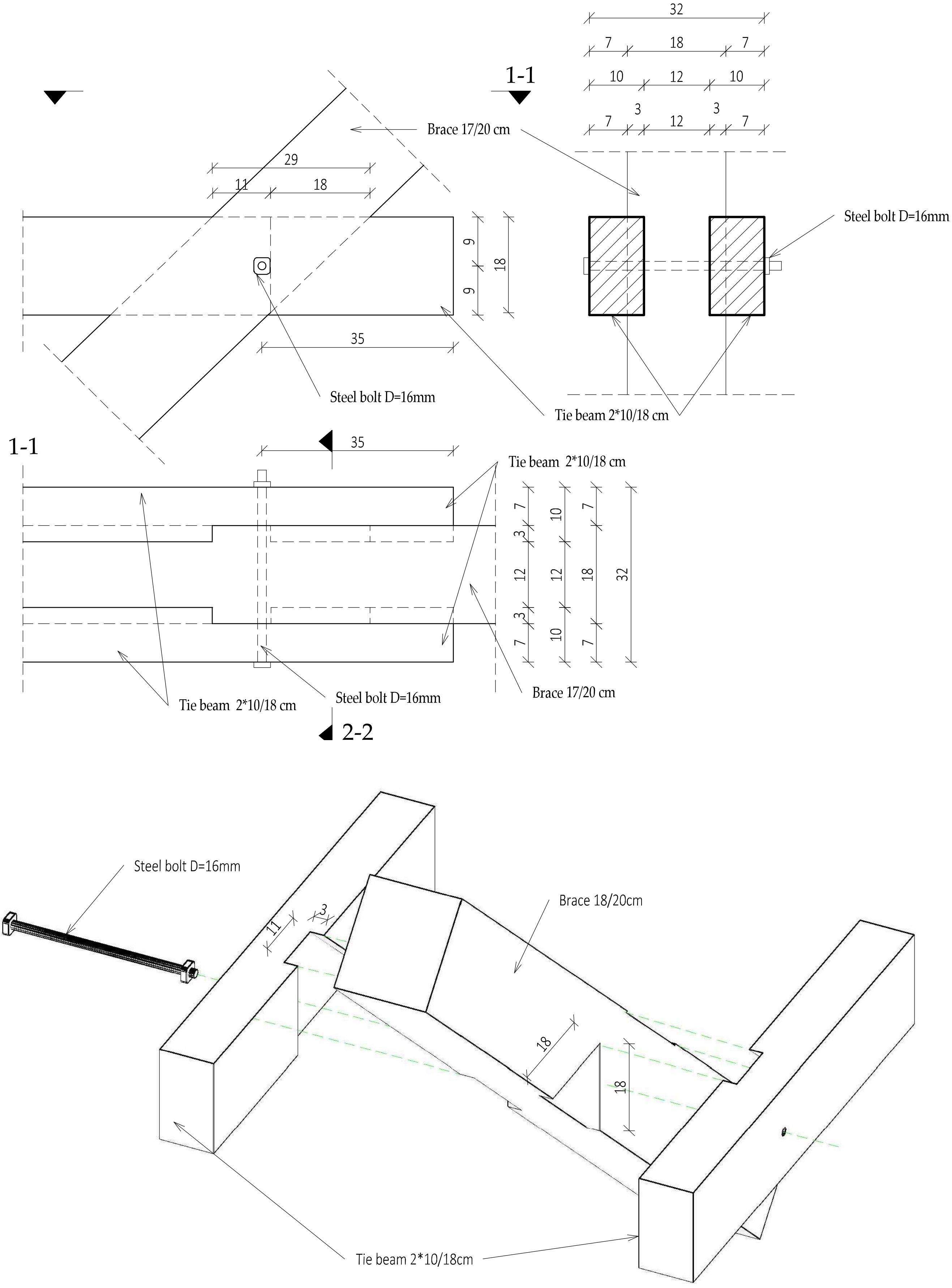
Detail: Rafter to Middle purlin M - 1:10





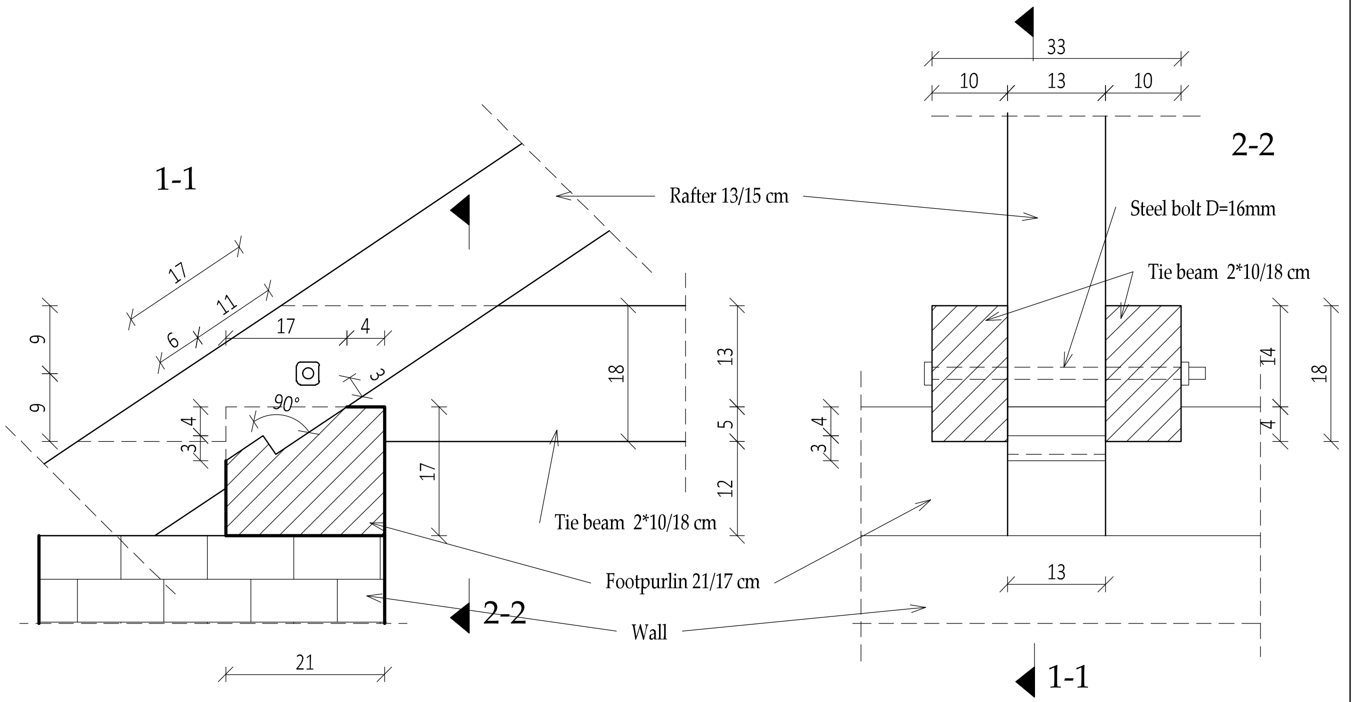
Plan number	Project:	Drawing:		Scale:
19.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Rafter to Middle purlin Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering TU Graz	Scale: 1:10 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

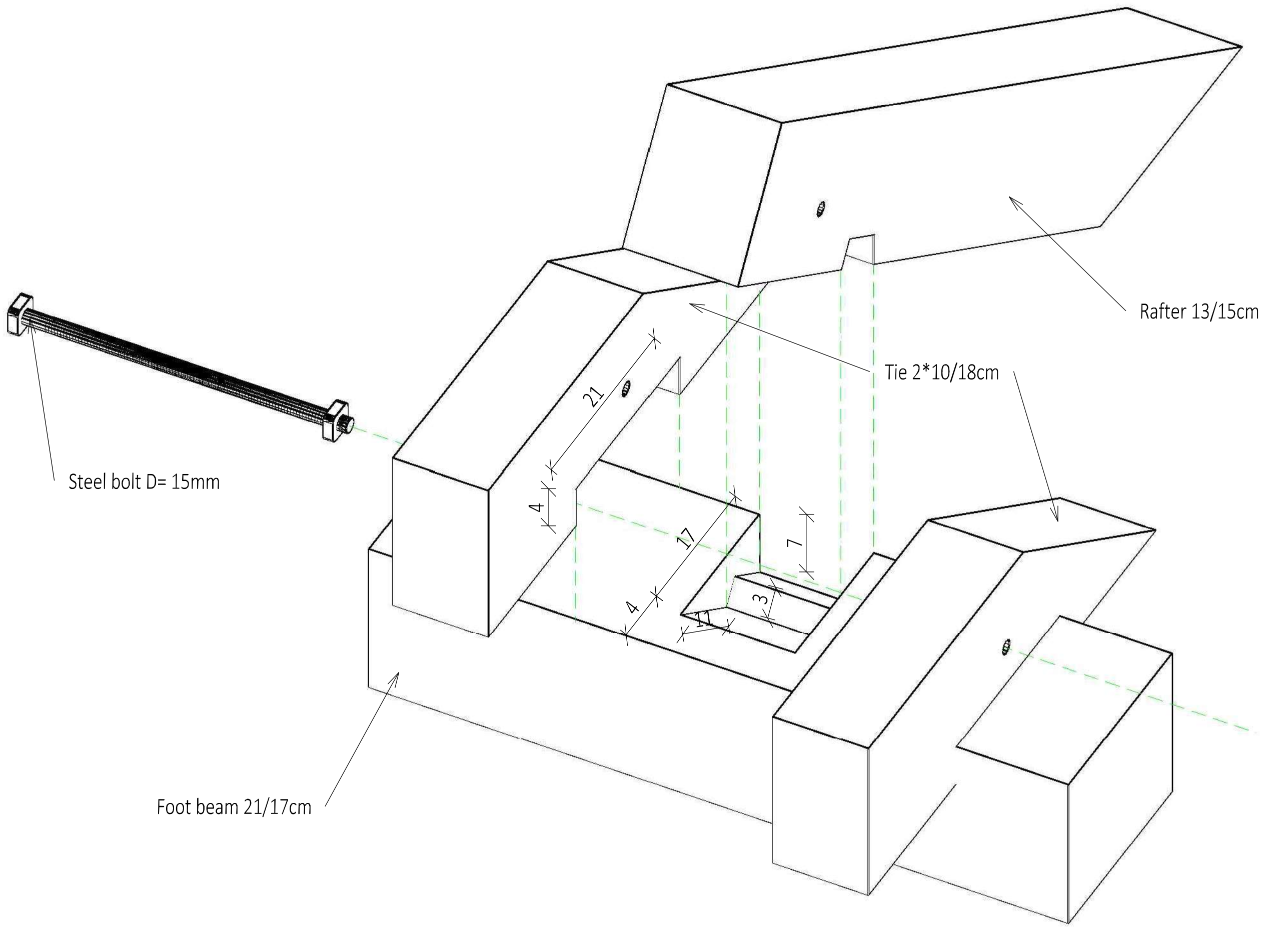
Detail: Brace to Ties M - 1:10



Plan number	Project:	Drawing:		Scale:
20.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Brace to Ties Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	lignum study research engineering / TU Graz	1:10 Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

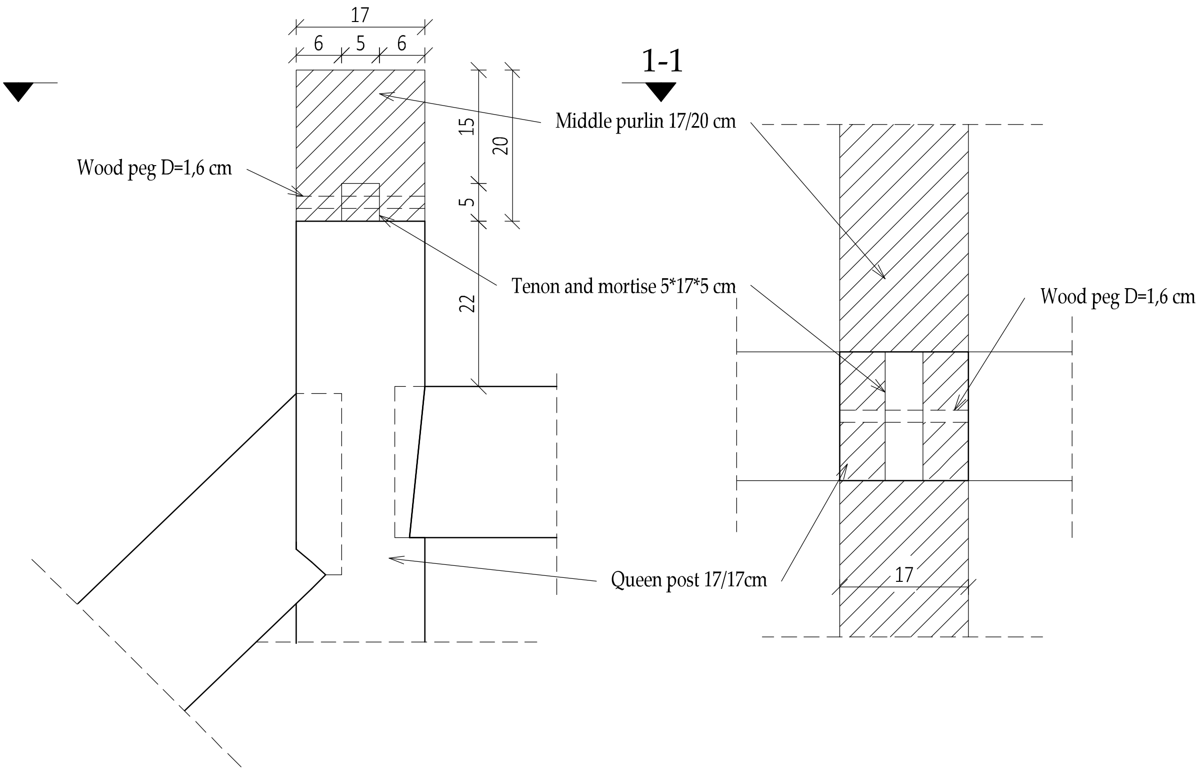
Detail: Rafter to Ties and Foot purlin M - 1:10

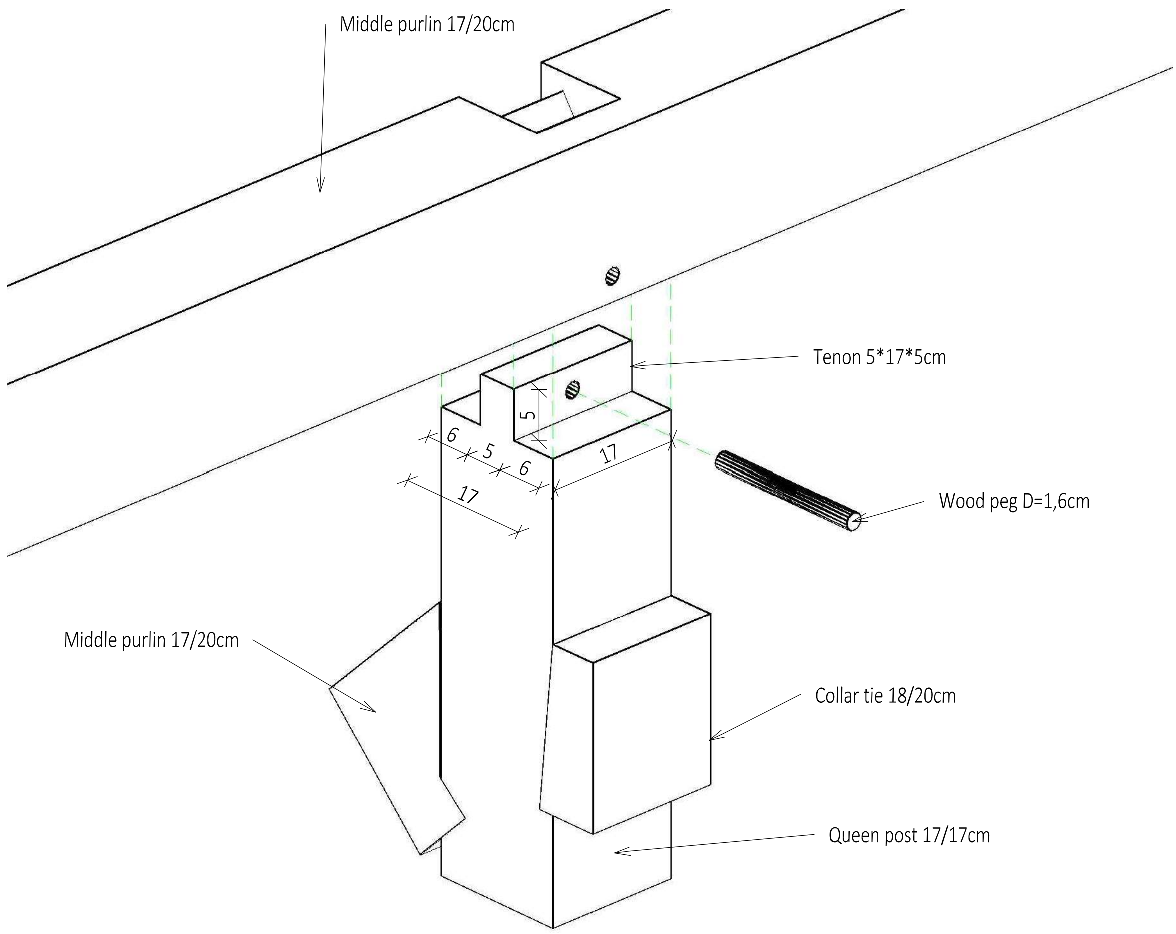




Plan number	Project:	Drawing:		Scale:
21.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Rafter to Ties and Foot purlin	lignum study research engineering	1:10
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19 Author: Alen Malagić Format: 210/297mm

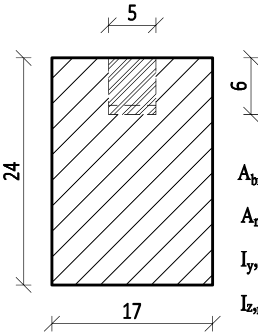
Detail: Middle purlin to Queen post M - 1:10





Plan number	Project:	Drawing:		Scale:
22.	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Detail: Middle purlin to Queen post	lignum study research engineering	1:10
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19
				Author: Alen Malagić
				Format: 210/297mm

Tie beam-queen post



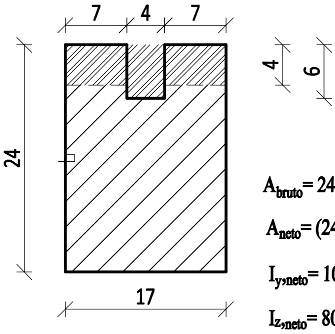
$A_{bruto} = 24{,}0cm*17{,}0cm = 408{,}0cm^2$

$A_{neto} = (24{,}0*17{,}0) - (5{,}0*6{,}0) = 378{,}0cm^2$

$I_{y,neto} = 16871\ cm^2$

$I_{z,neto} = 9763\ cm^2$

Tie beam-queen post



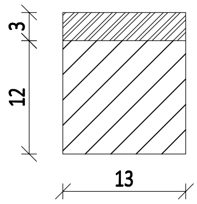
$A_{bruto} = 24{,}0cm*17{,}0cm = 408{,}0cm^2$

$A_{neto} = (24{,}0*17{,}0) - 2*(7{,}0*4{,}0) - (4{,}0*6{,}0) = 328{,}0cm^2$

$I_{y,neto} = 10446\ cm^4$

$I_{z,neto} = 8081\ cm^4$

rafter



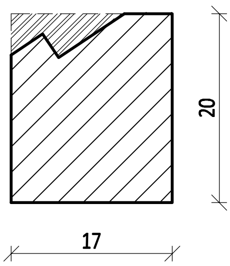
$A_{bruto} = 13{,}0\ cm*15{,}0\ cm = 195{,}0\ cm^2$

$A_{neto} = (13{,}0*15{,}0) - (3{,}0*13{,}0) = 156{,}0\ cm^2$

$I_{y,neto} = 1872\ cm^4$

$I_{z,neto} = 2197\ cm^4$

Middle purlin



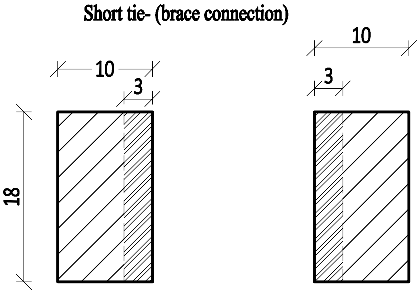
$A_{bruto} = 20{,}0cm*17{,}0cm = 340{,}0\ cm^2$

$A_{neto} = 307{,}23\ cm^2$

$I_{y,neto} = 8761\ cm^4$

$I_{z,neto} = 7424\ cm^4$

Short tie- (brace connection)



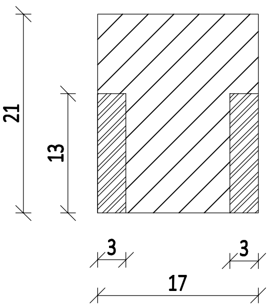
$A_{bruto} = 20{,}0\ cm*18{,}0\ cm = 360{,}0\ cm^2$

$A_{neto} = (20{,}0*18{,}0) - 2*(3{,}0*18{,}0) = 252{,}0\ cm^2$

$I_{y,neto} = 6804\ cm^4$

$I_{z,neto} = 1029\ cm^4$

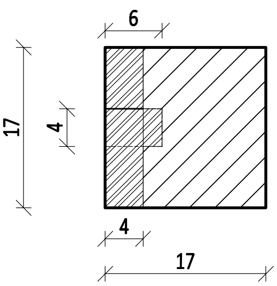
Brace



$A_{bruto} = 21{,}0\ cm*17{,}0\ cm = 357{,}0\ cm^2$

$A_{neto} = (21{,}0*17{,}0) - 2*(3{,}0*13{,}0) = 279{,}0\ cm^2$

queen post - brace-straining beam



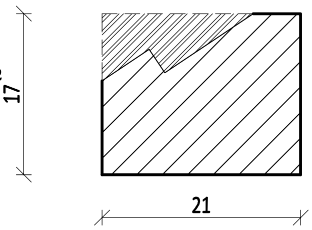
$A_{bruto} = 17{,}0cm*17{,}0cm = 289{,}0cm^2$

$A_{neto} = (17{,}0*17{,}0) - (6{,}0*4{,}0) - (4{,}0*17{,}0) = 197{,}0cm^2$

$I_{y,neto} = 5311\ cm^4$

$I_{z,neto} = 2858\ cm^4$

Foot purlin



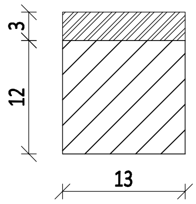
$A_{bruto} = 21{,}0\ cm*17{,}0\ cm = 357{,}0\ cm^2$

$A_{neto} = 292{,}0\ cm^2$

$I_{y,neto} = 5533\ cm^4$

$I_{z,neto} = 10603\ cm^4$

rafter (foot purlin connection)



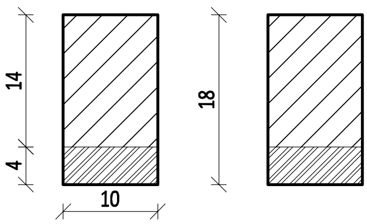
$A_{bruto} = 13{,}0\ cm*15{,}0\ cm = 195{,}0\ cm^2$

$A_{neto} = (13{,}0*15{,}0) - (3{,}0*13{,}0) = 156{,}0\ cm^2$

$I_{y,neto} = 1872\ cm^4$

$I_{z,neto} = 2197\ cm^4$

Short tie (foot purlin connection)



$A_{bruto} = 18{,}0\ cm*20{,}0\ cm = 360{,}0\ cm^2$

$A_{neto} = (18{,}0*20{,}0) - 2*(4{,}0*10{,}0) = 280{,}0\ cm^2$

$I_{y,neto} = 4573\ cm^4$

$I_{z,neto} = 2333\ cm^4$

Plan number	Project:	Drawing:		Scale:
-	Master thesis : Historic timber roof structures, assessment and analysis on example of roof structure " *Tu Graz Alte Technik*"	Neto cross sections with cross section properties	lignum study research engineering	1:10
		Graz University of Technology Institute for Timber engineering and Wood Technology Infeldgasse 24 8010 Graz , Tel: 43 (0)316 873 - 4601 ,www.lignum.at	TU Graz	Date: 26/03/19 Author: Alen Malagić Format: 210/297mm