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Practical Structural Design Guide for the Preservation and Renovation of Historical Stone Buildings

Bachelor's thesis
Degree Programme in Construction Engineering
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Degree Programme in Construction Engineering — Abstract
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This bachelor’s thesis aims to develop a comprehensive guide for the renovation of various historic structures, providing valuable examples and practical solutions for those involved in heritage building preservation. Designed as a resource for both inexperienced engineers and homeowners and structural engineers with experience, this guide offers innovative solutions to common structural challenges encountered in historic buildings renovation projects. The thesis is divided into two sections: the first presents the theoretical foundations of heritage preservation, and the second applies these principles to a specific renovation project.

The thesis outlines essential preservation concepts, historical materials and current guidelines. Topics include architectural styles, the importance of conservation and preservation of historical structures, geotechnical considerations and challenges during renovation, energy efficiency and structural engineering solutions connected to heritage buildings.

The second section explores the renovation of a historic priest's summer house dating back to the 1900s. The structure is located in the rural area of Latvia. The second part specifically focuses on improving structural integrity and energy performance, this project includes designing a new timber truss element for the roof, preserving original rubble masonry walls, and reinforcing the foundation with reinforced concrete. Additionally, energy efficiency is assessed by post-renovation calculations in the software. The project combines detailed architectural plans with sustainable solutions to balance historical preservation with modern sustainability standards, resulting in a practical and visually appealing proposal for the owner.

Ultimately, this thesis delivers a set of comprehensive plans and a sustainable design approach, serving both as a resource for the building’s owner and as a guide for other specialists working on similar heritage renovation projects.

Keywords: Renovation, stone, masonry
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Contents

Figures

Table

Appendices

1. Introduction

Natural stone and rubble as a building material have been used throughout the centuries, starting with the grand monuments and structures that are standing to this day. The peak of the usage of stone masonry came in the 12^{th} to 20^{th} century, when it was used as a durable material for fortifications and defensive structures of any kind. Today, natural stone and rubble masonry structures are often neglected and lost due to various factors which include ineffective methods used for the renovation of the structure, lack of knowledge during the renovation works and plenty of other factors that are affecting the structure's overall health and safety.

Nowadays the question of renovation and reconstruction of various historical structures is a part of sustainable urban development, it is crucial to create a comprehensive guide that will provide all the important information for the assessment of natural stone and rubble masonry structures so that it will both contribute to the modern sustainability standards and the raising market of real estate.

Furthermore, the main idea of the thesis is to provide a general understanding of the behaviour and related problems of the historical stone and rubble masonry structures, which would cover a wide range of topics including history, architecture, geotechnical aspects, energy efficiency, structural engineering and law. To enhance the understanding of the various processes the case study was developed, for the structure that is located in a rural area of Latvia, it implements both the international building codes and local law, which could contribute to the wide range of international collaboration in the questions related to renovation, reconstruction and sharing the knowledge among other professionals.

2. Methodology

This research adopts a qualitative approach to analyse historical stone and rubble masonry structures, focusing on their behaviour, preservation challenges, and renovation methods. The study integrates historical analysis, structural assessment, and legal frameworks to provide a comprehensive understanding of these structures. To gather relevant data, multiple sources were examined, including technical standards, legal regulations, and historical case studies. For gathering all the required data for the case study, numerous software's were used, including RFEM, AutoCAD, Revit, QGIS and other engineering software.

To ensure the readability and overall understanding of the topic, it is recommended that the thesis is read fully without missing any chapter. For more advanced users who are already familiar with the topic, the scientific work can be assessed from any point of interest.

3. Historical background

In Latvia, stone masonry structures have been used since the end of the 12th century, during the peak of the Northern Crusades in Eastern Europe. This period in the History of Latvia is marked by the significant innovations coming from the Western world, including new construction techniques and materials introduced by the Crusaders. Natural stones were primarily used for the outer walls of fortifications and buildings, serving as a strong defence against the local tribes. Stones were favoured over wood due to their durability and availability, which provided a great defence against attacks and possible fire damage.

At the beginning of the 13th century, the first brick manufacturer was established in the territory of modern Latvia, marking a new era in castle construction. The bricks as a building material significantly changed the architectural practices in the area. The outer shells of castles were constructed using bricks, which were more advantageous than natural stone due to their rectangular shape and faster manufacturing process. This allowed people to build more durable and beautiful designs. Alternatively, if the focus is on the internal parts of the wall, then they were often made from construction leftovers and lower-quality materials that cannot be used for the outer layer of fortifications. This practice was both economical and efficient, maximising the use of available resources.(Figure 1.) (Fm.Pl, 2016)

Figure 1. Turaida castle brick masonry walls with rubble stone core (Latvia) (*Alamy*, n.d.)

Bricks as a construction material were mainly used in areas with abundant clay deposits, which were crucial for the brick manufacturing process. In the other regions where clay deposits were not available, natural stone continued to be the primary building material. In fact, in the areas with limited clay deposits, the bricks were still used to form aisles, corners and decorative elements, as the brick provided an even and smooth surface for the door and window installation while at the same time adding the aesthetic touch to the overall structure. Over time, advances in technology and tools made it possible to use carved stones instead of natural stones. Although carved stones are much easier to install, their usage was still limited, mainly due to the lack of skilled stonemasons. As a result, carved stones were primarily used in areas with higher population and educational levels.(Blumberga et al., 2016)

When the second half of the 16th century approached, the quality of outside stone wall structures began to decline. This decrease was due to the shortage of skilled stone cutters and masons, which led to less precise construction techniques. As a result, natural stones started to be used more widely again, reaching a peak at the end of the 19th century. During this period, some of the most beautiful natural stone buildings in the territory of modern Latvia were built. (Blumberga et al., 2016)

The 17th century, however, was marked by a significant decrease in construction quality. Wall structures built during this period usually have the lowest quality because all the available materials in the area were used without the proper consideration of mechanical and physical properties. This led to the fact that structures were often unstable and subjected to deterioration. (Blumberga et al., 2016)

Innovations in stone wall construction were introduced only in the first half of the 19th century. For economic reasons, new stone walls were built with an air gap and a layer of bricks, which significantly reduced heat loss through the walls. This method improved the thermal efficiency of the building and became a standard practice in construction. The air gap mainly acted as an insulation layer. (Figure 2.)(RIBuild Consortium, 2020)

Figure 2. In the 20th century, a newly introduced insulation layer (Latvia). (February 24, 2025)

By the late 19th and 20th centuries, the culmination of these techniques resulted in the construction of some of Latvia's most beautiful stone masonry buildings.

4. Architecture

4.1. Romanesque

Since the early 13th century, the modern territory of Latvia has gone through various architectural styles that changed the architectural appearance of the cities and the rural areas. The romanesque architectural style was one of the earliest known for its decorative elements, which were made primarily from natural stones and bricks. An exceptional sign of this style was its use of different compositions to represent the stories from the Bible. However, only a few buildings in Latvia still retain romanesque architectural details. One of the most notable examples is Riga Dome Cathedral (Rīgas Doms) (Figure 3.), one of the oldest churches in both Latvia and Riga, built between 1211 and 1270 (Krastiņš. J, n.d.)

Figure 3. Riga Dome Cathedral. (Riga Dome Cathedral, 2001)

4.2. Gothic

Gothic architecture was one of the architectural styles that appeared later in medieval Latvia. This style dramatically transformed the appearance of buildings by introducing lighter wall structures, larger windows, and brighter, more spacious interiors. At the beginning of the 15th century, architectural designs became more complex in both decorative and structural elements. Rib vaults were an essential feature of Gothic architecture, enabling the construction of taller structures with large windows. In the late gothic period, vaults were often designed to be as aesthetically impressive as possible, which was achievable only due to the development of local craftsmanship. During this time, various types of vault structures were introduced, especially shapes and forms that could enhance the increase of visual space (Krastiņš. J, n.d.).

4.3. Renaissance

In the 16th century, the renaissance brought new ideas to urban planning and construction techniques. These ideas were mainly focused on developing new fortification structures, as the old medieval walls were no longer capable of withstanding attacks from cannons and gunpowder weapons. One of the key concepts of this period was bringing back the cultural heritage of previous centuries while also introducing new decorative elements.

Only a few buildings in Riga were constructed in this architectural style. One of the most notable examples is St. Johns Evangelical Lutheran Church (Svētā Jāņa Evanģēliski luteriskā baznīca)(Figure 4.). While the original structure was built during the period when Gothic architecture was dominant, later additions incorporated Renaissance decorative elements, making it one of the most unique buildings in Latvia's architectural heritage.

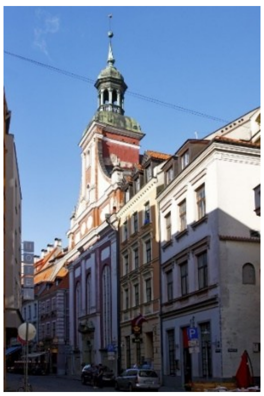
It is important to mention that the typical features of renaissance architecture include columns that follow Roman and Greek orders (doric, ionic, and corinthian), semi-circular arches, and vaults without ribs, reflecting the Renaissance ideas on returning to the classical ideals of the past (Krastiņš. J, n.d.)

Figure 4. Svētā Jāņa Evanģēliski luteriskā church. (RT Kompānija (n.d.)

4.4. Baroque

The Baroque is a highly decorative and ornate style, that appeared in the middle of the 17th century in central and eastern Europe. This style is characterized by its dramatic use of curved walls, columns, complex sculptures, and elaborate arches. Baroque architecture often features rich ornamentation, bold contrasts between light and shadow, and a sense of movement conveyed through dynamic forms. Domes, complex roof structures, and lavish interiors filled with frescoes and gilded details are also common. A great example of baroque architecture in Riga is the Reformation Church (Rīgas Reformātu baznīca)(Figure 5.) (Krastiņš. J, n.d.)

Figure 5. Rīgas reformātu baznīca. (Gulbis. A, 2014)



4.5. Classicism

In the 18th century, classicism became the dominant style in European architecture. It is mainly characterized by a focus on symmetry. Buildings were designed so that every element tried to point to a central, symmetrical axis. In addition to the general concept of symmetry, the style also features ornate decoration and various other decorative elements. Classicism is often associated with classical forms and was widely adopted by many religious groups as their preferred architectural style.

Figure 6. Riga town hall. (Rīga. Rātsnams, n.d.)

4.6. Rubble

As Latvians started to think about their own country, they needed to search for new artistic ways to express the local culture and utilize local materials for the façade finishing in the Latvian rural architecture of the 19th century. Thus, a completely new rubble style was developed.

In the 19th-century territory of modern Latvia, diverse architectural styles were found, but none of them could express the local culture and craftsmanship that had developed throughout the centuries. A key role is played by the application of different building materials, through which the external artistic image of the building is largely determined. Bricks, as a building material, became the symbol of the era. Brick was used not only in the local residential buildings, but also in many public buildings (schools, hospitals, courthouses, etc.), but also in structures such as religious buildings (churches) and industrially oriented facades. The material began to determine the artistic image of the building with different functional significance. (Zilgalvis, n.d.)

This encouraged the development of the so—called brick style: one of the formal types of eclectic style. Often, the red brick, and more rarely, the yellow brick combined with a rubble wall. Combining the two materials is a popular combination for satisfying the wall assembly's aesthetical appeal and mechanical properties. Overall, both the estates and the sacral architecture used bricks as the primary building material, primarily in the second half of the 19th century. (Zilgalvis, n.d.)

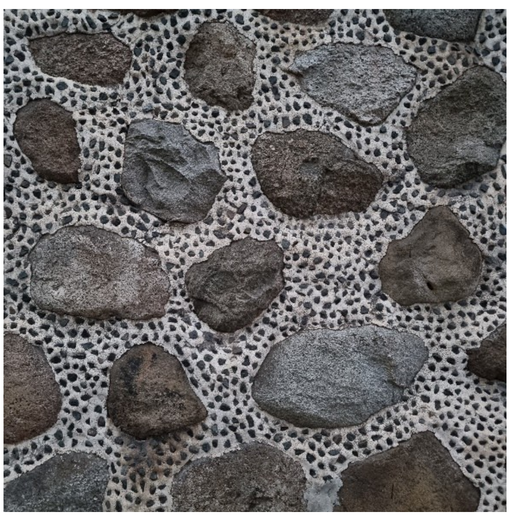
Bricks typically were used around the aisles in corners of buildings, eaves, and gable edges, as it was much easier to use bricks than carve boulders into different shapes, as the process of carving boulders required to have skilful stonemasons. At the same time, a formal stylistic trend was introduced, which we may call rubble style. Its peculiarity and character are mainly determined by the faced finishing technique- embedding granite chips in a mortar and combining these surfaces with plaster, red brick, and rubble walls. This combination can be divided into several groups: (Zilgalvis, n.d.)

Mortar, as a binding material for the joints, has lost its visual significance in some structures, as the boulders and rubble are so densely packed that the mortar is barely visible. While such examples are relatively rare, they can be found throughout Latvia in rural and sacral architecture. An imposing example is the Salnava Catholic church (Figure 7.), built in the second half of the 19th century. In some of the cases, a rare combination of brick and rubble was also used for construction. (Zilgalvis, n.d.)

Figure 7. Salnava Catholic Church. (August 23, 2024)

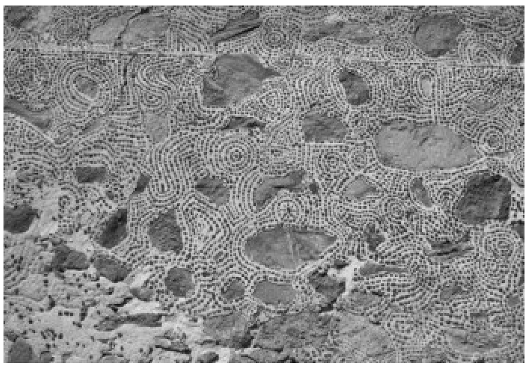
In the rubble style, there is a secondary type of structure where both the boulders and the mortar are equally essential and visually significant on the façade. Red brick is used in openings, cornices, and many other structurally necessary areas. his style is widespread across all regions of Latvia and can be seen in many buildings, including sacral structures, manor buildings, and other types of structures. A notable example in this category is the Varakļāni (Warkland) Lutheran Church, built in 1878 (Figure 8.).

Figure 8. Varakļāni (Warkland) Lutheran Church. (August 23, 2024)



The next group of rubble-style structures could be described as doodles, ornaments, and expressions of the stonemason's hobbies, worldviews, and momentary feelings. This is the most intriguing aspect of the rubble style in terms of creativity and unusual designs. Yet, it remains one of the most underappreciated rubble masonry styles in terms of its artistic value and historical significance. A unique example of this style can be found on the façade of one of the outbuildings at Nurmuiža Manor (Nurmhusen)(Figure 9.) from the second half of the 19th century. The rubble is arranged in parallel whorls with concentric circles embedded between them. Overall, the façade creates a mosaic effect that is highly expressive and unusual.

Figure 9. Nurmuiža Manor. (Zilgalvis, J. n.d.)



Another variation of the rubble style involves a façade where rubble imprints play the leading role. In this approach, the boulders appear to float in the sea of rubble, but unlike in other styles where the mortar is barely visible, here, the mortar-based surface has its own meaning and significance. Some of the facades were usually handmade, showcasing the stonemason's skill and attention to detail.

Figure 10. Variation of the rubble style in the city of Rujiena. (Latvia) (August 23, 2024)

4.7. Historicism

In 19th-century Europe, Historicism appeared as a prominent architectural style characterized by various historical styles. Architects were eager to blend contrasting architectural elements to create something new, reflecting the ideas of the century. Traditional masonry techniques, using materials like bricks and natural stones, were commonly applied in Historicism. However, by the late 19th century, metal structures began to be introduced as an innovative material for the load-bearing parts of the structures. (Historisms, Arhitektūrā, n.d.)

Starting in the 1840s, advances in glass manufacturing technology allowed the production of larger glass panels, leading to an increase in window sizes. The outer masonry walls typically served as a load-bearing element, while the facades were symmetrically decorated and ornamented. It also became popular to replicate the appearance of various natural materials, using cheaper, manufactured products, further expanding the decoration possibilities of the century. (Historisms, Arhitektūrā, n.d.)

Another architectural trend in the 19th century was the construction of residential castles- countryside manor palaces designed to remind medieval castles visually. These structures were primarily built at the request of wealthy German landowners. Various architectural styles were used, with Neo-Gothic and British Neo-Tudor being the most common. Unlike the actual medieval castles, these 19th-century residential castles used larger windows and more diverse interiors, as there was no longer a need to provide defensive capabilities of the structure. The 19th century was a century of unprecedented peace in the territory of modern Latvia, which, along with economic growth and technological advancements, allowed it to provide a level of comfort previously unknown. Towers were a popular feature, but their primary purpose was to offer scenic views rather than serve as defensive structures. (Žemaitis. A, 2016)

5. Ground investigation

5.1. Soil damage to structure

In the case of historical structures, various causes of damage and significant threats to the structure can be found; usually, they appear due to the following aspects:

- Subsidence of the ground substrate (Various cases can be mentioned, such as Leaning Tower in Pisa (Burland et al., 2003), the cathedral in Mexico City (Guerra, 1992), or the Church of St. John in Gdansk (Topolnicki M. 2001),
- Possible errors in the design process and during the implementation of construction solutions throughout the building process (Expansion of the Church of St. John in Gdansk that involved placing the structure on the foundation that was initially designed for smaller loads(Topolnicki M. 2001)
- Additional or unexpected loading (incl. fire, explosion, seismic vibrations, etc.)
- Reduction of the material strength due to the degradation of building materials caused by the various environmental conditions (impact of moisture, wind, frost, temperature change, pollution, etc.) (Raszczuk & Karolak, 2021)

All the previously mentioned aspects may affect the technical conditions of the structural elements and the entire structure itself. As a result, damage to the structural aspects of the historical building may lead to a dangerous threat to the structure's safety, which is even more concerning when the historical building is also essential as an architectural and historical object of the century. (Raszczuk & Karolak, 2021)

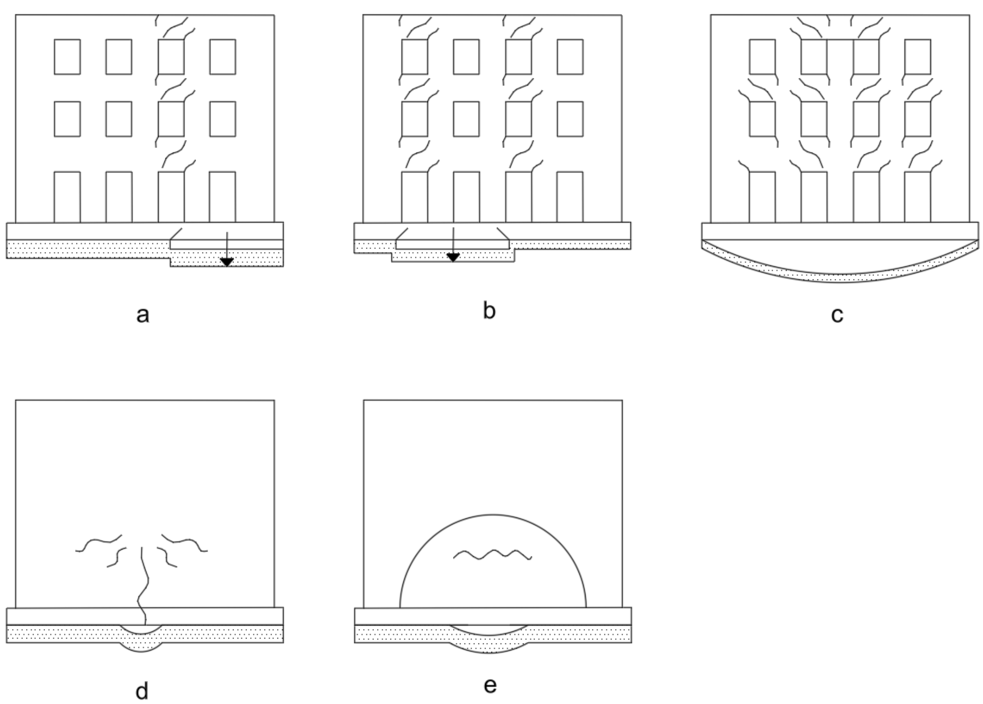
In the most common case, cracking of masonry structures is caused by the soil properties, more precisely, due to the unequal ground subsidence. (Pronozin et al., 2019) Unequal soil subsidence results in uneven settlement of the building foundation, which is the main reason why, in this process, cracks and mechanical defects of structural elements may appear. The most frequent causes of subsidence are: (Raszczuk & Karolak, 2021)

- Ground movement is caused by the soil movement underneath the structure, more precisely, under the building's foundation elements. The differing physical and mechanical properties of the soil cause the movement,
- The ground is insufficient for load-bearing purposes; for example, the soils are mainly found with a high percentage of clay and organic materials.
- Lack of soil compaction during the building process or errors in the foundation works.
- Varying groundwater level, causing changes in the volume and strength of the supporting ground substrate,

- Excess amount of the growing vegetation around the building that attracts the additional water to soil and causes natural soil degradation processes,
- Vibrations and ground movement related to nearby earthworks, traffic or natural seismic phenomena.

The problem of soil and foundation subsidence is a complex geotechnical engineering problem (Casalegno et al., 2013, pp. 187–207), especially in the case of historical heritage buildings, that were built tens or hundreds of years ago, often working with heterogenous soils, that possess poor load-bearing capabilities and are poorly compacted. As a result, due to the unequal soil subsidence, tensile stresses are created in the building structure, causing cracking, which leads to the degradation of the structure as a whole (Tokimatsu et al., 1996a, pp. 219–234). Usually, the visual inspections and checking of cracking patterns should provide a base for the assessment of structural safety (Alessandri et al., 2014, pp. 111–129) and for planning reinforcement repairs. Many authors conclude that analysis of cracking patterns and deformations may be a suitable method for the initial assessment of possible threats to structural safety. (Raszczuk & Karolak, 2021) (Figure 11.)

Figure 11. Typical wall cracking arising from irregular subsidence of the building foundation.



Note: **a**. subsidence of the corner of a building. **b**. small-scale subsidence of the central part of the foundation. **c**. large-scale subsidence of the central part of the foundation. **d**. localised subsidence of a foundation in the case of a weak foundation. **e**. localised subsidence of foundation in the case of a strong foundation

5.2. Building foundation investigation and diagnostic

Geometrical survey and recently developed H-BIM modelling as a basis for the diagnostic of superstructure and the foundation shows the rapid development of new diagnostic technologies for historical buildings; it means that they can be applied during the building's maintenance and use, as well as during the renovation works. Modern building diagnostics and modelling methods allow more precise identification and restoration of the internal structural elements or individual elements that provide aesthetics to the structure. These types of investigations are also important in the case when the documentation of the existing building is missing. Based on diagnostic test results, the appropriate conservation and strengthening plan can be carried out, also including possible preservation techniques. (Roca, 2011, pp. 151)

Table 1. Diagnostic methods

Methods:	**Method characteristic**	**Advantages/disadvantages**
Visual assessment	Assessment of external surfaces and cracking patterns	+Low cost +Does not require specialised equipment -Results are qualitative
Geotechnical investigation	Geological cores	+Assessment of differentiated subsidence rates of building foundation -High costs, especially with respect to deep boreholes
Geometrical survey including H-BIM	Interactive model built by the use of the historical architectural and construction documentation, photogrammetric techniques, laser scanning, and other data obtained from physical analysis of the building	+The possibility to understand, analyse, document, advertise and virtually reconstruct the whole structure +Enables also energy simulations, time, and cost calculations, and other functions that may improve the way to manage the maintenance and restoration process -Complex and time-consuming method

Nowadays Non-destructive testing methods are the most important among all the available diagnostic methods for historic buildings (Pérez-Gracia et al., 2013, pp. 40-47) because the building's historical details can be maximally preserved without losing the original look of the building, that is the most significant part in the process of conservation. The most common investigation methods include; visual assessment (Binda et al., 2000, pp. 199-233), geotechnical site investigation (Guerra, 1992, pp. 28-35; Topolnicki M., 2001), geometrical survey and H-BIM (Calì et al., 2020, pp. 421- 434), core sampling (Pelà et al., 2015), ultrasound testing (Carpinteri & Lacidogna, 2006a, pp.161-167), geo-radar (Ranalli et al., 2004, pp. 91-99), operational modal analysis (OMA)[(Aras et al., 2011, pp. 81-91)]. One of the most important phases in building diagnostics is to choose the appropriate method of investigation for the specific situation, it is required to consider all the pros and cons that the method can provide.

In many cases, visual assessment appears to be one of the easiest methods to apply, but it is generally recommended to apply several methods, that will provide enough information for the verification of results. Modern monitoring and advanced computer analysis methods combined with the visual assessment method are used in many important conservation projects, including St. Mark's Basilica in Venice, city cathedrals in Florence, Padua and Mexico, and towers (Carpinteri & Lacidogna, 2006b, pp. 1681-1690). Proper assessment and identification of the problem allow for the appropriate technical solution to be developed, because currently, the problem of structural damage identification is often too general. To increase testing quality, researchers and designers use integrated methods to precisely assess possible safety concerns, especially when it comes to historically significant buildings (Bosiljkov et al., 2010, pp. 239-249; Tokimatsu et al., 1996b, pp. 219-234). For economical purposes it is recommended to make visual assessments first, in the case of structural damage (cracks), the person who investigates the site may require carrying out geotechnical tests to verify that the condition of the building foundations is sufficient.

The H-BIM (Heritage BIM) is an innovative concept, which has been increasingly used by construction engineers and researchers in recent years. The method consists of creating a detailed three-dimensional model of existing historical buildings or historical objects. The model can be used for various purposes also including architectural and structural analysis. H-BIM is a complex method that allows the creation and storage of detailed building and structure models with documented history. In the first step, the H-BIM library is created using technologies: laser scanning, photogrammetry, and other data obtained from the physical analysis of the structure and historical archives. (Dore et al., 2015, pp. 351-357) Particularly the use of historical data introduced the possibility to include the details that are hidden under the surface, more precisely to determine the material, structural elements, historical and cultural significance, including the renovation status and maintenance program (Quattrini et al., 2017, pp. 129-139) Furthermore, it also provides the opportunity to enter the temporal data that can represent the possible incidents occurring during the life span of the building. It is worth mentioning that H-BIM provides not only the complex analysis of the structure but also helps to understand issues related to materials and construction techniques, as well as helping in various other things such as management, renovation and reconstruction processes of heritage buildings including those that no longer exist or have a lack of technical documentation.

5.3. Soil investigation methods

Various methods are available for the geotechnical investigation, however, in this section, the most common methods such as borehole drilling, test pit excavation, cone penetration testing and dynamic penetration testing methods will be summarized.

5.3.1. Borehole drilling

In Eastern Europe and Latvia, boreholes are typically drilled with hollow-stem augers, even though other forms of drilling such as wash boring, solid-stem augers, air rotary and other drilling techniques can be used. In the case of small properties and family houses, manual drilling techniques are often used, but it is worth mentioning that it may result in a less precise investigation outcome. Drilling technique and/or method should be chosen depending on the task and the complexity of the building's structural systems.

In cases when the soil consists of stones and boulders, it is generally recommended to carry out drilling using rotary diamond (core) drilling equipment, which provides sufficient hardness of the drilling tip to penetrate the stones and boulders. (ĢeO Eksperts - Geotechnical Surveys, n.d.)

A wide variety of onsite testing techniques may be carried out in the boreholes, including the following techniques:

- Standard Penetration Testing (SPT).
- ‘Quick’ in situ vane testing.
- Strain-rate controlled in situ vane testing (e.g., Nilcon vane testing).
- Piezocone (i.e., CPT) testing.

In the case of deeper boreholes, it is recommended to use piezometers. Typically, they consist of 19-millimetre to 50-millimetre nominal-size HDPE or PVC tubing with an opening over the bottom part. The device itself can be installed in boreholes to enable monitoring of the groundwater levels (City of Ottawa, n.d.).

5.3.2. Test pit excavation

Generally, test pit excavations are done using hydraulic excavators. Rubber-tired ‘backhoes’ or track-mounted excavators can be commonly found. During the pit excavation process, geotechnical samples can be obtained from the sides of the test pit (only if the geotechnical test pit is shallow enough to collect them safely) or from the excavated spoils. A limited amount of testing can be carried out in the test pits; the undrained shear strength of clayey soils can be measured using a ‘field’ vane. However, the results of such testing may not be as accurate as the results of onsite vane testing in boreholes; in such a case, further investigation and testing may be required (City of Ottawa, n.d.).

After the investigation works, the geotechnical engineer, property owner and project developer should consider that the test pit leaves a zone with disturbed soil, that is not suitable for the support of the superstructure. To avoid such a situation, in which the disturbed soil may lead to partial or full collapse of the superstructure, it can be recommended to increase the load-bearing capabilities of the soil by using engineered backfill materials, that are compacted according to the design requirements. Test pits are not always suitable for all situations and must be considered and identified in the early stages of geotechnical design. (City of Ottawa, n.d.).

In the case of high groundwater level, the test pits may not need to be stabilized, unless the test pit is left open for an extended period, which can cause the excavation pit walls to partially collapse. In this case, it must be safely barricaded and treated as a construction excavation. The depth of test pits is more limited compared to the drilled boreholes. The maximum depth of the investigation varies depending on the machinery used during the excavation. For the rubber-tired ‘backhoe’, the excavation depth is about 4 to 5 metres. Test pits excavated using a larger track-mounted excavator could reach around 8 to 10 meters.

The excavation of test pits suits best sites with shallow bedrock, where bedrock surface excavation can be economically reasonable at many locations. Finland and Estonia are examples of such soils.

In general, investigations using the test pits are acceptable only if they satisfy the following conditions:

- The weight of the structure is relatively light (Single-family houses or wood frame housing), so that the load of the structure does not affect the deeper soil.
- The surface of the rock is shallow, and the main goal is to investigate and profile the rock surface.
- The soil overburden is not highly compressible (in comparison to the loads from light structures), e.g., glacial till, stiff clays, etc. (City of Ottawa, n.d.)

Investigation by the test pits is not used in the case when the site is underlain by soft and sensitive marine clay (That has a shear strength of less than 25 kilopascals), since the measurements provided in the “inspections” may be inaccurate. The same as it wouldn’t be recommended to use the investigation with test pits in the areas where sandy soils are below the groundwater level (so-called ‘running’ sand), as the conditions don’t allow collection of the samples or observation of the excavated soil, due to the rapid inflow of groundwater.

5.3.3. Cone penetration testing

Cone penetration testing (CPT) is a much less common investigation method than the methods mentioned previously such as borehole drilling or test pit excavation. The CPT involves the pushing of an instrumented probe into the ground (vertically) at an essentially constant rate of penetration (City of Ottawa, n.d.). The gadgets on the probe typically measure both the force on the conical ‘tip’ of the probe, required to advance it into the ground, as well as the friction along the ‘sleeve’ of the probe. (City of Ottawa, n.d.). The technique can be used to measure numerous engineering properties of the soil, the same as evaluating the type of the soil. As an advantage, CPT testing provides continuous measurements of the soil properties and possible conditions. The probe can be upgraded for a variety of different tasks; for example, the gadget that measures the water pressure in the ground can be added; in this case, the probe will be called ‘piezocone’. The probe can also be upgraded with a geophone to allow for shear wave velocity testing, which allows for evaluation of the seismic design Site Class; this testing is known as seismic cone penetration testing (SCPT). (Cone Penetration Testing (CPT) | U.S. Geological Survey, 2003)

5.3.4. Dynamic penetration (DPSH)

Dynamic penetration super heavy (DPSH) is performed mainly in the case where the result of the testing cannot be achieved by using cone penetration, or the anchoring of the cone method, mainly due to the dense and hard soils. With the help of dynamic penetration, it is possible to specify the existing soil layer borderlines the same as determining the parameters and mechanical properties of the existing soils. The main principle of the work of DPSH is that the cone probe is continuously driven into the ground with a freely falling hammer (with a mass of 63.5kg dropped from the height of 0.75m). In addition, the number of hammer blows needed to push the probe is fixed in the range of 20cm recorded in the number of pushes. Then, the probing operator registers the information in the computer program, which later on provides visual information about the probing. The probing is done when the required depth is reached. (ĢeO Eksperts - Geotechnical Surveys, n.d.)

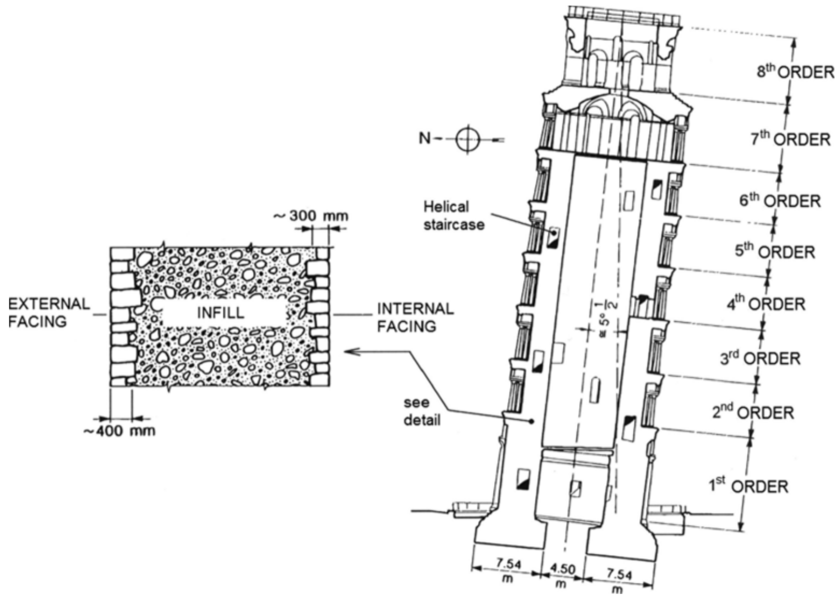
5.4. Foundation types

Many medieval-period buildings were founded on shallow foundations, as the more progressive methods were not developed yet. In the early medieval era, local materials such as timber, stones, rubbles, and later bricks were used, in combination with the various mortars that were used for the joints. In the pre-Romanesque period (5th to 10th centuries), the standard method of constructing foundations was to improve the bearing capacity of the soil by dropping rubble or debris into the excavated pit. The width of the foundation was usually equal to or slightly wider than the underground part of the building. The overall dimensions of the foundation and structure were determined more by the available space than by the active loads or soil-bearing capacity. A low-quality mortar was poured into the excavation pit to bond the stone rubble . (Przewłócki et al., 2005, pp. 363–372)

Romanesque foundations (11th to 13th centuries) were made in a way that the large boulders were located on the outer side as the main load-bearing elements; at the same time, the debris and smaller stones were used as a filling material in between the larger boulders. As a connection material, lime or clay-lime mortar was poured on the foundation to fill the crack and joints (Przewłócki et al., 2005, pp. 363–372). Considering that the mortar is of low quality, it is obvious that the foundation could not appropriately interact with the structure.

The Tower of Pisa is an example of the most famous Romanesque building founded on soft soil (Jamiołkowski,2001)(Figure 12.). Its foundation was constructed by limestone faced with marble, resting on a layer of cemented debris, with an underlying stone layer (Przewłócki et al., 2005, pp. 363–372)

Figure 12. Structure of the Tower of Pisa. (Jamiołkowski,2001)



Gothic foundations (12th to 16th centuries) were generally more precisely and adequately made, using good quality mortar and better-cut blocks; also, later, regular-sized bricks were introduced and became a basic construction practice. Often, the external layer of the foundation was lined with ashlar blocks. Many investigations of the historical building foundations dated to the Gothic period show that the building's location was usually (but not always) selected according to the load-bearing possibilities, and the soil conditions were often checked.

Figure 13. An example of a Gothic foundation in Poland (Saint Mary's Basilica). (Firlet, Kadłuczka, & Pianowski, 2011)

In the example above, it is possible to see the widening type of the stone foundation; moreover, soil conditions and the overall structure of the foundation show that the interactions between a structure and soil are considered. Many historical buildings had no foundation in the modern sense because the foundation often is the extension of basement walls, without any or with little widening. Foundations without widening in the weak soils may cause the possible over-loadings of the soil, so cracks or defects can appear.

Figure 14. Malbork Castle – the damage caused by the soil over-loading. (Mierzwiński, 1994)

Gothic buildings were sometimes supported on shafts and arches. The method was implemented only in cases where small sources of groundwater or archaeological remains of the foundation were found during the excavation. In this technique, regularly spaced deep excavations in the form of columns were made and filled with brick or stone masonry. (Strzelecka, 1958; Małachowicz, 1994)

Figure 15. Historical masonry foundation technology and connection. (**a**) The figure shows the buttress, upper offset and arch connection. (**b**) The lower part of the buttress. Visible foundation technology and profiled offset made of brick fittings.(Kmiecik & Szwed, 2022)

(a)

(b)

Some Gothic foundations were sealed off above groundwater level, often with clay, which provided all the necessary properties to prevent the capillary rise of moisture to the brickworks of facades and walls above. In Venice, Istrian marble (an impermeable calcareous rock) was commonly used as the sealing agent during this period. (Przewłócki et al., 2005, pp. 363–372)

The Renaissance (13th to 16th centuries) and Baroque era (16th to 18th centuries), witnessed the application of similar foundation methods to those applied in the previous period, with some examples of regression in the construction techniques. Various innovations in foundation engineering were introduced in the Neoclassical period (18th to 20th centuries). It was strongly connected to the technological advances and application of new materials, such as hydraulic mortar -called Roman concrete- and steel. (Przewłócki et al., 2005, pp. 363–372)

Most of the historical cities are situated along the rivers, where high groundwater levels are present. The methods and technological development of the medieval period did not allow excavations below the groundwater level to construct foundations. Due to the lack of mechanical tools and machines, it was impossible to replace the soft or organic soils with sand or gravel, which could improve the ground's load-bearing properties. The possible methods of dewatering of pits were also unknown. For soil improvement short and closely spaced timber piles (like a brush) were often driven into the weak soils to compact them and as much as possible transfer the load to the deeper soil layers, to improve the bearing capacity of the foundation. Such a technique for improving the foundation became commonly used in the Gothic period, although similar practices were used in ancient times. If during the excavation weak and compressible soils were found at the depth of excavation, the builders tried to improve the load-bearing properties of the soil, by often pushing stones of different sizes into the ground, as after the compacting they could create something like a layer of flat claystone or cobblestones. (Przewłócki et al., 2005, pp. 363–372)

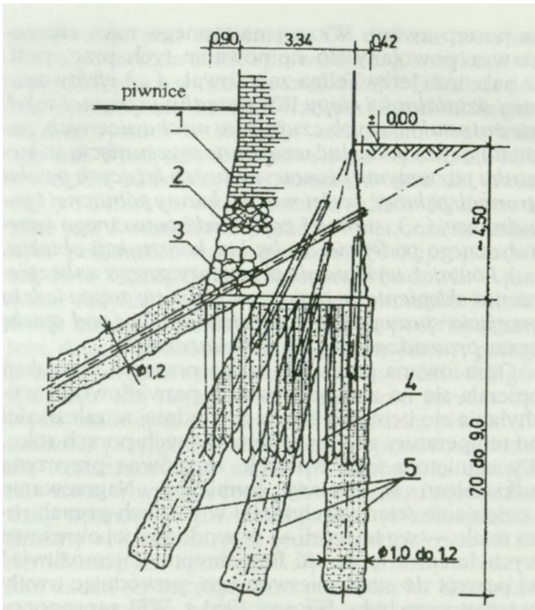
Usually, buildings located on soft soils had foundations in the form of rectangular platforms constructed on top of wooden piles, providing a bed for the rest of the structure. Local stones and bricks were used as the main material for the foundation caps. Sometimes, gravel, rocks, and weak mortar were placed inside the wooden moulds, which were pushed into the loose soil to increase the load-bearing capacity.

Timber piles were always placed below the lowest expected groundwater level in the area because builders with longer experience had noticed that timber does not decay in water. Piles were made from the local wood types (oak, pine, alder, beech, etc.), with almost no treatment, except for debranching. The piles were hammered into the ground with the thinner top-end downwards. Timber rafts were situated horizontally beneath layers of bricks or stones. Different types of timber construction have been used over the centuries (Borrmann, 1992; Ladjarevic & Goldscheider, 1997, pp. 215-223). The simplest method was to hammer down the sharpened short piles (1.5m - 3m long) driven close to one another into the soil. This type of foundation was used until the 18th century. Another method commonly used in the Middle Ages consisted of timber piles put lengthwise and crosswise under the foundation base. Later, such a method was improved by the use of both the piles and timber elements that were put lengthwise and crosswise, in addition to that newly developed iron connectors were used.

In the 16th century, technological advancements allowed the use of longer and thicker piles for the foundation, resulting in more complex structural systems that could transfer the load to the deeper layers of the hard soils below.

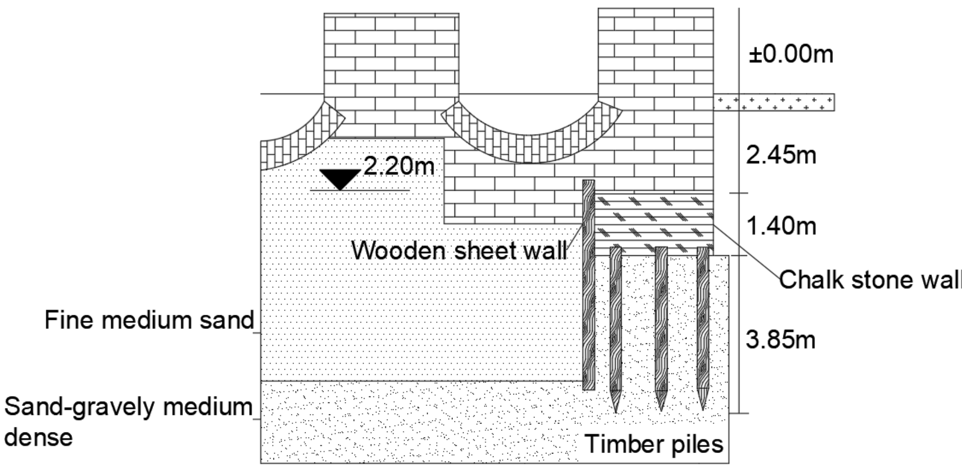
The idea, illustrated in Figure 16, represents a Gothic foundation type situated on soft subsoil(Przewłócki et al., 2005, pp. 363–372; Dembicki et al., 1995, pp. 67-81). It exemplifies a potential reinforcement concept proposed for Malbork Castle in Poland. The existing structure features a nearly 4-meter-wide brick wall founded on a stone layer, supported by timber piles, which were historically used to increase the soil's load-bearing capacity. In the proposed reinforcement plan, cement piles will be introduced to further increase the foundation's load-bearing capacity and stabilize the building in the horizontal direction.

Figure 16. A conception of reinforcing the foundations proposed by the GEOSERVICE company. (Mierzwiński, 1994)



The foundation of the Reichstag building is an example of good modern foundation design and engineering. (Quick et al., 1997, pp. 773–781)(Figure 17.). The connection between the adjacent deep, heavy-loaded sections of the building and the shallower, less-loaded foundation was done by the use of transitioning foundation thicknesses in the various parts the same as the general arrangement of inverted arches. That resulted in significant material savings and more regular pressure transfer across the foundation. The foundation scheme provides the information that the designers back then had a great knowledge of structural and soil mechanics. It is worth mentioning that, before the foundation works of the Reichstag the geotechnical survey was performed, several boreholes and excavations were done to ensure that soil conditions were appropriate for the foundation. In addition to that, some of the foundation elements were dug under the protection of wooden sheet walls.

Figure 17. foundations with inverted arches and timber piles of Reichstag building in Berlin (19th century)



5.5. Historical building preservation methods

Numerous historical buildings are still being affected by the various changes beneath the structure. The performance of building foundations is generally influenced by soil properties, foundation types, and the interaction between the soil and the structure. One of the main challenges faced in historical foundations is the poor characteristics of the subsoil, low quality of the mortar in the stone and brick masonry structures and in its foundations, and/ or the rotting of the supporting wooden structures. Many foundations have suffered significant damage due to factors such as rain infiltrations and changes in the groundwater, subsidence caused by the decrease of natural water level because of pumping from deep wells, dissolving of the various minerals, mining cavities, soil movement of nearby slopes, failure of the local sewage systems or careless occupation. (Przewłócki et al., 2005, pp. 363–372)

In many cases, the load distribution on the bearing elements was significantly modified by the numerous works done after the construction, such as restoration works or the construction of nearby buildings. Extensive destruction of the historical buildings was caused by industrial pollution, or by the deterioration of the structural elements exposed to long-lasting moisture. Historical buildings such as churches often were constructed on old foundations that were previously used for other types of buildings, as a result, it led to the large difference in the building settlement, caused by the nonuniform contact pressure. (Przewłócki et al., 2005, pp. 363–372)

For the timber piles, the aspect of the groundwater level can be crucial, as it has a significant influence on the durability of the timber piles. As usual, piles are exposed to the process of rotting when the groundwater level lowers. Even a temporary change in the surrounding environment of the piles may cause significant damage and decay to the piles. However, the effect of the decay development is strongly dependent on the type of soil that surrounds the timber foundation. Sandy soils have the perfect conditions to cause the timber foundation to decay, whereas soil such as clay has natural properties such as the capillary lifting of water and superficial activity of the clay, which lowers the possibility of the decaying process. In the peat, the development of decay is impossible, because it contains natural wood preservatives, which can hold up moisture.(Przewłócki et al., 2005, pp. 363–372)

To protect historical buildings, an exact picture of the structural conditions of the existing structure is required, as well as more precise details of the foundation type and the subsoil layers. To define the current state of the foundation, it is recommended to analyse the historical background of the structure in connection with the additional loading processes it had undergone. Unfortunately, it is difficult for various reasons; old documents are rarely available or sometimes even don’t exist anymore, past measurements are frequently unreliable, and past restoration processes have significantly damaged the façade of the building causing the covering of the original structure. (Przewłócki et al., 2005, pp. 363–372)

When dealing with historical structures it is important to have data that precisely represents the dimensions of the existing foundation and the underlying subsoil material properties. In the case of the timber piles, the knowledge about the preservation state of the timber and the dimensions of the piling should be checked. Previous archaeological studies may play an essential role in understanding the behaviour of the foundation. The type, dimensions and depth of the foundation should be determined by the subsurface exploration, which in general includes all the various methods mentioned previously. (Przewłócki et al., 2005, pp. 363–372)

Various Geotechnical problems play an essential role in the behaviour of historical buildings, and they influence the possible future preservation works. The geotechnical engineer’s role is to assess the behaviour of the foundation and the underlying subsoils, in addition to the proper use of modern foundation engineering techniques. The foundation of the historical building must fulfil sufficient load-bearing capacities while also not losing the serviceability of the building foundation. If the bearing capacity of the historical building foundation is insufficient, a suitable strengthening design of the foundation is essential. A small amount of the additional settlement can be accepted, and the action should be taken only in case if it endangers the serviceability of the structure.(Przewłócki et al., 2005, pp. 363–372)

To preserve a historical building subjected to the harmful influence of geotechnical factors, the foundations and/or the underlying subsoil must generally be improved. According to previously mentioned information (Calabresi & D'Agostino, 1997, pp. 75-83)., and some modifications by the present authors, the appropriate methods of preservation can be divided into the following groups(Przewłócki et al., 2005, pp. 363–372):

a) Replacing the foundation of the whole historical building.
b) Widening of the existing load-transfer surfaces, lowering the foundation level (make it deeper), and strengthening of the existing foundation.
c) Involvement of different structural elements such as piles, micro piles and tendons, and the possible addition of the underpinning of substituting foundation structures.
d) Improving the subsoil properties by chemical or types of cement grouting, or electro-osmosis
e) Modifying the effective stress field in the soil by drainage or consolidation.
f) Restoring or preserving the original state of the soil and structure (soil water content, masonry moisture conditions, anti-bacterial protection of wooden structure).

The main issue in the preservation of historical foundations is to make the best choice out of many methods available so that it brings satisfactory results and doesn't damage the building. The preferred method should do the least modifications to the building and the interacting soil, although it would be generally recommended to not use the cheapest method possible. If preservation aims to save the historical background of the used materials, any redundant change of its properties should be avoided. In any of the given cases, the various methods should be analysed, and the cons and pros of each method should be considered in relation to the particular foundation.

It should be mentioned that there is no universal method for all types of foundations. Even the most modern, effective, and reliable techniques, such as micro piles or jet grouting, can in some cases cause damage and deterioration of the foundation and lead to the partial or full collapse of the building. In some cases, it is recommended to do nothing other than to try to improve the structure in an unsuitable way.

5.5.1. Replacing the whole historical structure or its foundation

In occasional cases, the material history and value of historical buildings determine the method of intervention. For example, because of the dam construction, the only way how to preserve the Philae Temple complex (Egypt) was to dismantle the whole structure and rebuild it elsewhere, as the cost of losing part of the original structure and foundation was too high. Despite the high financial cost and enormous planning, the historical worth of this great heritage of Egyptian civilisation was more than justified in such a procedure. Another way how to preserve the historical value of the structure may be their relocation without dismantling. The first such case took place in the 15th century when the 25m high masonry bell tower of the Saint Maria Church in Bologna was displaced almost 25m.

Nowadays, available records describe numerous ‘engineering’ attempts to improve the foundations of historical buildings to prevent excessive settlements. Some of these attempts were very complex and required a lot of hard work and engineering work. An example is the total replacement of the foundation underlying one of the pillars of St John’s Church in Gdansk (Poland), executed a few centuries ago (Bukowski, 1948).

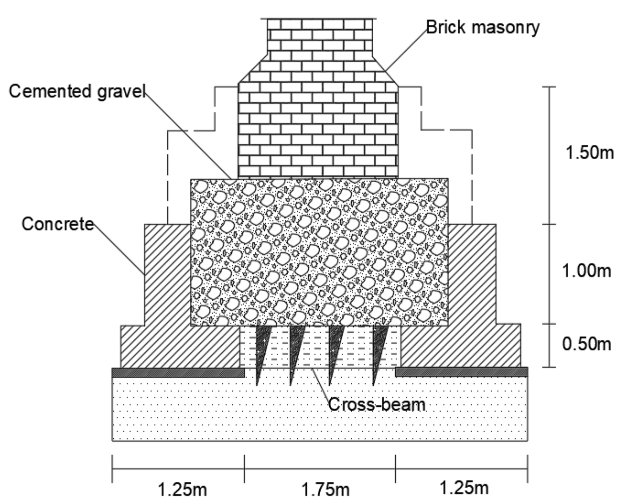
5.5.2. Strengthening, deepening, or widening the foundation

Some historical building foundations can be effectively reconstructed by grouting, using a cement-sand-clay-water mixture. Cement grouting can, in some cases, strengthen the Factor of Safety (FS) soil under and around the foundation. Traditional improvement methods can also be used, including increasing the foundation depth or enlarging the base and underpinning so that the load is distributed over a bigger area.

Reconstruction of the foundation by deepening is widely used when sumps, filled-up wells or subsoil-loosening zones exist under the building or in its vicinity. (Przewłócki et al., 2005, pp. 363–372)

Enlarging the base of the foundation while at the same time maintaining the original depth of the foundation should be used only in cases when small damage to the building has occurred. It allows the enlarged foundation to remain on the weak subsoil, as a result, the unit pressure on the ground can be significantly reduced. The concept of widening the foundation to increase the effective bearing area was used in the Cathedral of St Martin in Landshut (Germany)(Hilmer, 1981, pp. 111-116)(Figure 18.). The advantage of this technique is that the concrete shell around the foundation embraces and confines the old foundation.

Figure 18. Underpinning of the Cathedral of St Martin in Landshud with a new concrete foundation



The bell tower of St Mark’s in Venice is a spectacular example of foundation reconstruction. (Colombo & Colleselli, 1997,pp. 435-445)(Figure 19.).The original tower collapsed in 1902 due to poor load-bearing characteristics and various mistakes during the renovation. The tower was rebuilt in the same place as the foundation, which was found to be in good condition. It was retained and enlarged by a greater outer wall, which increased the load-bearing capacity of the foundation, which mainly consisted of over 3000 larch piles.

Figure 19. Foundation of the St Mark’s bell tower after the collapse. (Umberto Sartori, 2012)

5.5.3. Underpinning with piles

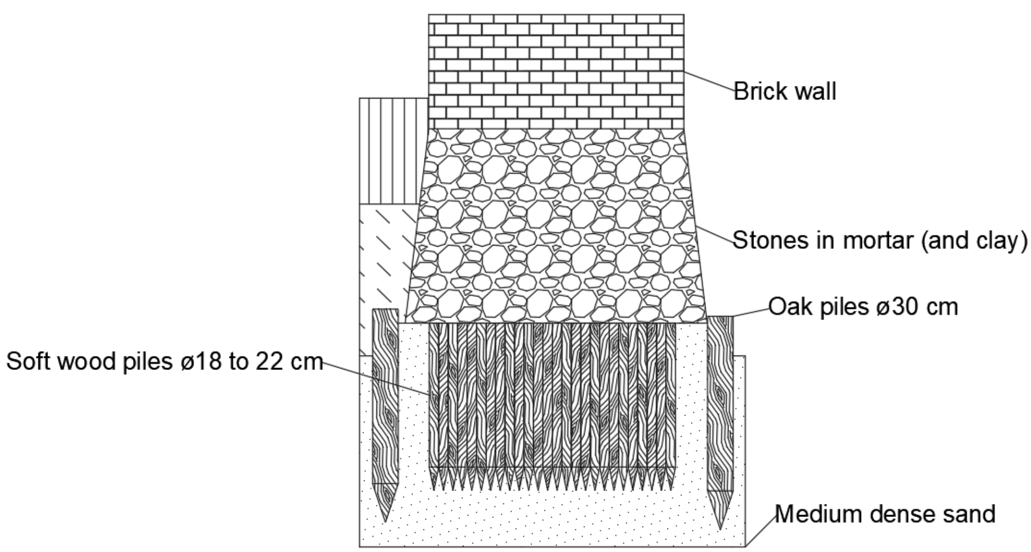
In the method of underpinning with the piles, vertical shafts are bored to the top of the rigid soil, which is usually called firm strata, and has a load-bearing capacity sufficient to handle the loads coming from the superstructure. Later the bore holes are filled with the concrete that is reinforced with the steel rebars. To transfer the loads from the structure to the newly made piles, the pile caps are constructed and attached to the old foundation with prestressing tendons. However, the quality of such a system is dependent mainly on the condition of the historical foundation. It is worth mentioning that this method may cause undesirable ‘hard spots’ beneath the strengthened foundation, which could lead to the redistribution of the stresses in the structure if additional ground settlement occurs.

Nowadays, micro piles are the most used method for underpinning. The technique can be applied to almost any structure, including the foundation of monuments and historical buildings. The method is often chosen due to various factors, such as the minor risk of uneven settlement during the installation and the relatively low cost of installation. The application of micro piles allows the operation of a piling rig in limited spaces (such as a cellar), which results in considerably reduced soil disturbance. The technique, in general, does not disturb the existing old foundation and is relatively free from vibration. The method's main disadvantage is that it relies mainly on the old foundation to transfer the load to the new pile caps.

In the design of the structure, it should be considered that the capacity of a single pile is low, so it may be necessary to install many piles, potentially weakening the historical foundation. Additionally, if geotechnical investigations reveal that load-bearing soils are deeper than expected, extending the pile length could lead to increased reconstruction costs of the structure. It should be noted that the underpinning with micro piles destroys the soil underlying the foundation, which reduces the possibility of the later analysis of the sedimentary layers of historical construction.

The piles can be drilled through the old foundation to connect the micro piles with the superstructure. In the case when the drilling through the foundation might significantly damage the structure, the piles can be drilled either on one side or at both sides close to the old foundation and then connected to the wall with anchors, crossbeams, or concrete slabs. The choice of the appropriate connection depends on the dimension of the supported wall or column, the bearing capacity of the soil layer and the overall budget of the construction works. An example of underpinning using micropiles for the castle of Malbork (Poland) is presented in Figure 20. (Dembicki et al., 1995).

Figure 20. Foundation of the western wall of Great Mess Hall (Malbork castle)



5.5.4. Underpinning with jet grouting

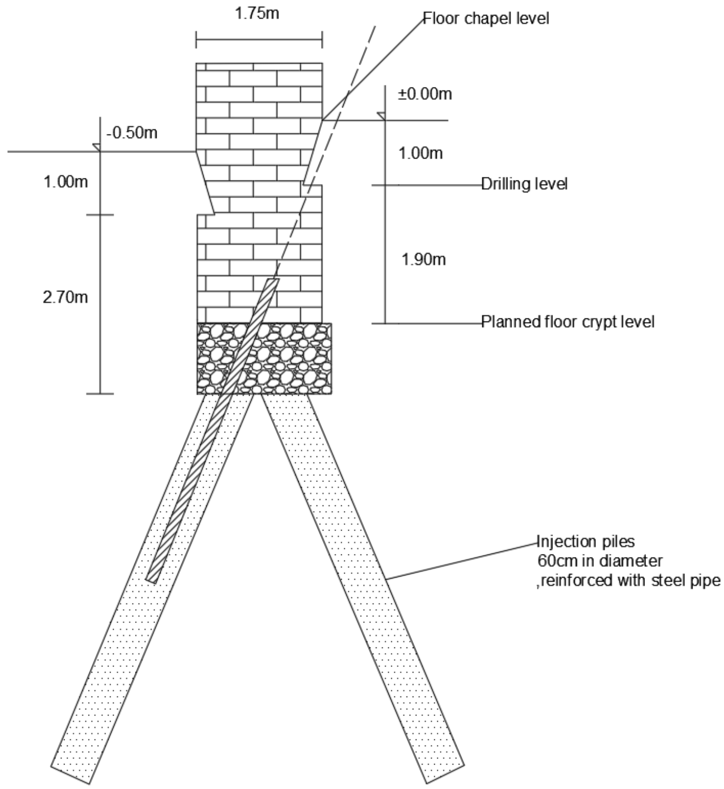
The Soilcrete jet grouting process, introduced by Keller Company, is a specialised soil stabilization technique that combines soil and concrete (or, more accurately, a cement suspension). The process involves using high-pressure jets, ranging from 20 to 40 MPa, with nozzle exit velocities of around 100 m/s, to cut into and erode the surrounding soil. (Przewłócki et al., 2005, pp. 363–372): The eroded soil is rearranged, mixed with cement suspension and partly flushed out to the top of the borehole through the annular space between the drilling jet grouting rod and the borehole. (Przewłócki et al., 2005, pp. 363–372): Different geometrical shapes of soilcrete elements can be created, such as lamellae, panels, and quarter/half/complete columns.

The distanced soil erosion caused by the jet varies according to the type of soil and the soilcrete. The distance may reach from 0.3m up to 2.5m in diameter. In practice, various jet grouting methods can provide the opportunity to create even soilcrete columns up to 3.5m in diameter and unconfined strength from 1 MPa for organic soils and up to 20MPa for sandy-gravely soils.

Using the jet grouting method requires both experience and flexibility in the design and careful execution of all the necessary steps during the main construction works. Usually, the beginning of the soilcrete works is the most challenging stage of the operation because the process itself may cause unacceptable settlement of the soil beneath the structure or sometimes even the local failure of the existing foundation. It is crucial to conduct a proper investigation and survey of the subsoil and inventory the existing foundations before the jet grouting process begins. Furthermore, it is required that the jet grouting process is precisely checked and monitored during the construction period. A few examples of the application of jet grouting methods are presented in the following paragraph:

The jet grouting method can be highly effective in stabilising foundations, but as it was mentioned, it requires planning, surveying, and monitoring due to the potential risks involved. The example from the Odra River flood in 1997 highlights a typical situation where jet grouting is used to stabilize a structure impacted by soil instability and degradation. Organic soils such as peat and mud reached a depth of 3m. The observed damage was associated with a significant decrease in the soil parameters and the decay of timber and organic soil compounds that were in the moisture for an extended amount of time. Initially, the castle was founded on short wooden piles located under the stone foundation. The two rows of the underpinning created using the jet grouting method are shown in the following figure (Noga & Kos´cik, 2001).

Figure 21. Jet grouting method in chapel of castle in Raciborz



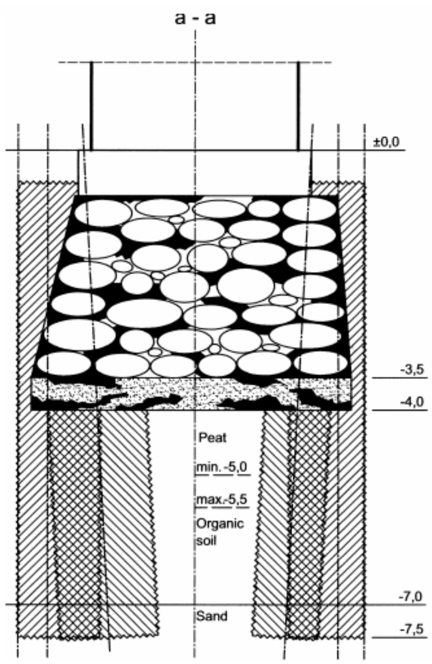
The first step involved drilling small rods through the brick and stone masonry and grouting them to strengthen the structure. Jet grouting piles, each 60 cm in diameter, reinforced with steel pipes, were installed below the existing foundation. These piles were spaced 1.0 to 1.5 meters apart along the chapel walls. It's worth noting that the original stone foundation had previously been reinforced with cement injections. Afterwards, the decayed wooden piles, which were part of the old foundation, were cut and removed. This process strengthens the foundation by replacing the old wooden piles with modern ones, ensuring better stability for the structure.

The important part of this example is that the jet grouting piles fulfilled their static function without any additional pile caps or reinforced beams, which are necessary when used with micro piles.

The second example is referred to the already mentioned St John's Church in Gdansk, where the previous attempts undertaken a few centuries ago, had failed. (Topolnicki, 2004,pp. 1-9) The eastern part of the walls and new columns of the church, located closer to the river and founded on deeper organic deposits, had settled over the centuries by up to 75cm, and the settlement rate has increased owing to the lowering of groundwater level. The eastern wall suffered tilting, resulting in 140 cm of horizontal displacement at the top of the wall. Six of the eastern pillars have shown excessive settlement, horizontal displacements, and dangerous buckling (Topolnicki, 2004,pp. 1-9).

The installation of the underpinning of the columns was the most complex task. Up to 32 Soilcrete elements (sectors, panels and complete columns) had to be installed for several weeks to stabilise just one column; the example of such a column can be seen in Figure 22. Before the jet grouting works started, the stone foundation was strengthened by low-pressure cement injection inside the whole foundation assembly to avoid the structural collapse of the whole stone structure by drilling and jet grouting. After the initial works, soilcrete panels were constructed to stabilize the structure and minimise additional horizontal displacement.

Figure 22.Usage of jet grouting method in St John's Church in Gdansk. (Topolnicki, 2004,pp. 1-9).



To avoid the unacceptable settlement of the structure, the jet grouting method is carried out step by step under all foundations to allow the previously installed piles to harden. During 100 days of underpinning works, the foundation of the eastern wall has settled, without causing any additional damage, by about 7mm and the columns by approximately 11mm. Geodetic measurements taken in the period of four years after the works finished (1995-2000), showed that there was no significant settlement, only within an average deviation of 0.5mm. (Topolnicki, 2004,pp. 1-9).

5.5.5. Other preservation techniques

The Tower of Pisa is one of the most spectacular cases that required foundation strengthening and structural protection. The stabilisation of the bell tower has proved to be one of the most complicated cases for civil engineers. A permanent solution was believed to result in only a small reduction of the tower inclination. Numerous methods have been considered to preserve the foundation, including drainage, consolidation beneath the north side by electro-osmosis, consolidation by loading the ground around part of the tower with slabs and loading with anchors. (Przewłócki et al., 2005, pp. 363–372) None of the previously mentioned methods weren't satisfactory, as there were too many doubts about their effectiveness. Any vibration to the ground on the inclined side could be potentially dangerous, which crossed out the chances to use methods such as underpinning and grouting. The unique technique of soil extraction provided the necessary properties that could increase the stability of a tower, while at the same time being entirely consistent with architectural conservation. (Jamiolkowski et al., 2002) The method itself requires the use of specific equipment that includes the installation of several soil extraction pipes just beneath the foundation. The overall implementation of this method requires advanced computer soil modelling, large-scale development tests, exceptional continuous monitoring, and a complex communication and control system.

6. Energy performance

As buildings age, many older structures increasingly require urgent repairs, particularly when the integrity of load-bearing walls is at risk. In such cases, aesthetic or full-scale renovations are often considered, especially for masonry exterior walls. With rising market demands, many commercial and industrial buildings are being renovated and repurposed for residential use. (Gonçalves, 2015, pp. 1-8)

To increase energy efficiency and occupant comfort in cold climates, additional measures for installing thermal insulation on the interior side of the solid masonry walls are often to be considered. However, adding the thermal insulation without consideration of many different aspects, such as vapour permeability, possible local moisture infiltrations through the cracks, or insufficiently made foundation, may lead to problems that could cause damage to the newly made insulation layer, also the fact that insulation constructed to the inside of previously uninsulated walls, decreases the temperature of the masonry during the heating season in the cold climate, which can increase the risk for condensation and water freezing inside the walls, as well as increasing the drying time of the assembly. This combination of many factors affects the integrity and durability of the building envelope. (Gonçalves, 2015, pp. 1-8)

Due to the lack of knowledge and information, there is currently much controversy and confusion among the building designers and construction community in the questions related to the addition of the thermal insulation on the existing building with solid masonry walls, that are exposed to the cold climate conditions. The main issue with insulating solid masonry walls is the question of improving the performance of the insulation layer without losing the overall durability, which can be affected by various aspects mentioned previously.(Gonçalves, 2015, pp. 1-8)

Usually, when the outside aesthetics of the building is the primary concern, thermal insulation is installed on the interior side of the solid masonry walls, which is often regarded as a retrofit option to increase energy efficiency and occupant comfort. In the original state, before any major renovation of the wall structures has been completed, solid masonry walls with little or no thermal insulation provide an important temperature gradient. During the heating season, physical processes occurring on the surface of solid masonry walls contribute to a temperature gradient that increases the wall's temperature. This elevation in temperature reduces the possibility of masonry and mortar cracking. Additionally, the established temperature gradient enhances the more effective drying of moisture within the masonry wall structure, promoting its overall durability and integrity. Although insulating from the exterior side would be the most efficient way of applying thermal insulation, in the case of historic buildings, where the aesthetic details and overall aesthetic features of the façade are to be conserved, the only option is to add the insulation layer on the interior side of the wall. (Building of Ireland, 2024,pp. 12-14)

In the case of thermal insulation on the inside of the historical building, various issues should be considered. One of the main aspects is the decrease of the overall temperature on the surface of the masonry wall during the heating season in areas with cold climates, which can increase the risk of condensation and water freezing processes, the same as extending the drying time of the wall assembly. In the cases of conversion of original commercial or industrial buildings into residential, diverse problems may arise, such as the increase of the relative humidity levels inside the structure. The combination of longer drying time, colder temperatures, and increased relative humidity can significantly affect the integrity and durability of the building envelope. To reduce the possible risks related to the increasing thermal resistance from the inside, it is recommended to follow several basic guidelines that provide sufficient instructions on the ways of controlling exterior and interior humidity sources, that may impact the structure. (Gonçalves, 2015, pp. 1-8; Building of Ireland, 2024, pp. 12-14)

Minimising the possible rainwater infiltration through the solid masonry wall: The moisture coming from the rain penetration is potentially a large water source that can damage the wall assembly and is often related to the poor quality and condition of the masonry units and mortar joints. The additional layers of thermal insulation need to be combined with detailed joint inspections and necessary repairs of the exterior face of the masonry to minimise water infiltration into the structure. (Gonçalves, 2015, pp. 1-8)

Decrease the penetration of indoor humidity into the wall assembly through water vapour diffusion: To prevent humidity transmission through the process of water vapour diffusion, an uninterrupted vapour barrier must be included within the wall assembly. The vapour possible transmission, location and the rate of condensation within a wall structure depends on multiple factors, including the type of wall materials used and their order of installation, the same as the local outside temperature in the different seasons and relative humidity conditions maintained inside the building during the heating season.(Gonçalves, 2015, pp. 1-8)

Reduce the penetration of indoor and rainwater humidity into the wall assembly through infiltration and exfiltration: The assembly have to contain an uninterrupted air barrier system across the entire surface to reduce the humidity transfer into the wall assembly due to the exfiltration of indoor air. The air barrier enhances the reduction of the air pressure difference across the masonry wall assembly and decreases the rainwater penetration caused by the air infiltration. Minimised air leakage also increases the level of occupant comfort and energy efficiency. (Gonçalves, 2015, pp. 1-8; Rousseau, 2003, pp. 1-11)

Minimize the temperature within the wall assembly: To ensure adequate occupant comfort and optimal energy performance in buildings, it is often necessary to enhance the overall thermal resistance of the exterior walls. However, increasing the thermal resistance on the interior surface of solid masonry walls can limit the transfer of indoor heat, preventing the masonry wall from warming up during the colder winter months. Consequently, the solid masonry wall may be exposed to lower temperatures, which increases the risk of condensation and frost formation within the wall. Therefore, it is essential to keep a balance between achieving the desired energy performance and maintaining the durability of the wall assembly.(Gonçalves, 2015, pp. 1-8)

Reduce the air pressure difference across the exterior wall assembly: The air pressure difference across the building envelope is the primary factor that causes warm, moist indoor air to exfiltrate through the wall assembly (due to positive internal pressure) and allows rainwater to penetrate and infiltrate inside the structure (due to negative internal pressure). This pressure difference can arise from factors such as the stack effect, wind loads, or insufficient work of mechanical ventilation systems. Preferably, mechanical systems should be balanced to maintain a nearly neutral indoor air pressure.(Gonçalves, 2015, pp. 1-8; Lstiburek, 1999, pp. 1–38)

6.1. Insulation order

When insulating a traditional building, wall insulation should be considered one of the last options, as other measures are often more cost-effective and simpler to implement. For example, insulating and adequately sealing windows and doors, as well as insulating the roof and floor, should be prioritized. (Historic Environment Scotland, n.d.)

Traditionally, single-glazed windows are found in historic buildings. In fact, they are not as efficient as windows nowadays. However, numerous ways exist to improve the thermal performance of windows without negatively affecting their historical appearance.

Possible ways of improving the thermal performance of windows:

- Draught proofing sashes- reduce possible air leakage by up to 80%
- Drawing full-length and lined, well-fitted curtains- to control draught and reduce possible heat loss by up to 14%
- Closing timber shutters- reduce the heat loss by up to 51%
- Applying a thin layer of aerogel blanket on insulating shutters – reduce the heat loss by up to 60%
- Adding secondary glazing and using other measures like blinds and shutters- to reduce the heat loss by more than 75%

Timber sash and case windows in good condition typically have minimal air leakage and may not require draught proofing. However, it is advisable to have the house inspected to assess its condition and determine if any of the previously mentioned solutions need adjustment. (Historic Environment Scotland, n.d.)

Heat loss from doors can be reduced by:

- Draught proofing the door edges, letterboxes, and keyholes
- Fitting insulation to the inside part of the door panels
- Usage of appropriate door frames with sufficient insulation.

To fit insulation to door panels:

1. Choose the insulation solution that allows the possible vapour permeability and is thin enough to fit in the panel without affecting the aesthetic look of the doors.
2. Fit the materials used in the door assembly using an adhesive (not nails or screws).
3. Apply a thin layer of plywood on top.
4. Fix in place new beads or moulding in the original style.
5. Paint the door. (Historic Environment Scotland, n.d.)

Roofs in general contribute to around 25% of heat loss, so loft insulation is one of the most popular and effective ways how to reduce the heat loss through the roof.

Insulation can be fitted either:

- At the ceiling level, which creates a cold roof space, that works as an additional insulation layer.
- Between the rafters for a warmer roof space

The most common practice for insulating the loft is to fit the insulation material on the horizontal upper side of the ceiling in the loft, rather than trying to fit the insulation between the rafters.

Depending on the area, it is usually recommended to fit at least 270 mm of the designed or chosen insulation for it to be fully effective. The possible loft access hatches should be well-insulated and draught-proofed. Possible roof ventilation should be designed appropriately to reduce the possible condensation risks. (Historic Environment Scotland, n.d.)

The floor absorbs heat from inside the building and transfers it to the foundation or ground. Additionally, cold air from sources like doors and windows impacts the floor's temperature and overall energy consumption.

When insulating a timber floor, the most effective method is to install insulation below the floorboards. If there's a crawl space, insulation can be added without disturbing the upper floor or occupants. If not, lifting the floorboards allows insulation materials to be fitted between the floor joists.

In the case of solid floors, various solutions can be found. In the first case, if the floor is still usable and doesn't require major renovations, it is possible to lay the thermal insulation on top of the existing floor. In other scenarios when the solid floor is not sufficient for further insulation works, it is recommended to renew or build a new concrete foundation with the insulation already included in the floor assemblies

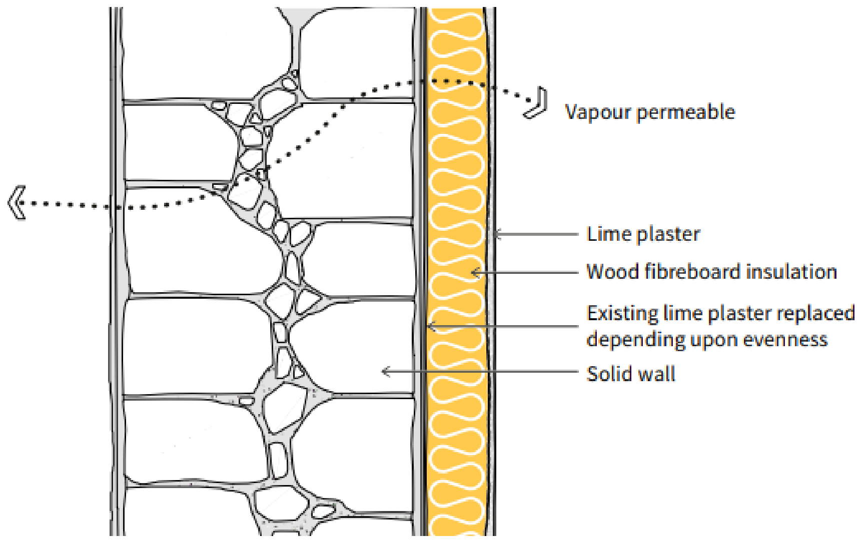
6.2. Various insulation options

Using mechanical fasteners or modifying existing fittings in walls can potentially damage the wall assembly. Such damage, including punctures made during the construction process, can create openings that allow moisture vapour to pass through. This vapour can later condense within or around the thermal insulation, which may lead to hidden rot and decay of timber elements.

While closed-cell foams are naturally vapour-impermeable, the joints between foam sections are vulnerable to moisture penetration. In traditional permeable wall structures, the weakest spots are often found at the perimeter. Moisture can bypass the physical vapour barrier through cracks or joints in the walls and floors.

However, many of these issues can be reduced by using insulation systems made from hygroscopic and vapour-permeable materials, such as wood fibre. These materials help to control moisture more effectively by allowing the wall to regulate the passage of vapour, reducing the risk of condensation and decay within the structure.

Figure 23. Internal solid wall insulation (with no vapour control layer). This shows a fully permeable insulation system using wood-fibre board and lime plaster. A new lime plaster may need to be added to the existing wall to provide an even surface if the existing plaster surface is particularly uneven or is made of gypsum. (Historic England, 2012)



6.2.1. Insulation with the framing

Where original lath and plaster wall linings are damaged or have been removed, the possible new fit of timber strapping or framing should be considered. The framing can be used to hold a thicker, board-based insulation material, which in general can increase the thermal resistance of the wall assembly

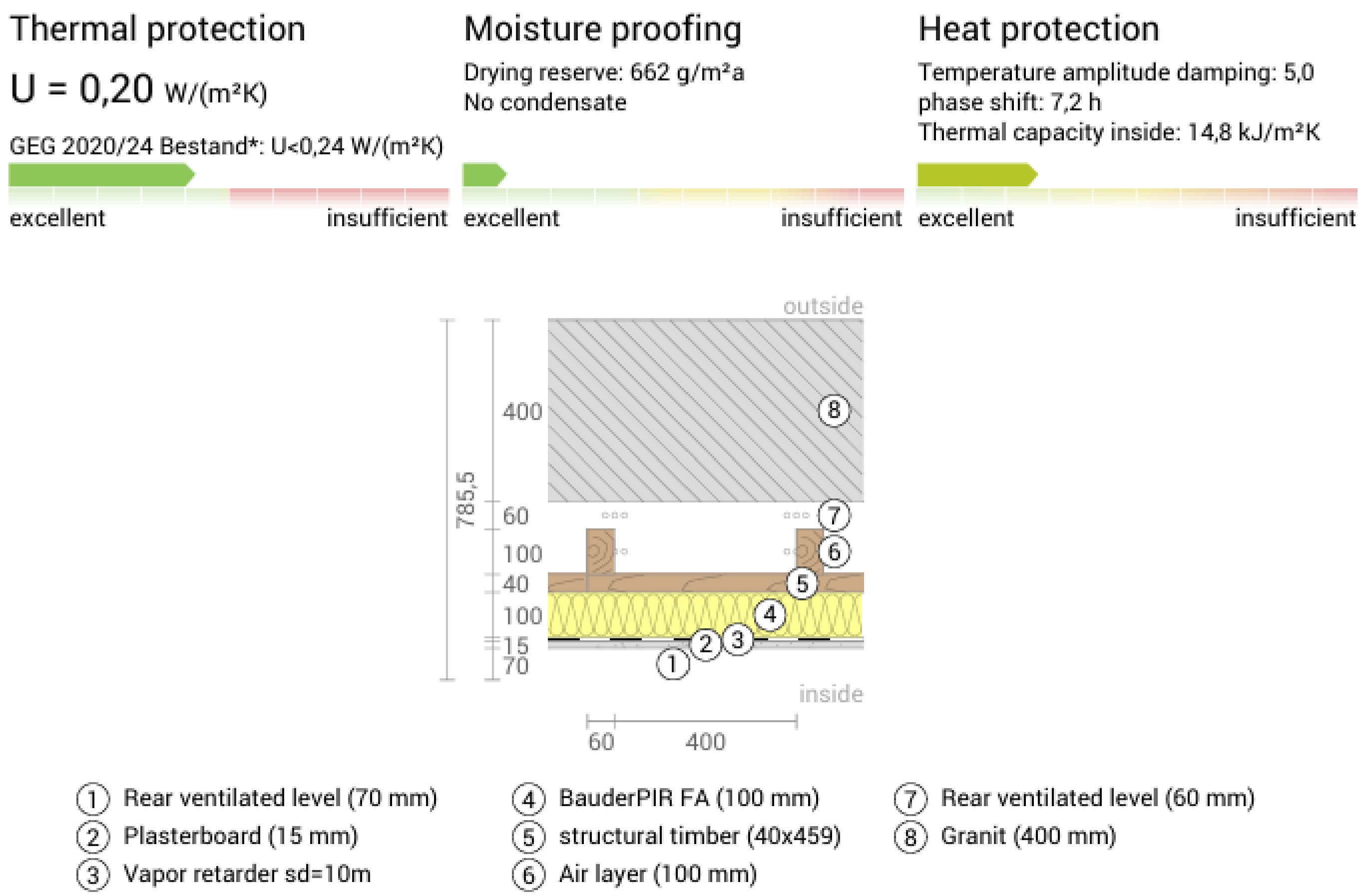
Timber framing is a well-known technique for ease of installation in the construction industry. Numerous suitable vapour-permeable insulation materials can be found in the local building material shops or the specified construction markets. Possible options are also to use rigid board to cover the framing or, in some cases, flexible batts.

Insulation materials may be used in different types of frame structures.

- Aerogel board
- Wood fibreboard
- Hemp board
- Board made from recycled textiles

The thermal improvement achieved will vary according to the type of chosen insulation material and the thickness of the assembly. It is recommended to consult the appropriate building physics specialist, as the choice of insulation may differ from building to building. (Historic Environment Scotland, n.d.)

Figure 24. Insulation of the stone wall by the use of timber framing.



6.2.2. Exterior rigid insulation

As it was mentioned earlier, insulating the stone wall on the exterior side of the wall assembly is the best solution in terms of energy efficiency perspective and moisture safety, in both forms as the rainwater penetrates through the cracks in the assembly and condensation from the interior vapour.

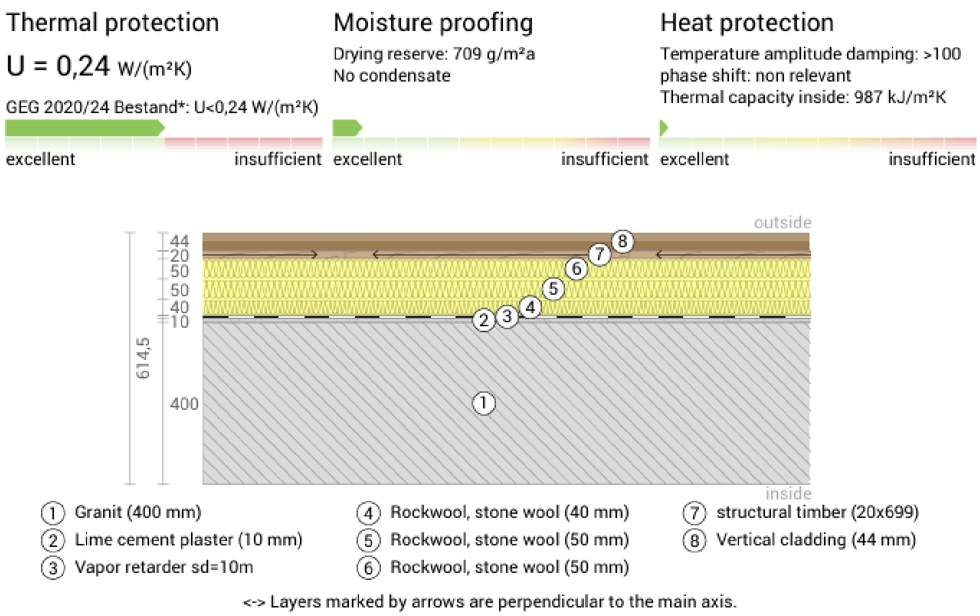
The exterior rigid insulation is made from the following materials:

1. Existing Stone Walls
2. Lime Cement Plaster
3. Fluid Applied Vapor Permeable Waterproofing
4. Rigid Mineral Wool Insulation (Rockwool board)
5. Timber battens
6. Lightweight Cladding

First, inspect the exterior stone or pebble walls for large cracks, gaps, and voids, as these buildings often have not been renovated for decades. Such openings frequently lead to increased air leakage and water infiltration. If various gaps and cracks are found, parge the stone wall with lime cement to cover the old mortar. This creates a uniform surface for the fluid-applied waterproofing, which will be installed to cover the openings. Note that the waterproofing coating cannot span gaps exceeding 3mm, which might be problematic in cases when the wall cover is not even.

In this detail (Figure 25.), the old stone walls have been parged with a lime cement rendering and waterproofed with a new fluid-applied membrane (water and air control layer). Timber battens are fastened into the stone walls along with rigid insulation layers. This detail uses a rock-wool board; however, any rigid insulation can work. Smaller timber batten strips are fastened through the insulation into the bigger batten strips that were attached to the stone walls, providing a base for a new lightweight cladding system.

Figure 25. Example of the exterior rigid insulation.



After installing the initial timber battens, which are fastened into the stone walls along with the rigid insulation, any type of rigid insulation can be used if it provides sufficient thermal properties as required by the project.

Smaller timber battens are then fastened through the insulation into the larger battens attached to the stone walls. These smaller timber battens provide a base for a new lightweight cladding system.
It is recommended to install a vapor-permeable membrane with an SD value of around 10. This allows the stone wall assembly to dry out in case of moisture infiltration. The membrane encourages the drying process on the exterior of the wall, which can help reduce the risk of blistering in fluid-applied coatings. (ASIRI Designs, n.d.)

As an alternative, one can choose to specify a cementitious parge coat or “Slurry” that can function as a waterproofing layer; mineral-based slurries can be used, which are designed to form a water-repellent chemical connection on the stone and masonry surfaces. However, if cracks start to appear on the surface, the water and air control layer might be damaged, resulting in loose of waterproofing properties. (ASIRI Designs, n.d.)

Then another layer of timber battens is fastened directly to the waterproof stone walls with certified and approved fasteners. These will provide a base for the battens attached to the rigid insulation layers. The rigid insulation layers are then fastened to the stone wall assembly so that the two layers of any rigid insulation are installed appropriately according to the manufacturer-provided information. In practice, the most long-lasting rigid insulation for the exterior used to be mineral wool, as it is composed of stone fibre; it does not rot and is not affected so much by thermal drifts; it is also highly resistant to moisture-induced damage. (ASIRI Designs, n.d.)

After the rigid insulation is installed, timber battens are fastened through the insulation layer into the 2nd timber batten layer that was fastened into the stone walls. This will provide the required load-bearing capacity for lightweight cladding, such as wood siding, aluminium panels, or fibre cement. (ASIRI Designs, n.d.)

6.2.3. Interior mineral wool and smart vapour retarder

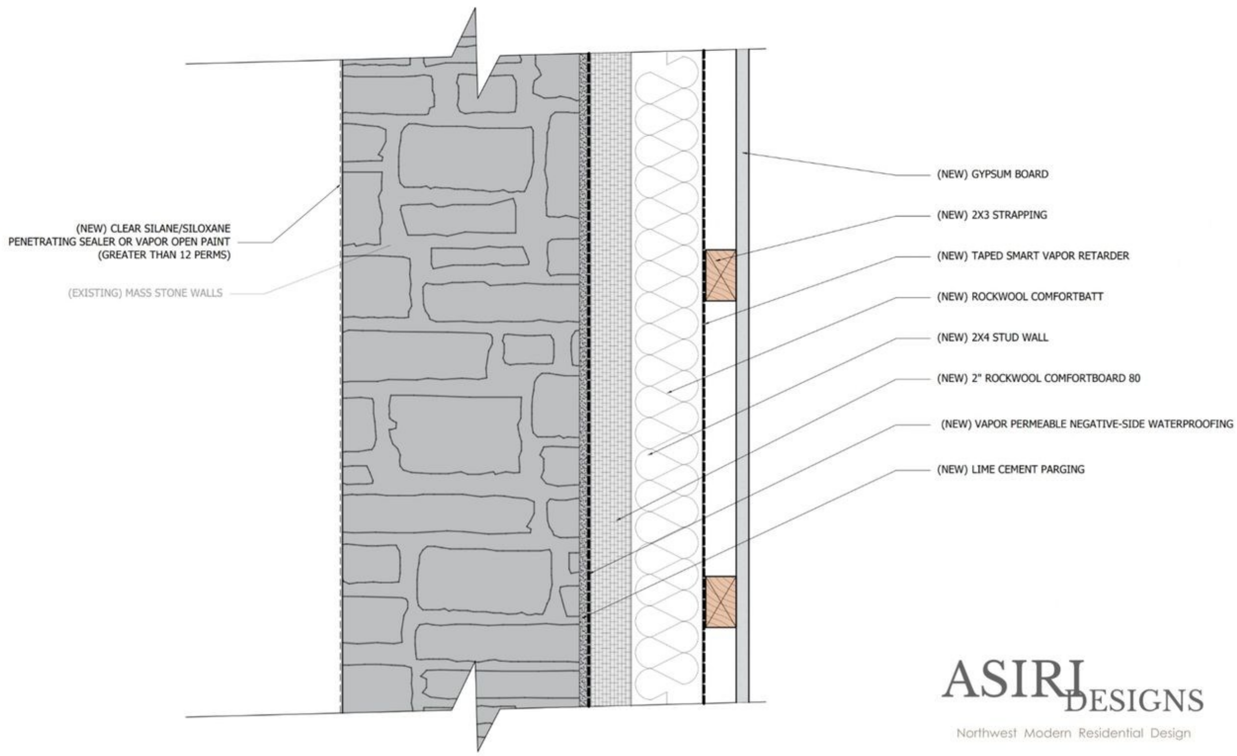
When the aesthetics of a historical stone building are a concern, exterior insulation may not be an option. In such cases, interior insulation can be considered. Using a combination of mineral wool insulation and a smart vapour retarder membrane allows the wall to dry out if moisture infiltrates, preventing interior vapour from condensing on cold stone surfaces. If the relative humidity inside the wall assembly exceeds 60%, the membrane allows vapour to escape. This differs from traditional vapour barriers, which trap moisture, potentially causing material degradation, similar to how timber frames rot when exposed to moisture for more extended periods.(ASIRI Designs, n.d.)

This wall assembly contains the following structural elements:

1. Existing stone walls
2. Lime cement parging
3. Applied vapour permeable negative-side waterproofing
4. Rigid mineral wool insulation
5. Framed walls with mineral wool batts
6. Taped Smart Vapor Retarder
7. Horizontal Furring Strips
8. Gypsum Board

In this detail, shown in Figure 26, the old stone walls have been parged on the interior with a lime cement parging. Water control is provided through a combination of a silane/siloxane penetrating sealer on the exterior of the walls to reduce the surface absorption of the stone and mortar joints, as well as a negative side interior waterproof coating to prevent water from wicking inside through cracks and gaps. The walls are insulated with mineral wool and air sealed with a smart vapour retarder membrane, which prevents vapour from diffusing into the wall assembly from the interior, which could result in condensation while allowing the wall cavity to dry to the interior if conditions within the wall become wet or humid. This strategy allows us to use air and vapour-permeable insulation types like mineral wool, wood fibre, etc. (ASIRI Designs, n.d.)

Figure 26. Insulation of the interior stone masonry wall (smart vapour retarder is used).(ASIRI Designs, n.d.)



First, the stone wall is coated in either a clear penetrating sealer or highly vapour permeable hydrophobic exterior paint to reduce the possible moisture infiltration through the cracks and mortar joints. Mineral silicate paints such as those by KEIM (can be any other brand that provides the same properties) bond to the stone's surface and span over any cracks and gaps in the stone, creating a smooth layer for the further installation of other wall assembly layers. The goal is to reduce the surface water absorption into the wall assembly. (ASIRI Designs, n.d.)

After the exterior wall has been painted or treated, the interior side can be inspected for possible cracks or general damage in the lime mortar of the stone assembly. If the walls are in good condition, a fluid-applied waterproofing layer can be applied directly to the stone surface. This will prevent moisture from infiltrating the structure through small cracks and gaps, which could otherwise be drawn into the framed wall cavity by capillary action. It is important that the fluid-applied coating is vapour-permeable, ideally with an Sd value of around 10 or higher, to ensure the stone wall can properly dry from the interior side.(ASIRI Designs, n.d.)

If the visual inspection reveals significant gaps and cracks on the interior face, it is recommended to apply a lime-cement parge coat to seal the larger voids before installing the membrane. It is crucial to use traditional lime-cement mortar, as modern Portland cement or polymer-modified products have lower permeability, which can trap moisture within the structure. Traditional lime mortars, on the other hand, allow for proper drying, helping to maintain the durability of the wall.(ASIRI Designs, n.d.)

After the interior side of the stone wall has been waterproofed, rigid mineral wool insulation is installed to provide a thermal break in the wall assembly, before the frame structure is installed. It significantly increases the thermal efficiency of the assembly since the timber frame is not connected to the stone wall, it also reduces the possible chance of timber element rotting.

In the case of mineral insulation, then is highly air and vapour-permeable, which allows the stone wall assembly to dry out. In the next stage, the framed walls can be installed along with optional mineral batts between the studs if a higher level of thermal resistance is desired. It is important to remember that walls should not be over-insulated in cold climates, as the stone walls still require heat flow to dry out. If the moisture presence inside the wall assembly during freezing temperature is too high, the possible freeze-thaw damage can, cause damage to the insulation layers and possible durability loss of the wall assembly. (ASIRI Designs, n.d.)

After the walls have been insulated with mineral wool, it is required to install taped smart vapour retarded membrane to prevent condensation in the wintertime, while also allowing the structure to dry out. A smart vapour retarded serves not only as a way of controlling the condensation inside the wall assembly but as the primary air barrier in the assembly. It is worth noting that the joints and fastening places are taped with high-quality air sealing tape to prevent moisture from infiltrating through the air leakages or damaged areas into the wall cavities, which could potentially result in the damaging of the wall assembly and possible mould growth. The benefit of a smart vapour retarder membrane is that it allows us to use a wider range of insulation types without having to depend on rigid foam or spray foam insulation, helping to reduce costs and allowing more flexibility in the design.(ASIRI Designs, n.d.)

The horizontal strapping is fastened to the studs through the smart vapour retarded member, to provide an airtight service cavity for any kind of installation and services can be installed. This can be combined with airtight enclosure boxes that can be taped directly to the smart vapour retarded membrane. This strategy allows the stone wall to continue functioning as a mass wall, while also providing the necessary moisture-safe insulation strategy. (ASIRI Designs, n.d.)

7. Structural engineering

7.1. Structural engineering in the preservation of cultural heritage buildings

Chapters 1 – 6 covered topics ranging from the significance of preserving architectural heritage to the essential geotechnical solutions that support the stability of the entire structure. This chapter, however, focuses on the role of structural engineering in cultural heritage preservation. Specifically, we will explore how structural engineers can contribute to both the success of preservation projects and the future development of cultural heritage. The following points will outline these contributions in detail.

- **Restoration and Repair** – Structural engineers play a crucial role in advising on renovating and repairing historical buildings and monuments. They ensure that the structure's original appearance is preserved while making the space safe for public use. (Cocke, n.d.)
- **Seismic Retrofitting**—In regions prone to earthquakes, historical retrofitting has become increasingly important. Specific guidelines must be followed to protect these structures from seismic events. Certified structural engineers are critical in developing solutions to improve the structural stability and earthquake resistance of structures with historical significance. (National Park Service, n.d.)
- **Conservation and Stabilization** – As previously discussed, the main challenge in many cases is not the restoration itself but rather issues with the foundation or improper conservation and stabilization practices over time. Structural engineers can find methods to stabilize both the foundation and the structure, while also protecting the site from erosion and weathering. (ProStruct Engineering, 2023)
- **Monitoring and Maintenance**—With their deep understanding of structural stability, integrity, and building physics, structural engineers are well-equipped to monitor the ongoing maintenance of historical structures. When potential issues or defects are detected, they can create a plan to prevent further deterioration, such as cracks or other factors that could affect the building's stability. (Fenton Holloway, 2021)

7.2. Roofing materials

Timber as a Historic roofing material made from organic sources, like wood shingles, often face limited lifespans due to natural decay and weathering. Over time, wood shingles erode from rain exposure and ultraviolet light. Some wood species are more durable than others, with heartwood being stronger and more decay-resistant than sapwood. For optimal performance, shingles should be split so the grain is perpendicular to the surface, reducing the risk of moisture infiltration, which accelerates decay. Prolonged dampness can also lead to moss and fungi growth, further retaining moisture and promoting rot. (Sweetser, 1984, pp. 1-8)

Sheet metals such as lead, copper, zinc, tin plate, terne plate, and galvanized iron are common in historic roofing but are prone to chemical deterioration. Pitting and streaking may result from exposure to airborne pollutants, acid rain, or organic acids from nearby wood or moss. Alkalis from materials like lime mortar or Portland cement can also cause damage. (Sweetser, 1984, pp. 1-8)

Galvanic corrosion occurs when different metals, like copper and iron, come into contact, especially in the presence of moisture. This is common in roofing when copper roofs are decorated with iron elements or steel nails are used with copper sheets. Plastic insulators or compatible fasteners can help prevent this reaction. (Sweetser, 1984, pp. 1-8)

Although roofing materials such as tiles are weather-resistant, they are fragile and can crack when struck by falling branches or mishandled. Like slate, tiles cannot support much weight, and poorly fired tiles may suffer from crazing and spalling, especially during freeze-thaw cycles. (Sweetser, 1984, pp. 1-8)

7.3. Failures of support systems

Once the condition of the roofing materials has been evaluated, it is essential to examine the associated features and support systems, both externally and internally. Regular gutters and downspout maintenance is critical, as accumulated debris can obstruct water flow, leading to water backup and seepage beneath the roofing elements. (National Park Service, 2010) Over time, this water exposure may result in the degradation of fasteners, sheathing, and other structural components. In winter, freeze-thaw cycles can cause ice formation beneath the roof surface. The pressure exerted by ice can dislodge roofing materials, such as slates, shingles, or tiles. Furthermore, the formation of ice dams above gutters can trap moisture, leading to the decay of sheathing or structural members. (National Park Service, 2010) Many large public buildings incorporate built-in gutters along the roof perimeter. These gutters may drain through downspouts within the walls or through outlets in the roof or parapet, leading to external downspouts. These systems can be effective if properly maintained, but inadequate roof slopes or clogged drains may cause water to pool on the roof surface, leading to potential damage. (National Park Service, 2010)

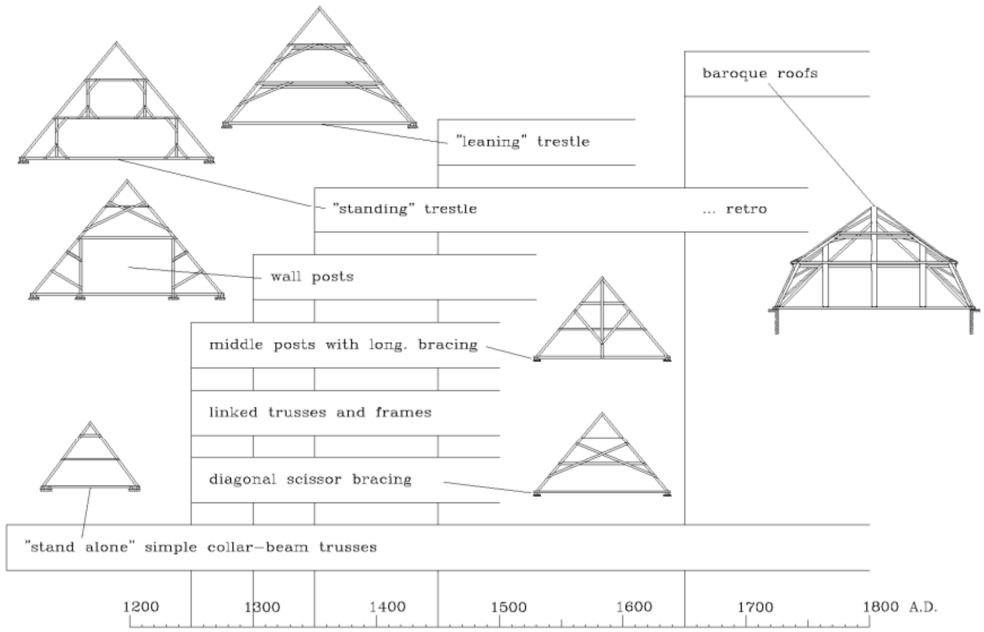
7.4. Historical research

In restoration projects, it is crucial to investigate both documents and the physical properties of the building to understand the history of the structural building elements. Document research may include original building plans found in the different archives, construction specifications, newspaper descriptions, and personal records related to the building's ownership or construction. Historical photos can also provide valuable insights into lost or altered details. (Sweetser, 2017) A physical inspection of the roofing can reveal additional information about buildings construction and renovation history. Observing changes in roof slope, material, or configuration—such as visible patchwork, alterations in exterior brickwork, or evidence of a converted gable to gambrel roof—can highlight past modifications. Similarly, stylistic changes in rooflines, dormers, or decorative elements may indicate significant alterations and guide further investigation. (Sweetser, 2017)

7.5. Structural roof elements

In renovation projects, it is necessary to preserve the historical character of the building while ensuring that the structural elements can carry the necessary loads. From the early medieval period through the 19th century, roof structures predominantly utilized timber truss systems. These systems were typically constructed from hardwood or softwood, forming the primary load-bearing elements of the roof. (Sweetser, 2017)

Figure 27. Development of the roof truss in Central Europe (Caston, 2006)



During the early medieval period, builders did not have access to modern structural calculations. Instead, they relied on empirical methods, observing that a larger cross-section provides a higher load-bearing capacity. This approach was crucial in constructing large structures like cathedrals and castles, where the roof's weight had to be supported without advanced engineering knowledge. (Caston, 2006)

Over time, as the understanding of compression and tension improved, architects began to develop more sophisticated designs. The Gothic period, for example, saw the emergence of complex roof trusses that allowed for higher and more elaborate ceilings. By the 17th and 18th centuries, advancements in mathematics and physics began to inform building practices, leading to more efficient use of materials and new architectural styles. (Caston, 2006)

The preference for oak was due to its durability, strength, and resistance to insect damage. Oak trusses have survived for centuries, demonstrating the material's effectiveness in historic structures. Other timber species, such as pine and fir, were also used, particularly in regions where oak was less available. The choice of materials often depended on local availability and the specific requirements of the building. (Aber Roof Truss, 2024)

In summary, the evolution of roof truss design reflects broader developments in engineering and architecture. While early builders relied on simple rules of thumb, later innovations allowed for more ambitious and varied architectural expressions.

7.6. Structural wall elements

In the preservation of historical buildings, structural wall materials commonly found in Central and Eastern Europe include bricks, stones, and timber. For timber elements, Eurocode 1995 provides general assessment guidelines (SFS-EN 1995-1-1/A1 Eurocode 5, 2004). However, a closer inspection of historical wall assemblies is essential before deciding on any major renovation.

For brick masonry structures, Eurocode 1996-1-1 offers general guidelines, while Eurocode 1996-1-3 provides simplified calculation methods for specific conditions (SFS-EN 1996-1-1:2022, Eurocode 6, 2022; SFS-EN 1996-3, Eurocode 6, 2006). Stone masonry and random rubble walls are more complicated to assess. Although stone masonry can be assessed using standards like BS 5628 (British Standards Institution, 2005), random rubble masonry requires both structural analysis and an in-depth literature review. In such cases, relying only on formulas from Eurocode 1996-1-1 and BS 5628 may not be reasonable. (SFS-EN 1996-1-1:2022, Eurocode 6, 2022; British Standards Institution, 2005). Older codes are often needed to provide additional understanding of structural possibilities.

The Institution of Civil Engineers (ICE) suggests that the London Building Acts 1894–1909 may be useful for assessing load and calculation methods in historical structures (Fletcher, 1914). CP 121.202 (1951) provides information about construction practices, including real-life examples (British Standards Institution, 1951), while CP 3: Chapter V (1944) outlines loading guidelines, which were expanded in the 1952 revision (British Standards Institution, 1944; British Standards Institution, 1952). Additional resources such as CP 111 and EN-771-6 may provide valuable information for the assessment of rubble and stone walls (British Standards Institution, 1970; SFS-EN 771-6 + A1, 2015).

On-site testing, such as the flat-jack method, is recommended for assessing masonry. Two relevant ASTM standards, C 1196-91 and C 1197-91, offer guidelines for in-situ compressive stress and deformability testing, respectively (ASTM, 1991a; ASTM, 1991b). European standards follow RILEM LUM.D.2 and LUM.D.3, introduced in 1990, for similar evaluations (RILEM, 1997; RILEM, 1990).

8. Local law – Latvia

8.1. Legal framework for cultural heritage preservation

Cultural heritage preservation and renovation are governed by laws designed to protect historical monuments and ensure their longevity (Supreme Council, 1992). This section examines the key legal instruments relevant to this field, focusing on the law issued by the Supreme Council in 1992, titled *"Par kultūras pieminekļu aizsardzību"* ("On Protection of Cultural Monuments") (Supreme Council, 1992), and the subsequent regulations, *"Kultūras pieminekļu uzskaites, aizsardzības, izmantošanas un restaurācijas noteikumi"* ("Regulations on the Registration, Protection, Use, and Restoration of Cultural Monuments") (National Heritage Board, 2021).

8.2. The 1992 law: "On Protection of Cultural Monuments"

The 1992 law outlines the fundamental principles and procedures for preserving historical monuments (Supreme Council, 1992). Chapter Four, titled "Preservation of Cultural Monuments," is particularly significant for understanding the legal requirements that affect construction work and the issuance of necessary permits (Supreme Council, 1992).

8.2.1. Section 21: Procedures for the research, conservation, restoration, and renovation of cultural monuments

This section mandates that any conservation, restoration, or renovation activities on cultural monuments of state and regional significance must be conducted with the written permission and oversight of the National Heritage Board (Supreme Council, 1992). Local monuments require permission from local government authorities (Supreme Council, 1992). Furthermore, research involving modifications to cultural monuments, including archaeological investigations, must also be authorized by the National Heritage Board (Supreme Council, 1992). Notably, the use of metal detectors or similar devices is strictly prohibited unless explicitly permitted (Supreme Council, 1992).

8.2.2. Section 22: Preservation of cultural monuments during construction

Before initiating construction or other economic activities, parties must survey the area for cultural values (Supreme Council, 1992). Discoveries of archaeological or historically significant objects must be reported to the National Heritage Board immediately, and all activities must be suspended until further guidance is provided (Supreme Council, 1992). This ensures that development projects do not inadvertently damage or destroy cultural heritage sites (Supreme Council, 1992).

8.2.3. Section 23: Cultural monument protection zones

Protection zones are established around immovable cultural monuments to safeguard their integrity (Supreme Council, 1992). The National Heritage Board determines these zones, which can extend up to 500 meters in rural areas and 100 meters in urban areas (Supreme Council,1992). Any activities within these zones that might impact the cultural and historical environment require prior approval from the Board (Supreme Council, 1992). The law also provides for adjustments to the size of protection zones in collaboration with local governments, depending on the specific needs of the monument (Supreme Council, 1992).

8.2.4. Regulations on the restoration of cultural monuments

In addition to the 1992 law, the 2021 law, "Regulations on the Registration, Protection, Use, and Restoration of Cultural Monuments," provides detailed guidelines for the restoration process, particularly in Section 4, "Restoration of Cultural Monuments" (National Heritage Board, 2021).

8.2.5. Restoration guidelines

The regulations emphasize the preservation of various historical layers during restoration activities (National Heritage Board, 2021). For instance, structures must maintain their original systems, materials, and finishes (National Heritage Board, 2021). Similar care must be taken with urban and rural groups of buildings, significant sites, archaeological monuments, and art monuments (National Heritage Board, 2021). These guidelines ensure that restoration efforts respect and preserve cultural monuments' original character and historical value (National Heritage Board, 2021).

8.3. Permit process

The administration is responsible for issuing permits for restoration, conservation, and repair works based on an application detailing the planned activities (National Heritage Board, 2021). This process involves evaluating documentation and, if necessary, consulting with local municipalities (National Heritage Board, 2021). The permit ensures that all work is conducted according to established standards and procedures (National Heritage Board, 2021).

8.4. Material and documentation requirements

Restoration work must use materials compatible with the original construction to maintain historical authenticity (National Heritage Board, 2021). Additionally, detailed reports documenting the work and any modifications to the original structure must be submitted to the administration and relevant municipalities (National Heritage Board, 2021). This documentation records the restoration process and helps maintain transparency and accountability (National Heritage Board, 2021).

9. Case study

The case study focuses on a partially historical rubble masonry building situated in a rural area of Latvia. The primary objective is to demonstrate how the guidelines for existing structures can be effectively applied to this particular case. The study will use the materials discussed in chapters 1 to 8, while also incorporating additional insights and details specific to the case study.

9.1. Historical background

As one of the aims of the thesis is to preserve the historical side of the building, it is crucial also to mention the local history that had a significant impact on the building and how it looks nowadays. The building is located in the manor by the name Mētaka Manor (From German: Metakshof); the original name of the manor was Sossenberg. The history starts with Zygmunt III Waza, who gifted it to Johann Tiesenhausen in 1592. However, he died without having heirs, and so the manor was taken over by his mother, born Vietinghoff. His sister, who was married to Reinhold Hörling, and their daughter Anna, who was married to Abraham Jackson, also wanted the manor. Throughout this period of time, the manor developed into a side manor of Vecumuiža, and in 1630, cattle breeding estates were formed on its territory near the village of Mētaka. However, the inheritance dispute officially ended only two years later, when Vecmuiža became formally the owner of Mētaka. In 1733, the manor was bought by the Aderkas nobel family. Around 1800, near the Mētaka manor, the priest's summer house was built (The building that is the main object of the research). In 1852, Baroness Luise Friederike Dorothea von Mengden became the manor's owner.

Figure 28. Mētaka Manor on a map from 1903.

9.2. Location

The plot is located in the northern part of Latvia, around 70 km from the capital city, Riga and 10km from the local town, Limbaži, which is also a regional centre in the local area. The plot area is around 3.68 ha and is located between two roads that have regional significance. In addition, the Rail Baltic project will also be constructed around 1km from the plot. As the location of the plot is close to the crossroads of regional roads, it is a perfect place for a small guest house, which is one of the main ideas why the renovation project is even considered. In addition to that, sustainability aspects should be taken into account, as the plot is situated in the National Biosphere and Zone C for environmental protection. As Latvia is a northern country, snow loads must be considered in the load-bearing calculations of the structural elements. Additionally, because the location is near a forest, the wind load category can be classified as III.

Figure 29. Location of the plot

9.3. Energy efficiency

Designed Floor Structure-

Top Layer: 10mm Lamination

- This layer is chosen primarily for its aesthetic appeal, enhancing the visual comfort for guests and the owner. Additionally, it contributes to the overall comfort inside the building

Second Layer: Nordic Fiber Board (+-5mm)

- As an underlay for the lamination, the Nordic Fibreboard effectively reduces the noise from footsteps while also improving the insulation properties of the wall, making the floor warmer.

Third Layer: 200mm Concrete Slab

- This layer provides the necessary load-bearing capacity of the floor, ensuring that the foundation can support the structure's loads.

Fourth and Fifth Layers: Polyfoam ECO Floorboard Standard (Both 100mm)

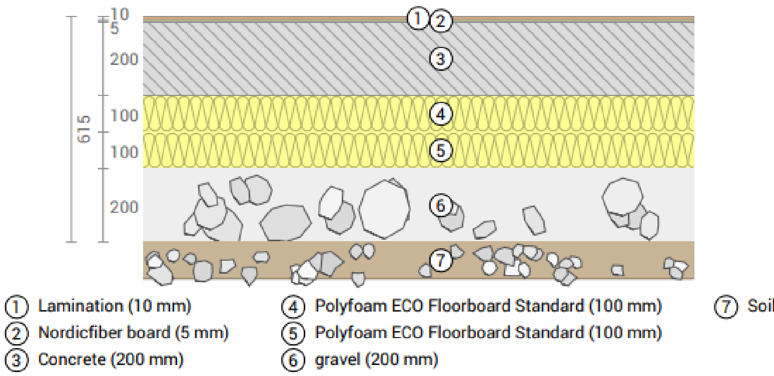
- These layers consist of Polyfoam ECO Floorboard Standard, known for its exceptionally high compressive strength, making it ideal for underfloor insulation. It is highly resistant to water absorption, crucial for maintaining the floor's integrity. Environmentally, this material is 100% recyclable and has a low global warming potential (<5). Which meets the requirements for the local area standards.

Bottom Layer: Gravel

- The final layer before the original soil is a gravel layer. This layer increases the load-bearing capacity and stabilises the concrete slab on the existing soil.

By incorporating these layers, the floor structure improves comfort and insulation and ensures durability and environmental sustainability.

Figure 30. Estimated floor structure



The building's floor is made out of multiple layers, each serving a specific function:

In the research on the exterior wall, information from Historic Environment Scotland's dedicated building conservation centre was utilised. The original structure has a 1-meter-thick stone exterior layer, which requires considering the damping that goes through the cracks and condensation that accumulates on the stones in the summer or late autumn, potentially causing structural damage to the following layers.

Designed wall structure-

Interior Layer: 120 mm Solid Bricks

- The first layer from the inside will be 120 mm solid bricks. These bricks, already present in the building's interior as a structural layer, will be reused to maintain an aesthetic appeal while contributing to the overall energy efficiency of the wall.

Insulation Layer: 120mm Polyurethane Boards

- Next, we will install 120mm polyurethane boards. These boards are highly efficient for insulation and are relatively easy to install, as the aspect of fast and easy installation is important for the local area due to the lack of qualified workers. This layer provides excellent thermal resistance without complicating the construction process

Supportive Frame: Spruce Battens and Spruce section (60x60mm)

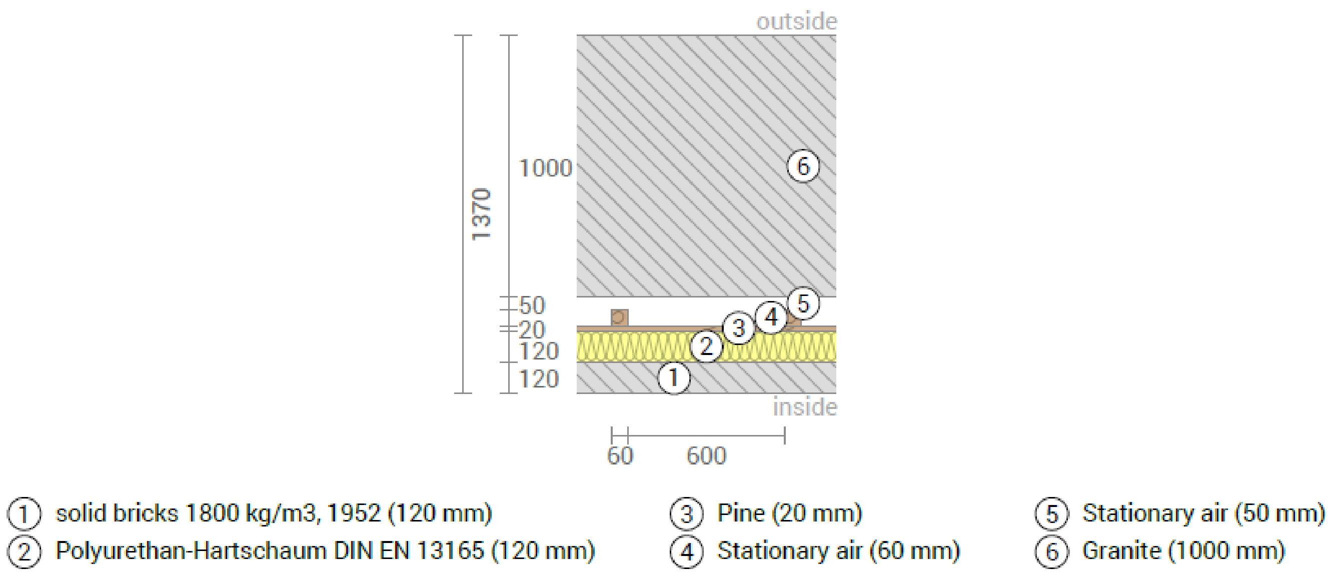
- The third and fourth layers consist of a supportive frame made from spruce battens and spruce sections (60x60mm). This structure not only provides overall support for the insulation but also creates an additional air layer for a better drying process.

Air Gap

- The frame structure in the previous layer allows the creation of an air gap, which plays a crucial role in preventing the new structure from rotting or any other forms of damage that can be caused by the moisture accumulated on the stone walls.

This multi-layered approach addresses the problems caused by moisture and significantly improves the building's thermal efficiency while also preserving the original design's aesthetic and structural integrity.

Figure 31. Wall structure



Designed roof structure-

In designing the roof structure, we have considered various practices from around the world to develop a model that best suits the local climate and weather conditions.

Interior Layer: Timber Rafter (Part of Truss element).

- The roof structure begins with timber rafters spaced at 780mm c/c. These rafters are part of the roof truss, providing the primary structural support.

Vapour Retarder

- The next layer is a vapour retarded, which doesn't allow the moisture to penetrate the structure as we have the roof cut with a thickness of almost 40 cm; the drying time is crucial for the overall health of the building. This barrier ensures efficient drying during the short Latvian summers.

Additional insulation: XPS 035 (Two panels each with a thickness of 140mm)

- A second layer of XPS 035 insulation (140mm) is added for the additional thermal resistance that ensures the layer compatibility with the given standards.

Breather Membrane: DuPont Tyvek

- The DuPont Tyvek breather membrane resists condensation within the roof structure, mainly protecting the internal layers. This membrane also allows the structure to "Breathe," preventing trapped vapour from appearing and maintaining internal dryness.

Battens: 30mm Thick

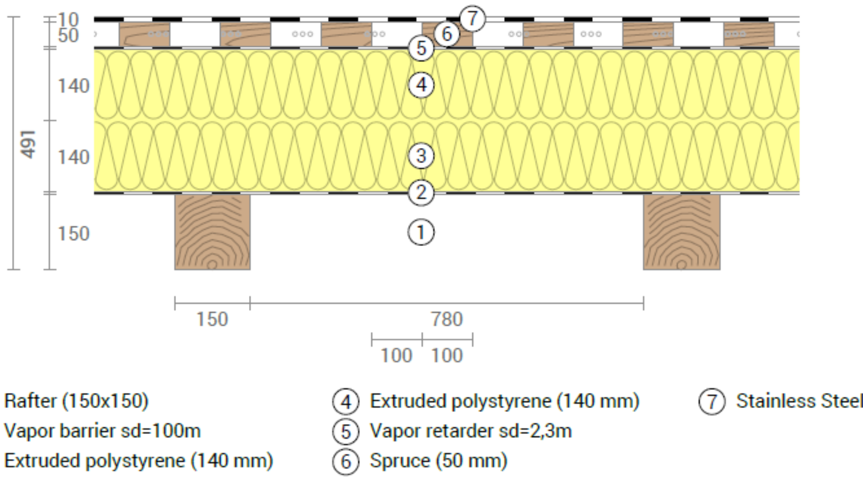
- 30 mm thick battens are installed next. These battens are impregnated with a special liquid that enhances their resistance to various negative effects caused by condensation on metal sheet roofs.

Exterior Layer: Stainless Steel Roof

- The final layer is a stainless-steel roof. This material is highly durable, requires minimal maintenance, and lasts 40 to 60 years. It is an environmentally sustainable choice, and the manufacturer can be selected based on the client's specific needs.

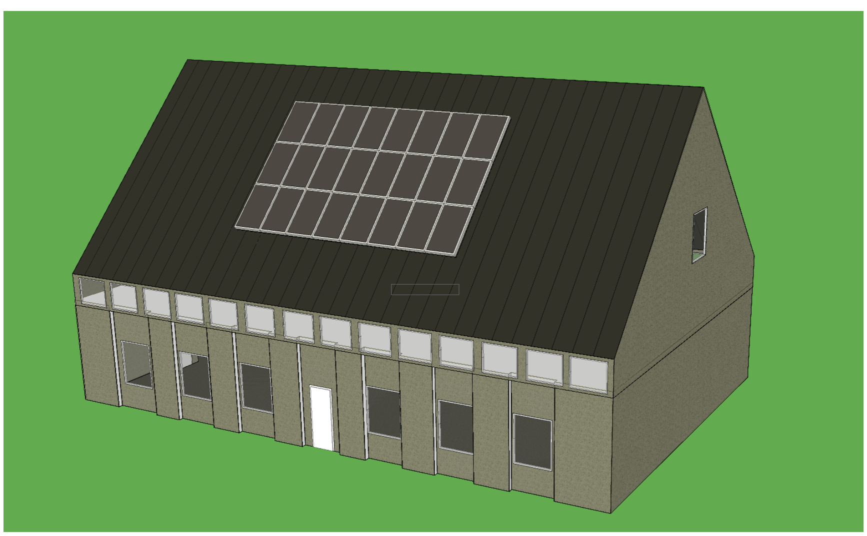
By incorporating these layers, the roof structure meets the thermal and moisture control requirements and ensures durability and sustainability in the local climate.

Figure 32. Roof structure



After defining all the structural layers required for a comfortable and efficient living environment, a sufficient IDA ICE energy balance model must be made to ensure that the chosen layer materials meet local energy standards.

Figure 33. IDA ICE model



After the modelling and a series of tests, it was found that the structure satisfies all the requirements and can be used as a living space. The analysis also demonstrated that the design achieves excellent energy balance and efficiency, with effective insulation, optimised thermal performance, and minimal energy losses, making it a sustainable and cost-effective solution for long-term use. The results are provided in Appendix 1.

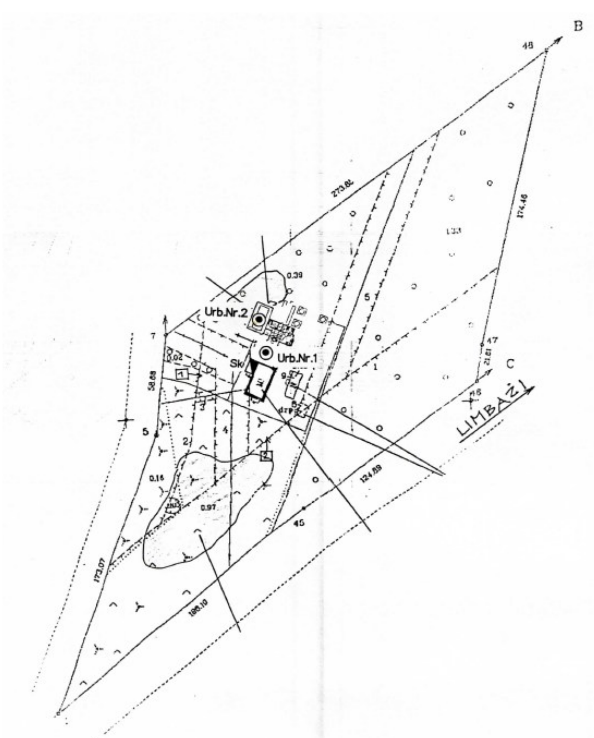
9.4. Geotechnical recommendations

Chapter 5 of this thesis mentions that it is crucial to ensure that the renovated structure can provide sufficient load-bearing capacity for newly introduced loadings, such as new roof structure and loads caused by the newly constructed second floor. To ensure that structure is capable of carrying such loads, it is essential to make sure that the foundation can transfer the loads to the ground without exceeding the serviceability settlement requirements while providing sufficient capacity during the ultimate limit state.

As the building itself is approximately 100 years old, it is important to conduct sufficient geotechnical surveys of the soil beneath. This will provide information for further assessment of the foundation capacity and possible solutions that can be implemented during the soil and foundation improvement process.

While checking the documentation from 2004 it was found that geotechnical surveying was done during the preparation of the previous project, moreover, it was found that the two boreholes drilled for the identification of load-bearing soils are acceptable only for the initial estimation/pre-design of the structure as it may provide some insight into the load bearing capacity of the soils. After consulting with the geotechnical engineer, it is generally recommended to make another 3-4 boreholes around the existing building to make sure that the soil beneath the structure is fully capable of carrying newly introduced loading. (personal project, n.d.)

Figure 34. The boreholes drilled during the 2004 geotechnical survey. (personal pictures, February 24, 2025)



9.5. Structural design

This chapter examines the structural design of the roof and walls, outlining the key assumptions and principles that guide the analysis. Emphasis is placed on the material properties, load considerations, and design methodologies relevant to the structural performance of these elements.

9.5.1. Roof structure

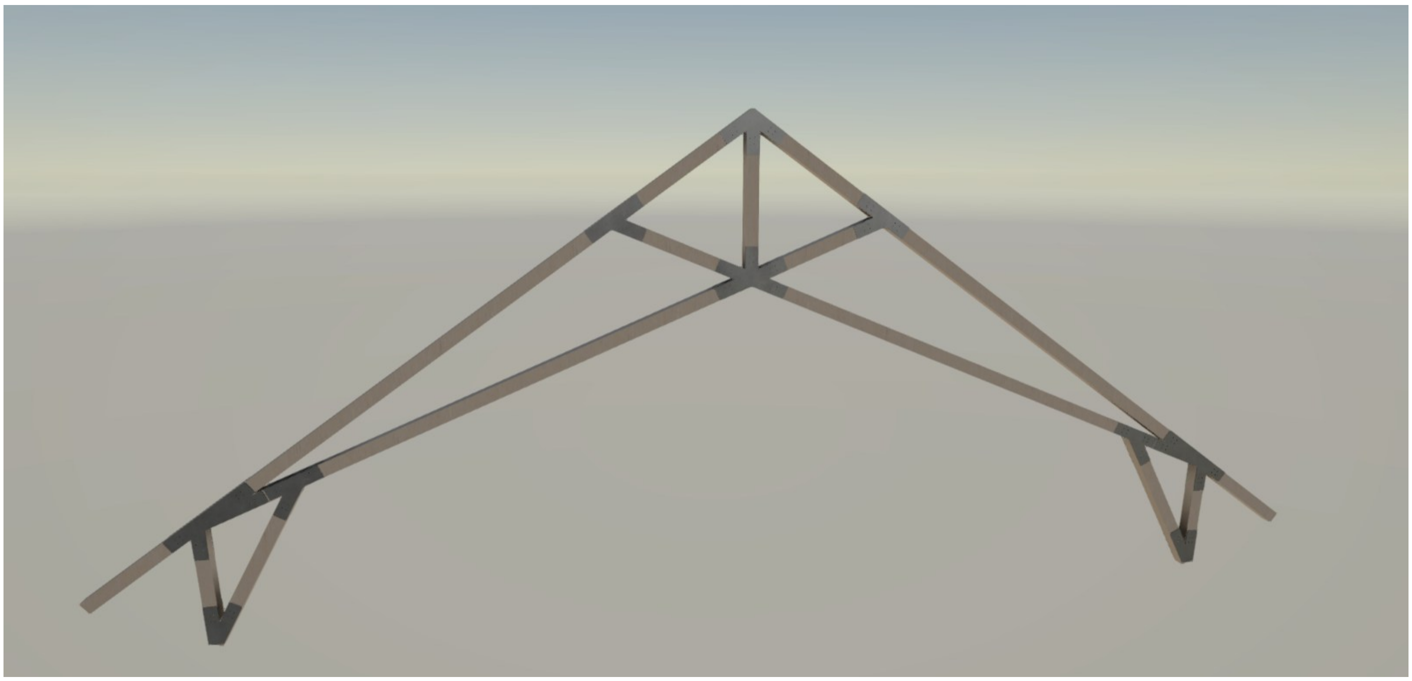
The structural design of historical buildings can be quite challenging due to potential damages that may be hidden within various structural elements. Before undertaking any major renovation work, a comprehensive assessment of the building is recommended. This evaluation should include an analysis of the quality of masonry units, as these significantly influence the load-bearing capacity of the entire structure. If any major defects or damages are identified during the assessment phase of the structures, they must be addressed before proceeding with further work. (personal project, n.d.)

The building's roof structure is composed of simple timber truss systems at both ends, while the central section is supported by timber columns connected to load-transfer elements (Figure 35). The roof covering is composed of asbestos, a material commonly used during the Soviet era in Latvia. Since the roof covering is intended for complete replacement, it is crucial to carefully remove the asbestos panels to prevent any environmental contamination. (personal project, n.d.)

Figure 35. The existing timber Roof structure. (February 24, 2025)

The new roof design incorporates the construction of new trusses, with the possibility of reusing existing timber elements that remain unaffected by environmental conditions or decay. The proposed trusses will be fabricated using timber classified as strength class C24, which is widely available. The cross-sectional dimensions of the timber truss elements will be 150 mm by 150 mm, with a total of 21 trusses planned for installation. Meanwhile, for the connections, four M16 bolts will be used in connection with two steel plates on both sides, each with a thickness of 8 mm, the truss design can be seen in Figure 36.

Figure 36. New timber truss design.



The load combinations for the roof truss assessment are derived from Eurocode 1990, while the loads acting on the structure are specified in Eurocode 1991-1-1. A more detailed analysis of snow and wind loads is conducted following Eurocode 1991-1-3 and 1991-1-4, with additional guidance from the Latvian national annexes.(SFS-EN 1991-1-1:en, 2002; SFS-EN 1991-1-3:en, 2003; SFS-EN 1991-1-4:en, 2005) These standards ensure that the structure can withstand extreme weather conditions specific to the region. Furthermore, the design incorporates safety factors as outlined in Eurocode 1990 to account for material uncertainties and variations in loading conditions. The timber elements and their connections are designed following the specifications in Eurocode 1995-1-1 (SFS-EN 1995-1-1/A1 Eurocode 5, 2004), which provides detailed guidance on the mechanical properties, durability, and joint detailing of timber structures. Additionally, Eurocode guidelines emphasize the importance of robust detailing in joints and fasteners to ensure long-term structural stability and resistance to dynamic loads.

The complete design of the timber trusses is provided in Appendix.2.

9.5.2. Wall structure

For the structural assessment of masonry units in the renovation project, Eurocode 1996-1-1 has been applied to establish general design principles, while Eurocode 1996-3 for the simplified calculation couldn't be applied because it doesn't satisfy the specific requirements for using this code (SFS-EN 1996-1-1:2022, Eurocode 6, 2022; SFS-EN 1996-3, Eurocode 6, 2006). However, the evaluation of the existing random rubble walls on-site has proven to be more challenging due to their irregular composition and historical construction methods.

To address the complexities of the existing project, the author used assessments based on BS 5628 (British Standards Institution, 2005) for stone masonry, supplemented by an extensive review of historical construction practices. For the random rubble masonry, reliance on Eurocode 1996-1-1 or BS 5628 formulas was deemed insufficient. Instead, older codes, including the London Building Acts of 1894–1909, were referenced for insights into historical loading and construction methods (Fletcher, 1914).

Key resources such as CP 121.202 (1951), which provides practical examples from mid-20th-century construction, and CP 3: Chapter V (1944, 1952 revisions) with its detailed loading guidelines, have been particularly valuable during the initial design process (British Standards Institution, 1944; British Standards Institution, 1952). Additionally, CP 111 and EN-771-6 have contributed critical information for assessing the structural capacity and material properties of rubble and stone walls in this heritage structure (British Standards Institution, 1970; SFS-EN 771-6 + A1, 2015).

After the information above has been reviewed, the main code that will be used in the design value assessment will be BS 5628, which provides general information about the strength of the random rubble masonry units (see Figure 37). The main calculations will be done using Eurocode 1996-1-1. Overall, the existing walls are constructed of random rubble with lime mortar as the bonding material. The walls are around 1m wide. Design checks can be observed in Appendix 3.

Figure 37. Masonry design consideration according to BS 5628. (British Standards Institution, 2005)

23.1.8 *Natural stone masonry*

Natural stone masonry should be designed on the basis of solid concrete blocks of an equivalent compressive strength. Where masonry is constructed from large, carefully shaped pieces with relatively thin joints, its loadbearing capacity is more closely related to the intrinsic strength of the stone than is the case where small structural units are used. Design stresses in excess of those obtained from this code may be allowed in such massive stone masonry, provided that the designer is satisfied that the properties of the stone warrant an increase.

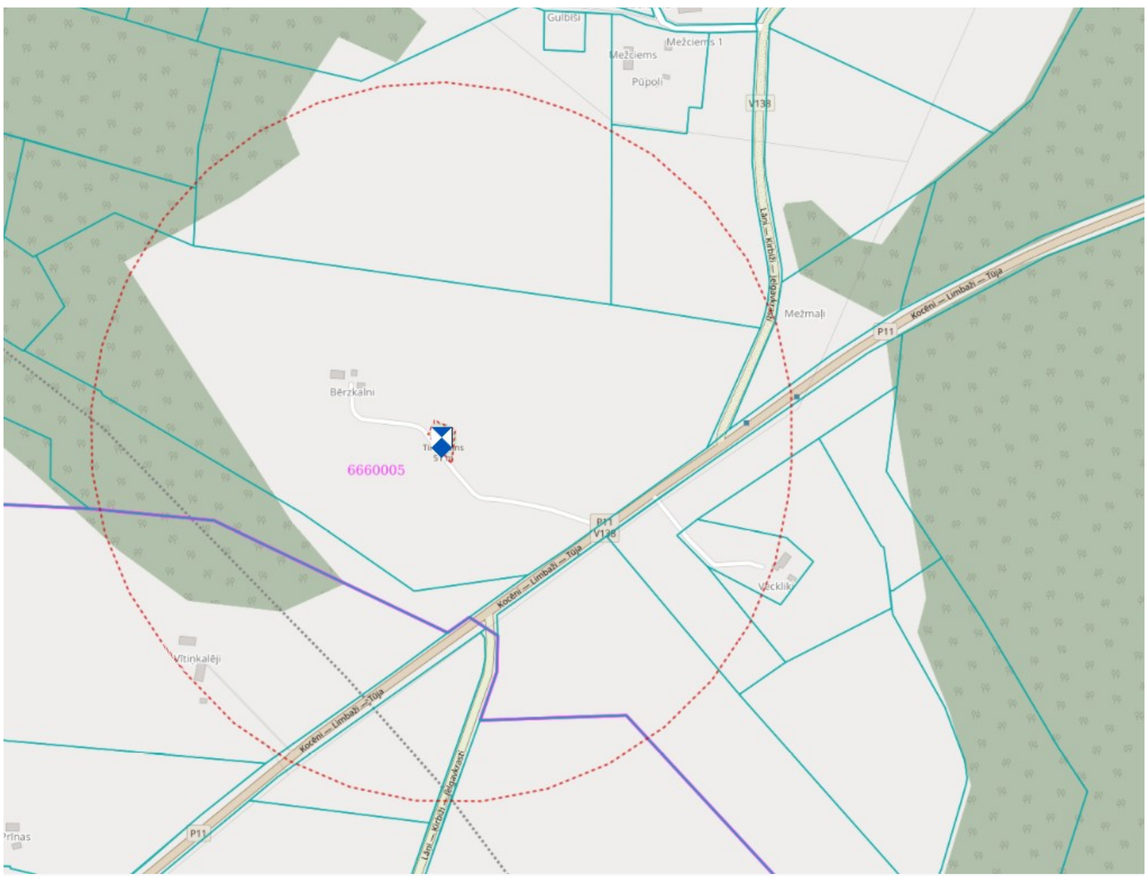
23.1.9 *Random rubble masonry*

The characteristic strength of random rubble masonry may be taken as 75 % of the corresponding strength of natural stone masonry built with similar materials. In the case of rubble masonry built with lime mortar, the characteristic strength may be taken as one-half of that for masonry in mortar designation (iv).

9.6. Law-related considerations (Latvia)

During the initial research on the historical aspects of a structure, it is crucial to review the applicable laws that might impact construction during the execution phase of the project. Such regulations can significantly extend the project schedule and increase the overall cost. In the case study, the structure is not listed as a historic heritage building in Latvia. However, it is essential to assess the surrounding area, as Latvian law specifies that buildings located within a 500-meter radius of a heritage-listed site must adhere to specific requirements before proceeding with the project. (National Heritage Board, 2021) Figure 39. illustrates the 500-meter radius around the relevant sites.

Figure 38. 500m radius of the restricted area.



In the context of our structure, specific requirements set by law are not applicable. However, the design and construction process will be guided by environmental safety considerations in the area, while preserving the owner's initial vision and ideas.

10. Conclusion

The thesis aims to provide numerous practices and methods that can be implemented during the renovation and reconstruction phase of the stone and rubble masonry structures, with an overall focus on modern construction practices with attention to the structure’s historical features. Moreover, it provides valuable insights from the numerous research, case studies, sites and other materials, that have been summarised in the thesis and successfully implemented during the creation of the case study. One of the main challenges encountered during the research process was managing the vast amount of data and addressing inconsistencies between sources. This required frequent consultations with professionals specialising in historical masonry restoration. Additionally, some articles and scientific studies provided unique yet incomparable insights, broadening the scope of the research. To further enhance the understanding of complex renovation processes, future research should incorporate additional case studies and materials. Expanding the study would allow for a more comprehensive analysis of effective restoration techniques, ultimately contributing to better-informed renovation and reconstruction projects for historical masonry structures.

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Appendix 1. Energy efficiency

Calculations for thermal insulation, moisture protection and heat protection

created on 10.1.2025 21:01

Content

	Component	U-value W/m²K	Condensate kg	TA-Attenuation	Thickness cm	Weight kg/m²	Page
1	ROOF FOR BACHELOR	0,12	0,011	4,8	49,10	31,9	2
2	FLOOR FOR BACHELOR	0,15	0,035	400,0	61,50	932,6	7
3	WALL FOR BACHELOR	0,22	-	1000,0	137,00	2844,0	10

Comparison with different maximum values*

Component	GEG 2020/24 Bestand	BEG Einzelmaßn.	GEG 2023/24 Neubau	DIN 4108
ROOF FOR BACHELOR	✓	✓	✓	✓
FLOOR FOR BACHELOR	✓	✓	✓	✓
WALL FOR BACHELOR				✓

*Comparison of the U-value with den Höchstwerten aus GEG Anlage 7 (GEG 2020-2024 Bestand); den techn. Mindestanforderungen für BEG Einzelmaßnahmen; 70% des U-Werts der Referenzausführung aus GEG 2023/2024 Anlage 1 (GEG Neubau); den R-Werten aus DIN 4108-2 Tabelle 3

ROOF FOR BACHELOR

Roof construction
created on 10.1.2025

Thermal protection

U = 0,12 W/(m²K)

GEG 2020/24 Bestand*: U<0,24 W/(m²K)

excellent insufficient

Moisture proofing

Drying reserve: 28 g/m²a
(leads to devaluation)

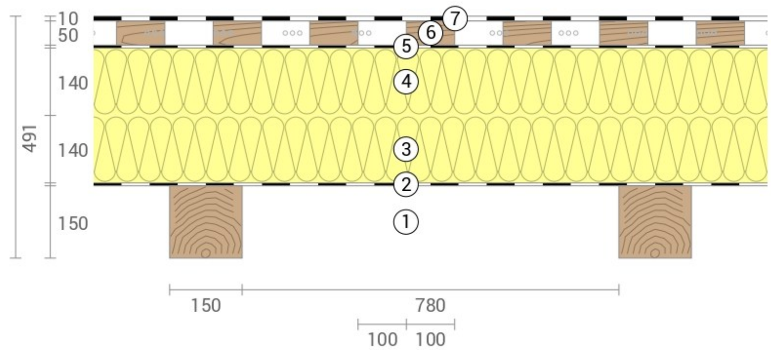
11 g/m² 31 Days +0,0%

excellent insufficient

Heat protection

Temperature amplitude damping: 4,8
phase shift: 10,0 h
Thermal capacity inside: 8,1 kJ/m²K

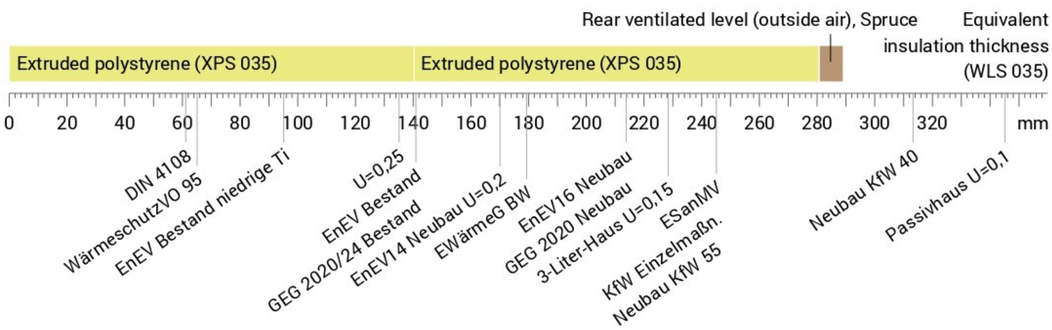
excellent insufficient



1. Rafter (150x150)
2. Vapor barrier sd=100m
3. Extruded polystyrene (140 mm)
4. Extruded polystyrene (140 mm)
5. Vapor retarder sd=2,3m
6. Spruce (50 mm)
7. Stainless Steel

Impact of each layer and comparison to reference values

For the following figure, the thermal resistances of the individual layers were converted in millimeters insulation. The scale refers to an insulation of thermal conductivity 0,035 W/mK.



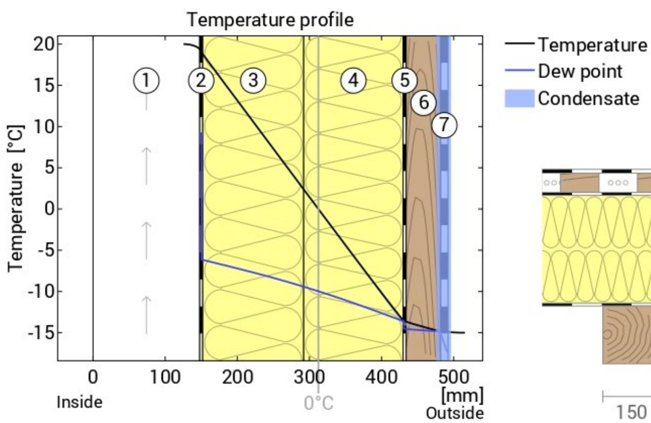
Inside air :	20,0°C / 50%			Thickness:	49,1 cm
Outside air:	-15,0°C / 80%	sd-value:	175,0 m	Weight:	32 kg/m²
Surface temperature.:	19,0°C / -14,8°C	Drying reserve:	28 g/m²a	Heat capacity:	32 kJ/m²K

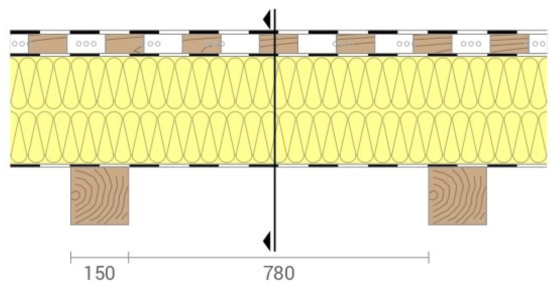
☑ GEG 2020/24 Bestand ☑ BEG Einzelmaßn. ☑ GEG 2023/24 Neubau ☑ DIN 4108

*Comparison of the U-value with den Höchstwerten aus GEG Anlage 7 (GEG 2020-2024 Bestand); den techn. Mindestanforderungen für BEG Einzelmaßnahmen; 70% des U-Werts der Referenzausführung aus GEG 2023/2024 Anlage 1 (GEG Neubau); den R-Werten aus DIN 4108-2 Tabelle 3

ROOF FOR BACHELOR, U=0,12 W/(m²K)

Temperature profile





① Rear ventilated level (150 mm)
② Vapor barrier sd=100m
③ Extruded polystyrene (140 mm)
④ Extruded polystyrene (140 mm)
⑤ Vapor retarder sd=2,3m
⑥ Spruce (50 mm)
⑦ Stainless Steel

Left: Temperature and dew-point temperature at the place marked in the right figure. The dew-point indicates the temperature, at which water vapour condensates. As long as the temperature of the component is everywhere above the dew point, no condensation occurs. If the curves have contact, condensation occurs at the corresponding position.
Right: The component, drawn to scale.

Layers (from inside to outside)

#		Material	λ [W/mK]	R [m²K/W]	Temperatur [°C] min	max	Weight [kg/m²]
1	15 cm	Rear ventilated level (room air)				20,0	0,0
		Thermal contact resistance*		0,100	19,0	20,0	
2	0,05 cm	Vapor barrier sd=100m	0,220	0,002	19,0	19,0	0,1
3	14 cm	Extruded polystyrene (XPS 035)	0,035	4,000	2,5	19,0	4,9
4	14 cm	Extruded polystyrene (XPS 035)	0,035	4,000	-14,1	2,6	4,9
5	0,05 cm	Vapor retarder sd=2,3m	0,220	0,002	-14,1	-13,6	0,1
6	5 cm	Spruce	0,130	0,385	-14,8	-13,6	10,9
	5 cm	Rear ventilated level (outside air) (50%)	0,313	0,160	-14,8	-13,9	0,0
7	1 cm	Stainless Steel (austenitic)	17,000	0,001	-14,8	-14,8	0,1
		Thermal contact resistance*		0,040	-15,0	-14,8	
	49,1 cm	Whole component		8,385			31,9

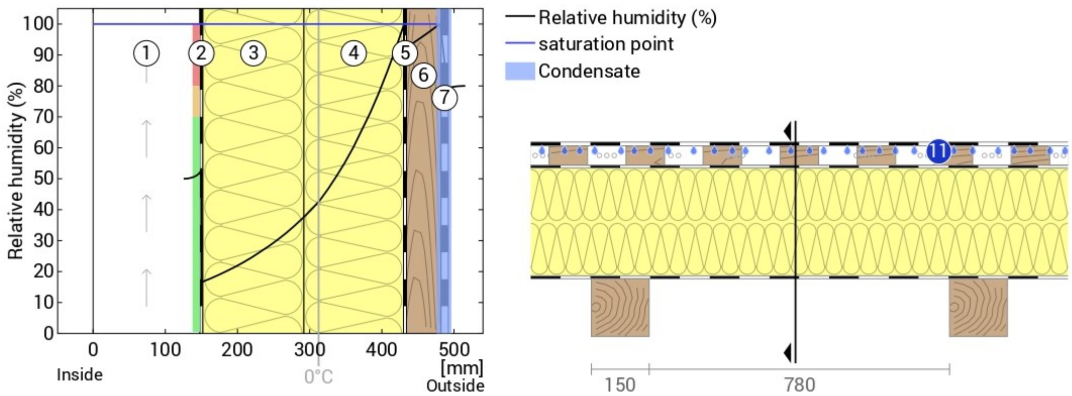
*Thermal contact resistances according to DIN 6946 for the U-value calculation. Rsi=0,25 and Rse=0,04 according to DIN 4108-3 were used for moisture proofing and temperature profile.

Surface temperature inside (min / average / max):	19,0°C	19,0°C	19,0°C
Surface temperature outside (min / average / max):	-14,8°C	-14,8°C	-14,8°C

ROOF FOR BACHELOR, U=0,12 W/(m²K)

Humidity

The temperature of the inside surface is 19,0 °C leading to a relative humidity on the surface of 53%.Mould formation is not expected under these conditions.
The following figure shows the relative humidity inside the component.



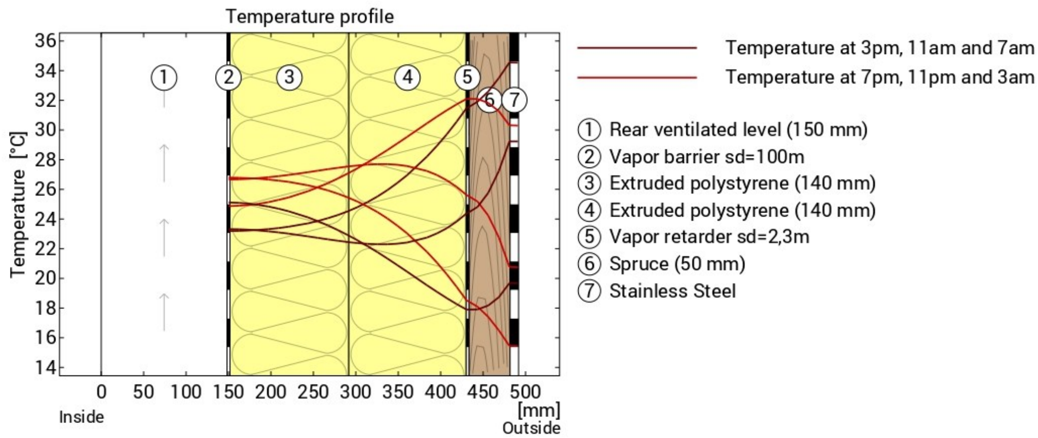
① Rear ventilated level (150 mm)
② Vapor barrier sd=100m
③ Extruded polystyrene (140 mm)
④ Extruded polystyrene (140 mm)
⑤ Vapor retarder sd=2,3m
⑥ Spruce (50 mm)
⑦ Stainless Steel

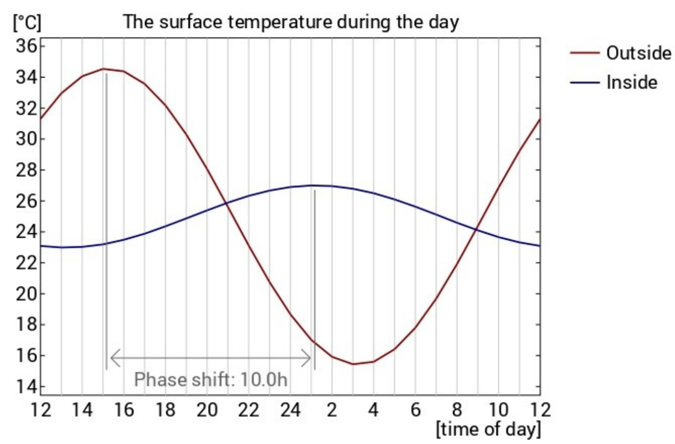
Notes: Calculation using the Ubakus 2D-FE method. Convection and the capillarity of the building materials were not considered. The drying time may take longer under unfavorable conditions (shading, damp / cool summers) than calculated here.

ROOF FOR BACHELOR, U=0,12 W/(m²K)

Heat protection

The following results are properties of the tested component alone and do not make any statement about the heat protection of the entire room:





Top: Temperature profile within the component at different times. From top to bottom, brown lines: at 3 pm, 11 am and 7 am and red lines at 7 pm , 11 pm and 3 am.
Bottom: Temperature on the outer (red) and inner (blue) surface in the course of a day. The arrows indicate the location of the temperature maximum values . The maximum of the inner surface temperature should preferably occur during the second half of the night.

Phase shift*	10,0 h	Heat storage capacity (whole component):	32 kJ/m²K
Amplitude attenuation **	4,8	Thermal capacity of inner layers:	8.1 kJ/m²K
TAV ***	0,209		

* The phase shift is the time in hours after which the temperature peak of the afternoon reaches the component interior.
** The amplitude attenuation describes the attenuation of the temperature wave when passing through the component. A value of 10 means that the temperature on the outside varies 10x stronger than on the inside, e.g. outside 15-35 °C, inside 24-26 °C.
*** The temperature amplitude ratio TAV is the reciprocal of the attenuation: TAV = 1 / amplitude attenuation

Note: The heat protection of a room is influenced by several factors, but essentially by the direct solar radiation through windows and the total amount of heat storage capacity (including floor, interior walls and furniture). A single component usually has only a very small influence on the heat protection of the room.

The calculations presented above have been created for a 1-dimensional cross-section of the component.

FLOOR FOR BACHELOR

Floor
created on 10.1.2025

Thermal protection

U = 0,15 W/(m²K)

GEG 2020/24 Bestand*: U<0,3 W/(m²K)

excellent insufficient

Moisture proofing

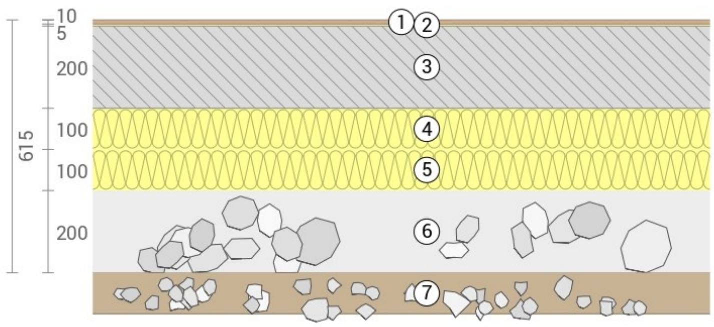
Dries 28 days
Condensate: 35 g/m²

excellent insufficient

Heat protection

Component is adjacent to earth:
TAV and phase non relevant
Thermal capacity inside: 442 kJ/m²K

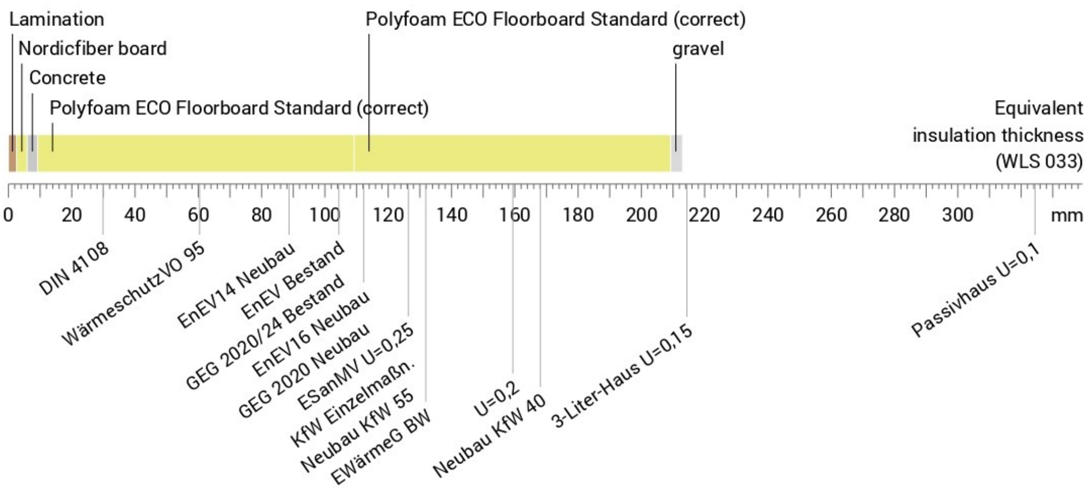
excellent insufficient



1. Lamination (10 mm)
2. Nordicfiber board (5 mm)
3. Concrete (200 mm)
4. Polyfoam ECO Floorboard Standard (100 mm)
5. Polyfoam ECO Floorboard Standard (100 mm)
6. gravel (200 mm)
7. Soil

Impact of each layer and comparison to reference values

For the following figure, the thermal resistances of the individual layers were converted in millimeters insulation. The scale refers to an insulation of thermal conductivity 0,033 W/mK.



Inside air :	20,0°C / 50%			Thickness:	61,5 cm
Ground:	0,0°C / 100%	sd-value:	32,3 m	Weight:	933 kg/m²
Surface temperature.:	19,3°C / 0,1°C			Heat capacity:	912 kJ/m²K

☑ GEG 2020/24 Bestand ☑ BEG Einzelmaßn. ☑ GEG 2023/24 Neubau ☑ DIN 4108

*Comparison of the U-value with den Höchstwerten aus GEG Anlage 7 (GEG 2020-2024 Bestand); den techn. Mindestanforderungen für BEG Einzelmaßnahmen; 70% des U-Werts der Referenzausführung aus GEG 2023/2024 Anlage 1 (GEG Neubau); den R-Werten aus DIN 4108-2 Tabelle 3

FLOOR FOR BACHELOR, U=0,15 W/(m²K)

Moisture proofing

For the calculation of the amount of condensation water, the component was exposed to the following constant climate for 90 days: inside: 20°C und 50% Humidity; outside: 0°C und 100% Humidity (Climate according to user input).

Under these conditions, a total of 0,035 kg of condensation water per square meter is accumulated.This quantity dries in summer in 28 days (Drying season according to DIN 4108-3:2018-10).

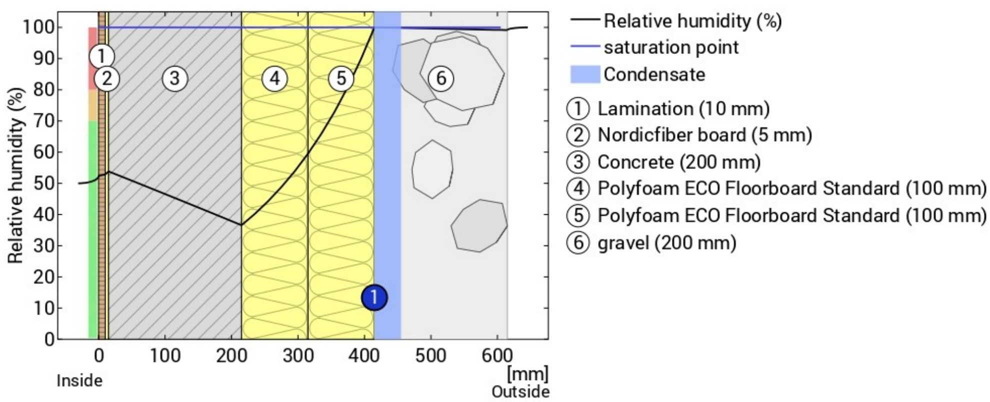
#		Material	sd-value [m]	Condensate [kg/m²]	Condensate [Gew.-%]	Weight [kg/m²]
1	1 cm	Lamination	0,30	-	-	5,0
2	0,5 cm	Nordicfiber board	0,01	-		1,3
3	20 cm	Concrete	16,00	-		480,0
4	10 cm	Polyfoam ECO Floorboard Standard (correct)	3,00	-		3,2
5	10 cm	Polyfoam ECO Floorboard Standard (correct)	3,00	0,035		3,2
6	20 cm	gravel	10,00	0,035		440,0
	61,5 cm	Whole component	32,31	0,035		932,7

Condensation areas

1 Condensate: 0,035 kg/m² Affected layers: gravel, Polyfoam ECO Floorboard Standard (correct)

Humidity

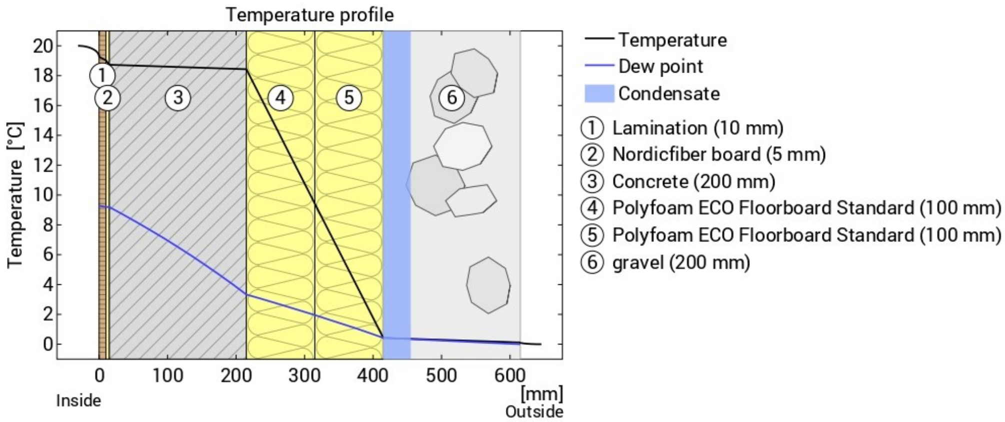
The temperature of the inside surface is 19,3 °C leading to a relative humidity on the surface of 52%.Mould formation is not expected under these conditions.
The following figure shows the relative humidity inside the component.



Notes: Calculation using the Ubakus 2D-FE method. Convection and the capillarity of the building materials were not considered. The drying time may take longer under unfavorable conditions (shading, damp / cool summers) than calculated here.

FLOOR FOR BACHELOR, U=0,15 W/(m²K)

Temperature profile



Temperature and dew-point temperature in the component. The dew-point indicates the temperature, at which water vapour condensates. As long as the temperature of the component is everywhere above the dew-point temperature, no condensation occurs. If the curves have contact, condensation occurs at the corresponding position.

Layers (from inside to outside)

#		Material	λ [W/mK]	R [m²K/W]	Temperatur [°C] min	max	Weight [kg/m²]
		Thermal contact resistance*		0,170	19,3	20,0	
1	1 cm	Lamination	0,130	0,077	19,0	19,3	5,0
2	0,5 cm	Nordicfiber board	0,049	0,102	18,7	19,0	1,3
3	20 cm	Concrete	2,000	0,100	18,4	18,7	480,0
4	10 cm	Polyfoam ECO Floorboard Standard (correct)	0,033	3,030	9,4	18,4	3,2
5	10 cm	Polyfoam ECO Floorboard Standard (correct)	0,033	3,030	0,4	9,4	3,2
6	20 cm	gravel	2,000	0,100	0,1	0,4	440,0
		Thermal contact resistance*		0,000	0,0	0,1	
7		Soil			0,0	0,0	104,6
	61,5 cm	Whole component		6,611			932,7

*Thermal contact resistances according to DIN 6946 for the U-value calculation. Rsi=0,25 and Rse=0,04 according to DIN 4108-3 were used for moisture proofing and temperature profile.

Surface temperature inside (min / average / max):	19,3°C	19,3°C	19,3°C
Surface temperature outside (min / average / max):	0,1°C	0,1°C	0,1°C

WALL FOR BACHELOR

Exterior wall
created on 10.1.2025

Thermal protection

U = 0,22 W/(m²K)

Interior insulation: No requirement*

excellent insufficient

Moisture proofing

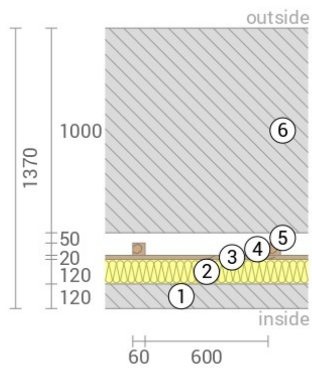
Drying reserve: 0 g/m²a
(leads to devaluation)
(Condensation on outer side of the cavity)

excellent insufficient

Heat protection

Temperature amplitude damping: >100
phase shift: non relevant
Thermal capacity inside: 300 kJ/m²K

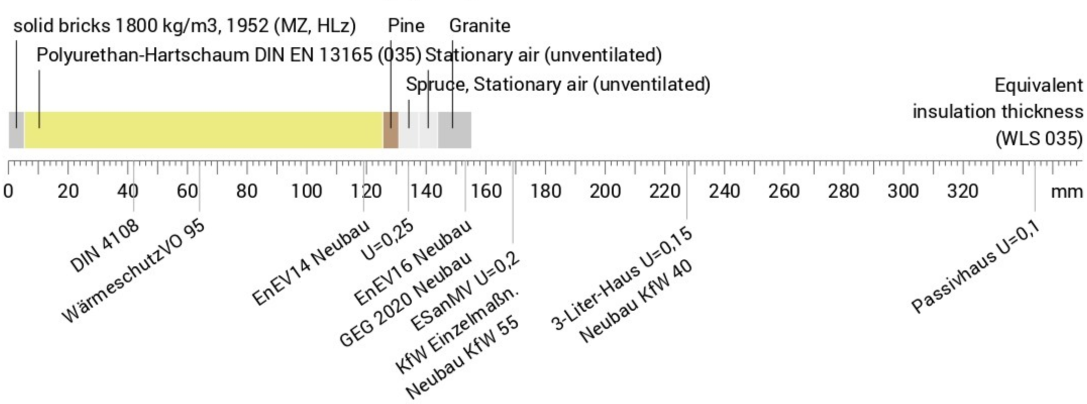
excellent insufficient



1. solid bricks 1800 kg/m3, 1952 (120 mm)
2. Polyurethan-Hartschaum DIN EN 13165 (120 mm)
3. Pine (20 mm)
4. Stationary air (60 mm)
5. Stationary air (50 mm)
6. Granite (1000 mm)

Impact of each layer and comparison to reference values

For the following figure, the thermal resistances of the individual layers were converted in millimeters insulation. The scale refers to an insulation of thermal conductivity 0,035 W/mK.



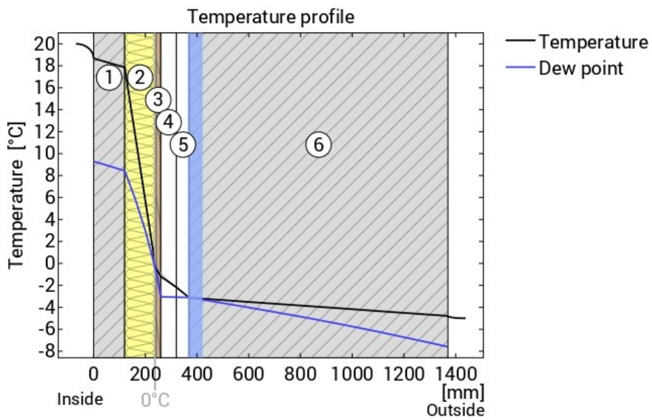
Inside air :	20,0°C / 50%		Thickness:	137,0 cm
Outside air:	-5,0°C / 80%	sd-value: 36,6 m	Weight:	2844 kg/m²
Surface temperature.:	18,7°C / -4,8°C		Heat capacity:	2102 kJ/m²K

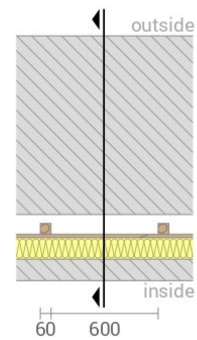
☐ GEG 2020/24 Bestand ☐ BEG Einzelmaßn. ☐ GEG 2023/24 Neubau ☑ DIN 4108

*Comparison of the U-value with den Höchstwerten aus GEG Anlage 7 (GEG 2020-2024 Bestand); den techn. Mindestanforderungen für BEG Einzelmaßnahmen; 70% des U-Werts der Referenzausführung aus GEG 2023/2024 Anlage 1 (GEG Neubau); den R-Werten aus DIN 4108-2 Tabelle 3

WALL FOR BACHELOR, U=0,22 W/(m²K)

Temperature profile





① solid bricks 1800 kg/m3, 1952 (120 m ③ Pine (20 mm) ⑤ Stationary air (50 mm)
② Polyurethan-Hartschaum DIN EN 1... ④ Stationary air (60 mm) ⑥ Granite (1000 mm)

Left: Temperature and dew-point temperature at the place marked in the right figure. The dew-point indicates the temperature, at which water vapour condensates. As long as the temperature of the component is everywhere above the dew point, no condensation occurs. If the curves have contact, condensation occurs at the corresponding position.
Right: The component, drawn to scale.

Layers (from inside to outside)

#		Material	λ [W/mK]	R [m²K/W]	Temperatur [°C] min	max	Weight [kg/m²]
		Thermal contact resistance*		0,130	18,7	20,0	
1	12 cm	solid bricks 1800 kg/m3, 1952 (MZ, HLz)	0,790	0,152	17,9	18,7	216,0
2	12 cm	Polyurethan-Hartschaum DIN EN 13165 (035)	0,035	3,429	-0,4	17,9	15,0
3	2 cm	Pine	0,130	0,154	-1,2	-0,1	10,4
4	6 cm	Stationary air (unventilated)	0,333	0,180	-2,2	-1,0	0,1
	6 cm	Spruce (9,1%)	0,130	0,462	-2,3	-0,8	2,5
5	5 cm	Stationary air (unventilated)	0,278	0,180	-3,1	-2,2	0,1
6	100 cm	Granite	3,200	0,313	-4,8	-3,1	2.600,0
		Thermal contact resistance*		0,040	-5,0	-4,8	
	137 cm	Whole component		4,592			2.844,0

*Thermal contact resistances according to DIN 6946 for the U-value calculation. Rsi=0,25 and Rse=0,04 according to DIN 4108-3 were used for moisture proofing and temperature profile.

Surface temperature inside (min / average / max):	18,7°C	18,7°C	18,7°C
Surface temperature outside (min / average / max):	-4,8°C	-4,8°C	-4,8°C

WALL FOR BACHELOR, U=0,22 W/(m²K)

Moisture proofing

For the calculation of the amount of condensation water, the component was exposed to the following constant climate for 90 days: inside: 20°C und 50% Humidity; outside: -5°C und 80% Humidity. This climate complies with DIN 4108-3.

Condensation forms only on the inside of the outer shell. Because core insulation and outer shell are made of moisture-resistant materials, this condensate is harmless. Please confirm this automatic assessment by an expert.

Drying reserve according to Ubakus 2D-FE method: 0 g/(m²a)
At least required by DIN 68800-2: 100 g/(m²a)
The moisture protection of this component is therefore rated poorly.

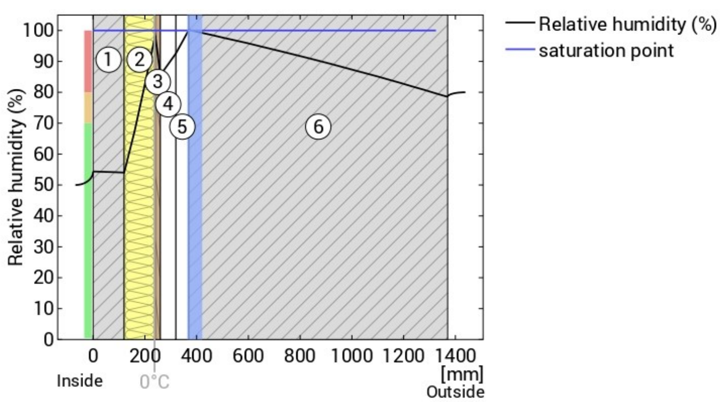
#		Material	sd-value [m]	Condensate [kg/m²]	Condensate [Gew.-%]	Weight [kg/m²]
1	12 cm	solid bricks 1800 kg/m3, 1952 (MZ, HLz)	0,60	-		216,0
2	12 cm	Polyurethan-Hartschaum DIN EN 13165 (035)	4,80	-		15,0
3	2 cm	Pine	1,00	-	-	10,4
4	6 cm	Stationary air (unventilated)	0,01	-		0,1
	6 cm	Spruce (9,1%)	3,00	-	-	2,5
5	5 cm	Stationary air (unventilated)	0,01	0,16		0,1
6	100 cm	Granite	30,00	0,16		2.600,0
	137 cm	Whole component	36,61	0		2.844,0

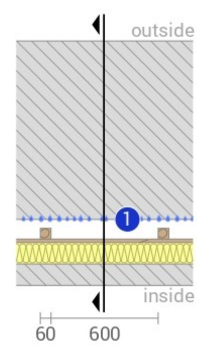
Condensation areas

1 Condensate: 0,16 kg/m² Affected layers: Granite, Stationary air (unventilated) Since all affected layers are insensitive to moisture, this condensation area does not downgrade the moisture proofing.

Humidity

The temperature of the inside surface is 18,7 °C leading to a relative humidity on the surface of 54%.Mould formation is not expected under these conditions.
The following figure shows the relative humidity inside the component.





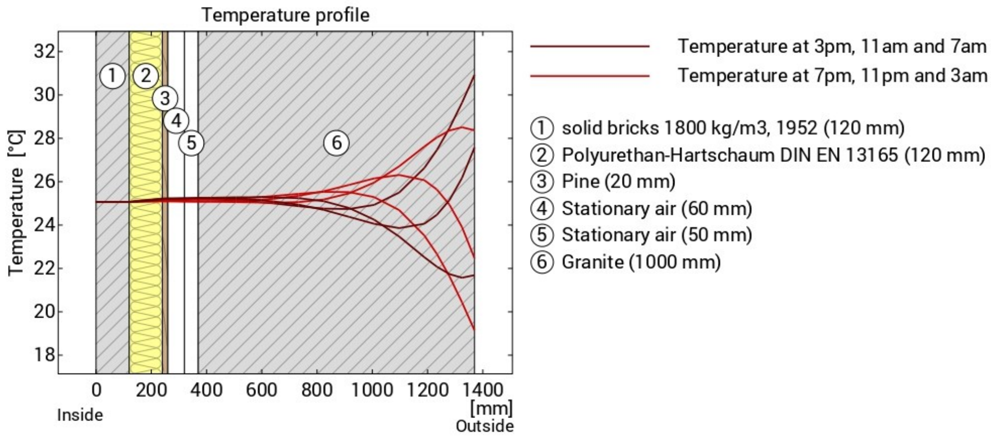
① solid bricks 1800 kg/m3, 1952 (120 m ③ Pine (20 mm)
② Polyurethan-Hartschaum DIN EN 1... ④ Stationary air (60 mm)
⑤ Stationary air (50 mm)
⑥ Granite (1000 mm)

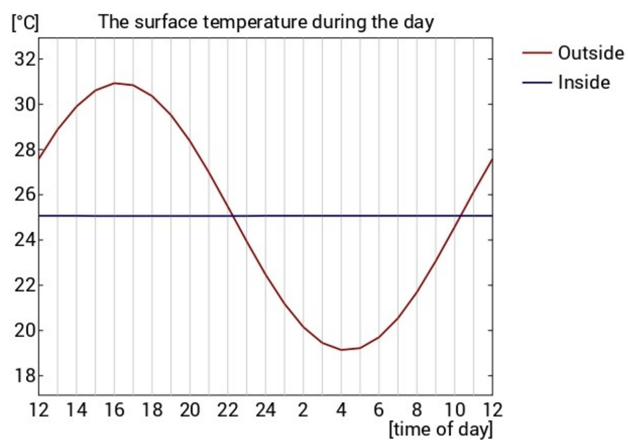
Notes: Calculation using the Ubakus 2D-FE method. Convection and the capillarity of the building materials were not considered. The drying time may take longer under unfavorable conditions (shading, damp / cool summers) than calculated here.

WALL FOR BACHELOR, U=0,22 W/(m²K)

Heat protection

The following results are properties of the tested component alone and do not make any statement about the heat protection of the entire room:





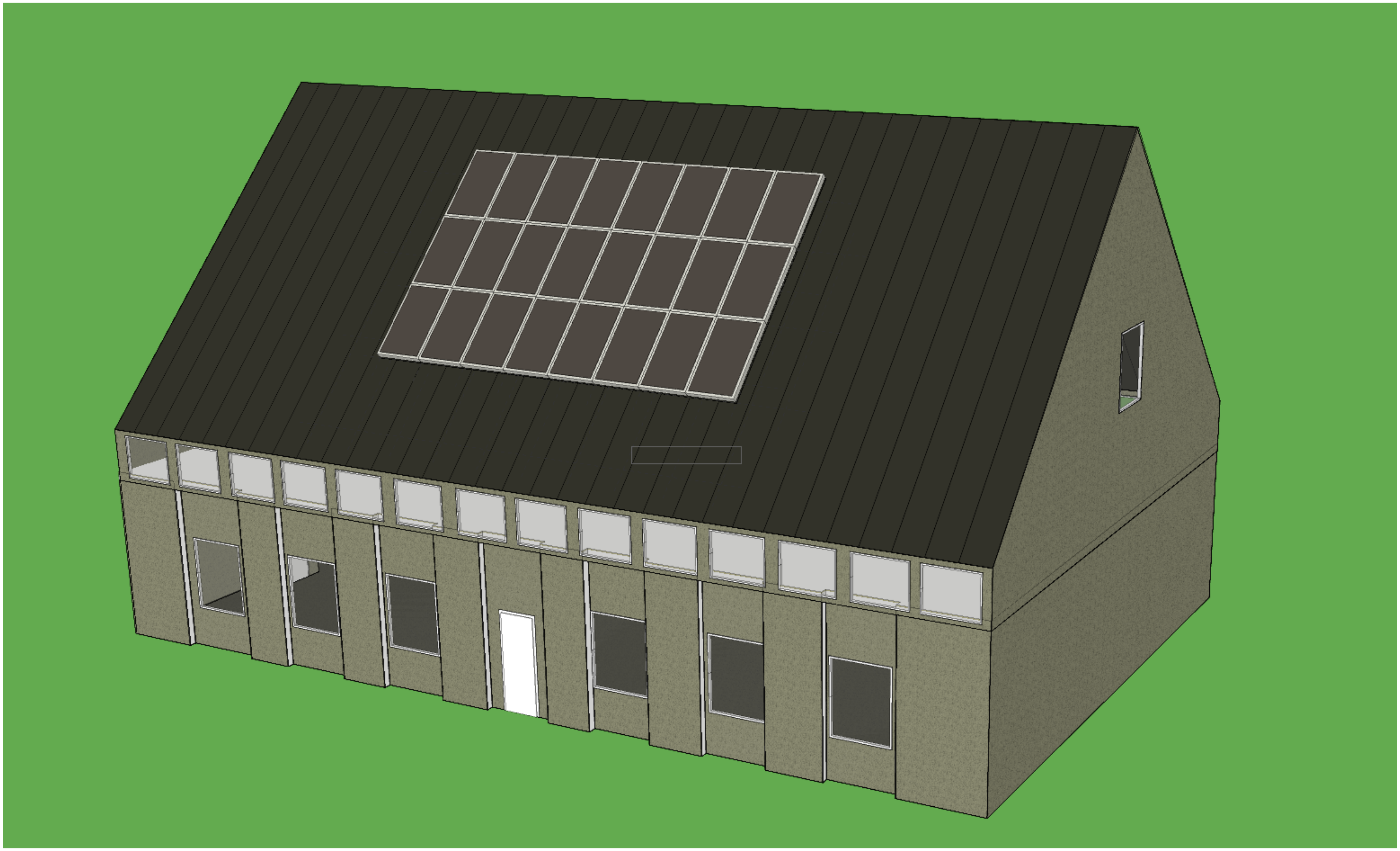
Top: Temperature profile within the component at different times. From top to bottom, brown lines: at 3 pm, 11 am and 7 am and red lines at 7 pm , 11 pm and 3 am.
Bottom: Temperature on the outer (red) and inner (blue) surface in the course of a day. The arrows indicate the location of the temperature maximum values . The maximum of the inner surface temperature should preferably occur during the second half of the night.

Phase shift*	non relevant	Heat storage capacity (whole component):	2102 kJ/m²K
Amplitude attenuation **	>100	Thermal capacity of inner layers:	300 kJ/m²K
TAV ***	0,001		

* The phase shift is the time in hours after which the temperature peak of the afternoon reaches the component interior.
** The amplitude attenuation describes the attenuation of the temperature wave when passing through the component. A value of 10 means that the temperature on the outside varies 10x stronger than on the inside, e.g. outside 15-35 °C, inside 24-26 °C.
*** The temperature amplitude ratio TAV is the reciprocal of the attenuation: TAV = 1 / amplitude attenuation

Note: The heat protection of a room is influenced by several factors, but essentially by the direct solar radiation through windows and the total amount of heat storage capacity (including floor, interior walls and furniture). A single component usually has only a very small influence on the heat protection of the room.

The calculations presented above have been created for a 1-dimensional cross-section of the component.



EQUA SIMULATION TECHNOLOGY GROUP		Delivered Energy Report	
Project		**Building**	
		Model floor area	459.2 m^2
Customer		Model volume	1737.2 m^3
Created by	martins viksna	Model ground area	323.6 m^2
Location	AINAZI_262290 (ASHRAE 2021)	Model envelope area	1268.9 m^2
Climate file	LVA_VALGA_262470(IW2)	Window/Envelope	4.2 %
Case	BACHELOR	Average U-value	0.2132 W/(m^2 K)
Simulated	10.01.2025 21:47:43	Envelope area per Volume	0.7304 m^2/m^3

Building Comfort Reference

Percentage of hours when operative temperature is above 27°C in worst zone	0 %
Percentage of hours when operative temperature is above 27°C in average zone	0 %
Percentage of total occupant hours with thermal dissatisfaction	13 %

Overall Energy Performance (ISO 52000-1, Chapter 9.6)

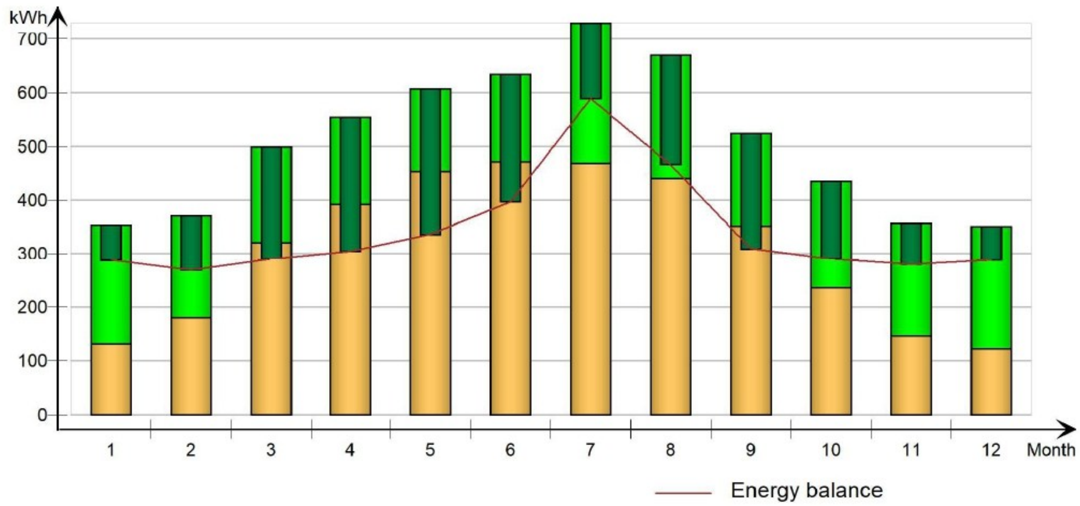
	Total		**Total primary energy**		**Non-renewable primary energy**		**CO2 Emission**	
	kWh	**kWh/m²**	**kWh**	**kWh/m²**	**kWh**	**kWh/m²**	**kg**	**kg/m²**
Produced by PV	3713.9	8.1	3713.9	8.1	0.0	0.0	0.0	0.0
Purchased by facility (el)	2368.8	5.2	5922.0	12.9	5448.2	11.9	994.9	2.2
Exported by facility (el)	-1975.8	-4.3	-4939.4	-10.8	-4544.3	-9.9	-829.8	-1.8
Total Electricity	4106.9	8.9	4696.5	10.2	904.0	2.0	165.1	0.4
Purchased by facility (fuel)	29178.3	63.5	32096.1	69.9	32096.1	69.9	6419.5	14.0
Total Fuel	29178.3	63.5	32096.1	69.9	32096.1	69.9	6419.5	14.0
Overall energy performance			36792.6 (2)	80.1	33000.1 (3)	71.9	6584.6	14.3
RER*			0.103	0.0				
RER on-site**			0.101	0.0				

* [(2) - (3)]/(2)

** Sum(prod*)/(2)

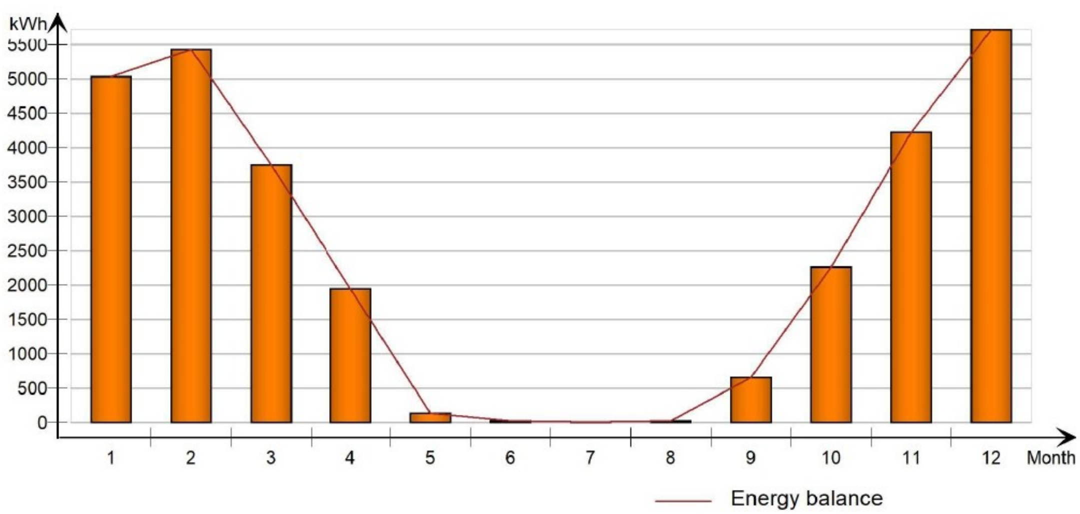
Monthly Energy Electricity

		Total		**Peak demand**	
		kWh	**kWh/m²**	**kW**	**Time**
■	Exported by facility (el)	-1975.8	-4.3	-1.258	05 Feb 13:04
■	Purchased by facility (el)	2368.8	5.2	3.105	04 Aug 13:37
■	Produced by PV	3714.0	8.1	1.551	06 Aug 15:28
	Total Electricity	4106.9	8.9		
	Overall energy performance	4106.9	8.9		



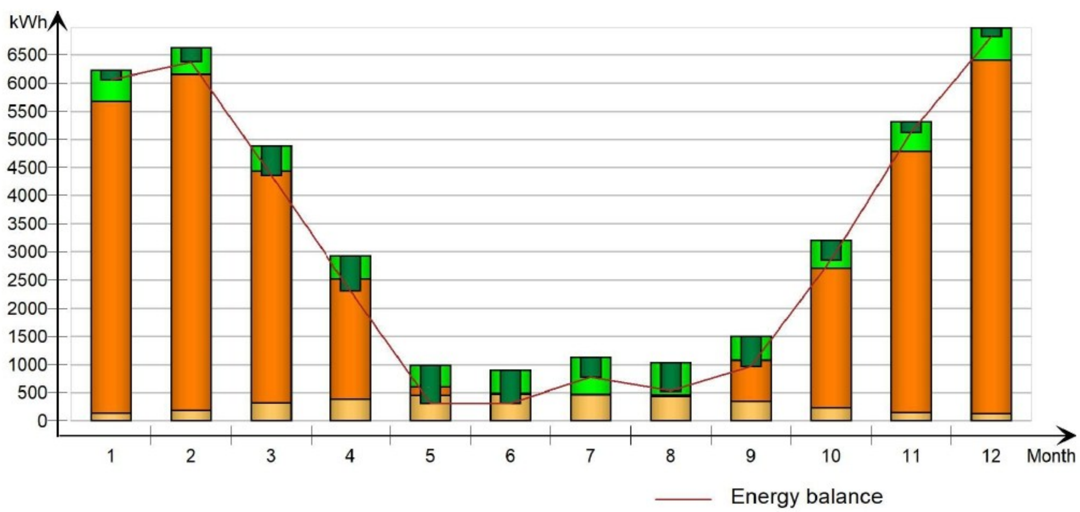
Monthly Energy Fuel

		Total		Peak demand	
		kWh	kWh /m^2	kW	Time
■	Purchased by facility (fuel)	29178.3	63.5	13.93	05 Feb 08:19
	Total Fuel	29178.3	63.5		
	Overall energy performance	29178.3	63.5		



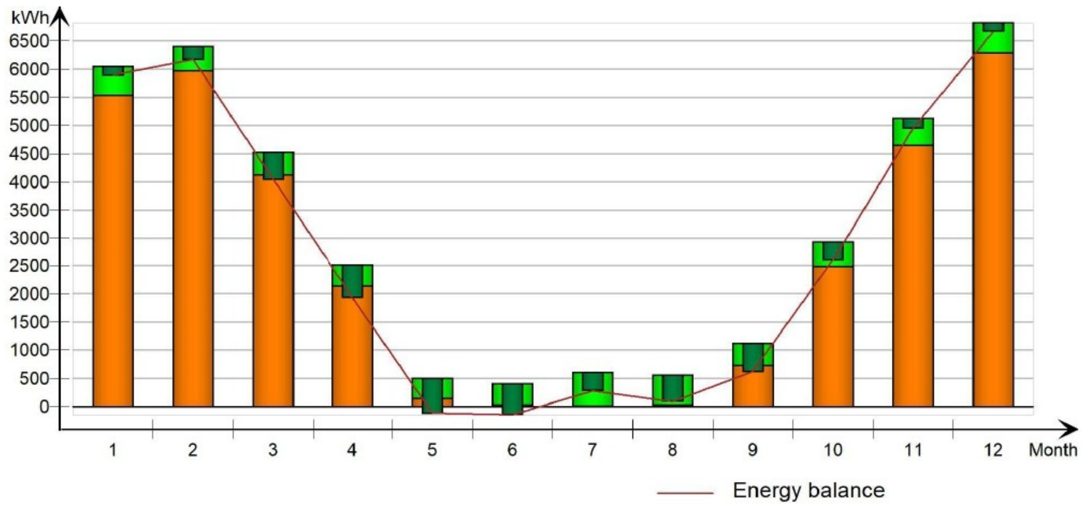
Monthly Total primary energy

		Total	
		kWh	kWh /m²
	Exported by facility (el)	-4939.4	-10.8
	Purchased by facility (el)	5922.0	12.9
	Produced by PV	3714.0	8.1
	Total Electricity	7204.3	15.7
	Purchased by facility (fuel)	32096.1	69.9
	Total Fuel	32096.1	69.9
	Overall energy performance	36792.6	80.1



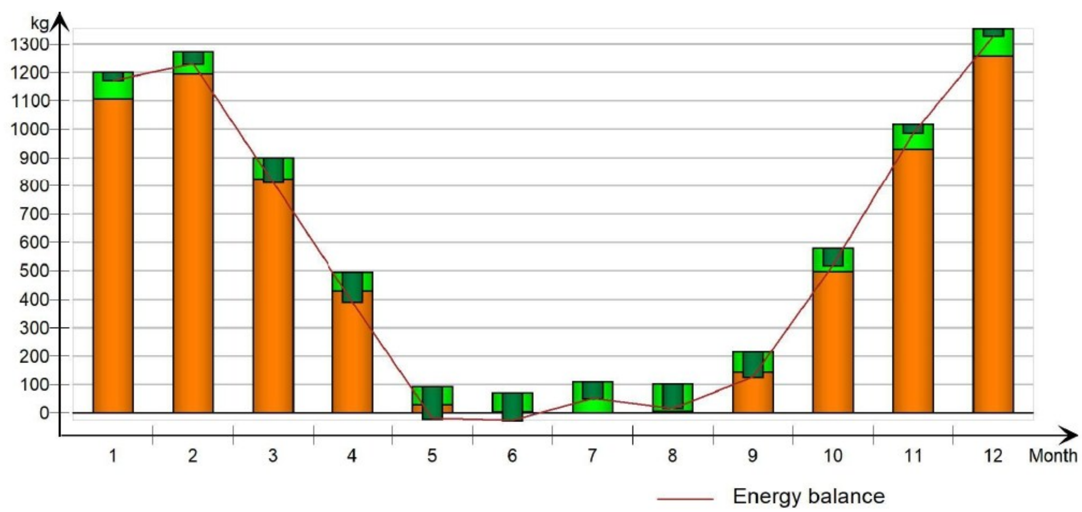
Monthly Non-renewable primary energy

		Total	
		kWh	kWh /m²
	Exported by facility (el)	-4544.3	-9.9
	Purchased by facility (el)	5448.2	11.9
	Total Electricity	3211.1	7.0
	Purchased by facility (fuel)	32096.1	69.9
	Total Fuel	32096.1	69.9
	Overall energy performance	33000.1	71.9

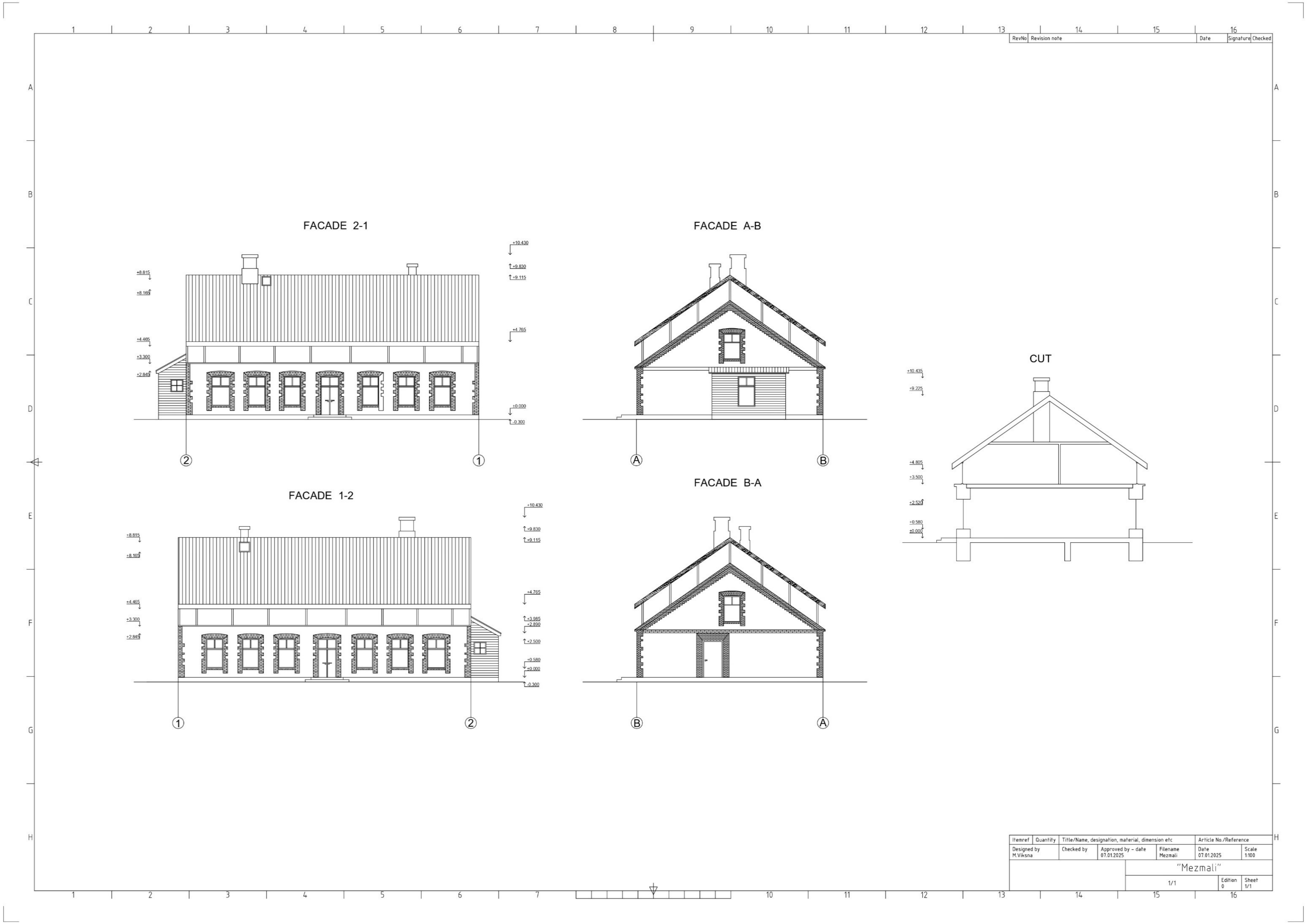


Monthly CO2 Emission

		Total	
		kg	kg /m²
■	Exported by facility (el)	-829.8	-1.8
■	Purchased by facility (el)	994.9	2.2
	Total Electricity	586.4	1.3
■	Purchased by facility (fuel)	6419.5	14.0
	Total Fuel	6419.5	14.0
	Overall energy performance	6584.6	14.3



IDA Indoor Climate and Energy
Version: 5.1
License: IDA40:ICE51X:25MAR/X0E1X (trial license)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FACADE 1-2
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Title/Name, designation, material, dimension etc
Article No./Reference
Designed by
M.Viksna
Checked by
Approved by - date
07.01.2025
Filename
Mezmali
Date
07.01.2025
Scale
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Edition
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Sheet
1/1

RevNo	Revision note	Date	Signature	Checked

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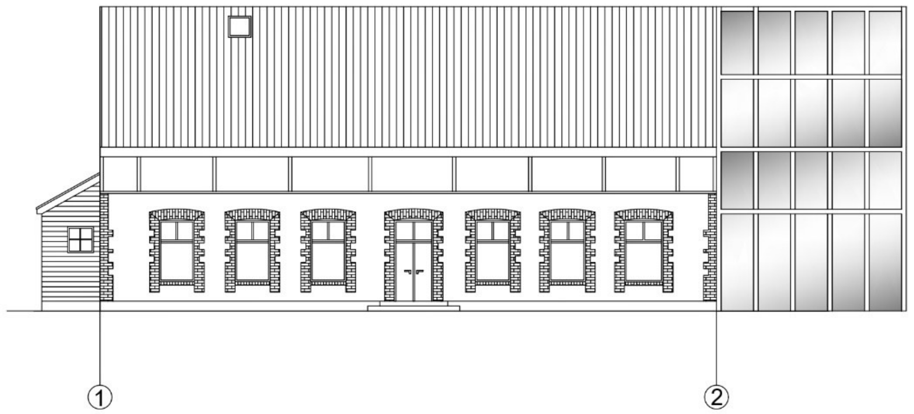

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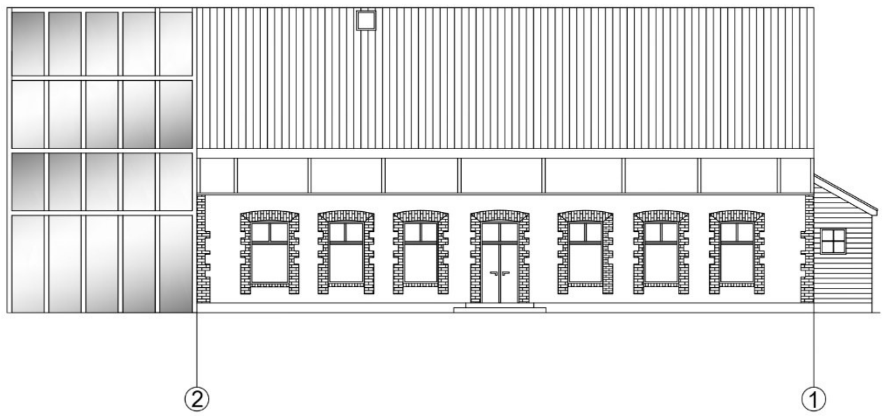

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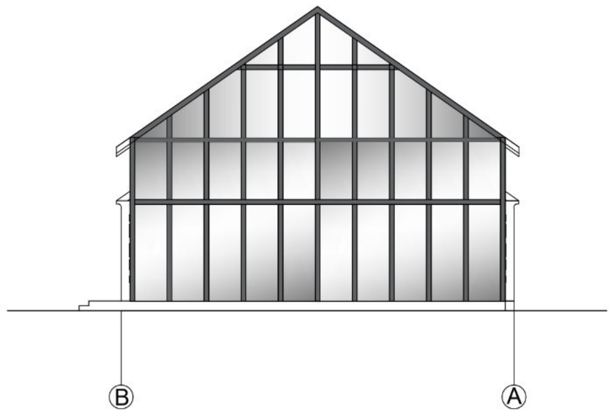

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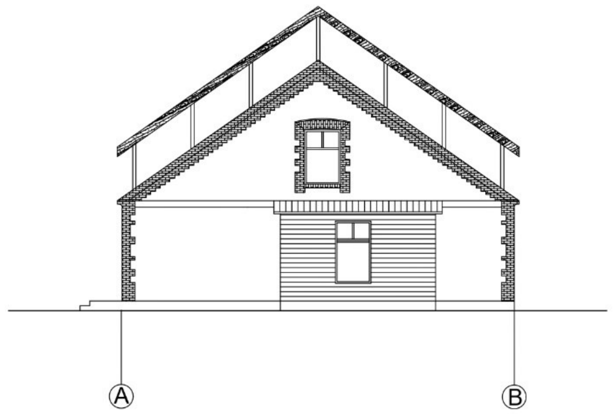

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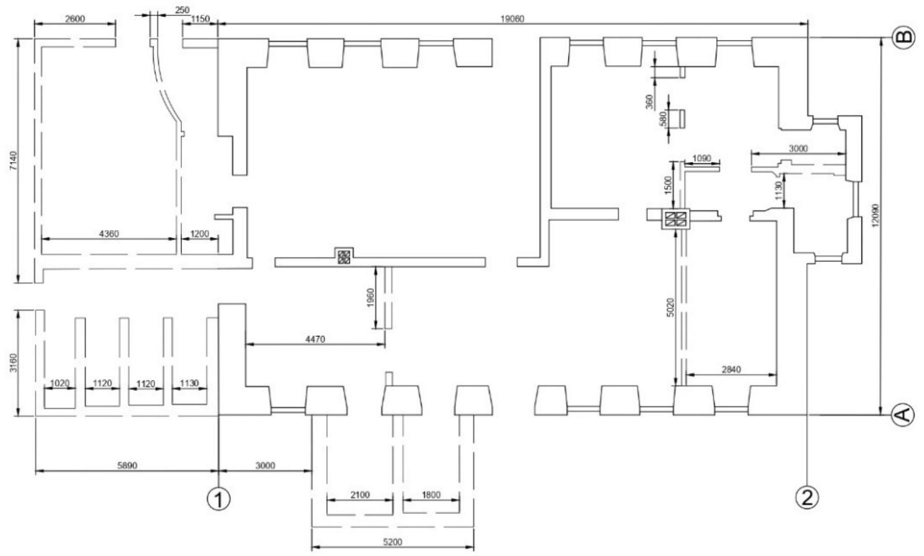
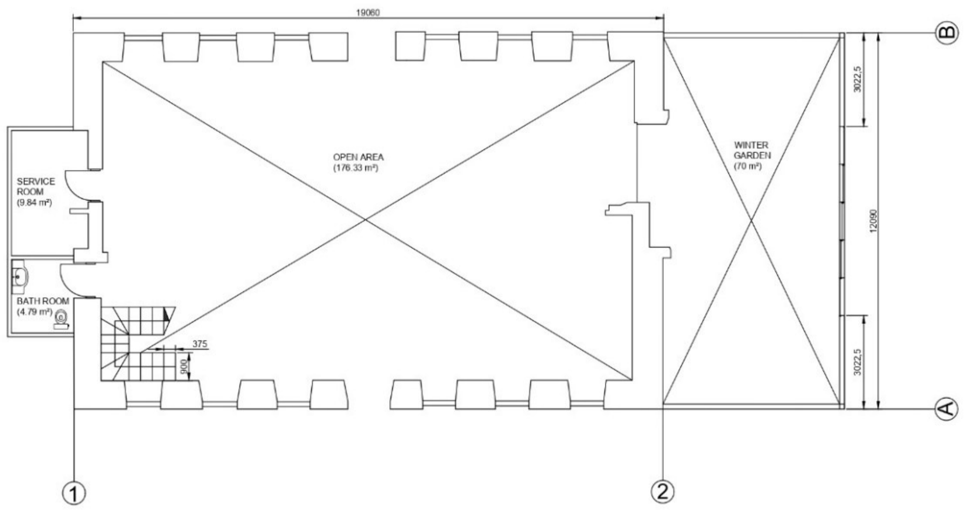
Designed by M.Viksna	Checked by	Approved by - date 07.01.2025	Filename Architecture	Date 07.01.2025	Scale 1:100
		Mezmali			
		1		Edition 1	Sheet 1/1

RevNo	Revision note	Date	Signature	Checked

First Floor



First Floor (after demolition)



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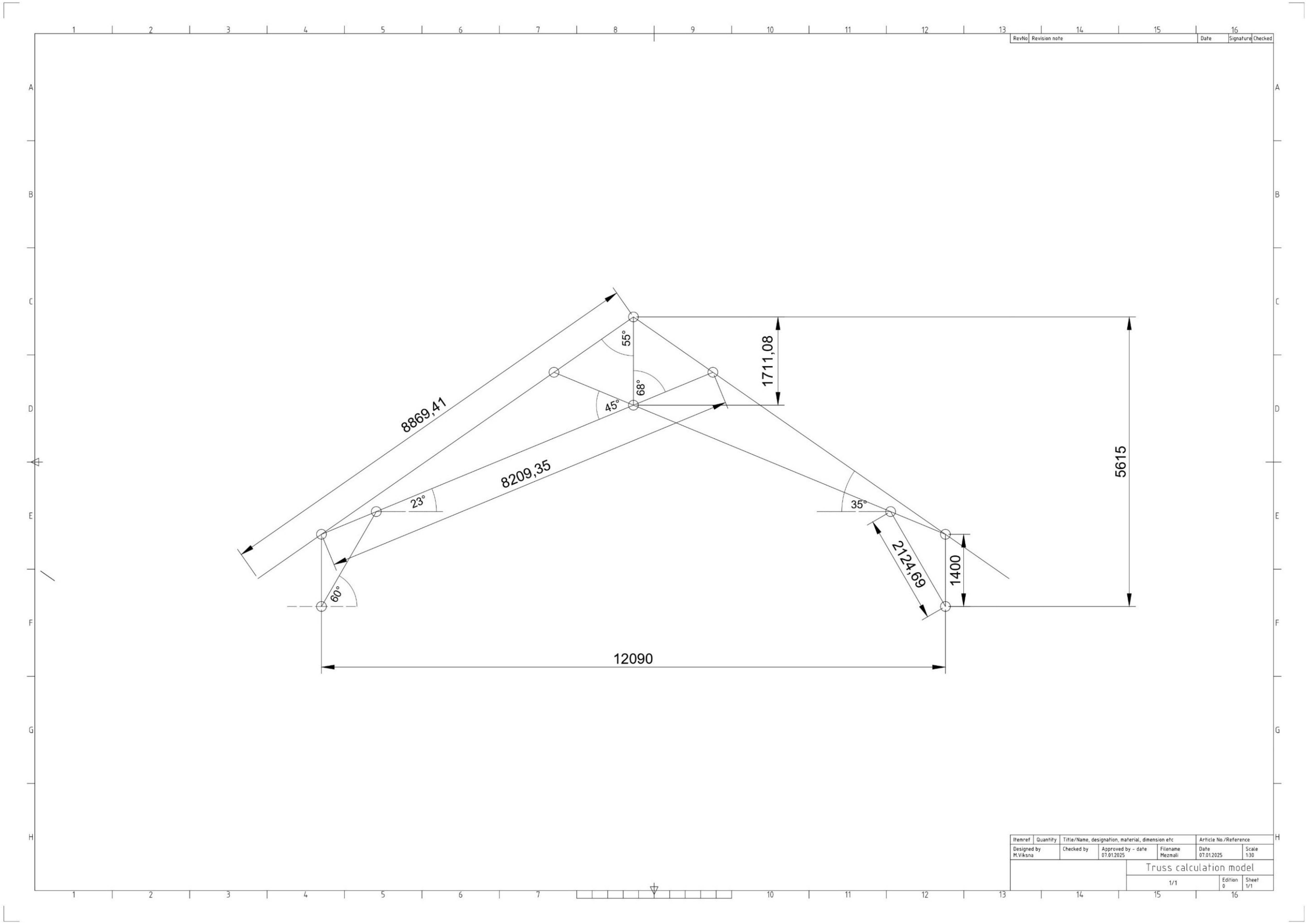
		1/1		Edition 0	Sheet 1/1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Designed by M.Viksna
Checked by
Approved by - date 07.01.2025
Filename Mezmali
Date 07.01.2025
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Truss calculation model
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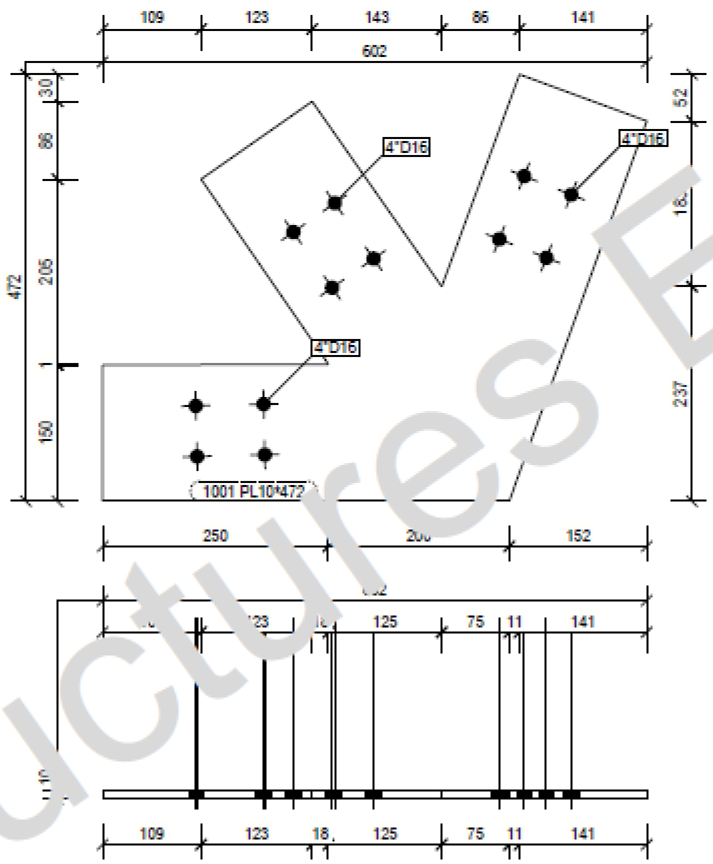
Edition 0
Sheet 1/1

PART LIST FOR ASSEMBLY A-1, MANUFACTURING AMOUNT 1 PIECES

PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIG[illegible]	PCS
1001	PL10*472	S235JR	602	0.3	12.[illegible]	1
			TOTAL:	0.3	12.6	

ASSEMBLY FASTENER LIST

ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	N[illegible]MBER
					TOTAL:			



FABRICATION	WELDS	SURFACE FINISH

DISTRICT/VILLAGE	BLOCK/ESTATE	BUILDING LOT	AUTHORIZATION NOTES	
Mezciems		66600050082		
BUILDING ACTION			DRAWING CATEGORY	ARCHIVING CODE
PROJECT NAME			DRAWING CONTENT	SCALES
"Mezmali"			ASSEMBLY DRAWING A-1, PLATE	1:5
DRAWER	DESIGNER			
M.Viksna	M.Viksna			
CHECKER	ACCEPTOR			
Powered by Trimble Tekla				

PROJECT NUMBER	SUB NUMBER	DWG. NO.	
1		A-1	
DESIGN GROUP	PAGE	DATE	REVISION
STR	1 / 0	07.01.2025	

PART LIST FOR ASSEMBLY A-2, MANUFACTURING AMOUNT 2 PIECES						
PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIGHT [kg]	PCS
1002	PL10*437	S235JR	1171	0.6	21.7	1
			TOTAL:	0.6	21.7	

ASSEMBLY FASTENER LIST								
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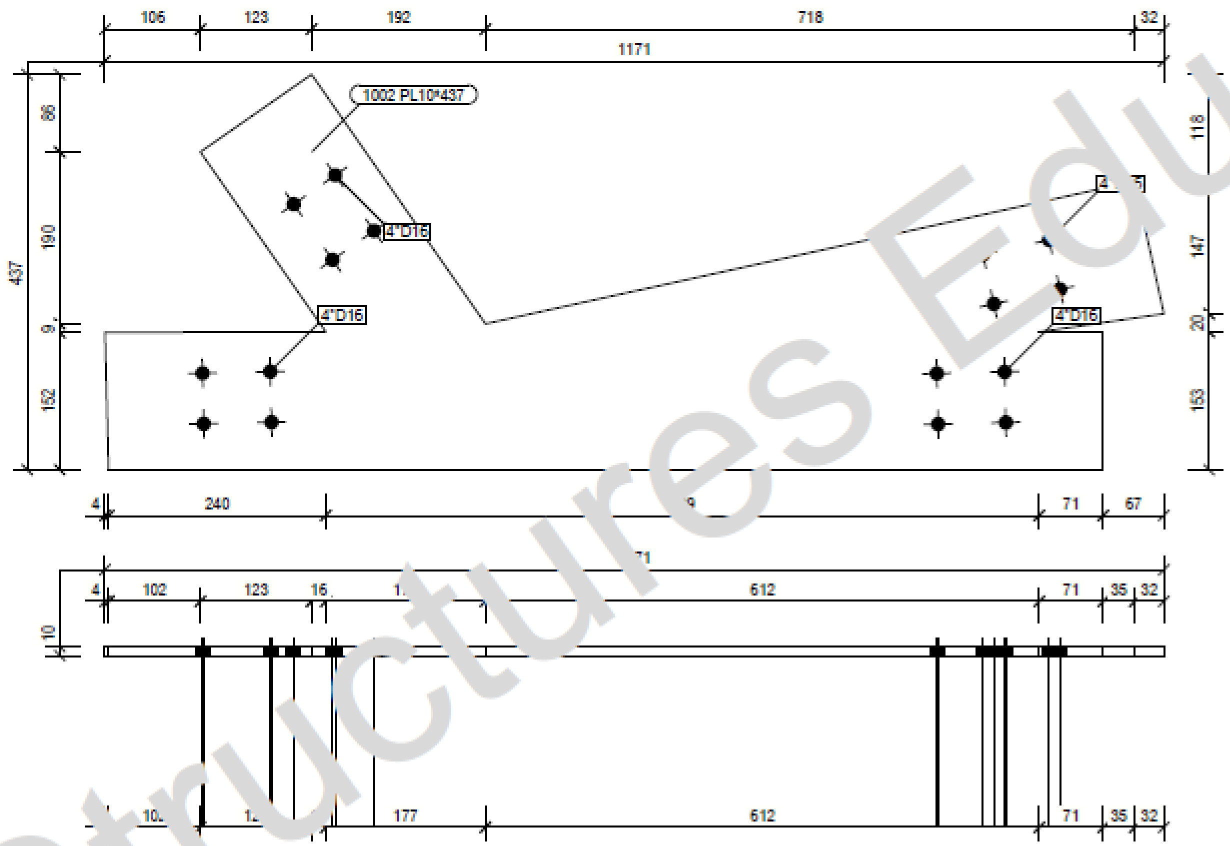
					TOTAL:			



DISTRICT/VILLAGE: Mezciems	BLOCK/ESTATE	BUILDING LOT: 66600050082
BUILDING ACTION		
PROJECT NAME: "Mezmali"		
DRAWER: M.Viksna	DESIGNER: M.Viksna	
CHECKER	ACCEPTOR	

AUTHORIZATION NOTES			
DRAWING CATEGORY			ARCHIVING CODE
DRAWING CONTENT: ASSEMBLY DRAWING A-2, PLATE			SCALES: 1:5
PROJECT NUMBER: 1	SUB NUMBER	DWG. NO.: A-2	
DESIGN GROUP: STR	PAGE: 1 / 0	DATE: 07.01.2025	REVISION

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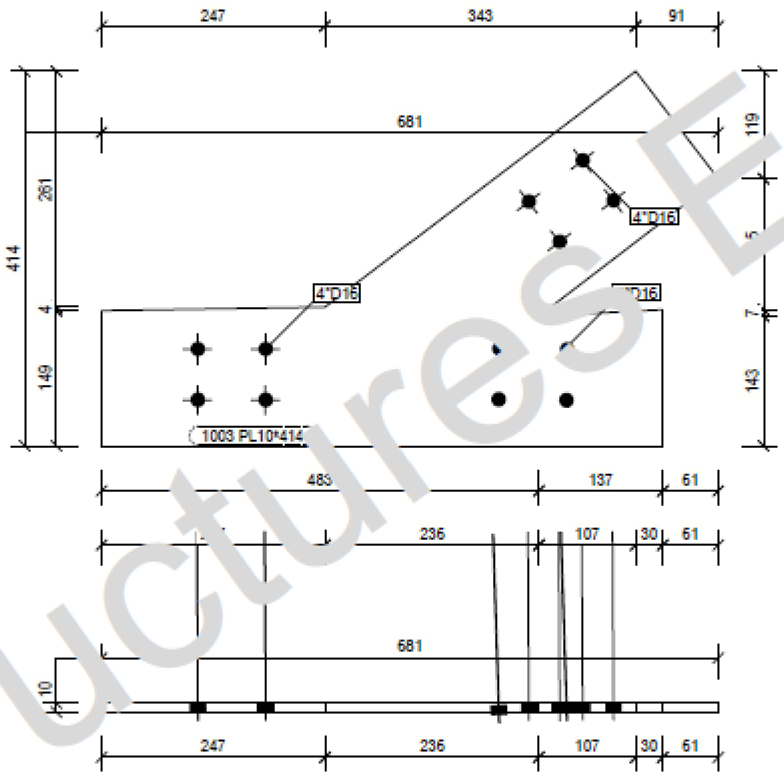
Tekla.

FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY A-3, MANUFACTURING AMOUNT 2 PIECES						
PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIGHT [kg]	PCS
1003	PL10*414	S235JR	681	0.3	11.3	1
			TOTAL:	0.3	11.3	

ASSEMBLY FASTENER LIST								
ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	NUMBER
					TOTAL:			



DISTRICT/VILLAGE: Mezciems	BLOCK/ESTATE	BUILDING LOT: 66600050082
BUILDING ACTION		
PROJECT NAME: "Mezmali"		
DRAWER: M.Viksna	DESIGNER: M.Viksna	
CHECKER	ACCEPTOR	

AUTHORIZATION NOTES			
DRAWING CATEGORY			ARCHIVING CODE
DRAWING CONTENT: ASSEMBLY DRAWING A-3, PLATE			SCALES: 1:5
PROJECT NUMBER: 1	SUB NUMBER	DWG. NO.: A-3	
DESIGN GROUP: STR	PAGE: 1 / 0	DATE: 07.01.2025	REVISION

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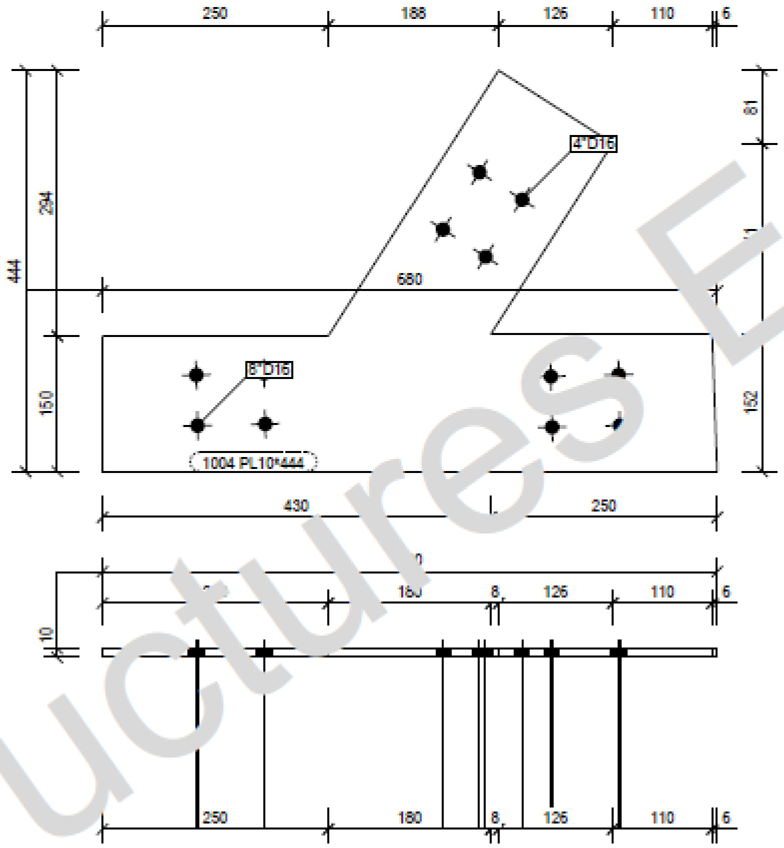
FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY A-4, MANUFACTURING AMOUNT 2 PIECES						
PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIGH[illegible]	PCS
1004	PL10*444	S235JR	680	0.3	11.[illegible]	1
			TOTAL:	0.3	11.6	

ASSEMBLY FASTENER LIST								
ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	NUMBER
					TOTAL:			



DISTRICT/VILLAGE: Mezciems	BLOCK/ESTATE	BUILDING LOT: 66600050082
BUILDING ACTION		
PROJECT NAME: "Mezmali"		
DRAWER: M.Viksna	DESIGNER: M.Viksna	
CHECKER	ACCEPTOR	

AUTHORIZATION NOTES			
DRAWING CATEGORY			ARCHIVING CODE
DRAWING CONTENT: ASSEMBLY DRAWING A-4, PLATE			SCALES: 1:5
PROJECT NUMBER: 1	SUB NUMBER	DWG. NO.: A-4	
DESIGN GROUP: STR	PAGE: 1 / 0	DATE: 07.01.2025	REVISION

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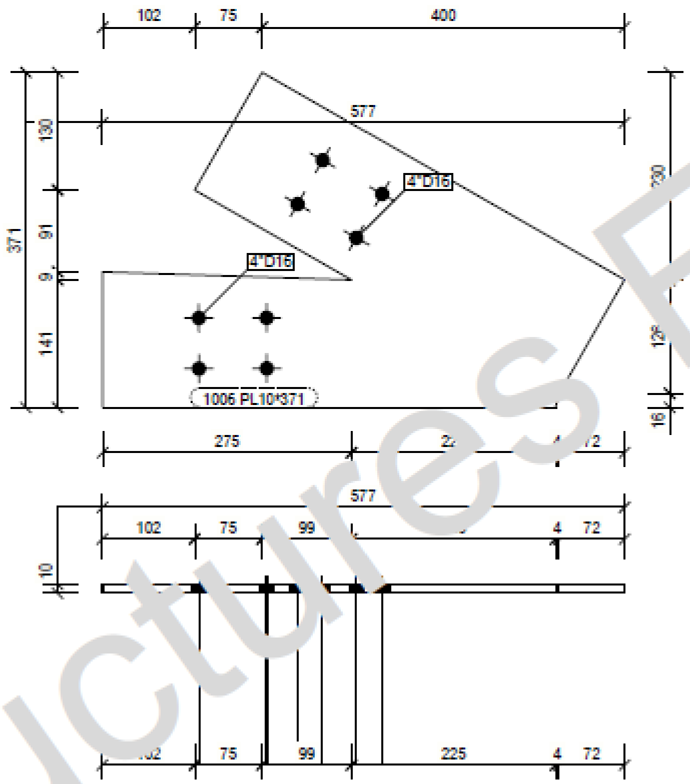
FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY A-6, MANUFACTURING AMOUNT 2 PIECES

PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIGH[illegible]	PCS
1006	PL10*371	S235JR	577	0.3	9.[illegible]	1
			TOTAL:	0.3	9.9	

ASSEMBLY FASTENER LIST

ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	N[illegible]MBER
					TOTAL:			



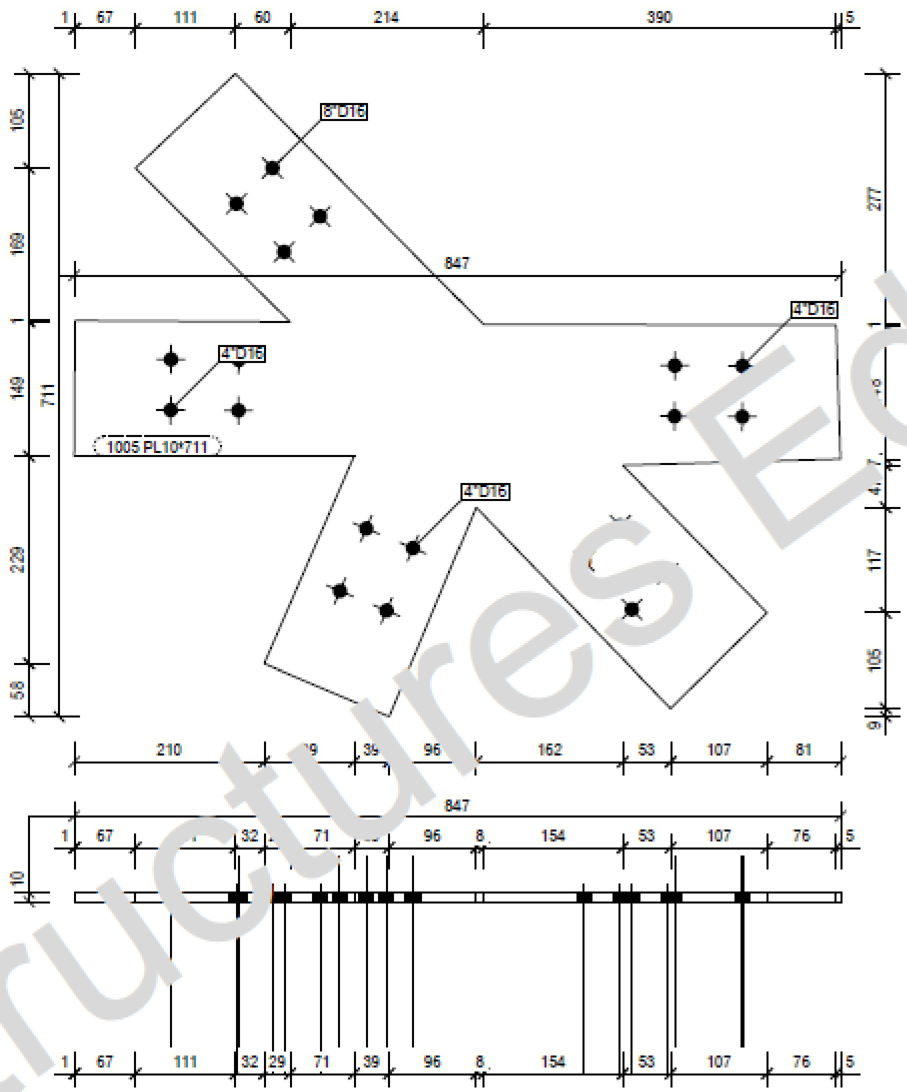
DISTRICT/VILLAGE Mezciems	BLOCK/ESTATE	BUILDING LOT 66600050082
BUILDING ACTION		
PROJECT NAME "Mezmali"		
DRAWER M.Viksna	DESIGNER M.Viksna	
CHECKER	ACCEPTOR	

AUTHORIZATION NOTES		
DRAWING CATEGORY		ARCHIVING CODE
DRAWING CONTENT ASSEMBLY DRAWING A-6, PLATE		SCALES 1:5 1:5
PROJECT NUMBER 1	SUB NUMBER	DWG. NO. A-6
DESIGN GROUP STR	PAGE 1 / 0	DATE 07.01.2025 REVISION

FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY A-16, MANUFACTURING AMOUNT 1 PIECES						
PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIG[illegible]]	PCS
1005	PL10*711	S235JR	847	0.6	20.4	1
			TOTAL:	0.6	20.4	

ASSEMBLY FASTENER LIST								
ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	NUMBER
					TOTAL:			



DISTRICT/VILLAGE	BLOCK/ESTATE	BUILDING/LOT	AUTHORIZATION NOTES	
Mezciems		66600050082		
BUILDING ACTION			DRAWING CATEGORY	ARCHIVING CODE
PROJECT NAME "Mezmali"			DRAWING CONTENT ASSEMBLY DRAWING A-16, PLATE	SCALES 1:5
DRAWER M.Viksna	DESIGNER M.Viksna			
CHECKER	ACCEPTOR			
PROJECT NUMBER 1	SUB NUMBER	DWG. NO. A-16		
DESIGN GROUP STR	PAGE 1 / 0	DATE 07.01.2025	REVISION	

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FABRICATION	WELDS	SURFACE FINISH

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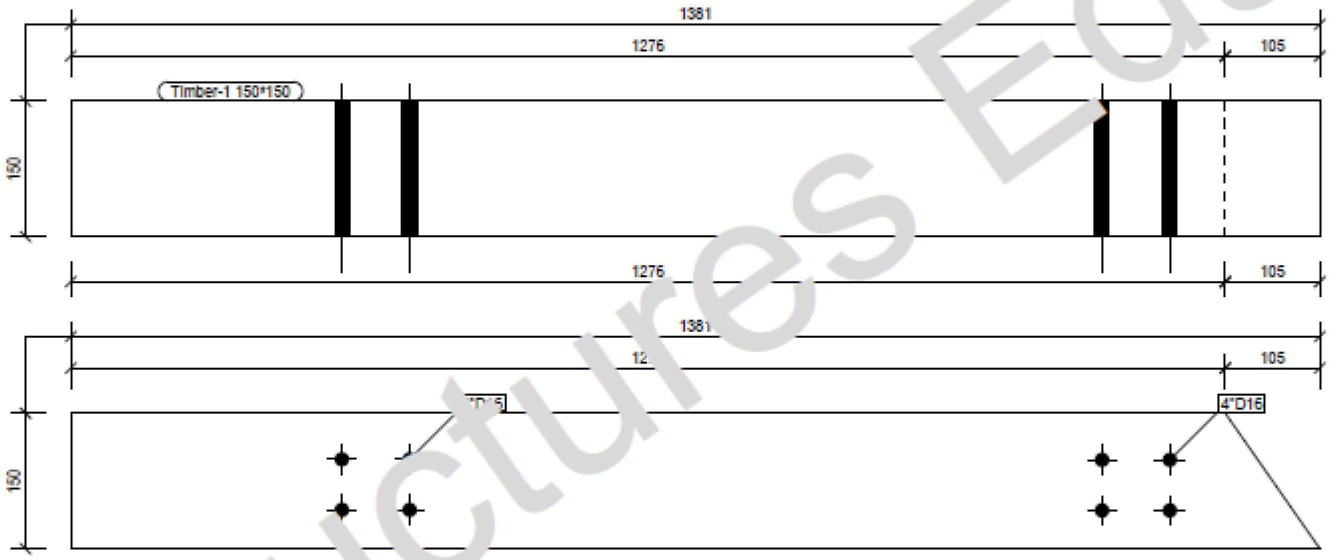
PART LIST FOR ASSEMBLY K-1, MANUFACTURING AMOUNT 1 PIECES						
PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIG[illegible]	PCS
Timber-1	150*150	C24	1381	0.8	12.[illegible]	1
			TOTAL:	0.8	12.6	

ASSEMBLY FASTENER LIST								
ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	N[illegible]MBER
					TOTAL:			



DISTRICT/VILLAGE: Mezciems	BLOCK/ESTATE	BUILDING LOT: 66600050082
BUILDING ACTION		
PROJECT NAME: "Mezmali"		
DRAWER: M.Viksna	DESIGNER: M.Viksna	
CHECKER	ACCEPTOR	

AUTHORIZATION NOTES			
DRAWING CATEGORY			ARCHIVING CODE
DRAWING CONTENT: ASSEMBLY DRAWING K-1, BEAM			SCALES: 1:5
PROJECT NUMBER: 1	SUB NUMBER	DWG. NO.: K-1	
DESIGN GROUP: STR	PAGE: 1 / 0	DATE: 07.01.2025	REVISION

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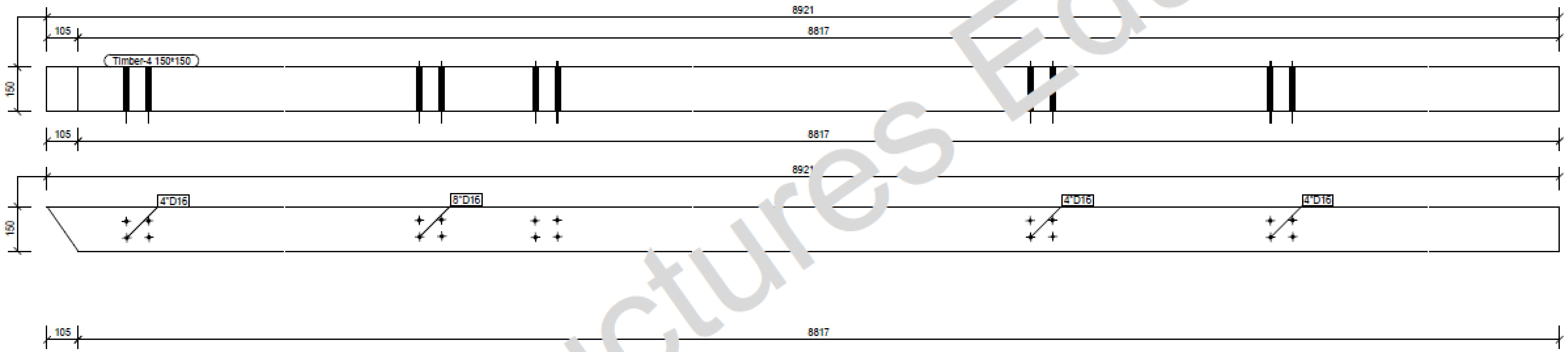
FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY K-5, MANUFACTURING AMOUNT 1 PIECES

PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIG[illegible]	PCS
Timber-4	150*150	C24	8921	5.4	83.[illegible]	1
			TOTAL:	5.4	83.8	

ASSEMBLY FASTENER LIST

ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	NUMBER
					TOTAL:			



DISTRICT/VILLAGE: Mezciems	BUILDING LOT: 66600050082
BUILDING ACTION	AUTHORIZATION NOTES
PROJECT NAME: "Mezmali"	DRAWING CONTENT: ASSEMBLY DRAWING K-5, BEAM
DRAWER: M.Viksna	DESIGNER: M.Viksna
SCALES: 1:10	PROJECT NUMBER: 1
DWG. NO. K-5	DESIGN GROUP: STR
PAGE: 1 / 0	DATE: 07.01.2025

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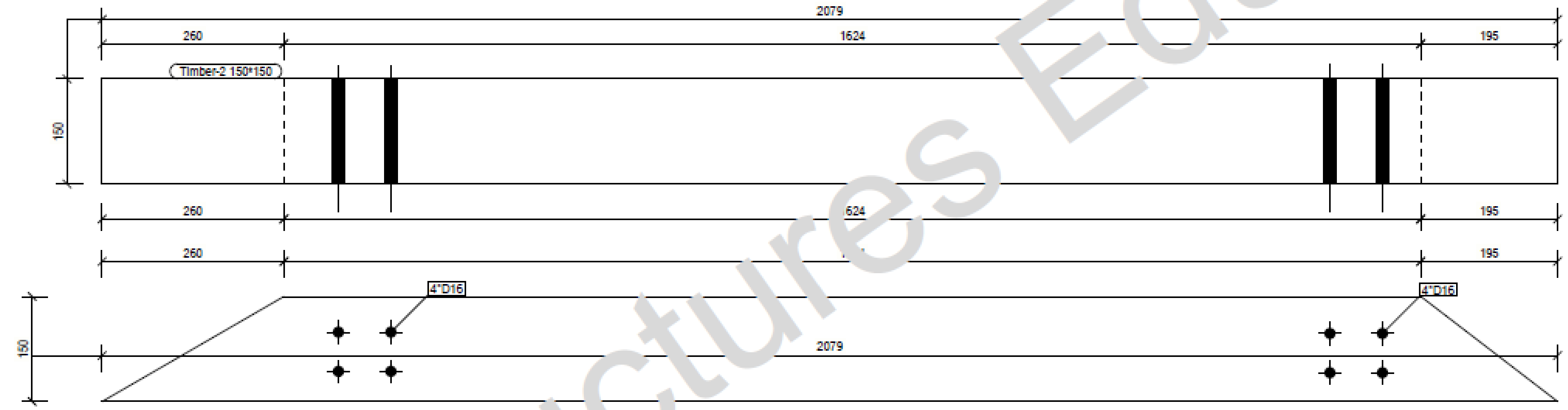
FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY K-3, MANUFACTURING AMOUNT 1 PIECES						
PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIGHT [kg]	PCS
Timber-2	150*150	C24	2079	1.2	17.5	1
			TOTAL:	1.2	17.5	

ASSEMBLY FASTENER LIST								
ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	NUMBER
					TOTAL:			



DISTRICT/VILLAGE Mezciems	BLOCK/ESTATE	BUILDING LOT 66600050082	AUTHORIZATION NOTES	
BUILDING ACTION			DRAWING CATEGORY	ARCHIVING CODE
PROJECT NAME "Mezmali"			DRAWING CONTENT ASSEMBLY DRAWING K-3, BEAM	SCALES 1:5
DRAWER M.Viksna	DESIGNER M.Viksna			
CHECKER	ACCEPTOR			
PROJECT NUMBER 1	SUB NUMBER	DWG. NO. K-3		
DESIGN GROUP STR	PAGE 1 / 0	DATE 07.01.2025	REVISION	

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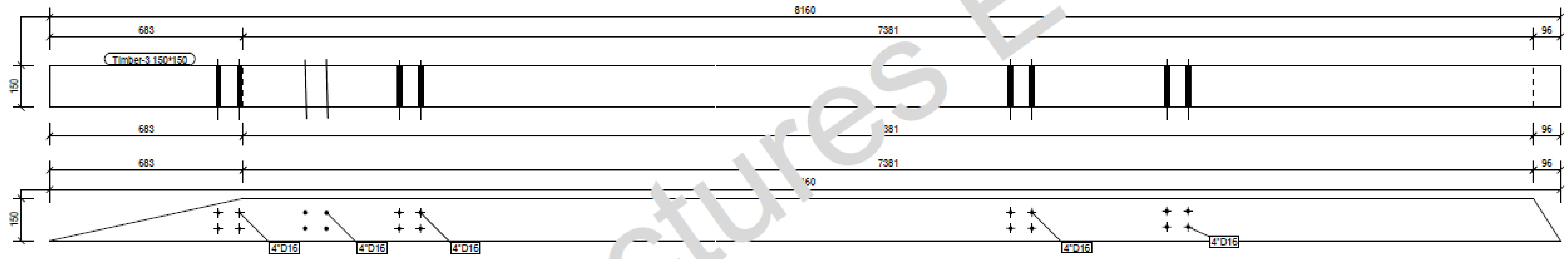
FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY K-4, MANUFACTURING AMOUNT 1 PIECES						
PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIGH[illegible]	PCS
Timber-3	150*150	C24	8160	4.8	73.4	1
			TOTAL:	4.8	73.4	

ASSEMBLY FASTENER LIST								
ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	NUMBER
					TOTAL:			



DISTRICT/VILLAGE Mezciems	BLOCK/ESTATE	BUILDING LOT 66600050082
BUILDING ACTION		
PROJECT NAME "Mezmali"		
DRAWER M.Viksna	DESIGNER M.Viksna	
CHECKER	ACCEPTOR	

AUTHORIZATION NOTES			
DRAWING CATEGORY			ARCHIVING CODE
DRAWING CONTENT ASSEMBLY DRAWING K-4, BEAM			SCALES 1:10
PROJECT NUMBER 1	SUB NUMBER	DWG. NO. K-4	
DESIGN GROUP STR	PAGE 1 / 0	DATE 07.01.2025	REVISION

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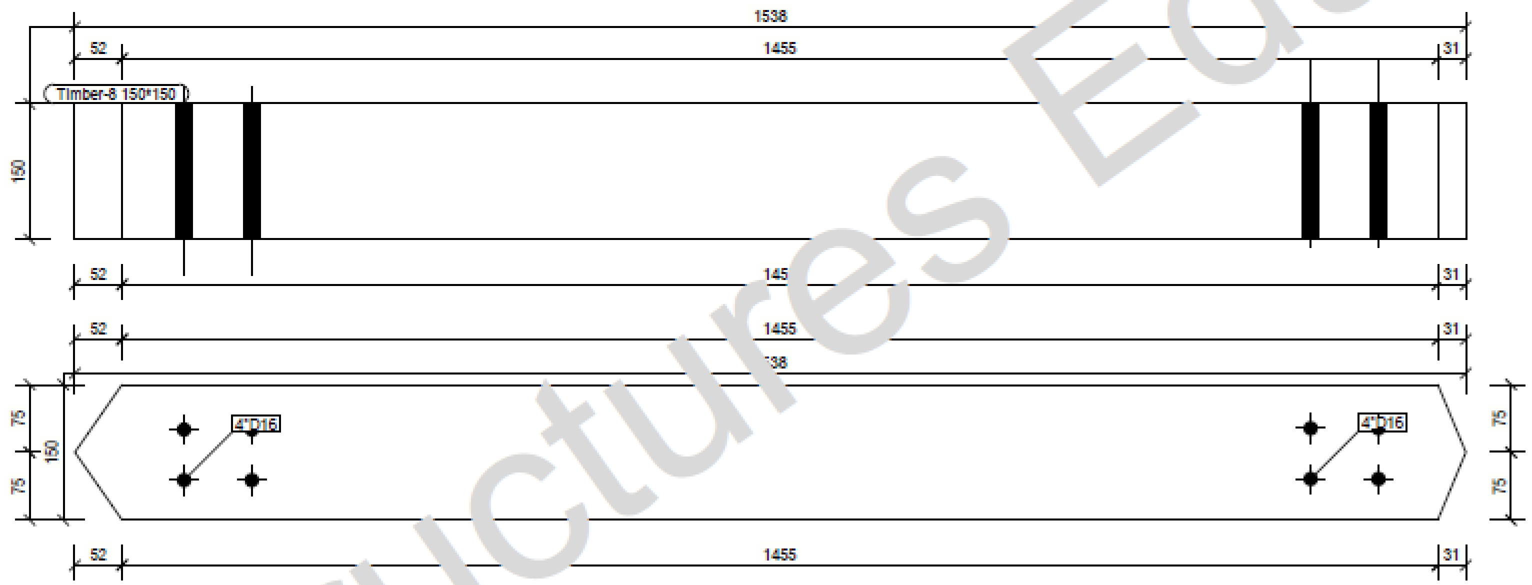
FABRICATION	WELDS	SURFACE FINISH

PART LIST FOR ASSEMBLY K-9, MANUFACTURING AMOUNT 1 PIECES

PART	PROFILE	MATERIAL	LENGTH [mm]	AREA [m2]	WEIG[illegible]	PCS
Timber-8	150*150	C24	1538	0.9	14.[illegible]	1
			TOTAL:	0.9	14.1	

ASSEMBLY FASTENER LIST

ITEM	DIA	SIZE	STANDARD	GRADE	MAT./FINISH	COLOR	[illegible]	NUMBER
					TOTAL:			



DISTRICT/VILLAGE: Mezciems	BLOCK/ESTATE	BUILDING LOT: 66600050082	AUTHORIZATION NOTES
BUILDING ACTION		DRAWING CATEGORY	ARCHIVING CODE
PROJECT NAME: "Mezmali"		DRAWING CONTENT: ASSEMBLY DRAWING K-9, BEAM	SCALES: 1:5
DRAWER: M.Viksna	DESIGNER: M.Viksna		
CHECKER	ACCEPTOR		
PROJECT NUMBER: 1	SUB NUMBER	DWG. NO.: K-9	
DESIGN GROUP: STR	PAGE: 1 / 0	DATE: 07.01.2025	REVISION

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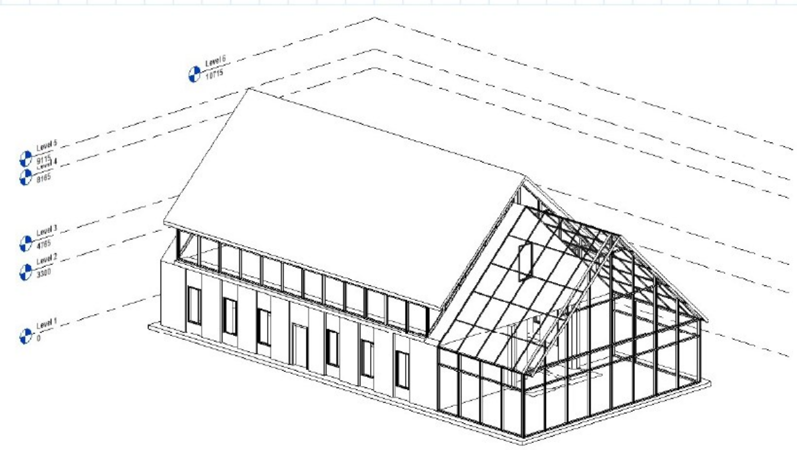
FABRICATION	WELDS	SURFACE FINISH

Appendix 4. Rubble masonry and timber truss calculations

Thesis calculations



Roof wind-load calculations

Terrain factor

$Z := 9.115\ m$ Height of the structure (m)

$\begin{bmatrix} Z_0 \\ Zmin \end{bmatrix} :=$ Terrain categories: 2

Table 4.1 — Terrain categories and terrain parameters

Terrain category		z_0 m	z_{min} m
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10

NOTE: The terrain categories are illustrated in A.1.

A.1 Illustrations of the upper roughness of each terrain category

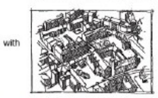
Terrain category 0
Sea, coastal area exposed to the open sea

Terrain category I
Lakes or area with negligible vegetation and without obstacles

Terrain category II
Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights

Terrain category III
Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)

Terrain category IV
Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m



$$Kr := 0.19 \left(\frac{Z_0}{0.05\ m} \right)^{0.07}$$

$Kr = 0.19$ Terrain factor

EN 1991-1-4 ; 4.3.2 Terrain roughness

$Zmax := 200\ m$

z_{max} is to be taken as 200 m

Terrain roughness

$$Cr(Z) := \begin{Vmatrix} \text{if } Zmin \le Z \le Zmax \\ \quad \left\| Kr \cdot \ln\left(\frac{Z}{Z_0}\right) \right. \\ \text{else if } Z \le Zmin \\ \quad \left\| Kr \cdot \ln\left(\frac{Zmin}{Z_0}\right) \right. \end{Vmatrix}$$

$$c_r(z) = k_r \cdot \ln\left(\frac{z}{z_0}\right) \quad \text{for} \quad z_{min} \le z \le z_{max}$$

$$c_r(z) = c_r(z_{min}) \quad \text{for} \quad z \le z_{min}$$

Expression (4.4)

$Cr(Z) = 0.98907$ Terrain roughness

Mean wind velocity

$k_I := 1$ Turbulence factor Check national annex

$C_0 := 1$ Orography factor

$v_b := 24\ \frac{m}{s}$ Basic wind velocity Mby putting picture from where

$v_m(Z) := Cr(Z) \cdot C_0 \cdot v_b$ $v_m(z) = c_r(z) \cdot c_o(z) \cdot v_b$ Expression (4.3)

$v_m(Z) = 23.738\ \frac{m}{s}$ Mean wind velocity

Wind turbulence

$$Iv(Z) := \begin{cases} \text{if } Zmin \le Z \le Zmax & \dfrac{k_I}{C_0 \cdot \ln\left(\dfrac{Z}{Z_0}\right)} \\ \text{else if } Z \le Zmin & \dfrac{k_I}{C_0 \cdot \ln\left(\dfrac{Zmin}{Z_0}\right)} \end{cases}$$

$I_v(z) = \frac{\sigma_v}{v_m(z)} = \frac{k_I}{c_o(z) \cdot \ln(z/z_0)}$ for $z_{min} \le z \le z_{max}$

$I_v(z) = I_v(z_{min})$ for $z < z_{min}$

Expression (4.7)

$Iv(Z) = 0.1921$ The turbulence intensity

Peak velocity pressure

$q_b := 360\ Pa$ Basic velocity pressure

$\rho := 1.25\ \frac{kg}{m^3}$ Air density in the region

$$q_p(Z) := (1 + 7 \cdot Iv(Z)) \cdot \frac{1}{2} \cdot \rho \cdot (v_m(Z))^2$$

$q_p(Z) = 825.745\ Pa$ Peak velocity pressure

$Ce(Z) := \frac{q_p(Z)}{q_b}$ $c_e(z) = \frac{q_p(z)}{q_b}$ Expression (4.9)

$Ce(Z) = 2.294$ Exposure factor

Duopitch roof

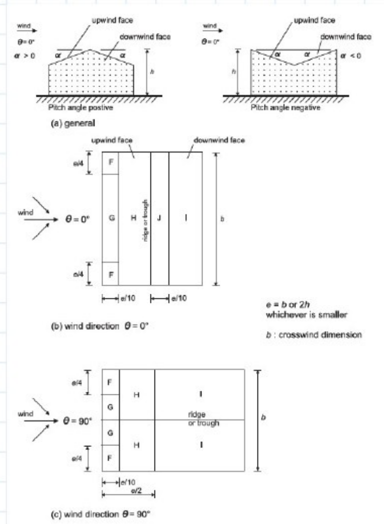


Figure 7.8 — Key for duopitch roofs

Wind 0 direction

$d := 12.09\ m$ Width dimension

$b := 19.06\ m$ Cross-wind dimension

$h := 9.115\ m$ Height of the building

$e := min(b, 2 \cdot h)$

$e = 18.23\ m$

$Direc. := 0$ Wind direction

$\alpha := 33\ °$ Roof slope

Zones for wind in 0 direction

$Fy := \frac{e}{4}$	$Fy = 4.558\ m$	Zone F length in y-direction
$Fx := \frac{e}{10}$	$Fx = 1.823\ m$	Zone F length in x-direction
$Gy := b - 2\left(\frac{e}{4}\right)$	$Gy = 9.945\ m$	Zone G length in y-direction
$Gx := \frac{e}{10}$	$Gx = 1.823\ m$	Zone G length in x-direction
$Hy := b$	$Hy = 19.06\ m$	Zone H length in y-direction
$Hx := \frac{d}{2} - \frac{e}{10}$	$Hx = 4.222\ m$	Zone H length in x-direction
$Jy := b$	$Jy = 19.06\ m$	Zone J length in y-direction
$Jx := \frac{e}{10}$	$Jx = 1.823\ m$	Zone J length in x-direction
$Iy := b$	$Iy = 19.06\ m$	Zone I length in y-direction
$Ix := \frac{d}{2} - \frac{e}{10}$	$Ix = 4.222\ m$	Zone I length in x-direction

Area of zones for wind in 0 direction

Area zone F	Area zone G	Area zone H	Area zone J	Area zone I
$A_F := Fy \cdot Fx$	$A_G := Gy \cdot Gx$	$A_H := Hy \cdot Hx$	$A_J := Jy \cdot Jx$	$A_I := Iy \cdot Ix$
$A_F = 8.308\ m^2$	$A_G = 18.13\ m^2$	$A_H = 80.471\ m^2$	$A_J = 34.746\ m^2$	$A_I = 80.471\ m^2$

External pressure coefficients

Table 7.4a — External pressure coefficients for duopitch roofs

Pitch Angle α	Zone for wind direction $\theta = 0°$									
	F		G		H		I		J	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-0,6		-0,6		-0,8		-0,7		-1,0	-1,5
-30°	-1,1	-2,0	-0,8	-1,5	-0,8		-0,6		-0,8	-1,4
-15°	-2,5	-2,8	-1,3	-2,0	-0,9	-1,2	-0,5		-0,7	-1,2
-5°	-2,3	-2,5	-1,2	-2,0	-0,8	-1,2	+0,2		+0,2	
							-0,6		-0,6	
5°	-1,7	-2,5	-1,2	-2,0	-0,6	-1,2	-0,6		+0,2	
	+0,0		+0,0		+0,0				-0,6	
15°	-0,9	-2,0	-0,8	-1,5	-0,3		-0,4		-1,0	-1,5
	+0,2		+0,2		+0,2		+0,0		+0,0	+0,0
30°	-0,5	-1,5	-0,5	-1,5	-0,2		-0,4		-0,5	
	+0,7		+0,7		+0,4		+0,0		+0,0	
45°	-0,0		-0,0		-0,0		-0,2		-0,3	
	+0,7		+0,7		+0,6		+0,0		+0,0	
60°	+0,7		+0,7		+0,7		-0,2		-0,3	
75°	+0,8		+0,8		+0,8		-0,2		-0,3	

For zones with area of less than 10m^2, we use Cpe,1 . For the rest Cpe,10

$ZoneF_{1a} := 0.7 \quad ZoneG_{1a} := 0.7 \quad ZoneH_{1a} := -0.2 \quad ZoneI_{1a} := -0.4 \quad ZoneJ_{1a} := -0.5$

$$Press.F_{1a} := \begin{array}{l} \text{if } ZoneF_{1a} < 0 \\ \quad ZoneF_{1a} - 0.2 \\ \text{else} \\ \quad ZoneF_{1a} + 0.3 \end{array} \qquad Press.F_{1a} = 1$$

$$Press.G_{1a} := \begin{array}{l} \text{if } ZoneG_{1a} < 0 \\ \quad ZoneG_{1a} - 0.2 \\ \text{else} \\ \quad ZoneG_{1a} + 0.3 \end{array} \qquad Press.G_{1a} = 1$$

$$Press.H_{1a} := \begin{array}{l} \text{if } ZoneH_{1a} < 0 \\ \quad ZoneH_{1a} - 0.2 \\ \text{else} \\ \quad ZoneH_{1a} + 0.3 \end{array} \qquad Press.H_{1a} = -0.4$$

$$Press.J_{1a} := \begin{array}{l} \text{if } ZoneJ_{1a} < 0 \\ \quad ZoneJ_{1a} - 0.2 \\ \text{else} \\ \quad ZoneJ_{1a} + 0.3 \end{array} \qquad Press.J_{1a} = -0.7$$

$$Press.I_{1a} := \begin{array}{l} \text{if } ZoneI_{1a} < 0 \\ \quad ZoneI_{1a} - 0.2 \\ \text{else} \\ \quad ZoneI_{1a} + 0.3 \end{array} \qquad Press.I_{1a} = -0.6$$

Pressure in the each Zone

$$\begin{bmatrix} Press.F_{1a} \cdot q_p(Z) \\ Press.G_{1a} \cdot q_p(Z) \\ Press.H_{1a} \cdot q_p(Z) \\ Press.J_{1a} \cdot q_p(Z) \\ Press.I_{1a} \cdot q_p(Z) \end{bmatrix} = \begin{bmatrix} 825.745 \\ 825.745 \\ -330.298 \\ -578.022 \\ -495.447 \end{bmatrix} Pa$$

Wind 90 direction

$d1 := 19.06\ m$ Width dimension

$b1 := 12.09\ m$ Cross-wind dimension

$h1 := 9.115\ m$ Height of the building

$e1 := min(b, 2 \cdot h)$

$e1 = 18.23\ m$

$Direction.90 := 90$ Wind direction

$\alpha := 35\ °$ Roof slope

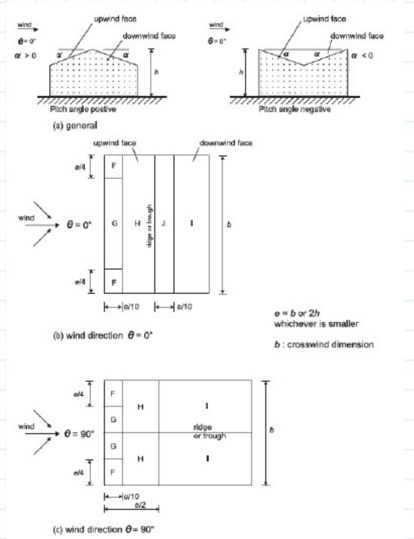


Figure 7.8 — Key for duopitch roofs

Zones for wind in 90 direction

$Fy1 := \frac{e1}{4}$	$Fy1 = 4.558\ m$	Zone F length in y-direction
$Fx1 := \frac{e1}{10}$	$Fx1 = 1.823\ m$	Zone F length in x-direction
$Gy1 := \frac{b1}{2} - \left(\frac{e1}{4}\right)$	$Gy1 = 1.488\ m$	Zone G length in y-direction
$Gx1 := \frac{e1}{10}$	$Gx1 = 1.823\ m$	Zone G length in x-direction
$Hy1 := \frac{b1}{2}$	$Hy1 = 6.045\ m$	Zone H length in y-direction
$Hx1 := \frac{e1}{2} - \frac{e1}{10}$	$Hx1 = 7.292\ m$	Zone H length in x-direction
$Iy1 := \frac{b1}{2}$	$Iy1 = 6.045\ m$	Zone I length in y-direction
$Ix1 := d1 - \frac{e1}{2}$	$Ix1 = 9.945\ m$	Zone I length in x-direction

Area of zones for wind in 90 direction

Area zone F	Area zone G	Area zone H	Area zone I
$A_{F1} := Fy1 \cdot Fx1$	$A_{G1} := Gy1 \cdot Gx1$	$A_{H1} := Hy1 \cdot Hx1$	$A_{I1} := Iy1 \cdot Ix1$
$A_{F1} = 8.308\ m^2$	$A_{G1} = 2.712\ m^2$	$A_{H1} = 44.08\ m^2$	$A_{I1} = 60.118\ m^2$

External pressure coefficients

Table 7.4b — External pressure coefficients for duopitch roofs

Pitch angle α	Zone for wind direction $\theta = 90°$							
	F		G		H		I	
	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
-45°	-1,4	-2,0	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-30°	-1,5	-2,1	-1,2	-2,0	-1,0	-1,3	-0,9	-1,2
-15°	-1,9	-2,5	-1,2	-2,0	-0,8	-1,2	-0,8	-1,2
-5°	-1,8	-2,5	-1,2	-2,0	-0,7	-1,2	-0,6	-1,2
5°	-1,6	-2,2	-1,3	-2,0	-0,7	-1,2	-0,6	
15°	-1,3	-2,0	-1,3	-2,0	-0,6	-1,2	-0,5	
30°	-1,1	-1,5	-1,4	-2,0	-0,8	-1,2	-0,5	
45°	-1,1	-1,5	-1,4	-2,0	-0,9	-1,2	-0,5	
60°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	
75°	-1,1	-1,5	-1,2	-2,0	-0,8	-1,0	-0,5	

For zones with area of less than 10m^2, we use Cpe,1 . For the rest Cpe,10

$ZoneF90_{1a} := -1.1 \quad ZoneG90_{1a} := -2.0 \quad ZoneH90_{1a} := -0.8 \quad ZoneI90_{1a} := -0.5$

$$Press.F90_{1a} := \begin{cases} ZoneF90_{1a} - 0.2 & \text{if } ZoneF90_{1a} < 0 \\ ZoneF90_{1a} + 0.3 & \text{else} \end{cases} \qquad Press.F90_{1a} = -1.3$$

$$Press.G90_{1a} := \begin{cases} ZoneG90_{1a} - 0.2 & \text{if } ZoneG90_{1a} < 0 \\ ZoneG90_{1a} + 0.3 & \text{else} \end{cases} \qquad Press.G90_{1a} = -2.2$$

$$Press.H90_{1a} := \begin{cases} ZoneH90_{1a} - 0.2 & \text{if } ZoneH90_{1a} < 0 \\ ZoneH90_{1a} + 0.3 & \text{else} \end{cases} \qquad Press.H90_{1a} = -1$$

$$Press.I90_{1a} := \begin{cases} ZoneI90_{1a} - 0.2 & \text{if } ZoneI90_{1a} < 0 \\ ZoneI90_{1a} + 0.3 & \text{else} \end{cases} \qquad Press.I90_{1a} = -0.7$$

$$\begin{bmatrix} Press.F90_{1a} \cdot q_p(Z) \\ Press.G90_{1a} \cdot q_p(Z) \\ Press.H90_{1a} \cdot q_p(Z) \\ Press.I90_{1a} \cdot q_p(Z) \end{bmatrix} = \begin{bmatrix} -1.073 \cdot 10^3 \\ -1.817 \cdot 10^3 \\ -825.745 \\ -578.022 \end{bmatrix} Pa$$

Acting pressure in each of the zones

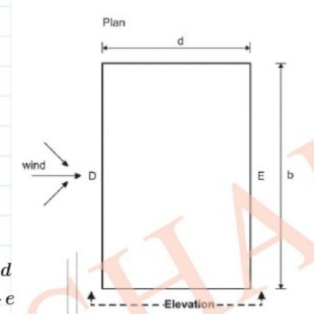
Wind on the walls

$d = 12.09\ m$ Width dimension

$b := 19.06\ m$ Cross-wind dimension

$h = 9.115\ m$ Height of the building

$e := min(b, 2 \cdot h)$ $e = 18.23\ m$



$$A := \begin{cases} \frac{e}{5} & \text{if } e < d \\ \frac{e}{5} & \text{else if } e \ge d \\ d & \text{else if } e \ge 5\ d \end{cases} \qquad B := \begin{cases} \frac{4\ e}{5} & \text{if } e < d \\ d - \frac{e}{5} & \text{else if } e \ge d \\ 0 & \text{else if } e \ge 5\ d \end{cases} \qquad C := \begin{cases} d - e & \text{if } e < d \\ 0 & \text{else if } e \ge d \\ 0 & \text{else if } e \ge 5\ d \end{cases}$$

$D := b$ $E := b$

$\frac{h}{d} = 0.754$

$A = 3.646\ m$ $B = 8.444\ m$ $C = 0$ $D = 19.06\ m$ $E = 19.06\ m$

Area of zones for wind on the wall

Area zone A	Area zone B	Area zone D	Area zone E
$area(A) := A \cdot h$	$area1(B) := B \cdot h$	$area3(D) := b \cdot h$	$area4(E) := b \cdot h$
$area(A) = 33.233\ m^2$	$area(B) = 76.967\ m^2$	$area(D) = 173.732\ m^2$	$area(E) = 173.732\ m^2$

External pressure coefficients

Table 7.1 — Recommended values of external pressure coefficients for vertical walls of rectangular plan buildings

Zone	A		B		C		D		E	
h/d	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$	$c_{pe,10}$	$c_{pe,1}$
5	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,7	
1	-1,2	-1,4	-0,8	-1,1	-0,5		+0,8	+1,0	-0,5	
≤ 0,25	-1,2	-1,4	-0,8	-1,1	-0,5		+0,7	+1,0	-0,3	

$ZoneA := -1.2$ $ZoneB := -0.8$ $ZoneD := 0.8$ $ZoneE := -0.5$

$$\begin{bmatrix} ZoneA \\ ZoneB \\ ZoneD \\ ZoneE \end{bmatrix} = \begin{bmatrix} -1.2 \\ -0.8 \\ 0.8 \\ -0.5 \end{bmatrix}$$

$$Press.A := \begin{array}{l} \text{if } ZoneA < 0 \\ \quad ZoneA - 0.2 \\ \text{else} \\ \quad ZoneA + 0.3 \end{array}$$

$$Press.A = -1.4$$

$$Press.B := \begin{array}{l} \text{if } ZoneB < 0 \\ \quad ZoneB - 0.2 \\ \text{else} \\ \quad ZoneB + 0.3 \end{array}$$

$$Press.B = -1$$

$$Press.D := \begin{array}{l} \text{if } ZoneD < 0 \\ \quad ZoneD - 0.2 \\ \text{else} \\ \quad ZoneD + 0.3 \end{array}$$

$$Press.D = 1.1$$

$$Press.E := \begin{array}{l} \text{if } ZoneE < 0 \\ \quad ZoneE - 0.2 \\ \text{else} \\ \quad ZoneE + 0.3 \end{array}$$

$$Press.E = -0.7$$

$$\begin{bmatrix} Press.A \\ Press.B \\ Press.D \\ Press.E \end{bmatrix} = \begin{bmatrix} -1.4 \\ -1 \\ 1.1 \\ -0.7 \end{bmatrix}$$

$$\begin{bmatrix} Press.A \cdot q_p(Z) \\ Press.B \cdot q_p(Z) \\ Press.D \cdot q_p(Z) \\ Press.E \cdot q_p(Z) \end{bmatrix} = \begin{bmatrix} -1.156 \cdot 10^3 \\ -825.745 \\ 908.32 \\ -578.022 \end{bmatrix} Pa$$

Acting pressure in each of the zones

Snow load

$C_e := 1$ Exposure coefficient

$C_t := 1$ Thermal coefficient

$s_k := 1.25 \cdot \dfrac{kN}{m^2}$ Characteristic value of snow load on the ground

$$\mu_i := 0.8 \cdot \left(\frac{60 - \frac{\alpha}{°}}{30} \right) = 0.667$$

$$s := \mu_i \cdot C_e \cdot C_t \cdot s_k = 0.833 \ \frac{kN}{m^2}$$

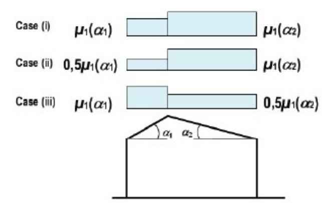


Figure 5.3: Snow load shape coefficients - pitched roofs

Load Combinations

$$\sum_{j \geq 1} \gamma_{G,j} G_{k,j} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i > 1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad (6.10)$$

$G_k := 0.32 \cdot \dfrac{kN}{m^2}$

$q_{k.1} := s = 0.833 \ \dfrac{kN}{m^2}$

$q_{k.2} := Press.F_{1a} \cdot q_p(Z) = 0.826 \ \dfrac{kN}{m^2}$

Ultimate limit state

$1.35 \cdot G_k = 0.432 \ \dfrac{kN}{m^2}$

$1.35 \cdot G_k + 1.5 \cdot q_{k.1} = 1.682 \ \dfrac{kN}{m^2}$

$1.35 \cdot G_k + 1.5 \cdot q_{k.1} + 1.5 \cdot 0.6 \cdot q_{k.2} = 2.425 \ \dfrac{kN}{m^2}$

$1.35 \cdot G_k + 1.5 \cdot q_{k.2} + 1.5 \cdot 0.7 \cdot q_{k.1} = 2.546 \ \dfrac{kN}{m^2}$

Truss calculations

$$Load := 1.35 \cdot G_k + 1.5 \cdot q_{k.2} + 1.5 \cdot 0.7 \cdot q_{k.1}$$

$$F1 := 1\ m \cdot (2.745\ m + 1.5\ m) \cdot Load = 10.806\ kN$$

$$F2 := 1\ m \cdot (2.745\ m + 0.94\ m) \cdot Load = 9.381\ kN$$

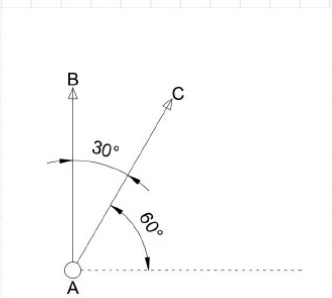
$$F3 := 1\ m \cdot (0.94\ m) \cdot Load = 2.393\ kN$$

Support reactions

$$R1 := \frac{2 \cdot F1 + 2 \cdot F2 + 2\ F3}{2} = 22.58\ kN$$

$$R2 := R1$$

Truss member internal forces



Joint 1

1.

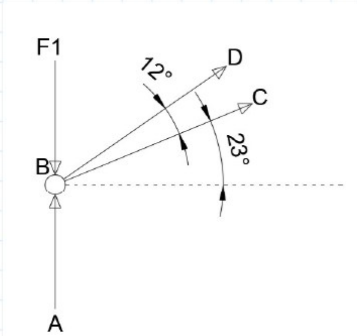
$$F_{x1} := 0 + A_c \cdot \cos(60\ °) = 0 \xrightarrow{solve, A_c} 0$$

$$A_c := 0$$

$$F_{y1} := R1 + A_c \cdot \sin(60\ °) - B_A = 0 \xrightarrow{solve, B_A} 22.579628884219641 \cdot kN$$

$$B_A := 22.58\ kN$$

2.



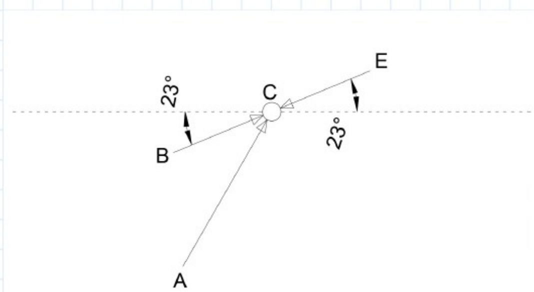
$F_{y2} := -F1 \cdot \cos(35\,°) + B_A \cdot \cos(35\,°) - \boxed{B_C} \cdot \sin(12\,°) = 0$

$$B_C := \frac{-F1 \cdot \cos(34.8869\,°) + B_A \cdot \cos(34.8869\,°)}{\sin(12.387\,°)} = 45.022\ kN$$

$F_{y.2.1} := -F1 + B_A + B_C \cdot sin(23\,°) + \boxed{B_D} \cdot sin(35\,°)$

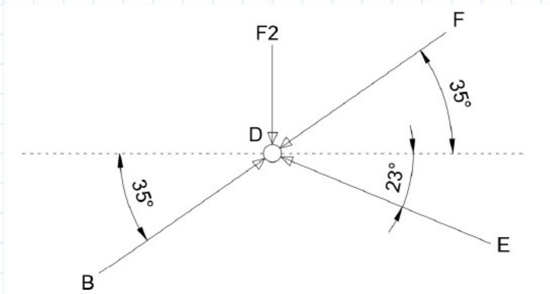
$$B_D := \frac{-F1 + B_A}{\sin(35\,°)} + \frac{B_C \cdot \sin(23\,°)}{\sin(35\,°)} = 51.197\ kN$$

3



$F_{x3} := -B_C \cdot \cos(35\,°) + \boxed{C_E} \cdot cos(35\,°)$

$$C_E := \frac{B_C \cdot \cos(35\,°)}{\cos(35\,°)} = 45.022\ kN$$



$F_{y4} := F2 \cdot cos(35\ °) + \boxed{B_E} \cdot cos(33\ °)$

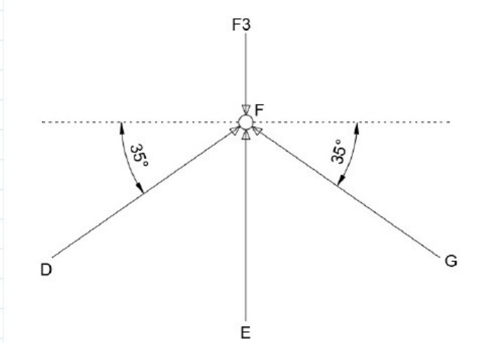
4.

$$D_E := \frac{F2 \cdot \cos(35\ °)}{\cos(33\ °)} = 9.162\ kN$$

$F_{x4} := B_D \cdot cos(35\ °) - \boxed{D_F} \cdot cos(35\ °) - D_E \cdot cos(23\ °)$

$$D_F := \frac{B_D \cdot \cos(35\ °)}{\cos(35\ °)} - \frac{D_E \cdot \cos(23\ °)}{\cos(35\ °)} = 40.901\ kN$$

5.



$G_F := D_F = 40.901\ kN$

$F_{y5} := -F3 + D_F \cdot \sin(35\ °) + G_F \cdot sin(35\ °) + \boxed{F_E}$

$F_E := 2\ F3 - D_F \cdot \sin(35\ °) - G_F \cdot \sin(35\ °) = -42.134\ kN$

Timber calculations

Property	C24	C27	C30	C35	C40
Strength values					
Bending parallel to grain $f_{m,k}$	**24**	27	30	35	40
Tension parallel to grain $f_{t,0,k}$	**14,5**	16,5	19	22,5	26
Tension perpendicular to grain $f_{t,90,k}$	**0,4**	0,4	0,4	0,4	0,4
Compression parallel to grain $f_{c,0,k}$	**21**	22	24	25	27
Compression perpendicular to grain $f_{c,90,k}$	**2,5**	2,5	2,7	2,7	2,8
Shear $f_{v,k}$	**4,0**	4,0	4,0	4,0	4,0
Stiffness value for capacity analysis					
Elastic modulus $E_{0,05}$	**7 400**	7 700	8 000	8 700	9 400
Stiffness values for deformation calculations, mean values					
Elastic modulus parallel to grain $E_{0,mean}$	**11 000**	11 500	12 000	13 000	14 000
Elastic modulus perpendicular to grain $E_{90,mean}$	**370**	380	400	430	470
Shear modulus G_{mean}	**690**	720	750	810	880
Density					
Density ρ_k [1]	**350**	360	380	390	400
Density ρ_{mean} [1]	**420**	430	460	470	480

$f_{m.k} := 24 \cdot MPa$ Bending parallel to grain

$f_{t.0.k} := 14.5\ MPa$ Tension parallel to grain

$f_{c.0.k} := 21\ MPa$ Compression parallel to grain

$f_{v.k} := 4\ MPa$ Shear

$E_{0.05} := 7400\ MPa$ Elastic modulus

$\gamma_M := 1.3$ Partial coefficient for material

Strength modification factor

Material	Associated material standard	Service class	Load duration class				
			P	L	M	S	I
Structural timber	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
		2	0,60	0,70	0,80	0,90	1,10
		3	0,50	0,55	0,65	0,70	0,90

$k_{mod} := 0.9$

Rafter design check

General information

$b_s := 150\ mm$ Width of the section

$h_s := 150\ mm$ Height of the section

$A_{comp.} := h_s \cdot b_s$

Compression

$B_D = 51.197\ kN$ Maximum compression force

$k_{sys} := 1$

$$f_{c.0.d} := f_{c.0.k} \cdot \frac{k_{mod}}{\gamma_M} \cdot k_{sys} = 14.538\ MPa$$

$$\sigma_{c.0.d} := \frac{B_D}{A_{comp.}} = 2.275\ MPa$$

$$UR.comp := \text{if } \frac{\sigma_{c.0.d}}{f_{c.0.d}} \le 1 \;\; \| \text{“Ok”} \;\; \text{else} \;\; \| \text{“Not Ok”}$$

$UR.comp = \text{“Ok”}$

$$\frac{\sigma_{c.0.d}}{f_{c.0.d}} = 0.157$$

Shear

$f_{v.d} := f_{v.k} \cdot \frac{k_{mod}}{\gamma_M} \cdot k_{sys} = 2.769\ MPa$ Design shear strength

$V_{ed} := 6.247\ kN$ Maximum shear force

$\tau_d := \frac{3 \cdot V_{ed}}{2 \cdot h_s \cdot b_s} = 0.416\ MPa$ Shear stress

$UR.shear := \text{if } \frac{\tau_d}{f_{v.d}} \le 1 \ \| \text{"Ok"} \ \text{else} \ \| \text{"Not Ok"}$

$UR.shear = \text{"Ok"}$ $\frac{\tau_d}{f_{v.d}} = 0.15$ Shear utility ratio

Bending

$M_{ULS} := 5.24\ kN \cdot m$ Maximum bending force

$k_h := min\left(\left(\frac{150}{\frac{h_s}{mm}}\right)^{0.2}, 1.3\right) = 1$ Size effect

$W_y := \frac{b_s \cdot h_s^2}{6} = (5.625 \cdot 10^5)\ mm^3$ Section modulus

$f_{m.d} := f_{m.k} \cdot \frac{k_{mod}}{\gamma_M} \cdot k_h \cdot k_{sys} = 16.615\ MPa$ Design bending strength

$\sigma_{m.d} := \frac{M_{ULS}}{W_y} = 9.316\ MPa$ Bending stress

$UR.bending := \text{if } \frac{\sigma_{m.d}}{f_{m.d}} \le 1 \ \| \text{"Ok"} \ \text{else} \ \| \text{"Not Ok"}$

$UR.bending = \text{"Ok"}$ $\frac{\sigma_{m.d}}{f_{m.d}} = 0.561$ Bending utility ratio

Combined bending and axial compression

$UR.b.c.1 := \left(\frac{\sigma_{c.0.d}}{f_{c.0.d}}\right)^2 + 0.7 \cdot \frac{\sigma_{m.d}}{f_{m.d}} = 0.417$

$UR.b.c.2 := \left(\frac{\sigma_{c.0.d}}{f_{c.0.d}}\right)^2 + \frac{\sigma_{m.d}}{f_{m.d}} = 0.585$

$UR.b.c := \text{if } UR.b.c.1 \le 1 \ \| \text{"Ok"} \ \text{else} \ \| \text{"Not Ok"}$ $UR.b.c = \text{"Ok"}$

$UR.b.c.' := \text{if } UR.b.c.2 \le 1 \ \| \text{"Ok"} \ \text{else} \ \| \text{"Not Ok"}$ $UR.b.c.' = \text{"Ok"}$

Timber column buckling

$k := 1$

$L := 2.746\ m$ Lenght of the section

$\beta_c := 0.2$

$L_{e.y} := k \cdot L$ Effective length

$$\lambda_y := \frac{L_{e.y}}{\frac{h_s}{\sqrt{12}}} = 63.416$$ Slenderness ratio

$$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.075$$ Relative slenderness ratio

$\sigma_{c.0.d} = 2.275\ MPa$ Compression design stress

$$k_y := 0.5\left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + \lambda_{rel.y}^{\ 2}\right) = 1.156$$

$$k_{c.y} := \frac{1}{k_y + \sqrt{k_y^{\ 2} - \lambda_{rel.y}^{\ 2}}} = 0.633$$ Instability factor

$$UR.buck := \text{if } \frac{\sigma_{c.0.d}}{f_{c.0.d} \cdot k_{c.y}} \le 1 \ \ \|\ \text{“Ok”} \ \ \text{else} \ \ \|\ \text{“Not Ok”}$$

$UR.buck = \text{“Ok”}$

$$\frac{\sigma_{c.0.d}}{f_{c.0.d} \cdot k_{c.y}} = 0.247$$ Buckling utility ratio

Combined bending and axial compression buckling

$$UR.b.c.b.1 := \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + 0.7 \cdot \frac{\sigma_{m.d}}{f_{m.d}} = 0.64$$

$$UR.b.c.b.2 := \frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.d}}{f_{m.d}} = 0.808$$

$$UR.b.c.b := \text{if } UR.b.c.b.1 \le 1 \ \ \|\ \text{“Ok”} \ \ \text{else} \ \ \|\ \text{“Not Ok”}$$

$UR.b.c.b = \text{“Ok”}$

$$UR.b.c.b. := \text{if } UR.b.c.b.1 \le 1 \ \ \|\ \text{“Ok”} \ \ \text{else} \ \ \|\ \text{“Not Ok”}$$

$UR.b.c.b. = \text{“Ok”}$

Bottom chord

Same section

$h_s = 150\ mm$

$b_s = 150\ mm$

$A_{tens} := h_s \cdot b_s = \left(2.25 \cdot 10^4\right)\ mm^2$

Tension

$C_E = 45.022\ kN$ Maximum tension force

$f_{t.0.d} := f_{t.0.k} \cdot \dfrac{k_{mod}}{\gamma_M} \cdot k_{sys} \cdot k_h = 10.038\ MPa$ Design tension strength

$\sigma_{t.0.d} := \dfrac{C_E}{A_{tens}} = 2.001\ MPa$ Design tension stress

$$UR.t := \text{if } \frac{\sigma_{t.0.d}}{f_{t.0.d}} \le 1 \;\; \| \text{"Ok"} \;\; \text{else} \;\; \| \text{"Not Ok"}$$

$UR.t = \text{"Ok"}$ $\dfrac{\sigma_{t.0.d}}{f_{t.0.d}} = 0.199$

Vertical support member

Same section

$h_s = 150\ mm$

$b_s = 150\ mm$

$A_{comp} := h_s \cdot b_s = \left(2.25 \cdot 10^4\right)\ mm^2$

Compression

$B_A = 22.58\ kN$ Maximum compression force

$k_{sys} = 1$

$f_{c.0.d} = 14.538\ MPa$

$\sigma_{c.0.d.} := \dfrac{B_A}{A_{comp}} = 1.004\ MPa$

$$UR.comp.v := \text{if } \frac{\sigma_{c.0.d.}}{f_{c.0.d}} \le 1 \;\; \| \text{"Ok"} \;\; \text{else} \;\; \| \text{"Not Ok"}$$

$UR.comp = \text{"Ok"}$ $\dfrac{\sigma_{c.0.d}}{f_{c.0.d}} = 0.157$

Timber column buckling

$k = 1$

$L1 := 1.4\ m$ Lenght of the section

$\beta_c = 0.2$

$L_{e.y.1} := k \cdot L$ Effective length

$\lambda_{y.1} := \dfrac{L_{e.y.1}}{\dfrac{h_s}{\sqrt{12}}} = 63.416$ Slenderness ratio

$\lambda_{rel.y.1} := \dfrac{\lambda_{y.1}}{\pi} \cdot \sqrt{\dfrac{f_{c.0.k}}{E_{0.05}}} = 1.075$ Relative slenderness ratio

$\sigma_{c.0.d.} = 1.004\ MPa$ Compression design stress

$k_{y.1} := 0.5\left(1 + \beta_c \cdot \left(\lambda_{rel.y.1} - 0.3\right) + {\lambda_{rel.y.1}}^2\right) = 1.156$

$k_{c.y.1} := \dfrac{1}{k_{y.1} + \sqrt{{k_{y.1}}^2 - {\lambda_{rel.y.1}}^2}} = 0.633$ Instability factor

$UR.buck.1 :=$ if $\dfrac{\sigma_{c.0.d.}}{f_{c.0.d} \cdot k_{c.y.1}} \le 1$
  "Ok"
else
  "Not Ok"

$UR.buck =$ "Ok" $\dfrac{\sigma_{c.0.d.}}{f_{c.0.d} \cdot k_{c.y.1}} = 0.109$ Buckling utility ratio

Connections

$B_C = 45.022\ kN$ Tension force

$t_s := 8\ mm$ Thickness of the steel plate

$d_b := 14$ Bolt diameter (mm)

$f_u := 360$ Tensile strength of bolt (MPa)

$\rho_k := 350$

Strength parameters

$M_{y.Rk} := 0.3 \cdot f_u \cdot d_b^{2.6} \cdot 1 = 1.031 \cdot 10^5$

The yield moment of the bolt (N*mm)

$\boxed{M_{y.Rk}} := M_{y.Rk} \cdot N \cdot mm = 103.123\ N \cdot m$

$f_{h.0.k} := (0.082 \cdot (1 - 0.01 \cdot d_b)) \cdot \rho_k = 24.682$

Embedding strength(N*mm)

$\boxed{f_{h.0.k}} := f_{h.0.k} \cdot MPa = 24.682\ MPa$

$f_{h.90.k} := \dfrac{(0.082 \cdot (1 - 0.01 \cdot d_b)) \cdot \rho_k}{1.35 + 0.015\ d_b}$

$\boxed{f_{h.90.k}} := f_{h.90.k} \cdot MPa = 15.822\ MPa$

$\boxed{d_b} := d_b \cdot mm = 14\ mm$

$\boxed{f_u} := f_u \cdot MPa = 360\ MPa$

Shear capacity

$t_2 := 150\ mm$ Thickness of the timber middle member

$f_{h.2.k} := f_{h.0.k}$

$F_{v.Rk} := min\left(0.5 \cdot f_{h.2.k} \cdot t_2 \cdot d,\ 1.15 \cdot \sqrt{2 \cdot M_{y.Rk} \cdot f_{h.2.k} \cdot d_b}\right) = 9.708\ kN$ Shear capacity

Withdrawal capacity

$f_{c.90.k} := 2.5\ MPa$

$D. := min\left(12\ t_s, 4 \cdot d_b\right) = 56\ mm$ Circular area (plate replacement)

$A_{s.washer} := \pi \cdot \dfrac{(D.^2 - d_b^2)}{4} = (2.309 \cdot 10^3)\ mm^2$ Area of washer (steel plate)

$F_{ax.washer.Rk} := 3 \cdot f_{c.90.k} \cdot A_{s.washer} = 17.318\ kN$ Widthdrawal capacity of a bolt

$F_{Rk} := F_{v.Rk} + 2 \cdot \dfrac{F_{ax.washer.Rk}}{4} = 18.367\ kN$ Characteristic resistance of one bolt

$F_{Rd} := k_{mod} \cdot \dfrac{F_{Rk}}{\gamma_M} = 12.716\ kN$ Design value of resistance

Connection layout

$$n_{rows} := \left(\frac{t_2 - 2 \cdot 3\ d_b}{4 \cdot d_b}\right) + 1 = 2$$

$$n_{grain} := 2 \cdot n_{ef} \cdot F_{Rd} = B_C \xrightarrow{solve, n_{ef}} 1.7703172378050464896$$

$$n_{rows}^{0.9} \cdot \sqrt[4]{\frac{7 \cdot d_b}{13 \cdot d_b}} = 1.726$$

$$n := min\left(n_{rows}^{0.9} \cdot \sqrt[4]{\frac{7 \cdot d_b}{13 \cdot d_b}}, n_{grain}\right) = 1.726$$

Distance check

Pre-drilling should be made if:
$\rho_k > 500$ kg/m³
$d > 6$ mm

$$\alpha_m := 23\ deg$$

Table 10.4 Minimum values of spacing and end and edge distances for bolts

Spacing and end/edge distances	Angle	Minimum spacing or distance
a_1 (parallel to grain)	$0° \le \alpha \le 360°$	$(4 + \lvert\cos\alpha\rvert)d$
a_2 (perpendicular to grain)	$0° \le \alpha \le 360°$	$4d$
$a_{3,t}$ (loaded end)	$-90° \le \alpha \le 90°$	max(7d; 80mm)
$a_{3,c}$ (unloaded end)	$90° \le \alpha < 150°$	$(1 + 6\sin\alpha)d$
	$150° \le \alpha < 210°$	$4d$
	$210° \le \alpha \le 270°$	$(1 + 6\lvert\sin\alpha\rvert)d$
$a_{4,t}$ (loaded edge)	$0° \le \alpha \le 180°$	max[(2 + 2 sinα)d; 3d]
$a_{4,c}$ (unloaded edge)	$180° \le \alpha \le 360°$	$3d$

$$a_1 := \left(4 + \left|\cos\left(\alpha_m\right)\right|\right) \cdot d_b = 68.887\ mm$$ Distance parallel to grain

$$a_{1.real} := \frac{a_1}{\cos\left(\alpha_m\right)} = 74.836\ mm$$

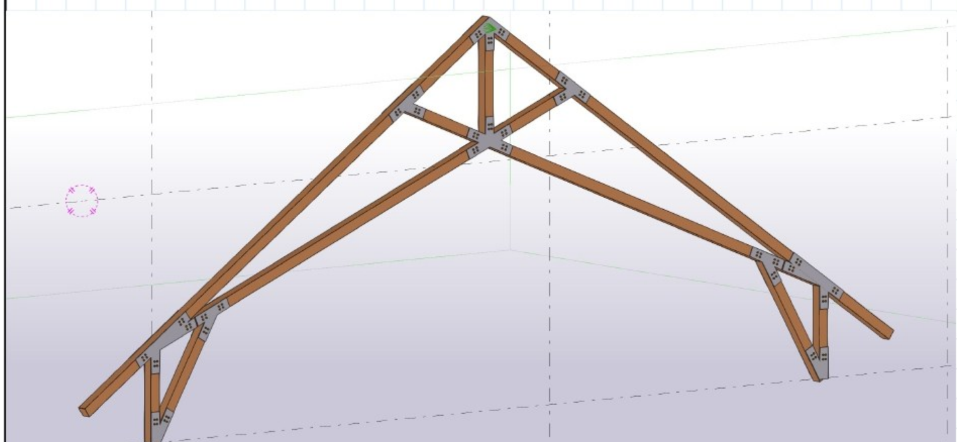
$$a_2 := 4 \cdot d_b = 56\ mm$$ Distance perpendicular to grain

$$a_3 := \max\left(7 \cdot d_b, 80\ mm\right) = 98\ mm$$ Distance from loaded end

$$a_{3.real} := \frac{a_3}{\cos\left(\alpha_m\right)} = 106.463\ mm$$

$$a_4 := 3 \cdot d_b = 42\ mm$$ Distance from unloaded edge

Visual representation of truss element



HCS SLAB CHOICE

$H_{screed} := 50\ mm$ $\qquad L_{slab} := 12.09\ m$

$\rho_{concrete} := 25\ \dfrac{kN}{m^3}$ $\qquad L_{wall} := 19.06\ m$

Loads coming on the slab

$$g_{screed} := H_{screed} \cdot \rho_{concrete} = 1.25\ \frac{kN}{m^2}$$

$$q_l := 2.0\ \frac{kN}{m^2}$$

Load combinations

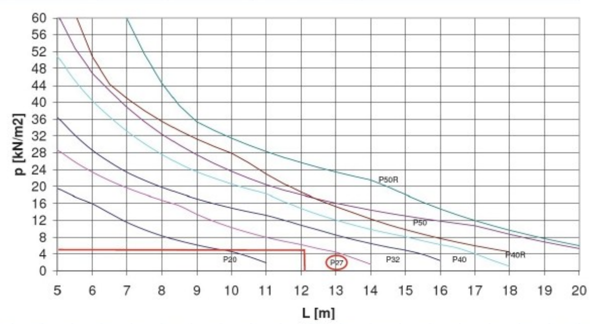
$$ULS_{slab} := \max\left(1.35 \cdot g_{screed}, 1.15 \cdot g_{screed} + 1.5 \cdot q_l\right) = 4.438\ \frac{kN}{m^2}$$

Choosing slab

PARMAN ONTELOLAATTOJEN KANTOKYVYT, KOOSTE

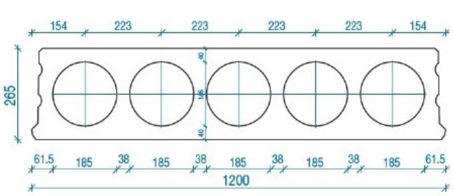
KANTOKYKY ONTELOLAATAT – asunnot, toimistot, lumikuorma (Koonti luokka 0.5)

Betoni C50
Teräs st.1630/1860
Alkujänn. 1000 MN/m²



P27-ONTELOLAATTA

POIKKILEIKKAUS



Loads coming to the wall assembly

$q_l = 2 \frac{kN}{m^2}$

$g_{screed} = 1.25 \frac{kN}{m^2}$

$g_{slab} := 380 \frac{kg}{m^2} \cdot g = 3.727 \frac{kN}{m^2}$

Ultimate limit state combination

$ULS_{wall} := \max\left(1.35 \cdot \left(g_{screed} + g_{slab}\right), 1.15 \cdot \left(g_{screed} + g_{slab}\right) + 1.5 \cdot q_l\right) = 8.723 \frac{kN}{m^2}$

Line load acting on the wall

$w := ULS_{wall} \cdot \frac{L_{slab}}{2} = 52.731 \frac{kN}{m}$

Characteristic compressive strength of the concrete masonry unit (Assumed number)

$f_k := 2.8\ MPa$

Characteristic compressive strength of the natural stone masonry unit (Assumed number)

$f_{k.ns} := f_k = 2.8\ MPa$

Characteristic compressive strength of the Random rubble masonry (Assumed number)

$f_{k.rb} := \frac{f_{k.ns}}{2} = 1.4\ MPa$

The effective height of a wall

$\rho_n := 0.75$

$h_{wall} := 3.115\ m$

$h_{ef} := \rho_n \cdot h_{wall} = 2.336\ m$

The effective thickness of the random rubble masonry

$t := 1\ m$

$t_{ef} := t$

The slenderness ratio

$$UR.s := \text{if } \frac{h_{ef}}{t_{ef}} \le 27 \ \| \ \text{“Ok”} \ \text{else} \ \| \ \text{“Not Ok”}$$

$UR.s = \text{“Ok”}$

Reduction factor for slenderness and eccentricity

$M_{id} := 0\ kN \cdot m$	Design bending moment resulting from the eccentricity
$N_{id} := R1 = 22.58\ kN$	Vertical load acting at the top of wall
$e_{he} := 0$	Eccentricity at the top resulting from the horizontal loads
$e_{init} := \frac{h_{ef}}{450} = 5.192\ mm$	Initial ecentricity
$e_i := \frac{M_{id}}{N_{id}} + e_{he} + e_{init} = 5.192\ mm$	Eccentricity at the top of the wall assembly

$$e_i := \text{if } e_i \ge 0.05\ t \ \| \ e_i \ \text{else} \ \| \ 0.05\ t$$

$\phi_i := 1 - 2 \frac{e_i}{t} = 0.9$	Reduction factor for slenderness and eccentricity

Reduction factor for middle part of the wall assembly

$M_{md} := 0\ kN \cdot m$	Design value of the greatest moment at the middle of the height of the wall
$N_{md} := 1\ kN$	Design value of the vertical force at the middle height of the wall
$e_{hm} := 0$	The eccentricity at mid-height resulting fro horizontal loads
$\varphi := 0.1$	Final creep coefficient
$e_m := \frac{M_{md}}{N_{md}} + e_{hm} + e_{init} = 0.005\ m$	Eccentricity due to the loads

$e_k := 0.002 \cdot \varphi \cdot \dfrac{h_{ef}}{t_{ef}} \cdot \sqrt{t \cdot e_m} = 0.034\ mm$ Eccentricity due to the creep

$e_{mk} := e_m + e_k = 5.225\ mm$ Eccentricity at the middle height of the wall

$$e_{mk} := \begin{array}{l} \text{if } e_{mk} \ge 0.05\ t \\ \quad \| \ e_i \\ \text{else} \\ \quad \| \ 0.05\ t \end{array} = 50\ mm$$

$E_{\cdot wall} := f_{k.rb} \cdot 1000 = 1.4\ GPa$ Modulus of elasticity

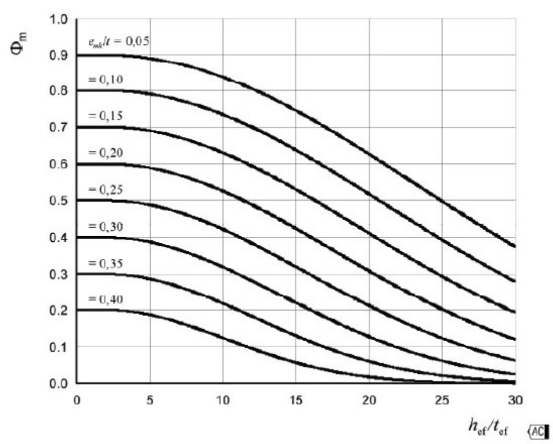


Figure G.1 — Values of Φ_m against slenderness ratio for different eccentricities, based on an E of 1 000 f_k

$\dfrac{e_{mk}}{t} = 0.05$ $\dfrac{h_{ef}}{t_{ef}} = 2.336$

$\phi_m := 0.9$ Reduction factor at the middle height

$\gamma_m := 3$ Masonry safety factor

$N_{Rd.1} := \phi_m \cdot t \cdot \dfrac{f_{k.rb}}{\gamma_m} = 420\ \dfrac{kN}{m}$ Design resistance at the top of the wall

$N_{Rd.2} := \phi_i \cdot t \cdot \dfrac{f_{k.rb}}{\gamma_m} = 420\ \dfrac{kN}{m}$ Design resistance at the middle height

$$N_{rd} := min\left(N_{Rd.2}, N_{Rd.1}\right) = 420\ \frac{kN}{m}$$

$$N_{ed} := w = 52.731\ \frac{kN}{m}$$

$$UR_{wall} := \text{if } \frac{N_{ed}}{N_{rd}} \le 1 \ \Big\| \ \text{"OK"} \ \text{else} \ \Big\| \ \text{"Not OK"}$$

$$UR_{wall} = \text{"OK"} \qquad \frac{N_{ed}}{N_{rd}} = 0.126$$

Serviceability Limit State

$$\frac{h}{t} = 9.115 \qquad \frac{L_{wall}}{t} = 19.06$$

(1) Notwithstanding the ability of a wall to satisfy the ultimate limit state, which must be verified, its size should be limited to that which results from use of figures F.1, F.2 or F.3, depending on the restraint conditions as shown on the figures, where h is the clear height of the wall, l is the length of the wall and t is the thickness of the wall; for cavity walls use t_{ef} in place of t.

(2) Where walls are restrained at the top but not at the ends, h should be limited to 30 t.

(3) This annex is valid when the thickness of the wall, or one leaf of a cavity wall, is not less than 100 mm.