POLITECNICO DI TORINO

Master's Degree in Civil Engineering

Master's Degree Thesis

Study of a lightweight structure in the mountains with two different materials through the Building Information Modelling

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Abstract

The building of high alpine environment facilities is certainly a current topic in a society that has seen in recent years a rapid increase in mountain tourism; in modern times, many interesting different types of new solutions are found and applied for these constructions whose priorities are, for the conditions they are built in, lightness, high resistance, high thermal insulation and also aesthetical fitting with the surrounding environment; for these purposes, the in most cases best satisfying material is wood, which can have different applications and come in different forms; one of them is the Structural Insulated Panel (SIP).

SIPs are, as the name suggests, panels that can have a structural function in a building and, if wood based, their composition consists in two timber sheets and an insulating core. They arise as a composite material potentially able to provide with excellent thermal performances and at the same time additional stiffness to strengthen the whole structure and possibly decrease, or in some cases also completely replace the presence of frame elements. In recent years, innovative types of SIPs with various sorts of wood and different thicknesses of the layers have been developed in order to try to further improve their performances, but on which, due to their young age, not much is known. Especially in Italy, where timber framing and crosslam panels are the most common wood construction systems, little information is present in literature about SIPs and their structural behaviour.

The current thesis aims to do a research about two different Structural Insulated Panel types in order to collect as much information possible about them and then create an accurate digital model of a high mountain building, more precisely a bivouac, employing those panels.

To reach this goal, Building Information Modelling methodology and its software have been used, which have proven to be a very useful tool for the collection of high amounts of intelligent data in one or more programs and then creation of digital models reproducing real-life products.

The building modelled in this thesis in not an existing bivouac but a project, present currently only in the form of drawings which were made and handed to me by the Leap Factory company.

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Introduction

Extreme environments have always been, throughout the history of humankind, a symbol of challenge and exploration, which eventually led, in some cases, to new discoveries and improvements of the society. In the case of building engineering it's no different; the hostile climatic conditions that are to be found when building in glacial or oceanic ambiances present difficulties that often require a deep attention of the single case study, in order to allow the structure to "survive" those conditions.
In this context, in recent years, particular effort has been put in the research of how to optimize structures situated on high altitudes in the mountains, as those territories have seen a more and more increasing anthropization in the past decades and centuries. This is probably due to many factors, such as the growth of interest for this environment for the practice of sports, as a place where to escape to from the usual city life, but also at times to the improved accessibility these locations have because of climate changes (e.g the retreat of glaciers, phenomenon often observable in the Alps).
One type of structure that has its "natural habitat" in these types of surroundings is the bivouac. It is a usually small and lightweight construction, which purpose is originally not to be called "home" by some inhabitants, but to offer shelter for those who need it. Therefore, generally these structures provide only with the bare essential, i.e. walls, roof for protection and a bed. Given the utmost challenging forces of nature to which they are subjected and the remote sites they are located in, bivouacs potentially represent an ideal *"...laboratory, of experimental nature, where to put to test, in an environmental context close to the limits, central issues of modern architecture: prefabrication, new materials, lightness related to stiffness, short times of building sites, but also the relation between the object the and alpine environment,..."* (Dini, Gibello, Girodo, Hoepli ed., 2018, translation by M.K.).
The above-mentioned issues require, for their often-complex nature, new and more effective approaches to study and solve them. One of them is offered, in the modern era, by the BIM (Building Information Modelling) methodology. The Building Information Modelling was developed in recent years, and it includes various tools (most of them consisting in computer files), which can be used and modified by one or more people and then exchanged, extracted or networked in a process of collaboration between professionals of different sectors of the construction industry. It has led to drastic changed in the engineering world, regarding the designing phase but also the construction and maintenance phase. Its basic concept consists in the realization of a digital model of the structure containing information about the characteristics of the latter, which allow it to work as a digital repository of the real building during its entire life cycle.

The aim of this thesis is to analyse a lightweight structure in the mountains, a bivouac, made with two different wood based structural insulated panels, in order to make a structural, energetic and cost based research and then comparison of the two construction systems. The study has been dealt with using the BIM methodology, with the purpose of creating a digital model of the building working with various software, to try to reach a high amount of information within the model and to put to test the interoperability amongst them.
The work is divided in 4 chapters: in the first, there will be a brief illustration of the history and the state of art of the bivouacs, an introduction to BIM, as well as a description of the territorial organization of the case and of the architectonical and structural shape of the structure. The second will display the used construction systems, meaning a detailed illustration of the two kinds of composite materials and the main differences between them, and the definition of the loads acting on the structure. The third chapter exposes the creation of the 3D BIM models of the specific case study, including a suggested solution of a structural model for both panels and a more architectonical model, and will examine the interoperability between the employed software. Finally, the fourth chapter exposes the obtained results regarding the structural, energetic and costs analysis.

Chapter 1 General introduction to bivouacs and BIM, territorial and structural framework of the case study

1.1 Historical background of bivouacs

When speaking about facilities providing shelter in high alpine environments, there are essentially two kinds that can be the taken into account. The most famous and relevant one is probably the mountain hut, which typically is a big structure able to, during summer season, host a relatively large amount of people, who usually are paying clients and get offered a service in the form of beds, served food, running water and a heating system by a staff living and working in the building. The second kind consists in the bivouac, a much smaller and simpler construction that is normally free for everyone and that doesn't provide with any sort of comfort. Bivouacs are generally composed by one or maximum two rooms that can contain tables, chairs, beds and possibly other kinds of supplies that can vary from case to case depending on the function and use of the bivouac.
Those two different types of construction have in common the primary necessity of *"hosting people in a domesticated space, in the least habitable areas of Europe."* (Dini, Luca Gibello, Stefano Girodo, Hoepli ed., 2018, translation by M.K.). They are the result of a relatively recent phenomenon, widespread throughout Europe, that consisted in the "conquest" of the highest peaks of the Alps by alpinists. It started in the late 18th century, (in fact, the first mountain hut, called "Temple de la Nature", was built in 1795 in Montenvers, on the Mont Blanc massif), and reached its peak in the second half of the 19th century, during the so called "golden age" of alpinism between 1854 and 1865, when the alps were considered to be *"the playground of Europe"* ("The playground of Europe", Stephen L., Longmans, Green, and Co., London, 1871).

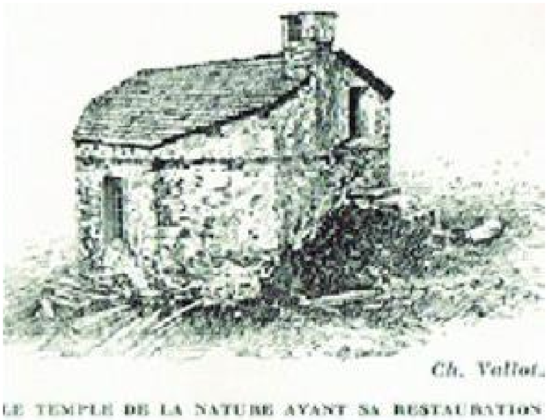

Figure 1.1: Drawing by Charles Vallot, illustrating the "Temple de la Nature" in its original shape.

In these years, in the midst of the mountain enthusiasm caused by these big ascents, also the first alpinism organisations and clubs were born in many countries throughout Europe; in England the Alpine Club was founded London in 1857, followed by similar associations other countries, such as Switzerland, Austria and Italy, where the Club Alpino Italiano (CAI) was founded in 1863. It is thanks to the CAI that, in Italy, a more or less organized construction of a net of bivouacs in the Italian territories of the Alps was conducted in the 1920s-1930s. The designing of most of them was entrusted to the brothers Rivera from Turin, whose typical structure model was, as illustrated in Figure 1.2, of semi-cylindrical form, with a height of 1.25m at the ridge, and 2.4 by 2 m base dimensions.

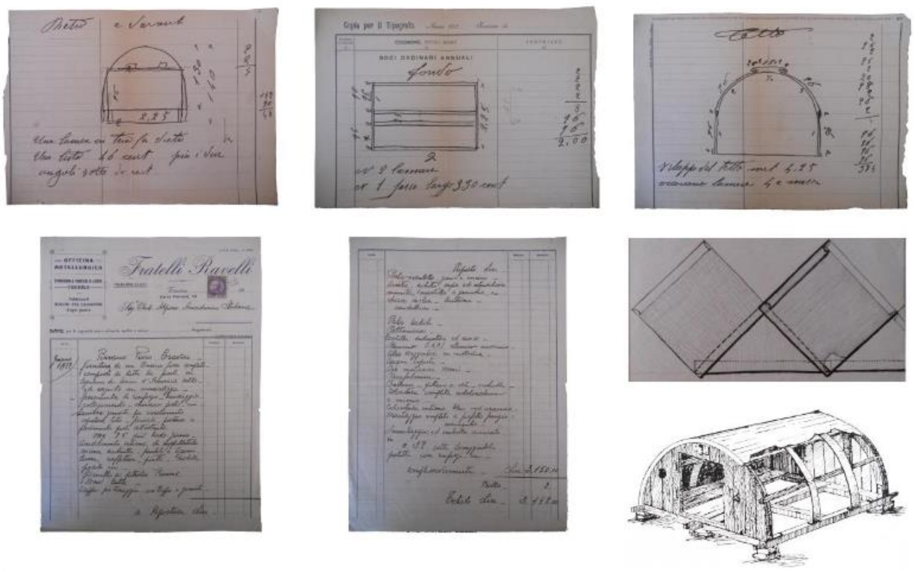

Figure 1.2: The "model Rivera", source Historic Archive of the CAI, Biblioteca Nazionale del CAI di Torino

The materials involved in those constructions where all easily mountable and removable, and lightweight, in order to be transportable by mules. As can be observed in figure Figure 1.2, the structure was in wood and had a cladding consisting in sheet metal made of zinc, internally covered with wooden boards covered by a layer of bituminous waterproofing. It could contain up to four people.

Dimensions grew and the form slightly changed during and after the second world war; this period saw, in Italy, two main very similar models designed one by the engineer Apollonio and the other one by the engineer Baroni, with the help of the Berti foundation. As can be seen in Figure 1.3, the semi-cylindrical shape was abandoned and replaced by bigger structures able to host more people (up to nine). Above in the figure, drawings of the Apollonio model show the typical arch of variable radius shaped roof, leaning on vertical walls that allowed the structure to reach heights of about 2.30 m. In the Berti-Baroni model, shown below in the figure, the slight difference can be observed in the roof;

it is formed by six sides of equal length but different inclination. The base dimensions (2,82x2,28m) as well as the materials on the other hand remained in both cases very similar to the model Rivera. The model Apollonio and the model Berti-Baroni are until today the most replicated models in the history of high altitude constructions.

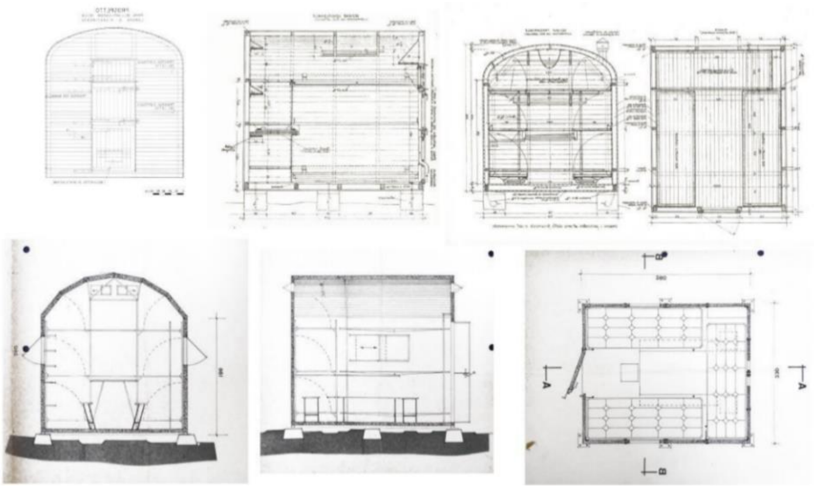

Figure 1.3 The Apollonio model (above) and the Berti-Baroni model (below). Source Historical Archive of the CAI, Biblioteca Nazionale del CAI di Torino

With the economic boom between the late 60s and 90s, a new revolution in the design of these structures took place. The newfound welfare allowed big steps forward in the progress of the technologies related to materials and construction techniques, resulting in highly technological buildings with futuristic shapes, perfectly reflecting the historical period they were built in, in which humanity was parallelly beginning to explore space. Although, differently to the previous cases, those configurations remained always experimental and were never replicated.

Figure 1.4 Above from the left: Bivacco Adolfo Hess (Rivera Model), built in 1925; bivacco Ivrea (Apollonio model), built in 1948. Below: two examples of futuristic shaped bivouacs from the 60s to 90s period. From the left: bivacco Bruno Ferrario, built in 1968 and bivouac du Dolent-La Maye built in 1973.

Nowadays there's an estimated two thousand (or more) structures considering mountain huts and bivouacs spread over the alpine territories of Italy, France, Monaco, Switzerland, Germany, Liechtenstein, Austria, and Slovenia. This high number translates in a large variety of architectonical and structural approaches regarding shapes and involved materials which, as seen in the paragraphs above, has gone through a big evolution in time, leading to today's situation.

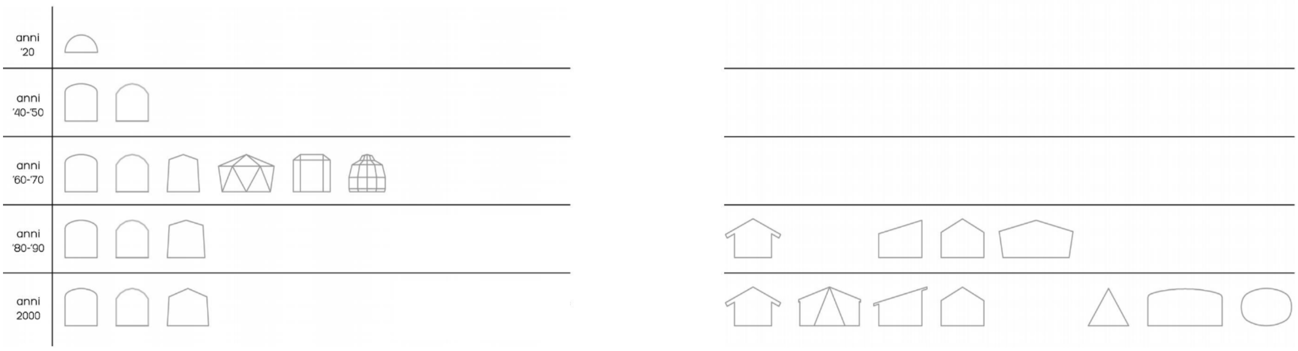

Figure 1.5 Evolution of the shape of the bivouacs in time

1.2 State of Art

1.2.1 Transportation

The construction sites are normally located in faraway areas, isolated and very hard to reach. It is therefore generally a good rule to maintain the construction times as short as possible; ice, snow and freezing cold temperatures which in some cases affect the site for twelve months a year, surely appear in large quantities or, when already present, increase during the winter season. In this context, a massive progress was brought in recent years by the establishment of the helicopter as a transportation tool, replacing the previously used mules or human shoulders. A revolution that allowed, apart from drastically reducing the construction times, many more possibilities not only in the involved materials but also in technical solutions, as the helicopter can work not only for the shipping of material but also for the assembly of heavy pieces together. Nevertheless, despite the obvious advantages implicated by its use, the aircraft transportation is a mean that still presents limits; the maximum weight capacity of a single helicopter is depending from various factors, but usually has an average of about 900 kg, corresponding more or less to that of a lorry. In some cases, in recent years, the solution taken has been that of designing extremely lightweight and small structures whose total weight was within those limits.

1.2.2 Foundation and structure

For what concerns the foundation, obviously the employment of materials such as concrete or steel represent an enormous obstacle with a view of a possible transportation of those, as an operation of that kind could become very expensive. With this in mind, it is often an adapted solution to choose a position which characteristics allow it to at least partially avoid the implementation of this part of the structure. This condition can occur in situations where the ground is already of extremely good quality, and it's enough to place wooden or metal spars on it, or when there are already pre-existing base plates. In case a foundation has to be built, an efficient solution is to place punctual concrete plinths, allowing to have a small digging surface so to minimize the damage to the ground. In addition, it is always appropriate to elevate the structure from the ground in order to better isolate it thermally.


Figure 1.6 From left to right: Plinth foundation of the Jubiläumsgrat bivouac; Bivacco Gervasutti, built in 2011

The structure itself has to face the massive challenge of being able to withstand the, in these environments, very big wind and snow loads, and at the same time being as lightweight as possible for the previously mentioned limitations imposed both by the transportation as well as by the lack of real mechanical lifting arms. The material that best suits the combination of those conditions is without a doubt wood, which is also usually the best solution from an aesthetic point of view, as it reduces to the minimum the contrast between nature and building. It has a high versatility in terms of processing, a good structural and thermal performance and is fairly durable and light; it is mostly used in the form of frame and panels. Other solutions adapted in recent years resorted to the metallic carpentry or in some cases to synthetic materials like plastic or fiberglass; an example of the latter would be the Gervasutti bivouac (Figure 1.6, right) on the Mont Blanc massif.

1.2.3 Claddings and openings

The external cladding, which has as primary function that of protecting the structure from atmospheric agents, is mostly used in its metallic forms; copper, sheet metal and aluminium are often adapted solutions. Internally, the most common cover is wood or plastic based. Is it important for these components to be light and easily installable, and for the internal claddings the vapour resistance can be a factor of significance in order to avoid interstitial condensation.

Given the original function of a bivouac, that is not designed for a visitor to stay for more than a certain amount of time, openings are usually a secondary element in the building. In general, it can be noticed that while in the past it was not uncommon to see a use of windows reduced to its minimum, today the tendency is to widen their surface in the construction (as can for example be seen in the Gervasutti bivouac, Figure 1.6, right), so as to "bring" the surrounding environment inside the bivouac. This is probably due to a phenomenon that started in recent years and is still ongoing today, which consists in a radical change of the approach towards these structures, as they are in many cases seen less and less as "only" a necessary shelter on the way of climbing before unreached peaks, and more as a "destination" in themselves. Therefore, the attention to the aesthetical aspect and the effort put in their design is much higher today than it was one century ago.

1.2.4 Spatial organization and technological aspect

As bivouacs are such small and essential buildings, the spatial organization plays a vital role in their assessment. A functional solution is to have one space with beds developing vertically, in order to optimize the space at the base; nevertheless, in the last years, more and more bivouacs are built with more than one space, significantly increasing their dimensions with respect to less recent times.

The technological equipment of the structure represents in today's time a new type of challenge and is at the same time a largely discussed matter in the high mountains world. On one side, those who have a more conservative view of alpinism and of the use of bivouacs favour a spartan type of configuration, refusing any type of possible technological progress; on the other side, the convinced innovators call for as much technological comfort as possible. Generally, it is today common to have a "basic set", provided with a small photovoltaic panel able to produce enough energy to feed a radio transmitter for SOS. Furthermore, a few constructions nowadays are equipped with additional supplies like a mechanical air circulation system, internet connection or a heating system. Those gadgets, as much as they can increase the quality of the permanence inside bivouacs, have the problem of requiring a lot of maintenance, which makes them often expensive and little practical.

1.3 Brief introduction to Building Information Modelling

Building Information Modelling, commonly referred to as "BIM", is as of today probably one of the hot topics in the construction world. A real milestone, considering the context in which it was born and the changes it eventually contributed to in the approach towards many aspects of the industry. Various definitions have been given to BIM; one is suggested by Eastman et al., 2008 in "BIM Handbook", where BIM is described as *"a verb or adjective phrase to describe tools, processes, and technologies that are facilitated by digital, machine readable documentation about a building and its performance, design, construction and operation"*. As the definition insinuates, BIM touches many sectors of the whole building process, and is therefore perceived in different ways from people from e.g. the design, construction and financial management field. In fact, BIM today refers to a product (the building information model), a process (the creation of all the intelligent data that can be used throughout the lifecycle of the construction, building information modelling) and to a system (the management of the net of data and files useful to increase quality and efficiency, building information management). One could say that, in a nutshell, the ultimate goal of BIM is to improve and maximize the design efficiency and the management of a project; this is done through the employment of digital modelling software that allow to have all the information needed in one single virtual building model which can be linked to numerical data, texts, images and other type of information, accessible and modifiable by all different professionals involved in the construction process.

1.3.1 Background

The construction sector employs 7 per cent of the world's working population and is one of the largest sectors in world economy, with about 10 trillion dollars spent on construction related goods every year, equivalent to 13 percent of Gross Domestic Product (McKingsey&Company, 2017). However, according to a study conducted by the McKingsey Global Institute in more than 20 countries and 30 companies, construction has suffered in the past decades from poor productivity relative to other sectors; compared with a 2.8 percent growth average per year for the total world economy and a 3.6 percent for manufacturing, construction sector labor-productivity growth averaged 1 percent per year over the past two decades. The labor-productivity performance of the construction sector is not equal throughout the world and there are obviously regional differences, with better and worst performing countries. For example, always refering to the analysis performed by McKingsey, in the United States the labor-productivity of the branch is lower today than it was in 1968.

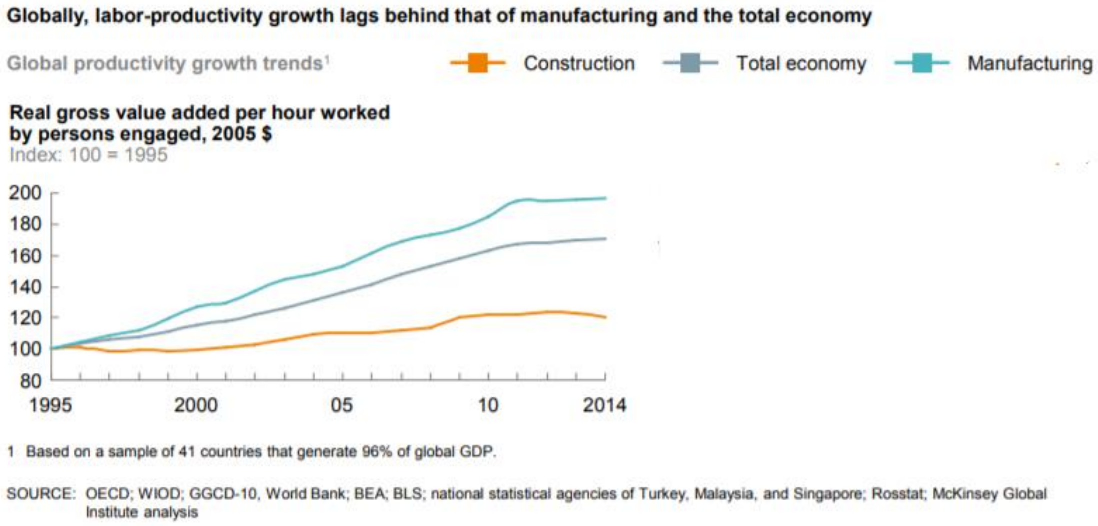

Figure 1.7 Productivity of construction industry in comparison with the manufacturing industry and the total economy

One of the main causes for these issues, is that many construction projects suffer from overruns in cost and time. In fact, for any type of civil construction project there are tens to hundreds of documents, blueprints and details that must be followed and interpreted, and this number can increase even drastically in the case of a major infrastructure projects. This high number of data combined with the oftentimes complicated communication taking place between the different parts involved in the project result in many situations in delays in the construction times, unexpected high field costs and also legal problems.

In this context, BIM can be an extremely useful tool in order to facilitate not only the creation of the various information concerning a construction i.e. drawings, schedules and specification details which are all contained in one digital model, but also the interaction between people from different fields of the building industry having to collaborate for the project.

1.3.2 General information

Being BIM, as seen in the previous paragraphs, such a multitasking tool, the question often arises of what exactly BIM is. In fact, a common mistake is to speak of BIM as of purely a set of software able to create a model. Probably, a more appropriate way to refer to BIM is of a methodology integrating all the professionals involved in the construction process (architects, engineers, contractors, etc) and creating a flow of information between them thanks to a representation of *"both the physical and intrinsic properties of the building as an object-oriented model tied to a database"* (Quirk, 2012).

Important features of BIM are listed below:

- All BIM software are able to update information automatically; this means that, as the model is developed, all other drawings within the project are adjusted accordingly without the need of any further intervention. This is particularly useful to reduce the times of the operations and avoid the human error when they are performed.
- The different parts involved in the project all work on the same model. This allows for a more fluid workflow and minimizes the loss of information during the process.
- The model is accessible and useful throughout every life-cycle phase of the structure or infrastructure, from the initial conceptual planning and designing phase to the in-operation maintenance, form the building phase to even the eventual deconstruction phase; this is an enormous simplification with respect to any other previous existing method, where all of those issues were in many occasions newly approached, making it often very complicated to know the necessary information about the building, for example in the case of maintenance, in order to operate.
- Since the model is accessible to professionals of different construction fields, everyone can add what for them is most essential for the project making it the closest thing to what can be a complete representation of reality.

Every BIM model is developed in different so called “dimensions” that describe the levels of information of the model, which will now be listed and explained:

- <u>1D, the idea:</u>

The first phase usually consists in a first vision of the project to be done; a location is defined, as well as the function of the to-build construction and possibly a qualitative first estimation of the order of magnitude of the project (how many people are involved, costs etc).

- <u>2D, drawings:</u>

The second dimension consists in the two-dimensional (x-y axis) drawings. Further information to be added can be the materials involved, the definition of a structural scheme and the applied loads.

- <u>3D, model:</u>

The z axis is added in the third dimension, and a three-dimensional model is created based on the information collected in the first two dimensions. This is, perhaps, the most commonly known kind of BIM, a concept that many are familiar with. The model however is not static, it evolves in time with the adding of more and more details and information. The final model is the “As-built” one, giving ideally a precise representation of the real product.

- <u>4D, the time element:</u>

4D BIM adds an additional dimension to the project data in the form of time scheduling of the different operations and construction phases. Information like the construction times or the order of installation of the different components can be added here, resulting in improved control over conflict detection or over the many changes occurring during a construction project.

- 5D, the costs:

The fifth dimension adds the component of the costs to the whole procedure. This variable is updated regularly on the software and allows users to visualise regularly where they stand cost-wise in the project and estimate the overall costing associated to the progress of the activities. Compared with the traditional methodology, where the costs aren't updated as regularly, cost managers will have to start operating earlier and perform more iterations but this will result in better outcomes, as it will make it easier to remain in the initially defined budgets.

- 6D, sustainability:

This dimension is used to assess the energetic performance of the construction during its operational phase. Sensors should allow to collect the needed data in order to define a strategy aiming to optimize the facility's energy consumption.

- 7D, facility management:

7D is where the BIM data can definitely make a difference. In fact, the operation and maintenance of a construction, that this dimension regards, can signify an important percentage of the total accumulated costs of a building during its life cycle, and starting a facility management program based on reliable data extracted from a well made as-built BIM model provides with the most effective solutions for the management of a construction.

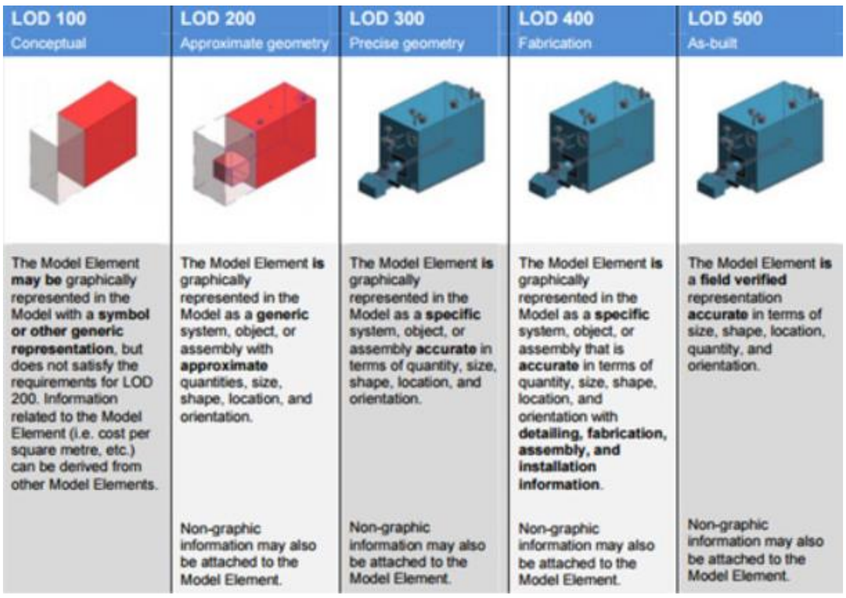

Figure 1.8 Level of Detail descriptions

Furthermore, all BIM models are characterised by a Level of Detail (LOD), that basically describes the quality of the model, in terms of total amount of information contained. This value usually starts at 100 at the very early stages of the procedure and increases over time with the progress of the project, eventually reaching maximum quantity of 500, that represents the As-built model containing all the possible information. An idea of the difference between the various LODs is given in Figure 1.8.

1.3.3 Interoperability

Interoperability is a very important aspect of BIM Software and of BIM in general. When creating a BIM model, it may be necessary at times to transfer data from one software to another in order to increase the amount of information of the model. In fact, some software may be specialized in some types of operations but lacking at possibilities in others, which on the other hand can be better handled by different products; interoperability should grant the opportunity to perform every operation with the best fitting tool, so to have the most complete model possible. It is therefore important that during this process the least amount of data goes lost. Additionally, one always has to take into account the limitations imposed in this case by the costs, since these software usually have a relatively high price. However, interoperability is ultimately defined by the ASUL interoperability working group as *"a characteristic of a product or system, whose interfaces are completely understood, to work with other products or systems, present or future, in either implementation or access, without any restrictions".* As will be noticed, it is described as a characteristic of the tool, and not as a process.

Since poor interoperability can represent a big obstacle to the progress of the project and possibly be the cause of financial losses, neutral, non-proprietary or open standards for sharing BIM data among different software applications have been developed in order to achieve the best outcome. Examples of BIM standards are the CIMSteel Integration Standard (CIS 2), which enables data exchange during the design and construction of steel framed structures, Construction Operations Building information exchange (COBie), useful particularly in the operation and maintenance phase, and finally the probably most known Industry Foundation Classes (IFC), developed by buildingSMART and recognised by the ISO. The latter has been an official standard, ISO 16739, since 2013.

1.3.4 BIM today

Nowadays, the implementation of BIM is spreading and increasing rapidly throughout the world. In the United Kingdom, undisputed world leader in the branch, the British Standard Institute (BSI) has strongly promoted the utilization of the methodology in recent years by producing various standard (BS 1192) to support the construction industry in the adoption of BIM. The methodology has been here classified in four “Levels of Maturity”, that go from 0 to 3; at level 0 there is a complete lack of Building Information Modelling and no collaboration between the different parts is taking place; level 1 implies that 2D and 3D models have been made but collaboration between the parties is not achieved yet; at level 2 the different professionals are collaborating on intelligent data in form of models and possibly additional information, but there is still a lack of a single source of data, and finally at level 3 there is complete and total collaboration in the planning, construction and operational life cycle of the facility and the information are all shared and stored in one single source of data. An idea of the development of the levels is given better in Figure 1.9.

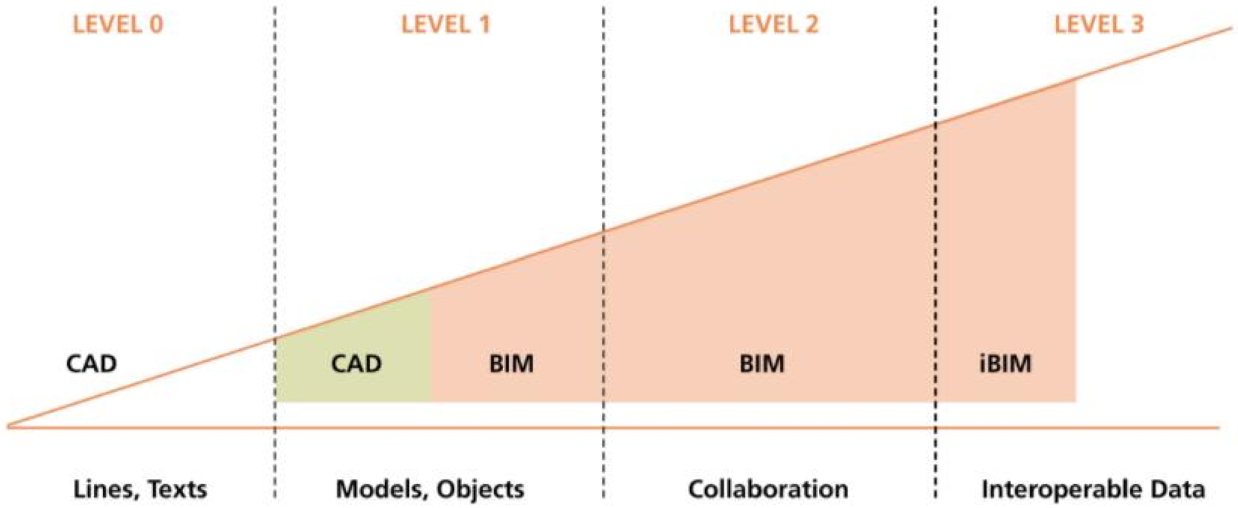

Figure 1.9 Levels of BIM Maturity

Since 2016, the British government has mandated the achievement of BIM level 2 Maturity for all publicly funded construction work.
Other countries where the implementation of BIM is mandatory today for public projects are, amongst others, Norway, Denmark, Finland, Sweden and Singapore. In many others, such the United Arab Emirates or Australia BIM is mandatory for public projects that exceed certain dimensions or costs. In the United States, BIM isn’t mandated across all the states yet but is expected to grow quickly. In 2010, the state of Wisconsin made it mandatory to implement BIM for public projects if equal or above the total budget of $5 million. Finally, also in Italy in recent times the government has been pushing with ordinances the increase of utilisation of the methodology for public works; since the 1/1/2019, the implementation of BIM is mandatory public works that exceed the budget of 100 million Euros, by 2020 the budget maximum budged was reduced to 50 million, from 2021 it will be

of 15M and this value will be further decreased (5.2M in 2022 and 1 million in 2023) until from 2025 it will be mandatory for all public projects.
Some of the leading firms in the BIM software industry are, as of today, Autodesk, producers of e.g. Revit, Robot, Advanced Steel and Naviswork, and Trimble, whose main products are Tekla and SketchUp. These, as well as other important tools, are illustrated in Figure 1.10. Apart from the more innovative software such as those mentioned above, also more traditional programs like Microsoft Excel and Microsoft Word can obviously be (and usually are) part of the BIM process.

Figure 1.10 List of useful BIM tools

1.4 The case study

The architectonical drawing of the building was handed to me by the company “Leap factory”, with the task of doing a comparison between the performances of two structural materials applied on it, in terms of structural and energetic behaviour. Leap factory is a company based in Turin who built, among others, the “Bivacco Gervasutti” in 2011, as well as other structures in extreme glacial environments like the “frame” project in Greenland. The studied structure that will be presented in the next paragraphs is not a typical bivouac; it is, in dimensions, more of a mixture between a bivouac and a mountain hut (bigger than a usual bivouac but not big enough to be considered a mountain hut, which is usually composed by more than one inside space). However, it is not thought to be managed by a staff living inside of it and providing additional services to the visitors, and its planned function, which is in the end what really matters, is therefore that of a bivouac.

1.4.1 Structural framework

As can be observed in Figure 1.11, the structure has a rectangular base of dimensions 6.12x4.8 m. Three of the four walls are structural, made, depending on the case, of one of the two studied composite materials, which both consist in wooden based panels; a fourth wall, on the short side of the building, is a glass non bearing surface.

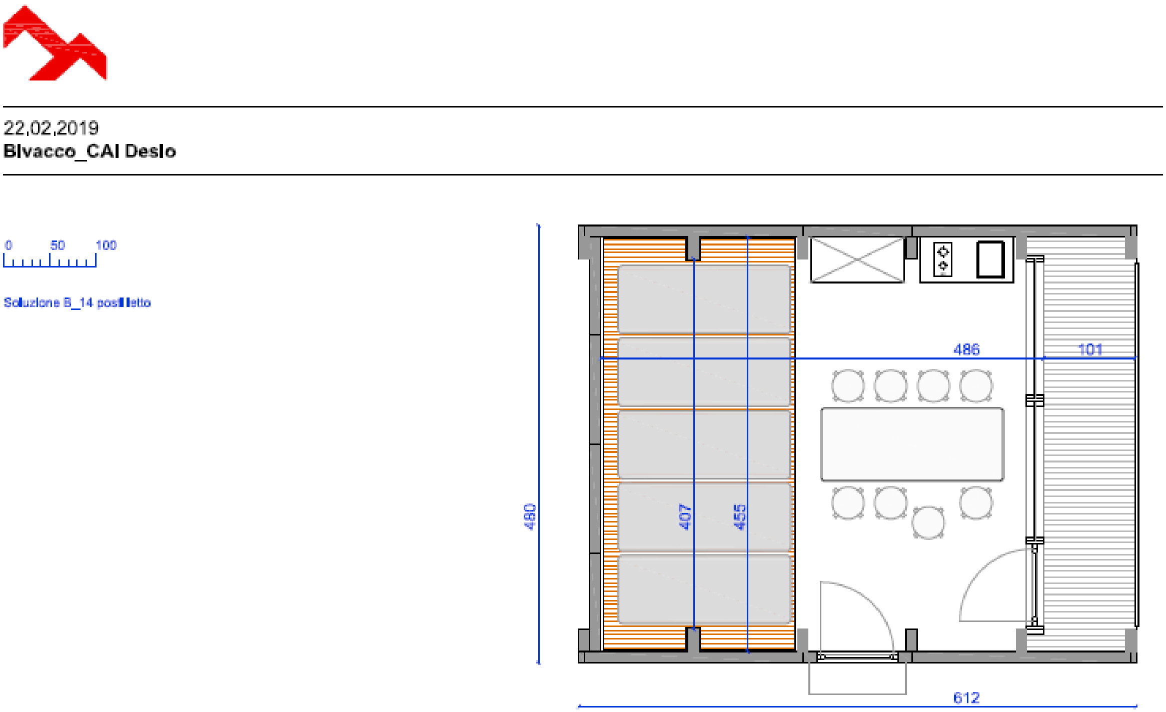

Figure 1.11 Floor plan of the building

There are two openings in the form of doors, one placed at the centre of one of the two long sides, the other one on the glass wall, leading to a terrace of dimensions 1.01x4,56m. In addition to the panel structure there can be added, if necessary, supporting frames every 1.2m, with a total of six possible frames. The frame system is not associated to any initial pre-dimensioning, it is thought to be added only in case the panels aren't enough to withstand the whole load the bivouac is subjected to, and therefore to be designed according to the "missing" resistance and stiffness, meaning that which the panels cannot provide. From a more architectonical point of view, the drawing shows a platform developing from one long side to the other and from the short panel side towards the glass wall for 2.4 m containing five beds, as well as a table allocated more or less at the centre of the internal space and some furniture placed against the long side opposite to the door.

Figure 1.12 illustrates a vertical section and the side view of the structure. The height from the intrados of the floor to the extrados of the ridge of the roof panels is of 4.81 m; as will be explained more specifically in chapter 4, for the creation of the model the thickness of the floor in the drawing has been assumed to be of 12 cm (since not specified), which generates a total height of the frame and of the entire structure of 5.05 m. It can be acknowledged, observing both drawings, first that the two walls on the long sides are of different heights, and second that the door is placed on the smaller wall.

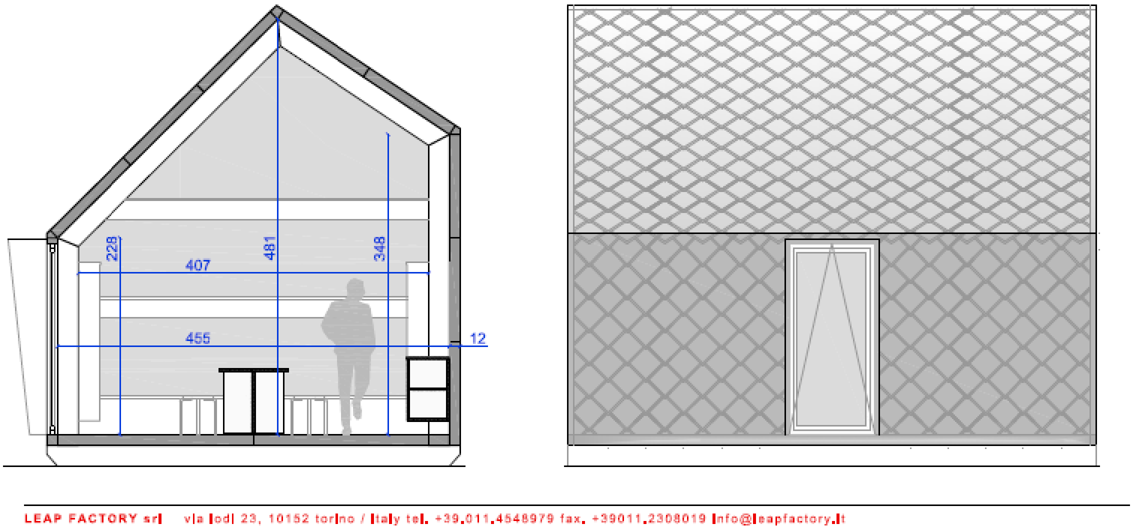

Figure 1.12 From left to right: cross section and side view of the building

The two long walls are one of height 2,28 m (left on the left drawing) and the other of height 3,48 m (right on the left drawing). The view of the cross section also allows to better recognise the supporting frame, composed by four elements, two columns and two rafter beams. One can also notice three levels at different heights which represent the platforms containing the beds, five on the first two and four on the last one, making a total of fourteen beds.

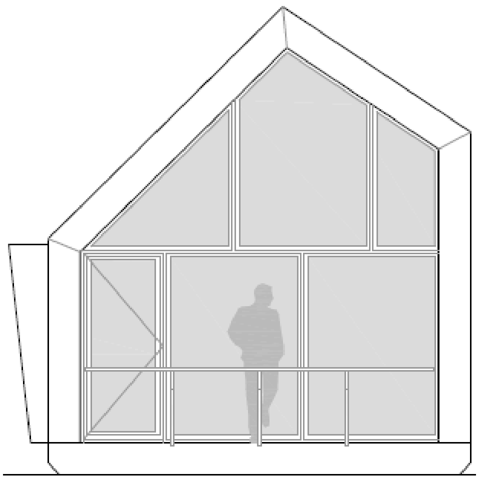

Figure 1.13 Front view of the building

Figure 1.13 shows a view of the building as if one were to stand in front of the glass wall. One can see the geometry of the frame supporting the glass surface, composed by elements of non-specified dimensions.

1.4.2 Setting of the structure

The studied structure doesn't have a specific location; it is placed in a generic point in the high altitudes of the alpine areas in Italy. A consideration to be made, at this point, concerns the problem of what can be considered "high" alpine environment. In fact, this denomination is not clearly defined, as it could, purely theoretically speaking, vary depending on the latitude of the area, the climate and other factors involved; nevertheless, for the sake of simplicity we will consider at high altitude those environments above 2500 m amsl.
In the case of this building, the company probably originally sent the drawings to the CAI section of Desio, as a keen eye will spot in Figure 1.11, which is a municipality situated in the Lombardy region, so the first intention was perhaps to build it somewhere in the Lombard Alps. However, the indications for this study from the company were to consider it to be at an altitude of 3000 m above the sea level, but no more specific information were given since it was of no particular advantage and therefore interest to give it an exact location. In any case, for the generation of e.g. temperature effects on the model, the building has been given a certain exposition; the glass is facing west, the higher longer wall faces south, and so on.

Chapter 2 2D BIM: description of the structural panels and definition of the loads on the structure

The study of the structure was conducted with two different wooden based panels; one is produced by the British company "KINGSPAN", it is a traditional SIP (structural insulated panel), composed of two OSB sheets separated by one layer of insulation material called PUR. The second on the other hand is composed by an internal layer of LVL wood, a sheet of PUR insulation and one external layer of OSB wood and is produced by a company from Estonia called "PANELO". Apart from the difference in the purely material composition of the panels, they are also characterized by two different overall thicknesses, resulting in unalike values of stiffness, resistance and thermal conductivity. The first part of this chapter will see a detailed description of those construction systems, while the second one will see, in order to conclude the second BIM dimension described in paragraph 1.3.2, the definition of the loads applied on the examined structure for the structural study.

2.1 What are SIPs?

Structural insulated panels are a form of sandwich panels, consisting of two structural faces and an insulating foam core sandwiched in between them. They are a high-performance building system for light construction developed during the first half of the last century in the United States, and while they are not particularly common in western Europe, they are an often adapted solution in north America and in Russia. The structural faces are usually made of OSB (oriented strand board) wood, but can in some cases also be sheet metal, cement or other types of timber like plywood or LVL (laminated veneer lumber), and the core is typically rigid Polyurethane (PUR), polyisocyanurate (PIR), polystyrene foam (EPS) or extruded polystyrene (XPS) and is significantly thicker than the structural layers. The structural properties of SIPs correspond to those of a I-beam or I-column, where the insulating layer works as a web and the two faces as the flanges (Figure 2.1); the axial and bending forces are therefore carried by the outer layers (the flanges) and the shear force by the core (the web). To connect one panel to the other there are different options; one is to use timber posts, dimensioned to fit in between the elements, which also work as a reinforcement for the structural purpose of the panel; the problem related to this solution is that the timber creates a thermal bridge, since its conductivity values are much higher than those of the isolation material. To prevent this connection splines can be utilised, usually slightly thinner that the panel (thickness equal to that of the insulation), but equally composed by two structural (OSB) sheets and an insulation core.

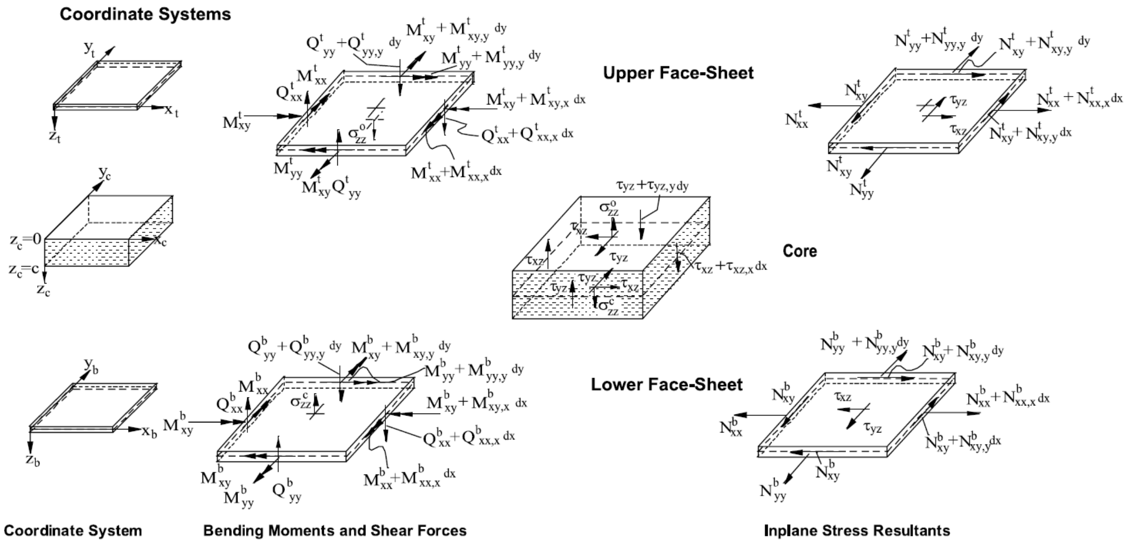

Figure 2.1 Typical sandwich plate; sign convention, stresses and internal stress resultants

2.1.1 Benefits and drawbacks

SIPs are very quick and quite easy to install, so much so that the placing of all the required panels for one entire house can last just two or even one day. They are generally considered to be a lightweight material, but are nevertheless heavy enough for it to be unlikely that one person alone is able to install them unless he/she is equipped with some mechanical arm, therefore usually a crew of people is necessary to build a house. Compared to a traditional timber stick framing construction system, SIPs have much better thermal insulation properties, as the timber elements in the framing system represent a thermal bridge, but they are also generally more expensive; although if one would be taking into account savings related to construction speed and smaller heating and cooling costs, SIPs could very well be just as or probably less expensive than stick framing in terms of total cycle costs. The main difference which in this evaluation often makes people opt for the framing system, is that while in framing construction one can buy one piece at a time and spread the costs more over time, in SIPs one necessarily has to buy the whole piece at once, inevitably making it a higher immediate expense. Moreover, an OSB skinned SIP structurally can outperform stick framed constructions in the case of axial load strength. A significant throwback of SIP panels is related to moisture, in the form of creation of interstitial condensation in between the layers during the heating season. This is due to the fact that, during the winter, water vapor tends to migrate from the warmer inner environment towards the colder outside. In case of SIPs however, especially the external OSB layer has a too low permeability, causing the water vapor to be subjected to a "blocking" effect and condensate. Finally, a downside of SIP panels that particularly affected this thesis is that there's no real standardization of them in Italy. Therefore, in the case the considered structure, their analysis was conducted with the use of the CNR-DT-206-R1-2018, which is a general standard for the design of wood, and a PDF file made by the Italian section of a wood promotion initiative of the Austrian company "pro:Holz" called promo_legno. The name of the PDF is *"Il calcolo dell`XLAM. Basi, normative, progettazione, applicazione"* and, as the title suggests, it actually concerns another type of wooden based panels, namely the crosslams (in Italy often referred to as XLAM), which now won't be illustrated in detail but in many aspects can be considered similar to the SIPs, at least in the initial approach.

2.2 The KINGSPAN system

The Kingspan panel is called "Kingspan TEK"; as mentioned in the first paragraph of chapter 2, it consists in two faces of OSB and an insulation core of PUR insulation. OSB stands for oriented strand board, and is a type of wood obtained by adding adhesives and then compressing together wood strands in a specific orientation, while PUR stands for rigid polyurethane; it is a polymer, which in this case, as the name suggests, comes in a rigid form. For what concerns the OSB, there are 5 classes of boards: OSB/0 doesn't have any added formaldehyde, which is one of the components forming the adhesive resins, OSB/1 and OSB/2 are to be used only in dry conditions (the difference relies in the load bearing capacity) and OSB/3 and 4 can be used in humid conditions, where OSB/4 is a heavy duty load-bearing board while OSB/3 is a regular load-bearing board; in the case of the Kingspan panel, the employed class is OSB/3. Regarding the standard characteristics of the panel concerning the thicknesses of the layers, dimensions etc, the datasheet made available by the company will be quoted (BBA Certificate, pag. 5): *"Each panel of the Kingspan TEK Building System is nominally 142 mm or 172 mm thick overall and has two outer skins of 15 mm thick OSB/3 (oriented strand board type 3), separated by a core of 112 mm or 142 mm thick, zero rated ozone-depleting potential (ODP) rigid urethane insulation (PUR). The panel mass is approximately 25 kg m^2 for the 142 mm and 172 mm thick panels. The panels are available in widths ranging from 200 mm to 1220 mm, and lengths up to 7500 mm. The panels are supplied in the appropriate shapes and sizes for each project, together with any expanding urethane sealant, fixings and jointing pieces that may be required.*

[...]

In addition to the panels, a number of other components are required to facilitate the assembly of the system:

For the 142 mm thick panels:

- *edge timbers — minimum 50 by 110 mm C16 graded or equivalent (+1/-1 tolerance on dimensions)*
- *structural timber posts — minimum 100 by 110 mm C24 graded or equivalent*
- *insulated splines — 100 mm (w) by 110 mm (d), comprising two OSB/3, 15 by 100 mm skins and rigid urethane insulation core (…).*

For the 172 mm thick panels:

- *edge timbers — minimum 38 by 140 mm C16 graded or equivalent (+1/-1 tolerance on dimensions)*
- *structural timber posts — minimum 80 by 140 mm C24 graded or equivalent*
- *insulated splines — 80 mm (w) by 140 mm (d), comprising two OSB/3, 15 by 80 mm skins and rigid urethane insulation core (…).”*

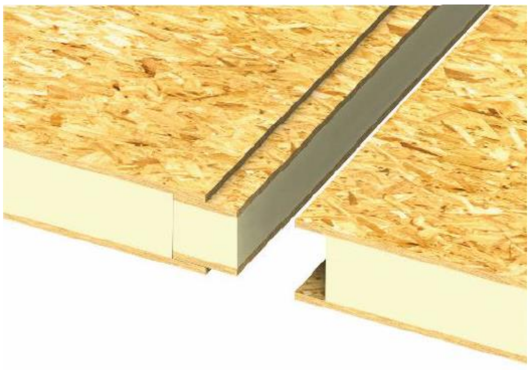

Figure 2.2 View of the panels connected by an insulated spline

Of the two possible panel sizes in terms of thickness (142 and 172 mm) described by the datasheet, the one chosen to perform the work was the TEK 142 mm system. This was done for the simple reason that the Kingspan company provides significantly more information related to the connection types to be used and to the standard details of the panel, all of which is of great help when dealing with BIM software and in general with structural design; accordingly, the composition of the panel is of 15 mm OSB, 112 mm PUR and 15 mm OSB.
The characteristics exposed in the next paragraphs regard the system’s strength and stability, the thermal performance, the vapour permeability and the condensation risk of the system as well as the construction details and were all taken from a different documents made public by the company: a certificate awarded by the BBA (British Board of Agrément) and approved by the ETA (European Technical Assessment) which are the UK and EU leading construction certification bodies, a brochure of the panel, and a specification manual of the TEK system.

2.2.1 Mechanical properties

The exact density of the different components of the TEK 142 system is provided by the BBA certificate (Table 2.1), same goes for the strength, resistance and stiffness values, that will be illustrated in different Tables and commented throughout the paragraph.

OSB board	(ρ = 650 kg/m^3)
Thermal insulation, PU	(ρ = 32-35 kg/m^3)

Table 2.1 Density of panel components

The density values were used, in relation to the thickness of the layers, to calculate on excel the total density and self-weight in $\frac{kN}{m^3}$ of the panel.

	t [mm]	ρ [kg/m^3]	m [kg/m^2]	$\gamma_{tot,KS}$ [kN/m^3]
OSB (ext)	15	650	9,75	/
PUR	112	35	3,92	/
OSB (int)	15	650	9,75	/
TOT,KINGSPAN	142	164,9295775	23,42	1,617959155

Table 2.2 Density and selfweight of the panel

Table 2.3 shows contains the design values of strength that should be compared to the worst loading case in the Ultimate Limit State (ULS).

Strength		Duration of load				
		Permanent	Long	Medium	Short	Instantaneous
Bending strength(2) (M_{Rd})	kN·m·m^{-1}	4.48	5.97	8.21	10.4	13.4
Shear strength(2) (V_{Rd})	kN·m^{-1}	3.81	7.62	11.4	15.2	15.2
Bearing strength(3) (B) min. 45mm bearing	kN·m^{-1}	3.66	7.32	11.0	14.6	14.6
Axial strength (*N*)						
wall height <2400 mm	kN·m^{-1}	38.5	50.8	57.9	64.7	78.1
wall height 2400 – 2700 mm	kN·m^{-1}	33.4	44.4	57.9	64.7	78.1
wall height 2700 – 3000 mm	kN·m^{-1}	29.0	38.8	57.9	64.7	78.1
wall height 3000 – 3500 mm	kN·m^{-1}	23.3	31.4	47.6	64.7	78.1
wall height 3500 – 4000 mm	kN·m^{-1}	18.9	25.7	39.3	64.7	78.1
wall height 4000 – 4800 mm	kN·m^{-1}	14.0	19.2	29.6	50.4	62.6
Racking strength(4)(5)(6) (*R*) with ø2.8x6.3mm smooth nails						
75 mm nail centres	kN·m^{-1}	N/A	N/A	N/A	8.89	10.9
100 mm nail centres	kN·m^{-1}	N/A	N/A	N/A	7.42	9.07
150 mm nail centres	kN·m^{-1}	N/A	N/A	N/A	5.58	6.82
Stiffness						
EI_{inst} for wind load checks	N·mm^{-2}	4.60E+11				
EI_{perm} for long-term deflection	N·mm^{-2}	EI_{inst} /(1 + 1.87)				
GA_{inst} for wind load checks	N	5.70E+05				
GA_{perm} for long-term deflection	N	GA_{inst} /(1 + 6.45)				

Table 2.3 Structural properties - limit state design - TEK 142

In addition to providing the typical bending, axial and shear strength and the stiffness, the given table also contains values describing the panel's racking resistance, which defines its in-plane lateral strength. More specifically, racking occurs when a wall (or panel in this case) is forced out of plumb and tilts due to high horizontal forces usually caused by wind or possibly also seismic actions; it is strictly related to the type connections involved, hence the specifications regarding the nail dimensions. In general, the layout of the data emphasizes the difference of strength values according to the duration of the load or, in the case of axial force, to the height of the wall.

According to the EC5, the NTC2018 and the CNR (specific for wood) the definitions of permanent, long, medium short term and instantaneous loads are the following:

-Permanent: more than 10 years

-Long-term: between 6 months and10 years

-Medium-term: between one week and six months

-Short-term: between one hour and one week

-Instantaneous: less than one hour

From a practical point of view, all the values which can be seen in Table 3 basically can be taken as they are, without further calculations, and be used to verify the stability of the panel in the studied case by comparing them with the calculated acting forces. To compute these forces, the arguably most important feature that must be known is the stiffness of the system, which is also given in Table 2.3 but can be observed, along with the shear modulus and the values of k_{def}, more in detail in another table (Table 2.4) provided by Kingspan. The k_{def} term is a factor taking into account the increase of deformability with time due to both creep and moisture content of the material. It is useful to determine the reduced stiffness, to be used in order to find the final deformation in the serviceability limit state, in the following way:

$$EI_{d,fin} = \frac{EI_d}{(1+k_{def})} \qquad (2.1)$$

Where EI_d is the design value of the bending rigidity of the material and $EI_{d,fin}$ is the reduced stiffness.

Flexural deflection			Shear deflection	
Rigidity *EI* (Nmm^2)	**Deflection factor $k_{def,(EI)}$**		**Rigidity *GA* (N)**	**Deflection factor $k_{def,(GA)}$**
4.60E+11	1.87		5.70E+05	6.45

Table 2.4 Bending and shear rigidity of the panel

2.2.2 Thermal performance

Regarding the thermal performance of the TEK 142 system, the datasheets come up with various information. In order to calculate the thermal transmittance (U) of the panel, the BBA certificate gives the thermal conductivity (λ) values that should be used, shown in Table 5, or alternatively, the total panel's thermal resistance (R), in this case given for the whole system. The thermal conductivities of Table 2.5 have unit measure $\frac{W}{mK}$ and fundamentally describe a material's tendency to conduct heat; the greater they are, the less the material is isolating. The value related to solid timber refers to the structural posts, which can potentially be used as a reinforcement for the panel.

PUR insulation	0.024 (1)
OSB	0.13
Solid timber	0.12

(1) λ_D

Table 2.5 Thermal conductivities of the TEK Kingspan components

The thermal resistance R values can be observed in Table 2.6, as a function of the panel thickness. They are computed using spline connections and not timber posts.

Element	Walls		Roofs	
Panel thickness (mm)	142	172	142	172
Solid timber bridging fraction	4%		1%	
Tek Panel R value ($m^2 \cdot K \cdot W^{-1}$) (1)	4.237	5.317	4.584	5.788

Table 2.6 R values of the system

Additionally to the above described layers, another document uploaded by the Kingspan company called "TEK specification manual" states that *"All Kingspan **TEK**® Building System panels should be lined internally with plasterboard"* (pag.13), adding a minimum thickness of 12.5 mm and a conductivity λ equal to 0.25 $\frac{W}{mK}$.

An example of thermal transmittance of a TEK system with an additional layer of Plasterboard of 15 mm, as well as other elements is provided by the BBA certificate and shown on Table 2.7.

Element (1)	Wall (2)		Roof (3)	
Panel thickness (mm)	142	172	142	172
Element U value ($W \cdot m^{-2} \cdot K^{-1}$)	0.20	0.16	0.20	0.16

(1) Includes a 25 mm services cavity (11.8% timber battens @ 0.13 $W \cdot m^{-1} \cdot K^{-1}$) and 15 mm plasterboard $\lambda = 0.25\ W \cdot m^{-1} \cdot K^{-1}$

(2) Includes 102.5 mm brickwork, 50 mm vented cavity, breather membrane, TEK Panel with 4% solid timber bridging and 2.6% cassette spline bridging and the internal finish in note (1)

(3) Includes slates/tiles, well ventilated air space, LR roof tile underlay, TEK Panel with 1% solid timber bridging and 5.6% cassette spline bridging and the internal finish in note (1)

Table 2.7 Example panel thermal transmittance

Hence, an approximated calculation of the panel's thermal transmittance U both including and excluding a 15 mm layer of plasterboard and without the employment of timber elements was performed, to make a comparison with the value given by the BBA certificate. This was done through the relation:

$$U = \frac{1}{\frac{1}{h_i}+\sum_{i=1}^{n}\frac{s_i}{\lambda_i}+\sum_{j=1}^{m}R_k+\frac{1}{h_e}} \qquad (2.2)$$

where n is the number of homogeneous layers of thickness s and thermal conductivity λ, m the number of non-homogenous layers for which a thermal resistance R is defined, h_i is the internal heat transfer coefficient and h_e the external heat transfer coefficient, both of the latter terms being given by the summation of a convective and a radiation component.

The coefficients h_i and h_e were chosen, according to what literature gives as common values, respectively to be 8 and 25 $\frac{W}{m^2K}$.

The values of s and λ for the respective layers were put in an excel table and the computation was made as can be seen in Tables 2.8 and 2.9.

	s [m]	λ [W/mK]	U_KINGSPAN [W/(m^2*K)]
Plasterboard	0,015	0,25	
OSB	0,015	0,13	
PUR	0,112	0,024	
OSB	0,015	0,13	0,195219622

Table 2.8 Example of calculation of the thermal transmittance U with 15 mm plasterboard

	s [m]	λ [W/mK]	U_KINGSPAN [W/(m^2*K)]
OSB	0,015	0,13	
PUR	0,112	0,024	
OSB	0,015	0,13	0,197533365

Table 2.9 Example of calculation of the thermal transmittance U without 15 mm plasterboard

It can be noticed both how the values on Table 2.8 and 2.9 for the 142 panel don't differ much from those given as standard by the company, and also how the thermal transmittance doesn't change relevantly between with and without the 15 mm plasterboard; this is due to the fact that the most important factor influencing the U value is the insulating core and not the other layers.

Further data provided by the Kingspan company consists in an exposure of other different U values in function of the thickness of a hypothetical additional insulation layer that can be added to the panel system as well as different types of claddings that could be used. Here it will be shown only the case of a ventilated timber cladding, illustrated in Figure 2.3.

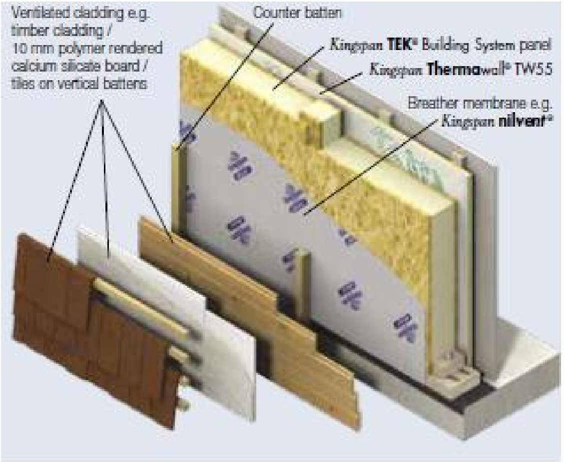

Figure 2.3 TEK system with ventilated timber cladding

The thermal transmittances in function of an additional insulation layer are shown in Table 2.10. In this specific case, the Table refers to an insulation board called *Kingspan* **Therma**wall® TW55.

Thickness of *Kingspan* **TEK®** Building System Panels (mm)	Thickness of *Kingspan* **Therma**wall® TW55 (mm)	U–value
142	0	0.20
142	20	0.16
142	25	0.15
142	30	0.15
142	40	0.14
142	50	0.13
142	60	0.12
142	70	0.12
142	75	0.11
142	80	0.11
142	90	0.11
172	0	0.17
172	20	0.14
172	25	0.13
172	30	0.13
172	40	0.12
172	50	0.11
172	60	0.11
172	70	0.10

Table 2.10 U values in function of thickness of TEK and thermawall

2.2.3 Vapour diffusion and condensation risk

To study the diffusion of water vapour inside the panel, originated by the difference in vapour pressure between the internal and external environment, the BBA certificate provides, similarly to the case of thermal conductivity, the values to be used when performing a calculation of the possible condensation point. In this case, the given quantities are those of the vapour diffusion resistance factor μ, which are dimensionless.

50	OSB warm side
60	PUR foam core
30	OSB cold side

Table 2.11 Vapour diffusion factor of the TEK components

Fundamentally, those values show how much more resistance a material applies to the diffusion of vapour inside of it with respect to air. In fact, to derive the material's permeability given its diffusion resistance factor, one puts in relation the latter to the air permeability, in the following way:

$$\delta_i = \frac{\delta_0}{\mu} \qquad (2.3)$$

where:

-δ_i is the material's vapour permeability in $\frac{kg}{m^2 * Pa * s}$,

-δ_o is the vapour permeability of air equal to approximately $2 * 10^{-10} \frac{kg}{m^2 * Pa * s}$,

-μ is the vapour resistance of the material.

Knowing the permeabilities, one can easily compute the total vapor diffusion resistance of the wall R_v, defined in $\frac{m^2 * Pa * s}{kg}$, with the formula:

$$R_v = \frac{1}{\beta_i} + \sum_{i=1}^{n} \frac{s_i}{\delta_i} + \frac{1}{\beta_e} \qquad (2.4)$$

being n the number of layers of the wall, s_i the thickness of the layer, δ_i the vapour permeability of the layer and β_i and β_e the water vapour transfer coefficients, whose value is so high that the first and last term of the equation can be assumed equal to zero. Therefore,

$$R_v = \sum_{i=1}^{n} \frac{s_i}{\delta_i} \qquad (2.5)$$

From the vapour resistance, one can compute the permeance of the wall, which is simply the inverse of the resistance:

$$M = \frac{1}{R_v} = \frac{1}{\sum_{i=1}^{n} \frac{s_i}{\delta_i}} \qquad (2.6)$$

It conceptually describes the ease with which water vapour molecules diffuse through the wall.

However, as mentioned in paragraph 2.1.1, SIP panels generally are at high risk of interstitial condensation, since they tend to "trap" the vapour which creates moisture. In fact, the "TEK specification manual" states that *"If a condensation risk is predicted, it can be controlled by ensuring there is a layer of high vapour resistance on the warm side of the insulation layer. If required, the vapour resistance of the wall lining can be increased by the use of a vapour check plasterboard*; the use of Kingspan **Therma**pitch® TP10 or **Therma**wall® TW55, both of which contain an integral vapour control layer*; the use of a layer of polythene sheeting*; or by the application of two coats of Gyproc Drywall Sealer to the plasterboard lining"* (pag.12).

In general, when trying to control interstitial condensation, it is always a good rule to place materials with decreasing water vapour resistance from the inside out.

Following the indications of the manual, a calculation of the vapour resistance and of the permeance of a wall composed by the Kingspan TEK system and a high vapour resistant plasterboard of thickness 15 mm, and one of a wall composed only by the TEK system were performed on Excel. The results are shown Table 2.12 and 2.13; they have obviously only a pure indicative value, as the humidity conditions the panel is exposed to play a significant role in the evaluation of those values.

	μ [-]	s [m]	λ [W/mK]	δ [kg/Pa*m*s]	Rᵥ [m^2*Pa*s/kg]	M [kg/m^2*Pa*s]
Plasterboard	800	0,015	0,25	2,5E-13	60000000000	1,66667E-11
OSB,int	50	0,015	0,13	4E-12	3750000000	2,66667E-10
PUR	60	0,112	0,024	3,33333E-12	33600000000	2,97619E-11
OSB,ext	30	0,015	0,13	6,66667E-12	2250000000	4,44444E-10
Tot	/	0,157	/	/	**99600000000**	1,00402E-11

Table 2.12 Water vapour resistance and permeance of the wall composed by the TEK system and a 15 mm plasterboard

	μ [-]	s [m]	λ [W/mK]	δ [kg/Pa*m*s]	Rᵥ [m^2*Pa*s/kg]	M [kg/m^2*Pa*s]
OSB,int	50	0,015	0,13	4E-12	3750000000	2,66667E-10
PUR	60	0,112	0,024	3,33333E-12	33600000000	2,97619E-11
OSB,ext	30	0,015	0,13	6,66667E-12	2250000000	4,44444E-10
Tot	/	0,142	/	/	**39600000000**	2,52525E-11

Table 2.13 Water vapour resistance and permeance of the wall composed only by the TEK system

It can be observed how here, unlike the thermal transmittance case, the plasterboard added to the wall makes a significant difference.

The vapour diffusion resistance factor of the plasterboard was taken from the webpage of a different British company, that produces plasterboards, called "British gypsum". It provides with the value of the total vapour resistance of a plasterboard named "Gyproc WallBoard Duplex", but since it obviously follows British literature the value is given in $\frac{MNs}{g}$ (Mega-Newton seconds per gram). Quoting[1]: *"The water vapour resistance of Gyproc WallBoard Duplex is 60MNs/g"*. One must multiply it times 10^9 to obtain the R_v value in $\frac{m^2*Pa*s}{kg}$ or, in order to convert it first into the vapour diffusion resistance factor μ, one has to multiply the original value times 0.2 $\frac{MNs}{g}$ (which is the value of vapour permeability of still air, equivalent to the δ_o term seen in (2.3)) and then divide it by the thickness of the layer in meters.

1 https://www.british-gypsum.com/technical-advice/faqs/063-what-is-the-vapour-resistance-of-gyproc-wallboard-duplex-plasterboard

2.2.4 Connections

To ensure continuity between one panel and the other there are two main elements that can be used; one provides higher performance in terms of structural resistance and the other provides better performance in terms of thermal insulation. The first one consists, as briefly described in paragraph 2.2, in a structural timber post, with a section of dimensions 100x112 mm. According to the BBA certificate, the timber to be used must be of class C14 or higher.
The second one consists in an insulated connection spline called "cassette", composed by the same materials the panel is made of, so OSB and PUR. Just as the timber post, its composite cross section is also 100 by 112 mm, where the 112 mm are 15 mm OSB, 82 mm PUR and then 15 mm OSB again. The two elements can be observed in detail in Figure 2.4.

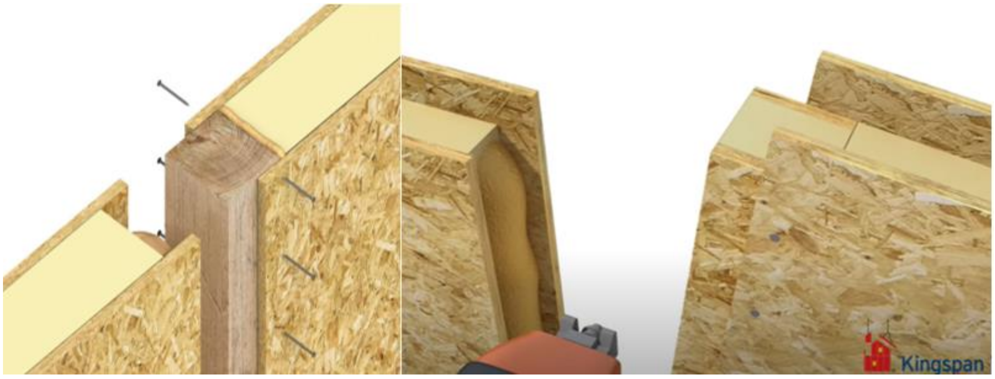

Figure 2.4 From left to right: connection through timber and cassette spline

To connect and fix the elements to each other different fastener types can be used, following the indications given by the company on the brochure of the TEK 142. The connection splines are fixed to the panel normally through the combination of nails and sealant, where according to the brochure the nails should be 2.8 mm x 63 mm galvanized ring-shank nails with a spacing of 50 mm for the timber posts and of 100 mm for the cassette splines, in both cases on both sides of the panel. The roof to wall connection, as well as the wall corner joints, are done with screws, the brochure suggests using 6 mm x 210 mm sparrennagel or 4.8 mm x 203 mm FastenMaster Headlok, with a spacing of 300 mm along the contact side. The details of the connection are shown in Figure 2.5, taken from the product brochure. For the roof to wall connection there are other options apart from that illustrated in Figure 2.5 depending on the structure; for instance, the geometry changes if there is a supporting beam. Nevertheless, the fastener suggested by the brochure is always the one described above and shown in Figure 2.5.

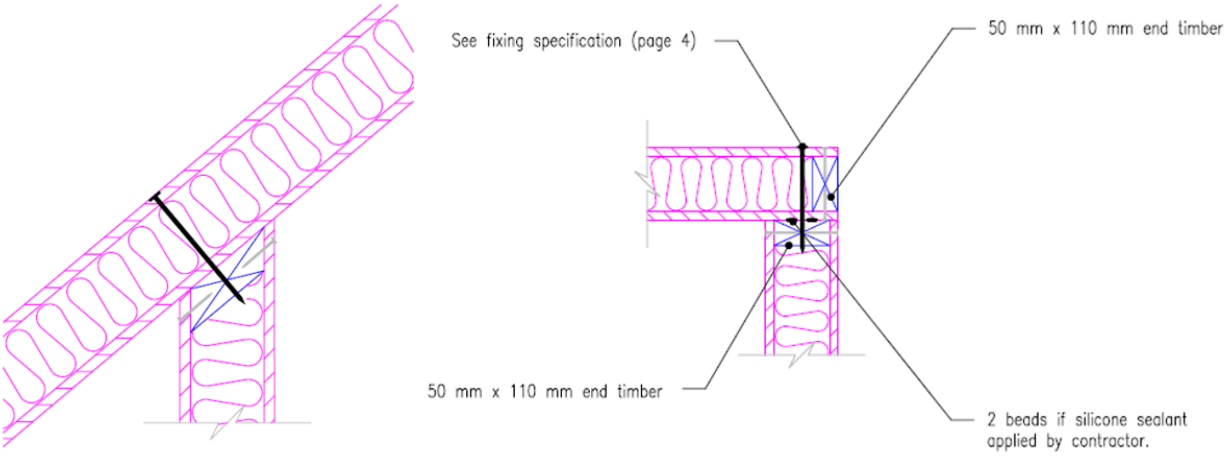

Figure 2.5 From left to right: roof to wall and wall to wall corner connection

As described in paragraph 2.1, the panels usually require edge timber elements to be "complete", in order to facilitate the assembly of the system. Those elements can clearly be seen in Figure 2.5 (on the left it is not specified, on the right it is the 50 mm 110 mm end timber element), in which also their importance is underlined; in fact, those elements have the function of giving more grip to the screw with respect to the normal panel elements, giving the whole connection more resistance. They are sealed to the rest of the panel and nailed to it with 2.8 mm x 63 mm galvanized ring-shank nails, to be placed on both sides of the panel every 50 mm. The roof ridge connection is done also with 6 mm x 210 mm sparrennagel or 4.8 mm x 203 mm FastenMaster Headlock screws; similarly to the roof to wall connection, the exact geometry can vary from case to case.

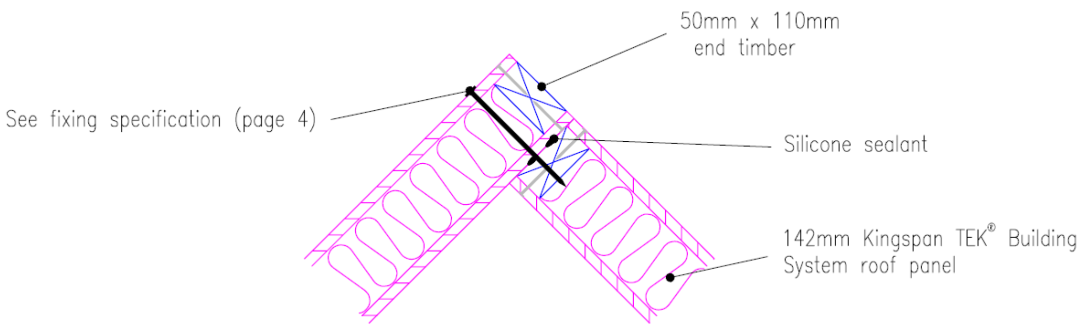

Figure 2.6 Ridge connection without support

Finally, the connection panel-foundation is the least specified in the product's brochure. A drawing of one possible solution is exposed in Figure 2.7; as can be seen, the system's bottom edge timber plate (50 mm x 112 mm) is fixed with nails to the (timber) soleplate of dimension 40 mm x 142 mm, which usually relies directly on the foundation. The suggested nail type to be used in this case according to the brochure is a 3.1 mm x 90 mm galvanized ring-shank nail, placed in two staggered rows along the plate with a spacing of 200 mm. Exact indications on the fixing of the bottom and soleplate system to the foundation are not given by the brochure, it states that, quoting: *"Specification should be in accordance with project structural engineers' recommendations based upon geography and project foundation substructure"* (pag 4). Although, Figure 2.7 suggests that some type of bolt fastener should be used.

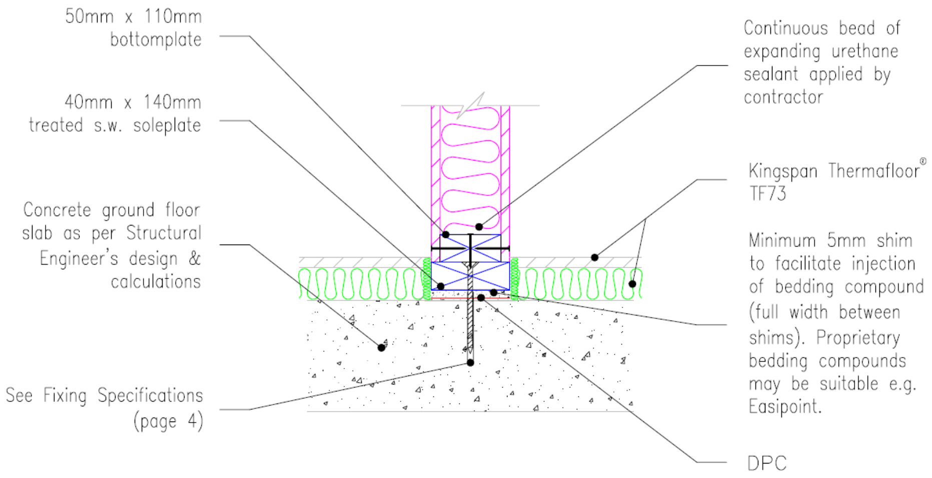

Figure 2.7 Panel-foundation connection

The exact indications given by the Kingspan company as they appear on the product's brochure can be seen in Table 2.14.

Application	Fastener Type	Spacing
Fixing soleplate or combined soleplate and bottomplate	Specification should be in accordance with project structural engineers' recommendations based upon geography and project foundation substructure	As per project structural engineers' recommendations
Panel straps to substructure / foundations	Specifications should be in accordance with project structural engineers' recommendations based upon geography and project foundation substructure	As per project structural engineers' recommendations
Fixing 50 mm x 110 mm bottomplates to soleplates	3.1 mm x 90 mm galvanized ring-shank nails	200 mm centres in two staggered rows
Fixing 15 mm x 100 mm OSB3 splines into *Kingspan* **TEK**® Building System panels	2.8 mm x 63 mm galvanized ring-shank nails	100 mm centres both sides of the panel
Fixing 50 mm x 110 mm bottomplates, headplates, end timbers, edge timbers into *Kingspan* **TEK**® Building System panels	2.8 mm x 63 mm galvanized ring-shank nails	50 mm centres both sides of the panel
Fixing 100 mm x 110 mm timber posts into *Kingspan* **TEK**® Building System panels	2.8 mm x 63 mm galvanized ring-shank nails	50 mm centres both sides of the panel
Fixing 110 mm x 150 mm bevelled headplate to *Kingspan* **TEK**® Building System panels	2.8 mm x 63 mm galvanized ring-shank nails	50 mm centres both sides of the panel
Fixing *Kingspan* **TEK**® Building System wall panels at corner joints	6.0 mm x 210 mm sparrennagel or 4.8 mm x 203 mm FastenMaster Headlok	Typically 300 mm centres for sparrennagel (4.0 mm dia. pre-drilled holes required.) Typically 350 mm centres for FastenMaster Headlok screws
Fixing *Kingspan* **TEK**® Building System roof sections at wall/floor junctions, ridge beams, intermediate purlins and gable walls	6.0 mm x 210 mm sparrennagel or 4.8 mm x 203 mm FastenMaster Headlok	Typically 300 mm centres for sparrennagel (4.0 mm dia. pre-drilled holes required.) Typically 350 mm centres for FastenMaster Headlok screws

Table 2.14 Fastener indications given by the TEK 142 brochure

2.2.5 Cost

The exact cost of the panel, which in literature is usually given in price per square meter, is not given by Kingspan. Some attempts were made, by sending emails to the company, to find out the commercial price of the panel, but those were never answered. However, after some research, it emerged that an average price of those kind of OSB type SIP panels is of usually around 50 Euro per square meter[2]. In addition, prices for the in paragraph 2.2.4 mentioned nail types i.e. 3.1 mm x 90 mm and 2.8 mm x 63 mm galvanized ring nails are very low (about some cents per nail[3]), but come in packs of usually around two thousand pieces which make it an expense of more or less 50 Euros in total. Screws are a little bit more expensive, in the specific case of 6 mm x 210 mm sparrennagel, the price can vary between 0,5 and 1 Euro[4] per piece depending on the manufacturer, but they also always come in bigger packs. Additional costs are related to the edge timber elements and to the sealants, where for both the price cannot be generally defined, since it strongly depends on the type of timber and sealing used.

2.3 The PANELO system

The second construction system that was studied on this thesis is quite innovative and produced by an Estonian company named "Panelo". It is, quoting the company's webpage[5]: *"[...] similar to SIP panels, but technically much more complex"*. The composition is, much alike the Kingspan panel, of three layers, although, in this case, the three layers consist also in three different materials.

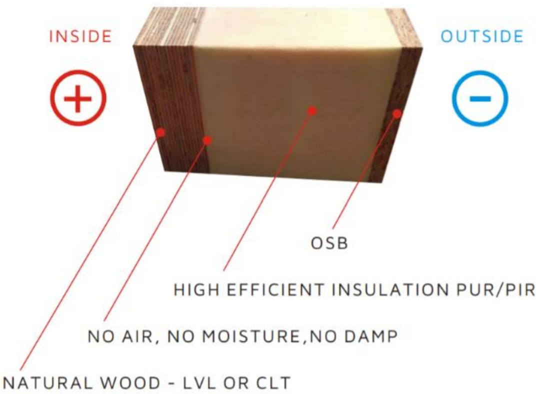

Figure 2.8 From left to right: Laminated Veneer Lumber, detail of Panelo panel

[2] http://www.acmepanel.com/sip-prices.asp
[3] https://www.toolstop.co.uk/dewalt-dnpt3190g12z-galvanised-plain-shank-timber-nails-3-1mm-x-90mm-box-of-2200/
[4] https://webshop.schachermayer.com/cat/de-IT/product/sparrennagel-6-0x210-verzinkt/104445152
[5] https://panelo.eu/panelo-wall-panels/

The insulation core is, like the TEK panel, in PUR material, while the outer sheets are in OSB on the inside and in LVL on the outside.

LVL, which of those three is the only not introduced material so far, stands for "Laminated Veneer Lumber"; it is a wood product consisting in multiple thin layers of "lumber" (North American English word for timber) assembled together with adhesives to form beams or two-dimensional boards. Differently from usual SIP panels, the two outer layers of the Panelo system don't have equal thickness; the LVL layer, already stiffer and stronger than the OSB layer, is also thicker. The exact thicknesses of the different layers depend in this case if the considered system is a roof or wall panel; the wall panel's layers have thicknesses 39 mm for the LVL, 145 mm for the PUR and 15 mm for the OSB, while the roof panels are composed with layers of thicknesses 27 mm LVL, 195 mm PUR and 15 mm OSB. As can be noticed, the wall panels have higher stiffness (more LVL), while the roof panels provide with better insulation (more PUR). The products' geometries are all given on the company's webpage Panelo.eu and shown here in Figure Table 2.15.

TYPE	LENGTH	WIDTH	HEIGTH	U-VALUE	TYPE	LENGTH	WIDTH	HEIGTH	U-VALUE
LVL(mm)-PUR(mm)-OSB(mm)	m	m	mm	W/(m²K)	LVL(mm)-PUR(mm)-OSB(mm)	m	m	mm	W/(m²K)
OW 39-145-15	max 12	max 3	199	0,157	Sample Panel C 27-195-15	max 12	max 3	237	0,12

Table 2.15 From left to right: Wall and roof panel geometry and U-values

Apart from the above described types, there are other systems both for roof with different geometries, which will not be analysed by this thesis.

2.3.1 Mechanical properties

Compared to the Kingspan company, Panelo provides with much less information about their product, especially for what concerns its strength and mechanical characteristics. This is probably due to the fact that their panel is more innovative, recent, and in general less studied by literature but in the first place also by the company itself. The weight of the system is given on Panelo's webpage in $\frac{kg}{m^2}$ for given thicknesses; however, the thickness values (Table 2.14) don't coincide with the values exposed in paragraph 2.3 and shown in Figure 2.13. This has been interpreted as a mistake done by the company, as the first value observable in Table 2.16 probably refers to the wall panel (39 mm LVL, 145 mm PUR, 15 mm OSB) and the last one to the roof panel (27 mm LVL, 195 mm PUR, 15 mm OSB).

Panel Type	Kg/m²
Exterior Wall - EW 39-170-21	45,32
Interior Wall - 45-170-15	45,90
Floor Panel - FP 21-195-15	39,06
Roof Panel RP 27-195-15	38,99

Table 2.16 System's unit mass in kg/m^2

No information is given about the system's strength and resistance, whether on the webpage nor on any uploaded certificate or brochure. In any case, an email address to contact is made public on the company's webpage, for anybody who's interested in the product and has questions to ask. This address was contacted for the first time around November of 2019, with the hope of obtaining some data, but without any success; the answer was that the panel hadn't been tested yet, therefore no information about its structural behaviour was available. Anyhow, some months later, in June 2020, the company was contacted again and the answer, as it was received, can be seen in Figure 2.9.

Hello Matthias,

Yes we do have test results. We have it for vertical load (simple measurement test; not conical) and PANELO panel 1,2x2,5 m took 32 tons,

before the first piece broke out of it. That means we can take for long turn load as 10 tons (with lots of weight in reserve).

The roof panels are also in the testing at the moment (1,2 x 6 meters), but no results yet.

Best regards,

Priit Vahe

Figure 2.9 Email by the Panelo company

From the data given by this e-mail, the design axial design resistance of the wall panel $N_{Rd,d}$ was calculated. To do this, initially the value in kg was converted in kN per meter by dividing it by the length of application of the load (1.2 meters), and multiplying it times the gravity acceleration 9,81 $\frac{m^2}{s}$.

Panel dimentions		Carried load			
width [m]	height [m]	kg	kg/m^2	kN/m^2	kN/m
1,2	2,5	32000	134003,3501	1314,572864	261,6

Table 2.17 Different unit measure conversions of the applied load

The value showed on the right in kN/m represents the characteristic strength $N_{Rd,k}$ of the panel.

The next steps were done following the CNR-DT-206-R1-2018 (the Italian code specifically written for wood design). In fact, according to it:

$$X_d = \frac{k_{mod} * X_k}{\gamma_m} \tag{2.7}$$

where X_d is the design property of a material, X_k the characteristic property, k_{mod} is a modification factor taking into account the effect, on the resistance parameters, both of the duration of the load and of the moisture content of the structure and γ_m is a partial factor for a material property.

For what concerns both the γ_m and the k_{mod} values, following other indications received per email from the Panelo company, the ones appearing on the CNR code referring to LVL wood were taken. Those are illustrated in the tables below.

Materiale	Riferimento	Classe di servizio	Classe di durata del carico				
			Permanente	Lunga	Media	Breve	Istantanea
Legno massiccio Legno lamellare incollato Microlamellare (LVL)	EN 14081-1 EN 14080 EN 14374,EN 14279	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

Table 2.18 kmod factor for different materials

The service class depends on the moisture content of the material and was in this calculation considered to be 3.

In the case of LVL wood, an interesting observation is that the values of γ_m given by the Italian NTC 2018 and those given by the CNR and the EC5 don't coincide, as can be seen in Table 2.19; however, the value indicated by the CNR were taken eventually.

Stati limite ultimi	Colonna A γ_M
combinazioni fondamentali	
legno massiccio	1,50
legno lamellare incollato	1,45
pannelli di tavole incollate a strati incrociati	1,45
pannelli di particelle o di fibre	1,50
LVL, compensato, pannelli di scaglie orientate	1,40
unioni	1,50
combinazioni eccezionali	1,00

Stati limite ultimi	γ_M
- combinazioni fondamentali	
legno massiccio	1.50
legno lamellare incollato	1.45
pannelli di particelle o di fibre	1.30
LVL, compensato, OSB	1.20
unioni	1.30
- combinazioni eccezionali	1.00

Table 2.19 From left to right: partial safety factor according to the NTC 2018 and to the CNR

The resulting design values can be observed in Table 2.20.

N_{Rd} (kN/m)				
Permanent	**Long**	**Medium**	**Short**	**Instantaneous**
109	119,9	141,7	152,6	196,2

Table 2.20 NRd design values

Doing an inverse procedure to find out the long term load in tons (in order to do a comparison with the 10 tons mentioned in the above showed email), it results being equal to 14.67 tons, which is plausible considering the “lots of weight in reserve” comment in the email (figure 2.10). Therefore, in order to have a safer approach, the same calculation was done again but taking this time as a reference value 10000 kg for the long term load and deriving from that all the others. The final results are presented in Table 2.21.

N_{Rd} (kN/m)				
Permanent	**Long**	**Medium**	**Short**	**Instantaneous**
74,3181818	81,75	96,61364	104,0454545	133,7727273

Table 2.21 NRd design values second iteration

Finally, the company also provides with no data about the stiffness of the panel. However, average elasticity modulus values for the single components of the system can be found in literature; given the background, normal values of Elasticity modulus along the main axis for OSB/3 boards is of the order of magnitude of 3-5 GPa[6], while a LVL board has usually higher stiffness. In the particular case of the Panelo system, assuming the OSB is of type 3 (no detailed information is given in regard) and of density 650 kg/m^3 and the PUR is of the same type as that of the Kingspan TEK panel (density equal to 35 kg/m^3), considering the total system’s unit weight shown in Table 2.16 the LVL used is most probably a heavy type, of density equal or greater than 700 kg/m^3. For this sort of LVL wood boards, the normal E values on the board’s major axis are of the order of magnitude of 15 GPa[7].

2.3.2 Thermal performance

The Panelo.eu website provides with the values of thermal transmittance U of the roof and wall panel given in $\frac{W}{m^2K}$, shown in Table 2.15 in paragraph 2.3. The exact thermal conductivities of each involved material are not furnished, however datasheet documents provided by LVL researches[8] suggest that the thermal conductivity λ of relatively heavy LVL wood (around 700 kg/m^3) is of 0.17 $\frac{W}{mK}$. The LVL density approximation was derived from the values in Table 2.16, assuming the OSB and PUR

[6] https://www.swisskrono.pl/en/mdb/OSB-boards/Technical-Data, https://baldolegnami.it/media/osb.pdf
[7] docecity.com_smartlvl-18-design-guide-lvl-18.pdf, Allgemeine Bauartgenehmigung BauBuche Platte Z-9.1-838.pdf
[8] https://puutuoteteollisuus.fi/images/pdf/LVL_bulletin_eng.pdf, http://scholar.google.it/scholar_url?url=http://ojs.cnr.ncsu.edu/index.php/BioRes/article/download/BioRes_04_2_0756_Uysal_KO_Thermal_Cond_Laminated_Veneer/376&hl=it&sa=X&ei=9DB7X8StDtKsmwGBsoywCg&scisig=AAGBfm09ekDPg_9jyxAmsXdWb_VGIC7g_w&nossl=1&oi=scholarr

materials have the same densities as in the Kingspan TEK case. Knowing the LVL conductivity and supposing the OSB and PUR have the same conductivities as defined in paragraph 2.2.2, Table 2.5, a calculation of an approximated thermal transmittance of the wall panel was performed for a comparison with the data provided by the company, using formula (2.2). Since like Kingspan, also Panelo suggests adding a plasterboard layer to the system (see Figure 2.11, paragraph 2.3.3), also in this case the calculation was made both with the plasterboard and without.

WALL			
	s [m]	λ [W/mK]	$U_{PANELO (WALL)}$ [W/(m^2*K)]
Plasterboard	0,015	0,25	
LVL	0,039	0,17	
PUR	0,145	0,024	
OSB	0,015	0,13	0,151252452

Table 2.22 Example of calculation of the thermal transmittance U of the wall panel with 15 mm plasterboard

WALL			
	s [m]	λ [W/mK]	$U_{PANELO (WALL)}$ [W/(m^2*K)]
LVL	0,039	0,17	
PUR	0,145	0,024	
OSB	0,015	0,13	0,152637662

Table 2.23 Example of calculation of the thermal transmittance U of the wall panel without 15 mm plasterboard

As the Panelo product differs in dimensions and characteristics between wall and roof panel, a same procedure was done for the roof case.

ROOF			
	s [m]	λ [W/mK]	$U_{PANELO (ROOF)}$ [W/(m^2*K)]
Plasterboard	0,015	0,25	
LVL	0,027	0,17	
PUR	0,195	0,024	
OSB	0,015	0,13	0,115952675

Table 2.24 Example of calculation of the thermal transmittance U of the roof panel with 15 mm plasterboard

ROOF			
	s [m]	λ [W/mK]	$U_{PANELO (ROOF)}$ [W/(m^2*K)]
LVL	0,027	0,17	
PUR	0,195	0,024	
OSB	0,015	0,13	0,116765028

Table 2.25 Example of calculation of the thermal transmittance U of the roof panel without 15 mm plasterboard

Also in this case, as it was for the TEK system, the resulting transmittance is not strongly dependent on whether or not the plasterboard is added, and the resulting value are all approximately similar to those seen in Table 2.15.

2.3.3 Vapour diffusion and condensation risk

There is not much to be said about the data provided by the company regarding the behaviour of the product in relation to vapour diffusion and condensation. In fact, Panelo don't provide with any sort of information to that respect, whether on their webpage nor on any other uploaded file. Anyhow, an approximated calculation of a hypothetical vapour diffusion resistance R_v and permeance M was performed, following the same steps described in paragraph 2.2.3.

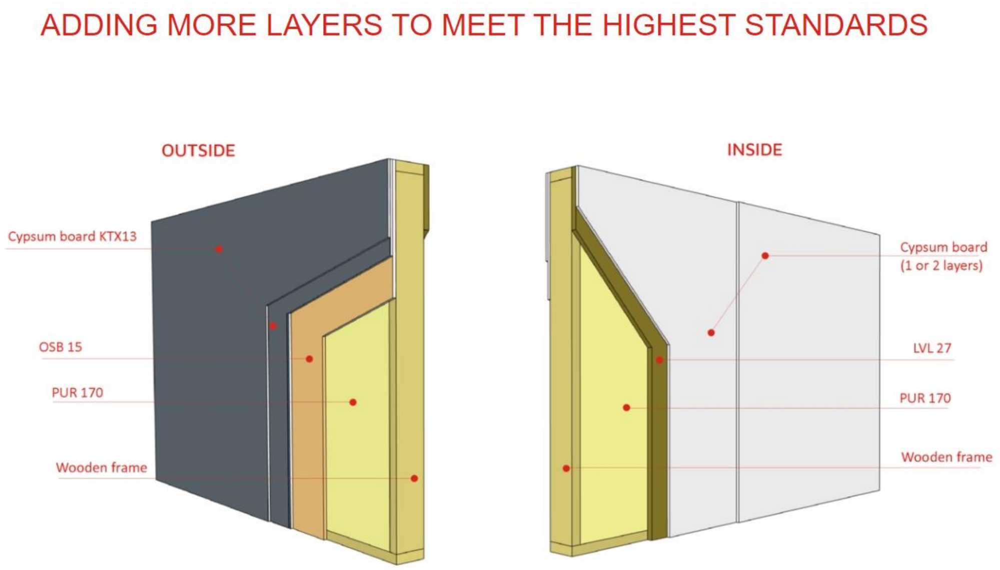

Figure 2.10 Possible solution of panel and additional layers to improve the panel's performance

The value of the LVL vapour diffusion resistance factor μ was defined following the indications given by an LVL wood producer named Metsawood[9], who provide with different values depending on the moisture content of the material and on the relative humidity conditions; however, it was assumed μ=70, which is the value the material assumes when tested at 23°C - 50/93RH% and to be applied when the mean relative humidity across the material is greater than or equal to 70 %. The vapour diffusion resistance factors of the OSB and the PUR were assumed to be equal to those defined for the Kingspan TEK system. Before exposing the results, a detail already mentioned in paragraph 2.3.2 important to underline, is that the producers suggest the use of Plasterboard in addition to the other components of the panel (Figure 2.10); therefore the calculation was done again both in the case of a Plasterboard internally added and not added to the roof panel and wall panel system.
The results of the calculations for the wall and roof panel are shown in the tables below. For the Plasterboard, the same characteristics used in paragraph 2.2.3 have been adopted.

[9]https://www.metsawood.com/global/Tools/MaterialArchive/MaterialArchive/Kerto-manual-lvl-moisture-behaviour.pdf

WALL						
	μ [-]	s [m]	λ [W/mK]	δ [kg/Pa*m*s]	R_v [m^2*Pa*s/kg]	M [kg/m^2*Pa*s]
Plasterboard	800	0,015	0,25	2,5E-13	60000000000	1,66667E-11
LVL	70	0,039	0,17	2,85714E-12	13650000000	7,32601E-11
PUR	60	0,145	0,024	3,33333E-12	43500000000	2,29885E-11
OSB,ext	30	0,015	0,13	6,66667E-12	2250000000	4,44444E-10
Tot	/	0,214	/	/	**1,194E+11**	8,37521E-12

Table 2.26 Water vapour resistance and permeance of the wall composed by the Panelo wall system and a 15 mm plasterboard

WALL						
	μ [-]	s [m]	λ [W/mK]	δ [kg/Pa*m*s]	R_v [m^2*Pa*s/kg]	M [kg/m^2*Pa*s]
LVL	70	0,039	0,17	2,85714E-12	13650000000	7,32601E-11
PUR	60	0,145	0,024	3,33333E-12	43500000000	2,29885E-11
OSB,ext	30	0,015	0,13	6,66667E-12	2250000000	4,44444E-10
Tot	/	0,199	/	/	**59400000000**	1,6835E-11

Table 2.27 Water vapour resistance and permeance of the wall composed only by the Panelo wall system

ROOF						
	μ [-]	s [m]	λ [W/mK]	δ [kg/Pa*m*s]	R_v [m^2*Pa*s/kg]	M [kg/m^2*Pa*s]
Plasterboard	800	0,015	0,25	2,5E-13	60000000000	1,66667E-11
LVL	70	0,027	0,17	2,85714E-12	9450000000	1,0582E-10
PUR	60	0,195	0,024	3,33333E-12	58500000000	1,7094E-11
OSB,ext	30	0,015	0,13	6,66667E-12	2250000000	4,44444E-10
Tot	/	0,252	/	/	**1,302E+11**	7,68049E-12

Table 2.28 Water vapour resistance and permeance of the roof composed by the Panelo roof system and a 15 mm plasterboard

ROOF						
	μ [-]	s [m]	λ [W/mK]	δ [kg/Pa*m*s]	R_v [m^2*Pa*s/kg]	M [kg/m^2*Pa*s]
LVL	70	0,027	0,17	2,85714E-12	9450000000	1,0582E-10
PUR	60	0,195	0,024	3,33333E-12	58500000000	1,7094E-11
OSB,ext	30	0,015	0,13	6,66667E-12	2250000000	4,44444E-10
Tot	/	0,237	/	/	**70200000000**	1,4245E-11

Table 2.29 Water vapour resistance and permeance of the roof composed only by the Panelo roof system

As for the Kingspan TEK 142 system, the use of a vapour resistant Plasterboard makes a significant difference.

2.3.3 Connections

Details of the connection types to be used to assemble together the components of the panel as well as the panels to each other are provided by three main documents; a drawing of the foundation to panel and panel to roof connection, a drawing of the panel wall to wall and wall to wall corner connection, and finally a step by step installation guide of the system. The panel requires an edge timber element of unspecified thickness and of width equal to the insulation thickness (so 145 mm for the wall panel and 195 mm for the roof panel). The wall-foundation connection is done by gluing the system to a soleplate either with wood sealing ribbon or with glue and PU foam or with rubber sealing ribbon and PU gun foam. If done with wood sealing ribbon, the "PANELO installation guide" specifies that *"Product like ISOVER SK-C 20x200 mm or such"* (pag 5) is to be used. Details of a possible solution to adopt are shown in Figure 2.11.

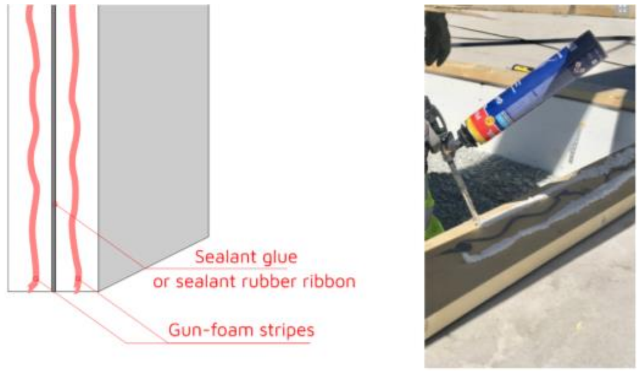

Figure 2.11 Detail of the gluing between panel and edge and soleplate

Additionally, the panel is fixed to the foundation with screws of dimensions, as suggested by the drawing, 8 mm x 150 mm and spacing 300 mm. The wall to roof connection is done in a very similar way by placing gluing materials between the wall and the roof as described for the panel-soleplate connection and adding fixing fasteners of dimensions 8 mm x 150 mm and spacing 300 mm. A graphic representation of what explained is illustrated in Figure 2.12. The red lines that can be seen at the connections represent the glue.

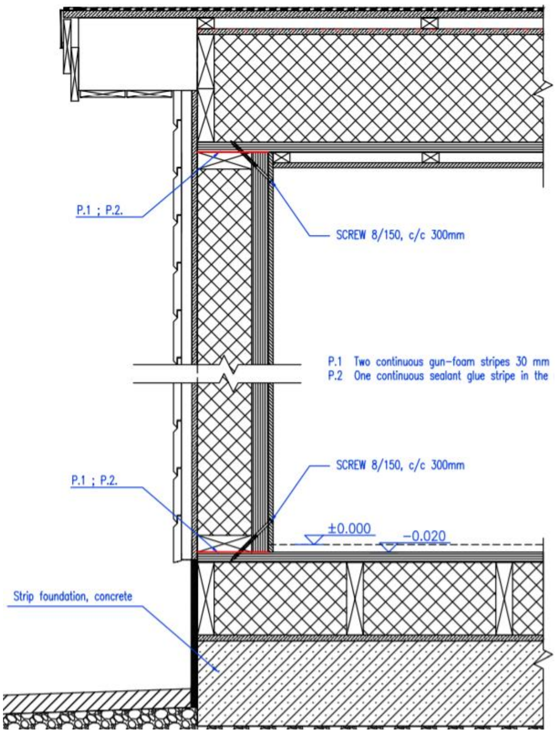

Figure 2.12 Foundation-wall and wall-roof connection

To connect the walls to each other, sealant glues and gun foam stripes are used as in the case of the panel to foundation and panel to roof joint and fixing screws are employed. Although, in this respect, the exact screw dimensions and spacings are not specified by the company but are to be defined *"according to load calculation"* (see Figure 2.13).

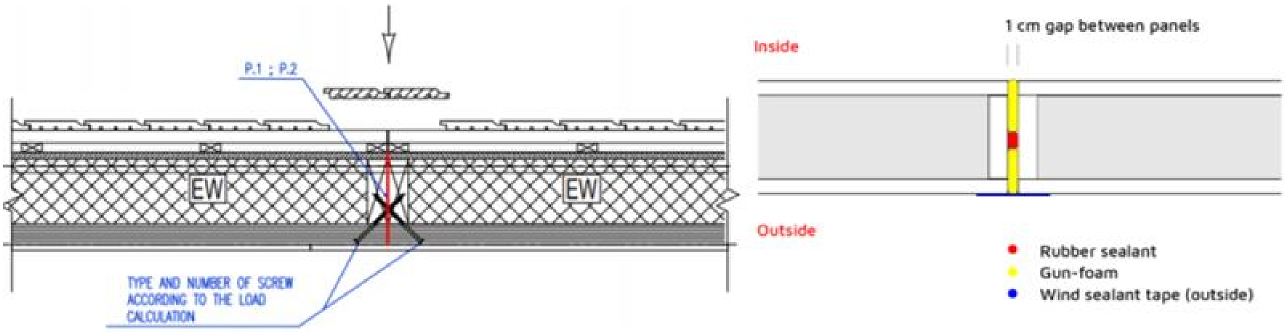

Figure 2.13 Details of the wall to wall connection

For what concerns the wall corner joints, everything that was said in the previous case counts. The only difference that can be spotted in Figure 2.14, is that in the side wall connection two fasteners are involved that are crossing each other, while in the corner wall connection (for obvious geometrical reasons) only one is.

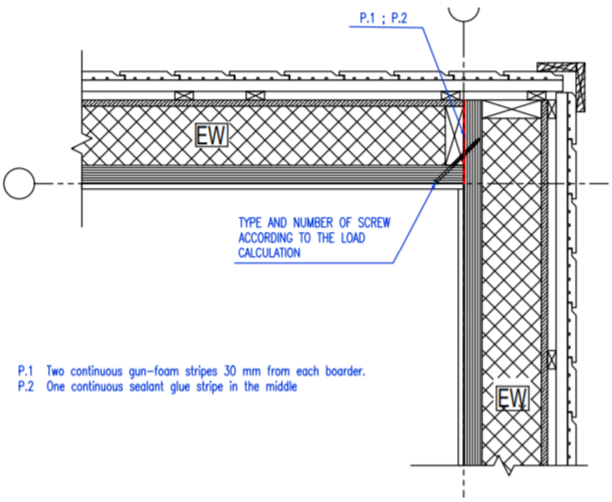

Figure 2.14 Wall to wall corner joint

2.3.3 Cost

The cost of the panel is not given on the company's webpage; however, similarly to other information exposed in the previous paragraphs, also this was eventually obtained per email from a Panelo representant, shown as it was received in Figure 2.15.

Hello,

Prices for the walls are in the range of 60-90 €/m2 depending on U value.

Roofs about 120-150 €/m2 depending on span and U value.

Kind regards,

Priit Vahe

P A N E L O / Wooden Sandwich Panels
+372 50 19 799 / priit.vahe@panelo.eu / www.panelo.eu
WAMP GROUP Ltd / Piloodi tee 2, Soodevahe, Rae vald, Estonia, EU

Figure 2.15 E-mail listing the prices for the Panelo panels

Furthermore, considering the cost of the connections, the price of 8/150 screws is of around 1.5 Euro per piece[10], while the price for a gluing material as ISOVER SK-C 20x200 is of more or less 2.50 Euro per meter[11].

2.4 Initial comparison

Doing a comparison of the presented data and the results obtained in the previous paragraphs for the respective panels, different considerations can be made. What first comes to the attention is the difference in thickness of the two systems, with the Panelo panels being thicker both in the insulation core and in the two outer sheets; moreover, LVL is, as exposed in paragraph 2.3.1, a generally stiffer and stronger material with respect to OSB. It can be deduced that the combination of these two factors (greater thickness and higher stiffness) will probably result, even if the exact bending and shear rigidity of the Panelo system are not known, in the Panelo panel (both the roof and the wall one) being in total stiffer and heavier than the Kingspan TEK 142 panel.
Additionally, greater thickness of the PUR core allows the Panelo system to also have better performances than the TEK system in terms of thermal insulation, as can be noticed comparing the tables of paragraphs 2.2.2 and 2.3.2, and the slightly higher vapour resistance of the LVL compared to the OSB (paragraphs 2.2.3 and 2.3.3) allows it to have better performance in terms of preventing interstitial condensation. All these advantages manifest themselves in the cost of the two panels, which is higher for the Panelo one than for the Kingspan one. Finally, as will have been noticed, much less information is known about the Panelo panels than about the Kingspan panels, due obviously to the fact that they are a more recent product. This represented a problem for the intentions of the work, and a lot of assumptions had to be made when creating and analysing the structural model of the panel.

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[10] https://www.westfieldfasteners.co.uk/Wood-Screws/Chipboard-Wood-Screw-Torx-Countersunk-8x150-A2-Stainless.html
[11] https://www.stark-suomi.fi/fi/eristyskaista-isover-sk-c-20x200-mm-14-m

2.5 Definition of the loads on the structure

The first and second dimension of the BIM methodology, that were first seen in paragraph 1.3.2, were basically applied to the study case and exposed in chapter 1 and in the first paragraphs of this second chapter of this thesis. In fact, in chapter 1 the first dimension is described, by introducing the function, the order of magnitude and the setting of the facility, while the 2D BIM is seen mainly in this second chapter with the description of the involved construction systems, but also partially in chapter 1 with the two dimensional CAD drawings; however, to complete the second dimension for what is the interest and purpose of this thesis, which is aiming to do a more structural study of the construction than a purely graphical one, an important information is missing which is the definition of the loads applied on the building, given the environment it is placed in.
Since the construction is placed in the mountains, the main expected loads are derived from the strong wind, the big snow falls and the freezing cold temperatures the facility is subjected to. Therefore, those were the loads taken in consideration, also under specific request of the Leap factory company. Their calculation was performed following the NTC 2018 and their definition is illustrated in the following paragraphs.

2.5.1 Wind

The Italian norm defines the basic wind velocity based on a certain altitude as:

$$v_b = v_{b,0} * c_a \qquad (2.8)$$

where $v_{b,0}$ is the basic wind velocity at sea level, assigned based on the zone the structure is located in (see Figure 2.16) and c_a is the altitude factor. The basic wind velocity is the mean value for 10 minutes, at 10 m height from the ground on a flat and homogeneous terrain of exposure category II (see Table 2.30) referred to a return period $T_R = 50$ years. Furthermore, the reference wind velocity, which depends on the design return period of the project, is calculated through the formula:

$$v_r = v_b * c_r \qquad (2.9)$$

being c_r the return coefficient, equal to 1 if $T_R = 50$.
However, the altitude factor c_a is on the norm only defined for altitudes equal or lower than 1500 m amsl, while for higher altitudes *"...the values of the reference velocity can be obtained through appropriate documentation or statistical surveys..."*. Therefore, following indications given by Leap Factory, it was defined:

$$v_r = 55 \ {}^{m}/_{s}.$$

Figure 2.16 Map of the Italian wind zones. The red zone is the one of interest for the study case

Knowing the reference velocity, one can determine the equivalent static action in the form of the reference velocity pressure:

$$q_r = \frac{1}{2}\rho v_r^2 \qquad (2.10)$$

in which ρ represents the air density, conventionally assumed to be constant equal to 1.25 $\frac{kg}{m^3}$. Finally, the wind pressure is given by the relation:

$$p = q_r * c_e * c_p * c_d \qquad (2.11)$$

where:

c_e is an exposure coefficient, to be calculated according to the category of exposure and the height of the structure,

c_p is a pressure coefficient, which depends on the shape and inclination of the surface hit by the wind,

c_d is a dynamic coefficient, to be assumable in this case equal to 1 (according to paragraph 3.3.9. of the code),

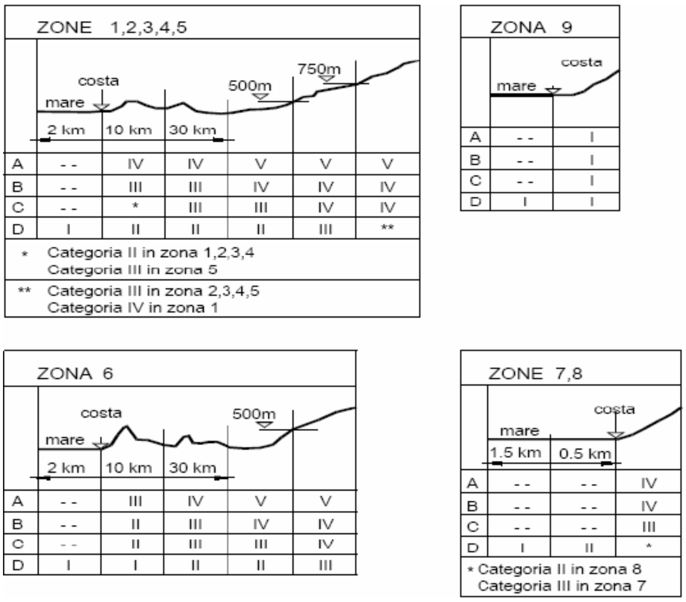
and the wind tangential pressure:

$$p_f = q_r * c_e * c_f \qquad (2.12)$$

being c_f a friction coefficient, in this case equal to 0,01[12].

The category of exposure can be defined thanks to Table 2.30; in the case of the Leap bivouac, the category of exposure is IV.

[12] http://biblus.acca.it/calcolo-del-vento-sulle-costruzioni-parte-1/



ZONE 1,2,3,4,5

A	- -	IV	IV	V	V	V
B	- -	III	III	IV	IV	IV
C	- -	*	III	III	IV	IV
D	I	II	II	II	III	**

* Categoria II in zona 1,2,3,4
Categoria III in zona 5

** Categoria III in zona 2,3,4,5
Categoria IV in zona 1

ZONA 9

A	- -	I
B	- -	I
C	- -	I
D	I	I

ZONA 6

A	- -	III	IV	V	V
B	- -	II	III	IV	IV
C	- -	II	III	III	IV
D	I	I	II	II	III

ZONE 7,8

A	- -	- -	IV
B	- -	- -	IV
C	- -	- -	III
D	I	II	*

* Categoria II in zona 8
Categoria III in zona 7

Table 2.30 Classification of the categories of exposure

The pressure coefficient c_p consists in both an internal component, $c_{p,i}$ and an external one, $c_{p,e}$ and is to be determined for every surface composing the building singularly, based on the direction of the wind and on the geometry of the surface. For the sake of this thesis, only the graphical representation of the case of interest for the studied building will be shown (Figure 2.17).

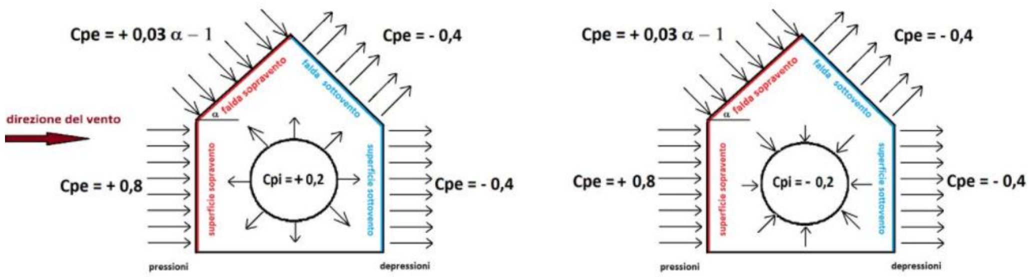


Figure 2.17 Definition of the pressure coefficient

Ultimately, $c_p = c_{p,e} + c_{p,i}$, with $c_{p,i}$ being positive or negative based on what begets the heavier combination.

The described procedure was applied and the wind pressure applied on each surface of the structure was calculated, with the wind coming from direction x+,x-,y+ and y-. The result was the definition four load cases, Wind 1, Wind 2, Wind 3 and Wind 4, which, as well as the tangential wind pressure p_f, are illustrated in the tables below.

q_r [N/m^2]	ρ [kg/m^3]	$c_e(z)$	k_r	z [m]	z_0 [m]
1890,625	1,25	1,62	0,22	5	0,3

z_{min} [m]	$c_e(z_{min})$	c_t	c_f	c_d	c_f	p_f [N/m^2]
8	1,62	1	0,01	1	0,01	30,628125

Table 2.31 Calculation of the reference wind pressure and the tangential wind pressure

x+ [WIND1]				y+ [WIND2]		
Superficie	c_p	p [N/m^2]		Superficie	c_p	p [N/m^2]
1	1	3062,8125		1	-0,6	-1837,6875
2	0,52	1592,6625		2	0,52	1592,6625
3	-0,6	-1837,6875		3	0,22	673,81875
4	-0,6	-1837,6875		4	-0,6	-1837,6875
5	-0,6	-1837,6875		5	-0,6	-1837,6875
6	-0,6	-1837,6875		6	1	3062,8125

x- [WIND3]				y- [WIND4]		
Superficie	c_p	p [N/m^2]		Superficie	c_p	p [N/m^2]
1	-0,6	-1837,6875		1	-0,6	-1837,6875
2	-0,6	-1837,6875		2	0,52	1592,6625
3	0,22	673,81875		3	0,22	673,81875
4	1	3062,8125		4	-0,6	-1837,6875
5	-0,6	-1837,6875		5	1	3062,8125
6	-0,6	-1837,6875		6	-0,6	-1837,6875

Table 2.32 Calculation of the wind pressures

inclinazione [°]	superficie 2	superficie 3
α	44	34

Table 2.33 Inclination of the roof surfaces

Figure 2.18 shows the numbering of the various surfaces of the building and the direction of the x and y axis.

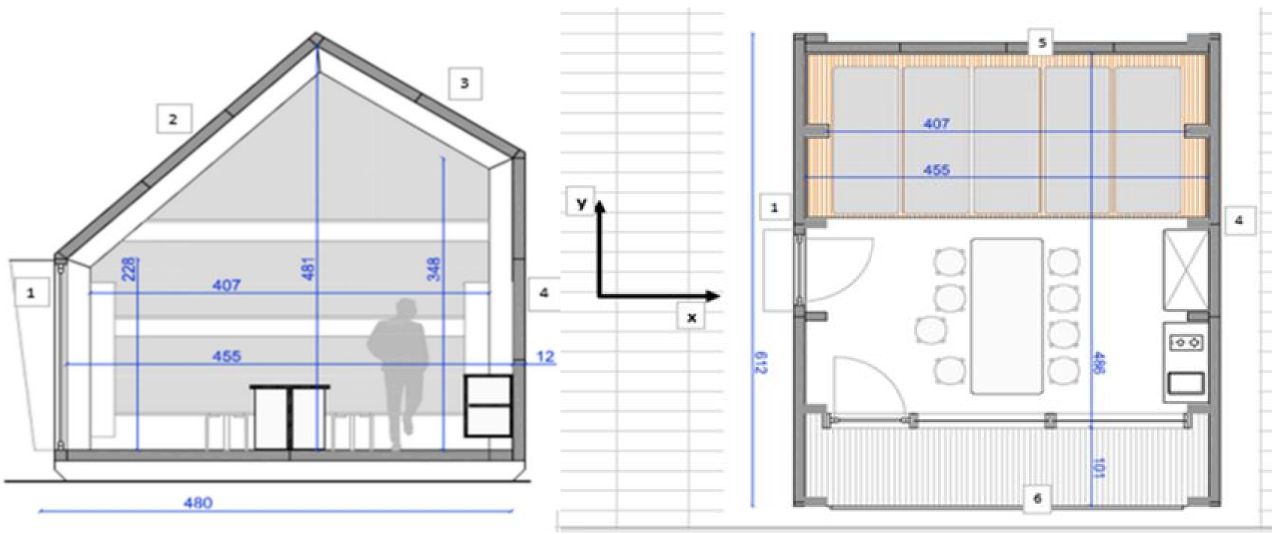


Figure 2.18 Geometry of the structure

2.5.2 Snow

The load caused by snow on the roof is determined, according to the NTC 2018, through the relation:

$$q_s = q_{sk} * \mu_i * C_E * C_t \qquad (2.13)$$

being:

q_{sk} the reference value of snow load on the ground, that depends on the zone the structure is located in,

μ_i the shape coefficient of the roof,

C_E the coefficient of exposition, taking into account the specific characteristics of the area surrounding the building. It is in this case assumed to be equal to 1.1 for heavier combination (following indications given in paragraph 3.4.4. of the NTC 2018).

C_t the thermal coefficient, in this case equal to 1 (according to paragraph 3.4.5. of the NTC 2018).

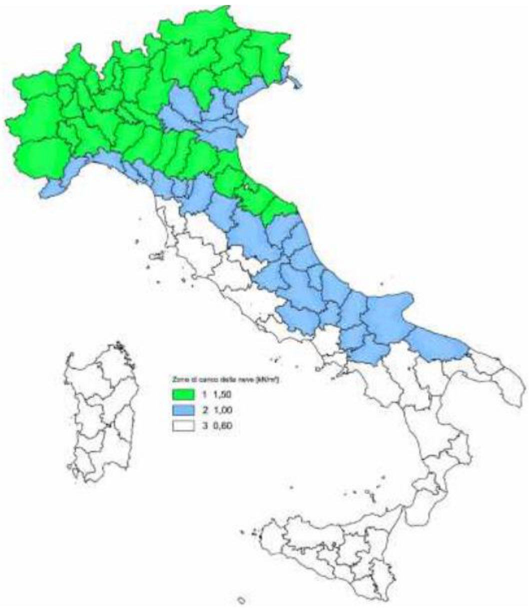


Figure 2.19 Map of the Italian snow zones. The Leap bivouac is located in the green zone 1

Similarly to the case of the wind, also for this calculation the component of the reference value q_{sk} is defined in the norm only for altitudes until 1500 m amsl. Under indications from Leap Factory, it was defined

$q_s = 11{,}88\ {}^{kN}/_{m^2}$,

which is equivalent to about 6 m^3 of snow per square meter of the roof, assuming for the snow a density of 200 $\frac{kg}{m^3}$.

The shape coefficient μ_i depends on the inclination of the roof surface, and has to be applied in (2.13) according to the indications given in Figure 2.20, which refers to a double-pitched roof, as is the case of the studied bivouac.
Equation (2.13) was employed with the μ_i values of the three different cases illustrated in Figure 2.20 to create three load cases, Snow 1, Snow 2 and Snow 3, whose values are exposed in Table 2.34. The numbering of the surfaces is the same as that seen in Figure 2.18.

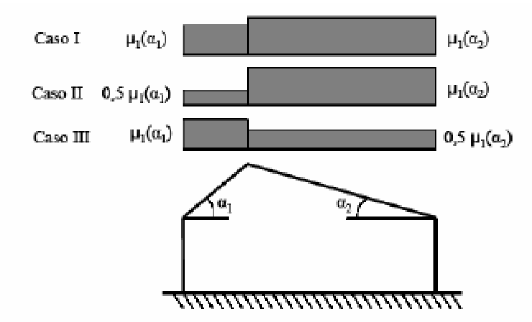


Figure 2.20 Load conditions for a double-pitched roof

						CASE 1-SNOW1	CASE 2-SNOW2	CASE 3-SNOW3
q_{sk} [kN/m^2]	a_s [m]	C_E	C_t	Superficie	μ_i	q_s [kN/m^2]	q_s [kN/m^2]	q_s [kN/m^2]
11,88088274	2000	1,1	1	2	0,426666667	5,576094301	2,788047151	5,576094301
				3	0,693333333	9,061153239	9,061153239	4,53057662

Table 2.34 Calculation of snow load cases

2.5.3 Temperature

For the action on the construction caused by the effect of temperature, the Italian code provides formulas to calculate the maximum and minimum outside air temperature, based on the zone the studied structure is located in. For zone 1, which is that of interest for the studied structure, the maximum temperature is defined as:

$$T_{max} = 42 - 6 * \frac{a_s}{1000} \qquad (2.14)$$

and the minimum:

$$T_{min} = -15 - 4 * \frac{a_s}{1000} \qquad (2.15)$$

where a_s is the elevation of the building above mean sea level in meters.
Being the defined elevation of the facility $a_s = 3000$ m amsl, the resulting values where:

$T_{max} = 24\ °C$,

and

$T_{min} = -27\ °C$.

Figure 2.21 Map of the Italian temperature zones. The zone of interest for the Leap bivouac is zone 1

These calculated temperatures can potentially cause stresses in the structure in two main ways; one is through a uniform temperature variation, which comes to happen from one season to another, in which the whole material composing the construction is subjected to a homogeneous cooling or warming, based on the season, and a consequential shrinkage (if it's cooling down) or expansion (if it's warming up), and will be referred to as ΔT_u. The second one is generated through a temperature difference between the inside and the outside environment of the structure, which is mostly observable in a heated building in the winter; in this case, the deformations within the material itself are different, creating a so called "butterfly" shape; the fibres subjected to warming will elongate while those subjected to cooling will shorten. It will be here referred to as ΔT_M.
The seasonal temperature variation is to be computed with the formula:

$$\Delta T_u = T - T_0, \qquad (2.16)$$

being T the mean actual temperature and T_0 the initial temperature when the structural element is constrained. Following documentation[13], T_0 was assumed to be equal to 10°C.
In the second case, the temperature is assumed to vary linearly within the structural element, as shown in Figure 2.22, and the temperature difference is determined through the relation:

$$\Delta T_m = T_{sup,int} - T_{sup,est}, \qquad (2.17)$$

in which $T_{sup,int}$ is the temperature of the internal surface and $T_{sup,est}$ of the outer one. They depend on the inner and outer air temperature and on the thermal properties of the material, in the case of the studied panels defined in paragraphs 2.2.2 and 2.3.2. The outer air temperature is that defined in equations (2.14) and (2.15) and the inner air temperature can be assumed to be of 20°C.

[13] https://eurocodes.jrc.ec.europa.eu/doc/WS2008/Holicky_Markova_2008.pdf

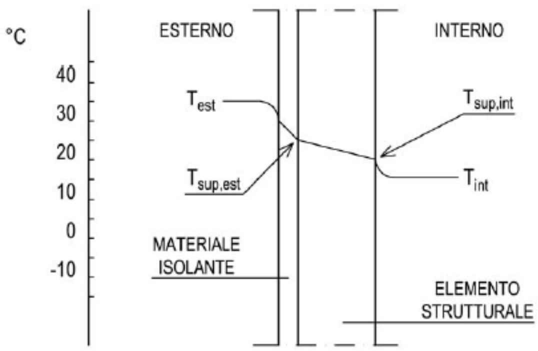


Figure 2.22 Temperature trend within the structural components

For both cases, one must consider the contribution of the solar radiation on the temperature of element, which can be evaluated according to Table 2.35.

Stagione	Natura della superficie	Incremento di Temperatura	
		superfici esposte a Nord-Est	superfici esposte a Sud-Ovest od orizzontali
Estate	Superficie riflettente	0 °C	18 °C
	Superficie chiara	2 °C	30 °C
	Superficie scura	4 °C	42 °C
Inverno		0 °C	0 °C

Table 2.35 Contribution of the solar radiation

If a difference of temperature between the inner and outer surface is present, the T term of equation (2.16) can be determined as the average of the two values.

Following the explained procedure, the uniform temperature variations ΔT_u for the summer and winter season as well as the linear temperature difference components ΔT_m were computed for the different surfaces of the building, according to their exposure. Since the studied construction is that of a bivouac, in which most of the times there is no heating system, four load cases were generated, TEMP1, TEMP2, TEMP3 and TEMP4; the first two for the summer and winter season not considering any heating source, and the other two for the summer and winter season considering a heating source that brings the inner environment to 20°C. The results are shown in Tables 2.37 and 2.38, while the input values are exposed in Table 2.36; the thermal transmittance, as well as the heat transfer coefficients, are those defined in paragraph 2.2.2.

a_s [m]	T_{max} [°C]	T_{min} [°C]	T_0 [°C]	U [W/(m^2*K)]	h,e [W/m^2K]	h,i [W/m^2K]
3000	24	-27	10	0,195219622	25	8

Table 2.36 Input data for the calculation of the ΔT values

Locale non riscaldato-TEMP1							
	Estate						
Esposizione superficie	T,e	T,i	Tp,e	Tp,i	Tmedia	ΔT,uniform	ΔT,butterfly
Nord	26	24	25,98438243	24,04880491	25,01659367	15,01659367	-1,935577525
Sud	54	24	53,76573645	24,73207358	39,24890502	29,24890502	-29,03366287
Est	26	24	25,98438243	24,04880491	25,01659367	15,01659367	-1,935577525
Ovest	54	24	53,76573645	24,73207358	39,24890502	29,24890502	-29,03366287

Locale riscaldato-TEMP2							
	Inverno						
Esposizione superficie	T,e	T,i	Tp,e	Tp,i	Tmedia	ΔT,uniform	ΔT,butterfly
Nord	-27	-27	-27	-27	-27	-37	0
Sud	-27	-27	-27	-27	-27	-37	0
Est	-27	-27	-27	-27	-27	-37	0
Ovest	-27	-27	-27	-27	-27	-37	0

Table 2.37 Calculation of the temperature variations without a heating source

Locale riscaldato-TEMP3							
	Estate						
Esposizione superficie	T,e	T,i	Tp,e	Tp,i	Tmedia	ΔT,uniform	ΔT,butterfly
Nord	26	24	25,98438243	24,04880491	25,01659367	15,01659367	-1,935577525
Sud	54	24	53,76573645	24,73207358	39,24890502	29,24890502	-29,03366287
Est	26	24	25,98438243	24,04880491	25,01659367	15,01659367	-1,935577525
Ovest	54	24	53,76573645	24,73207358	39,24890502	29,24890502	-29,03366287

Locale riscaldato-TEMP4							
	Inverno						
Esposizione superficie	T,e	T,i	Tp,e	Tp,i	Tmedia	ΔT,uniform	ΔT,butterfly
Nord	-27	20	-26,63298711	18,85308472	-3,889951195	-13,8899512	45,48607183
Sud	-27	20	-26,63298711	18,85308472	-3,889951195	-13,8899512	45,48607183
Est	-27	20	-26,63298711	18,85308472	-3,889951195	-13,8899512	45,48607183
Ovest	-27	20	-26,63298711	18,85308472	-3,889951195	-13,8899512	45,48607183

Table 2.38 Calculation of the temperature variations with a heating source

The mechanical stresses deriving from the defined ΔTs are directly proportional to the deformation ε, which, in turn, depends on the coefficient of thermal expansion α:

$$\varepsilon = \alpha \Delta T \qquad (2.18)$$

The standard α values for the different kinds of materials are shown in Table 2.39, taken from the norm.

Materiale	α_T [10^{-6}/°C]
Alluminio	24
Acciaio da carpenteria	12
Calcestruzzo strutturale	10
Strutture miste acciaio-calcestruzzo	12
Calcestruzzo alleggerito	7
Muratura	6 ÷ 10
Legno (parallelo alle fibre)	5
Legno (ortogonale alle fibre)	30 ÷ 70

Table 2.39 Thermal expansion coefficients

Considering the linear elastic constitutive equations, one has:

$$\sigma = E\varepsilon = E\alpha\Delta T \qquad (2.19)$$

For the case of the two studied panels, it has been assumed $\alpha = 5 * 10^{-6}$, in the absence of any further specifications.

Chapter 3 Creation of the 3D models and software interoperability

The 3D models exposed in this chapter are basically of two main types based on what was the purpose of the work: the more structural ones, made with software specialized on performing structural analysis, and the more graphical oriented architectonical ones, made with the purpose of reaching a high LOD in terms of graphical information.
The initial intention was to work mainly with the Autodesk produced programs Revit (for the architectonical model) and Robot as well as SAP2000 for the structural analysis, however, as will be exposed in the following paragraphs, some obstacles were encountered during the progress of the work that made the use of other software necessary, mainly that of the BIM tool Dlubal.

3.1 The initial steps: Revit, Robot and SAP2000

When creating a 3D model of a construction, it is always advisable to start with the structural model in order to dimension the core of the building which is the structure, before passing on to a more graphical detailed architectonical model in which also non-structural parts are considered. However, a first, basic model was made on the software Revit, in which only the general geometry was defined without giving any further information about materials, connections, exact thickness of the walls etc. The intention was to, once created, export it and continue the work on the structural software Robot, with the purpose of first testing how these two software work together. The reference-drawings that were used to create these models, as well as all the other models created in the next paragraphs, were the ones shown in paragraph 1.4.1.

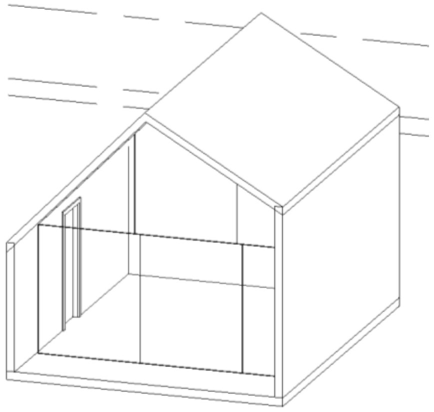


Figure 3.1 First model created in Revit

As mentioned, this first work was then exported on the software Robot with, as can be seen in Figure 3.2, decent results.

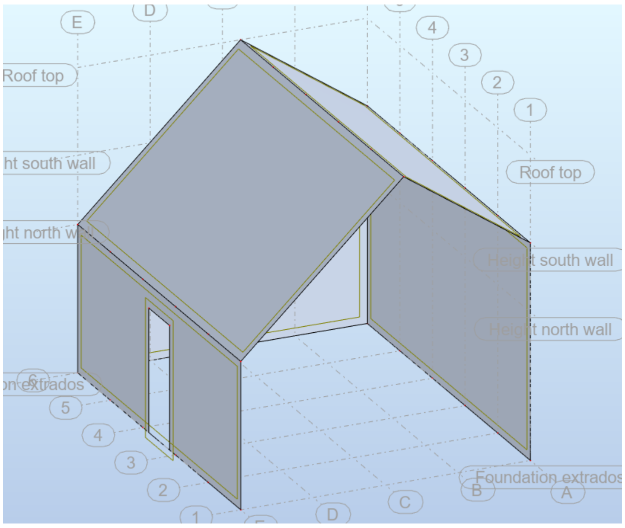


Figure 3.2 Robot model, imported from Revit

The structure was then further studied on Robot; a structural framework was defined, as is shown in Figure 3.3, and the Kingspan panels were modelled with the creation of a "Kingspan material". However, this software was then abandoned for the further steps of the work because of one main reason: it is not specialized in wood structures. For this reason, it doesn't for example allow the modelling of connections for timber frames and has a rather limited choice in timber materials. This was very important, because the intention was to consider basically the entire structure as wood-made. Parallelly, a trial of a model was started also on the software SAP2000 but was abandoned even earlier for the same reason, since it has less possibilities than Robot for what concerns timber.

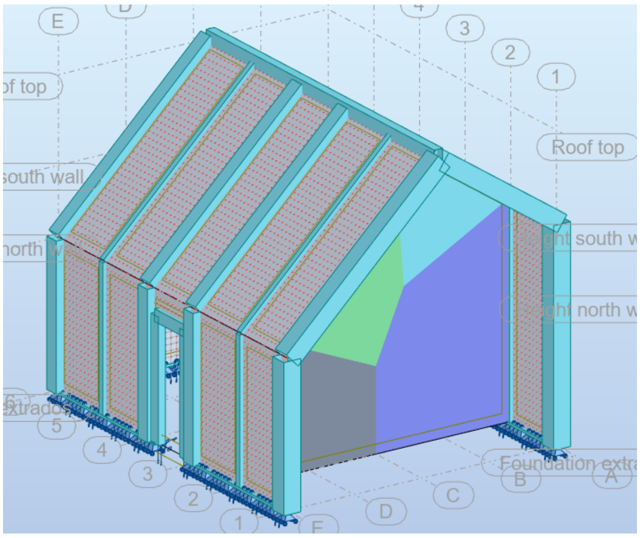


Figure 3.3 Robot model with elaborated structural framework

3.2 The Dlubal structural model

Eventually, the structural model was made on Dlubal, which is a BIM tool coming from Germany working with the Finite Element Method. This software has tens if not hundreds of different timber materials, from LVL to OSB types, from plywood to hardwood and softwood. Furthermore, it allows through add-ons the modelling of specific timber to steel and timber to timber connections and of complex section shapes for frame elements, which, as will be seen in the next paragraphs, was required for the performed work. All in all, it was by far more suitable to the project than the previous used software.
The next paragraphs expose an example of a possible solution of structural application of the panels illustrated in paragraphs 2.2 and 2.3 on the case study seen in paragraph 1.4.

3.2.1 The panels

To model in the program the two studied panels, KINGSPAN TEK 142 and PANELO, that where illustrated in paragraphs 2.2 and 2.3 of this thesis, various assumptions were made. Now, before starting with the description of the applied procedure, it would be appropriate to clarify on the purpose of the work presented in this paragraph: in fact, since the interest of this thesis was to do an investigation of the appropriateness of the panels in the case study and not a more detailed research on a single panel subjected to load case, the panels were considered as if to be composed of not different materials distributed on three layers but of only one equivalent material, whose stiffness E and shear modulus G result in the same EI and GA of the real composite panels. In other words, the distribution of the stresses within the real panel case, that would follow the sandwich theory (illustrated in image 2.1), was not taken into account and the panels were considered as if they were homogeneous.
For the computation of the stiffness E and the shear modulus G of the equivalent Kingspan system, the values of the design bending stiffness EI_d and shear rigidity GA_d given in Table 2.3 in paragraph 2.2.1 were used. According to the Table:
$EI_d = 4{,}6 * 10^{11}\ Nmm^2$
and
$GA_d = 5{,}7 * 10^5\ N.$

Knowing the thickness t=142 mm, the value of the width to use to calculate the area A and the inertial moment I was assumed to be of 1 m, in absence of further specification in any of the documents made public by the Kingspan company.

Consequently, the resulting E_d and G_d values are:

$E_d = 1928\ MPa$,

$G_d = 4\ MPa$.

What first comes to the attention is the very low shear modulus value compared to the Young's modulus E. However, as the panel can be considered as a plate element where the given EI assumes the role of the plate's bending stiffness D, the G value doesn't have much relevance for the problem in the first place.

Anyhow, the program requires as an input the mean value of stiffness and not the design one. To define it, the indications given by the Eurocode 5 where followed and not those given by the Italian codes; this was done because the panel is produced by a British company, which is more likely to provide the design values following the Eurocodes rather than the Italian ones. Consequently:

$$E_d = \frac{E_{mean}}{\gamma_m} \tag{3.1}$$

and

$$G_d = \frac{G_{mean}}{\gamma_m} \tag{3.2}$$

where γ_m is taken from the value of OSB given by the EC5 (same as that given by the CNR in Table 2.19, paragraph 2.3.1), being

$\gamma_m = 1{,}2$.

Therefore:

$E_{mean,KINGSPAN} = 2313{,}42\ MPa$,

$G_{mean,KINGSPAN} = 4{,}8\ MPa$.

Material Properties		OSB (EN 300), KINGSPAN PANEL | EN 12369-1:2001-01	
⊟ Main Properties			
Modulus of Elasticity	E	2313.4	N/mm2
Shear Modulus	G	4.8	N/mm2
Specific Weight	γ	1.73	kN/m3
Coefficient of Thermal Expansion	α	5.0000E-06	1/°C
Partial Safety Factor	γM	1.20	

Table 3.1 Kingspan material properties input in the program

Furthermore, following indications given by online tutorials provided by Dlubal, the material was modelled as an orthotropic elastic material, which is generally assumable for wood types, and the panel was accordingly modelled as a plane element in the form of an orthotropic 2D surface. For these kinds of surfaces, the different properties E, G and the Poisson's ratio υ are, unlike in isotropic materials, fully independent from each other. For what concerns the Poisson's ratio, there are equations to approximate its value which will here not be specified, however, always according to the Dlubal video classes, in the case of timber it can be that wood that's cut down in one place, for example in the US east coast and wood that's cut down in the US west coast of the exact same species have different Poisson's ratios, making it impossible to know its exact value; it is therefore common practice to assume $\upsilon = 0$.

Having modelled the panel as an orthotropic surface, the program asks as input also to define the stiffness properties on the axis orthogonal to the one of the main fibres. However, here it was made the assumption to consider the stiffness properties equal in all directions of the surface plane; this could seem at first sight as to be in contrast with what said before about wood materials, but since in this case the handled panel is of OSB, it has been considered as plausible. In fact, even though wood per se is usually considered to be an orthotropic material, OSB, as previously mentioned, stands for "Oriented Strand Board", which translates into many separate wood strands glued together while remaining placed in any direction, giving it presumably in all those directions similar properties. Therefore,

$$E_{mean,0} = E_{mean,90}, \qquad G_{mean,0} = G_{mean,90}. \qquad (3.3)$$

⊟ Additional Properties			
Modulus of Elasticity	$E_{x1,plate,0}$	2313.4	N/mm2
Modulus of Elasticity	$E_{y1,plate,90}$	2313.4	N/mm2

Table 3.2 Stiffness values in the two main directions of the panel

For the Panelo panel, the same approach as the Kingspan case was used in terms of modelling an equivalent panel, but a differend procedure was applied to calculate the stiffness of the material. In fact, since, as seen in paragraph 2.3.1, the company provides with no data relative to the bending stiffness of the system, an "engineered" stiffness was derived from the stiffness and thickness values of the single components of the panel. This was done in the following way:

$$E_{PANELO} = \frac{E_{LVL} * t_{LVL} + E_{PUR} * t_{PUR} + E_{OSB} * t_{OSB}}{t_{PANELO}} \qquad (3.4)$$

Considering:

-E_{LVL} derived from a heavy type LVL of characteristic weight 730 $\frac{kg}{m^3}$, specifically the BauBuche S Board, of stiffness $E_{LVL} = 16800\ Mpa$[14]

-$t_{LVL} = 39\ mm$ for the wall panels and 27 mm for the roof panels

[14] Allgemeine Bauartgenehmigung BauBuche Platte Z-9.1-838.pdf

-$E_{OSB} = 4800\ Mpa$, taken from the datasheet of the Kronoswiss OSB type 3 panel[15]

-$t_{OSB} = 15\ mm$

-E_{PUR} derived from an inverse procedure of the one described here applied to the Kingspan TEK system. In fact, knowing the E value and the thickness of the equivalent Kingspan material (defined in this paragraph) as well as the stiffness and the thickness of the OSB composing the Kingspan panel (the same as that defined here of the Kronoswiss OSB), it was possible to compute the stiffness of the Kingspan insulating core $E_{PUR} = 1647\ MPa$. It was then assumed that the same type of PUR composing the TEK panel is utilized for the insulating core of the PANELO system. This was done because the stiffnesses of PUR boards found in literature through research had a huge range of possible values (going more ore less from the order of magnitude of 5 MPa to 5GPa) depending from the density of the PUR, which was therefore assumed to be equal as that of the Kingspan panel.

-$t_{PUR} = 145\ mm$ for the wall panels and $195\ mm$ for the roof panels.

The LVL and OSB types were chosen according to the considerations made in paragraph 2.3.1.

Through (3.4), two values of mean stiffness, one for the wall panel and one for the roof panel were computed. The shear modulus G was not computed because, as mentioned in the case of the Kingspan panel, considered as of not great relevance for the problem.

The calculations of the stiffness values $E_{PANELO,WALL}$ and $E_{PANELO,ROOF}$ as well as the derivation of E_{PUR} from $E_{KINGSPAN}$ are schematized in Tables 3.3, 3.4 and 3.5.

FROM SWISS KRONO OSB/3 DATASHEET	
E_{OSB} [N/mm^2]	4800
FROM KINGSPAN	
$E_{KINGSPAN}$ [N/mm^2]	2313,424287
FROM BAUBUCHE S BOARD	
E_{LVL} [N/mm^2]	16800

Table 3.3 Stiffness values of OSB, KINGSPAN and PANELO

OSB		PUR		KINGSPAN PANEL	
t [mm]	E [N/mm^2]	t [mm]	E [N/mm^2]	t [mm]	E [N/mm^2]
30	4800	112	1647,377221	142	2313,424287

Table 3.4 Calculation of the PUR stiffness

FOR PANELO (WALL PANELS)		FOR PANELO (ROOF PANELS)	
E_{LVL} [N/mm^2]	t_{LVL} [mm]	E_{LVL} [N/mm^2]	t_{LVL} [mm]
16800	39	16800	27
E_{PUR} [N/mm^2]	t_{PUR} [mm]	E_{PUR} [N/mm^2]	t_{PUR} [mm]
1647,377221	145	1647,377221	195
E_{OSB} [N/mm^2]	t_{OSB} [mm]	E_{OSB} [N/mm^2]	t_{OSB} [mm]
4800	15	4800	15
E_{PANELO} [N/mm^2]	t_{PANELO} [mm]	E_{PANELO} [N/mm^2]	t_{PANELO} [mm]
4854,621593	199	3573,158473	237

Table 3.5 Calculation of the Panelo wall and roof panels stiffnesses

[15] https://baldolegnami.it/media/osb.pdf

As can be noticed, being the stiffest component the LVL, the wall panel has obviously greater stiffness than the roof panel. For what concerns all the other aspects regarding the modelling of the panels, the same assumptions as for the Kingspan TEK system were made.

3.2.2 The geometry

3.2.2.1 Structure with Kingspan TEK Panels

The Kingspan panel has maximum dimensions 1.22 m x 7.5 m. Therefore, the panels used in the model of the structure are all of width 1.2 m, while the height depends on the geometry of the walls and roof, exposed in paragraph 1.4.1.

The disposition and numbering of the panels as well as the structural framework of the model of the construction are shown in Figures 3.4 and 3.5. In this model, the x-axis is horizontal and its direction is parallel to east and west wall, the y-axis is horizontal and its direction is parallel to south and north wall and the z-axis is vertical. The origin of the system is in the corner between the foundation, the east wall and the north wall.

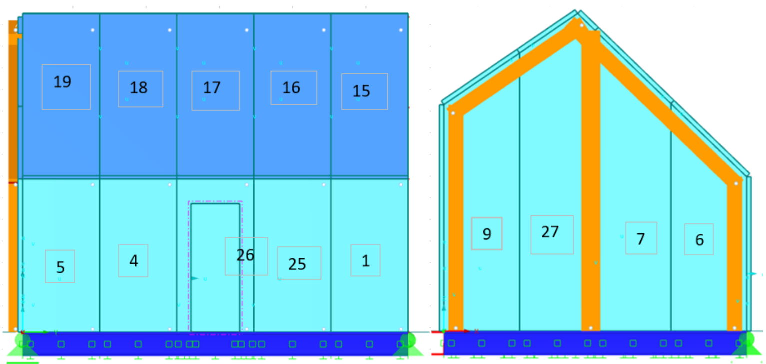


Figure 3.4 From left to right: north side and east side of the building

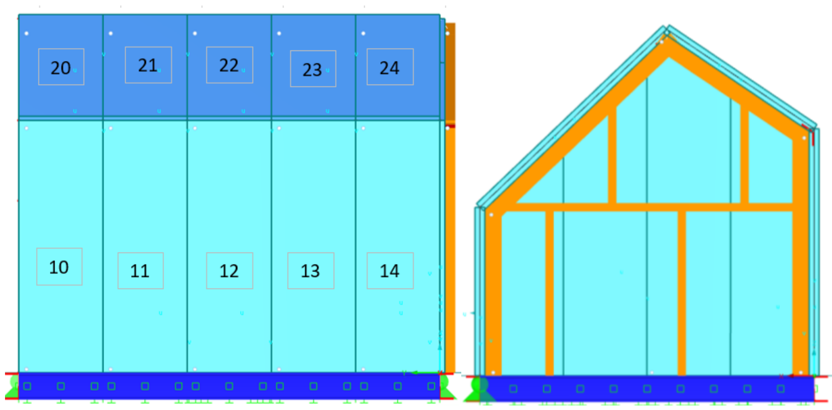


Figure 3.5 From left to right: south side and west side of the building

Table 3.6 shows the exact dimensions and exposure of each panel. For the panels with inclined sides, average height values have been considered. The base dimension of the modelled building is 4.8 m x 6 m.

Panel	Position	Width [m]	Height [m]	Area [m^2]
1	Muro nord	1,2	2,42	2,904
25	Muro nord	1,2	2,42	2,904
26	Muro nord	1,2	2,42	1,355616
4	Muro nord	1,2	2,42	2,904
5	Muro nord	1,2	2,42	2,904
15	Tetto nord	1,2	3,62	4,344
16	Tetto nord	1,2	3,62	4,344
17	Tetto nord	1,2	3,62	4,344
18	Tetto nord	1,2	3,62	4,344
19	Tetto nord	1,2	3,62	4,344
6	Muro est	1,2	2,991666667	3,59
7	Muro est	1,2	4,125	4,95
27	Muro est	1,2	4,716666667	5,66
9	Muro est	1,2	3,958333333	4,75
10	Muro sud	1,2	3,55	4,26
11	Muro sud	1,2	3,55	4,26
12	Muro sud	1,2	3,55	4,26
13	Muro sud	1,2	3,55	4,26
14	Muro sud	1,2	3,55	4,26
20	Tetto sud	1,2	2,56	3,072
21	Tetto sud	1,2	2,56	3,072
22	Tetto sud	1,2	2,56	3,072
23	Tetto sud	1,2	2,56	3,072
24	Tetto sud	1,2	2,56	3,072

Table 3.6 Dimensions and exposure of the panels

In the north wall, more precisely in panel 26, there's an opening in form of a door of dimensions 0.76 m x 2.03 m. As can be observed in Figures 3.4 and 3.5, a series wooden frames were added as a support to the panel structure based on the 2D drawings of paragraph 1.4, which can be seen in a more detailed way in Figure 3.6. More specifically, 6 main frames composed of two columns and a couple of rafter beams are placed with a spacing of 1.2 meters (corresponding to the width of the panels) along the longer sides of the building, so the y axis in the model. The two columns as well as the rafter beams composing those frames are made of C24 softwood timber and have section 220 mm x 240 mm. On the east wall of the bivouac, an additional column of the same material has been placed to contain the deformations of the panels as well as to take some of the vertical load coming to the wall from the roof. This column has section 220 mm x 300 mm. Of the 6 main frames, 5 are on the inner sides of the respective walls and roof they are connected to while another one, the one on the east side of the building, is on the outer side of the wall, more precisely the east wall. For the west wall, which, as was seen in the exposure of the 2D drawings in paragraph 1.4, is made of non-structural glass supported by a frame, only the wooden supporting structure of the glass was modelled. The structural timber elements of this frame have all dimensions 120 mm x 300 mm and are of C24 softwood timber.

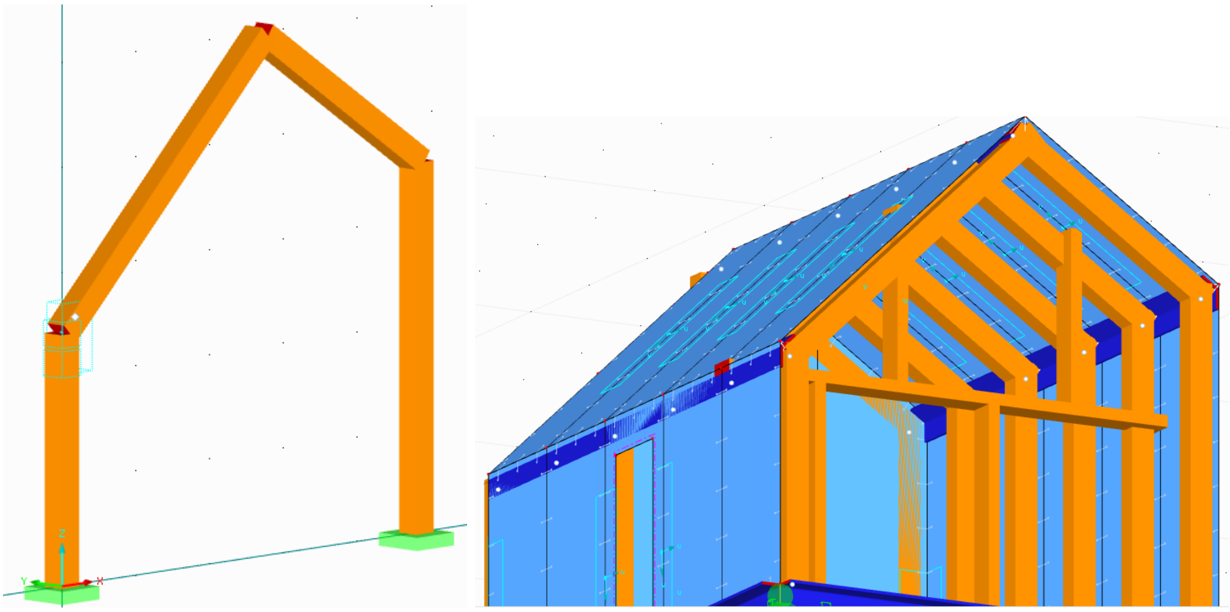


Figure 3.6 Details of the supporting frame

The frame connected to the east wall with the additional structural column as well as the timber glass supporting frame can be observed in Figure 3.7.

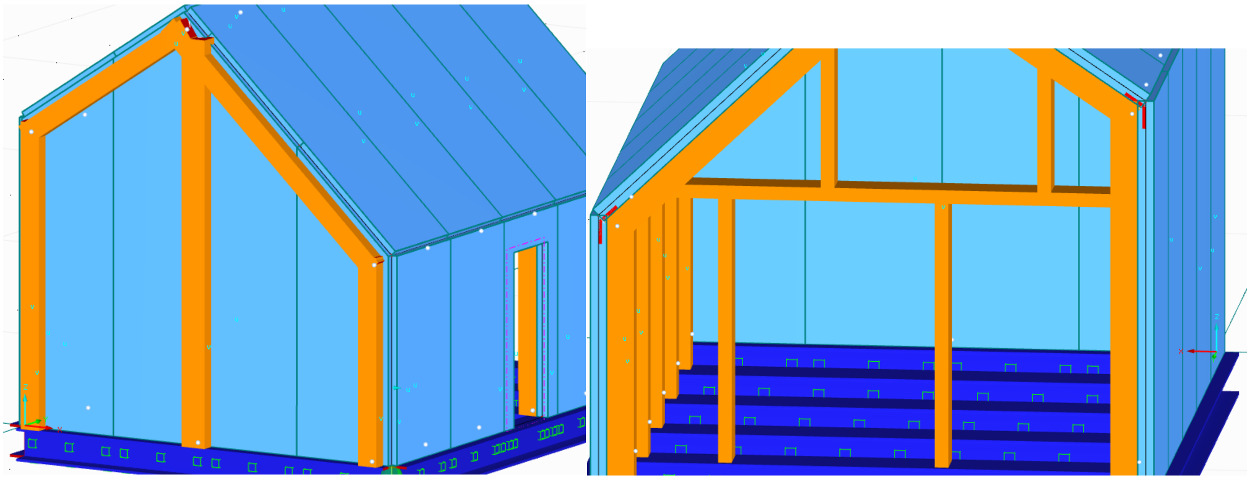


Figure 3.7 From the left to the right: Detail of the east wall frame and the glass wall supporting frame

The initial intention was to not model the west wall at all and consider the whole glass + supporting frame system as non-structural. However, after some launches of the structural analysis of the building without these components, it was noticed that in complete absence of some elements providing stiffness to the whole structural system in the direction of the x-axis (the red axis in Figure

3.7), the structure was experiencing a strong torsion around the vertical z-axis (the light blue axis in Figure 3.7), mostly in the case of wind loads "hitting" the north or south wall, so with direction parallel to the x-axis. This was due to the fact that the east wall, thanks to the panels, was providing the whole structure system with much more stiffness than the non-existing west wall which was obviously providing none. For this reason, the modelling of the glass frame was necessary.
Between one frame and the other, at the ridge of the roof and at the higher edge of the wall plates, steel truss elements were modelled that mostly serve for the purpose of connecting the different frames to each other and not to bear the weight of the roof. They are made of S 450 steel and were modelled with the Dlubal add on module "SHAPE THIN", specialized in the creation of sections for thin 1D elements. Their shape follows the geometry of the walls and roof system, their thickness is of 30 mm and their sides have length of 200 mm. Details of those elements are shown in Figures 3.8, 3.9 and 3.10.

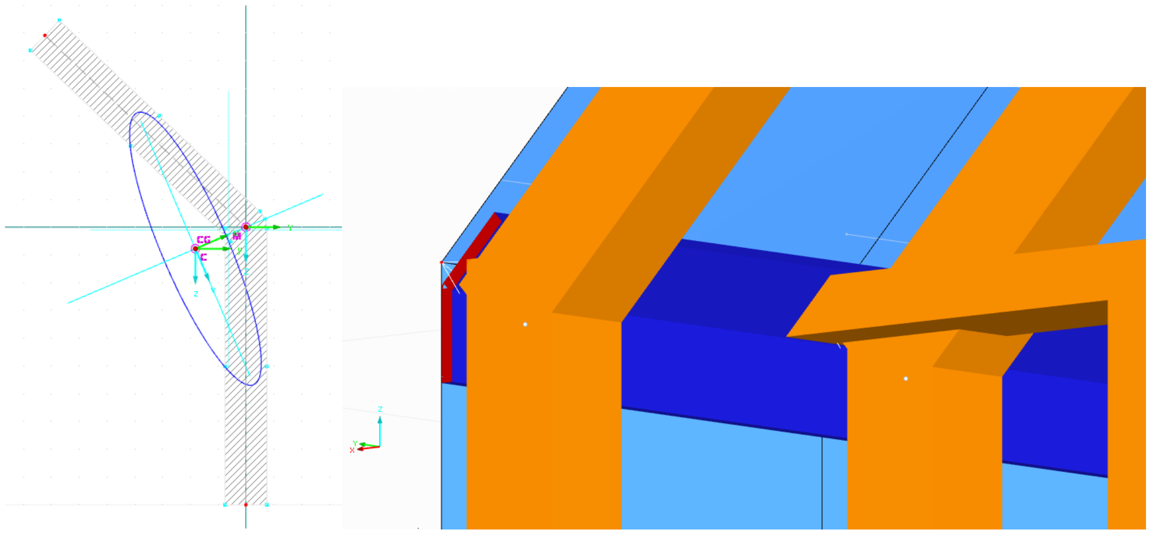


Figure 3.8 From the left to the right: modelling of the section of the north wall edge truss element and detail of it within the structure

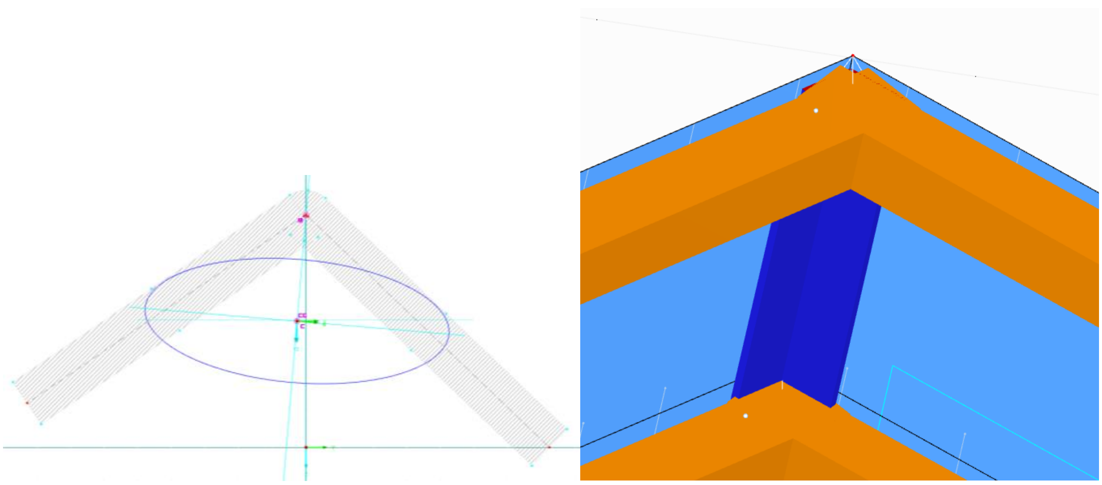


Figure 3.9 From the left to the right: modelling of the section of the roof ridge truss element and detail of it within the structure

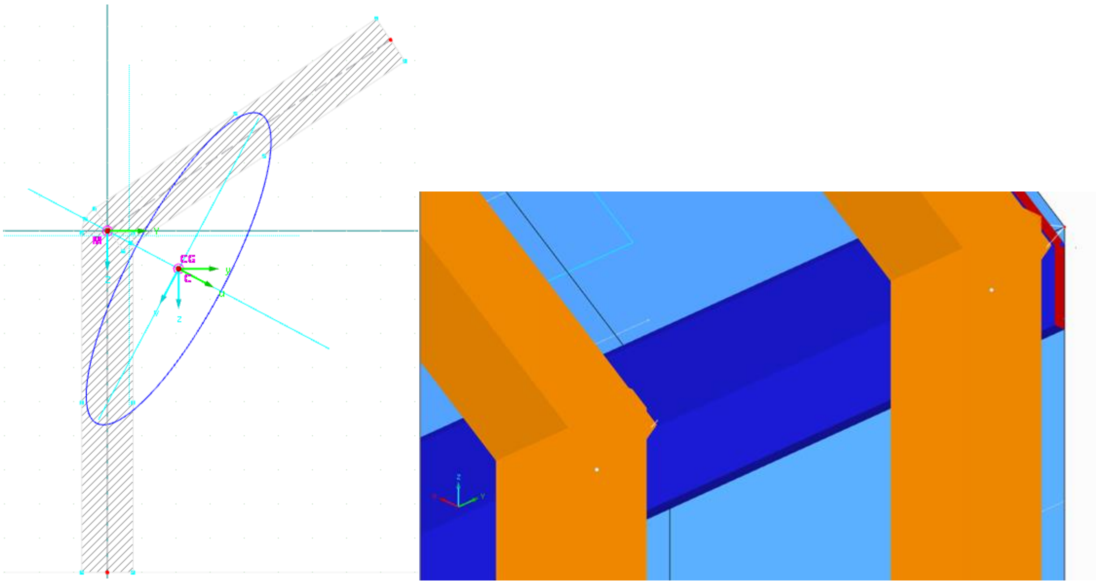


Figure 3.10 From the left to the right: modelling of the section of the north wall edge truss element and detail of it within the structure

No additional bracing is required between the frames, because this function is already carried out by the structural panels.

The foundation consists in a steel S450 grid composed of two longitudinal H beams and of six transversal H beams, with the steel elements having section of properties shown in Figure 3.11.

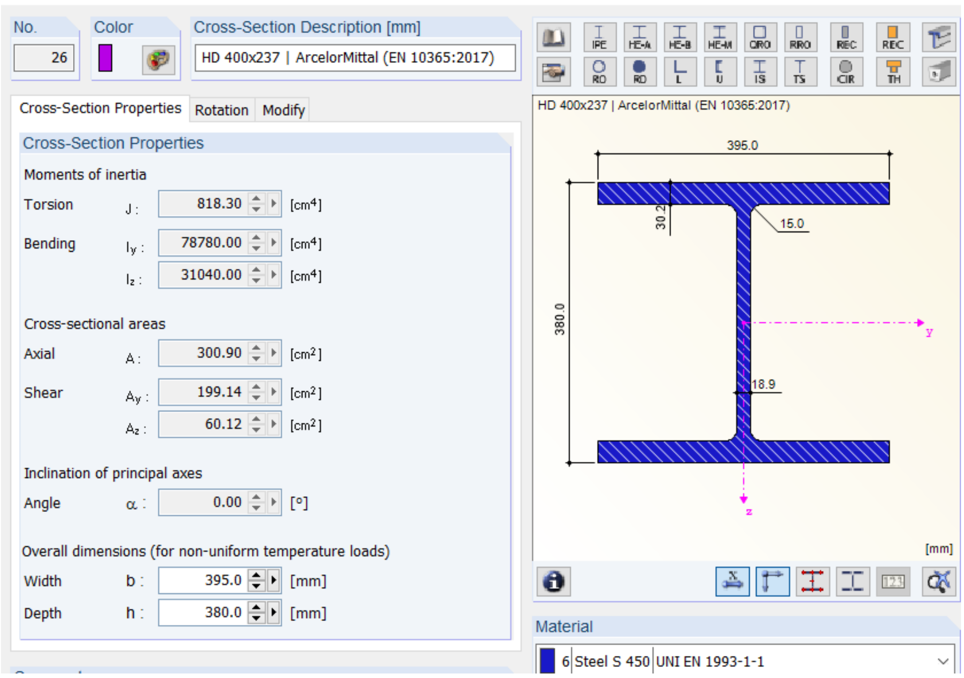


Figure 3.11 Cross section of the steel foundation elements

It has been modelled as if it was lying on infinitely stiff elastic ground (spring constant of the member elastic foundation in the vertical direction extremely high, see Figure 3.12)

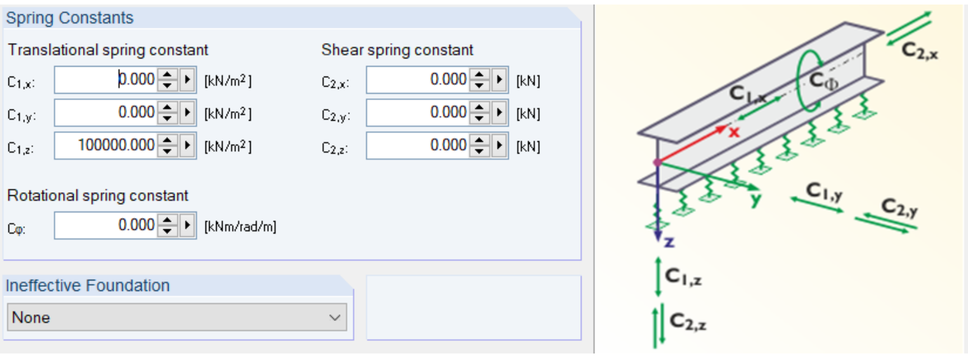


Figure 3.12 Spring constants of the foundation members

Furthermore, it has been constrained in the x and y direction in one vertex of the grid (at one end of one of the two longitudinal beams) and in the y direction in another vertex (at the other end of the same beam). Details of the foundation and of the constraints are to be seen in Figure 3.13.

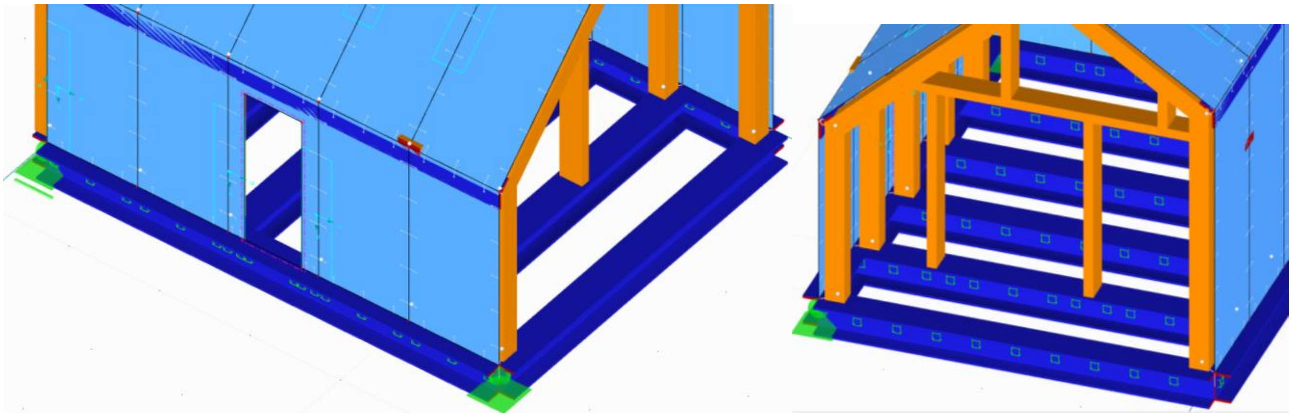


Figure 3.13 Details of the foundation and its constraints

All the structural elements were dimensioned based on an in-depth study of the structural behaviour of the building with applied loads those defined in paragraph 2.5.

3.2.2.2 Structure with Panelo Panels

The geometry of the structure with the Panelo panels is basically very similar to that defined with the Kingspan TEK panels, but there's two things that change. The first one is concerning the supporting frame system. In fact, since the Panelo panel is both thicker and stiffer, as seen in paragraph 3.2.1, this allowed to reduce by a significant amount the section of those elements, which was of dimensions 220 x 240 mm for those supporting the Kingspan Panels, and is in this case of dimensions 130 x 240 mm, always of C24 Softwood timber. Furthermore, the high stiffness allowed to also completely remove the additional column placed in paragraph 3.2.2.1 on the east wall of the building. The second thing that was changed was the geometry of the east wall. This was done because the Panelo company states on the brochure that the maximum dimensions of the panel are 12 m in width and 4 m in height[16], so it was impossible to model panels of height equal to that of the wall. The exact disposition and numbering of the panels as well as the structural framework of the model of the construction are shown in Figures 3.14 and 3.15.

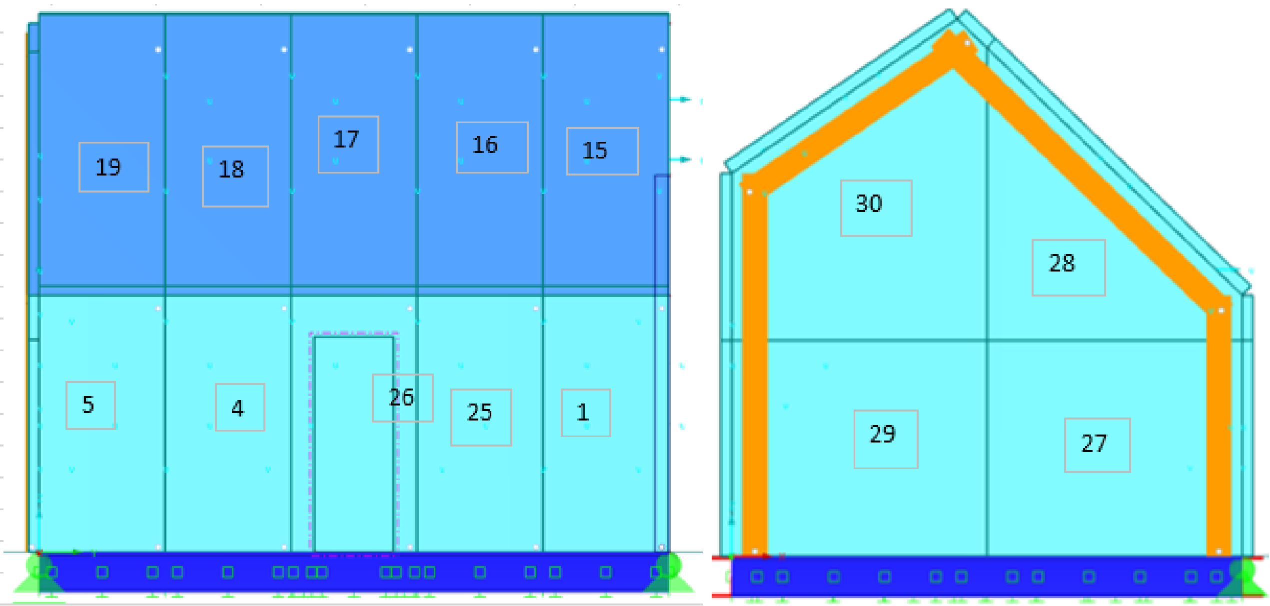


Figure 3.14 From left to right: north side and east side of the building

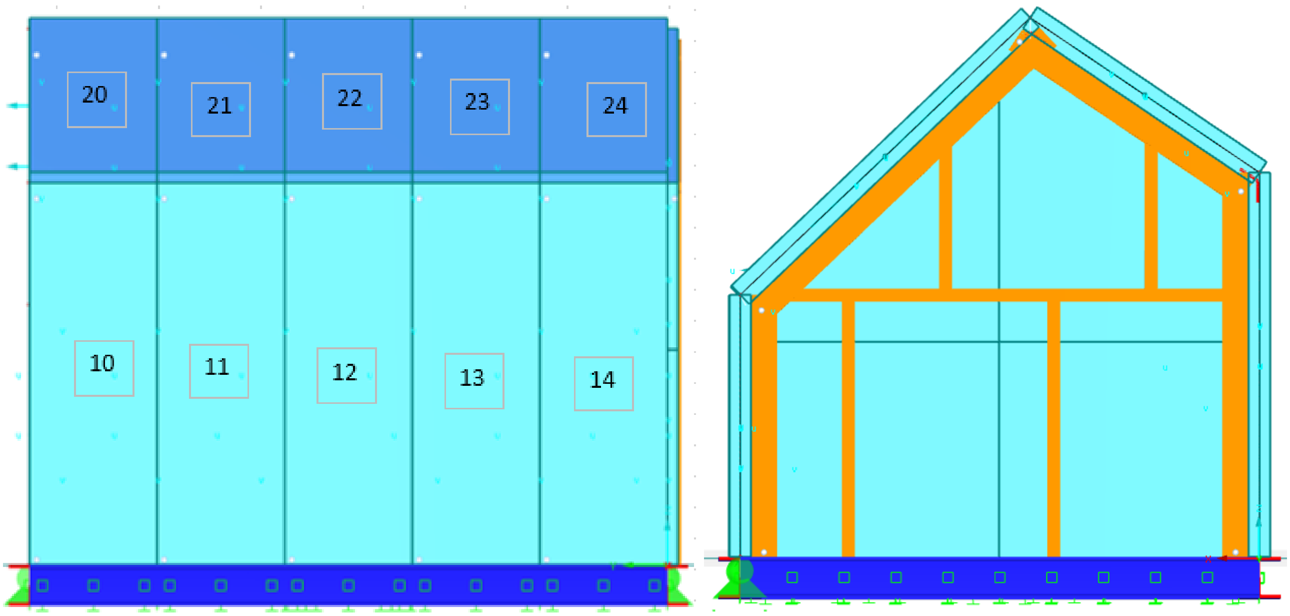


Figure 3.15 From left to right: south side and west side of the building

[16] https://panelo.eu/wp-content/uploads/2018/07/Brochure-ENG.pdf

Table 3.7 shows the dimensions and exposure of each panel. For the panels with inclined sides, average height values have been considered.

Panel	Position	Width	Height	Area
1	Muro nord	1,2	2,42	2,904
25	Muro nord	1,2	2,42	2,904
26	Muro nord	1,2	2,42	1,356
4	Muro nord	1,2	2,42	2,904
5	Muro nord	1,2	2,42	2,904
15	Tetto nord	1,2	3,62	4,344
16	Tetto nord	1,2	3,62	4,344
17	Tetto nord	1,2	3,62	4,344
18	Tetto nord	1,2	3,62	4,344
19	Tetto nord	1,2	3,62	4,344
27	Muro est	2,4	2	4,8
28	Muro est	2,4	1,566	3,759
29	Muro est	2,4	2	4,8
30	Muro est	2,4	2,333	5,599
10	Muro sud	1,2	3,55	4,26
11	Muro sud	1,2	3,55	4,26
12	Muro sud	1,2	3,55	4,26
13	Muro sud	1,2	3,55	4,26
14	Muro sud	1,2	3,55	4,26
20	Tetto sud	1,2	2,56	3,072
21	Tetto sud	1,2	2,56	3,072
22	Tetto sud	1,2	2,56	3,072
23	Tetto sud	1,2	2,56	3,072
24	Tetto sud	1,2	2,56	3,072

Table 3.7 Dimensions and exposure of the panels

The information regarding all the other structural components, as the glass supporting frame, the steel frame-connecting elements and the foundation are the same as those exposed in paragraph 3.2.2.1 for the structure with the Kingspan panels.

3.2.3 The panel connections

The panels, both for the case of the Kingpan system and of the Panelo system, are all connected to each other through insulated cassette splines and not through timber posts. This was done because having already the supporting frame, additional timber posts weren't needed. Having only the timber posts and not the supporting frames, which would be a more practical solution, was impossible because it was leading to failure in the system as the whole structure was not strong enough to bear the weight of snow and wind. Additionally, avoiding the use of timber posts and placing only insulated cassette splines also doesn't allow the creation of thermal bridges within the walls and the roof, which is a significant advantage. The next paragraphs explain how the fasteners mentioned already in paragraphs 2.2.4 and 2.3.4 connecting the panels, the splines and the frame or the foundation to each other were modelled, where with modelled what is meant is what constraints were given to the connection and, if constraints were given, with what rigidity.

3.2.3.1 Kingspan Panels

The panel-panel-frame connection works with a combination of both fasteners and gluing material. It is schematized with a section view in Figure 3.16. As can be observed, only one of the two panels as well as the spline are directly connected to the frame element with a fastener, the other one is only connected to the spline; the panel connected to the frame (the left one in Figure 3.16), is connected to it thanks to screw elements, more specifically the Sparennagel 6 mm x 210 mm, also seen in paragraph 2.2.4 which talks about the Kingspan connections. Always following the indications given by the company, those screws are placed in line along the vertical z axis with a spacing of 300 mm. The adjacent panel, in case of Figure 16 the right one, is jointed to the connection spline with nail type fasteners, again following the indications exposed in paragraph 2.2.4, which suggests the use of galvanized ring nails of dimesions 2.8 mm x 63 mm, placed with a spacing of 50 mm. In addition, the spline on this side is glued to the panel with a sealer, where for the sake of calculations it was also chosen a specific type, which is one called "SIP Seal", produced in the United States.

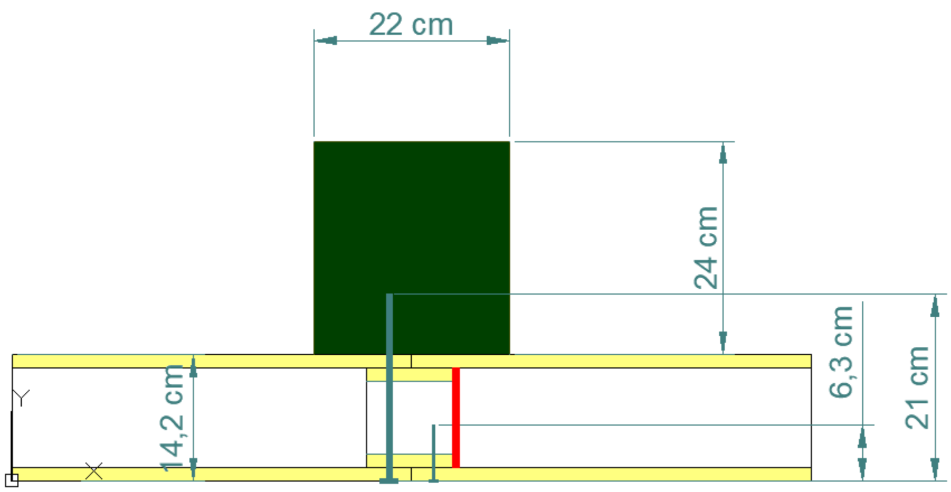


Figure 3.16 Panel to panel to frame connection. The red line represents the gluing material.

The purpose of this whole system was to consider one panel (the right one in figure 3.16) basically as if to form one single piece with the spline. In fact, those gluing materials are extremely strong and have a very high shear modulus, of some orders of magnitude greater[17] than what is usually the slip modulus of the fasteners, which will be seen in the next pages, and can therefore provide, combined with the nails, enough rigidity and resistance to completely fix the panel to the spline. Through this assumption, it is possible to consider both panels connected to each other and to the frame with the Sparrennagel 6 mm x 210 mm screw system, which makes it a little bit more intuitive to model and understand the behaviour of the connection.

For what concerns the wall to wall to frame corner connection, details are shown in Figure 3.17. It can be seen how in this case the one describe before was obviously not appliable; in this case, both are connected through Sparennagel screws, one to the adjacent panel and the other to the frame, both with a spacing of 300 mm; since the two rows cannot intersect, they will have to be staggered. However, similarly to the previous case, also here one panel is connected to the other one which is fastened to the frame, and it was also here considered as if both panels were connected both to each other and to the frame with a row of Sparennagel of dimensions 6 mm x 210 mm and spacing 300 mm. In short, the two connections of Figures 3.16 and 3.17 were modelled in the same way.

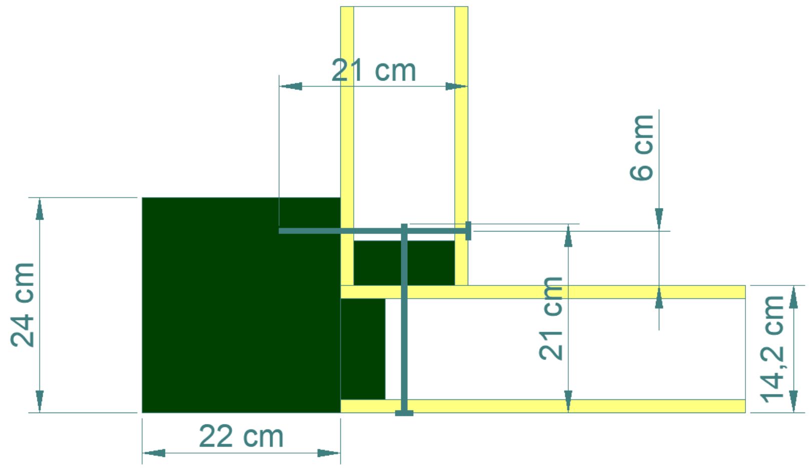


Figure 3.17 Wall to wall to frame connection

The ridge connections of the roof panels and the wall to roof edge panels connections are those suggested by the brochure of the TEK system and illustrated in paragraph 2.2.4 (Figure 2.5 and 2.6). In these cases, the panels are only connected to each other and not to any frame element; however, the geometry is here much simpler than in the previous two cases. It is therefore clear, that these connections have to be modelled as exactly what they are, which is a row of Sparrennagel spaced 300 mm from each other. Being this the case, also these connections were defined with the same characteristics as those of the first two that were described in this paragraph.

[17] Mechanicalshearpropertiesofadhesives.pdf

The described connections were modelled on Dlubal with the command "Line hinges", which allows to input constraint degrees for surfaces. The contact line between two more surfaces is there assigned an own reference system, that can be seen in Figure 3.18.

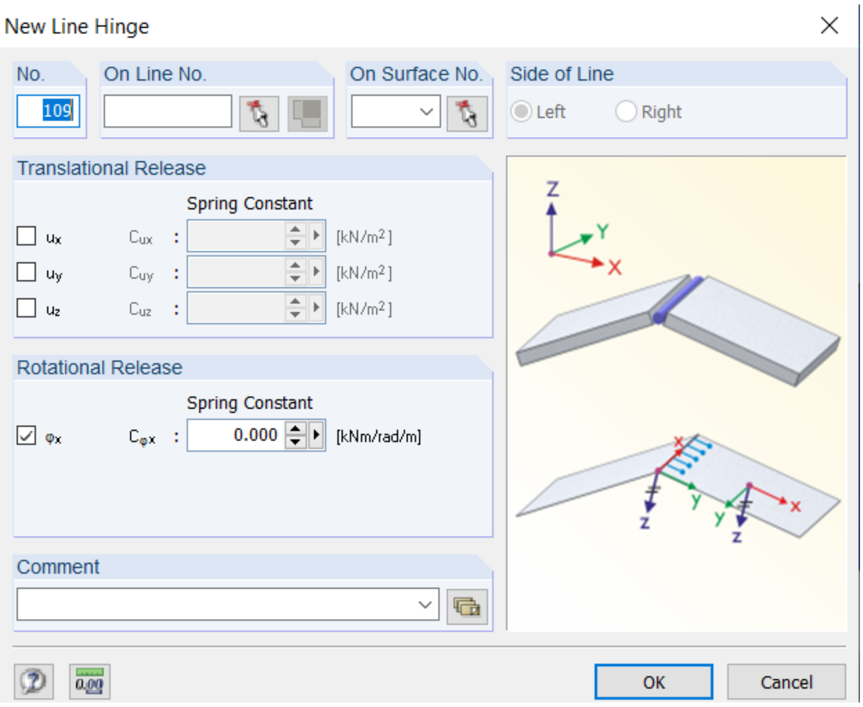


Figure 3.18 Line hinges in the Dlubal software

The first defined property was that the rotation around the main axes of the connection (x axis along the contact line between the surfaces in Figure 3.18) is completely allowed, so $C_{\varphi x} = 0$. Regarding the constraints for the displacement in the two in plane axes u_x and u_y, fasteners do have a limited stiffness caused by their long and thin shape, which can deform in the way illustrated in Figure 3.19.

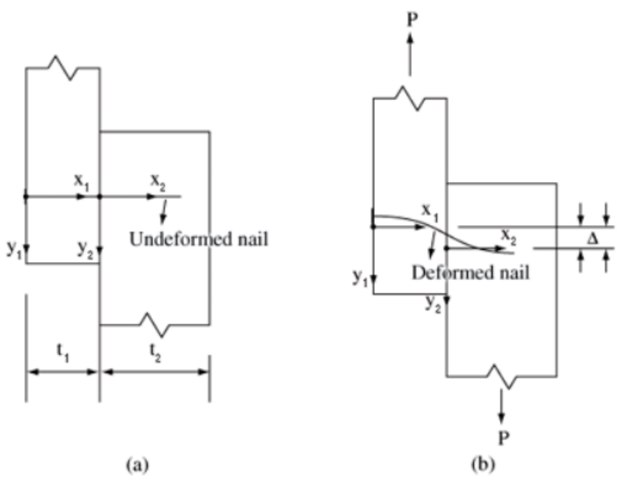


Figure 3.19 Deformed shape of the connection

This stiffness is defined, according to EC5, in the form of a slip modulus K_{ser}, whose value in $\frac{kN}{m}$ is computed, for wood fasteners inserted without drilling (like nails) through the relation:

$$K_{ser} = \rho_m^{1.5} \frac{d^{0.8}}{30} \qquad (3.5)$$

and for fasteners needing drilling (like screws and bolts), this equation becomes:

$$K_{ser} = \rho_m^{1.5} \frac{d}{23} \qquad (3.6)$$

being ρ_m the mean density of the involved wood in $\frac{kg}{m^3}$ and d the diameter in mm. If the two connected members are of two different wood materials, the density to consider is defined as $\rho_m = \sqrt{\rho_{m,1}\rho_{m,2}}$. In this case, the two considered densities were that of the Kingspan panel defined in paragraph 2.2.1 and of Softwood timber C24 of the frames. In order to have the value of (3.6) in $\frac{kN}{m^2}$, which is the input required by the program, an equivalent connection was modelled by applying

$$K_{ser,eq} = \frac{K_{ser}}{s} \qquad (3.7)$$

where s is the spacing of the fasteners.

What 3.7 does, is it creates a fictitious connection distributed uniformly along the connection line in the absolute absence of spacing (as can be observed in Figure 3.20), so basically in infinite number of infinitesimal nails or screws one attached to the next one, which result in the same stiffness as the real connection.

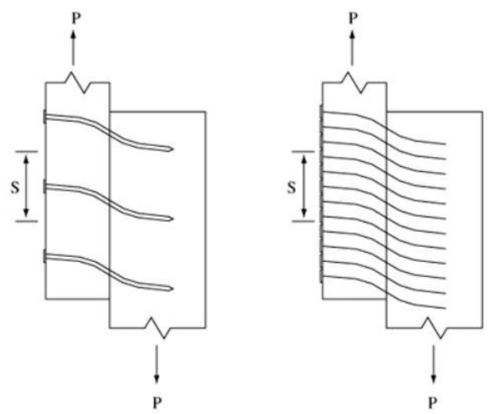


Figure 3.20 From the left to the right: real connection and fictitious connection

When performing a structural analysis at the Ultimate Limit State, the slip modulus should be taken as:

$$K_u = \frac{2}{3} K_{ser} \qquad (3.8)$$

The described procedure was applied and a value for the slip modulus was computed, assumed to be equal in the x and y connection axes. The resulting values are shown in Table 3.8.

KINGSPAN (PANEL-PANEL-FRAME) Slip		
$d_{Sparrennagel}$ [mm]	ρ_{KS+C24} [kg/m^3]	K_{ser} [kN/m]
6	263,1927479	1113,869448

KINGSPAN (PANEL-PANEL-FRAME) Slip			
s [m]	n	$K_{eq,ser}$ [kN/m^2]	$K_{eq,u}$ [kN/m^2]
0,3	1	3712,89816	2475,26544

Table 3.8 Kser, Keq,ser and Ku

Finally, the constraint in the z direction of the connection was imposed to be rigid.

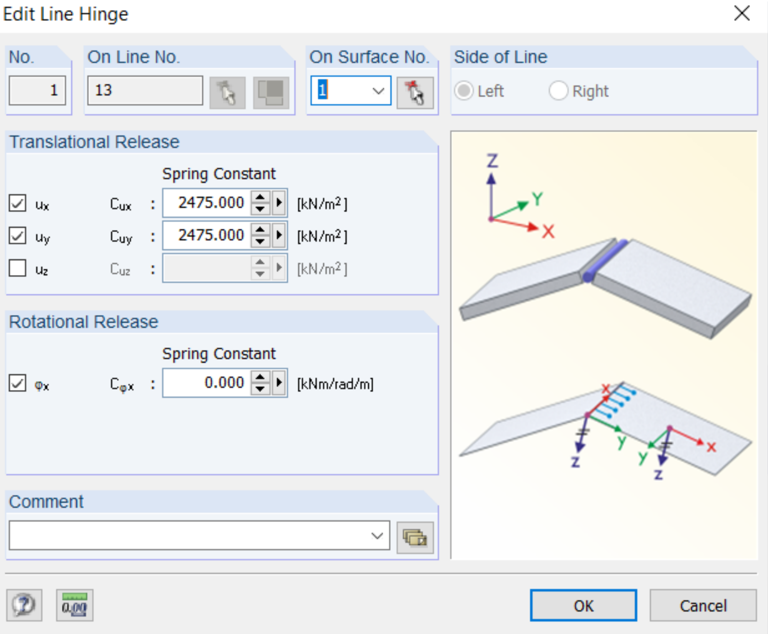


Figure 3.21 Input for the line hinge modelling at the ULS

For what concerns the panel-foundation connection, the considered geometry was the one seen in Figure 2.7 of paragraph 2.2.4. The panels are fixed through a bolt system, more precisely galvanized bolts dimensioned following the imperial system ½ inch x 5 inches, corresponding in the metric system to 12.7 mm x 127 mm, which fix the 50 mm x 112 mm panel bottomplates to the 40 mm x 142 mm soleplates (for more specifications see paragraph 2.2.4) to the steel members of the grid foundation, with a spacing of 0.6 m along the whole perimeter of the construction.

The slip modulus of this connection was computed in the same way as that of the Sparrennagel fastening systems, but with one main difference. In fact, since the bolt in this case connects wood to steel and not wood to wood as before, the ρ_m of (3.5) and (3.6) is equal, always following the indications of EC5, to the wooden element's density multiplied by 2.

The resulting values of the slip modulus are shown in Table 3.9.

KINGSPAN (PANEL-FOUNDATION) Slip						
$d_{Galvanized\ anchor\ bolts}$ [mm]	$\rho_{C24+S450}$ [kg/m^3]	K_{ser} [kN/m]	s [m]	n	$K_{eq,ser}$ [kN/m^2]	$K_{eq,u}$ [kN/m^2]
12,7	420	4752,803077	0,6	1	15842,67692	10561,78462

Table 3.9 Kser and Ku of the panel foundation bolt fasteners

In addition, consequently to some problems encountered in the initial modelling steps related to the formation of singularities in the stress distribution within the panels (see paragraph 4.1.1.3), which were then eventually solved in a different way, also an estimation of the vertical stiffness (meaning in direction of the global z-axis of the model) of the panel to foundation connection was performed, following different documents found on the web. However, the resulting spring constant value was so high (around 200000 $\frac{kN}{m^2}$) that it didn't make sense to insert it into the model, and the vertical constraint was eventually considered as rigid. The joint was therefore given, apart from the partial in plane restraints defined in Table 3.8, only freedom of rotation around the local x axis. The modelling of the foundation to panel connection can be seen in Figure 3.22.

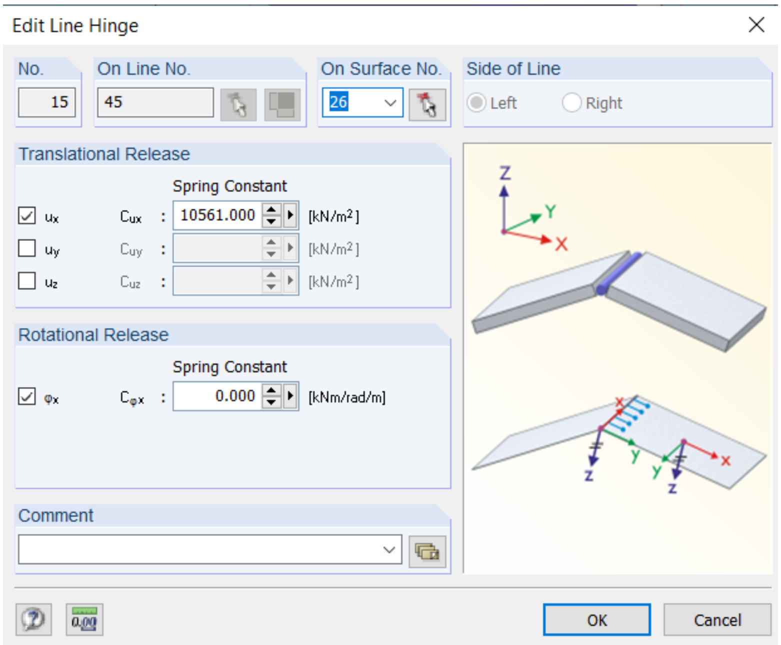


Figure 3.22 Modelling of the panel-foundation connection

3.2.3.2 Panelo Panels

The modelling of the Panelo connections was done in the exact same way as seen for the Kingspan TEK system in paragraph 3.2.3.1, only that instead of using Sparrennagel 6 mm x 210 mm screws, 8.0 mm x 300 mm Heco Topix screws were used to fasten the panels to the frame and coach screws 8.0 mm x 150 mm to each other. The panel foundation connection stayed the same also for what concerns the bolt type. The resulting stiffness values for the panel to panel to frame connections are given in Table 3.10. Since the Panelo systems have two different densities for the roof and wall panels, the two resulting joint types also have different spring constants.

PANELO WALL Slip						
$d_{Sparrennagel}$ [mm]	$\rho_{PANELO+C24}$ [kg/m^3]	K_{ser} [kN/m]	s [m]	n	$K_{eq,ser}$ [kN/m^2]	$K_{eq,u}$ [kN/m^2]
8	287,7824699	1698,082	0,3	1	5660,273282	3773,515521
PANELO ROOF Slip						
$d_{Sparrennagel}$ [mm]	$\rho_{PANELO+C24}$ [kg/m^3]	K_{ser} [kN/m]	s [m]	n	$K_{eq,ser}$ [kN/m^2]	$K_{eq,u}$ [kN/m^2]
8	250,5032909	1379,0573	0,3	1	4596,85759	3064,571727

Table 3.10 Stiffness values of the panel to panel to frame connections

3.2.4 The frame connections

Differently to the modelling of the panel connections which were done by hands, the joints of the supporting frame system were all modelled automatically by the Dlubal software thanks to the add-on module “RF-Joints”. This add-on allows to create the geometry of steel to timber connections and timber to timber connections between two or more beam members; once the geometry is set as an input and the program is run, it generates a rigid joint member in the node of interest connected to the beam elements (which on the other hand get completely released form the original node) with a certain stiffness. However, this additional joint element works really well when studying the behaviour only of the frame system, but it creates complications and “misunderstandings” in the model when modelling additionally to the 1D frame elements also 2D surfaces (as the panels are) connected to each other; for this reason, the geometry of the connections was defined on “RF-Joints” in a separate model in which only one frame was considered.

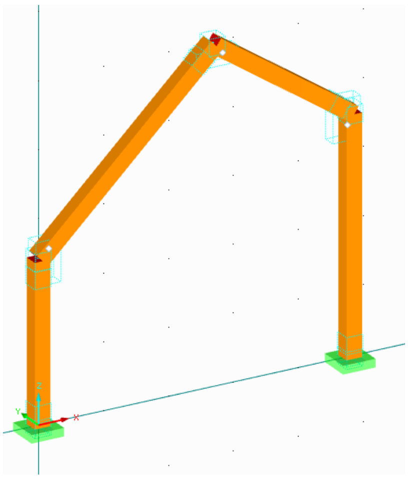


Figure 3.23 The separate "frame" model

It was then proceeded to take the output constraint stiffness values that the software provides for the modelled joint element and simply input this data in the traditional “member hinge” function, that the

Dlubal software offers, in both the Kingspan and the Panelo structural models. A representation of what explained is shown in the Figures below, which are all referring to the Kingspan building supporting frame, starting with the north wall column to north roof rafter beam connection.

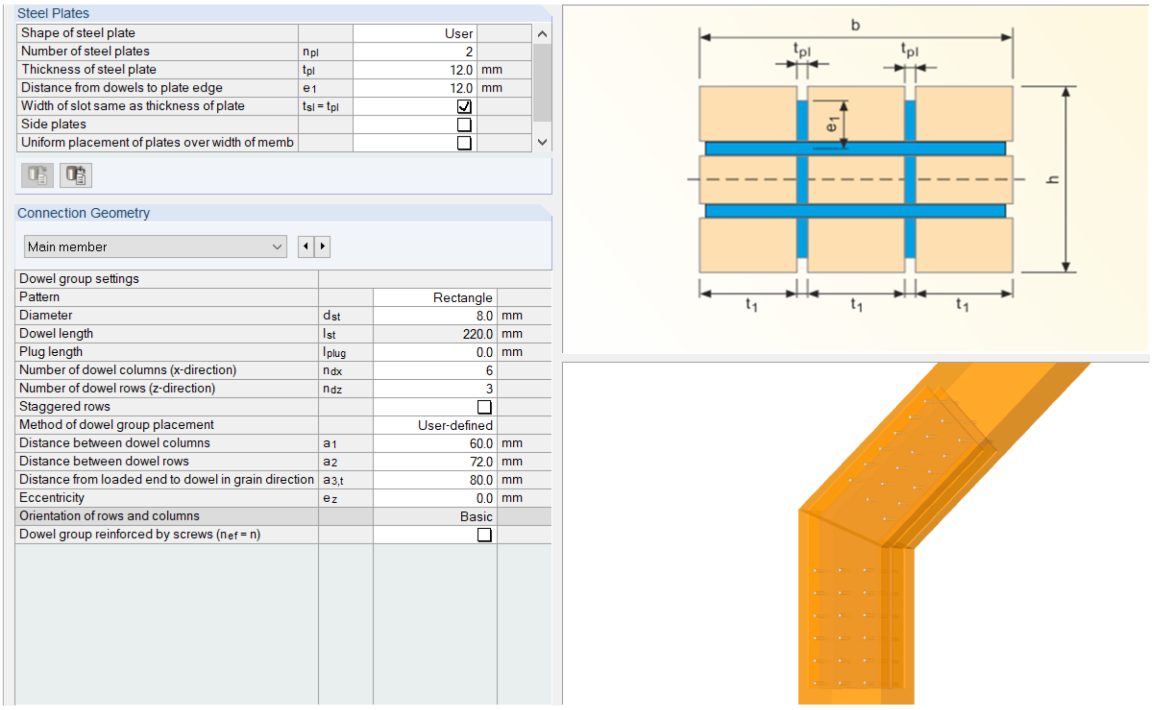


Figure 3.24 Details and representation of the north side edge frame connection

As can be seen in Figure 3.24, the connection is made with two plates of thickness 12 mm each, and a dowel group composed by 6 columns and 3 rows, with dowels of 8 mm diameter each and of length 220 mm (same as the width of the section). Further details are provided in Figures 3.25 and 3.26.

Figure 3.25 From the left to the right: side and front view of the east side edge frame connection

Figure 3.26 Close views of the east side edge frame connection

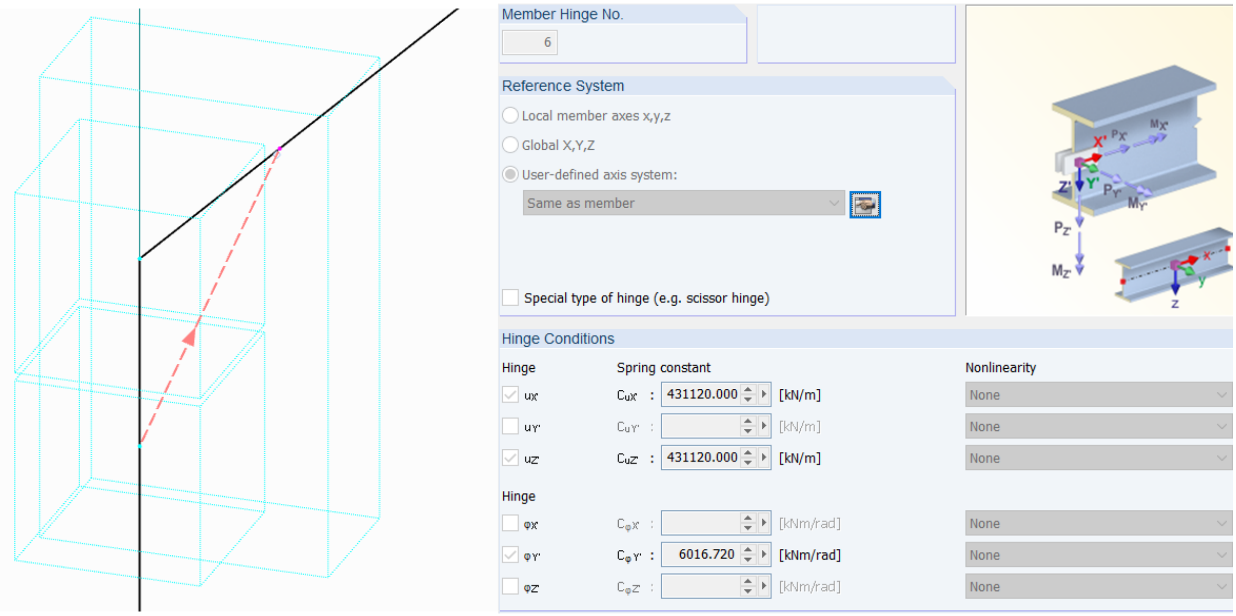


Figure 3.27 From the left to the right: Modelling of the connection by RF Joints, output stiffness values of the connection

Figure 3.27 shows on the left the automatic modelling done by the software of the connection through a joint element (the dashed red line) and node releases (the light blue cubes), and on the right the output connection constraints with relative stiffness given by the RF Joints module. The resulting spring constant related to the rotation about the y-axis (C_{φ}Y in Figure 3.28) was approximated to 6000 kN/rad and used to define the rotational stiffness of the member hinge on the Kingspan structure model of the same node The values of Figure 3.27 related to the stiffness in the x and z direction where on the other hand not considered because so high to allow the assumption of a rigid constraint. Figures 3.28 and 3.29 illustrate this step.

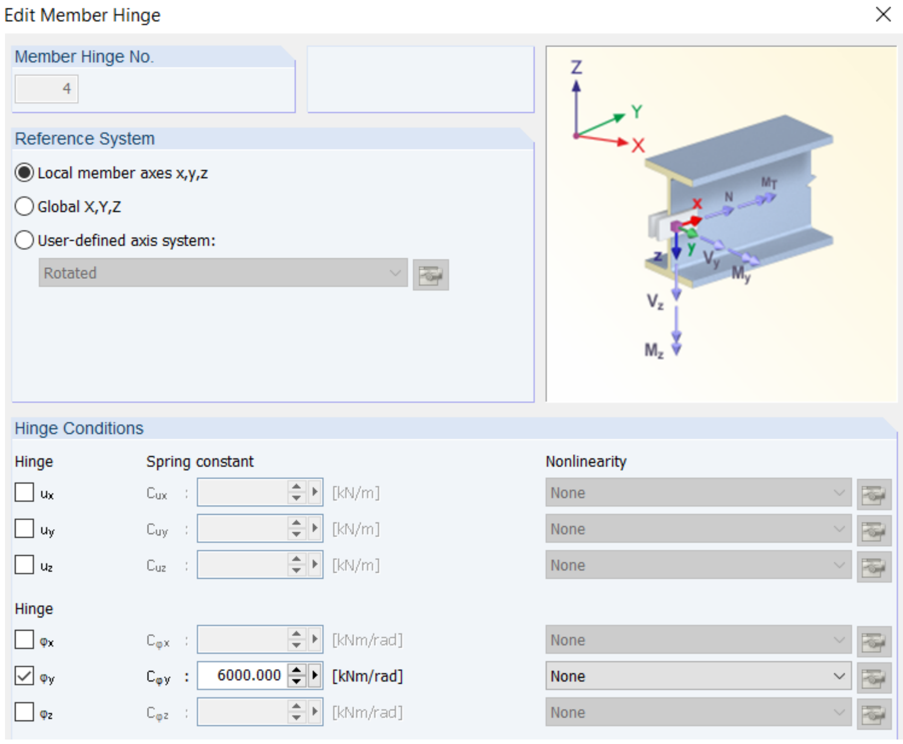


Figure 3.28 Modelling of the member line hinge on the Kingspan structural model

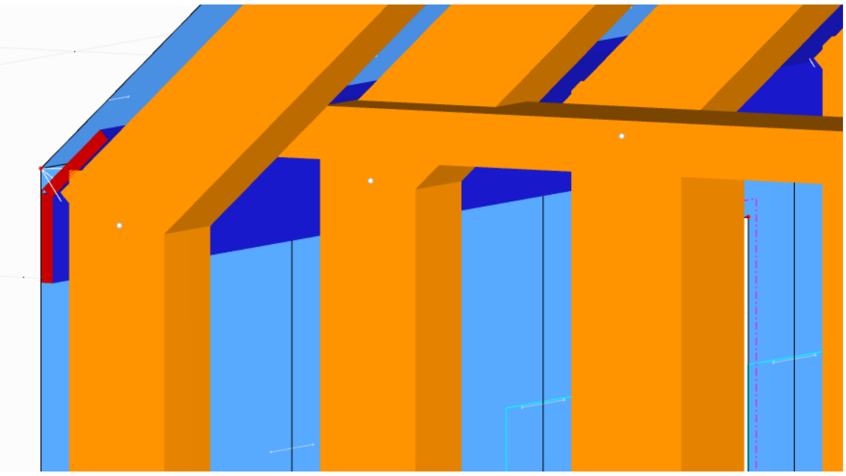


Figure 3.29 Rendering in the Kingspan structural model of the connection; the white dots represent the member hinges

For what concerns the joint of the rafter to rafter beams (ridge of the roof) and of the column to rafter beam on the south side of the building, the connection maintains the same properties as those seen now for the previous case being formed always by two plates and a dowel system of 3 columns by 6 rows of dowels of diameter 8 mm, but the general geometry, obviously following that of the structure, changes. The resulting bending stiffness is the same as that defined previously for the column to rafter connection, and it was modelled as seen in Figure 3.28 in the Kingspan structural model. Details of those connections are shown in the Figures below.

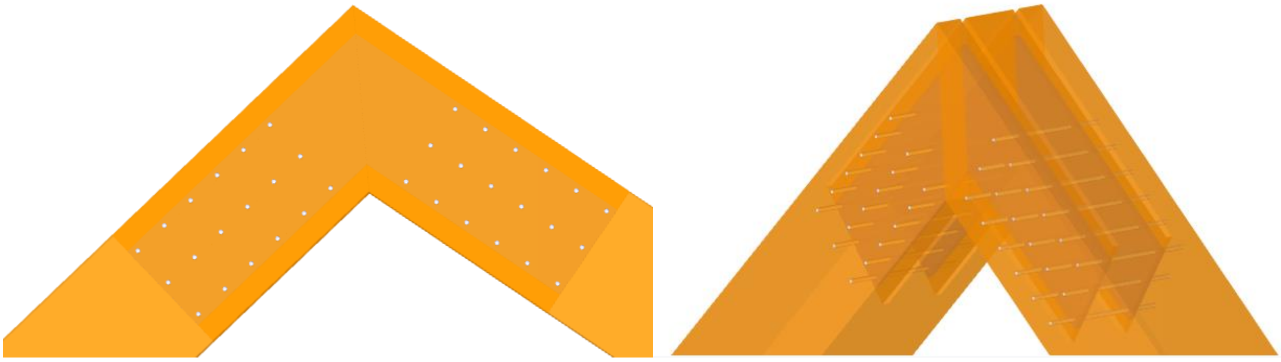


Figure 3.30 Details of the ridge connection

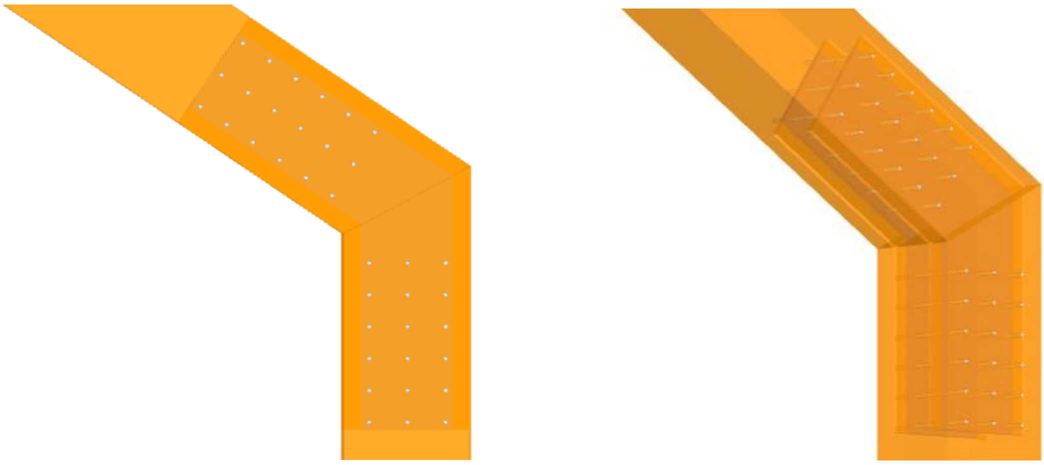


Figure 3.31 Details of the south side frame connection

Regarding the column to foundation connections, it is also formed by two steel plates 12 mm thick and dowels of diameter 8 mm, in this case placed in 5 columns and 4 rows. Details of the connection as well as its stiffness values provided by the RF Joints add-on and its modelling in the Kingspan structural model are shown in the figures below.

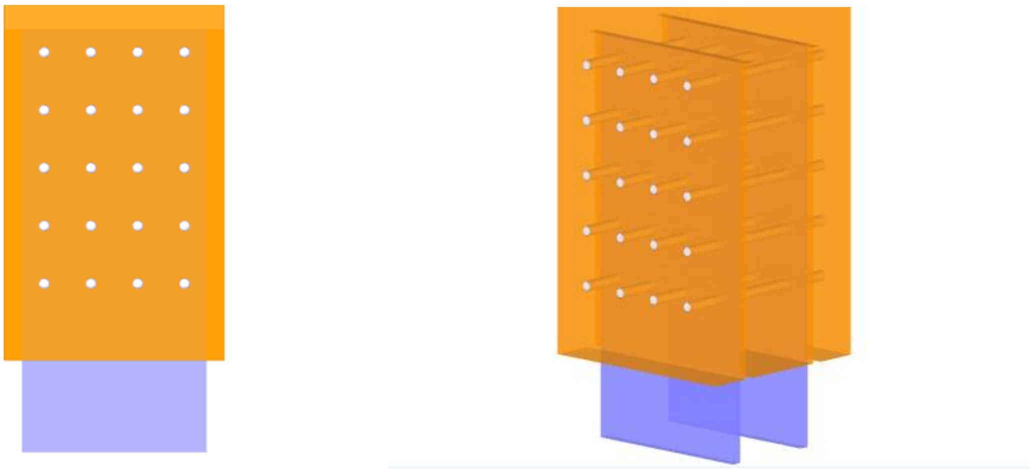


Figure 3.32 Details of the columns foundation connection

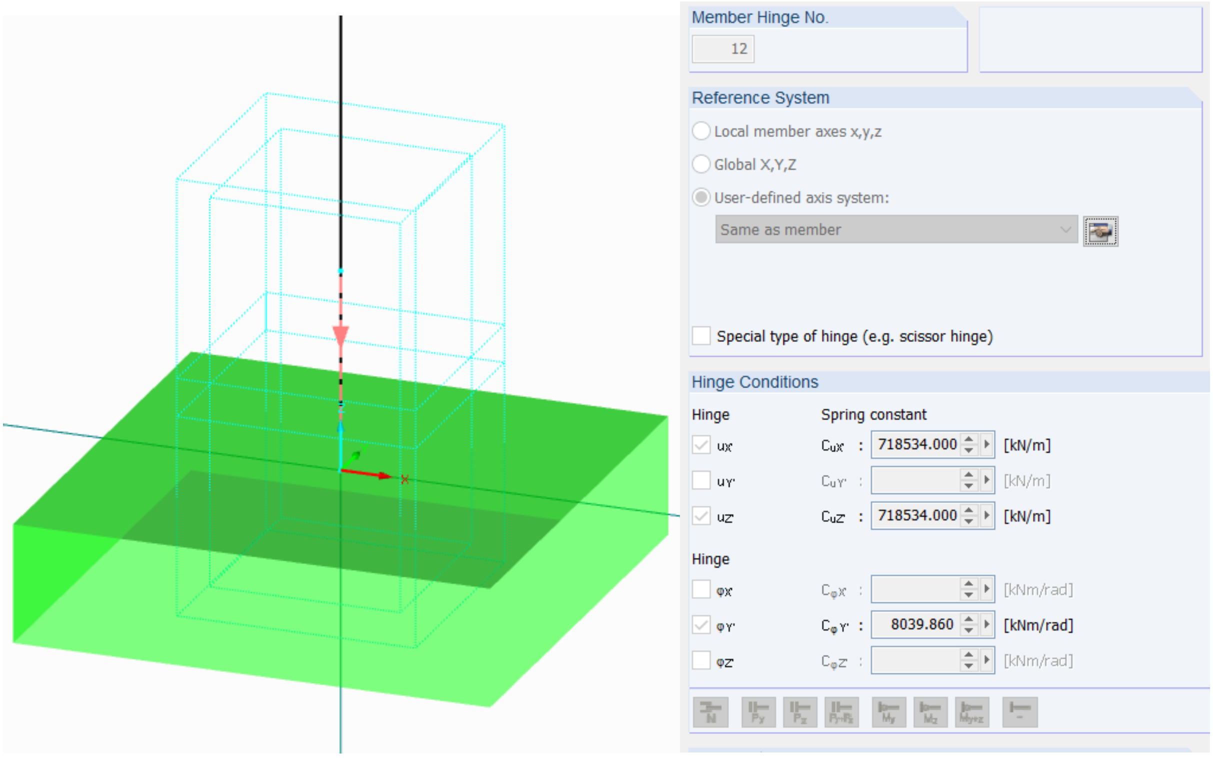


Figure 3.33 From the left to the right: Modelling of the connection by RF Joints, output stiffness values of the connection

The bending stiffness value of Figure 3.33 (8039.860 kNm/rad) was approximated to 8000 kNm/rad and inserted in the Kingspan structural model at the same node.

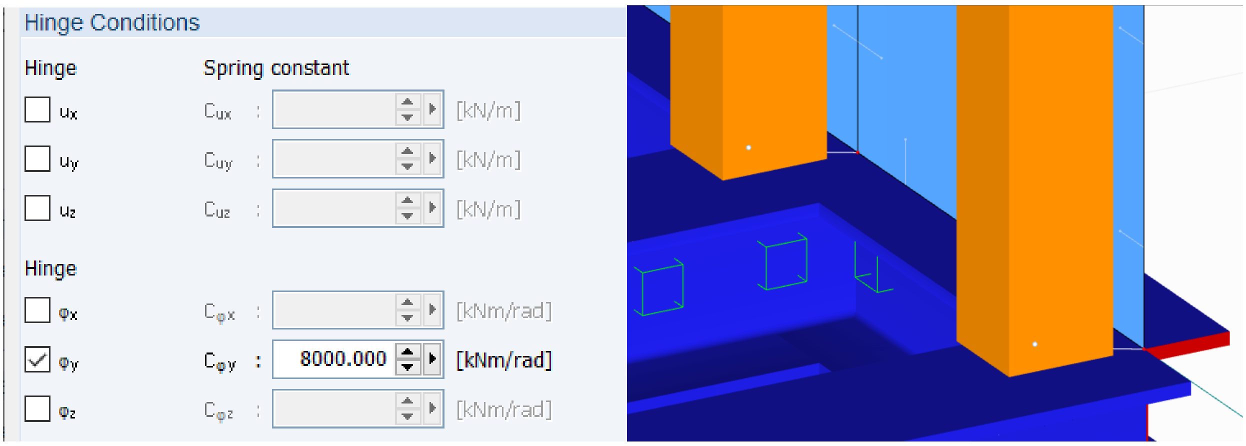


Figure 3.34 From the left to the right: modelling of the member line hinge on the Kingspan structural model, rendering in the Kingspan structural model of the connection; the white dots represent the member hinges

For what concerns the supporting frame system of the structure made with the Panelo panels, whose members have section, as mentioned in paragraph 3.2.2, of 13 cm x 24 cm, the exact geometry of the connections were not modelled on RF Joints. However, the connections were assumed to have similar characteristics to those defined above, and they were modelled in the Panelo structural model with the function “member hinge” at the single members with the same stiffness values as those found for the Kingspan structural model supporting frame.

3.2.5 The application of the loads

3.2.5.1 Snow loads

The snow loads are those defined in paragraph 2.5.2. They are applied vertically along the full span of the roof and, since the building is located at altitudes greater than 1000 m amsl, they are medium term loads. Their application in the models is shown in Figures 3.35, 3.36 and 3.37.

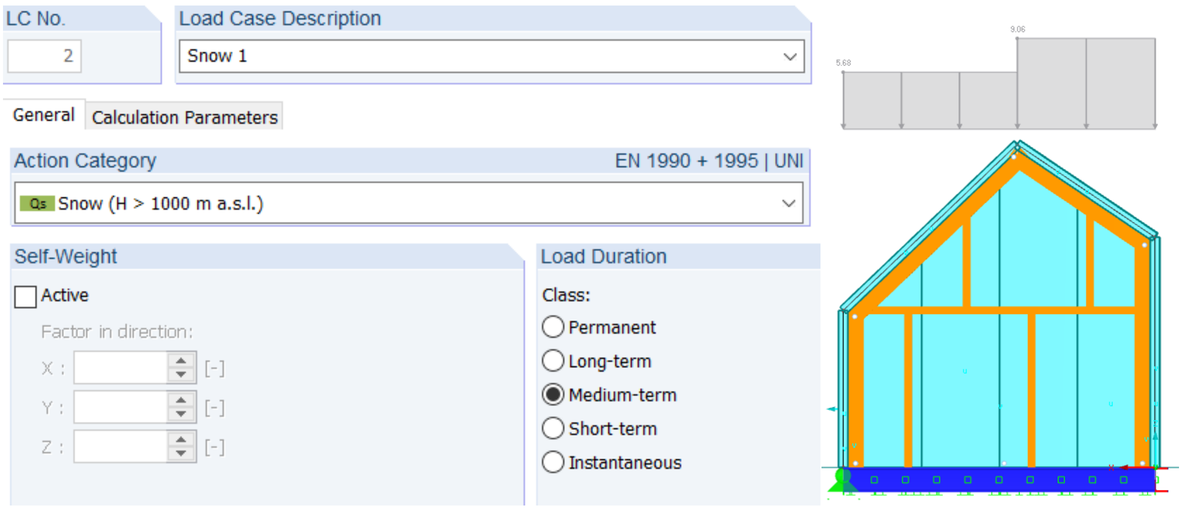


Figure 3.35 Definition and application of the load case Snow 1

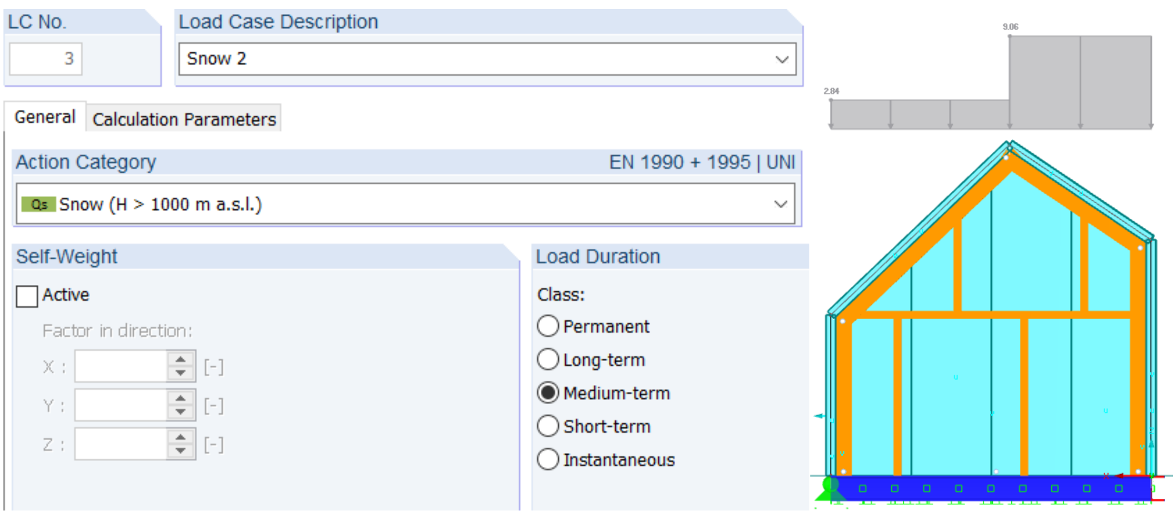


Figure 3.36 Definition and application of the load case Snow 2

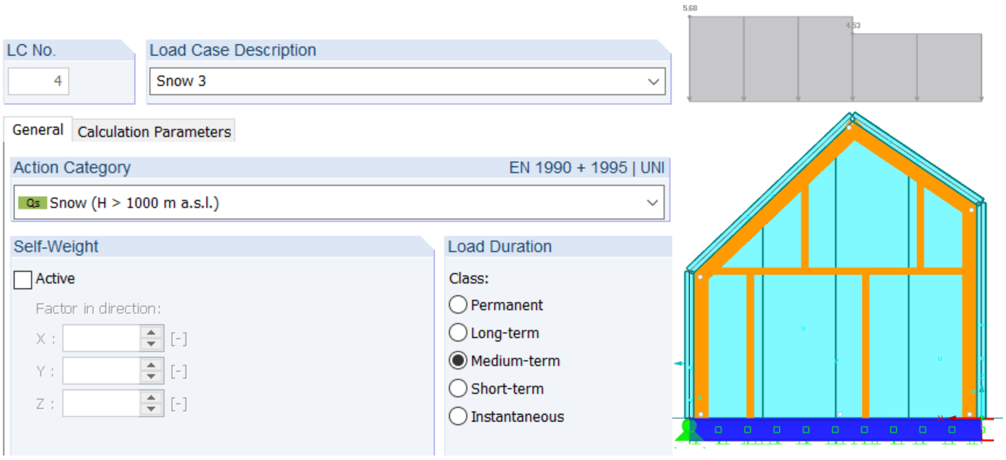


Figure 3.37 Definition and application of the load case Snow 3

3.2.5.2 Wind loads

The wind loads are those defined in paragraph 2.5.1. They are applied perpendicularly to each surface of the structure and are instantaneous loads. Their application in the models is shown in Figures 3.38, 3.39, 3.40 and 3.41.

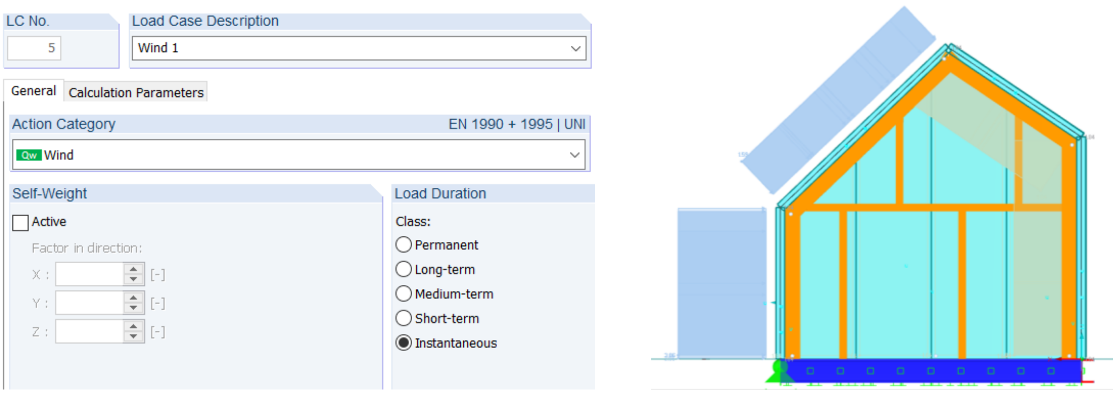


Figure 3.38 Definition and application of the load case Wind 1

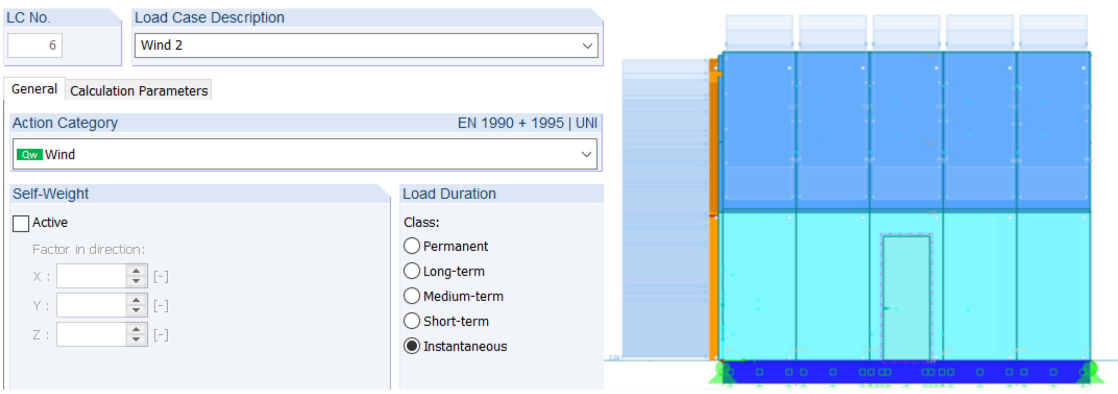


Figure 3.39 Definition and application of the load case Wind 2

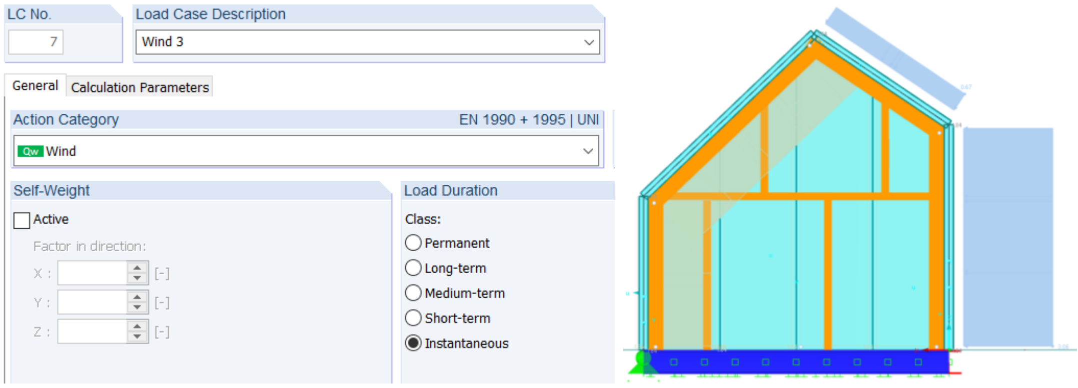


Figure 3.40 Definition and application of the load case Wind 3

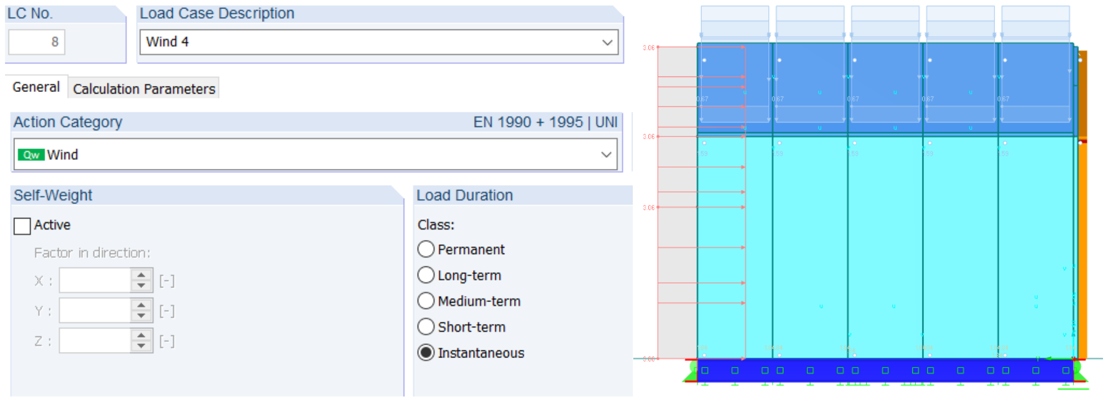


Figure 3.41 Definition and application of the load case Wind 4

3.2.5.3 Temperature

Regarding the effect of the temperature variations described in paragraph 2.5.3, their application in the Kingspan and Panelo models created a lot of problems in the initial stages of the modelling, mostly concerning the stresses that it created inside the panels. This was due to the fact that, at first, the panel joints were modelled as if only allowing rotation about the connection's main direction axis, while they were considered rigid for the in plane constraints, not allowing any translation at all along the x and y axes of the connection (see Figure 3.21 for further details). This hindered the panel completely to shrink or expand when subjected to temperature variations and created therefore huge normal stresses in it close to the connection line. In fact, this phenomenon is why the stiffnesses of the fastening connections were calculated and modelled in the first place. However, once this stiffness and partial freedom of movement was input inside the program, the effects of the temperature both in terms of stresses and strains became rather insignificant and negligible with respect to those generated by wind and snow. In terms of deformations, just to give an idea, if one were to apply the equation (2.18) ($\varepsilon = \alpha \Delta T$) with the maximum ΔT possible for the studied case, which is of 37° C, and knowing

$\alpha = 5 * 10^{-6}$, the total shrinkage in the longest span of the structure (6 m) is of 1.1 mm (with $\Delta L = \varepsilon * L$). Considering the types of connections used for the panels, it seems plausible that the system doesn't suffer too much from it, which in fact running the calculations with the software was the case. Furthermore, the addition of the temperature load cases was slowing down the computation times of the program for the analysis at the Ultimate and Serviceability Limit States (which had to be performed every time some detail was changed), and where therefore eventually removed from the model.

3.2.5.4 Limit States combinations

The Limit States load combinations applied were defined following the instructions of the NTC 2018. The analysis was performed for two combinations:

- The Fundamental Ultimate Limit State combination (for persistent and transient design situations)

$$\gamma_{G1} * G_1 + \gamma_{G2} * G_2 + \gamma_P * P + \gamma_{Q1} * Q_{k1} + \gamma_{Q2} * \psi_{02} * Q_{k2} + \gamma_{Q3} * \psi_{03} * Q_{k3} + \cdots \qquad (3.25)$$

- The Characteristic (or rare) for irreversible Serviceability Limit State combination

$$G_1 + G_2 + P + Q_{k1} + \psi_{02} * Q_{k2} + \psi_{03} * Q_{k3} + \cdots \qquad (3.26)$$

From these two equations, 96 combinations resulted, 64 ULS ones and 32 SLS.

The final load cases, load types and coefficients used for the combinations are shown in the Tables below.

		Coeff γ for ULS verification				Contemp		
		EQU		A1		Ψ_0	Ψ_1	Ψ_2
G = G1 + G2 + G3 + ...	permanent actions	0,9	1,1	1	1,3			
Qs	Snow (H > 1000 m a.s.l.)	0	1,5	0	1,5	0,7	0,5	0,2
Qw	Wind	0	1,5	0	1,5	0,6	0,2	0

Table 3.11 Partial safety and contemporaneity coefficients

	Load case	Load type	Action
1	Self weight	Permanent	G
2	Snow 1	Snow	Q_S
3	Snow 2	Snow	Q_S
4	Snow 3	Snow	Q_S
5	Wind 1	Wind	Q_W
6	Wind 2	Wind	Q_W
7	Wind 3	Wind	Q_W
8	Wind 4	Wind	Q_W

Table 3.12 List of load cases and load types

ULS Verification STR A1								
	Permanent	Snow	Snow	Snow	Wind	Wind	Wind	Wind
	G	Q_s			Q_w			
COMBO's name	LC1	LC2	LC3	LC4	LC5	LC6	LC7	LC8
CO1	1,3	/	/	/	/	/	/	/
CO2	1,3	1,5	/	/	/	/	/	/
CO3	1,3	/	1,5	/	/	/	/	/
CO4	1,3	/	/	1,5	/	/	/	/
CO5	1,3	1,5	/	/	0,9	/	/	/
CO6	1,3	1,5	/	/	/	0,9	/	/
CO7	1,3	1,5	/	/	/	/	0,9	/
CO8	1,3	1,5	/	/	/	/	/	0,9
CO9	1,3	/	1,5	/	0,9	/	/	/
CO10	1,3	/	1,5	/	/	0,9	/	/
CO11	1,3	/	1,5	/	/	/	0,9	/
CO12	1,3	/	1,5	/	/	/	/	0,9
CO13	1,3	/	/	1,5	0,9	/	/	/
CO14	1,3	/	/	1,5	/	0,9	/	/
CO15	1,3	/	/	1,5	/	/	0,9	/
CO16	1,3	/	/	1,5	/	/	/	0,9
CO17	1,3	/	/	/	1,5		/	/
CO18	1,3	/	/	/	/	1,5		/
CO19	1,3	/	/	/	/	/	1,5	/
CO20	1,3	/	/	/	/	/	/	1,5
CO21	1,3	1,05	/	/	1,5	/	/	/
CO22	1,3	1,05	/	/	/	1,5		/
CO23	1,3	1,05	/	/	/	/	1,5	/
CO24	1,3	1,05	/	/	/	/	/	1,5
CO25	1,3	/	1,05	/	1,5		/	/
CO26	1,3	/	1,05	/	/	1,5		/
CO27	1,3	/	1,05	/	/	/	1,5	/
CO28	1,3	/	1,05	/	/	/	/	1,5
CO29	1,3	/	/	1,05	1,5		/	/
CO30	1,3	/	/	1,05	/	1,5		/
CO31	1,3	/	/	1,05	/	/	1,5	/
CO32	1,3	/	/	1,05	/	/	/	1,5
CO33	1	/	/	/	/	/	/	/
CO34	1	1,5	/	/	/	/	/	/
CO35	1	/	1,5	/	/	/	/	/
CO36	1	/	/	1,5	/	/	/	/
CO37	1	1,5	/	/	0,9	/	/	/
CO38	1	1,5	/	/	/	0,9	/	/
CO39	1	1,5	/	/	/	/	0,9	/
CO40	1	1,5	/	/	/	/	/	0,9
CO41	1	/	1,5	/	0,9	/	/	/
CO42	1	/	1,5	/	/	0,9	/	/
CO43	1	/	1,5	/	/	/	0,9	/
CO44	1	/	1,5	/	/	/	/	0,9
CO45	1	/	/	1,5	0,9	/	/	/
CO46	1	/	/	1,5	/	0,9	/	/
CO47	1	/	/	1,5	/	/	0,9	/
CO48	1	/	/	1,5	/	/	/	0,9
CO49	1	/	/	/	1,5	/	/	/
CO50	1	/	/	/	/	1,5		/
CO51	1	/	/	/	/	/	1,5	/
CO52	1	/	/	/	/	/	/	1,5
CO53	1	1,05	/	/	1,5		/	/
CO54	1	1,05	/	/	/	1,5		/
CO55	1	1,05	/	/	/	/	1,5	/
CO56	1	1,05	/	/	/	/	/	1,5
CO57	1	/	1,05	/	1,5	/	/	/
CO58	1	/	1,05	/	/	1,5		/
CO59	1	/	1,05	/	/	/	1,5	/
CO60	1	/	1,05	/	/	/	/	1,5
CO61	1	/	/	1,05	1,5	/	/	/
CO62	1	/	/	1,05	/	1,5		/
CO63	1	/	/	1,05	/	/	1,5	/
CO64	1	/	/	1,05	/	/	/	1,5

Table 3.13 Multiplication coefficients of the load cases in the Ultimate Limit State combinations

SLS Carachteristic COMBO								
	Permanent	Snow	Snow	Snow	Wind	Wind	Wind	Wind
	G	Q_s			Q_w			
COMBO's name	**LC1**	**LC2**	**LC3**	**LC4**	**LC5**	**LC6**	**LC7**	**LC8**
CO65	1	/	/	/	/	/	/	/
CO66	1	1	/	/	/	/	/	/
CO67	1	/	1	/	/	/	/	/
CO68	1	/	/	1	/	/	/	/
CO69	1	1	/	/	0,6	/	/	/
CO70	1	1	/	/	/	0,6	/	/
CO71	1	1	/	/	/	/	0,6	/
CO72	1	1	/	/	/	/	/	0,6
CO73	1	/	1	/	0,6	/	/	/
CO74	1	/	1	/	/	0,6	/	/
CO75	1	/	1	/	/	/	0,6	/
CO76	1	/	1	/	/	/	/	0,6
CO77	1	/	/	1	0,6	/	/	/
CO78	1	/	/	1	/	0,6	/	/
CO79	1	/	/	1	/	/	0,6	/
CO80	1	/	/	1	/	/	/	0,6
CO81	1	/	/	/	1		/	/
CO82	1	/	/	/	/	1		/
CO83	1	/	/	/	/	/	1	/
CO84	1	/	/	/	/	/	/	1
CO85	1	0,7	/	/	1		/	/
CO86	1	0,7	/	/	/	1		/
CO87	1	0,7	/	/	/	/	1	/
CO88	1	0,7	/	/	/	/	/	1
CO89	1	/	0,7	/	1		/	/
CO90	1	/	0,7	/	/	1		/
CO91	1	/	0,7	/	/	/	1	/
CO92	1	/	0,7	/	/	/	/	1
CO93	1	/	/	0,7	1		/	/
CO94	1	/	/	0,7	/	1		/
CO95	1	/	/	0,7	/	/	1	/
CO96	1	/	/	0,7	/	/	/	1

Table 3.14 Multiplication coefficients of the load cases in the Serviceability Limit State combinations

3.3 The Revit model

After completing the structural analysis, the Dlubal model was then exported to Revit in order to create an architectonical more graphically detailed model; because of interoperability, and consequentially time related issues, this was accomplished only partially, and only for the case of the Kingspan panel; the Panelo structural system was not modelled on Revit.
In addition to what defined in Dlubal, the final Revit model includes architectonical details such as the stratigraphy of the panels, the spline cassettes, the timber edge elements, the wooden door, the glass wall and the glass door.
The work done will be illustrated in the following paragraphs.

3.3.1 The Panel

The Kingspan company makes available a digital model of their TEK 142 Panel system, which is accessible on the bimobject platform, a website that allows the download of different BIM models or "families" uploaded by software producers, manufacturers, etc. The file made available by Kingspan is shown in Figure 3.42.

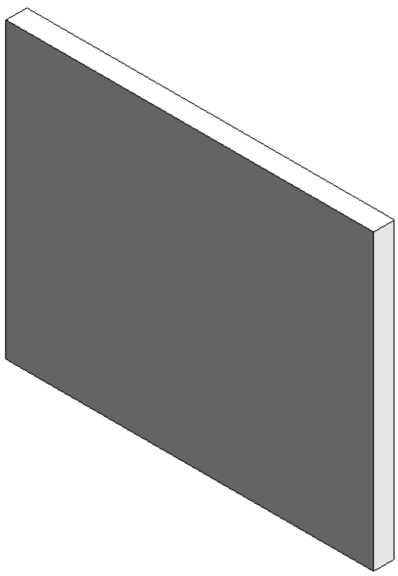


Figure 3.42 TEK digital object downloaded from bimobject

As can be seen, only a general geometry given, with one thickness; the different layers composing the panel are not modelled. However, in addition to this three-dimensional element, the file also contains information input about the panel's insulating properties as well as other kinds of data like the “GlobalWarmingPotential” and the Fire rating. Parts of these information are shown in Figure 3.44.

Famiglia: Famiglia di sistema: Muro di base

Tipo: NBS_KingspanInsulation_StructuralInsulatedWallPanels_Kingsp

Carica... Duplica... Rinomina...

Parametri tipo

Parametro	Valore
Proprietà analitiche	
Coefficiente di scambio termico (U)	
Resistenza termica (R)	
Massa termica	
Assorbimento	0.100000
Ruvidità	1
Dati identità	
Immagine tipo	
Nota chiave	
Modello	
Produttore	technical@kingspaninsulation.co.uk
Commenti sul tipo	
URL	
Descrizione	Structural Insulated Panels (SIPs) for infillin
Descrizione assieme	
Codice assieme	
Contrassegno tipo	
Resistenza al fuoco	

Parametri IFC	
AcousticRating	n/a
Combustible	☑
Compartmentation	☐
ExtendToStructure	☐
FireRating	Euroclass D
IsExternal	☐
LoadBearing	☑
Reference	n/a
Status	UNSET
SurfaceSpreadOfFlame	n/a
ThermalTransmittance	5.100000
IfcExportAs	IfcWallType
IfcExportType	USERDEFINED
Dati	
Framing	n/a
GlobalWarmingPotential	Less than 5
GreenGuideRating	A or A+ depending on the full wall construc
InsulationMaterial	Polyurethane (PUR) foam board
InsulationThickness	112.0
OtherRequirements	n/a
OverallThickness	142 mm
OzoneDepletionPotential	Zero
PanelFacings	15 mm OSB/3
Perforations	n/a
ThirdPartyProductCertification	BBA Certificate 02/S029
Altro	
AccessibilityPerformance	n/a
AssetType	Fixed
Category	Pr_20_93_85_84:Structural insulated wall pa
CodePerformance	n/a
Color	n/a

Table 3.15 TEK family provided information

In order to keep this given information, the family was imported on the main model exported from Dlubal and was then used to perform the modelling of the single panel layers. Details of the modelled composite panels are shown in the pictures below.

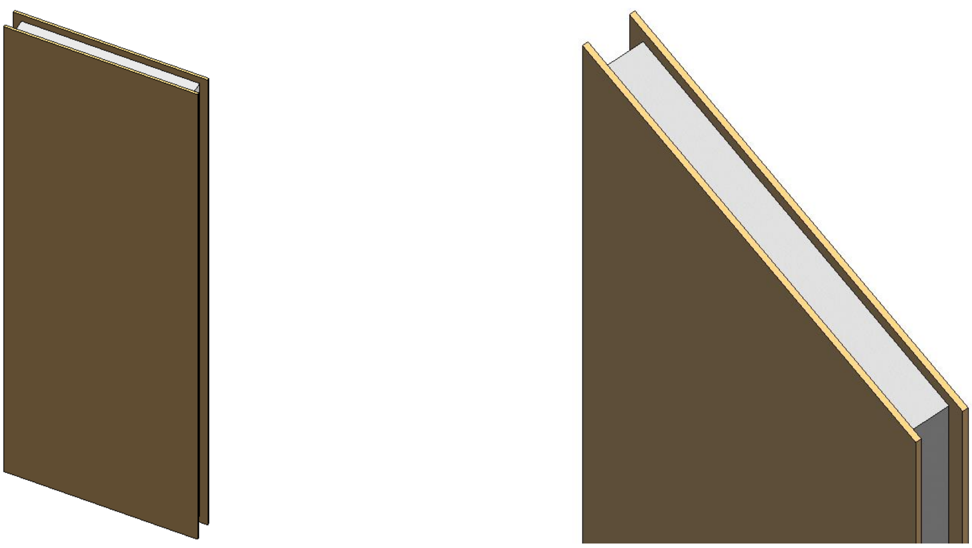


Figure 3.43 Composite panel modelling

Figure 3.45 shows the complete south wall, without the insulated splines.

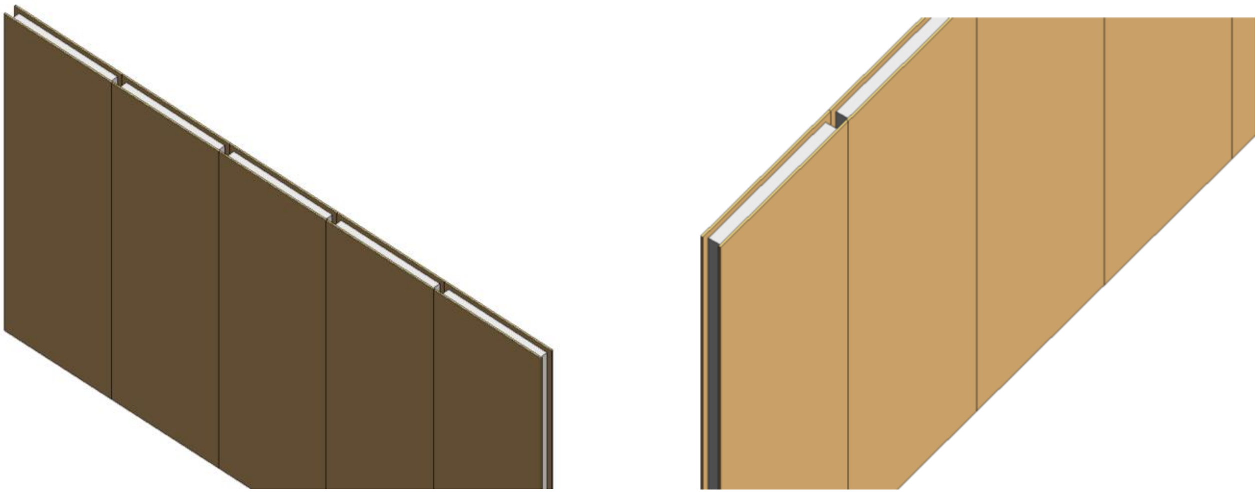


Figure 3.44 South wall with the TEK panels

3.3.2 The Insulated splines and timber edge elements

The insulated spline was modelled according to the indications given by the BBA certificate uploaded by the company and mentioned in paragraph 2.2; it has base dimensions 100 mm x 112 mm and is composed in thickness by 15 mm of OSB, 82mm of PUR and 15 mm of OSB.
Details of the spline are shown in Figures 3.45.

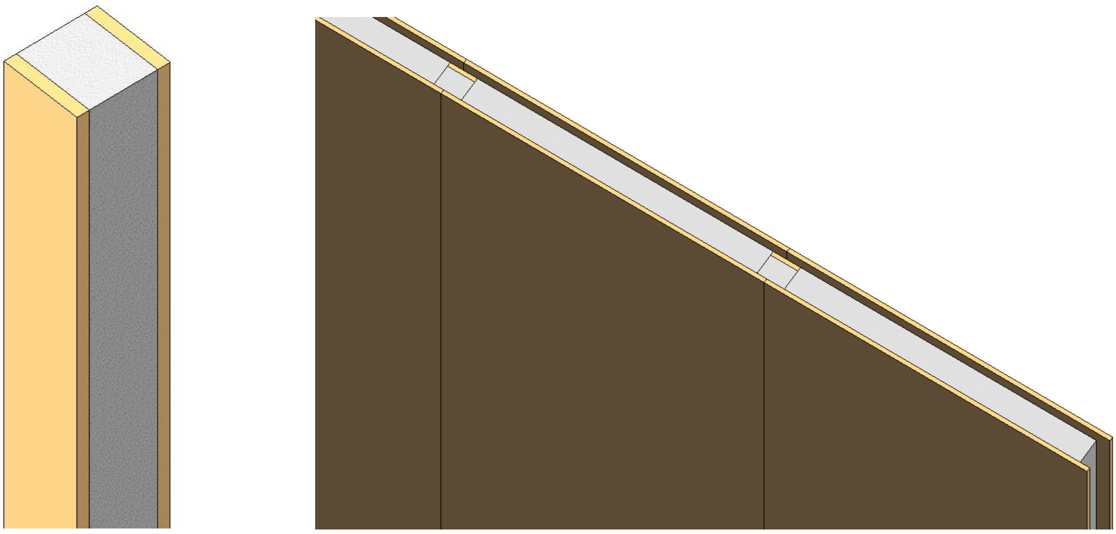


Figure 3.45 From the left to the right: detail of the insulated spline, south wall with the spline connections added

Moving on, Figure 3.46 and 3.47 show the adding to the model of the timber edge elements and of the soleplates to be placed between the panel and the foundation. The timber edge elements all have, as seen in paragraph 2.2.4, dimensions 50 mm x 112 mm while the soleplate has dimensions 40 x 142, because the whole panel base is lying on it.

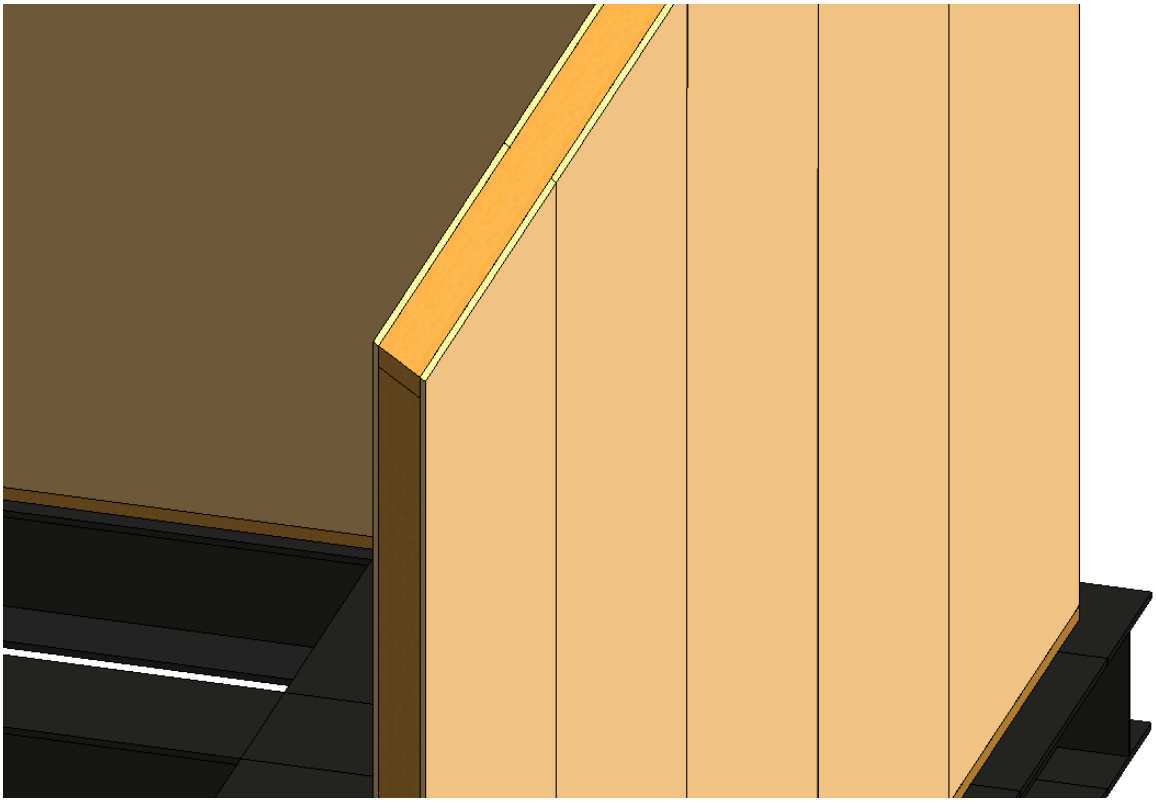


Figure 3.46 South wall with the added timber edge elements

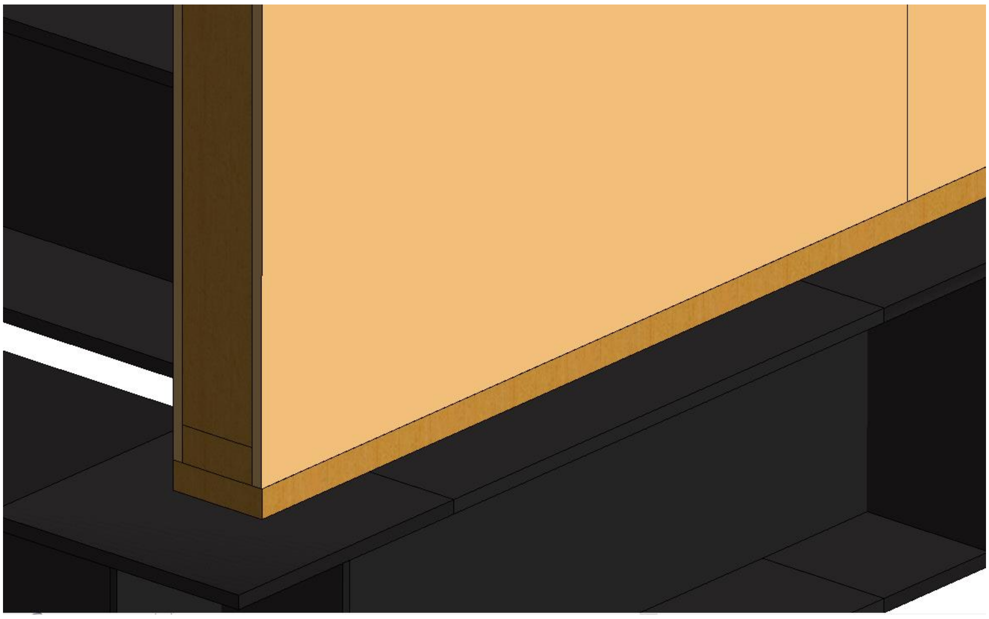


Figure 3.47 Detail of the panel, the soleplate and the foundation

3.3.3 The final geometry

The Figures presented in this paragraph show the final model of the case study as it was made on the Revit software, following the shape suggested by the structural model made on Dlubal, meaning including the foundation, the supporting frame, the steel frame-connecting elements, etc.

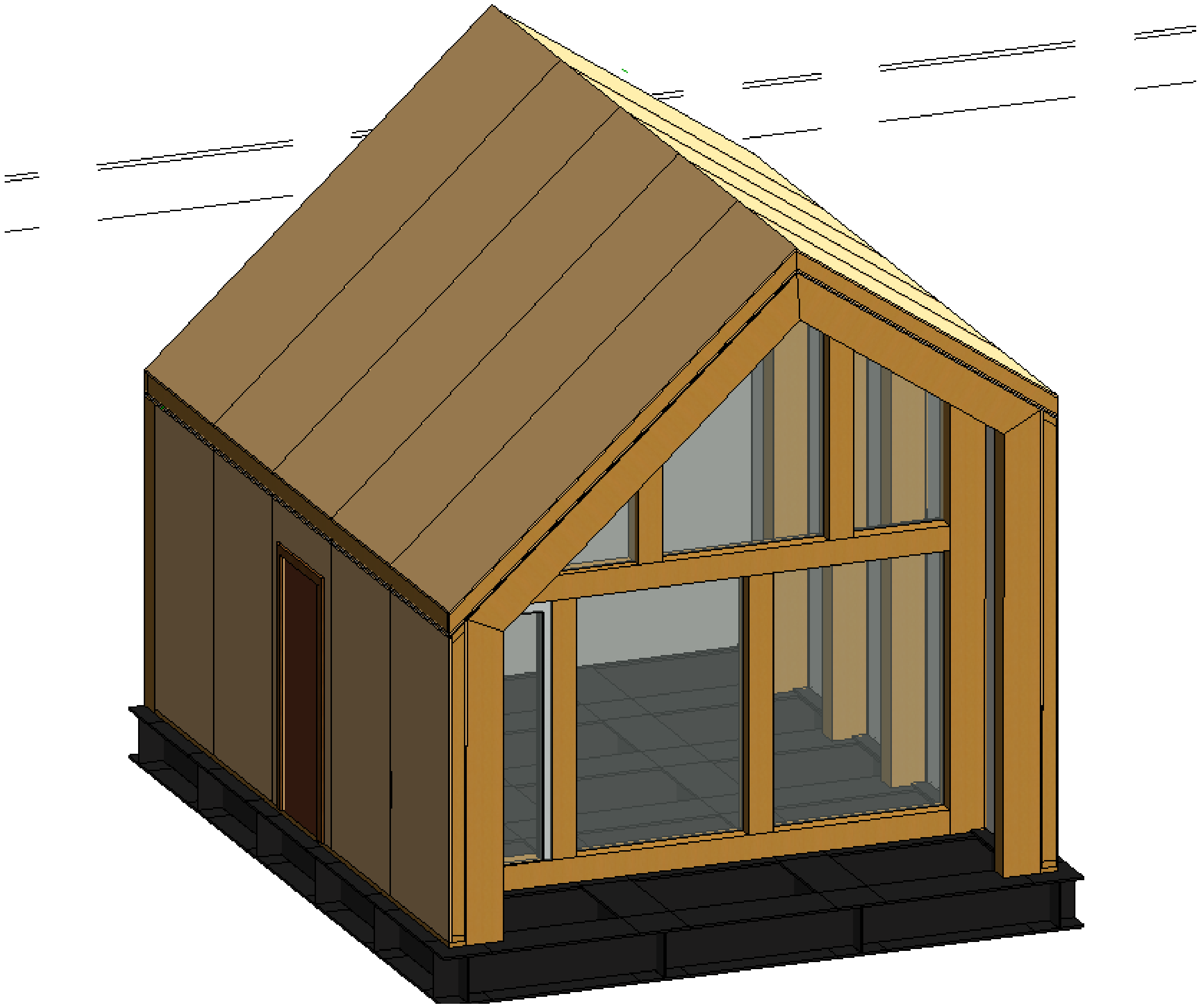


Figure 3.48 Final Revit model

Following the suggestions given by the TEK specification manual and specified already in paragraphs 2.2.2 and 2.2.3, the TEK panels were lined internally with a 15 mm layer plasterboard, which was mostly useful (as will be seen in the next chapter) for the prevention of interstitial condensation inside the panel. The fastening connections of the panels as well of those of the frame members were not added because of shortness of time.

Figure 3.48 Detail of the frame connecting member and of the internal side plasterboard lining

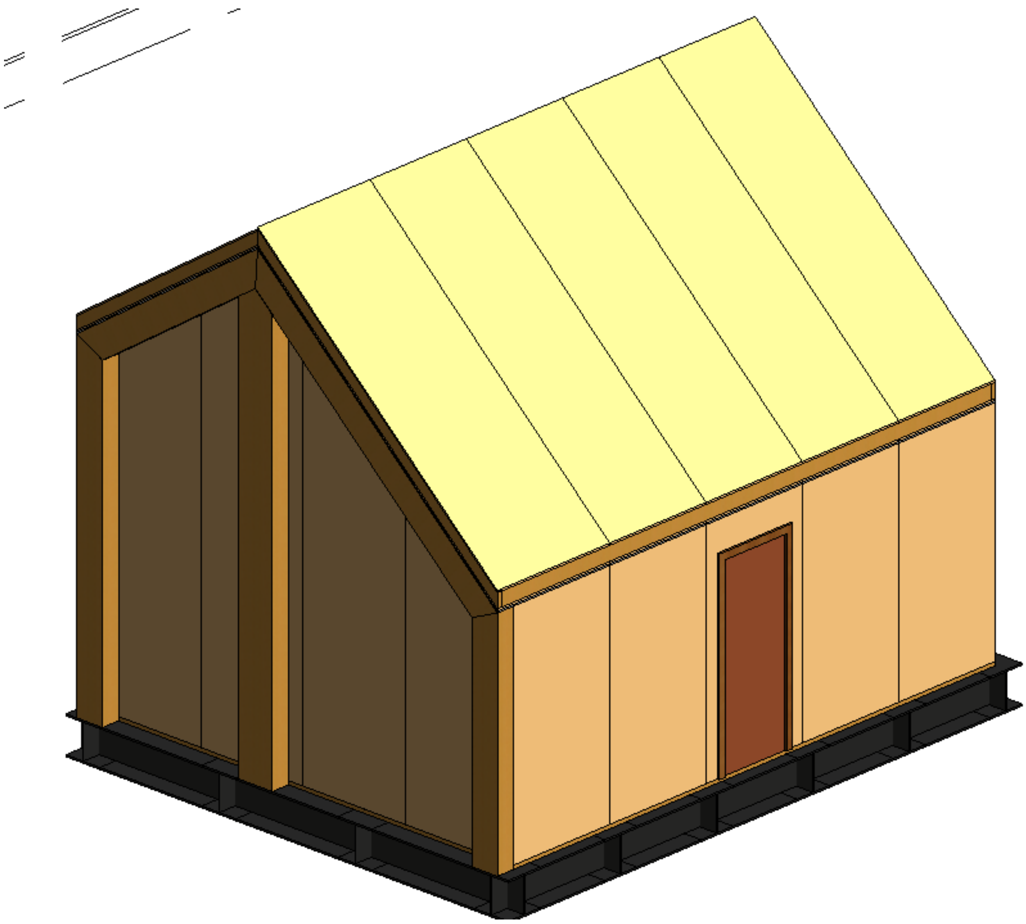


Figure 3.49 View of the east side frame of the building

Figure 3.50 The glass wall, the glass door (1m x 2.42 m) and the glass supporting frame

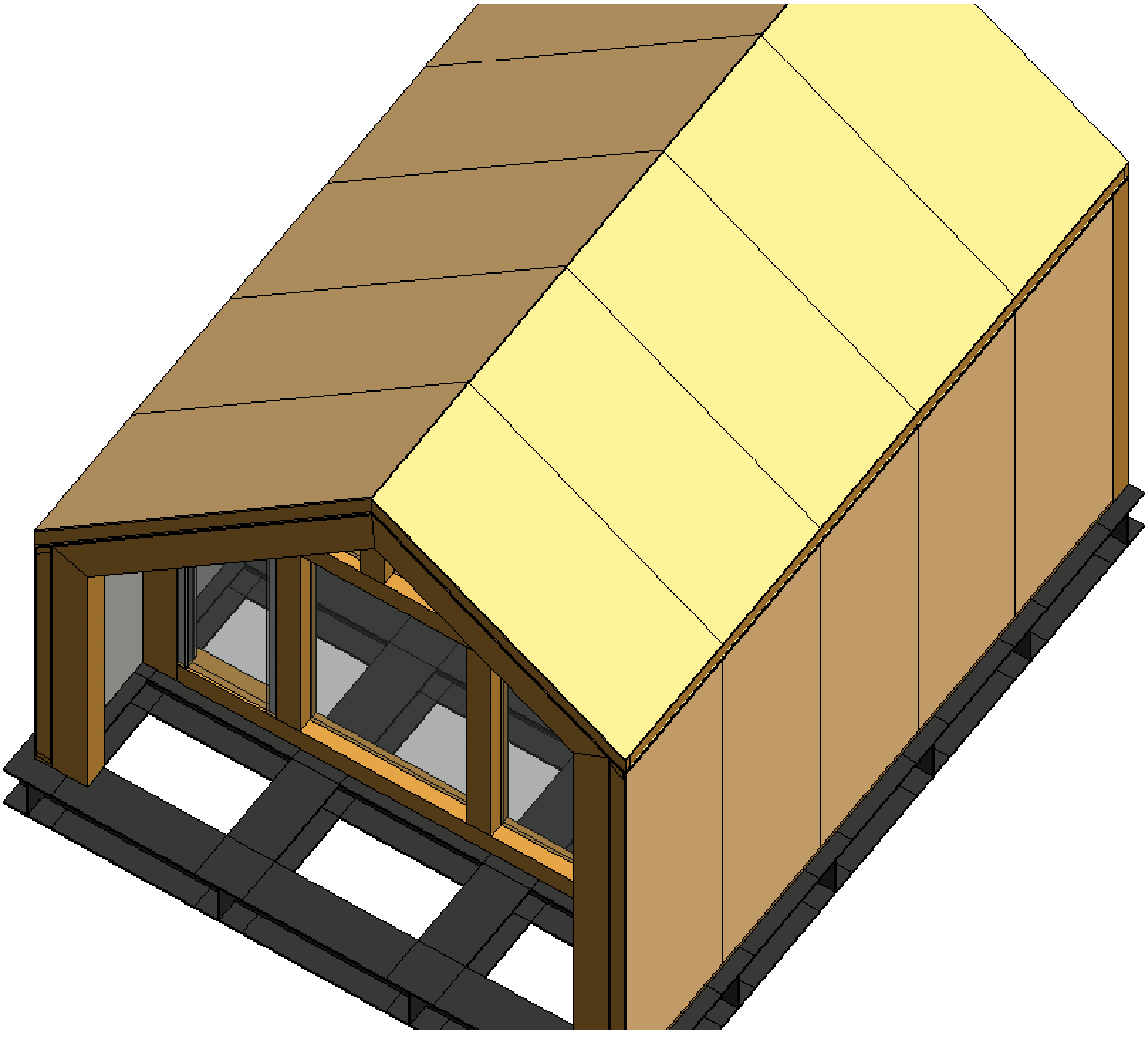


Figure 3.51 View of the building from above

3.4 Interoperability

The Dlubal-Revit interoperability was rather problematic. Before exposing the difficulties encountered when moving from one software to the other though, a reminder must be done of the definition of interoperability, already introduced in paragraph 1.3.3. In fact, as mentioned in that paragraph, one of the huge advantages of the BIM methodology is that it allows to create one single model containing all the information needed about the structure, in the ideal case (which was here not reached) an As-built digital building that faithfully recreates the real product; however, seeing how in this thesis two different software are employed and for pages and pages the “Dlubal model” and the “Revit model” are described, one could get confused about the meaning of “single” model. In reality, the final BIM model is neither the Dlubal model nor the Revit model nor both, but much more the IFC file (or a similar standardized type of file, see paragraph 1.3.3 for further specifications) that is created through these programs and that can then be exported from one software to the other and be further elaborated by these, in order to get improved in amount of information. In this sense, in order to achieve real interoperability and test its effectiveness the programs must rely on a common information exchange reference model, which is indeed the IFC. To do this, the model created on a software must first be converted to an IFC file, and can then be opened on another software; there are then tools that allow to convert the opened IFC data in local file format of the employed software (for example .rvt for Revit). Many programs may have additional tools that allow direct data transfer with other programs, but this is not interoperability, it’s compatibility.

Returning to the performed work, the two interoperability steps that were performed, or tried to perform, were all regarding the Kingspan structural model; once from Dlubal to Revit after the structural analysis to further model the building and once from Revit to Dlubal, just for the sake of testing how well the interoperability in both ways between these two software works.

In the first case of Dlubal to Revit, the interoperability basically didn’t work at all; in fact, the Revit program, when trying to open the IFC file, always gave error messages. After many trials in different ways, the Dlubal model was eventually exported on Revit through a compatibility tool offered by these two software. The result of the exportation are shown in Figures 3.52 and 3.53; as can be seen, a lot of data regarding the geometry of the structure, like the eccentricity of the members and the element’s materials was lost. By performing the compatibility different times, different outcomes were obtained. For example, as can be seen, in Figure 3.52 some elements didn’t lose their material information (the timber frame columns), while in Figure 3.53 all elements lost their information regarding materials, but less data concerning the sections shapes was lost; in both cases however, a lot of elaboration had to be performed in order to restore the right geometry.

Figure 3.52 Dlubal to Revit compatibility 1st trial

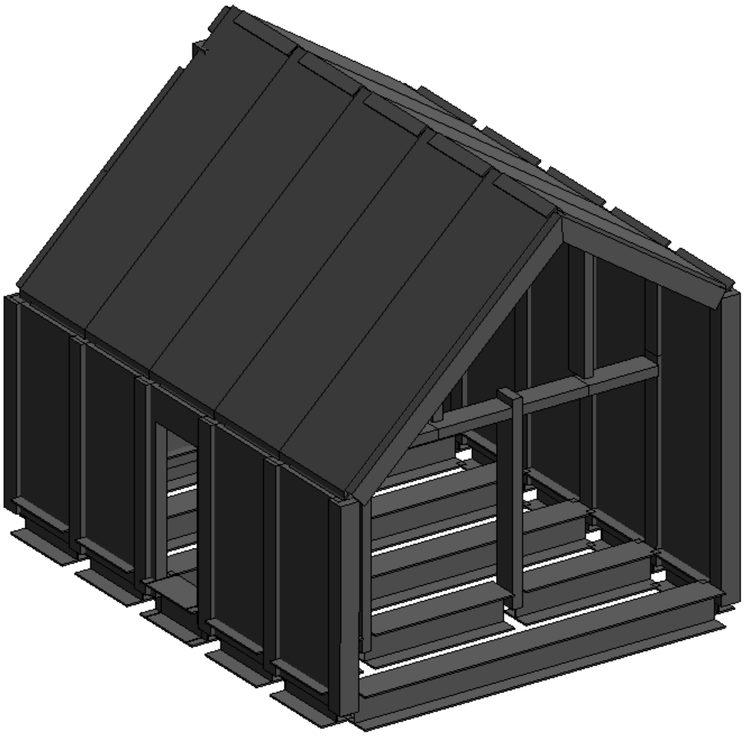


Figure 3.53 Dlubal to Revit compatibility 2nd trial

The Revit to Dlubal interoperability on the other hand worked, but also here with a lot of limitations. In fact, no additional work only was performed only because the intention was not to further model the building. However, the Revit model was exported on Dlubal once with an IFC file and once with the compatibility tool, and the results can be compared in Figures 3.54 and 3.55.

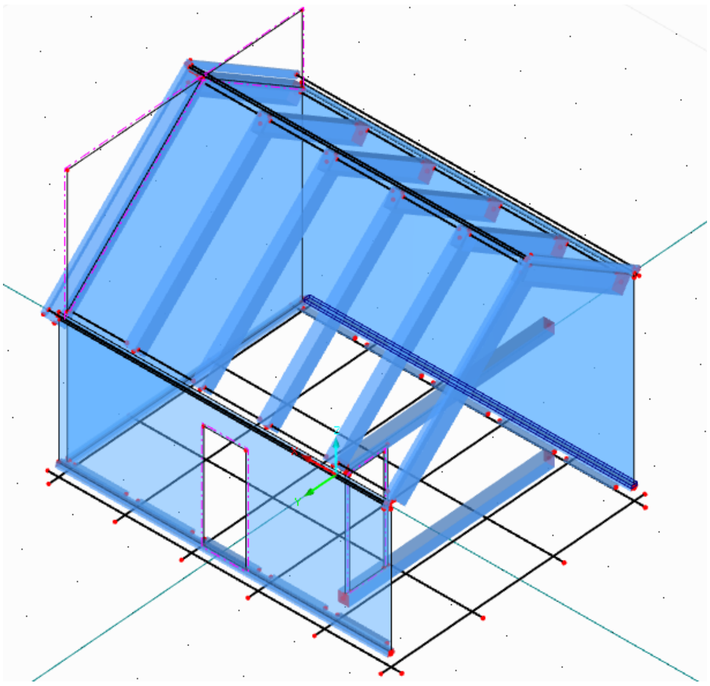


Figure 3.54 Revit-Dlubal interoperability with the IFC file

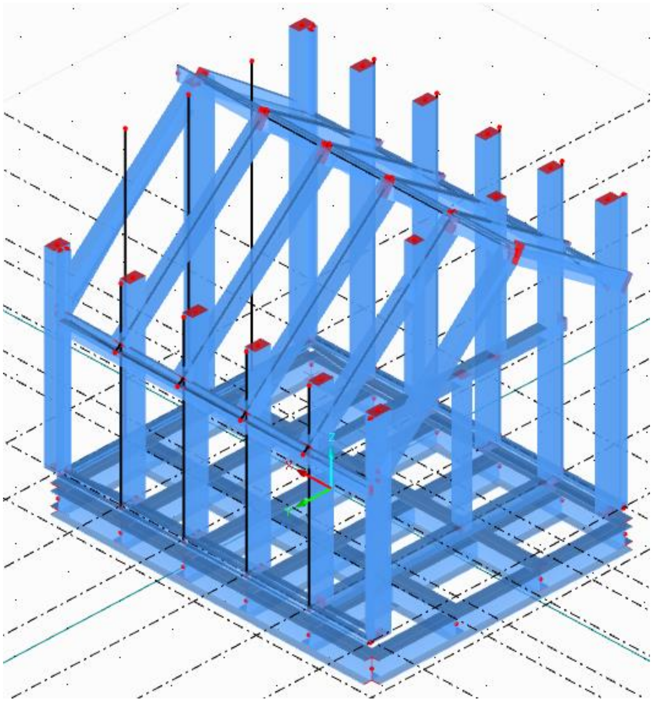


Figure 3.55 Revit-Dlubal compatibility

As can be seen, in the interoperability process (Figure 3.55) the vertical surfaces are kept while all the columns as well as the foundation members are deleted. On the contrary, with the compatibility tool (Figure 3.56) all the surfaces are lost while columns and foundation are kept even if with a wrong geometry. In both cases however, different elements are lost in the process.

Chapter 4 Results: Structural analysis, thermal performance, interstitial condensation and costs

This chapter aims to expose and compare the obtained results in terms of structural and thermal behaviour of the panels, risk of water vapour condensation and partial costs.

4.1 Structural analysis

The structural analysis was performed on the Dlubal model described in paragraph 3.2, by applying the loads defined in paragraph 2.5. The verifications at the Fundamental Ultimate Limit State and Characteristic Serviceability Limit State were applied following indications given by the CNR codes and the PDF file *"Il calcolo dell`XLAM. Basi, normative, progettazione, applicazione"* (Bernasconi) and were done only for the panels since it was the interest of this thesis. The frame elements and the single connections were not verified.

4.1.1 ULS

4.1.1.1 Kingspan structural model

The following Figures are a graphical representation of the resulting forces obtained for the Ultimate Limit State Design in the case of the Kingspan structural model.

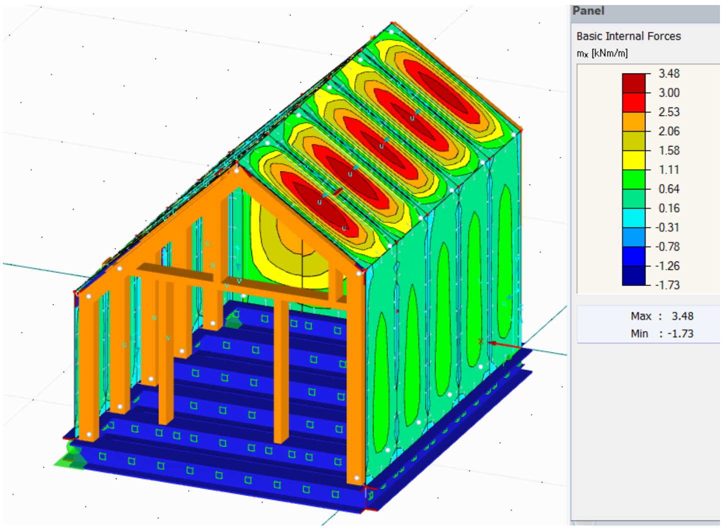


Figure 4.1 Mx at ULS

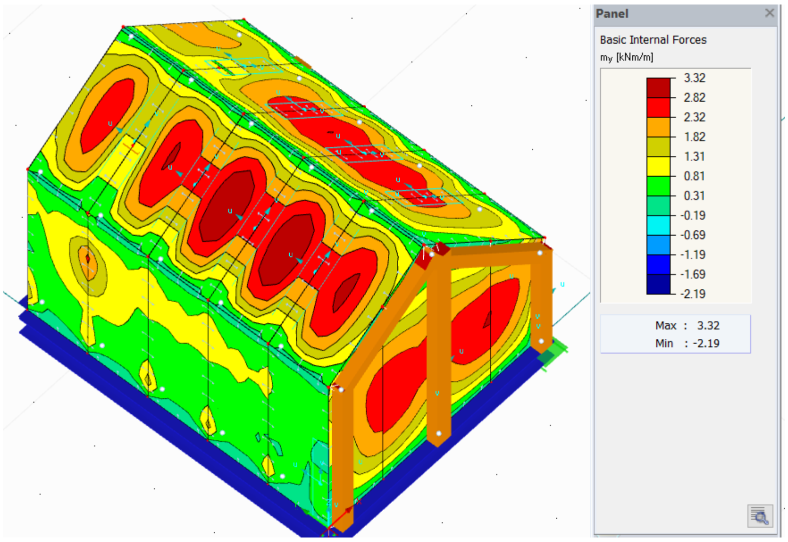


Figure 4.2 My at ULS

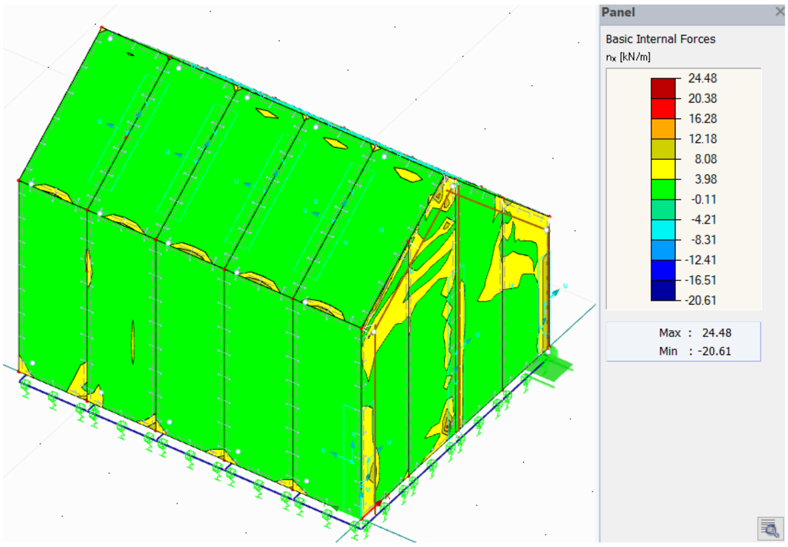


Figure 4.3 Nx at ULS

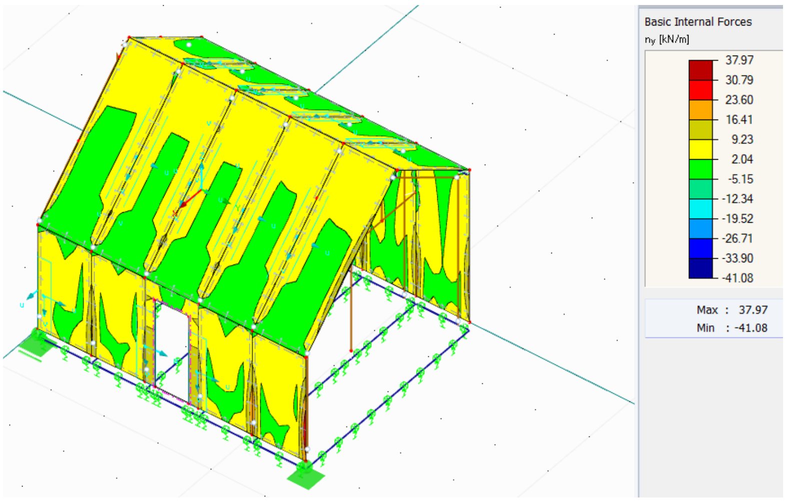


Figure 4.4 Ny at ULS

Table 4.1 shows a sample of the forces per unit length and bending moments per unit length on panel 1 obtained from the structural analysis. The values have been taken dividing the panels in grids of 10 cm * 10 cm.

Surface	Grid	Grid Point Coordinates [m]			Moments [kNm/m]			Shear Forces [kN/m]		Axial Forces [kN/m]		
No.	Point	X	Y	Z	m_x	m_y	m_{xy}	v_x	v_y	n_x	n_y	n_{xy}
1	1	4,800	4,800	0,000	0,04	0,13	0,51	0,16	0,17	13,21	19,21	1,99
					-0,04	-0,11	-0,46	-0,19	-0,06	-12,38	-16,32	-2,34
	2	4,800	4,900	0,000	0,07	0,11	0,30	0,13	0,93	10,47	10,46	1,83
					-0,06	-0,09	-0,28	-0,14	-0,26	-9,95	-9,99	-2,19
	3	4,800	5,000	0,000	0,10	0,09	0,08	0,09	1,70	7,72	1,72	1,67
					-0,08	-0,08	-0,09	-0,10	-0,46	-7,52	-3,66	-2,03
	4	4,800	5,100	0,000	0,15	0,05	0,04	0,07	1,67	5,51	3,56	1,21
					-0,12	-0,05	-0,04	-0,07	-0,80	-5,52	-5,05	-1,42
	5	4,800	5,200	0,000	0,21	0,01	0,00	0,05	1,64	3,29	5,41	0,74
					-0,16	-0,01	0,00	-0,04	-1,14	-3,52	-6,43	-0,80
	6	4,800	5,300	0,000	0,20	0,01	0,00	0,05	1,70	2,57	3,06	0,77
					-0,14	-0,02	-0,01	-0,04	-1,19	-2,74	-4,23	-0,81
	7	4,800	5,400	0,000	0,19	0,00	0,00	0,04	1,76	1,85	0,71	0,79
					-0,13	-0,03	-0,01	-0,04	-1,24	-1,96	-2,02	-0,83
	8	4,800	5,500	0,000	0,17	0,01	0,01	0,04	1,72	3,11	3,10	0,81
					-0,12	-0,02	-0,02	-0,03	-1,16	-2,76	-5,30	-0,84
	9	4,800	5,600	0,000	0,15	0,01	0,02	0,03	1,68	4,37	5,49	0,83
					-0,11	-0,01	-0,02	-0,03	-1,08	-3,56	-8,58	-0,85
	10	4,800	5,700	0,000	0,10	0,05	0,05	0,04	1,79	6,77	4,95	1,58
					-0,09	-0,05	-0,05	-0,03	-0,87	-5,58	-6,52	-1,62
	11	4,800	5,800	0,000	0,05	0,10	0,08	0,05	1,91	9,18	4,41	2,33
					-0,07	-0,09	-0,07	-0,04	-0,66	-7,60	-4,46	-2,38
	12	4,800	5,900	0,000	0,03	0,11	0,27	0,11	1,06	11,99	13,08	2,64
					-0,05	-0,09	-0,29	-0,10	-0,44	-10,04	-10,28	-2,16

Table 4.1 Sample of forces in Panel 1 obtained from the structural analysis

Table 4.2 shows the maximum and minimum values for each force and bending moment per unit length, for all panels.

max							
Moments [kNm/m]			Shear Forces [kN/m]		Axial Forces [kN/m]		
m_x	m_y	m_{xy}	v_x	v_y	n_x	n_y	n_{xy}
3,48	**3,31**	**0,86**	**11,16**	**8,84**	**23,30**	**36,77**	**6,09**

min							
Moments [kNm/m]			Shear Forces [kN/m]		Axial Forces [kN/m]		
m_x	m_y	m_{xy}	v_x	v_y	n_x	n_y	n_{xy}
-1,70	**-2,10**	**-0,81**	**-11,59**	**-7,92**	**-20,61**	**-41,03**	**-6,16**

Table 4.2

Table 4.3 contains the resistance values of the panels already seen in paragraph 2.2.1.

RESISTANCE VALUES		
N_{Rd} [kN/m]	V_{Rd} [kN/m]	M_{Rd} [kN*m/m]
50,4	**15,2**	**10,4**

Table 4.3

Table 4.4 shows, always in samples, the verifications that were performed and the equations they followed.

Surface	Grid	Grid Point Coordinates [m]						
No.	Point	X	Y	Z	$N_{Ed,x}/N_{Rd,x}$	$N_{Ed,y}/N_{Rd,y}$	$M_{Ed,x}/M_{Rd,x}$	$M_{Ed,y}/M_{Rd,y}$
1	1	4,800	4,800	0,000	0,262103175	0,381150794	0,003846154	0,0125
					0,245634921	0,323809524	0,003846154	0,010576923
	2	4,800	4,900	0,000	0,207738095	0,207539683	0,006730769	0,010576923
					0,197420635	0,198214286	0,005769231	0,008653846
	3	4,800	5,000	0,000	0,153174603	0,034126984	0,009615385	0,008653846
					0,149206349	0,072619048	0,007692308	0,007692308
	4	4,800	5,100	0,000	0,109325397	0,070634921	0,014423077	0,004807692
					0,10952381	0,100198413	0,011538462	0,004807692
	5	4,800	5,200	0,000	0,065277778	0,10734127	0,020192308	0,000961538
					0,06984127	0,127579365	0,015384615	0,000961538
	6	4,800	5,300	0,000	0,050992063	0,060714286	0,019230769	0,000961538
					0,054365079	0,083928571	0,013461538	0,001923077
	7	4,800	5,400	0,000	0,036706349	0,014087302	0,018269231	0
					0,038888889	0,040079365	0,0125	0,002884615
	8	4,800	5,500	0,000	0,061706349	0,061507937	0,016346154	0,000961538
					0,054761905	0,10515873	0,011538462	0,001923077
	9	4,800	5,600	0,000	0,086706349	0,108928571	0,014423077	0,000961538

Grid	$V_{Ed,x} < V_{Rd}$	$V_{Ed,y} < V_{Rd}$	$N_{Ed,x} < N_{Rd}$	$N_{Ed,y} < N_{Rd}$
	OK	**OK**	**OK**	**OK**
Point	**NO CRITICAL AREA**	**NO CRITICAL AREA**	**NO CRITICAL AREA**	**NO CRITICAL AREA**
1	OK	OK	OK	OK
	OK	OK	OK	OK
2	OK	OK	OK	OK
	OK	OK	OK	OK
3	OK	OK	OK	OK
	OK	OK	OK	OK
4	OK	OK	OK	OK
	OK	OK	OK	OK
5	OK	OK	OK	OK
	OK	OK	OK	OK
6	OK	OK	OK	OK
	OK	OK	OK	OK
7	OK	OK	OK	OK
	OK	OK	OK	OK
8	OK	OK	OK	OK
	OK	OK	OK	OK
9	OK	OK	OK	OK
	OK	OK	OK	OK
10	OK	OK	OK	OK
	OK	OK	OK	OK
11	OK	OK	OK	OK

Table 4.4 From above to below: Acting-resistance normal stress ratio, shear and normal force verifications

Grid	$M_{Ed,x} < M_{Rd}$	$M_{Ed,y} < M_{Rd}$	$N_{Ed,x}/N_{Rd,x} + M_{Ed,x}/M_{Rd,x} <= 1$	$N_{Ed,y}/N_{Rd,y} + M_{Ed,y}/M_{Rd,y} <= 1$
	OK	**OK**	**OK**	**OK**
Point	**NO CRITICAL AREA**	**NO CRITICAL AREA**	**NO CRITICAL AREA**	**NO CRITICAL AREA**
1	OK	OK	OK	OK
	OK	OK	OK	OK
2	OK	OK	OK	OK
	OK	OK	OK	OK
3	OK	OK	OK	OK
	OK	OK	OK	OK
4	OK	OK	OK	OK
	OK	OK	OK	OK
5	OK	OK	OK	OK
	OK	OK	OK	OK
6	OK	OK	OK	OK
	OK	OK	OK	OK
7	OK	OK	OK	OK
	OK	OK	OK	OK
8	OK	OK	OK	OK
	OK	OK	OK	OK
9	OK	OK	OK	OK
	OK	OK	OK	OK
10	OK	OK	OK	OK
	OK	OK	OK	OK
11	OK	OK	OK	OK

Table 4.5 Bending moment verification

4.1.1.1 Dlubal structural model

The following Figures are a graphical representation of the resulting forces obtained for the Ultimate Limit State Design in the case of the Dlubal structural model.

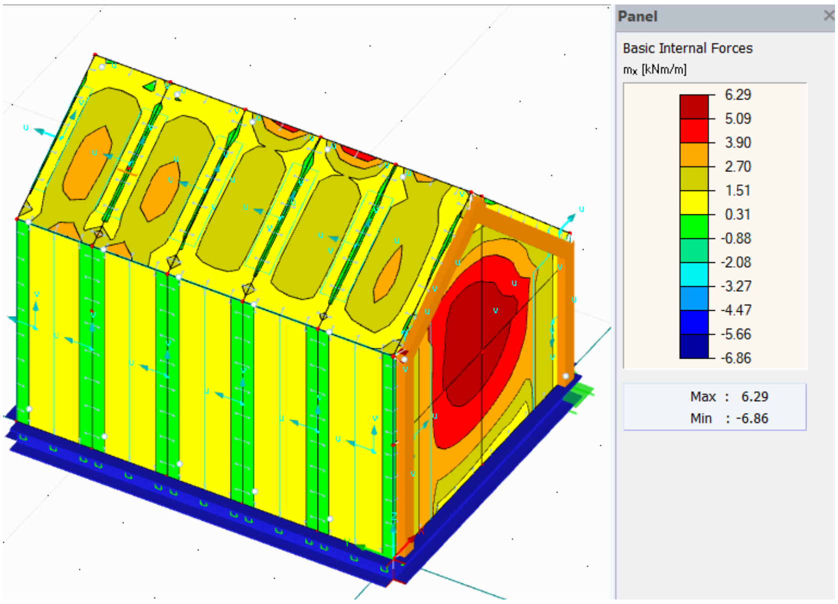


Figure 4.5 Mx at ULS

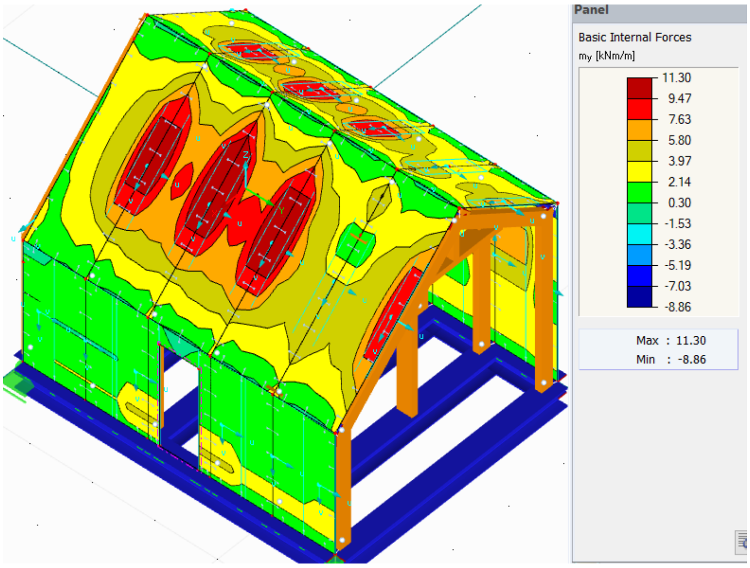


Figure 4.6 My at ULS

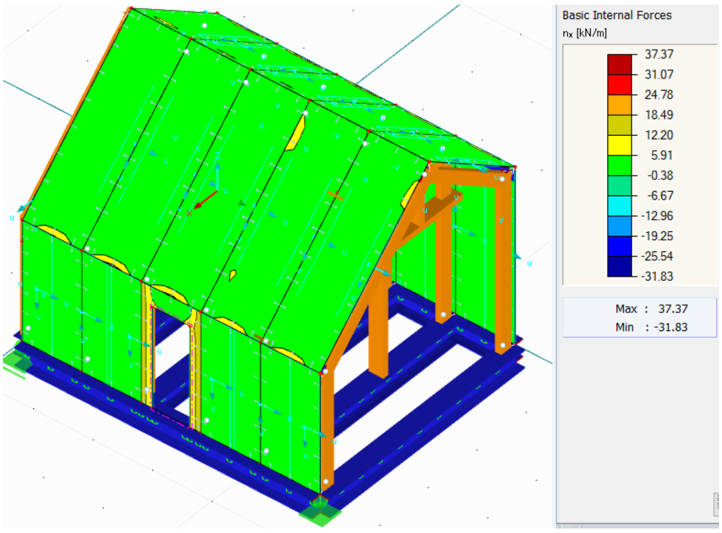


Figure 4.7 Nx at ULS

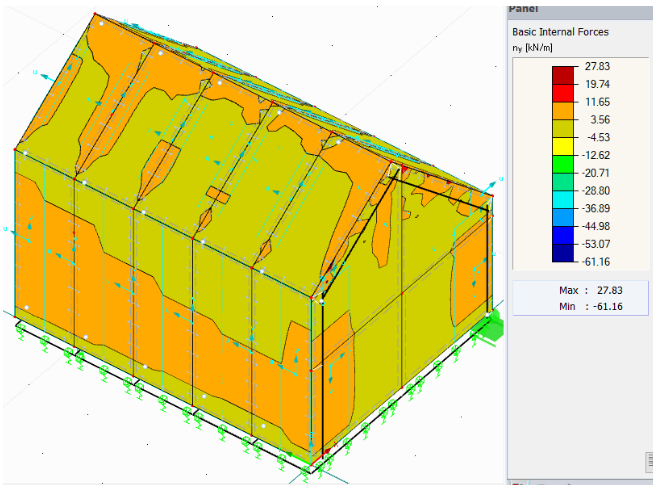


Figure 4.8 Ny at ULS

Table 4.6 shows a sample of the forces per unit length and bending moments per unit length on panel 1 obtained from the structural analysis. The values have been taken dividing the panels in grids of 10 cm * 10 cm.

Surface	Grid	Grid Point Coordinates [m]			Moments [kNm/m]			Shear Forces [kN/m]		Axial Forces [kN/m]		
No.	Point	X	Y	Z	m_x	m_y	m_{xy}	v_x	v_y	n_x	n_y	n_{xy}
1	1	4,800	4,800	0,000	0,05	0,35	0,09	2,45	2,72	2,88	0,36	4,64
					-0,07	-0,33	-0,09	-1,69	-1,10	-2,78	-1,11	-5,53
	2	4,800	4,900	0,000	0,22	0,35	0,09	2,04	2,72	2,18	0,36	4,64
					-0,21	-0,33	-0,09	-1,41	-1,10	-2,19	-1,11	-5,53
	3	4,800	5,000	0,000	0,38	0,35	0,09	1,63	2,72	1,47	0,36	4,64
					-0,35	-0,33	-0,09	-1,13	-1,10	-1,60	-1,11	-5,53
	4	4,800	5,100	0,000	0,51	0,35	0,09	1,24	2,72	0,97	0,36	4,64
					-0,47	-0,33	-0,09	-0,85	-1,10	-1,18	-1,11	-5,53
	5	4,800	5,200	0,000	0,63	0,35	0,09	0,84	2,72	0,46	0,36	4,64
					-0,58	-0,33	-0,09	-0,58	-1,10	-0,75	-1,11	-5,53
	6	4,800	5,300	0,000	0,67	0,35	0,09	0,45	2,72	0,47	0,36	4,64
					-0,59	-0,33	-0,09	-0,30	-1,10	-0,63	-1,11	-5,53
	7	4,800	5,400	0,000	0,72	0,35	0,09	0,06	2,72	0,48	0,36	4,64
					-0,60	-0,33	-0,09	-0,03	-1,10	-0,51	-1,11	-5,53
	8	4,800	5,500	0,000	0,68	0,40	0,09	0,30	3,11	0,78	1,83	4,53
					-0,55	-0,37	-0,09	-0,40	-1,52	-0,66	-2,52	-5,76
	9	4,800	5,600	0,000	0,65	0,45	0,09	0,54	3,49	1,08	3,29	4,42
					-0,50	-0,41	-0,09	-0,77	-1,93	-0,82	-3,92	-5,99
	10	4,800	5,700	0,000	0,51	0,45	0,09	0,86	3,49	1,80	3,29	4,42
					-0,40	-0,41	-0,09	-1,24	-1,93	-1,23	-3,92	-5,99
	11	4,800	5,800	0,000	0,36	0,45	0,08	1,17	3,49	2,51	3,29	4,42
					-0,31	-0,41	-0,09	-1,71	-1,93	-1,64	-3,92	-5,99
	12	4,800	5,900	0,000	0,24	0,45	0,09	1,50	3,49	3,49	3,29	4,42
					-0,17	-0,41	-0,09	-2,21	-1,93	-2,20	-3,92	-5,99

Table 4.6 Sample of forces in Panel 1 obtained from the structural analysis

Table 4.7 shows the maximum and minimum values for each force and bending moment per unit length for all panels.

max							
Moments [kNm/m]			Shear Forces [kN/m]		Axial Forces [kN/m]		
m_x	m_y	m_{xy}	v_x	v_y	n_x	n_y	n_{xy}
6,29	**11,28**	**5,38**	**10,20**	**12,35**	**34,18**	**20,58**	**8,39**

min							
Moments [kNm/m]			Shear Forces [kN/m]		Axial Forces [kN/m]		
m_x	m_y	m_{xy}	v_x	v_y	n_x	n_y	n_{xy}
-6,86	**-8,80**	**-5,40**	**-9,82**	**-11,39**	**-24,55**	**-59,41**	**-8,98**

Table 4.7

Table 4.8 contains the resistance values of the panel already seen in paragraph 2.3.1.

RESISTANCE VALUES
N_{Rd} [kN/m]
104,0454545

Table 4.8

Table 4.9 shows, always in samples, the verifications that were performed and the equations they followed. Since for the Panelo case only the N_{Rd} is given, only the relative verification was performed.

Surface	Grid	Grid Point Coordinates [m]				
No.	Point	X	Y	Z	$N_{Ed,x}/N_{Rd,x}$	$N_{Ed,y}/N_{Rd,y}$
1	1	4,800	4,800	0,000	0,026719091	0,010668414
					0,020952381	0,003460026
	2	4,800	4,900	0,000	0,021048493	0,010668414
					0,01412844	0,003460026
	3	4,800	5,000	0,000	0,015377894	0,010668414
					0,009322848	0,003460026
	4	4,800	5,100	0,000	0,011341197	0,010668414
					0,004421145	0,003460026
	5	4,800	5,200	0,000	0,007208388	0,010668414
					0,004517256	0,003460026
	6	4,800	5,300	0,000	0,006055046	0,010668414
					0,004613368	0,003460026
	7	4,800	5,400	0,000	0,004901704	0,010668414
					0,007496723	0,017588467
	8	4,800	5,500	0,000	0,006343381	0,024220183
					0,010380079	0,031620795
	9	4,800	5,600	0,000	0,007881171	0,037675841

Grid	$N_{Ed,x} < N_{Rd}$	$N_{Ed,y} < N_{Rd}$
	OK	OK
Point	**NO CRITICAL AREA**	**NO CRITICAL AREA**
1	OK	OK
	OK	OK
2	OK	OK
	OK	OK
3	OK	OK
	OK	OK
4	OK	OK
	OK	OK
5	OK	OK
	OK	OK
6	OK	OK
	OK	OK
7	OK	OK
	OK	OK
8	OK	OK
	OK	OK
9	OK	OK

Table 4.9 From above to below: Acting-resistance normal stress ratio, normal force verifications

4.1.1.3 The problem of the singularities

When performing a structural analysis on 2D surfaces, it is possible to encounter in the phenomenon of singularities. Singularities occur in a limited area due to the concentration of the stress-dependent result values, and are conditioned by the FEA methodology. In theory, the stiffness and/or the stress in an infinite size concentrate on an infinitesimal small area. They often occur at the corners of the modelled surfaces and, in reality, those singularities or the resulting stress concentrations do not occur, at least not in the extent they appear in the model. This represented a huge problem for both the Kingspan and the Dlubal model, since it was present in both cases and for a long period of time no solution was found. It was eventually solved in two ways: by applying the function "Average region" of Dlubal and, following the suggestion of the thesis supervisor, by "not caring too much about it", since it obviously doesn't represent reality.

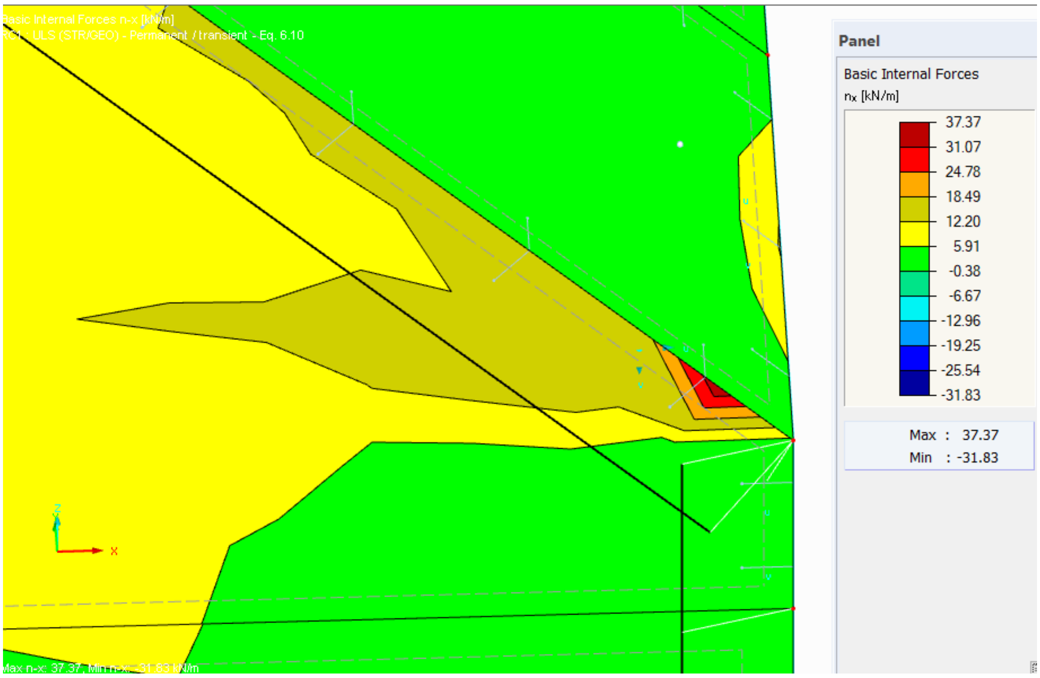


Figure 4.9 Example of singularity

4.1.2 SLS

4.1.2.1 Kingspan structural model

The following Figure shows the deformation of the Kingspan structural model obtained for the Serviceability Limit State Characteristic combination.

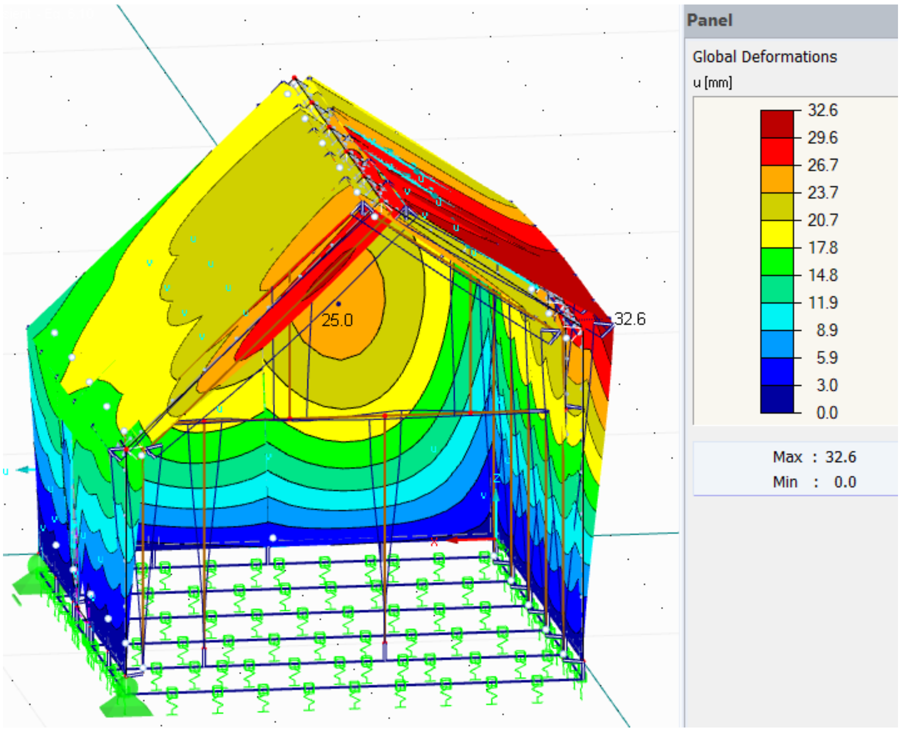


Figure 4.10 Deformation at SLS

Table 4.10 shows a sample of the values of displacement for the single grid points composing Panel 1.

Surface	Grid	Grid Point Coordinates [m]			Displacements [mm]			Rotations [mrad]		
No.	Point	X	Y	Z	u_x	u_y	u_z	φ_x	φ_y	φ_z
1	1	4,800	4,800	0,000	0,2	0,2	0,2	2,2	0,2	6,7
					-0,2	0,0	-0,2	-2,5	-0,2	-6,8
	2	4,800	4,900	0,000	0,2	0,2	0,2	3,0	0,2	3,4
					-0,2	0,0	-0,2	-3,4	-0,2	-3,5
	3	4,800	5,000	0,000	0,2	0,2	0,2	3,8	0,2	0,0
					-0,2	0,0	-0,2	-4,3	-0,2	-0,1
	4	4,800	5,100	0,000	0,2	0,1	0,1	3,8	0,2	0,0
					-0,1	0,0	-0,2	-4,3	-0,2	-0,1
	5	4,800	5,200	0,000	0,1	0,1	0,1	3,8	0,2	0,0
					-0,1	0,0	-0,1	-4,3	-0,2	-0,1
	6	4,800	5,300	0,000	0,1	0,1	0,1	3,9	0,2	0,0
					-0,1	0,0	-0,1	-4,3	-0,2	-0,1
	7	4,800	5,400	0,000	0,1	0,1	0,1	3,9	0,2	0,0
					-0,1	0,0	-0,1	-4,4	-0,2	-0,1
	8	4,800	5,500	0,000	0,1	0,1	0,1	3,9	0,1	0,0
					-0,1	0,0	-0,1	-4,3	-0,2	-0,1
	9	4,800	5,600	0,000	0,1	0,1	0,0	3,8	0,1	0,0
					0,0	0,0	-0,1	-4,2	-0,1	-0,1
	10	4,800	5,700	0,000	0,0	0,0	0,0	3,9	0,1	0,0
					0,0	0,0	0,0	-4,2	-0,1	-0,1
	11	4,800	5,800	0,000	0,0	0,0	0,0	3,9	0,1	0,0
					0,0	0,0	0,0	-4,2	-0,1	-0,2
	12	4,800	5,900	0,000	0,0	0,0	0,0	3,0	0,1	4,0
					0,0	0,0	0,0	-3,2	-0,1	-3,9

Table 4.10

Figure 4.11 shows an example of deflection of a single panel.

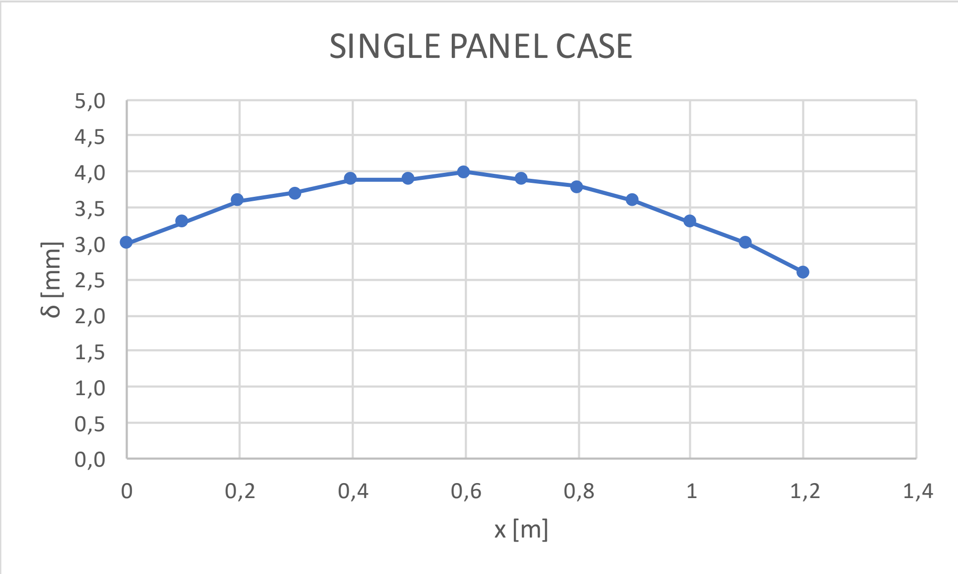


Figure 4.11

Table 4.11 gives a sample of a verification of the deflection of the centre point of the panel 1 considering the average of the deflection at the two edges of the panel. It must be verified that $\delta < \frac{L}{200}$.

Panel No	H [m]	z_1 [mm]	z_2 [mm]	z_c [mm]	$z_m=(z_1+z_2)/2$ [mm]	$\delta=z_m-z_c$ [mm]
1	0	0,2	0,0	0,1	0,1	0,0
1	0	-0,2	0,0	-0,1	-0,1	0,0
1	0,1	0,5	0,2	0,6	0,4	0,3
1	0,1	-0,5	-0,2	-0,5	-0,4	-0,2
1	0,2	0,7	0,4	1,2	0,6	0,7
1	0,2	-0,7	-0,4	-1,0	-0,6	-0,5
1	0,3	1,0	0,7	1,7	0,9	0,9
1	0,3	-1,0	-0,7	-1,4	-0,9	-0,6
1	0,4	1,4	1,0	2,1	1,2	0,9
1	0,4	-1,3	-1,0	-1,8	-1,2	-0,7
1	0,5	1,7	1,3	2,6	1,5	1,1
1	0,5	-1,6	-1,3	-2,2	-1,5	-0,8
1	0,6	2,1	1,7	3,0	1,9	1,1
1	0,6	-2,0	-1,6	-2,5	-1,8	-0,7
1	0,7	2,5	2,0	3,4	2,3	1,2
1	0,7	-2,3	-2,0	-2,9	-2,2	-0,8
1	0,8	2,9	2,4	3,9	2,7	1,3
1	0,8	-2,7	-2,4	-3,3	-2,6	-0,8
1	0,9	3,4	2,8	4,3	3,1	1,2
1	0,9	-3,1	-2,8	-3,7	-3,0	-0,8

$\delta_{max,abs}$ [mm]	$\delta_{min,abs}$ [mm]	L [m]	L/200 [mm]	CHECK
4,3	2,3	1,2	6	OK

Table 4.11

Figure 4.12 shows an example of deflection of the north wall.

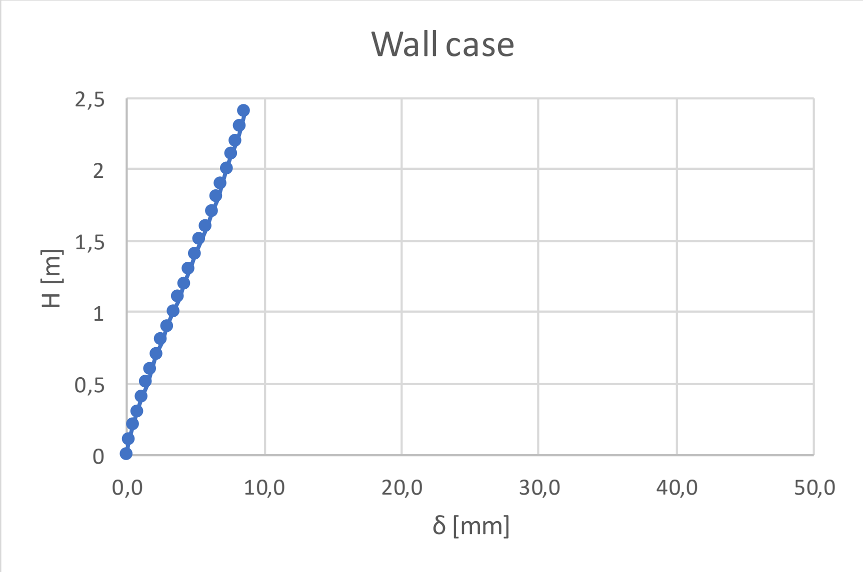


Figure 4.12

Table 4.12 gives a sample of a verification of the deflection of the north wall edge.

Wall/Roof	L [m]	H [m]	X [m]	z_{edge}//z_{ridge} [mm]	δ=z_{edge}-z_{ridge} [mm]
Edge north wall	2,4	2,4	0	9,6	9,6
Edge north wall	2,4	2,4	0	-8,8	-8,8
Edge north wall	2,4	2,4	0,1	9,6	9,6
Edge north wall	2,4	2,4	0,1	-8,8	-8,8
Edge north wall	2,4	2,4	0,2	9,6	9,6
Edge north wall	2,4	2,4	0,2	-8,9	-8,9
Edge north wall	2,4	2,4	0,3	9,6	9,6
Edge north wall	2,4	2,4	0,3	-9,0	-9,0
Edge north wall	2,4	2,4	0,4	9,6	9,6
Edge north wall	2,4	2,4	0,4	-9,0	-9,0
Edge north wall	2,4	2,4	0,5	9,6	9,6
Edge north wall	2,4	2,4	0,5	-9,0	-9,0
Edge north wall	2,4	2,4	0,6	9,6	9,6
Edge north wall	2,4	2,4	0,6	-9,1	-9,1
Edge north wall	2,4	2,4	0,7	9,6	9,6
Edge north wall	2,4	2,4	0,7	-9,1	-9,1
Edge north wall	2,4	2,4	0,8	9,6	9,6
Edge north wall	2,4	2,4	0,8	-9,1	-9,1
Edge north wall	2,4	2,4	0,9	9,7	9,7
Edge north wall	2,4	2,4	0,9	-9,1	-9,1
Edge north wall	2,4	2,4	1	9,7	9,7
Edge north wall	2,4	2,4	1	-9,1	-9,1
Edge north wall	2,4	2,4	1,1	9,7	9,7
Edge north wall	2,4	2,4	1,1	-9,1	-9,1

|$δ_{max}$| [mm]	|$δ_{min}$| [mm]	L/200 [mm]	CHECK
9,7	9,6	12	OK

Table 4.12 Edge displacement verification of the panels

4.1.2.2 Panelo structural model

The following Figure shows the deformation of the Kingspan structural model obtained for the Serviceability Limit State Characteristic combination.

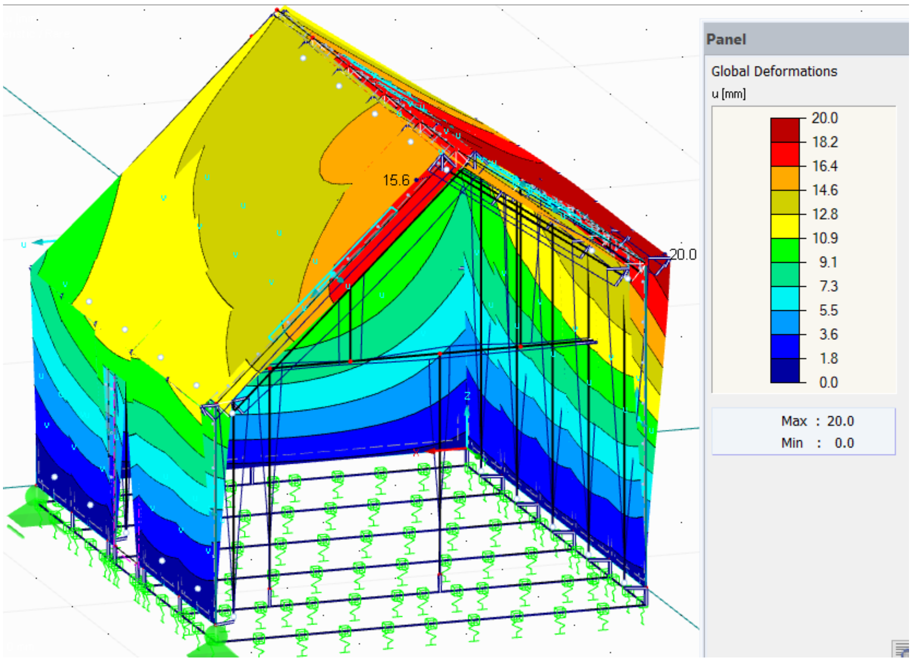


Figure 4.13 Deformation at SLS

Table 4.13 shows a sample of the values of displacement for the single grid points composing Panel 1.

Surface	Grid	Grid Point Coordinates [m]			Displacements [mm]			Rotations [mrad]		
No.	Point	X	Y	Z	u_x	u_y	u_z	φ_x	φ_y	φ_z
1	1	4,800	4,800	0,000	0,2	0,2	0,2	2,4	0,2	5,3
					-0,2	0,0	-0,2	-2,7	-0,2	-6,6
	2	4,800	4,900	0,000	0,2	0,2	0,2	3,3	0,2	2,6
					-0,2	0,0	-0,2	-3,6	-0,2	-3,3
	3	4,800	5,000	0,000	0,2	0,2	0,2	4,2	0,2	0,0
					-0,2	0,0	-0,2	-4,5	-0,2	-0,1
	4	4,800	5,100	0,000	0,2	0,1	0,1	4,1	0,2	0,0
					-0,1	0,0	-0,2	-4,4	-0,2	-0,1
	5	4,800	5,200	0,000	0,1	0,1	0,1	4,1	0,2	0,0
					-0,1	0,0	-0,1	-4,3	-0,2	-0,1
	6	4,800	5,300	0,000	0,1	0,1	0,1	4,1	0,2	0,0
					-0,1	0,0	-0,1	-4,3	-0,2	-0,1
	7	4,800	5,400	0,000	0,1	0,1	0,1	4,2	0,2	0,0
					-0,1	0,0	-0,1	-4,4	-0,2	-0,1
	8	4,800	5,500	0,000	0,1	0,1	0,1	4,2	0,1	0,0
					-0,1	0,0	-0,1	-4,3	-0,2	-0,1
	9	4,800	5,600	0,000	0,0	0,1	0,0	4,2	0,1	0,0
					0,0	0,0	0,0	-4,3	-0,1	-0,1
	10	4,800	5,700	0,000	0,0	0,1	0,0	4,3	0,1	0,0
					0,0	0,0	0,0	-4,4	-0,1	-0,1

Table 4.13

Table 4.14 gives a sample of a verification of the deflection of the centre point of the panel 1 considering the average of the deflection at the two edges of the panel. It must be verified that $\delta < \frac{L}{200}$.

Panel No	H [m]	z_1 [mm]	z_2 [mm]	z_c [mm]	$z_m=(z_1+z_2)/2$ [mm]	$\delta=z_m-z_c$ [mm]
1	0	0,2	0,0	0,1	0,1	0,0
1	0	-0,2	0,0	-0,1	-0,1	0,0
1	0,1	0,5	0,2	0,5	0,4	0,2
1	0,1	-0,5	-0,2	-0,5	-0,4	-0,2
1	0,2	0,8	0,5	0,9	0,7	0,3
1	0,2	-0,7	-0,5	-0,9	-0,6	-0,3
1	0,3	1,1	0,8	1,4	1,0	0,5
1	0,3	-1,1	-0,8	-1,3	-1,0	-0,4
1	0,4	1,5	1,1	1,8	1,3	0,5
1	0,4	-1,4	-1,1	-1,7	-1,3	-0,5
1	0,5	1,9	1,5	2,2	1,7	0,5
1	0,5	-1,8	-1,5	-2,1	-1,7	-0,5
1	0,6	2,4	1,9	2,6	2,2	0,5
1	0,6	-2,2	-1,9	-2,5	-2,1	-0,5
1	0,7	2,8	2,3	3,1	2,6	0,6
1	0,7	-2,6	-2,3	-2,9	-2,5	-0,5
1	0,8	3,3	2,7	3,5	3,0	0,5
1	0,8	-3,1	-2,8	-3,3	-3,0	-0,4
1	0,9	3,8	3,2	4,0	3,5	0,5
1	0,9	-3,5	-3,3	-3,7	-3,4	-0,3

$\delta_{max,abs}$ [mm]	$\delta_{min,abs}$ [mm]	L [m]	L/200 [mm]	CHECK
1,1	0,7	1,2	6	OK

Table 4.14

Table 4.15 gives a sample of a verification of the deflection of each wall's or roof's edge.

Wall/Roof	L [m]	H [m]	X [m]	$z_{edge}//z_{ridge}$ [mm]	$\delta=z_{edge}-z_{ridge}$ [mm]
Edge north wall	2,4	2,4	0	10,4	10,4
Edge north wall	2,4	2,4	0	-9,6	-9,6
Edge north wall	2,4	2,4	0,1	10,4	10,4
Edge north wall	2,4	2,4	0,1	-9,8	-9,8
Edge north wall	2,4	2,4	0,2	10,4	10,4
Edge north wall	2,4	2,4	0,2	-9,9	-9,9
Edge north wall	2,4	2,4	0,3	10,4	10,4
Edge north wall	2,4	2,4	0,3	-10,0	-10,0
Edge north wall	2,4	2,4	0,4	10,5	10,5
Edge north wall	2,4	2,4	0,4	-10,1	-10,1
Edge north wall	2,4	2,4	0,5	10,5	10,5
Edge north wall	2,4	2,4	0,5	-10,2	-10,2

Edge north wall	2,4	2,4	0,6	10,5	10,5
Edge north wall	2,4	2,4	0,6	-10,2	-10,2
Edge north wall	2,4	2,4	0,7	10,6	10,6
Edge north wall	2,4	2,4	0,7	-10,3	-10,3
Edge north wall	2,4	2,4	0,8	10,6	10,6
Edge north wall	2,4	2,4	0,8	-10,4	-10,4
Edge north wall	2,4	2,4	0,9	10,6	10,6
Edge north wall	2,4	2,4	0,9	-10,4	-10,4

$\lvert\delta_{max}\rvert$ [mm]	$\lvert\delta_{min}\rvert$ [mm]	L/200 [mm]	CHECK
10,7	10,6	12	OK

Table 4.15 Edge displacement verification of the panels

4.2 Thermal performance

Following the thermal transmittance values defined in the paragraphs 2.2.2 (Table 2.7) and 2.3.2 (Table 2.15) for the Kingspan system and Panelo system, a calculation of a hypothetical thermal energy needed to keep the internal building environment at 20° C with outside temperatures of -27° C (minimum temperature according to the NTC 2018) and at -10 °C (assumed average winter temperature at 3000 m a.s.l.), assuming the door to be insulated with the same characteristics of the Panels (same thermal transmittance) and the Glass to have thermal transmittance $U = 1{,}1\ \frac{W}{m^2K}$[18].

The results are shown in the Tables below.

Con riscaldamento a 20° e temperatura minima			
	KINGSPAN	GLASS	Tot
T,e [°C]	-27		
T,i [°C]	20		
ΔT [°C]	47		
A [m^2]	84,686	18,95	103,636
U [W/(m^2*K)]	0,2	1,1	0,364566367
Tp,e [°C]	-26,624	-24,932	-26,31461523
Tp,i [°C]	18,825	13,5375	17,85817259
φ [W/m^2]	9,4	51,7	17,13461924
Q [W]	796,0484	979,715	1775,7634

Table 4.16

[18] https://uk.saint-gobain-building-glass.com/en-gb/glass-and-thermal-insulation

Con riscaldamento a 20° e temperatura media invernale			
	KINGSPAN	GLASS	Tot
T,e [°C]	-10		
T,i [°C]	20		
ΔT [°C]	30		
A [m^2]	84,686	18,95	103,636
U [W/(m^2*K)]	0,2	1,1	0,364566367
Tp,e [°C]	-9,76	-8,68	-9,56252036
Tp,i [°C]	19,25	15,875	18,63287612
φ [W/m^2]	6	33	10,93699101
Q [W]	508,116	625,35	1133,466

Table 4.17

Con riscaldamento a 20° e temperatura minima				
	PANELO (WALL)	PANELO (ROOF)	GLASS	Tot
T,e [°C]	-27			
T,i [°C]	20			
ΔT [°C]	47			
A [m^2]	47,606	37,08	18,95	103,636
U [W/(m^2*K)]	0,157	0,12	0,9	0,27962
Tp,e [°C]	-26,70484	-26,7744	-25,308	-26,474
Tp,i [°C]	19,077625	19,295	14,7125	18,3572
φ [W/m^2]	7,379	5,64	42,3	13,1422
Q [W]	351,284674	209,1312	801,585	1362

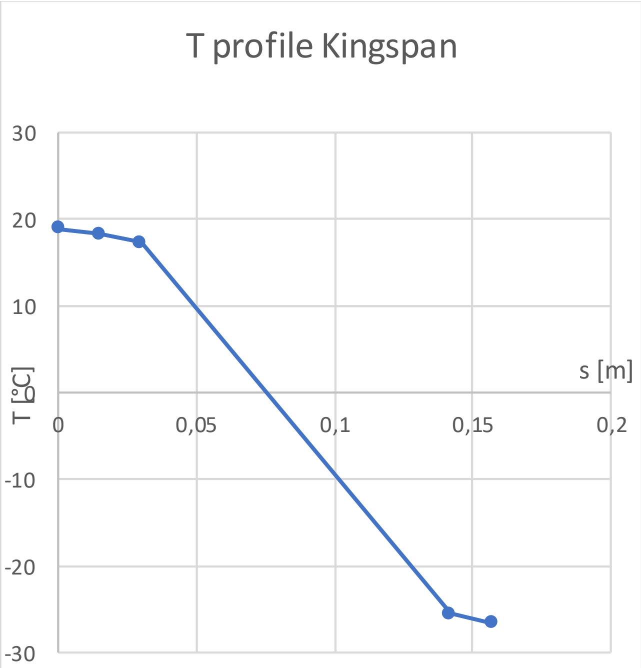
Table 4.18

Con riscaldamento a 20° e temperatura media invernale				
	PANELO (WALL)	PANELO (ROOF)	GLASS	Tot
T,e [°C]	-10			
T,i [°C]	20			
ΔT [°C]	30			
A [m^2]	47,606	37,08	18,95	103,636
U [W/(m^2*K)]	0,157	0,12	0,9	0,27962
Tp,e [°C]	-9,8116	-9,856	-8,92	-9,6645
Tp,i [°C]	19,41125	19,55	16,625	18,9514
φ [W/m^2]	4,71	3,6	27	8,38861
Q [W]	224,22426	133,488	511,65	869,362

Table 4.19

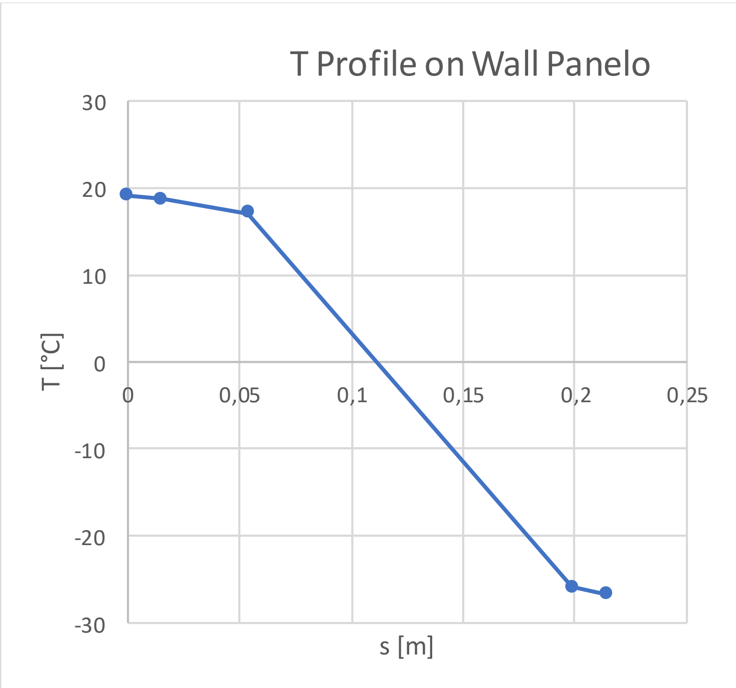
4.3 Surface condensation

Following the water vapour resistance values defined in the paragraphs 2.2.3 (Table 2.12) and 2.3.3 (Table 2.26 and 2.28) for the Kingspan system and Panelo system, a calculation of a hypothetical condensation risk was performed. This was done taking into consideration as temperatures at the different layers those shown in Figure 4.20, 4.21 and 4.22, computed thanks to the thermal transmittance values defined in paragraphs 2.2.2 (Tale 2.8) and 2.3.2 (Table 2.22 and 2.24), considering as outside temperature -27 °C and inside temperature 20 °C The different layers composing the panels are (from the inside to the outside): Plasterboard-15 mm, OSB-15 mm, PUR 112 mm, OSB-15mm for the Kingspan panel, Plasterboard-15 mm, LVL-39 mm, PUR 145 mm, OSB-15mm for the Panelo wall panel and Plasterboard-15 mm, LVL-27 mm, PUR 195 mm, OSB-15mm for the Panelo roof panel.



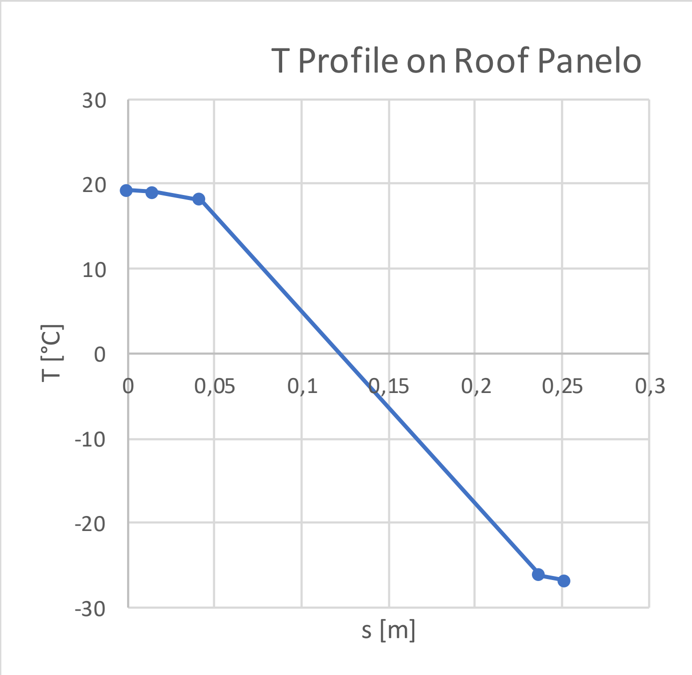
T profile		Stot [mm]
Tpi	18,85308472	0
T2	18,30256539	0,015
T3	17,24387436	0,03
T4	-25,57429608	0,142
Tpe	-26,63298711	0,157

Table 4.20 T-profile Kingspan



T profile (WALL)		Stot [mm]
Tpi	19,11139184	0
T2	18,68485993	0,015
T3	17,0540026	0,054
T4	-25,89539171	0,199
Tpe	-26,71564539	0,214

Table 4.21 T-profile Panelo wall



T profile (ROOF)		Stot [mm]
Tpi	19,31877804	0
T2	18,9917915	0,015
T3	18,12623888	0,042
T4	-26,1531887	0,237
Tpe	-26,78200897	0,252

Table 4.22 T- profile Panelo roof

All the panels have been lined internally with a plasterboard of thickness 15 mm. Furthermore, the computation was performed assuming a relative humidity in the inside environment of 70% and for two cases of relative humidity in the outside environment: on 60% and once 65%.

The Glaser diagram as well as the Tables with the vapour pressure values for both panels and for both relative humidity cases are shown below.

60% outside relative humidity case:

Layer	P_v [Pa]	$P_{,sat}$ [Pa]	Condensation
Int	1633,847159	2334,06737	NO
2	673,7566167	2100,118532	NO
3	613,7509578	1964,832979	NO
4	76,10025433	76,17743012	NO
Ext	40,096859	66,82809833	NO

Table 4.23 Vapour pressure values within the Kingspan panel and saturation pressure values compared

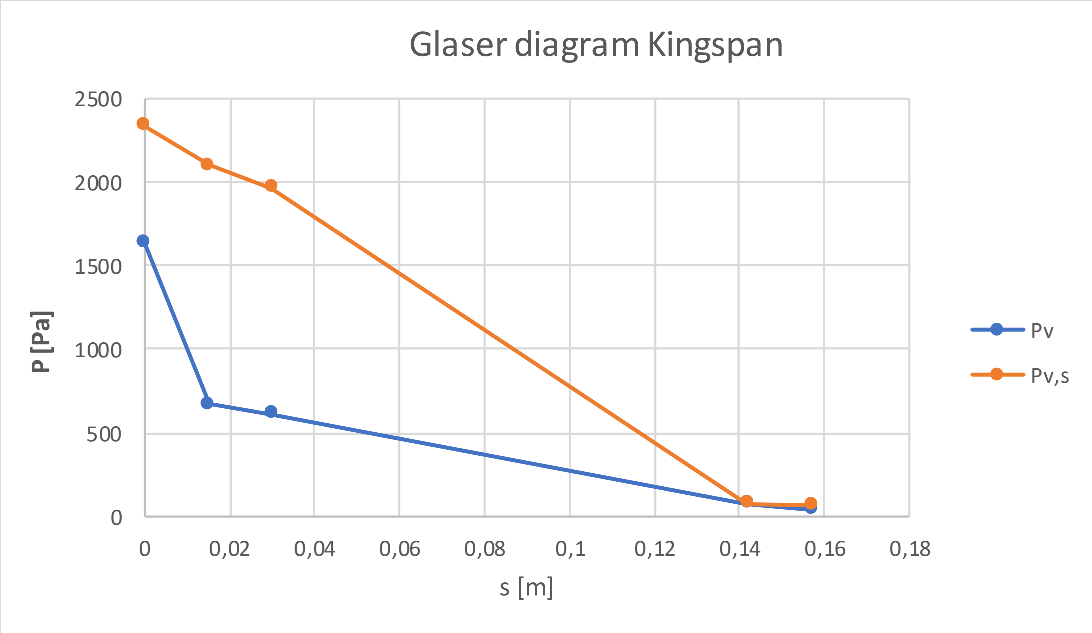


Figure 4.14 Glaser diagram within the Kingspan panel

WALL			
Layer	**Pv [Pa]**	**P,sat [Pa]**	**Condensation**
Int	1633,847159	2334,06737	NO
2	832,9676111	2150,93606	NO
3	650,767514	1941,394833	NO
4	70,12984204	73,97509545	NO
Ext	40,096859	66,82809833	NO

Table 4.24 Vapour pressure values within the Panelo wall panel and saturation pressure values compared

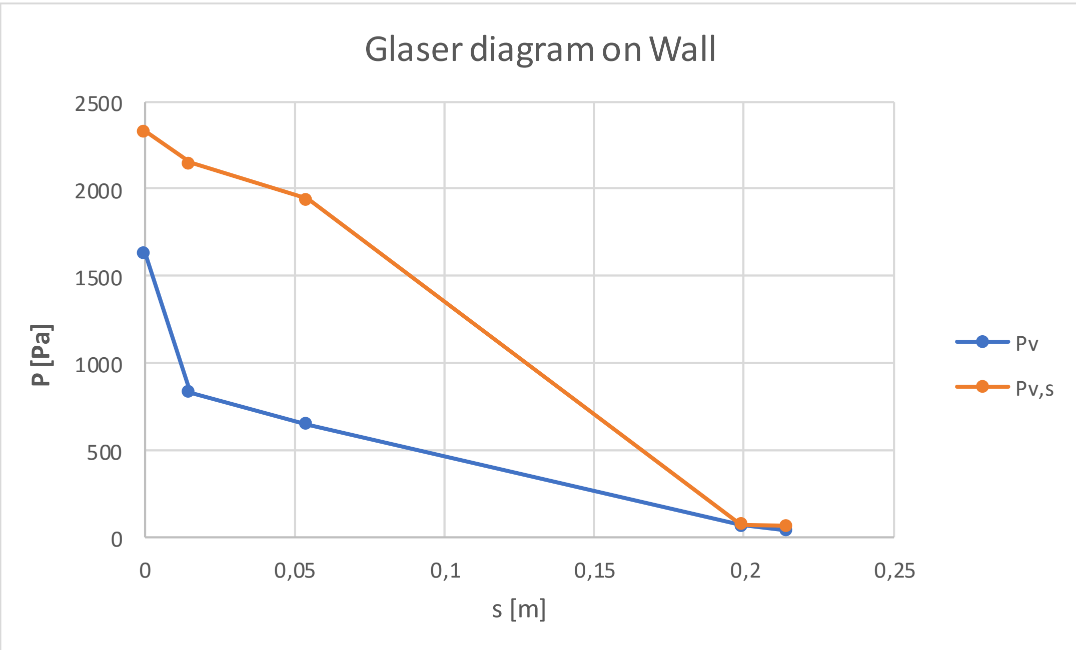


Figure 4.15 Glaser diagram within the Panelo wall panel

ROOF			
Layer	P_v [Pa]	$P_{,sat}$ [Pa]	Condensation
Int	1633,847159	2334,06737	NO
2	899,400016	2150,93606	NO
3	783,724591	1941,394833	NO
4	67,63862685	73,97509545	NO
Ext	40,096859	66,82809833	NO

Table 4.25 Vapour pressure values within the Panelo roof panel and saturation pressure values compared

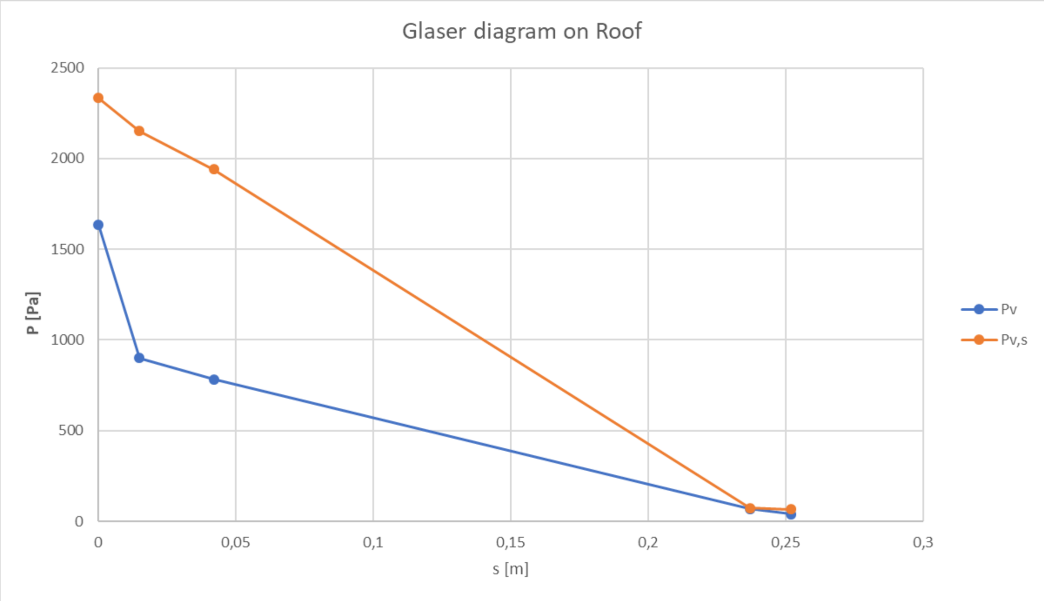


Figure 4.16 Glaser diagram within the Panelo roof panel

65% outside relative humidity case:

Layer	P_v [Pa]	$P_{,sat}$ [Pa]	Condensation
Int	1633,847159	2334,06737	NO
2	675,7695112	2100,118532	NO
3	615,8896583	1964,832979	NO
4	79,3661757	76,17743012	YES
Ext	43,43826392	66,82809833	NO

Table 4.26 Vapour pressure values within the Kingspan panel and saturation pressure values compared

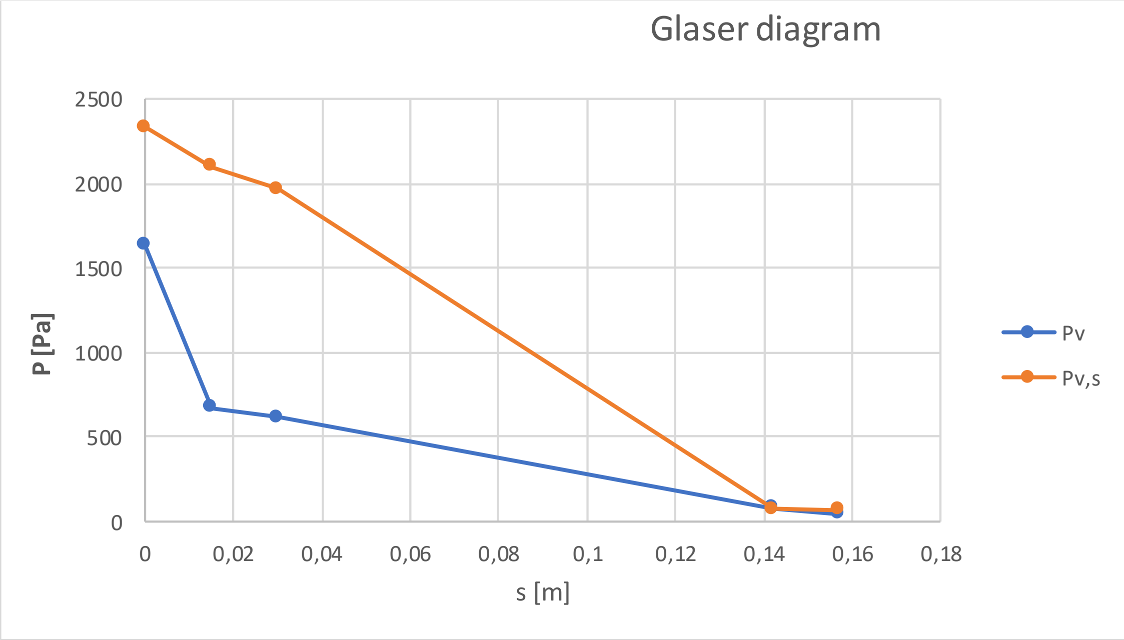


Figure 4.17 Glaser diagram within the Kingspan panel

WALL			
Layer	**P_v [Pa]**	**$P_{,sat}$ [Pa]**	**Condensation**
Int	1633,847159	2334,06737	NO
2	834,6467091	2150,93606	NO
3	652,8286068	1941,394833	NO
4	73,40828078	73,97509545	NO
Ext	43,43826392	66,82809833	NO

Table 4.27 Vapour pressure values within the Panelo wall panel and saturation pressure values compared

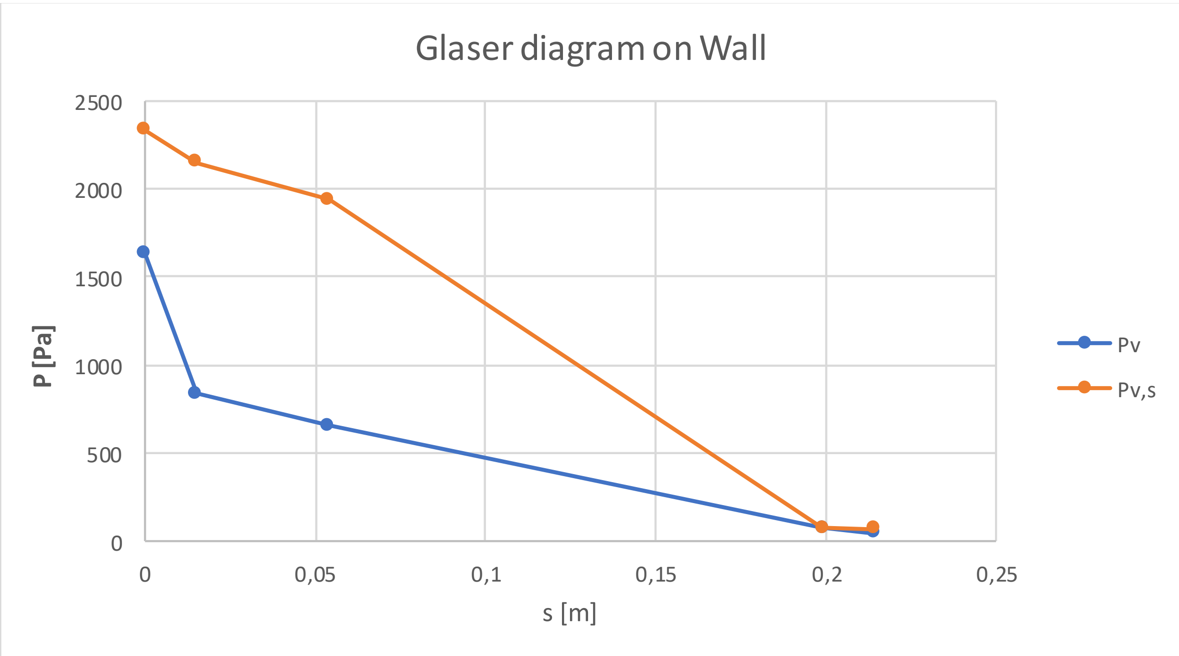


Figure 4.18 Glaser diagram within the Panelo wall panel

ROOF			
Layer	**P_v [Pa]**	**$P_{,sat}$ [Pa]**	**Condensation**
Int	1633,847159	2334,06737	NO
2	900,9398339	2150,93606	NO
3	785,5069303	1941,394833	NO
4	70,9222886	73,97509545	NO
Ext	43,43826392	66,82809833	NO

Table 4.28 Vapour pressure values within the Panelo roof panel and saturation pressure values compared

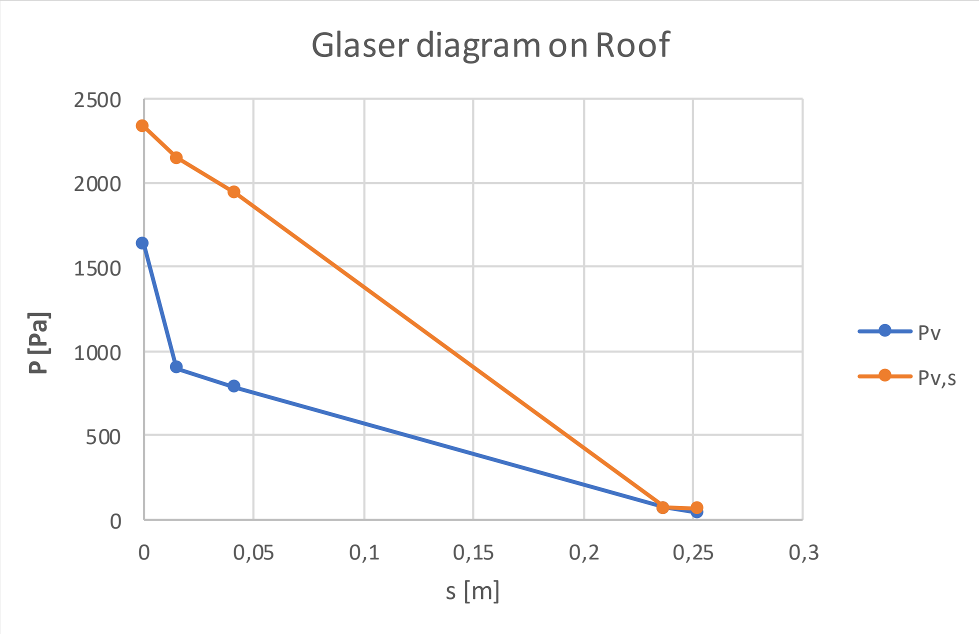


Figure 4.19 Glaser diagram within the Panelo roof panel

4.4 Costs

Following what said in paragraphs 2.2.3 and 2.3.3 about the cost of the panels, a computation of the total cost of only the panels composing the building studied has been computed. For this computation, a price of 55 EURO/square meter has been considered for the Kingspan panels and of 60 EURO/square mater for the wall panels as well as 90 EURO/square meter for the roof panels of the Panelo system. The total cost of the panels for the two cases is given in tables 4.29 and 4.30.

Kingspan Panels		
Price [EUR/m^2]	**A [m^2]**	**Tot Cost [EUR]**
55	91,85	5051,75

Table 4.29 Total cost of the Kingspan panels

Panelo Wall Panels		
Price [EUR/m^2]	A [m^2]	Tot Cost [EUR]
60	54,77	3286,2
Panelo Roof Panels		
Price [EUR/m^2]	A [m^2]	Tot Cost [EUR]
90	37,08	3337,2
TOT PANELS		
6623,4	EURO	

Table 4.30 Total cost of Panelo panels

In addition, an approximated calculation of a cost of only the supporting frame, without the connections, has been performed, considering for the frames the cross sections defined in paragraphs 3.2.2. The cost of the wood elements is defined to be 500 EURO/meter cube[19]. The resulting costs are given in Tables 4.31 and 4.32.

KINGSPAN		
Vtot [m^3]	Cost [EUR/m^3]	Tot cost [EUR]
4,1778	500	2088,9

Table 4.31 Total cost of Kingspan supporting frame

PANELO		
Vtot [m^3]	Cost [EUR/m^3]	Tot cost [EUR]
2,27448	500	1137,24

Table 4.32 Total cost of Panelo supporting frame

4.5 Final comparison

From what seen in the results, the Panelo panel provides as expected with much more stiffness to the structural system, resulting in having similar deformations to the Kingspan structural system with a much less rigid supporting frame. Nevertheless, this higher stiffness also means, as seen in paragraph 4.1.1, greater stresses inside the panel when subjected to the loads; since, apart from the axial resistance, all the other strength characteristics of this panel have yet to be defined, it has to be seen if the panel has enough resistance to resist these loads. Furthermore, paragraph 4.2 shows that the Panelo panel provides with significant energy savings related to the heating of the inside environment. However, since the studied structure is a bivouac which most commonly doesn't have any heating

[19]Avarage value, https://www.rifaidate.it/pareti-solai/travi/costo-travi-in-legno.asp#:~:text=Per%20quanto%20riguarda%20il%20costo,530%20euro%20per%20metro%20cubo.

system it is unsure whether these energy savings will also translate into financial savings or if they “only” result in greater comfort for those arriving in need of shelter; although, from a more general perspective this represents a clear advantage also economically for other types of building constructions. In paragraph 4.2, the results clearly show that for the Kingspan panel the condensation risk is higher, seeing that with slightly increased relative humidity there is interstitial condensation. Anyhow, for both panels the condensation risk is present, also due to the huge temperature difference between inside and outside. This is probably attributable to the outer OSB sheet, as can be seen in all Glaser diagrams shown, which really creates a blocking barrier, and may be solved with the use either of additional high water vapour resisting plasterboard, or with the use of breather membranes which allow the blocked water to quickly leave the panels. Finally, paragraph 4.4 exposes a difference in total price of the employed panels of more than 1500 Euros, which is reasonable considering all the advantages the Panelo panel provides with, and this difference is already partially balanced by the smaller total price of the supporting frame. However, to have real comparison of the costs of the building phase of the facility, one should take into account, apart from the strictly material related expenses, also the construction phase. In this sense, no research has been done to further deepen the issue, which leaves a lot of open possibilities for the outcome of total expenses. Considering that the Panelo system is bigger and heavier than the Kingspan one, it could very well be that more people are necessary for the assembly of the panels, or, if as seen in paragraph 1.2.1, if the assembly and the transportation is done by helicopter (which is for these types of structures often the case), it could significantly reduce the number of panels that the helicopter is able to lift and translate into higher transportation related expenses.

Conclusions and future works

This thesis was initially conceived with the purpose of creating a digital model of a building in the mountains with the Building Information Modelling. During the course of the work however, the main aims and leitmotivs changed and evolved while the thesis was taking form, and thanks to the collaboration with the Leap Factory company it became additionally addressed to a research about new and innovative construction systems as are the Kingspan TEK and the Panelo system.

For what concerns the conclusions drawn from the performed work and the possible future works on the topics covered by this thesis, there's some considerations to be made:

- The Panelo insulated panel could generally speaking be a very convenient construction system considering the performance and cost comparisons seen with a more "traditional" structural insulated panel such as the Kingspan one. However, the high number of assumptions made in this work to study its rendering is a clear evidence that the amount of information given by the producers is not yet enough to confirm such a statement. It would be interesting to perform sample laboratory tests on it to gain the missing data, and in that sense, it will just be about waiting what progresses the next months will bring, since they are in progress right now at the company. Anyhow, there are plenty of possible future applications regarding not only extreme environments buildings like the one seen in this thesis but e.g. also normal single-family houses, for which this system could represent a both in comfort and in finances appealing solution, and it is to follow how in the next years the product will do on the market.
- The application of both panels on the study case brought decent results in every aspect addressed by the work, and they are structurally wise surely an appropriate solution for the building. A further comparison with e.g. the wood framing or the traditional x-lam panels could also give an idea of their performance and costs compared to the most used wooden based construction systems in Italy.
- BIM allowed to combine the use of two software to create an accurate model in terms of quantity, size, shape and material. Still, the complicated interoperability process between the two employed programs that led to big data losses significantly slowed down the working progress and partially contributed to preventing the finalisation of adding of architectonical details such as inside furniture, pavement etc. There is therefore, regarding this aspect, big room for improvement.
- Modelling two dimensional surfaces in Finite Element Method based programs is certainly not an easy task if one doesn't have the appropriate knowledge to begin with; first of all, the

stress distribution resulting from load applications is less intuitive than in beam members, and secondly it presents very tricky obstacles like the singularities which, if one isn't prepared on how to handle them, can lead to the loss of a lot of time in the quest of trying to find "fair" solutions (where with fair what is meant is without the help of any tool given by the software whose function is specifically that of deleting singularities).

Finally, from a personal point of view, this thesis surely gave me the chance to put to test my skills learned in those years at the Politecnico di Torino university, and more importantly to learn so much that I could use for future applications, that it would be hard to put it all into a list; from notions, to the approach towards the problem, to the interaction with the different people that certainly helped me during this work. For example, to mention some of the things I surely approached in the wrong way initially, a very important aspect to follow when creating a digital model of a construction is to always start with the structural model to dimension the core of the structure before passing on to software like Revit that allow to increase the graphical information. Secondly, the employed software definitely does make the difference, both in the structural and in the graphical modelling; decisive were in this case the possibilities related to the shaping of geometry, the materials and the connection types. Thirdly, the connection type and its characteristics are when studying wood-based panels extremely important for the stiffness of the whole system and should be inserted in the structural model. Lastly, an additional lesson that this thesis gave me is surely to not try to be too perfectionist.

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The conclusion of this thesis means also the conclusion of a very difficult, challenging and beautiful part of my life which is that of the university studies, which so wouldn't have come to happen without the help of many people that I would like to thank.
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Thank you all,
Matthias Kather

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