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ECONOMIC EFFICIENCY OF FRAME MATERIALS IN A CASE-BUILDING

Comparing Costs of Glued-laminated Timber and Steel Frames

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Nita Viitasaari

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Työn nimi Economic Efficiency of Frame Materials in a Case-building: Comparing Costs of Glued-laminated Timber and Steel Frames
Ohjaaja Kalle Rohola

Opinnäytetyö pyrkii vertailemaan liimapuun ja teräksen hyötyjä ja puutteita runkomateriaaleina sekä vertailee eri ratkaisujen kustannuksia. Näiden seikkojen pohjalta on pyritty löytämään kustannustehokkaampi ratkaisu case-kohteelle. Tämän kaltainen analyysi on avuksi esisuunnitteluvaiheessa ja rakennusprojektin alkuun saattamisessa. Opinnäytetyö ottaa kantaa myös rakenteen muuntojoustavuuteen sekä tutkii rakenteen kantavuutta tilanteessa, jossa rakenteeseen lisättäisiin myöhemmin nostinpalkki.

Alkuun opinnäytetyö käsittelee liimapuun ja teräksen osalta hieman teoriaa historian ja materiaaliominaisuuksien osalta. Tämän jälkeen on esitelty case-projekti, jonka pohjalta lähdetään työstämään runkojen tieto- ja laskentamalleja. Laskentamallin tueksi rakenteiden mitoitusprosessissa on käytetty tarvittaessa myös käsin laskentaa. Laskentamallin pohjalta laaditaan karkea kustannusarvio runkomateriaalien menekkien mukaan. Kustannusarvio laaditaan Rakennustiedon Kustannuslaskenta-sivuston sekä liimapuutavaraa tuottavalta yritykseltä saadun arvion pohjalta. Opinnäytetyön päättää yhteenveto, jossa on käsitelty tuloksia ja muita huomioita. Laskentatulokset on kerätty opinnäytetyön loppuun liitetiedostoiksi.

Opinnäytetyötä aloitettaessa oletus oli, että liimapuurunkoinen tulisi todennäköisesti olemaan vertailussa hintavampi. Tämä oletus osoittautui vääräksi, joskin arvioiden vertailtavuuteen vaikutti tietyt muuttujat.

Avainsanat Liimapuu, puurakenteet, teräsrakenteet, kustannusarviot
Sivut 24 sivua, ja liitteitä 34 sivua

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Degree Programme in Construction Engineering | Abstract

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Subject Economic Efficiency of Frame Materials in a Case-building: Comparing Costs of Glued-laminated Timber and Steel Frames

Supervisor Kalle Rohola

This thesis aspires to compare glued-laminated timber and steel's advantages and disadvantages as frame materials. Different solutions costs are compared. Based on these factors, the thesis pursues to find out the most economical structural solution in a case-building. This sort of analysis will help with the pre-design phase of a construction project. The thesis also examines the space and time flexibility of the selected frame type, and whether it can carry the load coming from a crane beam if a crane would be added later.

In the beginning, theory related to glued-laminated timber and steel and their history and material properties are discussed. After this, a case-project is introduced which acts as a base for the building information and calculation models. To support the calculation model, some calculations by hand were also carried out. A rough cost estimation was made based on calculation models. The cost estimations were created using Rakennustieto Kustannuslaskenta platform and information received from a glued-laminated timber manufacturing company. The thesis is finished with a conclusive chapter which discusses the results and other observations. The calculation results are combined as appendices at the end of the thesis.

At the beginning of the writing process of the thesis, the preliminary assumption was that glued-laminated timber frame would be more expensive. It was discovered that this was not the case, although there were some variables affecting the compatibility of the results.

Keywords Glued-laminated timber, timber structures, steel structures, cost estimations

Pages 24 pages and appendices 34 pages

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1 Introduction

This thesis compares glued-laminated timber and steel costs as frame materials. It examines cost variation between glued-laminated timber and steel frames in a case-building. The aim is to design as similar structures to one another as possible, to get as comparable results as possible. The purpose of the thesis is not to provide undisputed answers on the matter, but to offer referential solutions regarding the case-building presented.

The thesis aims to find the most economical solution between the two structural options. The financial aspect is evaluated only, ignoring the environmental point of view, leaving that aspect for the reader to weigh in. For the sake of simplification, the topic of building services is also left out of scope, as well as geotechnical aspects. Those will not be reviewed any further than what is needed for the calculations.

The prices in RT-kustannuslaskenta are based on manufacturer's, importer's and hardware and wooden goods stores' retail prices. For that reason, it is good to keep in mind that the prices represented in the report, obtained from the cost estimation platform, are most likely higher than when ordering material and labor work based on tenders.

1.1 Terminology

Thesis includes some terminology related to the topic. To make it easier to follow the thesis, some of the terminology is gathered below and explained shortly. Abbreviations or symbols used in the calculations have been explained along the calculations.

Talo 2000 is a nomenclature used to classify information regarding a building project. It is used as a supportive tool for implementation and management. The nomenclature is divided into several parts describing building parts and technical parts, as well as tasks such as project, property or user related tasks (Rakennustieto, n.d.).

BIM-model stands for building information model. Building information model refers to a model containing multi-disciplinary data. It makes it possible to produce a digital representation of a building, which can be modified throughout the lifecycle of it (Autodesk, n.d.).

1.2 Tools and research methods

Research for this thesis is implemented by designing two different framing systems: glued-laminated timber frame and steel frame. Tekla Structures is used for creating BIM-models of both structures. Tekla Structures is used, because it is a familiar tool to use, and especially for steel structures, it is convenient as the material catalogue is quite comprehensive. Autodesk Robot is used to create calculation models of the structures. For calculating the loads and dimensioning the profile sizes by hand, PTC Mathcad Prime 9.0.0.0 is utilized.

After the models are finalized, and the material quantities are final, the costs for both framing systems are examined based on the calculations and calculation model's results. For cost estimation, RT-Kustannuslaskenta is used. The costs are evaluated according to Talo 2000. RT-Kustannuslaskenta is practical, since the prices are up-to-date, and reports can be created directly from the software. However, in addition, it is necessary to reach out to companies regarding some of the material, which cannot be obtained from RT-kustannuslaskenta.

2 Glued-laminated timber

Glued-laminated timber is an engineered wooden product produced by bonding longitudinally parallel wooden laminations with adhesives. The thickness of timber laminations does not exceed 45 millimeters (Puuinfo, 2020). Structural glued-laminated timber's properties are set according to standard SFS-EN 14080 (SFS Online).

Glued-laminated timber can be produced using homogeneous laminations. However, the external laminations often receive greater stresses, and for this reason, different strength class's timber is combined (Suomen Liimapuuyhdistys ry, n.d.). In Finland, structural glued-laminated timber products are produced using either pine or spruce but mostly pine (Puutuoteteollisuus, 2021). Later in the thesis, glued-laminated timber may be referred to as glulam.

2.1 History of glued-laminated timber

Glued-laminated timber was a revolutionary invention, what came to timber construction. The dimensional and shape related limitations of timber as a material could be overcome using glulam. Timber could finally compete with long span-structures with steel and reinforced concrete.

The history of glued laminated timber goes way back to approximately a decade ago. German Otto Hetzer was the first one to proof that beams and arcs could be manufactured by bonding timber to obtain large sections, so that timber could be used in structures that demanded long spans (Suomen Liimapuuyhdistys ry & Puuinfo Oy, 2014, s.8).

In 1906 Hetzer patented an invention that bonded timber boards together to construct curved structures. Glued-laminated timber products are still produced based on this method these days. The most significant realizations were that the shape of the structure was no longer dependent on the dimension of trees, and that the defects of timber were no longer such an issue on the structural capacity (Suomen Liimapuuyhdistys ry & Puuinfo Oy, 2014). Glulam offered economical advantage in comparison with steel or reinforced concrete. Timber could also withstand corrosion better than steel.

Manufacturing of glued-laminated timber in Finland started in 1945, as Laivateollisuus Oy manufactured ship frames using glulam. In construction industry glued-laminated timber has been in use since 1958 in Finland. Production of the material in Finland is around 300 000 cubic meters per year. Roughly 50 000 cubic meters of the amount is used in Finland and the rest is exported (Suomen Liimapuuyhdistys Ry & Puuinfo Oy, 2014, s.11).

2.2 Advantages and disadvantages of glued-laminated timber

Glued-laminated timber is a highly non-combustible material, and it does not bend under a high temperature. The charring depth after an hour under fire is around 36 millimeters and it happens at speed of about 0,6 millimeters per minute. The reinforcement inside the timber product is protected at the same time (Puuinfo, 2020). The production process of glued-laminated timber contains a treatment against, for example, harmful insects and mold. The material has also high resistance to defects due to adverse environmental circumstances. In addition to this, glued-laminated timber as a building material often appeals to the aesthetic eye. It enables soft and natural appearances for built structures.

Compared to the advantages of glued-laminated timber, the list of disadvantages is short. When reviewing the environmental aspects, considering the fabrication process as well as disposal, the adhesives used for combining the wooden layers or lamella are considered quite toxic, and are harmful for the environment. Glu-lam timber's production process involves many different stages which affect the cost of the material.

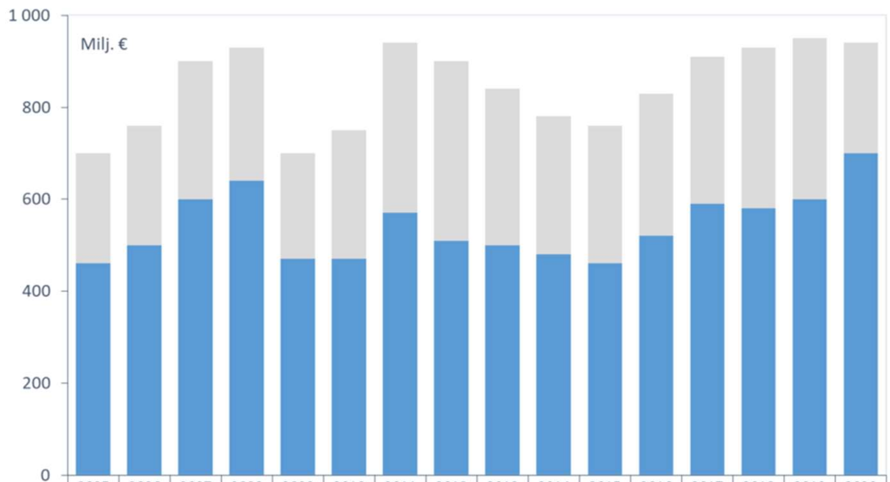
3 Steel

Weldable steel is an iron alloy with a low carbon content of 0.2-1%. Steel is produced removing carbon from molten metal. Compared to wrought iron, steel is stronger, achieving the yield strength of 235-700 MPa in both compression and tension. In addition, it is affordable and easy to process.

Manufacturing processes have developed over time. This has had a positive effect on the quality of iron and a decreasing effect on the costs. The development of welding and weldable steel production has had a significant impact in joining technology. As a result, larger structures have been constructed of steel with relatively low costs.

The production of structural steel in Finland has grown remarkably during the few decades. Export of structural steel has also been significant. A diagram in Figure 1, cited from Teräsrakenneyhdistys website shows, that the net worth of steel structure production between 2019-2020 was around 940-950 million euros (Teräsrakenneyhdistys, n.d.). This demonstrates how significant building material steel is alone in Finland.

Figure 1 Steel's production worth in Finland between 2019-2020



	2005	2006	2007	2008	2009	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019	2020
Vienti	240	260	300	290	230	280	370	390	340	300	300	310	320	350	350	240
Kotimaa	460	500	600	640	470	470	570	510	500	480	460	520	590	580	600	700

From all new buildings in Finland, about 20% are constructed from steel frames. Steel is mostly used in industrial buildings in which steel holds up to 55% of the market share. Steel is a popular structural material also in agricultural buildings, taking up to 50% of the market share (Teräsrakenneyhdistys, n.d.).

3.1 History of steel

The history of steel goes way back to centuries ago, when pig iron casting process was invented. Process of rolling of structural steel started in the Middle Europe at end of 18th century. Production started from flat steel, developing to L-profiles in 1830s and already in 1850's production of I-profile steel began. In 1856, an English inventor called Bessemer came up with a process, to produce steel with low enough carbon percentage at same cost as cast iron. This process was based on the idea of air being blown into molten cast iron, to burn the carbon. It was called the Bessemer's converter. This invention revolutionized structural steel production. Steel as it is in the present day, has been used in building construction since the early 1900's.

3.2 Advantages and disadvantages of steel

Steel is a strong material in both compression and tension. Steel is lightweight and it enables long spans constructed with minimal amount of material, which makes it an economical material. One of steel's advantages as a building material is its malleability. It is a ductile material, so it can be easily shaped into multiple different shapes and forms.

Steel as a building material is highly versatile. It is used in various applications and it offers flexibility as the already manufactured components are easy to install on site. This saves time and expense. Steel connections are easy to implement. Steel is also considered quite flexible in terms of its use, as it can be still used even if intended use of a building or space is changed.

When thinking about disadvantages of steel, arises its susceptibility to corrosion and thermal deformation, and therefore, it takes some extra consideration to protect the material against these environmental circumstances. This resonates to the maintenance costs of steel being quite high. The production process of steel also burdens the environment. Even though beauty is in the eye of the beholder, steel structures can also often be seen as quite bleak.

4 Case: An industrial hall

An industrial hall, that is thought to be built in an industrial park in Lempäälä, located in Pirkanmaa region, is used as a case example for this thesis. The case-building is designed only for this thesis. The building has a total area of 450m^2, divided between a 375m^2 storage and 75 m^2 office and personnel space. The case building is designed using both glued-laminated timber frame and steel frame. For purpose of simplification, the ground is assumed to be load-bearing. Neither the geotechnical aspects and groundwork, nor the foundation are to be taken into further consideration in this thesis though.

The industrial hall is designed to include spaces such as personnel and office space, and sufficient storage space. In the hall, cranes, forklifts and other machinery might be moving around, causing vibration that must be considered in the design if needed. The building is designed to belong to consequence classCC2 end execution class EXC2.

4.1 Loads

When designing a building, excessive loading is to be avoided. Loads can be of permanent, quasi-permanent or transient type. They can also be classified as dynamic or static. Self-weight of the structures is to be considered in the calculations. Dead load can be seen as a permanent load, which is affecting the load-bearing structures. Roof elements or loads coming from HVAC systems, for example can be classified as dead load. There can also be live load loading a building. Since the case-building is a one-storey building, service load will not be needed to consider.

4.1.1 Self-weight and suspension load

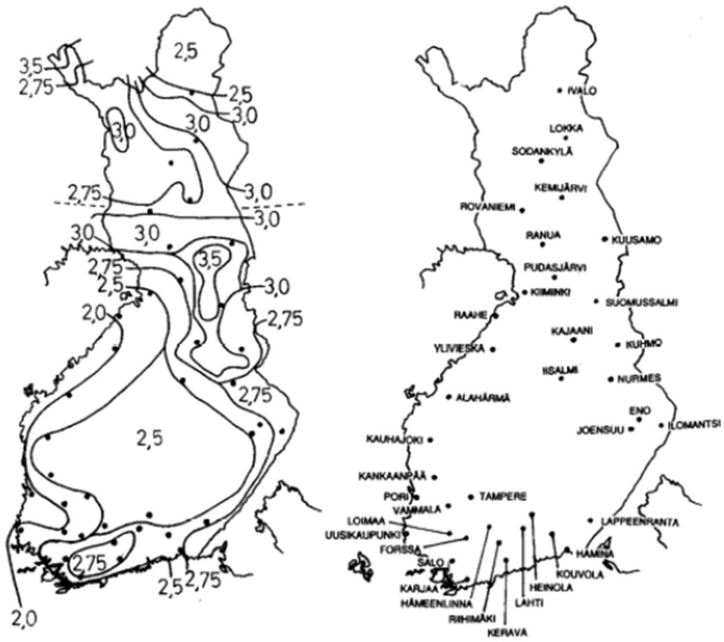
The self-weight of the roof structure contains the self-weight of each member of the roof, and according to Eurocode, it is classified as permanent, fixed load. This load is transmitted through the beams and columns to the ground. The information of the self-weight of the structure can easily be obtained from the BIM-model created. In addition, an estimation of the self-weight of the roofing elements was needed for calculations. This was evaluated to be 0,5 kN/m^2.

A suspension load from the HVAC system and piping is an additive load to consider. The load can be assumed to be 0,2 kN/m^2. This load also classifies as permanent and fixed.

4.1.2 Snow load

Snow load is a vertical, variable load to be considered in Finland. Snow load is calculated according to the standard SFS-EN 1991-1-3. The value of the designed snow load depends on the roof shape and the building surroundings. Finland is divided into multiple sections based on the characteristic snow load on the ground. The division is demonstrated below in **Error! Reference source not found.**. With the characteristic snow load on the ground, and the shape coefficient of the roof, a characteristic value for snow load on the roof can be obtained.

Figure 2. The characteristic values of snow load on the ground, s_k (Puuinfo, 2020)



The characteristic value of snow load on the roof, is taken to be 2,5 kN/m^2. In case of roof abutting and close to taller construction works, value for drifting snow needs to be taken to account. This is done by finding out a shape coefficient for the drifted snow, which is needed for calculating the characteristic value of drifted snow. In that case, a drifting length also needs to be defined.

4.1.3 Wind load

Calculation of wind load has been done following Puurakenteiden lyhennetty suunnitteluohje by Puuinfo (Puuinfo, 2020). It follows the standard SFS-EN 1991-1-4, Finnish National Annex and guides RIL 205-1-2017 and RIL 205-2-2019. The procedure can be used for calculating wind load in buildings located in Finland, which are not classified as demanding. For wind load calculations, the two parts of the building would be considered separately.

Wind is calculated perpendicular to the longer side of the storage hall. Velocity pressure is calculated based on the terrain category and orography of the area. After this, a structural factor is defined. To find out the force coefficient, effective slenderness ratio and dimensional relation between the sides of the building need to be obtained. With this information, the total characteristic wind load may be found. Same procedure is followed to get the wind load in other direction too.

4.1.4 Crane-induced loads

In Eurocode standard SFS-EN 1991-3 particularly for cranes and machinery, it is stated that "actions induced by cranes shall be classified as variable and accidental actions…" (SFS-EN 1991-3/2007, s. 23). However, the weight of the crane is considered as permanent action.

Crane-induced loads for the case-building are also considered. The loads have been calculated for a hanging crane. The crane weight was taken as 1 kN point load. In addition, a brake load was considered as 1 kN horizontal load.

4.2 Framing system

When selecting the framing system, it needs to be considered what are the special features that are needed in terms of the use of the building. For storage space, a broad, free space with no unnecessary limitations is needed. Free height needs to be also sufficient, to ensure enough space for lifting cranes and possibly other machinery. In the case-building, the free height of five meters was estimated to be sufficient. All in all, the building height would be approximately seven meters.

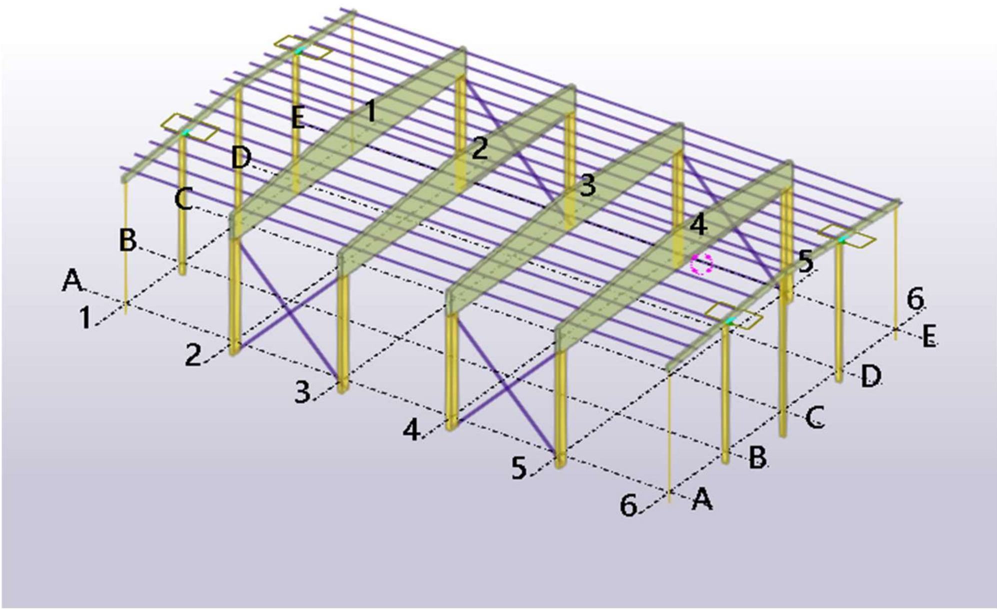
4.2.1 Glued-laminated timber frame

The frame type of the case-building will be a pillar-beam frame. This type of frame creates a mast stiffening transversely for the building. Longitudinally, the building would be stiffened using either timber or steel diagonal bracings. However, the diagonals are not to be dimensioned, and therefore not considered in the estimation. In addition, the wind pillars are dimensioned as mast pillars, that also act as a stiffening system.

Regarding roof structure, two options were contemplated. A mono-pitched roof was initially considered, but for more of an aesthetically appealing design, a double-pitched roof structure was selected. This was originally supposed to be implemented using pitched cambered glued-laminated beams. However, due to the price of these being remarkably higher, ridge beams were selected.

Ridge beams on the larger building part are supported on mast pillars. At the ends of the storage hall, there will be glued-laminated sections supported on corner columns and wind columns. Columns are attached to the foundation with a fixed connection using column shoes. Loads from the roofing are transferred to the beams via purlins. As said, building is stiffened in the longitudinal direction using diagonal bracing systems, which is not calculated in the thesis. In Figure 3, the frame type is shown from a BIM-model.

Figure 3. BIM-model of the glued-laminated frame



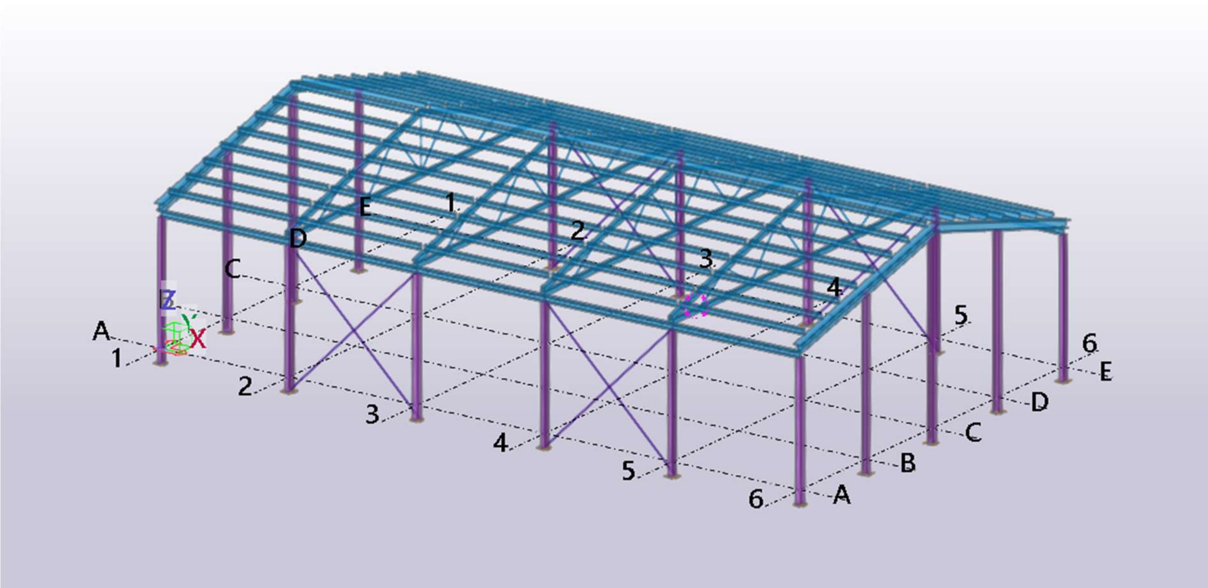
Wind columns are designed as GL30c 115x270 mm sections. Calculation of these may be found in appendix 6. Corner columns, calculated in appendix 8, are designed using GL30c 140x315 mm profile sizes.

4.2.2 Steel frame

Double pitched roof structure is designed using double howe steel trusses. In the case of steel framing, beams and columns are designed using open profiles of structural steel S355. Trusses are designed using cold-formed hollow-core sections. I-profile purlins will transfer the loads from roofing to the trusses. The roof structure will lay on top of single-span girder beams, that transfer the weight load to the columns. Mast pillars are attached to the foundation with fixed connections.

The stiffening system can be compared to what is designed in the glued-laminated timber frame option. The larger frame part is stiffened using bracing systems longitudinally in both ends of the structure. Demonstration of the frame is shown in Figure 4.

Figure 4. A BIM-model of the steel frame



4.2.3 Space and time flexibility

It is convenient, if not necessary, to keep in mind already in the design phase the future use of a building. Building’s use might change from the preliminary designed use and even if not, businesses usually aim to growth which is why the frame type is better to design so, that expansion is possible and easy to implement if needed. The pillar-beam frame is flexible, and it is easy to expand in the future if more space is needed.

Regarding the space and time flexibility, in the page 15, the load-bearing capacity of the structures is also reviewed briefly in case of a later installation of a crane, which would be executed as monorail hoist block underslung hanging from a runway beam.

4.3 Calculation and model

Project was started by creating preliminary BIM-models of the structures using Tekla Structures 2023.0. At this point, the profile sizes were more of an approximate guess just to outline the model. After modeling the building, dimensioning of the profiles was started. After obtaining the calculation results, the profiles could be edited to the existing BIM-model of the structure quite effortlessly.

The calculations did not consider the joints. It was researched, whether information related to the prices of the joints would have been available. However, there was not to be found any relevant information regarding the kilo prices or approximate share of joints in the costs. Therefore, this had to be left out, and only notified that this needs to be considered when reviewing the results.

4.3.1 Double tapered glued-laminated beams and mast pillars

Dimensioning of ridge beam was started using Puuinfo's Excel created for this purpose. The preliminary calculation was done based on the results obtained from this Excel shown in appendix 1. (Puuinfo, 2020). After this, a calculation model was created of the mast frame, including the tapered beams and mast pillars.

The ridge beam was estimated to be GL30c 190 millimeters thick, 1100 millimeters high at the supports, and 1700 millimeters high at the ridge. When carrying out the hand calculation, it appeared, that the bearing pressure capacity with the mast pillars of 190x450 would not be enough. However, it was concluded, that it would not be reasonable to increase the column size any more than it was, so the ridge beams would need to be designed with steel plates, to bear the pressure, on top of the supporting columns. This is not in scope of the thesis, though.

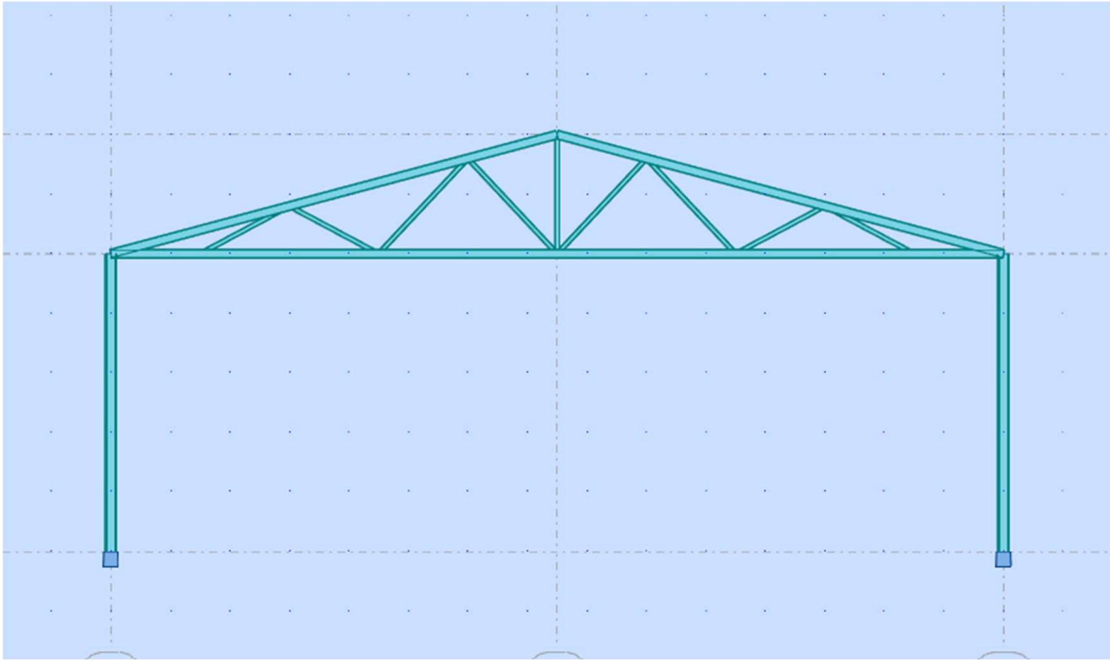
According to Robot, the sections input to the program, were sufficient. Mast pillars were to be designed as GL30c 190x450 mm sections and the double tapered ridge beam as GL30c 190 mm thick, 1100-1700-1100 beam, noting the need for dimensioning the bearing plates. Calculations regarding these can be found in appendices 2 and 3.

4.3.2 Steel trusses and mast pillars

Mast frame, that included truss and supporting columns, was modelled to Robot as shown in Figure 5. Column profiles were approximated to be HEA 200 and truss's top and bottom chords was approximated as RHS 120x120x5. After calculation, it was discovered, that the profiles of the columns and the chords of the trusses would not withstand the loads. Column profiles were changed to HEB 200 and chord profiles to RHS 150x150x5. Profile sizes of truss bars were adjusted so, that they would be RHS 80x80x5.

Mast pillars were calculated according to Eurocode 1993. Dimensioning calculations of the steel profiles differ a little from calculating glued-laminated timber profiles. For one thing, a cross-sectional class needs to be taken into consideration. The purpose of cross-section classification is to recognize, to which extent a local buckling limits the durability and torsional ability of cross-sections (SFS-EN 1993-1-1 s. 42). Cross-sectional classes for open profiles used in the hand calculations of this thesis were not calculated but obtained from a table of EurocodeApplied.com -website. Calculations regarding the mast pillars can be found in the appendix 4.

Figure 5 A model of the steel mast frame for structural analysis



4.3.3 Wind and corner columns

Glued-laminated columns wind and corner columns were dimensioned according to Puuinfo's publication "EC5 Sovelluslaskelmat – Hallirakennus" (Puuinfo, 2020) and RIL 205-1-2007. At first, wind column profiles were approximated as 90x190 mm sections. Calculations showed, that with this cross-section the bending strength would not be sufficient, so the cross-section was increased to 225 mm. Bending strength's utilization ratio also appeared to exceed the limit value, so cross-section was increased in total to 115x270 mm, and therefore, it fulfilled the requirements, and it was ensured, that was sufficient to act as a mast pillar also.

Corner columns were sketched as 115x115 mm profiles. This was checked to be a stock size profile also, which would probably be economical from that aspect also, even though in this case it was not as relevant, since the estimation had already been obtained from the manufacturer. However, the bending strength would have not been enough, so the profile size was calculated to be 140x315 mm. Calculations regarding the glulam wind and corner columns may be found in appendices 5 and 6.

For wind and corner columns of the steel frame, the same calculation procedure was followed, as with steel mast pillars. Based on the calculation results, HEA 200 and HEA 220 profiles were selected for these.

4.4 Cost estimations

According to the results obtained by manual calculation, a calculation model was created using Autodesk Robot Structural Analysis Professional 2022. The software was used for checking the durability of the structures. Based on the results of the dimensioning, the information regarding the materials and quantities was gathered to RT Kustannuslaskenta for comparison. Report regarding the total costs of the frame options is attached as an appendix 8. Cost estimation regarding the glued-laminated timber frame is based on the rough estimation received from a company manufacturing glued-laminated products, though.

4.4.1 Costs of the glued-laminated timber frame

For price information regarding the roof beams, a head of sales division in a Finnish timber product manufacturing company, was contacted. For comparison, price information for pitched cambered beams and ridge beams were inquired. The two options presented substantial differences. The received rough estimation included the columns, beams, purlins, column shoes and wooden forked boards installed on factory. To area of Southern Finland, these prices would include a freight also. Prices do not include value added taxes.

Estimation for pitched cambered beams including all the parts mentioned above was 67 000 euros. However, for ridge beams the estimation was 47 000 euros, the difference being around 20 000 euros. Since the price difference was remarkable, regular ridge beams were selected for the design and cost comparison. This price information obtained from the company, was taken to the cost estimation as it was, since this price included the total share of glued-laminated timber in this-sized building, and it would have been difficult to start finding out the precise share of each type of structures without inviting tenders. Furthermore, RT-Kustannuslaskenta is still quite stiff platform, since there are very few options of profile sizes for glued-laminated timber.

4.4.2 Costs of the steel frame

Regarding the steel truss cost estimation, two companies manufacturing steel trusses, were reached out for more specific estimation of the costs. Unfortunately, there were no answers received, so the cost estimation of the trusses is based on only RT-Kustannuslaskenta prices. Information regarding linear metres of the profiles used, was obtained from the BIM-model and input to RT-Kustannuslaskenta. If the profiles were not to be found in RT-Kustannuslaskenta, the estimation was done so, that the relation between two profile sizes was calculated based on prices of online stores and estimated from the RT-Kustannuslaskenta prices according to this relation.

5 Analysis: Load-bearing capacity for a crane

As mentioned earlier, it was also examined whether the structures could withstand a crane for a later installation, keeping in mind the space and time flexibility. The structures are designed and calculated so, that they can withstand the crane loads used in this example. Crane is considered as a hanging crane, installed to the roof spanning member. Crane loads are taken to be as follows: a self-weight of the crane as 0,7 kN/m, a point load of 1 kN, and the brake load of 1 kN also.

Other options were examined also. Other crane types, that were thought of, were cranes installed either on top of or below runway beams. However, this type of installation would need some changes or additions to the structure and would increase the costs for sure.

6 Results and conclusions

From the cost estimation in the appendix 8, it may be noticed that the steel frame’s cost estimation is slightly higher than in case of glued-laminated timber frame. For more thorough cost estimation, a more specific examination, that considers all the building parts such as foundation, insulation, façade material, and ground-work and other relevant factors, would be needed. As mentioned earlier, the results in this thesis are not to be used as anything more than an approximate estimation of the costs of the two different structural materials in this case-project. The costs are always to be evaluated separately for each project, and the latest prices available be applied, to get the most reliable results.

Inviting tenders, which was not done in this case, has also a major effect on the price range. The joints are also a relevant part of the costs and therefore, it is to be noted, that the share of the joints of the steel frame, is not calculated. There was not found any relevant information of the kilo prices, or the approximate share of joints of the costs, so this had to be left out. Furthermore, it is to be considered, that in case of the glued-laminated frame's cost estimation, the joints of the ridge beams to the columns, and the columns to the foundation, were calculated.

In addition, costs of the assembling at the mechanical workshop have not been calculated to the costs. Transport also brings additive costs. The fact, that in this thesis optimization was not done, but the selected profiles are on the conservative side, also has a major impact. If the structures would have been optimized to the limits, the price range would have been certainly different. One thing to consider is, that in the prices obtained from RT-Kustannuslaskenta, the labor costs are already included, and the labor costs from installing the glued-laminated structures is not considered in the estimation.

6.1 Further research

Thesis could be reviewed further in many ways; for example, by taking the environmental aspect into account. That would be very meaningful and interesting topic to pursue. In this thesis, it was left out of scope only to draw a line somewhere. Otherwise, the topic would have expanded too much, and the information flood would have been difficult to control and structure. One way of expanding the topic could also be reviewing the joints more deeply. Joints might have a significant impact on the costs. The calculations regarding the office part of the building were left out, so that could be one thing to review.

6.2 Personal insights

The thesis process was very educational, and it helped with understanding the designing process as whole. The preliminary assumption was that glued-laminated frame would be economically more expensive. The most surprising thing was, as mentioned earlier, the price difference between a pitched cambered beam and a ridge beam. This helped in realizing how much design choices truly affect the costs of a building project, and how important it is to have some idea of the price effect of certain choices already in the beginning.

When looking back the total thesis process, there has been a lot to learn from. If I could start the thesis process again, I would think more about the structuring of the thesis already in the beginning. I would also try to be more organized with the procedure order. In conclusion, this process has taught a lot of a designing and writing process, and about what things to do, and especially what not to do.

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Appendix 1. Preliminary dimensioning of the double tapered glulam beam

Versio 1.1

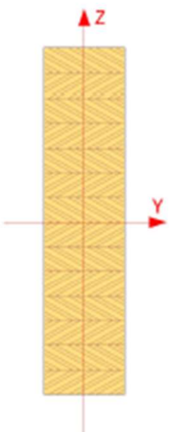
Suunnittelutoimisto	Työn nro		Sivu
	Päiväys	Tekijä	1 / 1
		N.V	
Rakennuskohde	Sisältö		
Opinnäytetyö	Liimapuuharjapalkin mitoitusohjelma (EC 5)		

1.0 RAKENTEEN TIEDOT

Palkkijako	k =	5000 mm
Palkin jänneväli (15…32 m)	L =	15000 mm
Palkin korkeus tuella	H_1 =	1300 mm
Palkin korkeus harjalla (max 2000 mm)	H_2 =	1850 mm
Yläreunan ja syysuunnan välinen kulma	α =	4,19 °

Sekundäärin jatkuvuus	1-aukkoinen (kerroin 1,00)
Palkin leveys	b = 190 mm (vakiotuote)
Tuettu Y-suunnassa	5 kpl poikittaistuettuja kenttiä
Taipuma (w_{fin} : $w_{net,fin}$)	L/200 : L/300 : esikorotus L/400
Lujuusluokka	GL30c
Käyttöluokka	KL 1
Aikaluokka	Keskipitkä
Kuormitus tulee palkille	Puristetulta reunalta
Kuormituksen tyyppi	Symmetrinen
Pintakäsittely	Ei estä kosteuden siirtymistä

Poikkileikkaus



Pysyvä kuorma	g_k =	5,7 kN/m2
Muuttuva kuorma (vasen lape)	$q_{k,1}$ =	2,0 kN/m2
Muuttuva kuorma (oikea lape)	$q_{k,2}$ =	2,0 kN/m2
Muuttuvan kuorman pitkäaikaisosuus	Ψ_2 =	0,2
Palkin omapaino	$g_{k,palkki}$ =	1,50 kN/m

2.0 MITOITUSTULOKSET

Info

Taivutuskestävyys	Taivutuskestävyys	Kiepahduskestävyys	Leikkauskestävyys
$M_{d,mit}$	$M_{d,max}$	$M_{d,mit}$	$V_{d,mit}$
OK	OK	OK	OK
80 %	76 %	99 %	78 %

Poikittainen vetokestävyys	Taipuma		Y-suunnan stabiloiva tuki			Tukipituus
$M_{d,harja} + V_{d,harja}$	w_{fin}	$w_{net,fin}$	F_d [kN]	C [N/mm]	a [mm]	ℓ_{min} [mm]
OK	OK	OK	2,0 kN	618,0	3000	784
97 %	68 %	27 %	Tuen voiman määrää 1. muoto			

Appendix 2. Double tapered glued-laminated beam: calculations

Ridge beams, glued-laminated timber, GL30c

Material properties and beam dimensions

$f_{m.k} := 30\ MPa$	Bending strength
$f_{v.k} := 3.5\ MPa$	Shear strength
$f_{c.0.k} := 24.5$	Compression strength parallel to grains
$f_{c.90.k} := 2.5\ MPa$	Compression strength perpendicular to grains
$f_{t.90.k} := 0.5\ MPa$	Tension perpendicular to grains
$k_{def} := 0.6$	Factor considering moisture effect to deflection
$E_{mean} := 13000\ MPa$	Modulus of elasticity
$E_{0.05} := 10800\ MPa$	Modulus of elasticity parallel to grain
$G_{mean} := 650\ MPa$	Shear modulus
$L := 15000\ mm = 15\ m$	Span length of the beam
$k_b := 5\ m$	Center to center span
$h := 1100\ mm = 1.1\ m$	Height of the beam ends
$h_r := 1700\ mm = 1.7\ m$	Height at the ridge
$b := 190\ mm = 0.19\ m$	Width of the beam
$l := 450\ mm = 0.45\ m$	Length of the support
$\alpha := 4.57\ °$	Roof slope

Surface of the beam is treated with a weather proof coating.

$\gamma_M := 1.2$ — Partial safety factor of the material

$k_{mod} := 0.8$ — RIL 205-1-2007 Table 3.1

Storage building dimensions:

$h_1 := 7\ m$ $\quad l_c := 5\ m$ $\quad k := 5\ m$

$d_1 := 25\ m$ $\quad b_1 := 15\ m$

$K_{FI} := 1$

Vertical loads:

Dead load and self-weight

$g_1 := 1.4\ \dfrac{kN}{m}$ — Self-weight of the beam

$g_2 := 0.5\ \dfrac{kN}{m^2}$ — Dead load of roof elements

$g_3 := 0.2\ \dfrac{kN}{m^2}$ — Suspension load of HVAC systems

$g_4 := 0.7\ \dfrac{kN}{m}$ — Dead load of the crane

$P_{G.SLS} := g_1 + g_4 + k \cdot (g_2 + g_3) = 5.6\ \dfrac{kN}{m}$ — Self-weight of water roof in SLS

Crane loads:

$P_C := 1\ kN$ — Point load from a crane

$P_{C.ULS} := 1.5 \cdot K_{FI} \cdot P_C = 1.5\ kN$ — Crane load in ULS

Snow load:

$$s_k := 2.5\ \frac{kN}{m^2}$$ Characteristic snow load on the ground

$$\mu_1 := 0.8$$ Shape coefficient of the roof

$$q_k := \mu_1 \cdot s_k = 2\ \frac{kN}{m^2}$$ Characteristic value of snow load on the roof

$$P_{Q.SLS} := q_k \cdot k = 10\ \frac{kN}{m}$$ Snow load in SLS

$$P_{Q.ULS} := 1.5 \cdot K_{FI} \cdot q_k = 3\ \frac{kN}{m^2}$$ Snow load in ULS

$$P_{Ed} := 1.15 \cdot K_{FI} \cdot P_{G.SLS} + 1.5 \cdot K_{FI} \cdot P_{Q.SLS} = 21.4\ \frac{kN}{m}$$ Combined self-weight and snow load in ULS

Bending strength in the critical cross-section

$$x := \frac{h}{h_r} \cdot \frac{L}{2} - \frac{l}{2} = 4.628\ m$$ Critical cross-section

$$hx := \frac{x}{16} + h = 1.389\ m$$ Height of the beam at the critical cross-section

$$M_d := \frac{P_{Ed} \cdot L^2}{8} = 603\ kN \cdot m$$ Design bending moment

$$R := \frac{M_d}{\frac{L}{2}} \cdot 2 = 160.8\ kN$$ Reaction force

$$M_{d.cr} := \frac{P_{Ed} \cdot L \cdot x}{2} \cdot \left(1 - \frac{x}{L}\right) = 514.574\ kN \cdot m$$ Bending moment in the critical cross-section

$$\sigma_{m.\alpha.d} := \frac{6 \cdot M_{d.cr}}{b \cdot h^2} = 13.43 \; MPa$$

Bending stress at the critical cross-section

$$f_{m.d} := \frac{f_{m.k} \cdot k_{mod}}{\gamma_M} = 20 \; MPa$$

Bending strength

$$f_{v.d} := \frac{f_{v.k} \cdot k_{mod}}{\gamma_M} = 2.333 \; MPa$$

Shear strength

$$f_{c.90.d} := \frac{f_{c.90.k} \cdot k_{mod}}{\gamma_M} = 1.667 \; MPa$$

Compression strength

$$k_{m.\alpha} := \frac{1}{\sqrt{1 + \left(\frac{f_{m.d}}{1.5 \cdot f_{v.d}} \cdot \tan(\alpha)\right)^2 + \left(\frac{f_{m.d}}{f_{c.90.d}} \cdot \tan(\alpha)^2\right)^2}} = 0.907$$

$k_{m.\alpha}$ factor

$$\sigma_{m.\alpha.d} \le k_{m.\alpha} \cdot f_{m.d}$$

Dimensioning condition for bending strength at the critical cross-section

$$\frac{\sigma_{m.\alpha.d}}{k_{m.\alpha} \cdot f_{m.d}} = 0.74$$

Utilisation ratio OK

<u>Bending strength at the ridge</u>

$$k_l := 1 + 1.4 \cdot \tan(\alpha) + 5.4 \cdot \tan(\alpha)^2 = 1.146$$

$$\sigma_{m.d} := \frac{k_l \cdot 6 \cdot M_d}{b \cdot {h_r}^2} = 7.554 \; MPa$$

Bending stress at the ridge

$$k_r := 1$$

For a ridge beam
RIL 205-1-2007

$$\sigma_{m.d} \le k_r \cdot f_{m.d}$$

Dimensioning condition for bending stress at the ridge

$$\frac{\sigma_{m.d}}{k_r \cdot f_{m.d}} = 0.378$$

Utilisation ratio OK

Transversal tensile strength at the ridge

$k_p := 0.2 \cdot \tan(\alpha) = 0.016$ — RIL 205-1-2007

$\sigma_{t.90.d} := \frac{k_p \cdot 6 \cdot M_d}{b \cdot h_r^2} - 0.6 \cdot \frac{P_{Ed}}{b} = 0.038\ MPa$ — Transversal tensile stress at the ridge

$f_{t.90.d} := \frac{f_{t.90.k} \cdot k_{mod}}{\gamma_M} = 0.333\ MPa$

$k_{dis} := 1.4$ — Factor for a ridge beam

$V_0 := 0.01\ m^3$ — Comparative volume

$V := h_r \cdot (2 \cdot (0.5 \cdot h_r)) \cdot b = 0.549\ m^3$ — Volume of the ridge area (1850mm x 1850 mm)

$k_{vol} := \left(\frac{V_0}{V}\right)^{0.2} = 0.449$

$\sigma_{t.90.d} \le k_{dis} \cdot k_{vol} \cdot f_{t.90.d}$ — Dimensioning condition for tensile stress at the ridge

$\frac{\sigma_{t.90.d}}{k_{dis} \cdot k_{vol} \cdot f_{t.90.d}} = 0.18$ — Utilisation ratio OK

Combined tensile and shear strength at the ridge

$V_d := 28.1\ kN$

$\tau_d := \frac{3}{2} \cdot \frac{V_d}{b \cdot h_r} = 0.13\ MPa$ — Design shear stress at the ridge

$\sigma_{t.90.d} := \frac{k_p \cdot 6 \cdot M_d}{b \cdot h_r^2} - 0.6 \cdot \frac{P_{Ed}}{b} = 0.038\ MPa$ — Tranversal tensile strength at the ridge

$\frac{\tau_d}{f_{v.d}} + \frac{\sigma_{t.90.d}}{k_{dis} \cdot k_{vol} \cdot f_{t.90.d}} \le 1$ — Dimensioning condition

$\frac{\tau_d}{f_{v.d}} + \frac{\sigma_{t.90.d}}{k_{dis} \cdot k_{vol} \cdot f_{t.90.d}} = 0.236$ — OK

Shear strength

$V_{d.s} := P_{Ed} \cdot \left(\frac{L}{2}\right) = 160.8\ kN$

Dimensioning shear force at the support

$\tau_d = 0.13\ MPa$

Shear force induced by a distibuted load can be reduced as follows

RIL 205-1-2007

$V_{red} := V_{d.s} \cdot \left(1 - \frac{h+l}{L}\right) = 144.184\ kN$

Reduced shear force

$h_{cr2} := h + \frac{h+l}{16} = 1.197\ m$

Critical cross-section

$\tau_d := \frac{3}{2} \cdot \frac{V_d}{b \cdot h_{cr2}} = 0.185\ MPa$

Shear stress at the support

$\tau_d \le \frac{f_{v.k} \cdot k_{mod}}{\gamma_M}$

Dimensioning condition

$\frac{\tau_d}{\frac{f_{v.k} \cdot k_{mod}}{\gamma_M}} = 0.079$

Utilisation ratio OK

Bearing pressure strength

$N_d := R$

$\sigma_{c.90.d} := \frac{N_d}{b \cdot l} = 1.881\ MPa$

Compression stress in the beam

$f_{c.90d} := \frac{f_{c.90.k} \cdot k_{mod}}{\gamma_M} = 1.667\ MPa$

Compression strength perpendicular to grains

$k_{c.90} := 1$

RIL 205-1-2007

$\sigma_{c.90.d} \le k_{c.90} \cdot f_{c.90.d}$

Dimensioning condition

$\frac{\sigma_{c.90.d}}{k_{c.90} \cdot f_{c.90.d}} = 1.128$

Utilisation ratio EXCEEDS THE LIMIT

Deflection

$I_y := \frac{b \cdot hx^3}{12} = (4.245 \cdot 10^{10})\ mm^4$

$w_{inst.G} := \frac{5 \cdot P_{G.SLS} \cdot L^4}{384 \cdot E_{mean} \cdot I_y} + \frac{P_C \cdot L^3}{48 \cdot E_{mean} \cdot I_y} = 6.816\ mm$ — Transient deflection due to permanent loads

$w_{inst.Q} := \frac{5 \cdot P_{Q.SLS} \cdot L^4}{384 \cdot E_{mean} \cdot I_y} = 11.944\ mm$ — Transient deflection due to variable loads

$k_{def} := 0.6$ — For glu-lam in use category 1

$\psi_{2.1} := 0.2$

$w_{net.G} := (1 + k_{def}) \cdot w_{inst.G} = 10.906\ mm$

$w_{net.Q} := (1 + \psi_{2.1} \cdot k_{def}) \cdot w_{inst.Q} = 13.377\ mm$

$w_{fin} := w_{net.G} + w_{net.Q} = 24.283\ mm$ — Total deflection

$w_{fin} \le \frac{L}{200}$

$\frac{L}{300} = 50\ mm$

$\frac{15.52}{50} = 0.31$ — Utilisation ratio OK

Lateral buckling strength

$\sigma_{m.a.d} := \frac{6 \cdot M_{d.cr}}{b \cdot h^2} = 13.43\ MPa$ — Bending stress in the critical cross-section

$a := 2500\ mm$ — Width of a roof element

$l_{eff} := a + 2 \cdot hx = 5278.493\ mm$ — Effective span length of a laterally supported beam

$c := 0.71$ — For glu-lam

$$\sigma_{m.crit} := \frac{c \cdot b^2}{hx \cdot l_{eff}} \cdot E_{0.05} = 37.749\ MPa$$

Critical bending stress of a rectangular cross-sectioned beam

$$\lambda_{rel.m} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} = 0.891$$

Relative slenderness ratio

$$k_{crit} := 1$$

RIL 205-1-2007

$$\sigma_{m.\alpha.d} \le k_{crit} \cdot f_{m.d}$$

Dimensioning condition

$$\frac{\sigma_{m.\alpha.d}}{k_{crit} \cdot f_{m.d}} = 0.671$$

Utilisation ratio OK

NOTE! The beam is to be installed steel plates on top of the supports, since the bearing pressure capacity is not enough as it is. However, since the utilisation ratio of the beam is otherwise very conservative, it might not be wise to increase the profile size. That is not reviewed any further in this thesis.

Appendix 3. Glued-laminated mast pillars: calculations

Mast pillars, glued-laminated timber GL30c

Column dimensions

$h := 450\ mm$	Height of the column cross-section
$b := 190\ mm$	Width of the column cross-section
$A := h \cdot b = 85500\ mm^2$	Column's cross-sectional area
$L := 5\ m$	Length of the column
$B := 15\ m$	Width of the frame
$H := 7\ m = 7000\ mm$	Height of the frame
$k := 5\ m$	Center to center dimension of beams

Vertical loads:

$K_{FI} := 1$

Dead load and self-weight

$g_1 := 1.4\ \frac{kN}{m}$	Self-weight of the beam
$g_2 := 0.5\ \frac{kN}{m^2}$	Dead load of roof elements
$g_3 := 0.2\ \frac{kN}{m^2}$	Suspension load of HVAC systems
$g_4 := 0.7\ \frac{kN}{m}$	Dead load of the crane
$P_{G\,SLS} := g_1 + g_4 + k \cdot (g_2 + g_3) = 5.6\ \frac{kN}{m}$	Self-weight of water roof in SLS

Crane loads:

$P_C := 1\ kN$ — Point load from a crane

$P_{C.ULS} := 1.5 \cdot K_{FI} \cdot P_C = 1.5\ kN$ — Crane load in ULS

Snow load:

$s_k := 2.5\ \frac{kN}{m^2}$ — Characteristic snow load on the ground

$\mu_1 := 0.8$ — Shape coefficient of the roof

$q_k := \mu_1 \cdot s_k = 2\ \frac{kN}{m^2}$ — Characteristic value of snow load on the roof

$P_{Q.SLS} := q_k \cdot k = 10\ \frac{kN}{m}$ — Snow load in SLS

$P_{Q.ULS} := 1.5 \cdot K_{FI} \cdot q_k = 3\ \frac{kN}{m^2}$ — Snow load in ULS

$P_{Ed} := 1.15 \cdot K_{FI} \cdot P_{G.SLS} + 1.5 \cdot K_{FI} \cdot P_{Q.SLS} = 21.4\ \frac{kN}{m}$ — Combined self-weight and snow load in ULS

$\Psi_{0.W} := 0.6$ $\quad \Psi_{0.S} := 0.7$ $\quad \Psi_{0.C} := 1$

Horizontal loads:

$C_b := 1\ kN$ — Brake load from the crane

$P_{Cb.ULS} := 1.5 \cdot K_{FI} \cdot C_b = 1.5\ kN$ — Crane load in ULS

Wind load perpendicular to the longer side

$q_p(h) := 0.61 \ \frac{kN}{m^2}$

Velocity pressure
SFS-EN 1991-1-4, 4.5
Terrain category II, flat orography

$c_s c_d := 1.0$

Structural factor
SFS-EN 1991-1-4, 7.2

$c_f := 1.8$

Force coefficient

$q_{w.ULS} := 1.5 \cdot K_{FI} \cdot c_s c_d \cdot c_f \cdot q_p(h) \cdot k = 8.235 \ \frac{kN}{m}$

Wind load of the wall in ULS

$F_{w.d} := q_{w.ULS} \cdot (H - L) = 16.47 \ kN$

Wind load of the roof in ULS

Maximum normal force of the column in ULS

$N_d := \left(1.15 \cdot K_{FI} \cdot (g_2 + g_3) + 1.5 \cdot K_{FI} \cdot \Psi_{0.S} \cdot q_k\right) \cdot k \cdot \frac{B}{2} + 1.15 \cdot K_{FI} \cdot g_1 \cdot \frac{B}{2} = 121.013 \ kN$

$M_{d.q} := \frac{5 \cdot q_{w.ULS} \cdot L^2}{16} = 64.336 \ kN \cdot m$

Distributed bending moment of the column

$M_{d.P} := \frac{F_{w.d} \cdot L}{2} = 41.175 \ kN \cdot m$ $\qquad m$

$M_{Ed} := M_{d.q} + M_{d.P} = 105.511 \ kN \cdot m$

Total bending moment on the column

$V_{Ed} := \frac{4 \cdot q_{w.ULS} \cdot L}{5} + \frac{F_{w.d}}{2} = 41.175 \ kN$

Shear force of the column in ULS

$L_{c.z} := 2.5 \cdot L = 12.5 \ m$

$I_y := \frac{b \cdot h^3}{12} = \left(1.443 \cdot 10^9\right) \ mm^4$

$i_y := \sqrt{\frac{I_y}{A}} = 129.904 \ mm$

$\lambda_y := \frac{L_{c.z}}{i_y} = 96.225$

Slenderness ratio

$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 1.459$ Relative slenderness ratio

$\beta_c := 0.1$ Factor for glu-lam
RIL 205-1-2007

$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 1.622$ ky-factor

$k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.429$ Buckling coefficient, $k_{c.y} < 1$
OK

$\sigma_{c.0.d} := \frac{N_d}{b \cdot h} = 1.415\ MPa$ Compression stress

$k_{mod} := 1.1$

$f_{c.0.d} := \frac{f_{c.0.k} \cdot k_{mod}}{\gamma_M} = 22.458\ MPa$ Compression strength

$\sigma_{m.y.d} := \frac{6 \cdot M_{Ed}}{b \cdot h^2} = 16.454\ MPa$ Bending stress

$f_{m.d} := \frac{f_{m.k} \cdot k_{mod}}{\gamma_M} = 27.5\ MPa$ Bending strength

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.y.d}} \le 1$$

$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.d}} = 0.745$ Combined effect of compression and bending stress, utilisation ratio OK

$q_{w.d.2} := 1.5 \cdot K_{FI} \cdot \Psi_{0.W} \cdot c_s c_d \cdot c_f \cdot q_p(h) \cdot k = 4.941\ \frac{kN}{m}$ Wind load of the wall in ULS

$F_{w.d.2} := q_{w.d.2} \cdot (H - L) = 9.882\ kN$ Wind load of rooof in ULS

Normal force of the column in ULS

$$N_{d.2} := \left(1.15 \cdot K_{FI} \cdot \left(g_2 + g_3\right) + 1.5 \cdot K_{FI} \cdot q_k\right) \cdot k \cdot \frac{B}{2} + 1.15 \cdot K_{FI} \cdot g_1 \cdot \frac{B}{2} = 154.763\ kN$$

$M_{d.q.2} := \dfrac{5 \cdot q_{w.d.2} \cdot L^2}{16} = 38.602\ kN \cdot m$ — Distributed bending moment in ULS

$M_{d.P.2} := \dfrac{\left(F_{w.d.2} + P_{Cb.ULS}\right) \cdot L}{2} = 28.455\ kN \cdot m$ — Point load of bending moment in ULS

$M_{Ed.2} := M_{d.q.2} + M_{d.P.2} = 67.057\ kN \cdot m$ — Maximum bending moment in ULS

$V_{d.c} := \dfrac{4 \cdot q_{w.d.2} \cdot L}{5} = 19.764\ kN$ — Shear force on the column in ULS

$\sigma_{c.0.d.2} := \dfrac{N_{d.2}}{b \cdot h} = 1.81\ MPa$ — Bending stress

$\sigma_{m.y.d.2} := \dfrac{6 \cdot M_{Ed.2}}{b \cdot h^2} = 10.457\ MPa$ — Bending stress

$$\frac{\sigma_{c.0.d.2}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d.2}}{f_{m.d}} \le 1$$

$\dfrac{\sigma_{c.0.d.2}}{k_{c.y} \cdot f_{c.0.d}} + \dfrac{\sigma_{m.y.d.2}}{f_{m.d}} = 0.568$ — The combined effect of compression and bending stress, utilisation ratio OK

Lateral buckling strength

$\sigma_{m.y.d} := \dfrac{6 \cdot M_{Ed}}{b \cdot h^2} = 16.454\ MPa$ — Bending stress

$a := 5\ m$ — Lateral buckling support span

$l_{eff.q} := 0.5 \cdot L = 2.5\ m$ — Effective span length for distibuted load

$l_{eff.q.r} := l_{eff.q} - 0.5 \cdot h = 2.275\ m$ — Reduced effective span length, due to the location of the load: on the tension side

$l_{eff.P} := 0.8 \cdot L = 4\ m$

$$l_{eff.qP} := \frac{M_{d.q.2} \cdot l_{eff.q.r} + M_{d.P.2} \cdot l_{eff.P}}{M_{d.q.2} + M_{d.P.2}} = 3006.992\ mm$$

Effective span length for distributed and point load

$c := 0.71$

For glu-lam

$$\sigma_{m.crit} := \frac{c \cdot b^2}{h \cdot l_{eff.qP}} \cdot E_{0.05} = 204.571\ MPa$$

Critical bending stress

$$\lambda_{rel.m} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} = 0.383$$

Relative slenderness ratio

$k_{crit} := 1$

$$\tau_d := \frac{3}{2} \cdot \frac{V_{Ed}}{b \cdot h} = 0.722\ MPa$$

Shear force at the support

$$f_{v.d} := \frac{f_{v.k} \cdot k_{mod}}{\gamma_M} = 3.208\ MPa$$

Shear strength

$\tau_d \le f_{v.d}$

$$\frac{\tau_d}{f_{v.d}} = 0.225$$

Shear strength condition, utilisation ratio OK

Appendix 4. Mast pillar – steel: calculations

Mast pillar, steel S355J2

Material properties: (HEB200)

$f_y := 355\ MPa$	Yield strength
$E := 210000\ MPa$	Modulus of elasticity
$\gamma_{M0} := 1$	
$\gamma_{M1} := 1$	
$L := 5\ m$	Column length
$h := 200\ mm$	Profile height
$b := 200\ mm$	Profile width
$t_f := 15\ mm$	Thickness of the flange
$t_w := 9\ mm$	Thickness of the web
$h_w := h - (2 \cdot t_f) = 170\ mm$	Height of the web
$r := 18\ mm$	Root radius
$i_y := 85.4\ mm$	Radius of gyration
$A := 7808\ mm^2$	Cross-sectional area
$W := 569.6 \cdot 10^3\ mm^3$	Section modulus
$I := 56.96 \cdot 10^6\ mm^4$	Moment of inertia
$B := 15\ m$	Width of the frame
$H := 7\ m = 7000\ mm$	Height of the frame
$k := 5\ m$	Center to center dimension of beams

Vertical loads:

$K_{FI} := 1$

<u>Dead load and self-weight</u>

$g_1 := 0.6\ \frac{kN}{m}$ Self-weight of the truss

$g_2 := 0.5\ \frac{kN}{m^2}$ Dead load of roof elements

$g_3 := 0.2\ \frac{kN}{m^2}$ Suspension load of HVAC systems

$g_4 := 0.7\ \frac{kN}{m}$ Dead load of the crane

$P_{G.SLS} := g_1 + g_4 + k \cdot (g_2 + g_3) = 4.8\ \frac{kN}{m}$ Self-weight of water roof in SLS

<u>Crane loads:</u>

$P_C := 1\ kN$ Point load from a crane

$P_{C.ULS} := 1.5 \cdot K_{FI} \cdot P_C = 1.5\ kN$ Crane load in ULS

<u>Snow load:</u>

$s_k := 2.5\ \frac{kN}{m^2}$ Characteristic snow load on the ground

$\mu_1 := 0.8$ Shape coefficient of the roof

$q_k := \mu_1 \cdot s_k = 2\ \frac{kN}{m^2}$ Characteristic value of snow load on the roof

$P_{Q.SLS} := q_k \cdot k = 10\ \frac{kN}{m}$ Snow load in SLS

$P_{Q.ULS} := 1.5 \cdot K_{FI} \cdot q_k = 3\ \frac{kN}{m^2}$ Snow load in ULS

Combined self-weight and snow load in ULS

$$P_{Ed} := 1.15 \cdot K_{FI} \cdot P_{G.SLS} + 1.5 \cdot K_{FI} \cdot P_{Q.SLS} + \frac{\left(\frac{P_C}{2}\right)}{\frac{B}{2}} = 20.6\ \frac{kN}{m}$$

$\Psi_{0.W} := 0.6$ $\Psi_{0.S} := 0.7$ $\Psi_{0.C} := 1$

Horizontal loads:

$C_b := 1\ kN$ Brake load from the crane

$P_{Cb.ULS} := 1.5 \cdot K_{FI} \cdot C_b = 1.5\ kN$ Crane load in ULS

<u>Wind load perpendicular to the longer side</u>

$q_p(h) := 0.61\ \frac{kN}{m^2}$ Velocity pressure
SFS-EN 1991-1-4, 4.5
Terrain category II, flat orography

$c_s c_d := 1.0$ Structural factor
SFS-EN 1991-1-4, 7.2

$c_f := 1.8$ Force coefficient

$q_{w.ULS} := 1.5 \cdot K_{FI} \cdot c_s c_d \cdot c_f \cdot q_p(h) \cdot k = 8.235\ \frac{kN}{m}$ Wind load of the wall in ULS

$F_{w.d} := q_{w.ULS} \cdot (H - L) = 16.47\ kN$ Wind load of the roof in ULS

Maximum normal force of the column in ULS

$$N_{Ed} := \left(1.15 \cdot K_{FI} \cdot (g_2 + g_3) + 1.5 \cdot K_{FI} \cdot \Psi_{0.S} \cdot q_k\right) \cdot k \cdot \frac{B}{2} + 1.15 \cdot K_{FI} \cdot g_1 \cdot \frac{B}{2} = 114.113\ kN$$

$M_{d.q} := \frac{5 \cdot q_{w.ULS} \cdot L^2}{16} = 64.336\ kN \cdot m$ Distributed bending moment of the column

$M_{d.P} := \frac{F_{w.d} \cdot L}{2} = 41.175\ kN \cdot m$

$M_{Ed} := M_{d.q} + M_{d.P} = 105.511\ kN \cdot m$ Total bending moment on the column

$V_{Ed} := \frac{4 \cdot q_{w.ULS} \cdot L}{5} + \frac{F_{w.d}}{2} = 41.175\ kN$ Shear force of the column in ULS

Cross-sectional class:

Profile's cross-sectional properties obtained from EurocodeApplied.com (cross-section class 1)

$M_{c.Rd} := 108.56\ kN \cdot m$ Bending strength

$\frac{M_{Ed}}{M_{c.Rd}} = 0.972$ Utilisation ratio OK

$A_v := 2518\ mm^2$ Shear area of cross-section

$V_{pl.Rd} := 508.94\ kN$ Shear strength

$\frac{V_{Ed}}{V_{pl.Rd}} = 0.081$ Utilisation ratio OK

$N_{pl.Rd} := 2771.88\ kN$ Compression strength

$\frac{N_{Ed}}{N_{pl.Rd}} = 0.041$ Utilisation ratio OK

$\frac{N_{Ed}}{0.25 \cdot N_{pl.Rd}} = 0.165$ OK

$\frac{N_{Ed}}{\left(\frac{0.5 \cdot h_w \cdot t_w \cdot f_y}{\gamma_{M0}}\right)} = 0.42$ OK

-> OK, therefore no need to check the combined effect of bending and normal force

Buckling

$\lambda := \pi \cdot \sqrt{\left(\frac{E}{f_y}\right)} = 76.409$ Slenderness ratio

$K_c := 2$ Factor for buckling length

$L_{cr} := K_c \cdot L = 10\ m$ Buckling length

$\lambda_{rel} := \frac{L_{cr}}{i_y} \cdot \frac{1}{\lambda} = 1.532$

Relative slenderness $\lambda_{rel} \geq 0.2$, so buckling needs to be checked

$\frac{h}{b} = 1$

-> Buckling curve c -> $\alpha := 0.49$ Imperfection factor
Table 6.1
SFS-EN 1993-1-1 s. 62

$\phi := 0.5 \cdot \left(1 + \alpha \cdot \left(\lambda_{rel} - 0.2\right) + \lambda_{rel}^{\ 2}\right) = 2.001$

$\chi := \frac{1}{\phi + \sqrt{\phi^2 - \lambda_{rel}^{\ 2}}} = 0.304$ $\chi \leq 1.0$ OK

$N_{b.Rd} := \frac{\chi \cdot A \cdot f_y}{\gamma_{M1}} = 843.29\ kN$

$\frac{N_{Ed}}{N_{b.Rd}} = 0.135$ Utilisation ratio OK

Lateral buckling

$c_1 := 1.00$

$\lambda_{Lt} := \frac{\frac{L_{cr}}{i_y}}{\sqrt{c_1} \cdot \sqrt{1 + \frac{1}{20}\left(\left(\frac{L_{cr}}{i_y}\right) \cdot \left(\frac{h}{t_f}\right)^{-1}\right)^2}} = 53.136$ Relative slenderness ratio

$\lambda_{LT} := \frac{\lambda_{Lt}}{\lambda} = 0.695$

$\lambda_{LT} \geq 0.2$ -> Lateral buckling needs to be checked

$\frac{h}{b} \leq 2$ Recommended lateral buckling curve: b

$\alpha_{LT2} := 0.34$

$\lambda_{LT.0} := 0.4$

$\beta := 0.75$

Recommended values for hot formed profiles

$\phi_{LT} := 0.5 \cdot \left(1 + \alpha_{LT2} \cdot \left(\lambda_{LT} - \lambda_{LT.0}\right) + \beta \cdot {\lambda_{LT}}^2\right) = 0.732$

$\chi_{LT} := \frac{1}{\phi_{LT} + \sqrt{{\phi_{LT}}^2 - \beta \cdot {\lambda_{LT}}^2}} = 0.872$

Decrease factor

$M_{b.Rd} := \chi_{LT} \cdot W \cdot \frac{f_y}{\gamma_{M1}} = 176.309\ kN \cdot m$

Bending moment resistance

$\frac{M_{Ed}}{M_{b.Rd}} = 0.598$

Utilisation ratio OK

Combined bending and compression

$$\frac{N_{Ed}}{\frac{\chi_y \cdot N_{Rk}}{\gamma_{M1}}} + k_{yy} \cdot \frac{M_{y.Ed} + \Delta M_{y.Ed}}{\frac{M_{y.Rk} \cdot \chi_{LT}}{\gamma_{M1}}} + k_{yz} \cdot \frac{M_{z.Ed} + \Delta M_{z.Ed}}{\frac{M_{z.Rk}}{\gamma_{M1}}} \leq 1$$

SFS-EN 1993-1-1
Formula 6.61 & 6.62 s.70

$$\frac{N_{Ed}}{\chi_y \cdot N_{Rk}} + k_{yy} \cdot \frac{M_{y.Ed}}{M_{y.Rk} \cdot \chi_{LT}} \leq 1$$

$N_{Rk} := A \cdot f_y = 2771.84\ kN$

Normal force resistance

$M_{y.Rk} := W \cdot f_y = 202.208\ kN \cdot m$

α_s = Ms/Mh = 1

SFS-EN 1993-1-1
Table B.3 s. 87

$\alpha_s := 1$

$c_{my} := \max\left(0.2 + 0.8 \cdot \alpha_s, 0.4\right) = 1$ Moment factor

$k_{yy} := c_{my} \cdot \left(1 + \left(\lambda_{rel} - 0.2\right) \cdot \dfrac{N_{Ed}}{\chi \cdot \dfrac{N_{Rk}}{\gamma_{M1}}}\right) = 1.18$ Combined effect factor

$M_{y.Ed} := M_{Ed}$

$\dfrac{N_{Ed}}{\chi \cdot N_{Rk}} + k_{yy} \cdot \dfrac{M_{y.Ed}}{\chi_{LT} \cdot M_{y.Rk}} = 0.842$ Utilisation ratio OK

Appendix 5. Glued-laminated wind column: calculations

Storage building and column dimensions:

$k := 5\ m$ $\quad B := 15\ m$ $\quad D := 25\ m$

Column dimensions

$h := 270\ mm$	Height of the column cross-section
$b := 115\ mm$	Width of the column cross-section
$A := h \cdot b = (3.105 \cdot 10^4)\ mm^2$	Column's cross-sectional area
$L := 6.5\ m$	Length of the column
$a := L$	Lateral buckling length
$B := 3.75\ m$	Loading width
$\Psi_{0.W} := 0.7$	Combination factor for wind
$K_{FI} := 1$	
$k_{mod} := 1.1$	
$c := 0.71$	For glu-lam

Vertical loads:

Dead load and self-weight

$g_1 := 0.1 \ \frac{kN}{m}$ Self-weight of the beam

$g_2 := 0.5 \ \frac{kN}{m^2}$ Dead load of roof elements

$g_3 := 0.2 \ \frac{kN}{m^2}$ Suspension load of HVAC systems

Snow load:

$s_k := 2.5 \ \frac{kN}{m^2}$ Characteristic snow load on the ground

$\mu_1 := 0.8$ Shape coefficient of the roof

$q_k := \mu_1 \cdot s_k = 2 \ \frac{kN}{m^2}$ Characteristic value of snow load on the roof

Wind load perpendicular to the shorter side

$q_p(h_1) := 0.61 \, \frac{kN}{m^2}$

Velocity pressure
SFS-EN 1991-1-4, 4.5
Terrain category II, flat orography

$c_f := 1.1$

Force coefficient
Interpolated from the chart
in Figure 3

$c_s c_d := 1$

Strctural factor

$\Psi_{0.W} := 0.7$

Combination factor for wind

$q_w := c_s c_d \cdot c_f \cdot q_p(H) = 0.671 \, \frac{kN}{m^2}$

Total characteristic wind load

$q_{w.k} := c_s c_d \cdot c_f \cdot q_w \cdot B = 2.768 \, \frac{kN}{m}$

Wind load of the wall in SLS

$q_{w.ULS} := 1.5 \cdot K_{FI} \cdot q_{w.k} = 4.152 \, \frac{kN}{m}$

Wind load in ULS

Wind load on the roof in ULS

$N_{Ed} := \left(1.15 \cdot K_{FI} \cdot (g_2 + g_3) + 1.5 \cdot K_{FI} \cdot \Psi_{0.W} \cdot q_k\right) \cdot \frac{k}{2} \cdot B + 1.15 \cdot g_1 \cdot B = 27.666 \, kN$

$M_d := \frac{9 \cdot q_{w.ULS} \cdot L^2}{128} = 12.334 \, kN \cdot m$

Bending moment at the
bottom of the column of
wind load in ULS

$V_{Ed} := \frac{5 \cdot q_{w.ULS} \cdot L}{8} = 16.867 \, kN$

Shear force of the column
due to wind load

$L_{c.z} := 0.85 \cdot L = 5.525 \, m$

Buckling length of the
column
RIL 205-1-2007

$I_y := \frac{b \cdot h^3}{12} = \left(1.886 \cdot 10^8\right) \, mm^4$

Moment of inertia

$i_y := \sqrt{\frac{I_y}{A}} = 77.942 \, mm$

$\lambda_y := \frac{L_{c.z}}{i_y} = 70.886$

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

$\beta_c := 0.1$ Factor of the pre curviness of glu-lam

$k_y := 0.5 \cdot \left(1 + \beta_c \cdot (\lambda_{rel.y} - 0.3) + {\lambda_{rel.y}}^2\right) = 1.116$

$k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.705$ Buckling factor

$\sigma_{c.0.d} := \frac{N_{Ed}}{b \cdot h} = 0.891\ MPa$ Compression stress

$f_{c.0.d} := \frac{f_{c.0.k} \cdot k_{mod}}{\gamma_M} = 22.458\ MPa$ Compression strength

$\sigma_{m.y.d} := \frac{6 \cdot M_d}{b \cdot h^2} = 8.827\ MPa$ Bending stress

$f_{m.d} := \frac{f_{m.k} \cdot k_{mod}}{\gamma_M} = 27.5\ MPa$ Bending strength

$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.d}} = 0.377$ Utilisation ratio OK

$l_{eff} := L$

Effective span length shall be increased

$l_{ef} := l_{eff} + 2 \cdot h = 7.04\ m$

$\sigma_{m.crit} := \dfrac{c \cdot b^2}{h \cdot l_{eff}} \cdot E_{0.05} = 57.783\ MPa$ Critical bending stress

$\lambda_{rel.m} := \sqrt{\dfrac{f_{m.k}}{\sigma_{m.crit}}} = 0.721$ Relative slenderness

$k_{crit} := 1$ Lateral buckling can be ignored

Shear strength

$\tau_d := \dfrac{3}{2} \cdot \dfrac{V_{Ed}}{b \cdot h} = 0.815\ MPa$ Shear stress at the support

$f_{v.d} := \dfrac{f_{v.k} \cdot k_{mod}}{\gamma_M} = 3.208\ MPa$ Shear strength

$\dfrac{\tau_d}{f_{v.d}} = 0.254$ Shear strength utilisation ratio OK

Bending strength

$w_{inst.Q} := \dfrac{q_{w.k} \cdot L^4}{185 \cdot E_{mean} \cdot I_y} = 10.891\ mm$ Transient deflection

$w_{net.fin} := w_{inst.Q}$

$\dfrac{w_{net.fin}}{\dfrac{L}{300}} = 0.503$ Bending strength utilisation ratio OK

Appendix 6. Glued-laminated corner column: calculations

Material properties

$f_{m.k} := 30\ MPa$	Bending strength
$f_{v.k} := 3.5\ MPa$	Shear strength
$f_{c.0.k} := 24.5\ MPa$	Compression strength parallel to grains
$f_{c.90.k} := 2.5\ MPa$	Compression strength perpendicular to grains
$f_{t.90.k} := 0.5$	Tension perpendicular to grains
$k_{def} := 0.6$	Factor that considers moistur effect to the deflection
$E_{mean} := 13000\ MPa = \left(1.3 \cdot 10^4\right) \dfrac{N}{mm^2}$	Modulus of elasticity
$E_{0.05} := 10800\ MPa$	
$G_{mean} := 650\ MPa = 650 \dfrac{N}{mm^2}$	Shear modulus
$\gamma_M := 1.2$	Partial safety factor

Corner pillars, glued-laminated timber GL30c

Column dimensions

$h := 315\ mm$ Height of the column cross-section

$b := 140\ mm$ Width of the column cross-section

$A := h \cdot b = (4.41 \cdot 10^4)\ mm^2$ Column's cross-sectional area

$L := 6\ m$ Length of the column

$a := L$

$k := 3.75\ m$ Center to center-distance of columns

$B := 15\ m$ Building width

$H := 7\ m$ Building height

$c := 0.71$

$c_s c_d := 1$ Structural factor

$c_f := 1.8$ Force coefficient perpendicular to the shorter side

$\Psi_{0.W} := 0.6$ $\Psi_{0.S} := 0.7$ $\Psi_{0.D} := 1$ $K_{FI} := 1$ $k_{mod} := 1.1$

Vertical loads:

Dead load and self-weight

$g_1 := 0.1\ \dfrac{kN}{m}$ Self-weight of the beam

$g_2 := 0.5\ \dfrac{kN}{m^2}$ Dead load of roof elements

$g_3 := 0.2\ \frac{kN}{m^2}$ Suspension load of HVAC systems

Snow load:

$s_k := 2.5\ \frac{kN}{m^2}$ Characteristic snow load on the ground

$\mu_1 := 0.8$ Shape coefficient of the roof

$q_k := \mu_1 \cdot s_k = 2\ \frac{kN}{m^2}$ Characteristic value of snow load on the roof

Wind load

$q_p(H) := 0.61\ \frac{kN}{m^2}$ Velocity pressure
SFS-EN 1991-1-4, 4.5
Terrain category II, flat orography

$q_{w.k} := c_s c_d \cdot c_f \cdot q_p(H) \cdot \frac{k}{2} = 2.059\ \frac{kN}{m}$ Wind load of the wall in SLS

$F_{w.k} := q_{w.k} \cdot (H - L) = 2.059\ kN$ Wind load of roof in SLS

$q_{w.d} := 1.5 \cdot K_{FI} \cdot c_s c_d \cdot c_f \cdot q_p(H) \cdot \frac{k}{2} = 3.088\ \frac{kN}{m}$ Wind load of the wall in ULS

$F_{w.d} := q_{w.d} \cdot (H - L) = 3.088\ kN$ Wind load of roof in ULS

Normal force in ULS

$$N_d := \left(1.15 \cdot K_{FI} \cdot (g_2 + g_3) + 1.5 \cdot K_{FI} \cdot \Psi_{0.S} \cdot q_k\right) \cdot \frac{k}{2} \cdot \frac{B}{8} + 1.15 \cdot K_{FI} \cdot g_1 \cdot \frac{B}{8} = 10.429\ kN$$

$M_{d.q} := \frac{5 \cdot q_{w.d} \cdot L^2}{16} = 34.741\ kN \cdot m$ Bending moment for distributed load

$M_{d.P} := \frac{F_{w.d} \cdot L}{2} = 9.264\ kN \cdot m$ Bending moment for point load

$M_{Ed} := M_{d.q} + M_{d.P} = 44.006\ kN \cdot m$ Maximum bending moment in ULS

$V_d := \frac{4 \cdot q_{w.d} \cdot L}{5} + \frac{F_{w.d}}{2} = 16.367\ kN$ Shear force of the column in ULS

<u>Buckling strength</u>

$L_{c.z} := 2.5 \cdot L = 15\ m$ RIL 205-1-2007

$I_y := \frac{b \cdot h^3}{12} = \left(3.647 \cdot 10^8\right)\ mm^4$ Moment of inertia

$i_y := \sqrt{\frac{I_y}{A}} = 90.933\ mm$

$\lambda_y := \frac{L_{c.z}}{i_y} = 164.957$

$\lambda_{rel.y} := \frac{\lambda_y}{\pi} \cdot \sqrt{\frac{f_{c.0.k}}{E_{0.05}}} = 2.501$ Relative slenderness ratio

$\beta_c := 0.1$ Pre-curviness factor or glu-lam

$k_y := 0.5 \cdot \left(1 + \beta_c \cdot \left(\lambda_{rel.y} - 0.3\right) + {\lambda_{rel.y}}^2\right) = 3.737$

$k_{c.y} := \frac{1}{k_y + \sqrt{{k_y}^2 - {\lambda_{rel.y}}^2}} = 0.154$ Buckling factor

$\sigma_{c.0.d} := \frac{N_d}{b \cdot h} = 0.236\ MPa$ Compression stress

$f_{c.0.d} := \frac{f_{c.0.k} \cdot k_{mod}}{\gamma_M} = 22.458\ MPa$ Compression strength

$\sigma_{m.y.d} := \frac{6 \cdot M_{Ed}}{b \cdot h^2} = 19.007\ MPa$ Bending stress

$$f_{m.d} := \frac{f_{m.k} \cdot k_{mod}}{\gamma_M} = 27.5\ MPa$$

Bending strength

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.d}} \le 1$$

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.d}} = 0.76$$

$$q_{w.d} := 1.5 \cdot K_{FI} \cdot \Psi_{0.W} \cdot c_s c_d \cdot c_f \cdot q_p(H) \cdot \frac{k}{2} = 1.853\ \frac{kN}{m}$$

Wind load on the wall in ULS

$$F_{w.d} := q_{w.d} \cdot (H - L) = 1.853\ kN$$

Wind load of the roof in ULS

Normal force on a column in ULS

$$N_d := \left(1.15 \cdot K_{FI} \cdot \left(g_2 + g_3\right) + 1.5 \cdot K_{FI} \cdot q_k\right) \cdot \frac{k}{2} \cdot \frac{B}{2} + 1.15 \cdot K_{FI} \cdot g_1 \cdot \frac{B}{8} = 53.723\ kN$$

$$M_{d.q} := \frac{5 \cdot q_{w.d} \cdot L^2}{16} = 20.845\ kN \cdot m$$

Bending moment due to the distributed load

$$M_{d.P} := \frac{F_{w.d} \cdot L}{2} = 5.559\ kN \cdot m$$

Bending moment due to the point load

$$M_d := M_{d.q} + M_{d.P} = 26.403\ kN \cdot m$$

Maximum bending moment

$$V_d := \frac{4 \cdot q_{w.d} \cdot L}{5} + \frac{F_{w.d}}{2} = 9.82\ kN$$

Shear force on the column in ULS

$$\sigma_{c.0.d} := \frac{N_d}{b \cdot h} = 1.218\ MPa$$

Compression stress

$$\sigma_{m.y.d} := \frac{6 \cdot M_d}{b \cdot h^2} = 11.404\ MPa$$

Bending stress

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.d}} \le 1$$

$$\frac{\sigma_{c.0.d}}{k_{c.y} \cdot f_{c.0.d}} + \frac{\sigma_{m.y.d}}{f_{m.d}} = 0.768$$

Bending strength utilisation ratio OK

Lateral buckling strength

$$\sigma_{m.y.d1} := \frac{6 \cdot M_{Ed}}{b \cdot h^2} = 19.007\ MPa$$

$$l_{eff} := 0.5 \cdot L = 3\ m$$

Effective length of a laterally non-supported column for distributed load

$$l_{eff.r} := l_{eff} - 0.5 \cdot h = 2.843\ m$$

Reduced effective length due to the load locating on tension side

$$l_{eff.P} := L \cdot 0.8 = 4.8\ m$$

Effective length of a laterally non-supported column for point load

$$l_{eff.1} := \frac{l_{eff} \cdot M_{d.q} + l_{eff.P} \cdot M_{d.P}}{M_{d.q} + M_{d.P}} = 3378.947\ mm$$

Effective length for combined distributed and point load

$$\sigma_{m.crit} := \frac{c \cdot b^2}{h \cdot l_{eff.1}} \cdot E_{0.05} = 141.204\ MPa$$

$$\lambda_{rel.m} := \sqrt{\frac{f_{m.k}}{\sigma_{m.crit}}} = 0.461$$

$$k_{crit} := 1$$

$$\tau_d := \frac{3}{2} \cdot \frac{V_d}{b \cdot h} = 0.334\ MPa$$

Shear stress

$$f_{v.d} := \frac{f_{v.k} \cdot k_{mod}}{\gamma_M} = 3.208\ MPa$$

Shear strength

$$\frac{\tau_d}{f_{v.d}} = 0.104$$

Shear strength utilisation ratio OK

Appendix 7. Cost estimation

Cost estimation steel	Case-building

Profile	Grade	Qty	Length(mm)	Net Area(m²) for one	Net Area(m²) for all	Net Weight(kg) for one	Net Weight(kg) for all	Total price estimation
CFSHS80X80X5	S355J2	8	1749,29	0,53	4,24	20,60	164,78	
CFSHS80X80X5	S355J2	8	1750,89	0,53	4,24	20,62	164,93	
CFSHS80X80X5	S355J2	4	2000	0,61	2,42	23,55	94,20	
CFSHS80X80X5	S355J2	16	2267,16	0,69	10,99	26,70	427,13	
	Total	36	72275		21,90		851,05	3 415 €
CFSHS150X150X5	S355J2	8	7858,75	4,58	36,65	178,90	1431,24	
CFSHS150X150X5	S355J2	4	15200	8,86	35,45	346,03	1384,11	
	Total	12	123670		72,10		2815,35	10 250 €
HEA200	S355J2	10	5000	5,68	56,80	200,37	2003,71	
HEA200	S355J2	4	7938,44	9,02	36,07	318,13	1272,51	
	Total	14	81753		92,87		3276,22	10 850 €
HEA220	S355J2	3	6328,41	7,94	23,83	304,39	913,18	
HEA220	S355J2	2	6980	8,76	17,52	337,31	674,61	
	Total	5	32945		41,35		1587,79	5 120 €
HEB200	S355J2	11	4980	5,73	63,05	294,37	3238,07	
HEB200	S355J2	1	6325,75	7,28	7,28	372,33	372,33	
	Total	12	61105		70,33		3610,41	11 500 €
IPE100	S355J2	80	4880	3,75	299,83	104,38	8350,53	
	Total	80	390399		299,83		8350,53	12 200 €
							VAT 0 %	53 335 €

Cost estimation glulam	Case-building

Profile								Total price estimation
Purlins								
Double pitched ridge beams								
Wooden forked boards								
Columns								
Column shoes								
							VAT 0 %	47 000 €