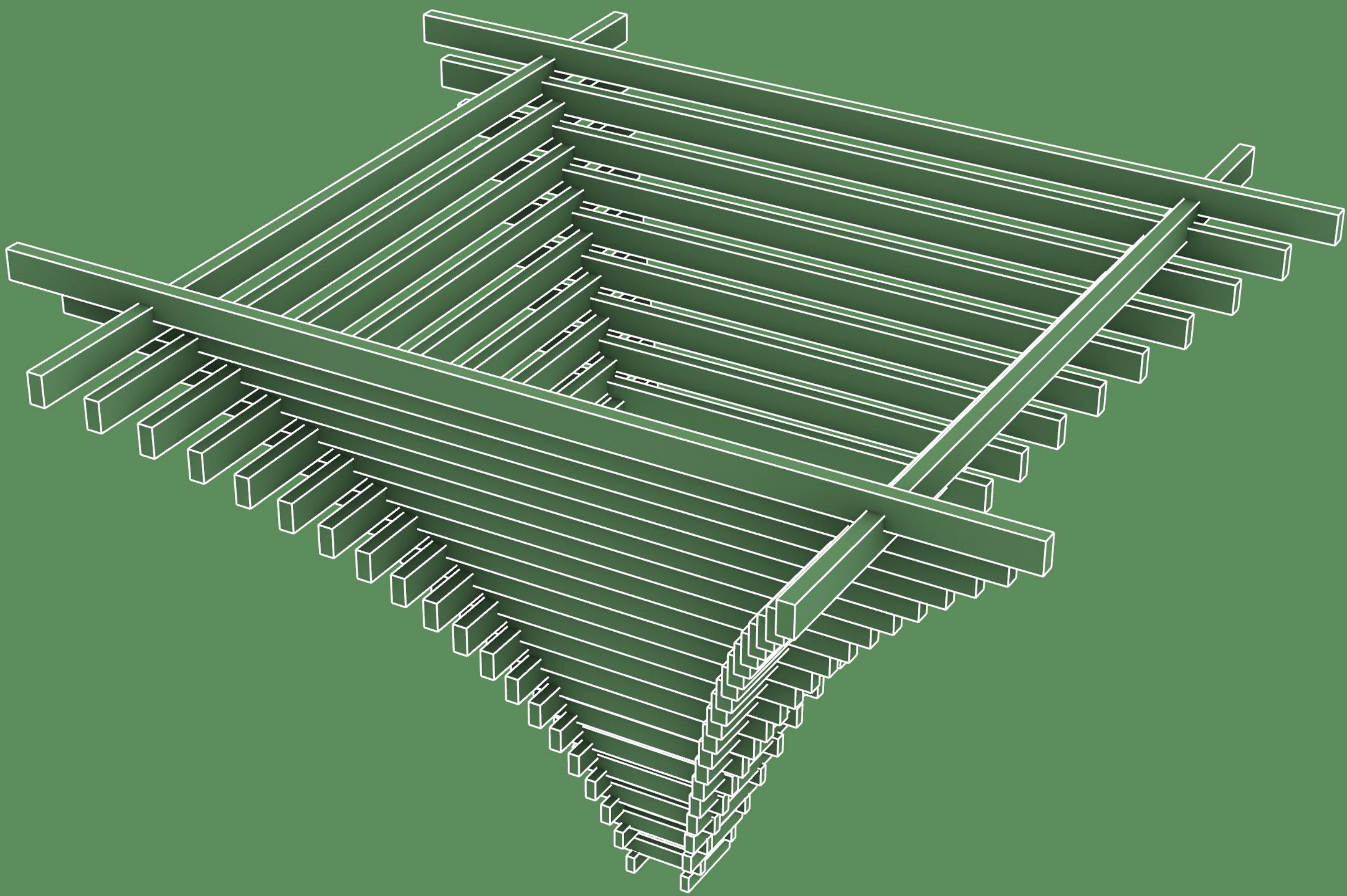
Structural performance of reversible discrete timber systems



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Preface

This thesis presents the last part of my master Building Technology at the Faculty of Architecture, Delft University of Technology.

I hope that the research presented in this thesis will in some way or form contribute in helping reduce the problem of global climate change and influences others to do so as well.

I want to thank my mentors, Dr. Stijn Brancart and Prof. Alex de Rijke for their time, guidance and useful insights & feedback that helped me steer this research in the right direction.

Bryan Zwakkenberg

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Abstract

With 39%, the construction industry accounts for a significant part of the global greenhouse gas (GHG) emissions, additionally it is accountable for 40% of all extracted materials, 40% of primary energy usage and 40% of the total waste generated. This all adds up to CO^2-emissions in the form of embodied or operational carbon. Meanwhile the temperature limit set in 2015 signed Paris Agreement with the sole purpose of reducing the global temperature increase, is on the brink of being exceeded.

If the global GHG emissions are not drastically reduced the global temperature rise will increase even more, and with that the severity of the climate change consequences too. GHG emissions in the building sector can be decreased by reducing operational and/or embodied carbon. Since embodied carbon is getting increasingly higher but lacks in research and innovative solutions, this research explores the possibility of reducing embodied carbon by desiging a reversible discrete timber system as alternative to a conventional structural timber element or system.

For this a joint that is reversible and applicable to discrete timber elements must be found and tested. Initially there was a collection of 198 wood joints, which was narrowed down to 14 that met the criteria to be applicable on perpendicularly combined elements. From these 14 joints, 5 were selected based on simplicity of the joint. In order to determine the loads on the discrete timber system, a case study was set to be a post war apartment block in Rotterdam South.

The selected joints were analysed in a Finite Element Analysis-software to determine what would happen if a load is applied to this element. The load showed highly localized peak stresses in the joints, indicating that the joints might be the weakness of the discrete timber elements. Next to the joints, the discrete timber elements showed expected behaviour to the applied load. In the end the simple square cog joint turned out to be the most suitable joint, because is resulted the best average stresses, but also due to its simplicity. In the end it was clear that the joints are not only the weaknesses in the system, but also the most crucial part in creating a reversible system.

Besides testing the discrete timber elements, the discrete timber systems also needed to be tested to see how they would react under the applied loads, and what the resulting maximum displacement and utilization values would be. Here various ways of aggregating discrete timber elements was tested. These aggregations were influenced by parameter such as the scaling factor (where a normal straight column, was scaled variedly into a mushroom-like column), dimensions of the base of the discrete system, the dimensions of the cross sections and the material of the discrete elements. The results from these tests showed the effects from the various parameters on the maximum displacement and utilization of the discrete timber system.

However, there are also some gaps with regards to which joints are suitable, moreover, can the joints be made in timber or should there be resorted to a reinforced joint in a different material than timber? One of the main strengths of a discrete timber system lays in its reversibility, and for the system to be reversible there must be a demountable joint.

Discrete timber systems can be a feasible alternative to conventional structural timber by ensuring that the discrete systems have strong, reversible joints that are simple in production and construction.

Keywords:

Climate change, discrete timber systems, structural capacity, reversible joints, top-up

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1. Introduction

1.1 Background

As per 2015, 196 countries (that account for 97% of global emissions) in the world entered into an agreement addressing the urge of climate change and the resulting, negative impacts. This agreement goes under the name “Paris Agreement”, and the main focus of this agreement is to drastically reduce global Greenhouse Gas (GHG) Emissions so that subsequently the resulting global temperature increase will also be reduced, limiting this temperature increase to a maximum of 1.5 degrees Celsius (National Resources Defense Council (NRDC), 2017, 2023). However, the European Centre for Medium-Range Weather Forecasts (ECMWF)’ most recent climate reanalysis production (ERA5) measured new record temperatures in the month of June 2023, as well as record sea-surfaces temperatures during the month of May in 2023. In addition, there is a 98% chance that one out of the next five years will read new record temperatures, the next five years together will see a new temperature record, and there is a 32% likelihood that the next five-year period will have a mean temperature increment higher than the 1.5°C limit set in the Paris Agreement (Copernicus Climate Change Services, 2018, 2023).

The construction industry accounts for a significant part of the global GHG emissions, placing it as the second largest CO2 emitter. Emission numbers can vary per source, according to the New Building Institute (NBI) (2023b) the GHG emissions from the construction industry are 39%, Pomponi and Moncaster (2016) say it is 35%, and according to Rahla et al., (2021) & Kisku et al., (2017) it accounts for 33% of the total. There are also sources that elaborate on other pollutions, uses or wastes; Solís-Guzmán et al., (2014) and Kisku et al., (2017) state that the construction industry is accountable for 40% of all extracted materials, 40% of primary energy usage, and for 40% of total waste generated. Pomponi and Moncaster (2016) limit their statement to the European Union, where the construction industry is accountable for 42% of primary energy usage and 50% of extracted materials. Greenhouse gas emissions include the following gases: Fluoro Carbons (FC), Nitrous Oxide (N2O), carbon dioxide (CO2), sulphur hexafluoride (SF6), and methane (CH4), but in the construction stage the CO2, N2O and CH4 are the main greenhouse gases (Iwata & Okada, 2014).

CO2-emissions in the construction industry can be divided into two categories: embodied and operational carbon (Hekma et al., 2021; Jarrett, 2023). Abel (2020) defines embodied carbon as: “the amount of carbon emitted during the making of a building. This includes extraction of raw materials, manufacture and refinement of materials, transport, the building phase of the product or structure, and the deconstruction and disposal of materials at the end of life” and operational carbon as: “the amount of carbon emitted during the operational or in-use phase of a building. This includes the use, management, and maintenance of a product or structure”. divides a full building lifecycle into modules, and assigns GHG emissions to either the embodied or operational stage.

CO2 emissions can be reduced by making changes to the processes in figure 2, but CO2 emissions also vary for different materials. Findings indicate that increased use of wood-based materials can help in mitigating climate change (Dodoo et al., 2014; Hart et al., 2021; Sandanayake et al., 2018; Werner & Richter, 2007). Sandanyake et al., (2018) conducted a case study and found that using timber instead of concrete reduced GHG emissions in material usage, material emissions, and transportation emissions. A study by Gustavsson, Pingoud & Sathre (2006) found that fully (100 percent) replacing reinforced concrete with engineered timber in a mid-rise building can save 26 MtCO2-eq. Sathre & O’Connor (2010) conducted

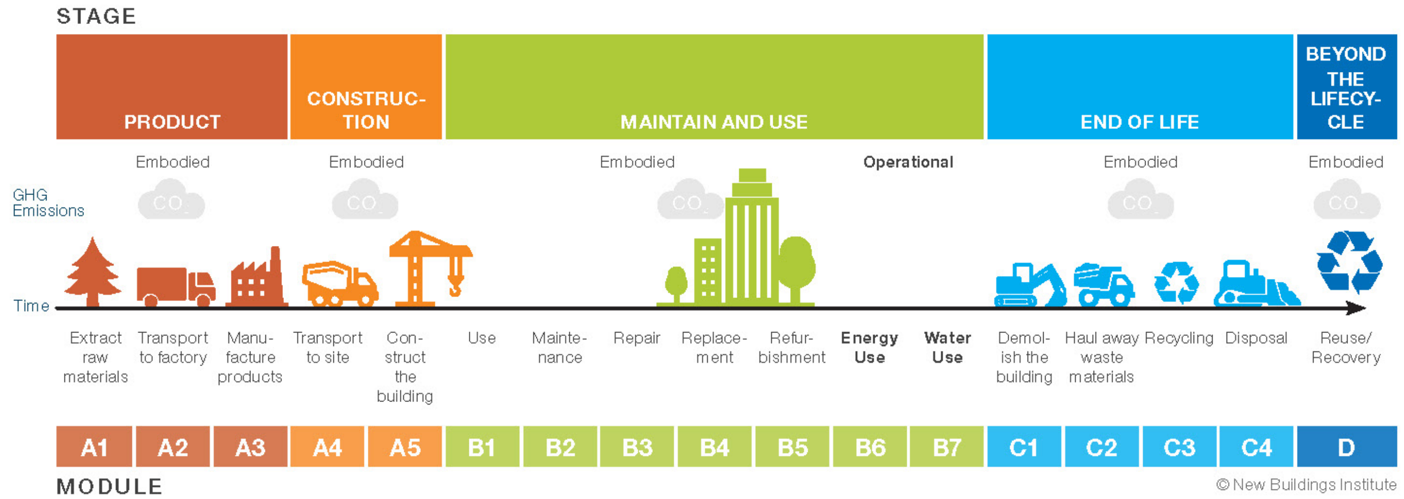

Figure 1.1: Building lifecycle stages(New Buildings Institute (NBI), 2023a)

a meta-analysis and found that by using timber instead of non-timber products GHG emissions can be reduced, per tonne of timber the reduction is about 3.9 tonne of CO2-eq. A comparative study by Hart et al., (2021) analysed building superstructures with matching engineered timber, steel and concrete frame constructions, and found the median Whole Life Embodied Carbon to be: 119 kgCO2-eq/m2 for engineered timber, 228 kgCO2-eq/m2 for steel, and 185 kgCO2-eq/m2 for concrete, see Figure 1.2.

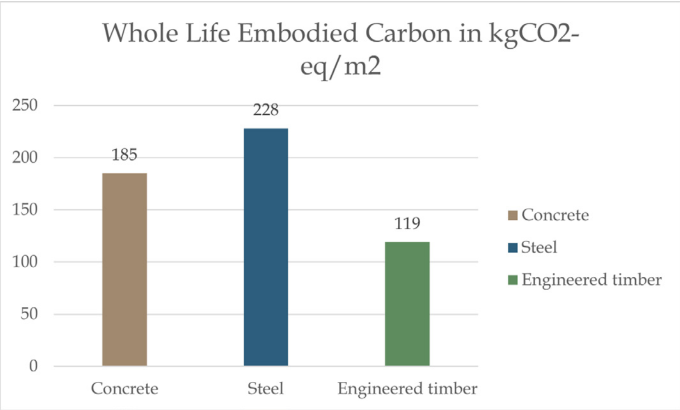

Figure 1.2: Whole Life Embodied Carbon emissions for concrete, steel and engineered timber (Adapted from Hart el al., 2021)

1.2 Problem statement

If no changes occur to global greenhouse gas (GHG) emissions, it is likely that the temperature goals that have been set in the Paris Agreement from 2015, are not reached (Copernicus Climate Change Services, 2018, 2023). The construction industry plays an important role in reducing the global GHG emissions, because it is accountable for approximately one third of the total global GHG emissions.

Greenhouse gas emissions in the construction industry are divided into two groups: operational and embodied carbon. Greenhouse gas emissions in the operational phase are assumed to be higher and have therefore been subject to more research and innovation in order to reduce this, resulting in various energy efficiency and renewable energy solutions. However, different studies have shown the increasing share of embodied carbon emissions. Reducing embodied carbon emissions can be done by replacing materials that are generating high emissions in production, but also by improving the end-of-life scenarios to extend a building life, reusing materials elsewhere and this way postponing the need for newly produced materials (Hekma et al., 2021; Ibn-Mohammed et al., 2013; Jarrett, 2023). Improved end-of-life scenarios can be ensured by following different design principles for structural adaptability, such as Design for Disassembly/Deconstruction, design in layers, and design for reuse (Ottenhaus et al., 2023).

There are multiple studies showing the advantages of using timber as a construction material instead of concrete or steel for example. Hart et al., (2021) showed that the whole life embodied carbon for engineered timber frame constructions is 35.7% lower than for its equivalent in concrete, and even 47.8% less than its equivalent in steel.

There is also research stating that structural discrete timber systems offer some advantages over conventional structural timber (sawn, massive, and engineered). Discrete in 'discrete systems' comes from 'discreteness', which refers to something being separate and individual. In the context of 'discrete systems' this translates to the elements in this system being somewhere in the spectrum between building element and particle, only having a function when they are combined with other discrete elements (Retsin, 2016b, 2019a, 2019b).

For example, it fits in economies of scale by producing only a single digit number of parts (that do not have a pre-defined function) rather than all pre-defined, customised, and optimized building parts, which results in a more time and cost efficient production. Furthermore, discrete systems are closely related to automation, resulting in fast assembly and complexity (Retsin, 2016a, 2019b). Additionally, in current society it is more efficient and cheaper to waste material instead of labour. Allowing material waste seems contradictory, which is the case for concrete for example, but for timber it can be said to be advantageous due to the carbon sequestering happening (Retsin, 2019b).

However, the application of discrete timber systems is mainly limited to just theoretical research, research prototypes, or small scale projects without calculations on the loadbearing capacity. Projects that did take the loadbearing capacity, or stress, into consideration are the 're-voxlam truss', 'robotic reversible timber beam', and 'reconwood slab' by SDU Create (CREATE SDU, 2019, 2023a, 2023b). Only these were more specific applications to their design, making it less applicable to other cases. Additionally, the current discrete timber systems are not optimally tailored following a design principle for structural adaptability.

To make discrete timber systems scalable in the industry, a method for structurally verifying different systems, that are designed for structural adaptability, is needed.

1.3 Research limitations

The focus of this research is to introduce a method to calculate the structural capacity of (a) discrete timber system(s). As this will be done computationally, the result are not immediately ready to use in real-life. Real strength tests are needed amongst other factors before this can be used. Because the structural calculations are done computationally, it can occur that the set-up for this calculation is not precisely how it would be in real-life.

1.4 Research goal

The aim of this thesis is to contribute to the current research by creating a method for structurally calculating a discrete timber system. To provide information on reversible joints for discrete timber elements, and apply this to the use case of a discrete timber system. By having a method to calculate structural capacity in a discrete timber system, it becomes much more feasible to use, not only the exact same system as used in this research, but also the logic can be applied in researching other discrete timber systems. The reversible joints and discrete timber elements will be combined to improve the end-of-life scenario for a structure, additionally with focusing on sustainability, this research aims to use only bio-based materials; timber.

1.5 Research questions

Main research question
"How can a reversible discrete timber system be a feasible alternative to conventional structural timber?"

Sub-research questions
- What makes a discrete timber system feasible?
- What defines the structural performance of discrete timber systems?
- Which reversible joinery techniques and joints are applicable to use in discrete timber systems?
- What is the structural performance of joints between discrete timber elements?

1.6 Methodology

In order to answer the aforementioned research questions, the framework in Figure 1.3 will be followed. This framework consists of five separate parts, the introduction, literature review, research for design, case study, and finally application of design to case study.

The introduction covers the background to set the context regarding discrete timber and the place this has within the problem statement.

The literature review is an exploration to the already available knowledge within the field regarding topics such timber construction and joints in timber, discrete system, and design for deconstruction and reuse.

The literature review is then used as a basis for conducting the research on "Structural calculation of discrete timber systems with reversible joints". For which structural calculations are conducted on two levels. Chapter 6 focusses on the structural design on element level by looking at the stresses and deflection in one element or a few layers of elements, chapter 7 focusses on structural design on a system level by checking various element aggregations and their effect. Both use calculated loads that are expected on top of the column, partially based on the case study defined in chapter 5.

The goal is for chapter 6 and 7 to yield results informing on which joint design, and element aggregations are more suitable in the use case of this design.

It is possible to depict this research in one image as a funnel. At the start it is broad, looking at all the available joints and connection methods in timber. However throughout the research it becomes clear that some joints are not suitable and others are, basically narrowing down the possibilities, or rather getting further down the funnel. Then the joints still deemed suitable are tested to see if they are as good structurally as they seemed earlier. At which again some joints will succeed and others will fail. This goes on until at the end is a discrete timber system aggregated from discrete timber elements reversible joints.

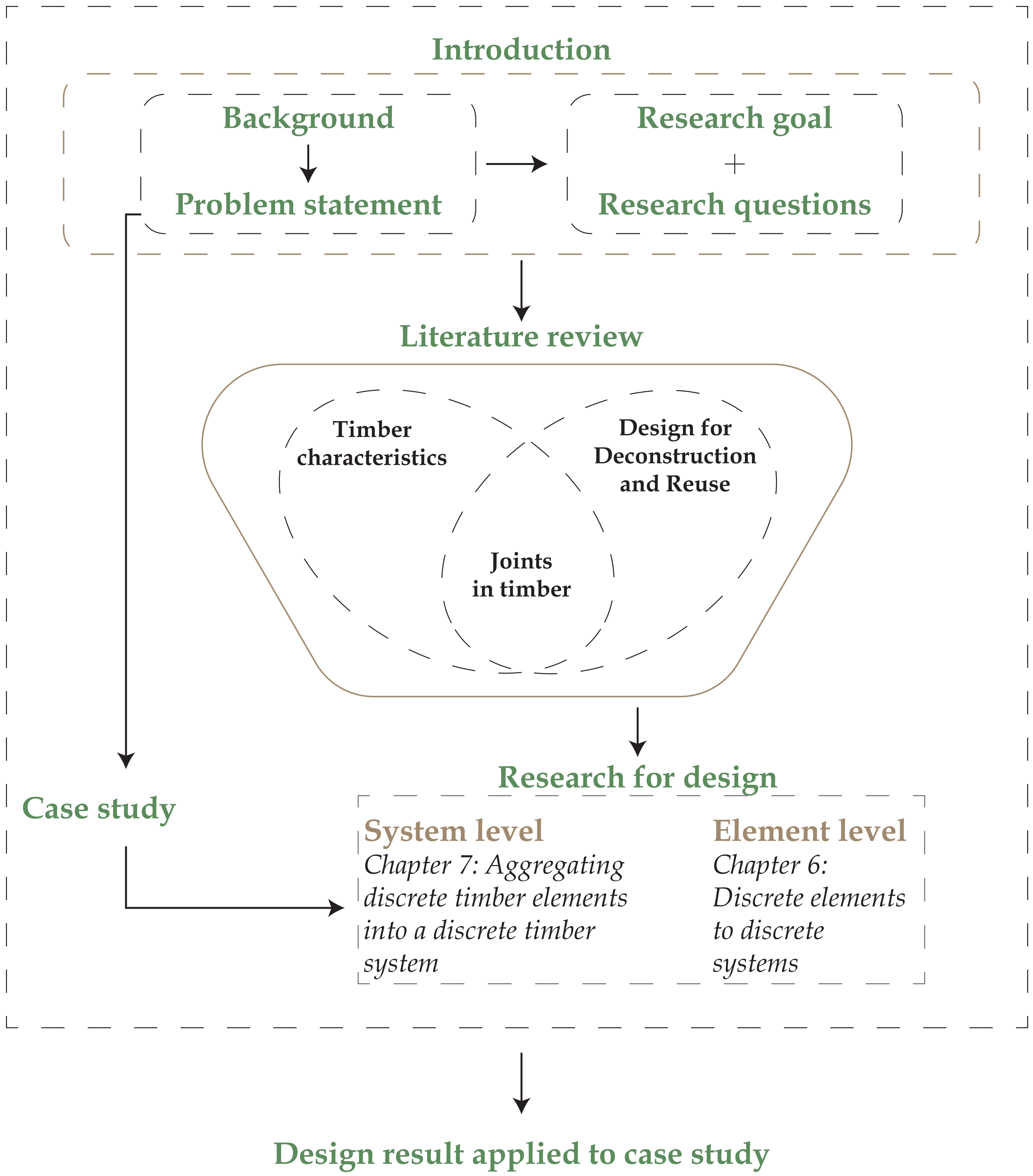

Figure 1.3: Research methodology (own work, 2024)

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2. Characteristics of timber

Trees and their resulting produce of, amongst others, timber and wood have been known to human kind as a material to construct/build things with for ages. It is a naturally occurring and organic material that is still being used in current day. The different names occurring such as wood and timber can also indicate different use of the material. The definitions of timber and wood are understood and from here on out used as; timber is used for structural or building purposes and wood as a base for 'making' things (consumer products) and for fuel, derived from the following definitions:

Timber:
"wood from trees that is used for building, or trees grown for this use" and "wood that has been specially produced for use in the building industry" (Cambridge University Press, n.d.-a).

Wood:
"a hard substance that forms the branches and trunk of trees and can be used as a building material, for making things or as a fuel" (Cambridge University Press, n.d.-b).

Figure 2.1: The Horyu Ji Pagoda in Japan (Cartwright, 2017a)

2.1 History of timber

The earliest findings of wood go back to 8 wooden spears of 400.000 years ago being found in a mine in north-western Germany (Radkau, 2012), today wood is still used in instruments, furniture, and things such as toothpicks. Timber is estimated to date back between 300.000 to even 1.000.000 years ago. This was in the form of primitive shelters built by the 'Peking man' in current day China. These buildings were nothing like how we know buildings today, but more shelters composed of branches and reed. There was however some sort of structural and fire-safety mechanism taken into account in these shel-ters. Timber used for load-bearing elements goes as far back as the Neolithic (7.000 – 1.700 BCE), where different societies have used timber in various ways, but each in such a way that they were using the characteristics of timber to their advantage (Bukauskas et al., 2019). Looking more to buildings as we know them today, we find the building that is believed to be the oldest and still standing timber building in Japan. The Horyu-ji Buddhist temple , see Figure 2.1, is a pagoda of five storeys high, and assumed to have been built around the 8th or even 7th century (Cartwright, 2017; Smith & Snow, 2008).

The current growth in interest and use of timber as a construction material is partially due to innovations made in the previous century. Novel timber products, such as cross-laminated timber, with the similar load-bearing capacity as concrete, but with a significantly lower material mass make timber for an attractive alternative. Additionally, this opens up the use for timber to construct higher buildings (Hough, 2019; Prins & van Roeden, 2021).

A study by Haisma et al., (2023) on construction material use in Europe found that, in nine by them defined building typologies, not for a single building typology wood accounts for more than 10 percent of the used material. Among these typologies are single, and multi-family homes. Which in the current situation consist of mainly non-biobased material such as concrete and masonry. To turn this into more sustainable family homes, the use of biobased materials within context is explored and the concrete floors and walls can be replaced by timber frame or CLT, the masonry façade and insulation can also be substituted by a biobased alternative, see Figure 2.2. This proves that there are possibilities for using biobased materials for dwellings, but are also applicable to other building typologies. However, to be able to deliver the materials needed to meet the demand for engineered wood products in 2030 it is expected that the EWP production needs to increase by factor of 5.

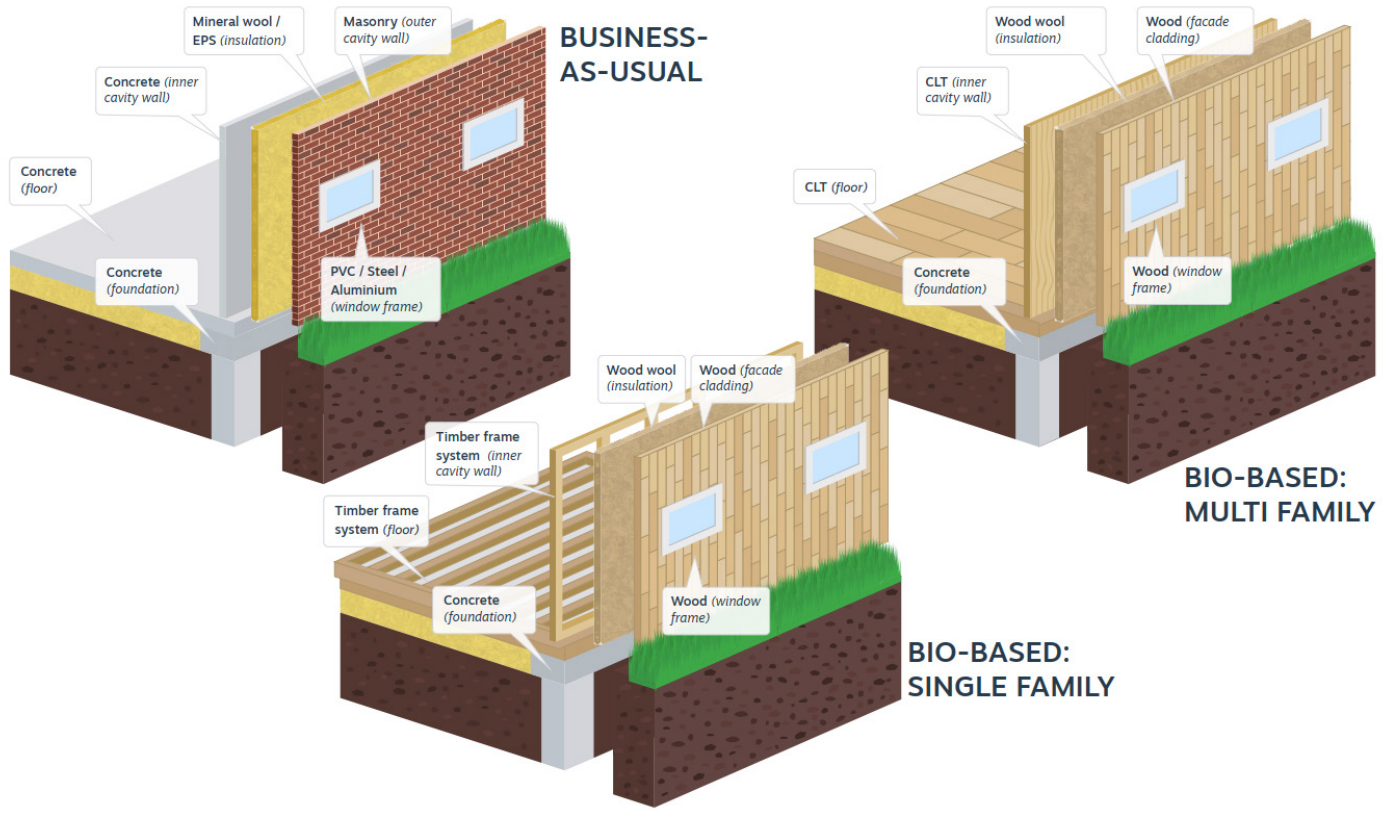


Figure 2.2: Transition from current dwellings to biobased dwellings (Haisma et al., 2023)

2.2 Harvesting of timber

Timber is a naturally grown, organic material, which can take from 25 years for softwoods to 100 years for hardwoods to harvest (Hart et al., 2021; Porteous & Kermani, 2007). In 2020, around 31 percent of the total global land area was covered by forests, this comes down to around 4.1 billion hectares (Global Forest Area 2020, n.d.), Figure 2.3 shows the division of forest area in percentage per continent.

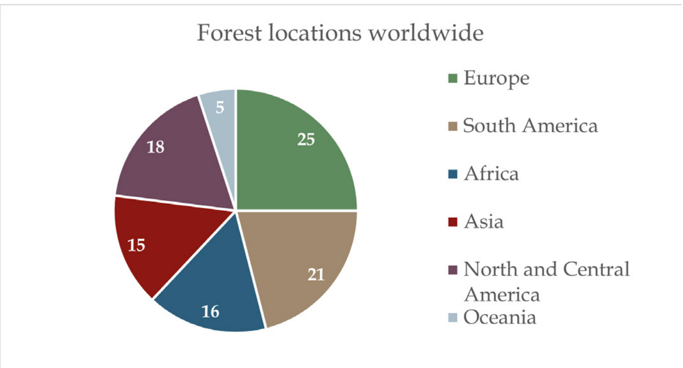


Figure 2.3: Percentage of total forest area per continent (adapted from Crocetti et al., 2016)

Remarkably, not even half of the global wood is used for industrial purposes such as paper, pulp and wood products, the other part (more than half) is used as fuel. Using wood for fuel mainly occurs in developing countries. A large chunk of the sawn timber is produced in Sweden, Russia, Canada, USA and Germany. Of these five countries, Sweden, Russia and Canada are main global exporters of timber worldwide (Crocetti et al., 2016).

Logically, just like the use of timber, the harvesting of timber has been known to be part of human activity for ages. However, it is the way of harvesting that has undergone some severe changes to, amongst others, better preserve the global forests. These changes are related to novel technologies such as mechanization of work, but also sustainable forestry management by agencies such as: "Sustainable Forestry Initiative (SFI), Forest Stewardship Council (FSC), and Program for the Endorsement of Forest Certification (PEFC)" (Asiz, 2023). The presence of such agencies is important to enforce sustainable forestry, which for example ensures that forest products meet certain sustainability criteria, but they also ensure that the percentage of global forests will not go in deficit, because forest resources are crucial for multiple aspects, such as the following seven aspects named in the Montreal Process by Siry et al., (2005):

- conservation of biological diversity;
- maintenance of productive capacity of productive ecosystems;
- maintenance of forest ecosystem health and vitality;
- conservation and maintenance of soil and water resources;
- maintenance of forest contribution to carbon cycles;
- maintenance and enhancement of long-term socio-economic benefits to meet the needs of societies and;
- development of legal, institutional and economic framework for forest conservation and sustainable management

The way trees are harvested depend on multiple factors such as, species composition, tree size, forest density, silvicultural treatment, site conditions, and the respective country's economic condition. When looking at Europe, Ireland, the UK and Scandinavian countries harvest almost fully (100%) mechanical, whereas this happens significantly less in Eastern European countries. Most commonly used harvesting methods are cut-to-length (CTL) and tree-length (TL). A main difference between the two is the length of the harvested parts. CTL harvesting is a more time consuming method, and therefore we see TL harvesting being applied more in situations where efficiency and speed are more sought after aspects (Moskalik et al., 2017). After harvesting, trees are cut into pieces, generally according to one of the cutting patterns shown in Figure 2.4 (Porteous & Kermani, 2007).

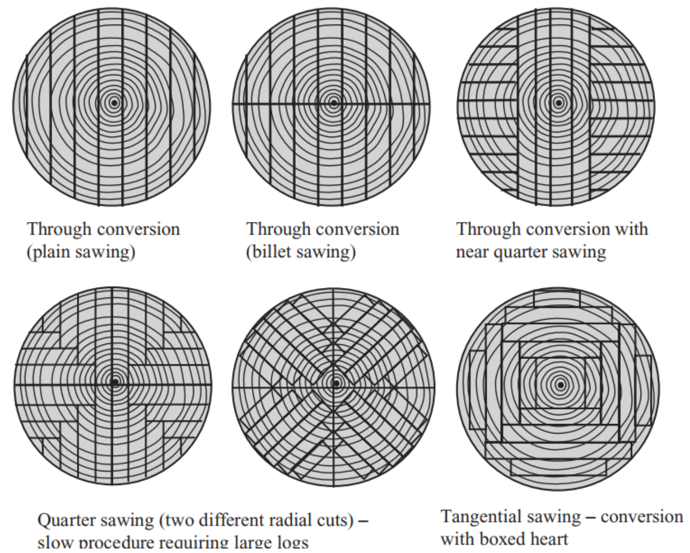


Figure 2.4: Typical sawing patterns (Porteous & Kermani, 2007)

2.3 Hardwood and softwood

Trees fall in either of the following two categories: hardwood species or softwood species. The categorization does not refer to the tree being hard or soft - but to the botanical origin. Hardwood trees commonly are deciduous (trees with broad leaves) and softwood trees commonly evergreen with needle leaves. The main differences between hardwood and softwoods are their growth rate, costliness, strength (durability). The growth rate for hardwood is way slower and can take more than 100 years to mature, softwood on the other hand mature earlier and can be felled in as fast as 30 years. The fibres within hardwood trees are much denser than softwood trees, which makes hardwood trees heavier and harder compared to softwood trees. When looking at the various application of trees, this makes hardwood trees the more durable (i.e. weather and fire resistance) of the two types, and hardwood trees also have higher strength characteristics. However, hardwood is more expensive than softwood, also softwood can more easily be processed (with light duty tools). Additionally, softwoods have a larger percentage of usable stem wood (see Figure 2.5) (Krackler et al., 2011; Porteous & Kermani, 2007; Urmila Mou, n.d.).

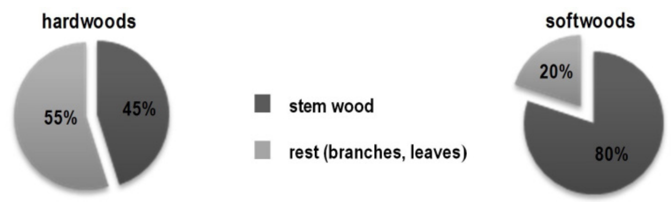


Figure 2.5: Usable percentage of stem wood (Krackler et al., 2011)

2.4 Structural characteristics

The anisotropic nature of timber is one of the aspects in which it differs from materials such as concrete or steel. The anisotropic characteristic means timber has different properties, depending on the direction in which the stress is applied compared to the direction of the grain/fibres. The natural characteristic of timber makes it impossible to control the variation in properties of the material, this variation can be significant to the point where more knowledge is needed to safely use an element in a construction. Norway Spruce can have a bending strength of 90MPa or 10 MPa. Timber is therefore categorized into different strength classes, this is done by either visual or machine strength grading the bending strength, density and its young modulus. Machine strength grading is the more accurate and preferred method because visual strength grading relies on a human grader determining within a few seconds the impact of certain defects. Although visual strength grading was used until around the 1950s, machine strength grading is a good replacement of grading in a non-destructive manner (Crocetti et al., 2016).

As a result of the structure of wood, the material has different properties depending on the directions on the x,- y,- and z-axis from the centre of a tree, making it an anisotropic material. The highest strength is found parallel to the direction of the grain, and the lowest strength is perpendicular to the grain.

Tension in wood elements
If wood elements are subject to tensile force, the material shows a brittle failure mode. Tensile loads applied perpendicular to the grain is how wood is at its weakest (because this pulls the fibres within wood apart from each other), with generally acceptable loads of only up to 2 N/mm². The young's modulus is also up to 30 times smaller perpendicular to the grain.

Compression in wood elements
Wood reacts differently under applied compressive loads. It is especially strong with forces parallel to the grain because this is the same direction as the fibres within wood, and it so happens to be that these fibres can resist high forces under axial loading. However, if the forces become too high these fibres will buckle. Generally wood can resist compressive forces of 80MPa parallel to the grain. Whereas under perpendicular loading the wood can generally only resist forces of 3 to 5 MPa (Crocetti et al., 2016).

2.5 Fire safety characteristics

Fire safety plays a big role in any building, but intuitively even more so in buildings with timber structures, as it is a combustible material. While timber is one of the oldest building materials, it is only recently experiencing an uprise again after quite a dip. This dip can be explained by the invention and growth of steel and reinforced concrete at structural materials, but also in part due to safety precautions and worries, in the United States for example following the great Chicago fire in 1871 (Faulstick, 2019).

However, also since the decline and fire safety issues with timber buildings there have been significant changes to building codes to ensure safety in sawn timber elements. In addition, innovations in timber engineering leading to new products such as Cross Laminated Timber (CLT), come with a whole new way of ensuring safety during fire. In the event of a fire, the different layers of CLT form a charring layer on the surface, but the inner parts stay protected and keep their structural integrity to ensure sufficient escape time. If wood would burn however, it happens at a constant rate which means that it can be calculated very precisely how long a structural element remains strong enough (Hough, 2019; Prins & van Roeden, 2021).

2.6 Moisture

The properties of timber and wood can highly vary between species but also within the same species, growing location, but is also highly influenced by water and the moisture content within the wood or timber. Difference in moisture content means that the weight, the strength, stability, fire and pest resistance and consistency all vary. The moisture content (ratio 'u' as percentage) is expressed as the weight of water divided by the dry weight of the wood, and the dryer the wood (thus lower moisture content) the higher the strength and stiffness. Table 2.1 nammes three different moisture contents.

Table 2.1: Moisture contents in wood (adapted from Steiger, 2017).

State	Moisture content
Green	> 30% wood moisture
Semi-dry	Between 20% and 30% wood moisture
Dry	< 20% wood moisture

As said before, with a lower moisture content the wood will be stronger and stiffer. Between 8 and 20% moisture content the following changes in strength occur with a Δ moisture content of 1%:

Table 2.2: Approximate change in mechancial properties of clear wood for a 1% change in moisture content (adapted from Crocetti et al., 2016).

Property	Change (%)
Compressions strength parallel to the fibre direction	5
Compression strength perpendicular to the fibre direction	5
Bending strength parallel to the fibre direction	4
Tension strength parallel to the fibre direction	2,5
Tension strength perpendicular to the fibre direction	2
Shear strength parallel to the fibre direction	3
Modulus of elasticity parallel to the fibre direction	1,5

As a result of more or less moisture, the material shows swelling and shrinking respectively, which is also called 'timber movement'. Swelling and shrinking happens in different factors, which are depending on the direction to the grain. Typically the following values represent the shrinkage for each direction as a result of Δ moisture content of 1%: tangential 0,0030; radial 0,0015; longitudinal 0,0001.

As a result of varying moisture contents and subsequent shrinking and/or swelling the wood can experience a few different geometrical changes which make it more difficult to use. It is therefore important to build with timber when it is in the 'dry' state, or to match the moisture content to the moisture level at the building location. This helps preventing sudden shrinking or swelling when the material is moved to the construction site and when the system is connected to other building parts (Crocetti et al., 2016; Steiger, 2017).

2.7 Engineered timber

Recent innovations have improved usability of timber in more structural complex and larger spanning constructions. Before that, the size of structural timber was directly related to the size of the trees available. The size of readily available single timber pieces has gone from 150 x 450 mm x 20m to 75 x 225mm x 5m, and the larger pieces now are uncommon and expensive. The successful innovation of connecting multiple smaller pieces of timber to each other makes it possible to form these large timber elements nonetheless, these large timber aggregations are named engineered wood products (EWP(s)). This not only allows for use of structural timber in more complex and larger spanning constructions, but also helps diminishing the effects of flaws, such as knots, that can occur in a single piece of timber and result in more consistent material properties. The consistency in material properties is part of the reason for the increased loadbearing capacity of EWPs (Blaß & Sandhaas, 2017). There are a handful of different EWPs that can be categorized into four groups based on the type of timber that is used to make the EWP'. There are EWPs based on 1) sawn timber boards, 2) fibres, chips or strands, 3) on veneers, and a fourth one is built up structures. Many of the EWPs, in beam and panel forms, were invented in the 20th century, in North America this originally started because the lack of sizeable and strong timber elements has led to using new tree species, trees with a smaller diameter, and lower quality timber.

Sawn timber board to create engineered wood products

A distinguishment in engineered wood products from timber boards can be made based on the direction to grain of the separate elements in the product. This method knows glued laminated timber and cross-laminated timber.

Glued laminated timber (glulam)
Glued laminated timber (glulam) is a product in which all timber elements are arranged parallel to the grain and glued together by applying an adhesive on the surface in contact with another element. Using timber in such away provides some useful advantages for construction purposes. Where single timber elements have a large variety in properties due to its fibres being cut while preparing the material, homogenisation of the material, which is happening in creating EWPs, improves and generalizes the properties. In a single timber beam with a knot, the knot has significant influence on the cross section, however with a glulam element the cross section size is increased and therefore the effect of the same knot will be significantly less (Blaß & Sandhaas, 2017; Crocetti et al., 2016). By using glulam instead of regular timber, other flaws such as shrinkage cracks and pith are also eliminated, see Figure 2.7.

Glulam elements consist of at least four layer laminated together, the combined layers can have different strength levels to form different aggregations. In symmetrical and asymmetrical aggregations elements with a higher strength are used for the top and bottom layers in a symmetrical aggregation and for the bottom layers in an asymmetrical aggregation, a third option is a homogenous glulam element in

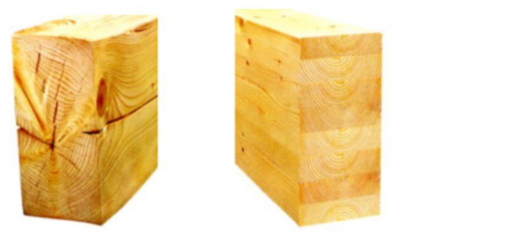


Figure 2.7: Squared timber cross-section with cracks compared to a glulam cross-section (Blaß & Sandhaas, 2017)

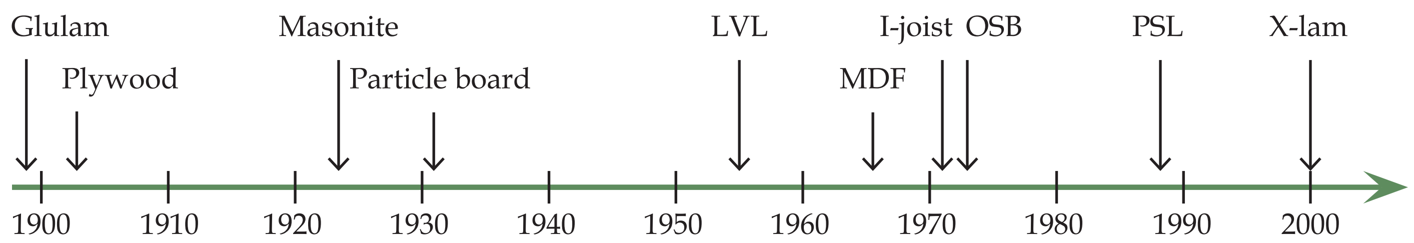


Figure 2.6: Timeline of development of various Engineered Wood Products (Design of timber structures-1, 2016). LVL - Laminated Veneer Lumber, MDF - Medium Density Fibreboard, OSB - Oriented Strand Board, PSL - Parallel Strand Lumber, X-lam - Cross-Laminated Timber

which all the layers have the same strength, the three options are shown in Figure 2.8. In symmetrical aggregations the top and bottom, or high strength, parts each are at least larger than or equal to 17% of the total height of the glulam element (Blaß & Sandhaas, 2017; Crocetti et al., 2016).

Cross-laminated timber (CLT)
Where the elements in glulam are arranged parallel to the grain, the elements in cross-laminated timber are placed perpendicular to the layers above and below. The elements can be glued to each other like glulam, but mechanical joining is also possible with dowels or nails. The top and bottom layers of CLT are always placed in the same direction, by following that logic we find that CLT is always using an uneven number of layers, starting at 3 layers up until at least 7 layers, with the possibility of more layers. There is the freedom to have partially filled CLT panels where there are some spaces left open in the inner layers. The individual layers are not only varying in their direction, but can also vary in how thick they are. Placing layers perpendicular to each other reduces variance in the properties, ensures a more isotropic nature instead of anisotropic, and improves dimensional stability (Blaß & Sandhaas, 2017; Crocetti et al., 2016).

The fact that the layering of elements in CLT starts a 3 but could go to at least 7 means that the thickness of CLT panels also has wide range (about 60 to 500mm).

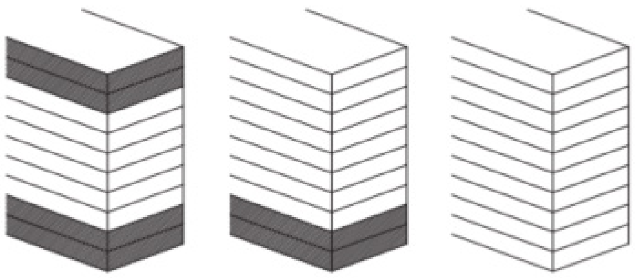


Figure 2.8: Symmetrical and asymmetrical combined and homogenous glulam (Blaß & Sandhaas, 2017)

The length of a single CLT panel could go up to 24m and they can be 3m wide. CLT panels are often used for loadbearing walls and as stiff floor elements, and since they can easily be prefabricated (including window and door holes and applied insulation) they are known to be a fast construction method (Blaß & Sandhaas, 2017; Crocetti et al., 2016).

Fibres, chips or strand to create engineered wood products

Smaller timber parts such as fibres, chips and strands can also be used to create EWPs. By glueing these together, different sized panels can be created. The way these panels are processed generally cause for higher density in the outer surfaces and lower density in the middle part of the panels. The strength is mainly determined by the amount and type of adhesive, but also for a small part by the way of producing.

Chip, particle or fibre board
Chip-and particleboards are relatively similar to each other. The main difference being the size of the single elements in the board. Chipboards are generally made from chips smaller than the wooden strands used in OSB. Particleboards are composed of saw dust together with adhesives. The 'mixture' of wood chips or particles and adhesives is then pressed together and after that have a finishing touch process to finalize the boards.

Within fibreboards however are some differences in the production process. There are variations for producing fibreboards in which more or less adhesives and chemicals are used. For example, wet production of fibreboards have minimal to no adhesives added, and supports largely on natural bonding between fibres. Opposite of that is a dry production method for fibreboards, which uses a significant amount of adhesive (Blaß & Sandhaas, 2017).

Oriented strand board (OSB)
OSB is easily the most common used panel for structural purposes. OSB is produced by combining longitudinal wood strands of about 0.8x13x100mm with adhesive, with a ratio of 95% wood strands and 5% adhesive the mixture is then exposed to heat and pressurized into a panel. Ideally, the strands in the upper layer are placed parallel to the production direction, and then by placing the strands in the middle layers perpendicular to the production direction or at random the OSB will show different properties in the different directions. OSB panel sizes vary largely, but frequently used are panels sized 1.2 by 2.4m with a thickness between 6 and 25mm. It is possible to have OSB panels up to a length of 25m, a width of 3m and thickness of 75mm (Crocetti et al., 2016).

Veneers to create engineered wood products

Veneer is a relatively thin layer of timber. The process of getting veneer is to remove bark from the logs, steaming the residue, and then peeling of layers of the timber in rotary motion, as shown in Figure 2.9. The thin layers coming off of this need to be dried in order to reduce the moisture content to the range of 6-12%, and then adhesives and hot-pressing are used to glue the veneers together into differently sized structural elements, the production process is shown in figure x (Blaß & Sandhaas, 2017; Crocetti et al., 2016). Plywood and laminated veneer lumber are the most commons veneer EWPs, the difference between these two are comparable to the differences between glulam and CLT.

Laminated veneer lumber (LVL)
LVL consists of veneer layers, glued together with adhesive into panels of 20-90mm thick, and can reach sizes of up to 3 by 24 meters. The veneer layers are all placed with the fibre direction the same way, this direction generally the long direction of the end product. There are however options to create LVL with a higher stiffness throughout the panel, to reach this some layers are placed perpendicular to the fibre direction. As with other EWPs where adhesive is used to glue different elements to each other, in LVL it also helps to create elements with a more consistent strength (Crocetti et al., 2016).

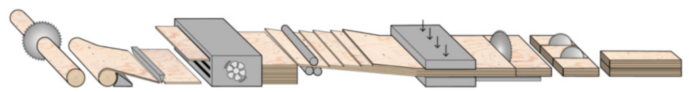


Figure 2.9: Production method for veneered materials (Blaß & Sandhaas, 2017)

Plywood
Where with LVL the layers are glued all in the same direction, for plywood the veneer layers are placed perpendicular to each other. Earlier the comparison between LVL and plywood & glulam and CLT has been made already, plywood is in this comparison similar to CLT. Not only with regards to the placing of the veneer layers, but also the logic that the top and bottom layer are always in the same direction, resulting plywood always having an uneven number of layers. Plywood generally comes in sizes 1200 by 2400mm or 1220 by 2440mm, and thickness between 12 and 24mm (Crocetti et al., 2016).

2.8 Conclusion

Timber is one of the oldest structural materials known to mankind. Initially used for tools but eventually found its way into building structures. Timber is an organic material and comes from trees, before it can be used it needs to be harvested, historically this was done manually, but nowadays it is almost done completely mechanically. After harvesting the trees are processed by cutting them into pieces, generally according to a few standard cutting patterns.

There are numerous different tree species, but they all fall into two categories of hardwood or softwood. This categorization is based on the botanical origin, and the main difference between the categories is the growth rate, costliness, and strength (durability).

Timber is an anisotropic material, meaning that it has different strength properties depending on the direction of the applied load. Because of this characteristic it is not possible to control the variation in properties and therefore visual and/or machines are used to grade the strength properties of timber. Timber is strongest when the load is applied parallel to the grain, and in tension.

While wood is widely used for its combustible characteristics, to make fires for warmth or barbecuing, that does however not make timber use in building unsafe. Quite the opposite, since exposing timber to fire causes the outer layer to char and directly act as a protective layer for the inside. Besides fire, moisture is also an important factor in timber, as this influences the weight, strengths, stability, fire and pest resistance of timber.

Where initially only solid timber was used, somewhere around the start of the 1900s innovations in timber have introduced new products in the market under the umbrella name 'engineered timber'.

3. Design for Deconstruction and Reuse

3.1 Design philosophy

The use of timber as a construction material is often supported by the carbon sequestering nature of timber, and while it is true that the photosynthesis process in trees captures CO2 and gives timber an initial advantages compared to construction materials such as steel and concrete. It is also true that timber will (partially) return the sequestered CO2 back into the atmosphere upon various end-of-life scenarios such as decaying and timber incineration. It is therefore important to manage the initially captured CO2 well at the end-of-life, though options such as reusing, recycling, biomass energy extraction, and anaerobic burial. In the case of timber buildings the best option is to disassemble, adapt and reuse at the end-of-life (Hough, 2019).

Several end-of-life scenarios are e.g. design for deconstruction, design for adaptability, and design for reuse, and fall in general within the design philosophy of 'Design for Deconstruction and Reuse' or DfDR (Hough, 2019). Russell & Moffatt (2001) explain that DfDr is: "the design of the buildings so that the parts are easily dismantled and separated from each other for reuse or recycling". For DfDr it is important to take into account that the parts may not break upon reparation or dismantling, and that reusing is always preferred over recycling, in most situations the amount of environmental gain is directly related to the amount of reuse.

3.2 Joints

In designing a building that can be dismantled, a general rule is; the difficulty of assembling is directly related to the difficulty of disassembling, so an easy assembly process is most likely related to a (relatively) easy disassembly process. One of the most important things in DfD is that components need to be sorted by their specific recycle requirements and service life, e.g. installations and loadbearing structures fall in different component categories. Then, these need to be clearly separated so that each category can be dismantled and reused separately, there is a rule of thumb here that the shorter the categories' service life, the easier its dismantling should be.

When focussing on joints there are some aspects to take into account which will improve the potential to disassemble the joints. Joints should be dismantled in a non-destructive manner, they should be easy to access, the number of unique connections and fasteners in the joints should be minimized, and adhesives should be replaced with mechanical fasteners (Hradil et al., 2014). Hradil et al. (2014) analysed different connections in timber structures and their suitability in DfD, see Table 3.1.

There is currently not an Eurocode or official regulation regarding design for disassembly. However, Laasonen & Pajunen (2023) conducted a small meta-analysis comparing DfD principles/ criteria according to the International Organization for Standardization (ISO) with other studies, and also collecting criteria not named by the ISO.

A study by Huuhka et al. (2018) defines glulam, solid wood panels, and LVL as suitable structures for disassembly. Another important piece of information from this study is that the location of the joint is influential on the strength and modifiability of the timber. It is preferable to reuse the join because that ensures similar structural properties. If the joint is damaged and needs to be sawn off, recreating a similar joint is the best method to ensure similar structural properties.

Possible challenges

Besides the positive aspects of DfDR, there are also some challenges related to designing this way. There are some events that can cause permanent deformations and affect the potential to de-construct. These deformations generally occur in cyclical loading and deform joints that only have a ductile behaviour in the first or earlier cycles. These deformations are specifically hindering cyclic loading events such as seismic loading or wind loads. Additionally, material effects on fasteners, such as corrosion, hinder the actual de-constructing by making it difficult to i.e. remove bolts. The re-usability of elements can

Table 3.1: Suitability of different connections in timber structures (Hradil et al., 2014)

Connections	Suitability	Note
Glued connections	Not suitable	Cannot be separated without damaging the elements.
Carpentry joints	Sometimes suitable	Notches can cause stress concentration if the elements are used in different configuration
Nails, staples	Sometimes suitable	Fail in bending, and are therefore difficult to remove without damaging the element.
Screws	Mostly suitable	The same connector is not as effective in the same hole.
Bolts, dowels	Suitable	The hole and the cracks should be checked.

Table 3.2: Comparison of ISO 20887 criteria with other studies (adapted from Laasonen & Pajunen, 2023)

Criteria	ISO 20888	Guy and Ciarimbol	Pozzi	Casagrande et al.	Yan et al.	Piccardio and Huges
Ease of access to components	x	x	x	-	-	-
Speed	-	-	-	x	-	-
Visibility of connection	x	x	-	-	x	-
Independence	x	*	*	-	-	x
Degree of freedom	-	-	x	-	-	-
Stiffness of connection	x	-	-	x	x	-
Strength and ductility of connection	x	-	-	x	x	-
Avoidance of unnecessary treatment	x	-	*	-	-	-
Finishes	x	-	x	-	-	-
Minimize or eliminate chemical connections	x	x	-	-	-	*
Supporting reuse circular economy business models	x	-	*	-	*	-
Reusability	x	-	x	-	x	-
Material selection	x	x	-	-	-	-
Simplicity	x	x	*	x	-	-
Joint assembly	x	*	-	-	-	x
Number of elements	x	*	x	-	-	-
Element complexity	x	-	x	-	-	-
Standardization	x	*	-	-	x	x
Prefabrication	x	-	x	x	x	x
Use bolted and screwed connections	-	x	*	*	*	*
Interchangeability	*	x	-	-	-	-
Safety of disassembly	x	x	-	-	-	-
Documentation	x	x	-	-	-	-

Table 3.3: Criteria outside ISO 20887, and their mentioning in various sources (adapted from Laasonen & Pajunen, 2023)

Criteria	ISO 20888	Guy and Ciarimbol	Pozzi	Casagrande et al.	Yan et al.	Piccardio and Huges
Stiffness of structure	-	-	x	x	-	-
Ease of assembly	-	-	x	x	x	-
Ease of disassembly	-	*	x	-	-	-
Weight	-	-	-	x	-	-
Seperate mechanical, electrical, and plumbing systems	-	x	-	-	-	*
Design to the worker and labor of seperation	-	x	-	-	-	-
Costs	-	-	x	-	-	x
End-of-waste cycle	-	-	x	-	-	-

"-" means that the criterion is not mentioned, "*" means that the criterion is mentioned but not used, "x" means that the criterion is mentioned and used

Table 3.4: Modified load modification factors for recycled timber (adapted from Ottenhaus et al., 2023)

Load duration	Original modification factor	New modification factor
Short term loads (<5 days)	6% reduction	No reduction
Service loads (<5 months)	20% reduction	2% reduction
Permanent loads (>5 months)	43% reduction	10% reduction

be affected by various aspects as well. Permanent deformations can happen to the joint, affecting the ability to de-construct, but if permanent deformations happen to the fastener hole it can be difficult to reuse the fastener hole in a new aggregation. This can also happen to metal plate connections where the connection holes are deformed due to loading, making the plates non usable due to exceeding the acceptable tolerances. There is also the case of metal fasteners that have exceeded the elastic limit, is it then impossible to determine the capacity for future reuse. For both of the previously named implications the characteristics are difficult to determine, affecting the re-usability (Ottenhaus et al., 2023).

Besides the actual connections between the elements, the elements themselves are also influential of the de-construct and reuse potential, as the load history affects the timber elements too, i.e. decreasing the load-bearing capacity of the element. Not a lot of research is available to counter this, but an Australian industry standard for recycled timber has some guidelines for grading recycled hardwood timber (see Table 3.4). They indicate that the elastic properties of timber remain unaffected by previous long term loading (Ottenhaus et al., 2023). For the strength of timber, increased modification factors should account for the long-term loading in reused timber, the guidelines in Table 3.4 have been set up as a result of previous long term loading of the reusable timber elements.

3.3 Kit of Parts system in DfD

Howe et al. (1999) defines kit-of-parts as the following: "a collection of discrete building components that are pre-engineered and designed to be assembled in a variety of ways to define a finished building". Advantages of using a kit-of-parts system include the ease of manufacturing and ability to use certain constraints for this, such as size for convenience in handling or shipping. A kit-of-parts system is closely comparable to LEGO, but then a few chosen bricks to construct the whole building with. It is also important to have standardized connections, this way the form itself has more freedom. The selected kit-of-parts can be prefabricated, but compared to 'regular' prefabrication, kit-of-parts can also be de-constructed and reused. Kit-of-parts systems can generally be categorized into four different categories, which are as follows.

Module-based
Module based kit-of-part systems are complete, pre-assembled blocks that are set into place on the building site. The size of one module can go up to a single building unit.

Panel-based
Panel-based kit-of-parts are among the earliest prefabricated systems. Panel-based systems are an aggregation of structural, and façade and floor cladding components in one. For panel-based systems to work properly, connections/joints need to have the ability to be deconstructed. The panel-based ones are smaller scale than module-based.

Joint-based
Joint-based kit-of-parts look at single prefabricated elements. There are more and less advanced solutions within this category. For the more advanced ones you will find clearer distinctions between element and connection. Furthermore, the connection needs to be designed so that it improves speed of assembly and disassembly.

Special types
In the special types of kit-of-part systems are inflatable and deployable structures. These systems are generally perfectly balanced, and upon removing a single element the system could fail.

3.4 Conclusion

Timber is initially already a more sustainable material than for example, steel or concrete, as a result of the carbon sequestering that happens in timber. However to keep this advantage over the other materials, an improved end-of-life scenario (to decay or incineration) must be followed - such as reusing or recycling. This is however not applicable to all timber structures because it was not designed with the de-construction in mind. There are however several end-of-life scenarios, such as Design for Deconstruction and Reuse, that focus on constructing so that it can be easily de-constructed. An important part of the construction for this are the joints, and while there are no Eurocode guidelines for this, there is some research that compares criteria for easier de-construction.

4. Connections in timber

Multiple aspects need to be taken into account and are important for a successful structural design. This consists of material decisions and compositions, but also calculations of the materials to verify the structural feasibility and meet building code requirements. These materials however need to be kept in place, and the forces need to be transferred from one material to another material, this is where the necessity of joints come into play. Joints are important in every material, but for timber there is an extra attention point, namely that timber elements generally have a higher load bearing capacity than the joints connecting them to each other. Additional factors influencing the selection process of timber joints are the production process, stakeholder preferences, erection process, aesthetic of the joint, and the costs (Blaß & Sandhaas, 2017). There are three primary types of joints used timber construction:

1. connections with metal fasteners;
2. dry timber joints
3. glued joints.

4.1 Connections with metal fasteners

Metal fasteners can be categorised into groups based on how they transfer load between members. The two most commonly appearing types are dowel-type fasteners, and surface-type fasteners.

Dowel-type fasteners

In dowel-type fasteners tensile and bending stresses occur in the fastener under stress, and shear stress occurs in the wood. Main fasteners in this group are: dowels, nails, bolts, staples, threaded rods, and screws.

Nails
Most commonly used in timber construction are nails, it is without surprise that they come in many different sizes, materials and shapes. The sizes generally fall within the range of 2 to 8mm for the diameter and 40 to 200mm for the length. A smooth-shank nail with circular cross-section is the most commonly used nail, these nails have a minimum tensile strength of 600 N/mm2. Loadbearing capacity of nails can be enhanced by some modifications, such as changing the smooth surface of the nail into rings or spirals. When using nailed joints it can be useful to pre-drill the holes so that timber won't split, predrilling is also handy in higher-density species (Blaß & Sandhaas, 2017).

Staples
Staples allow for rapid construction and are therefore a commonly used fastener in timber buildings. In its production process, staples will be reshaped under 90° angles, it is therefore useful, if not needed, that the staples are of a high-tensile and ductile steel. The steel grade in staples generally is significantly higher than that for nails, where nails have a minimum tensile strength of 600 N/mm2, staples have a minimum tensile strength of 800 N/mm2. A rule of thumb for stapled joints is that one staple is equivalent in loadbearing capacity to two similar diameter nails (Blaß & Sandhaas, 2017).

Bolts, dowels and threaded rods
Bolts, dowels and threaded rods are usually made of steel. Bolts also have a square or hexagonal head and corresponding diameter nuts to fasten it. Predrilled holes of the bolts' diameter + 1mm make it easier to insert the bolts. Bolts can have a negative effect on the aesthetic of the connection, in such case dowels or bolts that do no pop out of the connection can be used (Blaß & Sandhaas, 2017).

Screws
The most commonly used screws in timber construction are self-tapping screws, this type of screw tap their own threads when they are screwed into the material (Blaß & Sandhaas, 2017).

Surface-type fasteners

Examples of surface-type fasteners are toothed-plate connectors and split rings. For these type of fasteners, a large part of the force focusses on the surface area of the connector (Blaß & Sandhaas, 2017).

Connectors
In this type of surface fastener fall toothed-plate connectors, shear plates and split rings (Blaß & Sandhaas, 2017).

Punched metal plate fasteners
The punched metal plate fasteners, or nail plates are usually combined with screws or nails. In order to meet buildings codes, these punched metal plate fasteners come already pre-drilled. The 'nails' that are attached to the plate are bended, for this reason these plates are never thicker than 2mm. Using punched metal plate fasteners is relatively simple, the elements just need to be pushed into the wood.

The way that these plates are used is that they are pushed in the wood,

4.2 Dry timber joints

Carpentry joints, also called traditional timber joints, are used because of unavailability of steel dowels or other connectors. Sometimes a stronger timber species is used to make stabilising dowels. Making these joints the traditional way is done by hand and thus a very time consuming activity. These type of joints are not very well in transferring tension (Crocetti et al., 2016, p. 1)

Some of the commonly found (traditional) dry timber joints are as follows (Branco & Descamps, 2015).

- Lap joints, distinguishes between full lap joints and half lap joint. In the full lap joint no material is removed, the elements are stacked onto each other and the result is a joint with the thickness of both materials combined. The half lap joint has in both elements half of the height removed, so that the resulting joint is just flat, the height of the elements (see Figure 4.1).
- Scarf joint, these joints can connect two elemens end-to-end and are generally used to create elements which is not available in the desired length in one piece. This is the strongest dry timber joint (see Figure 4.2).
- Notched joints, these joints are used to make frame structures. In a bottom element a piece of the element is notched out so that another element can be placed here diagonally, if this is done symmetrically you have a frame (see Figure 4.3).
- Tenon and mortise joints, these joints usually exist between elements that form an ‘L’ or ‘T’ shape. It can be compared with a male female connection, where the tenon is the male part and the mortise is the female part. If the tenon is longer than the mortise it is used in, it can be locked into place with a pin or dowel (see Figure 4.4).

While dry timber joints are more common in historical buildings, they do offer some sustainability related advantages over for example metal fasteners. Dry timber joints generally have to deal with the same issues that ‘common timber’ has to deal with, in the form of natural defects, moisture and fire sensitivity and loss of structural capacity after use.

4.3 Glued joints

Glued joints have seen significant innovations come through in the last years. Creating durable adhesives with high stiffness and strength. These high strength adhesives are in timber generally used for producing Engineered timber products, but are also used to create glued joints. The advantage of glued joints over other joints is the improved aesthetics (the joint is not visible as it would be with e.g. bolts), stiffness of the joint is generally better, and possibly improved fire resistance. Disadvantages of glued joints lay their demand for quality control and the degrading characteristic.

Timber joints that make use of adhesives are different kinds of beams such as I-beams, composite panels such as Oriented Strand Board, but also in finger joints and scarf joints (Porteous & Kermani, 2007).

4.4 Joints for reversibility

With focus on the research question, and the need for the joints to be reversible the glued joints directly are not a good option. Within the category of metal fasteners there are some possibilities to create reversible joints. However the use of nails, staples or metal fastener plates does not work with reversible joints. Bolts or dowels can potentially be used to joint elements and later deconstruct them, however over time the pre-drilled hole for this might wear out. Some of the dry timber joints are suitable for reversible joints as well, however when choosing dry timber joints it is important to remember that these joints are not great in tension.

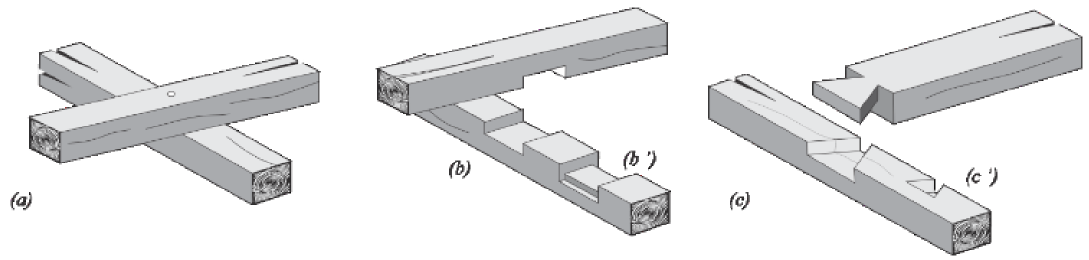


Figure 4.1: (a) full lap joint (b) half-lap joint (b’) cogged half-lap joint (c) through dovetail lap joint or (c’) wedged lap joint (Branco & Descamps, 2015)

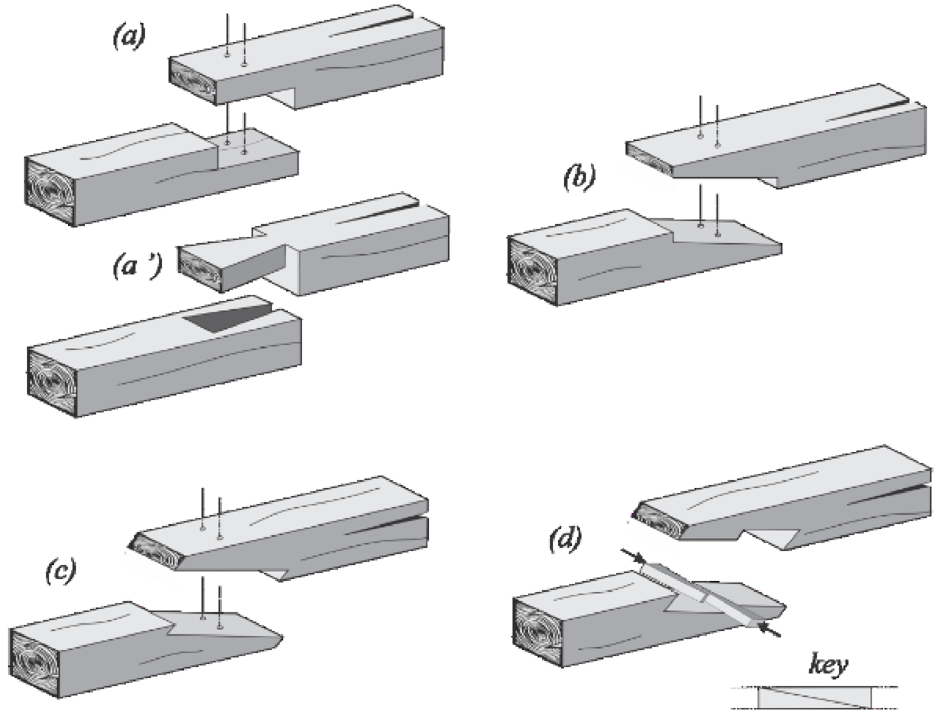


Figure 4.2: (a) common and simplest halved-scarf joint (or half-lap splice joint (a’) lapped dovetail scarf joint (b) scarf joint (c) scarf joint with under squinted ends (d) trait de jupiter (Branco & Descamps, 2015)

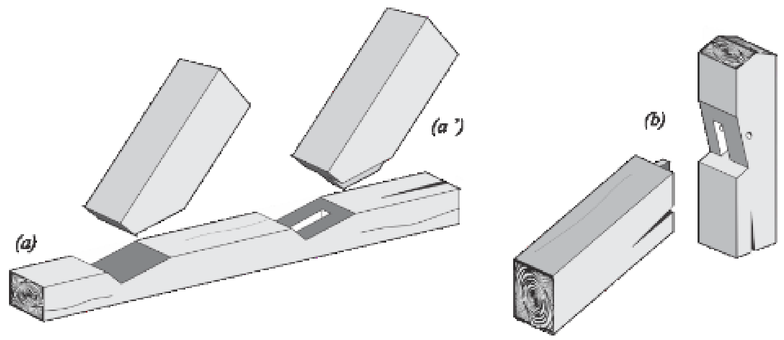


Figure 4.3: (a) notched joint between main rafters and tie beam (a’) a skewed tenon may be used to help in keeping all timber pieces co -planar (b) peak joint with a notched joint (main rafters and post (Branco & Descamps, 2015)

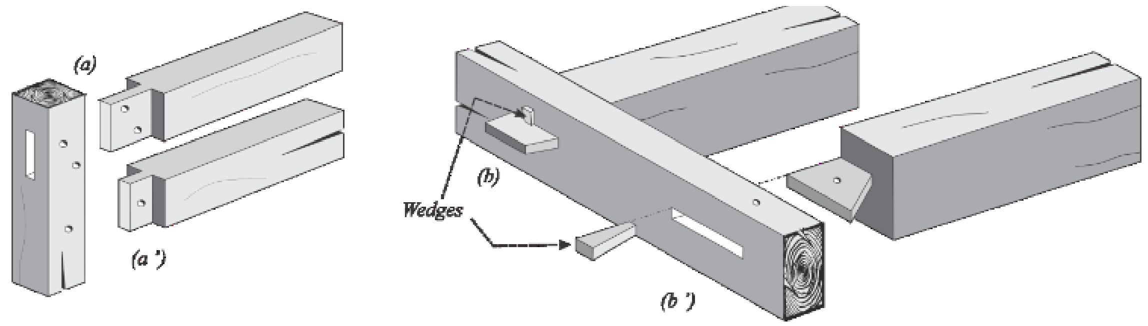


Figure 4.4: (a) through pinned mortise and tenon (a’) blind pinned mortise and tenon (b) through tenon with outside wedges (b’) wedged and pinned dovetail through mortise and tennon (Branco & Descamps, 2015)

5. Top-up on apartment blocks

Chapters 6 and 7, which come after this one, will cover research on discrete timber elements and discrete timber systems, connections between discrete timber elements, and computational calculation of the strength of the resulting discrete timber system.

For the research that will follow, some engineering decisions have to be made. These decisions are easier to make with a certain framework to follow. This framework will be set in the shape of a case study. Additionally, the final system will be used in a design for this case study. The selected case study will be a reference project (within a building typology) in designing a discrete timber system.

The ultimate goal of course is to see the system being used in a variety of different building typologies and for different problems, but for a start it aims to help in solving a problem closer to home, namely the housing shortage in the Netherlands.

5.1 Housing market in the Netherlands

In 2023, the housing shortage in the Netherlands increased from 3.9% to 4.8%, and next to that it is estimated that the amount of households will increase more than what was initially expected. A bottleneck in the flow of elderly people to smaller houses, household dilution*, and migration have a great influence on this rising statistic. Looking at the assignment ahead, this comes down to the Netherlands needing to increase its housing stock with 981.000 houses by 2030 (Koninkrijksrelaties, 2023b).

For a chance of reaching the goal of 981.000 houses in time the problem needs to be approached from all sides. That means not only looking at building new houses on newly prepared building lots, but also looking at the existing building and housing stock and utilising the opportunities that are presented here. Such as empty buildings that can be renovated into houses, splitting existing houses from one house into multiple independent houses, and topping up existing buildings with one or more new layers (Koninkrijksrelaties, 2023a). The latter is the method of focus for this research, aided by a statement from outgoing minister Hugo de Jonge that topping up is a great way to realize up to 100.000 new houses (Hannema, 2024).

** When the number of people per household decreases by part of the household moving out and thus forming its own, new household (BNR Webredactie, 2023).*

5.2 Topping up

Topping up is defined by Koninkrijksrelaties (2023c) as: "making houses by adding an additional layer or layers to existing buildings", Figure 5.1 is a schematic visual of different sizes and shapes of top ups. Figures 5.2 through 5.8 show the different possibilities for top ups in amongst others, size, shape, and number of floors. As said section 5.1, topping up can realize up to 100.000 new houses, there are however some prerequisites in topping up.

Firstly, the building on which the top up will be placed needs to have a flat roof, it is otherwise difficult to make an additional structure on the existing one. Secondly, in most top ups it is common to use the already existing load bearing structure for the top up too (Varamedia, 2021) - thus the load bearing structure needs to be strong enough to support an extra layer or layers and still meet the building codes.

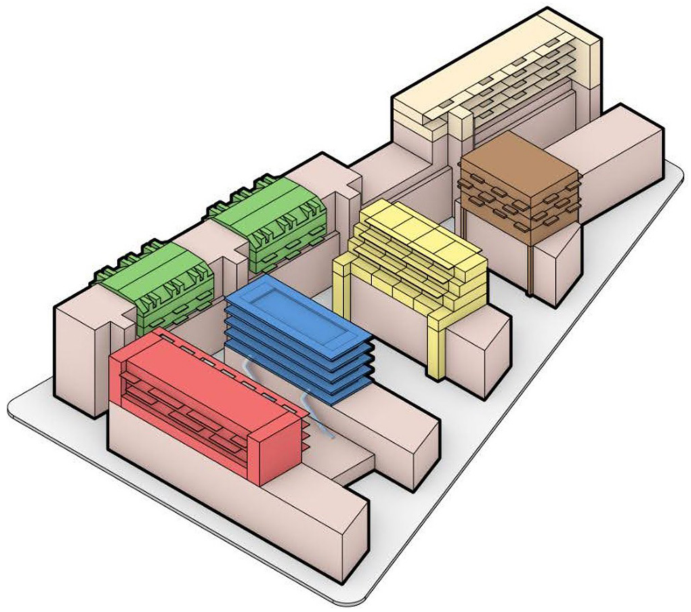


Figure 5.1: Top-ups in different shapes and sizes (Boom, n.d.)

In the Netherlands there is around 400 km^2 of unused flat roof surface, and according to research by 'Stec Groep' the potential for topping up is the largest in the province of 'Zuid-Holland', where as much as 28.800 houses can be added to the existing building stock. Focussing on the largest city in this Province, Rotterdam, there is about 18 km^2 of unused flat roof surface which can be used for e.g. top ups (Hannema, 2024; Monster, 2023; Wassenberg, 2022). According to Monster (2023) about two thirds of the expected potential of 100.000 houses on the flat roof surfaces can be realized on multi-family houses owned by housing corporations. An important fact here is that a significant part of these corporation owned apartment blocks originate from the post war period (1960, 1970, and 1980), and what these apartment blocks share, is that they are made in standard grid sizes. Which makes topping up more easily scalable and possibly a standardized process. Vastenhoud (2020) and Verburg (2000) describe more advantages to the post war apartment block typology, among these advantages are:

Helping the old with the new

Adding new built apartments on top of significantly older ones can turn out useful for the existing part in multiple ways.

First, the age of the existing apartment blocks range from 50 to 70 years, indicating that if no in-between renovations have taken place, the building physics of the apartments are in a bad state compared to current standards. The profits/cash flow from the new built apartments can help in improving the building physical aspects of the existing apartments, making them more sustainable - which in its turn reduces costs such as energy costs too. This increase in profits may however be partially needed for the instalment of an elevator, which is mandatory for apartments with a main entrance higher than 12.5m, which on the positive side provides the already existing apartments with an elevator too, and increasing house value and accessibility.

Second, for the maintenance of common spaces in an apartment block, there is a monthly fee for each household. With the addition of new households, the monthly fee will be divided into more parts resulting in a lower amount per household.

Already existing infrastructure and services

Besides the need for empty space to build new houses on new building lots, there is also infrastructure, services such as public transport and stores, and mechanical, electrical and plumbing installations that need to be arranged. In the case of topping up, all these aspects have been created and provided already. The increased population in an area is positive for the local economy as well, because this increases the number of potential customers for local stores and can improve the liveability by having a more lively environment.

Architectural improvement

The existing apartment blocks generally consist of a specific housing type. Topping up provides the opportunity to diversify in this, the top up itself can be a different housing type, such as studios or co-living spaces instead of multi family houses, but the space can also be used for neighbourhood hubs or roof gardens. Next to that, the post war apartment blocks are part of the architectural history, topping up will increase their lifespan and thereby the preserve the historical value they have in their area.

Finally, the load bearing structure of post war apartment blocks, on average, can carry an extra 10-12% weight. To stay below this number it is crucial to top up with a lightweight material, such as timber in this case. Timber is known to be a sustainable material by capturing CO_2 and the fact that trees are regenerative. Using a sustainable material can add to the sustainable image of the whole apartment block, and potentially increase the value of them too.

From the aforementioned advantages, it can be concluded that topping up on post war apartment blocks is a feasible way to use unused flat roof space and realize houses in this time of a shortage.

Figures 5.2 through 5.8 have been referred to earlier on already. These images show different ways of applying a top up. Whereas all seven designs have a housing function and have been built in a later time than the lower part, there are also some differences between the designs.

Standalone load-bearing structure or using the existing buildings' load bearing structure

Figures 5.2 through 5.6 show smaller scale top ups, which use the already existing load bearing structure of the building below it. The discrete system in this research it will also be added on top of the existing load-bearing structure, and because of the way that loads are transferred between elements, it is logical to place the new load-bearing structure directly above the existing one.

The post war apartment blocks are all constructed with load-bearing walls that cover the width of the block. Not only back then, but also currently this is a common way to have load-bearing elements in a building, however this also creates inflexible floor plans because the walls already divide the floor plan.

Figure 5.2: Top-up house designed and inhabited by architect Tjeerd Bloothoofd (C. van der Kooy, n.d.)

Figure 5.3: Top up on storage unit by Qupus Architectuur (Qupus architectuur, n.d.)

Figure 5.4: Top up design by Symbiotic Urban Movement TU Delft (SUM, n.d.)

Figure 5.5: Didden village by MVRDV (R.'t Hart, n.d.)

Figure 5.7: Fenix I top up by Mei architecten (Mei Architecten, n.d.)

Figure 5.6: Apartment block top up in Amstelveen (L. Kramer, n.d.)

Figure 5.8: Top up block Karel Doorman by Ibelings van Tilburg architecten (O. van Duivenbode, n.d.)

Figure 5.9: Topographical map of Rotterdam South with area around Pirandellostraat highlighted (adapted from Apple Maps, n.d.)

5.3 Apartment block 'Pirandellostraat'

Following the findings of the previous sections, the case study project will be a post war apartment block located in Rotterdam. The building selected for the case study is an apartment block on the 'Pirandellostraat'.

Figure 5.10: Topographical map of Rotterdam South with Pirandellostraat highlighted (adapted from Apple Maps, n.d.)

'Pirandellostraat' is a street in the 'Homerus' neighbourhood (see highlighted in Figure 5.9 and Figure 5.10 for a smaller scale) within Lombardijen, a post war expansion neighbourhood in Rotterdam South. It is one of the 'Garden cities' (as developed by Ebenezer Howard first in 1898) of Rotterdam. However, in some parts of the neighbourhood liveability has increasingly come under pressure. Focus aspects and ambitions for the future of the neighbourhood have been drawn up, and in the end these ambitions should make the neighbourhood resilient again. Opportunities within the neighbourhood are its sustainable connection to the centre of Rotterdam and on a regional level - which makes the neighbourhood interesting to develop new houses and (public) facilities in. It is characterised as a 'family-neighbourhood' that is in need of housing differentiation to retain its residents. It is said that the 'Homerusbuurt' is one of the three areas in Lomardijen with the most challenging and urgent tasks.

Current residents and various organizations/groups categorized eight goals for the neighbourhood. With the relevant ones for this research being (Rungs, 2023):

- improved social safety and a stronger social network, e.g. more social facilities - which can be located in one of the top-ups;
- differentiating the housing inventory, both in size and by rent/purchase ratio;
- improving sustainability, meaning a climate adaptive neighbourhood with e.g. proper water drainage and making existing houses more sustainble. The top ups can be equiped with the right water drainage systems, and the cash flow of the top-ups can be used to make the existing houses on which it is built more sustainable (Vastenhoud, 2020; Verbug, 2020).

Housing and social statistics Lomdardijen

Figure 5.11 through Figure 5.16 show various statistics on construction year, housing and owner type, and energy labels for the housing stock in Lombardijen, and there is one statistic on loneliness in Lombardijen.

A large of the houses in Lombardijen are from the pre 2000 period (Figure 5.12), with a notable quanitity of houses constructed between 1950 and 1970 (Figure 5.11). Also, 71% of the houses in Lombardijen are apartments (Figure 5.14), and more than half of all the houses are owned by a housing corporation (Figure 5.13). From these statistics it can be concluded that there is a large potential for topping up in this area. The housing typology and construction year indicate the presence of a large number of post war apartment blocks, and the fact that more than half of all houses is owned by housing corporations can lead to believe that topping up one apartment block can have a 'snowball effect' into topping up numerous other blocks, more so since the apartment block on 'Pirandellostraat' is not unique, but just one of twelve of the same apartment blocks in the area.

With topping up, the housing corporations can make their existing building stock more sustainable since more than 40% of the houses in Lomdarijen have energy label D or worse (Figure 5.15). Additionally, there is need for social interaction (spaces) for the people in Lombardijen (which can be housed in the new top-ups), because for the age groups 18-65 and 65+ 60% say to be lonely, and even very seriously lonely for 19% and 16% respectively.

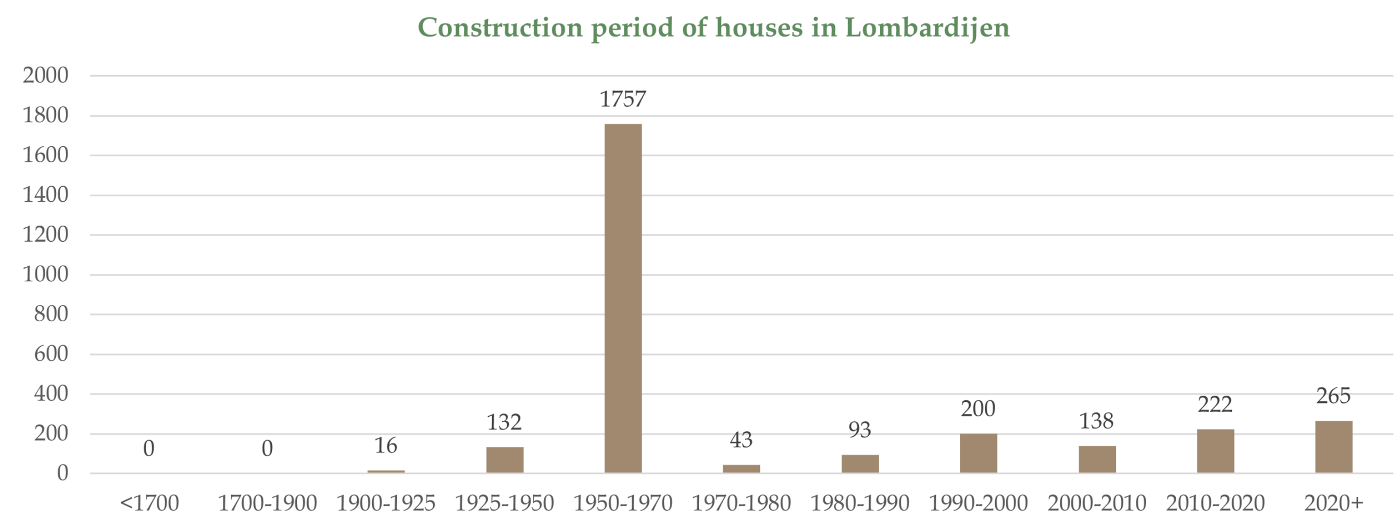


Figure 5.11: Construction period of houses in Lombardijen (Adapted from AlleCijfers, 2024)

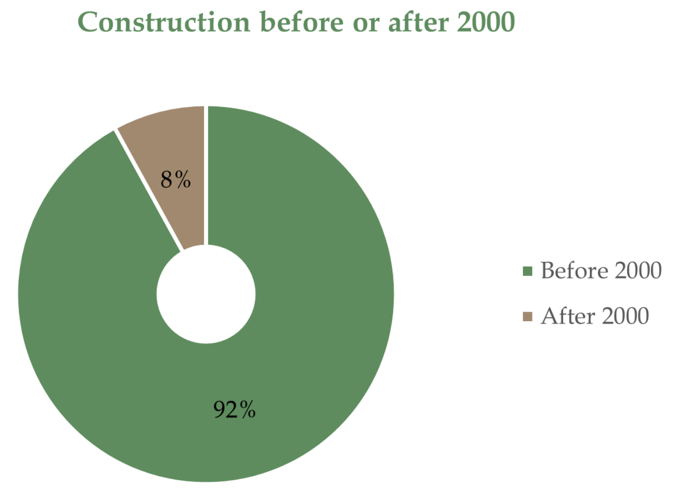


Figure 5.12: Construction of houses before and after 2000 in Lombardijen (Adapted from AlleCijfers, 2024)

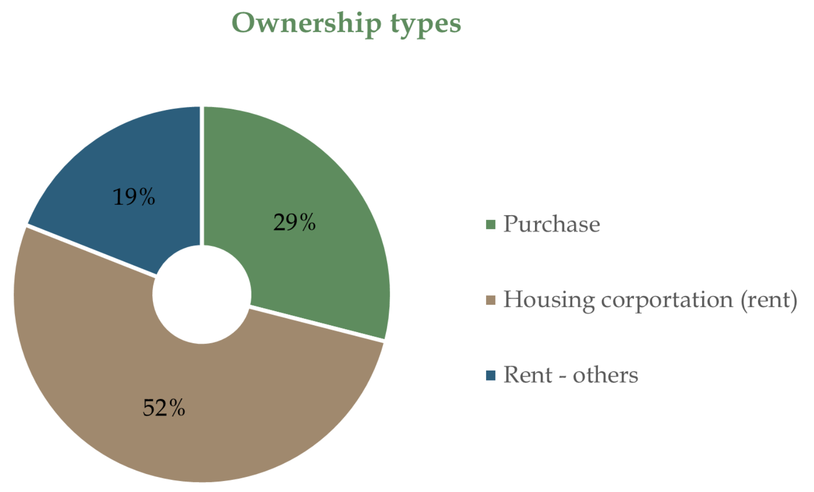


Figure 5.13: Ownership types of houses in Lombardijen (Adapted from AlleCijfers, 2024)

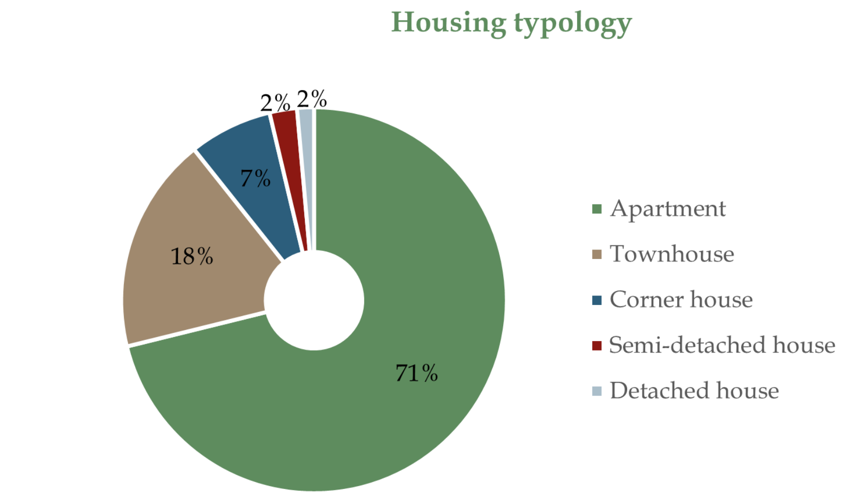


Figure 5.14: Housing typology in Lombardijen (Adapted from AlleCijfers, 2024)

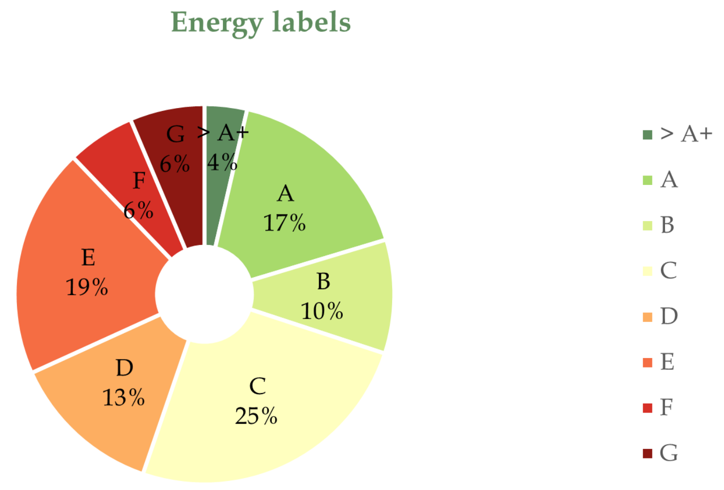


Figure 5.15: Energy labels of houses in Lombardijen (Adapted from AlleCijfers, 2024)

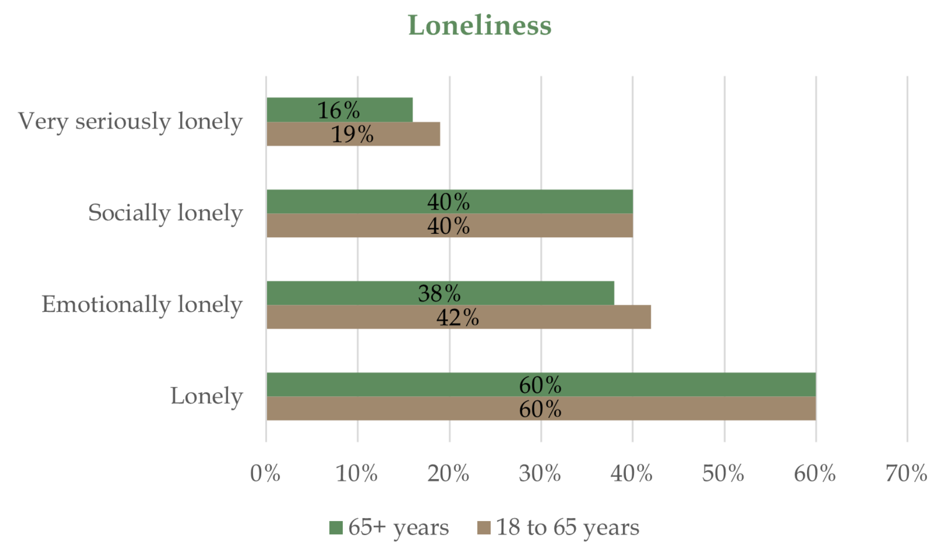


Figure 5.16: Loneliness percentages in Lombardijen (Adapted from AlleCijfers, 2024)

6. Discrete elements to discrete systems

A discrete system is an aggregation of discrete elements whether or not in a specific order. Crucial however, is that the discrete elements has a limited amount of possible connections (Retsin, 2016a). The discrete elements can be a variety of different geometries and materials, but also the way they are connected and the joints used in this connection can vary.

For this research the material is already established as timber, both for its lightweight characteristic and sustainability, and the joint needs to be (easily) demountable. This chapter will focus on the other aspects of geometry, joints and connection type.

6.1 Existing discrete timber elements

Currently there is little to no extensive research on existing discrete timber elements. The research available is usually focused on the application of one specific typology of discrete element into a discrete system and often connected to the design of a pavilion or statue like structure with this discrete system. However, it is crucial to have a good overview of the available options, so that a well-advised decision can be made when picking a discrete element typology.

In his research de Paula (2023) analysed and presented existing discrete elements as part of a discrete system, fFigure 6.1 shows a visual summary with the pros and cons for each typology as a result from this analysis. There are nine resulting typologies that are categorised according to their geometry.

Within these nine typologies, three categories are leading in which either discrete element would fall, these are blocks, plates and beams. Subsequently, within these categories there is a general division between solid and hollow elements.

When picking a discrete element typology that first the use case of this research, there are some criteria to run the nine options by. Since this research aims to answer "how a reversible discrete timber system can be a feasible alternative to conventional structural timber" and the discrete timber system will be used for a topping up on a post war apartment block, the criteria that make a discrete element a fitting one are: lightweight, ease of assembly and disassembly, and the ability to reconfigure.

As a first step in selecting a fitting discrete timber element, these criteria are compared with the pros and cons listed in fFigure 6.1. The discrete elements that can be disregarded as a result of this comparison then are the ones where the elements are connected with adhesives such as mortar or glue, the highly specific elements. The glue or mortar makes it difficult to disassembly the elements without damaging them, and the specific elements are tricky to reconfigure because they are designed for a unique use case.

With this knowledge solid blocks and solid-bar blocks become unfit as they are fixed with mortar and shape-specific beams and complex blocks are unfit too because these elements are too specific.

In theory this leaves the following five discrete element typologies (highlighted in fFigure 6.1) as suitable options for designing this discrete timber system:

- hollow blocks;
- hollow-bar blocks;
- solid plates;
- hollow plates;
- orthogonal beams.

However, in practice there are some differences between these options that make some better than others. The biggest downside of the hollow-bar blocks, solid plates, and hollow plates are that these typologies consist of many smaller parts. For a discrete system that could consist of up to a hundred discrete elements this becomes inefficient are perhaps non-feasible. The hollow blocks can also consist of a number of smaller parts but this can easily be prefabricated into the larger discrete element that will be used in construction.

In the end leaving the hollow blocks and orthogonal beams as two of the better discrete element typologies for this use case. Hollow blocks and orthogonal beams are both material efficient typologies, which is a strong advantage when you want to design something sustainable. The human scale lightweight, meaning that it can be constructed and de-constructed without the aid of big machinery, can be a good advantage for high density areas and also in combination with reconfigurable discrete elements. Last but not least, most important even, the reversible connections in a system of these discrete elements are a must in the scope of this research.

6.2 Examples of hollow block and orthogonal beam projects

With a given typology there are still numerous ways

Figure 6.1: Discrete element typology analysis (A. de Paula, 2023)

of connecting the discrete elements to each other, and also many different applications for the discrete system. This part aims to show some of the potential designs with hollow blocks and orthogonal beams, it does not focus yet on the connection between the elements in depth, but rather covers it briefly.

Hollow blocks

Hollow blocks are a relatively light typology. The elements usually consist of fabricated hollow geometries from individual elements such as sheet materials or a combination of different individual elements

Figure 6.2: Hollow OSB blocks (I. Tedbury, 2018)

Hollow OSB blocks

The first example within the hollow blocks typology is a hollow element from folded OSB plates designed by Ivo Tedbury in combination with an automated housing construction platform. The final elements are trapezium shaped with the sides having a 45 degree angle. The geometry allows for an infinite amount of possible combinations. The elements are connected through a steel plate connected to the side of the final element (the part where the steel plate is connected to is visible on Figure 6.2) with fasteners (screws or bolts).

STEKO®

Another option with hollow blocks is such as STEKO® designed as a new way of building faster, easier, cheaper, and with human and environmental health in consideration, see Figure 6.3. It is a discrete element built up from a number of smaller parts, practically a timber brick. The elements are topologically interlocking through a male-female connection (the top part containts the male part of the connection) and through dowels, these dowels are then placed in the elements connecting both sides (for all four elements a hole is visible at the top).

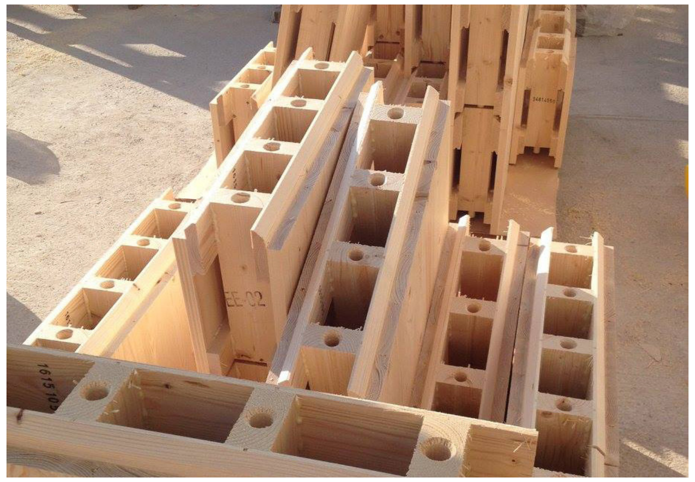


Figure 6.3: Steko building system (STEKO®, 2017)

Orthogonal beams

The other typology is the orthogonal beam, which is a plain timber beam that, without considering its joints, can only be placed in an orthogonal manner (only with a 90 degree angle). However, with help of joints it can also succeed under different angles. This is perfectly shown by the example system shown in Figure 6.4.

Combinatorial nest

This figure shows the design of the 'combinatorial nest', a competition entry for the 2019 Tallinn Biennial by a multi-disciplinary team. Sanchez et al. (2019) define the combinatorial nest as: "a discrete open-ended tectonic system, that relies on the patterning of material units to grow volumetrically with different motifs". The connection piece between the individual elements aids in the volumetric growth and enables for numerous different motifs and three dimensional growth, and not just a monotone growth pattern.

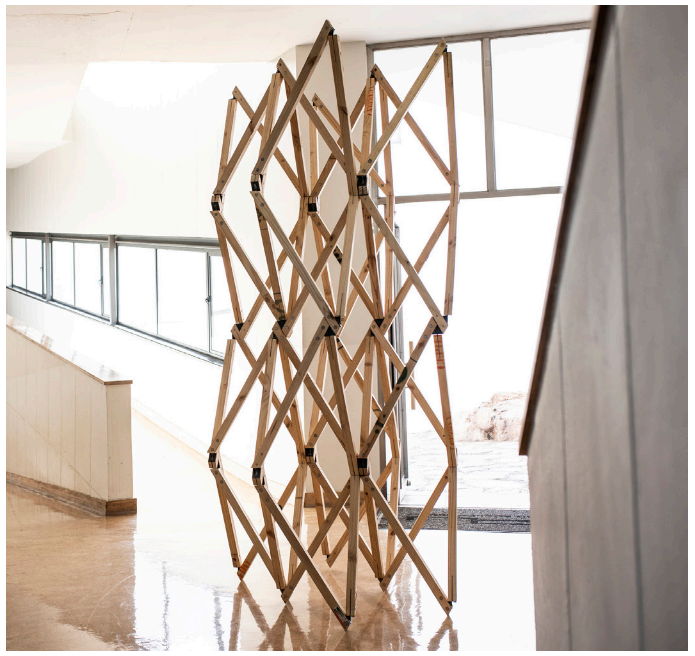


Figure 6.4: Folly.age system developed with Diego Pinochet and Felipe Veliz (Plethora-Project, 2019)

Yusuhara wooden bridge museum

The Yusuhara wooden bridge museum is a bridge like structure in its namesake village 'Yusuhara' in Japan. "The project's challenge is that it spans a vast distance with small structural elements" (Viva Arquitectura, 2014). The larger whole is constructed from hundreds of smaller beams, with a larger span to cover there are also more beams to the top. The elements support on each other, but in Figure 6.5 it is also visible that the upper element falls into a carved out part of the lower element.

Figure 6.5: Yusuhara wooden bridge museum by Kengo Kuma (T. Ota, n.d.)

Coeda house

The Coeda house is a café designed by Kengo Kuma Architects, on a location with a breathtaking view of the pacific ocean. This café is characterized by its single column structure standing in the middle, to create an unhindered view (Figure 6.6. This structure is achieved by randomly stacking 8 by 8 cm timber beams, with the length of the elements increasing to the top resulting in a visual representation of a treelike structure. In the structure are some hidden rods used to connect different parts of the structure to each other (Viva Arquitectura, 2019).

Figure 6.6: Coeda house tree-like structure (Kawasumi-Kobayashi Kenji Photograph Office, n.d.)

6.3 Connecting discrete elements

Although a structure of any material serves multiple purposes, in timber it is the joints that might be one of the more important aspects of the structure, because as Blaß & Sandhaas (2017) put it: "since those (joints) used in timber connection tend to be weaker than the members being joined". Various other important aspects exclude the existence of one master solution that is a good fit for all problems.

Besides the aspects needed to consider because of the material being timber, this research also pursues the fact that the elements need to be reversible and solely made from timber - which are characteristics defined and ensured by the joints. Within these requirements there are still numerous ways to connect the discrete timber elements to each other.

Hollow blocks

Hollow blocks are commonly stacked on one another, small interventions such as placing them in a bond or topologically interlocking elements(see Figure 6.6) can already enhance stability. Besides that, there are multiple methods of connecting the elements to each other.

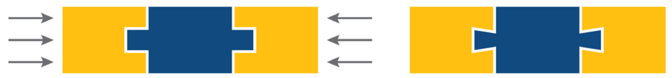


Figure 6.7: Conceptual examples of topological (left) and geometrical (right) interlocking (Estrin et al., 2021)

Interlocking

Interlocking elements emits the need for actual connectors or binders to keep elements in place. The keeping in place is realized by the geometry of the neighbouring element or elements. In Figure 6.7 blue element is kept in place through its interlocking with the yellow elements. Within interlocking there is a difference between topological interlocking and geometrical interlocking. With topological interlocking (Figure 6.8 top), the element are held together by a peripheral force, the element can be

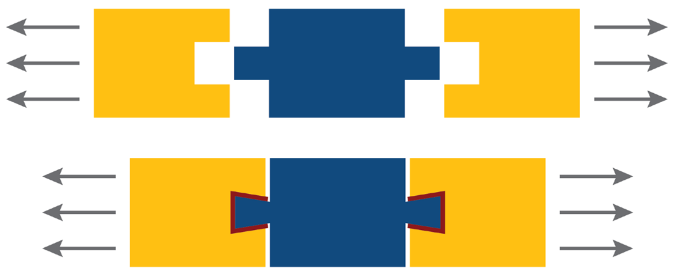


Figure 6.8: Examples of topological (top) and geometrical (bottom) interlocking when under tension (own work, 2024)

(dis)assembled by moving or rotating them. With geographical interlocking (Figure 6.7, right), the elements can only be (dis)assembled by lifting one element, because of deformation of one or more elements, and by breaking one or more elements into pieces (Estrin et al., 2021). Geometrical interlocking evokes the thought of a stronger connection because upon tension the elements lock into each other, see Figure 6.8for a schematic visual, where in the right visual the red part indicates where the elements lock into each other.

Interlocking with hollow blocks for example can be when an upper block is being kept in place with the geometry of the lower block (highlighted green in Figure 6.9).

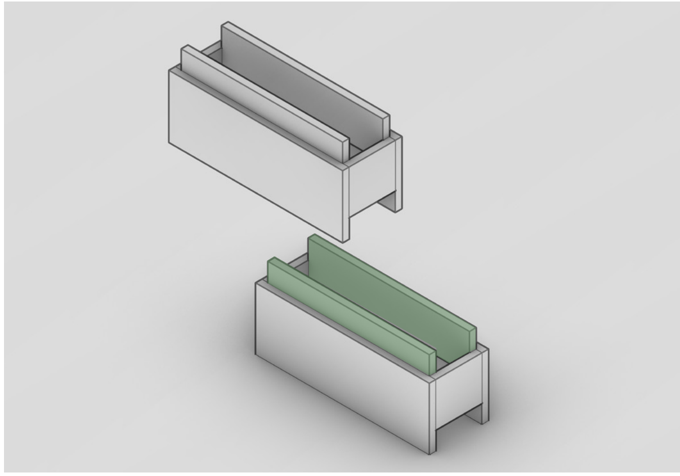


Figure 6.9: Topological interlocking in hollow blocks (Own work, 2024)

Adding vertical supports in the hollow parts

The hollow blocks can also be connected to each other with help of vertical support elements, a solution similar to placing vertical steel rebar elements in hollow concrete blocks and pouring the holes with concrete (see Figure 6.10). There are multiple advantages to using hollow versions of normally solid elements, among these are reduced weight for a full structure and material efficiency. The hollowed out parts can also be utilised for insulation purposes, or for adding extra support by placing vertical (and

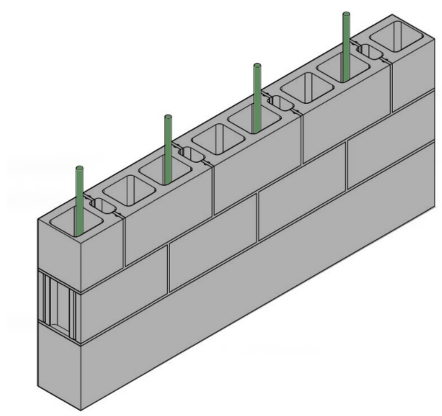


Figure 6.10: Hollow concrete blocks with rebar reinforcement (adapted from A.J.J. Sparling, 2015)

optionally horizontal) structural members that provide extra strength, stiffness and stability. This logic is not only applicable to concrete blocks and steel support elements, but can also be translated to hollow timber blocks and vertical timber elements. Figure 6.11 shows a set-up where timber hollow blocks with four sections are stacked onto a vertical support element, in this example the support elements are placed every fourth hole, this way each block is attached to two vertical support elements.

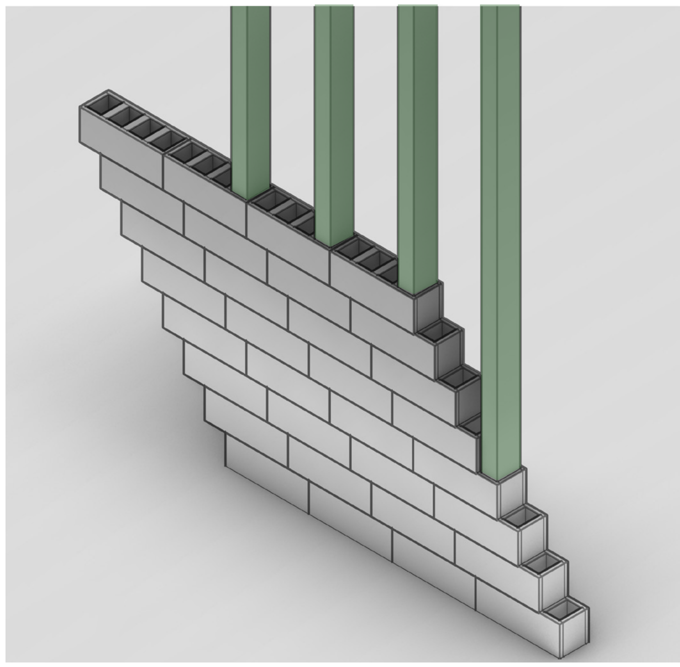


Figure 6.11: Vertical support elements in timber hollow blocks (Own work, 2024)

Dowel connections

A third option for connection hollow blocks with each other is with the help of vertical and/or horizontal dowels. In an existing system such as STEKO® we already see the application of vertical dowels, holes on top of the STEKO® block are visible in Figure 6.3 - by placing a dowel here it allows for connecting elements stacks on each other. However, by designing a hollow block in such a way that it has holes in the side, in combination with interlocking elements as explained in Figure 6.9, it creates the

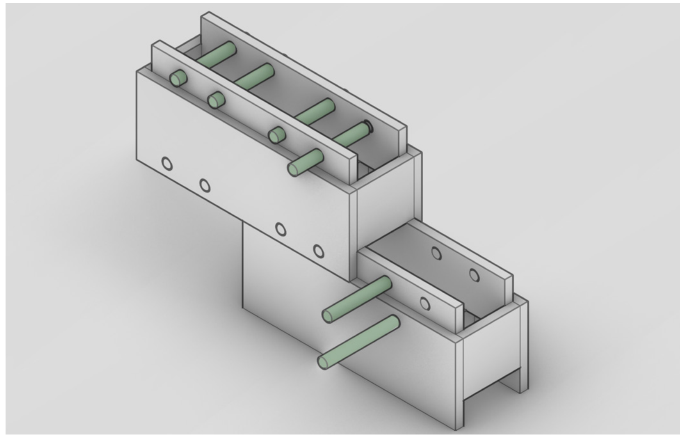


Figure 6.12: Horizontal dowels in hollow blocks (Own work, 2024)

possibility to use dowels horizontally, locking two stacked elements to each other - and this way creating a more uniform construction.

Orthogonal beams

Orthogonal beams is the second discrete element typology fitting this research. Orthogonal beams can be placed in such a way that they are laying on each other (such as in Figure 6.14) or stacked on to or next to each other (such as in Figure 6.13c). To connect the elements, disregarding the way they are combined, they can be connected through dowels or by interlocking.

Dowel connections
Using dowel connections to connect beam elements works with the same principle as with bolts, there are holes in the elements through which dowels are placed and the elements are connected to each other. It is important to either place multiple dowels next to each other or make use of diagonal bracing (see Figure 6.14) to ensure stiffness and prevent rotational movement. Placing dowels works similarly for both ways of stacking the orthogonal beams, deconstructing the elements also works the same for both options, namely drilling the dowels to recreate the hole which in a new use case can be filled with a dowel again.

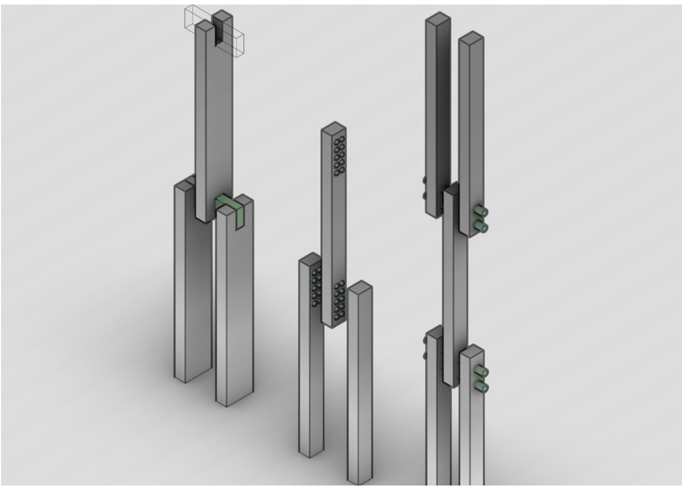


Figure 6.13: Vertical stacking of orthogonal beams with (from left to right) a) toplogical interlocking b) geometric interlocking c) dowels connection (Own work, 2024)

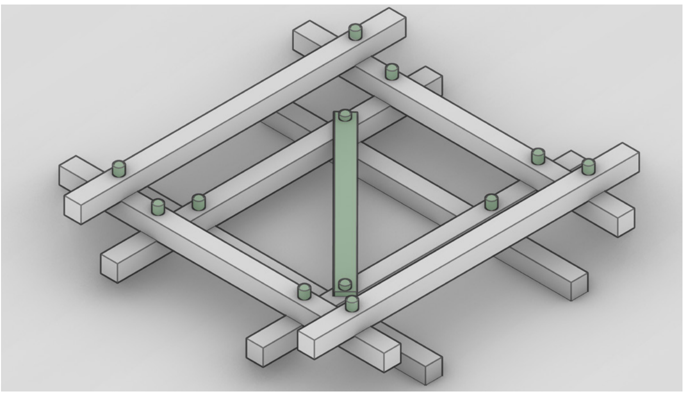


Figure 6.14: Placing orthogonal beams horizontally with dowels and diagonal bracing (Own work, 2024)

Interlocking
Orthogonal beams can also be made so that they interlock with each other, additionally this could be combined with dowels. This way of interlocking looks more like Japanese joints or dry joint techniques, also possibly with the help of inlays. This also means that there are numerous possibilities for applying joints to the orthogonal beams*.

Interlocking can be applied to orthogonal beams combined both vertically and horizontally. When stacked in a vertical manner the connections can be topological and geometrical. Regardless which way of interlocking, they are not attached end-to-end as this would create relatively unstable systems. It would be a system with alternating even-uneven amount of beams that are clamping to each other as can be seen in Figure 6.13.

The other way is to stack the beams horizontally, but for this to result in a stable structural element it needs to have more than one beam per 'layer' of the system, Figure 6.15 shows two options for stacking, left would create a more stable structural system. With the structure on the left being far more stable, this is also the option that is being taken into consideration when looking at possible interlocking joints for horizontally stacking orthogonal beams.

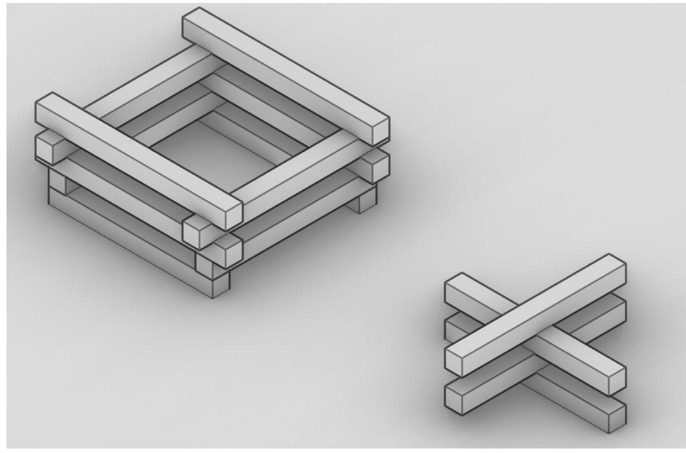


Figure 6.15: Two ways of stacking orthogonal beams horizontally (Own work, 2024)

This option however only has four layers, the goal is to create a floor height structural column by stacking beams horizontally, continuing the sequence as in Figure 6.16. Visually, this could turn out to look like the column in Figure 6.16. An argument for the column looking like this is the 'mushroom column' (see Figure 6.17) - and what this essentially does is reduce peaks stresses where the floor in attached.

Beams that are placed in a perpendicular order also need to have a reversible connection that support this way of aggregating. When comparing this to

** The possibilities shown in this research are not all the possibilities out there, however it is not possible to discuss all of them.*

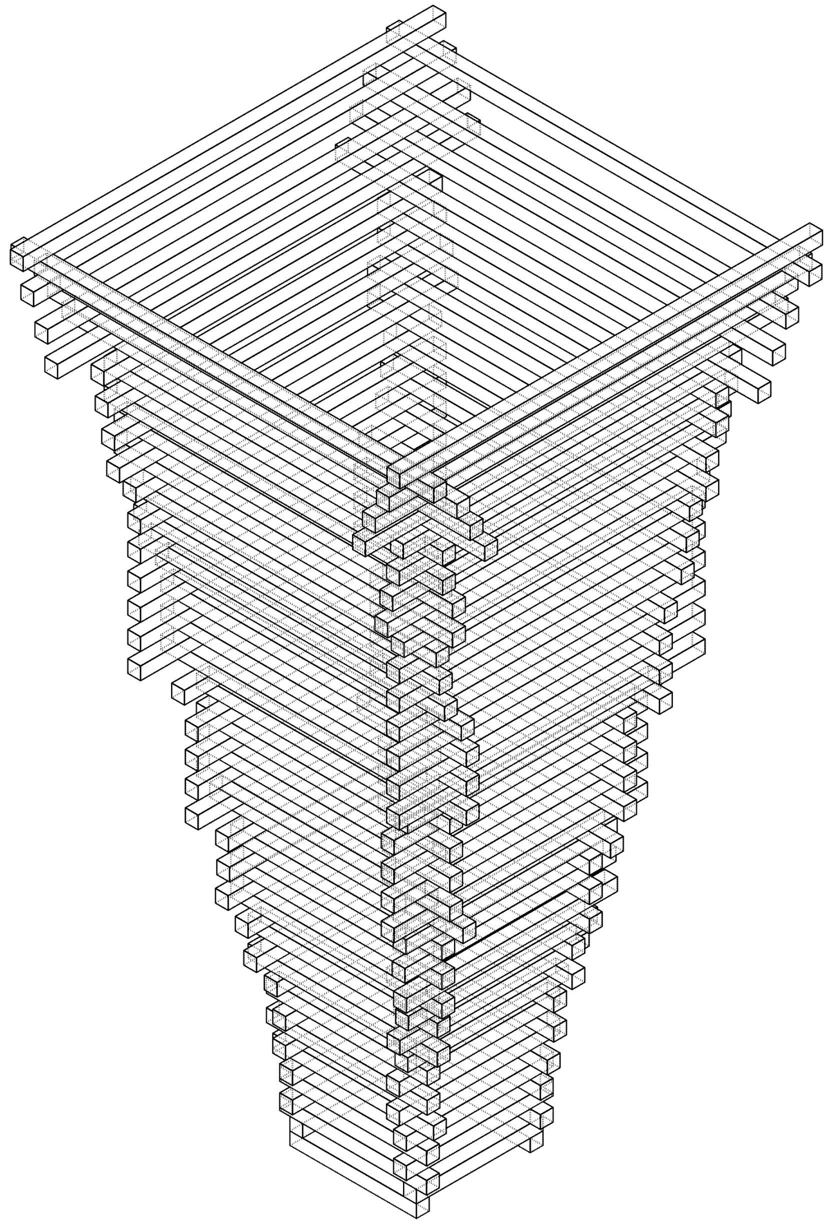
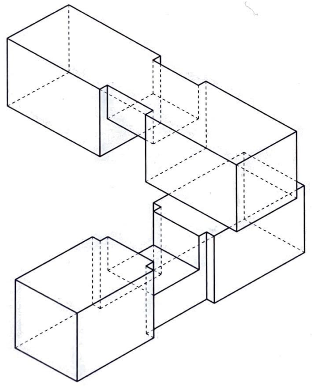
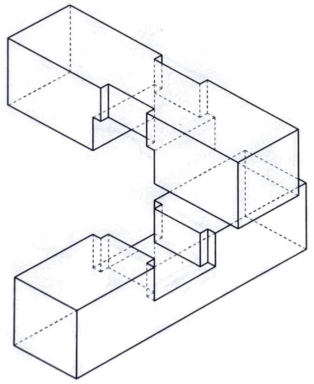


Figure 6.16: Discrete system column by horizontally stacking orthogonal beams (Own work, 2024)

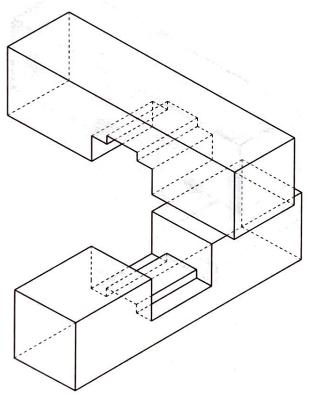
Figure 6.17: Mushroom columns in de Van Nelle factory (Tjasker, n.d.)



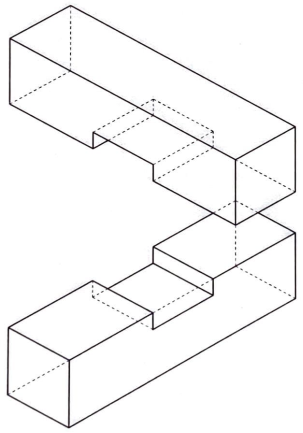
1. *Offset cross-lap*



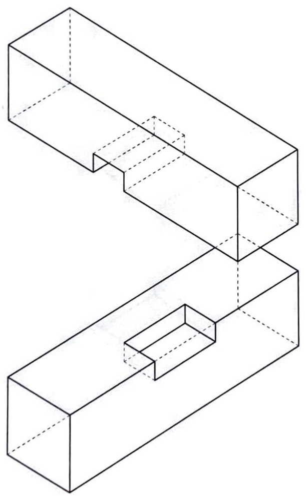
2. *Double-shouldered offset cross-lap*



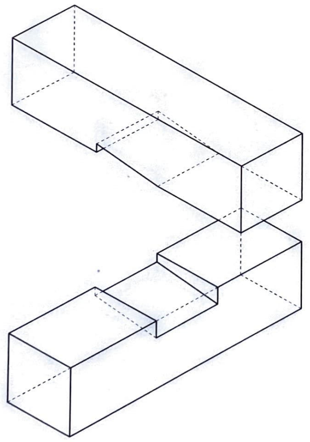
3. *Tabled cross-lap*



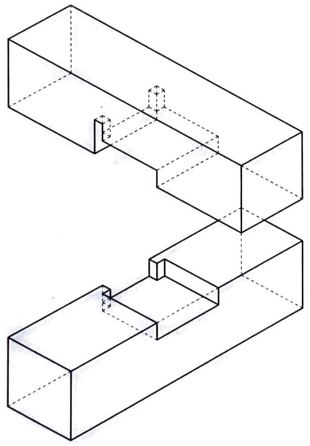
4. *Simple square cog*



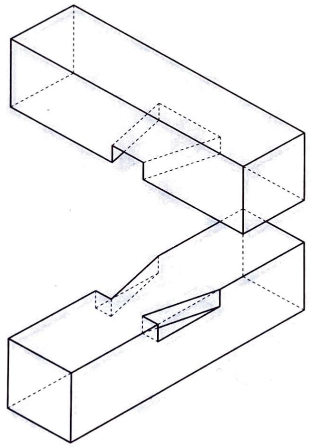
5. *Simple cog*



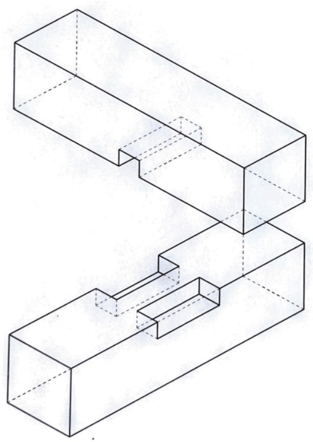
6. *Oblique cog*



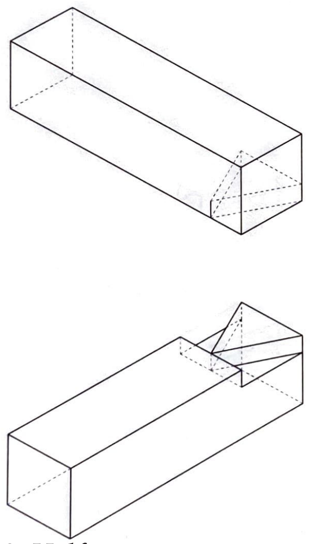
7. *Offset double cog*



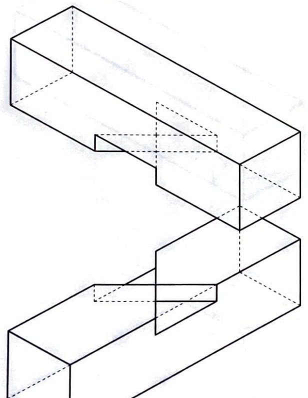
8. *Dovetail cog*



9. *Double cog*

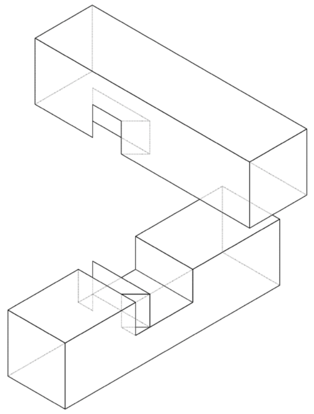


10. *Half-cross corner cog*

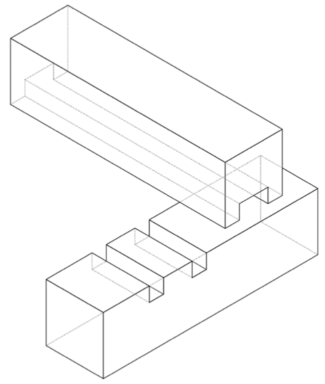


11. *Cross cog*

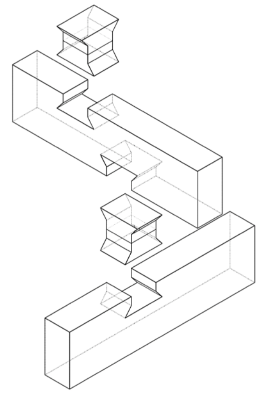
Figure 6.18: Perpendicular wood joints that can potentially be used on the discrete timber elements (Guenoun, 2019)



1. *Dovetail lap*



2. *Inverse double cog*



3. *Alternative loose goose-neck*

Figure 6.19: Three alternative joints for perpendicular beams, inspired by Guenoun 2019 (Own work, 2024).

the vertically stacked beams, which are connected end-to-end, this quite a different way of connecting beams. Guenoun (2019) presents in his book 198 wood joints, Figure 6.18 presents 11 examples from this book that showcase joints with perpendicular elements. Additionally, Figure 6.19 shows three connections that can be used on perpendicular beams too, the option in here are own ideas based on example from Guenoun (2019).

Typology advantages and disadvantages

It can be easily concluded that for both discrete element typologies there are numerous possibilities in creating a structural system for a post war apartment block top up. Using a hollow block variation results in a more conventional structure and continuation of the existing structure below it. Additionally, the hollow parts can be used for insulation or for placing building installations such as cables. However, these hollow block walls will create clearly divided spaces and by doing so taking away part of the (functional) flexibility. Next to that, the way of interlocking the hollow blocks make it more difficult to reconfigure an already built up structure. Both by the fact that in an in-use structure the bottom block can not so easily be reached and replaced, but also because of low flexibility in how the structure can be built up.

By using the orthogonal beam structure to make column-like structural systems a more open floor plan can be designed which can be partitioned by various self-supporting wall elements into a variety of differently sized and shapes spaces. The orthogonal beam system also has more functional flexibility, for example in an outdoor space it is not preferred to have closed walls wherever the load-bearing structure is. By making a column from either vertically or horizontally stacked orthogonal beams, a significantly open outdoor space can be realized, and the hollow part within a horizontally stacked column can be used for e.g. building installations. Additionally, by contrasting the type of existing load-bearing structure an interesting architectural composition can occur. Downsides to this typology is that for the vertically stacked orthogonal beams the stability might be an issue whether or not solvable by using large elements, which results in a lower material efficiency. Material efficiency might also be lower for horizontally stacked orthogonal beams.

With the criteria for the structural system being:

- Reversible and reconfigurable
- Creates flexible floor plans
- Speed and ease of assembly and production
- Lightweight systems
- Structurally sound (obviously a criteria for any structural system)

The orthogonal beams comes out to be the better discrete element typology to use for topping up on post war apartment blocks. More specifically the horizontally stacked orthogonal beams, mainly because of the lack of stability in the vertically stacked option, but also by the increased architectural value.

Connecting orthogonal discrete timber elements

First and foremost, there are hundreds of interlocking joints in timber, some definitely are not suitable for the use case in this research, but there sure are tens (or also even hundreds) of joints more that potentially could have been used in this discrete timber system. The joints that are visualized in Figure 6.18 are taken from a book called '198 wood joints' by Elias Guenoun. These joints are selected based on simplicity and whether or not they could be applicable to perpendicular (stacked) beams, as opposed to e.g. end-end connections. The joints in Figure 6.19 are inspired adaptations and combinations of existing joints from the same book.

To find out which joint or joints are most fitting for this research the possible options from Figure 6.18

and Figure 6.19 will be checked on a set of criteria relating to the joint, namely:

- Ease of (dis)assembly
- Complexity in relation to production speed (by analysing the complexity of the cut and if there are loose elements)
- Flexibility on element level (can one element be used to make a system of various different sizes?)
- Strength of the joint (by stress and deformation tests in Ansys)

The strength of the joints will be checked in the following section, however because there is some similarity in the joints and bullet points 1 to 3 can already separate the more and less good options, not all the options from Figure 6.18 and Figure 6.19 will be analysed in Ansys.

In Figure 6.18, joints 1, 2 and 3 are similar, however the connections for these elements are half the height of the total and will therefore need significant adjustments to the geometry to be used how it needs to for this research.

Joints 4, 6 and 7 share similarities, however option 7 is more complex and seemingly fragile with smaller elements, and option 6 is a more complex and weak connection due to the oblique cut-out.

Joints 5 and 9 are similar, however the double cog as options 9 would be more stable with the loads divided equally to both sides in a structural system such as in this research.

Joints 8, 10 and 11 are not discussed in any of the previous 'groups' yet, option 10 is only applicable for a corner which makes it not a good option for this system, and option 11 consists of relatively small parts. Option 8 can be good options and therefore will be tested in Ansys among the other options.

In Figure 6.19 option 3 is too complex and would require a large number of separate elements for the connections to even construct one structural element. Options 1 and 2 however both can be good options and will therefore also be tested in Ansys.

This results in options 4, 8, and 9 from Figure 6.18 and options 1 and 2 from Figure 6.19 too be tested in Ansys.

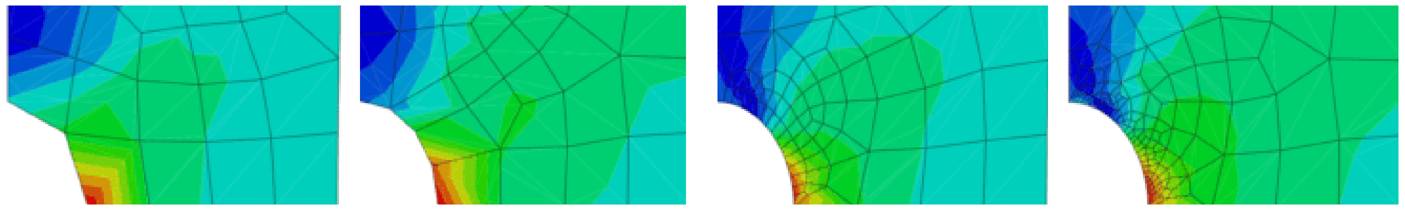


Figure 6.20: Mesh convergence (Harish, 2024)

6.4 Ansys set-up

Introduction

Within the Ansys environment, Ansys Workbench (2021 R2) will be used to conduct a Finite Element Analysis (FEA) on the different joint options for the horizontally stacked orthogonal beams. A FEA is computational way to see how a product, such as a joint, will react in a real life situation to forces such as loads, but it can also be used for other physical effects such as a fluid flow analysis. The results from this analysis then show whether or not the joint (in this case) is strong enough or will break, but also where maximum and minimum occurrences of e.g. stresses and deformation in the joint will be. It does so by dividing the (a) beam into a given amount of smaller parts; 'finite elements', and for each of these elements a mathematical equation aims to predict how this specific element reacts under a given load, force, or other external factor (Autodesk Inc., n.d.).

The amount of finite elements that the beam is divided into can have an influence on the final results. This is because the e.g. stress in a beam per finite element is averaged over this whole finite element. With a process called 'convergence' the finite elements are stepwise refined into smaller pieces, with the goal of yielding a more accurate result. Figure 6.20 shows this process, with from left to right more accurate division of the surface into finite elements and thus getting a better idea of where exactly the higher stress occur. However, more refinement is linked to a heavier computing tasks and longer to solve the solution.

Ansys will be used to see what the (extreme) stresses and deformations are in the element and in the connection, under the assumption that it is the lowest column in the top-up (and carries the highest load) see Appendix A for the load calculation. The information from the simulation results on stress and deformation will be used to decide which joint is a better option for connecting these discrete element..

In order to get accurate results it is crucial that the geometry for each joint is identical (or as similar as possible), each joint is tested in the same way, and exposed to the same external factors:

- The beam length, width and height should be the same for all elements, as well as the joints where possible - otherwise it should be the same scale.
- The location of the joints on these elements should be as similar as possible, however due to the geometry of the joint this might not always be identical.
- Each element should be of the same material.
- The loads that the elements are subjected to, the location where these loads are on the elements, and the support points should be identical

Chapter 2 discussed some of the characteristics of timber, also the fact that tree species are either hardwood or softwood, both with its advantages and disadvantages. For the discrete elements (and thus discrete system) in this research, hardwood is used, and more specific Oak hardwood. The 'ANSYS GRANTA Materials Data for Simulation (Sample)' library within Ansys already provides material information on Oak, therefore this material is used.

There are a few arguments to support the decision for hardwood over softwood. First the durability, which is especially important for the first layer of the top-up which can be an outdoor space and be exposed to rougher conditions than inside. Another durability argument is the fact that the aim is to design a reversible system, having system that is durable but not a timber type to meet this durability would be illogical.

Simulation set-up

Uniformly checking the various joints is important, as described in the previous part. This section will elaborate on the set-up for the geometry, loads etc. that were used to test the joints.

Geometry

The idea is to perform the simulations on five layers of beams stacked alternatively in an orthogonal way. The thought behind this is that the effect and/or constraint the elements possibly have on each other can be seen. This set of layers has been taken from the middle of a mock set-up for the column (see Figure 6.21), resulting in point loads occurring on the spots where in the full system the next layers are connected, and supports where the layers below are connected.

The bottom beam is 1000mm long and each following layer increases with 150mm in total, so 75mm on the left and 75mm on the right. Figure 6.23 shows a side view of three beams, and with length increments of 150mm per layer this means that the steps in this view are 1000mm-1300mm-1600mm. If the increments have a linear relationship from this point to the top, the topmost beam will be 3625mm in length. The cross section of the beam is 50mm by 100mm, and the interlocking joints will be cut out from the top

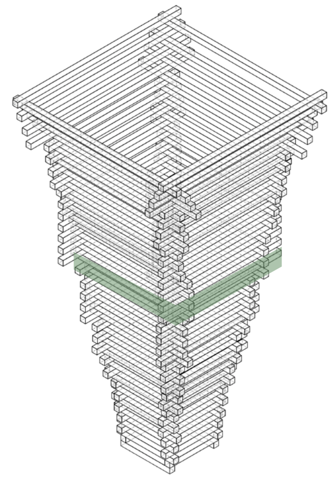


Figure 6.21: Discrete system column by horizontally stacking orthogonal beams (Own work, 2024)

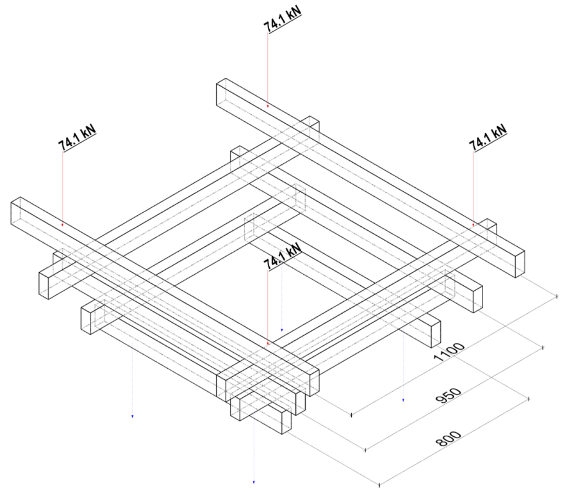


Figure 6.22: Baseline discrete system set up for ansys with loads, supports and dimensions (Own work, 2024)

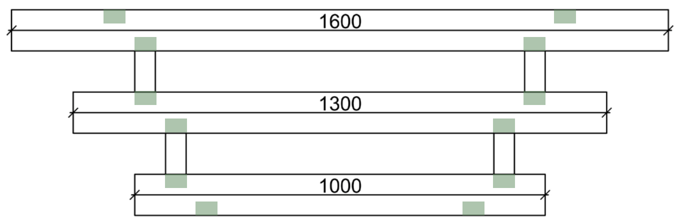


Figure 6.23: Element sizes - side view - for Ansys (Own work, 2024)

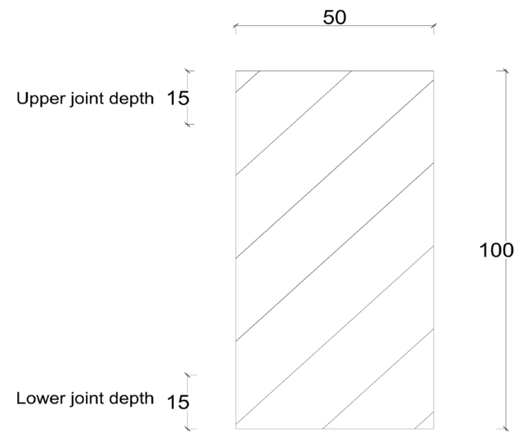


Figure 6.24: Cross section of the beam for Ansys (Own work, 2024)

and bottom 30mm of the beam (see Figure 6.23), the cut out for the joint will be square (e.g. if the beam is 50mm wide, the cut out will be 50mm by 50mm). The initial mesh size is set to 20mm.

Loads
Figure 6.22 shows a schematic visual of this, with the loads denoted with red arrows and the supports with blue arrows, the direction of the arrow show the direction of the load. The loads acting on this column have been calculated in a simplified manner, for this calculation please refer to Appendix A.

Joints
In order to enable the size increment of the beams to the top, the joints are placed not directly above each other, but for each beam, the joint in the upper part of the beam is moved outward compared to the joint in the lower part of the beam. Ansys automatically assigns where contact points between different elements are, which in this case occur between the joints, the green rectangles in Figure 6.23. While this is a good thing, the joint can behave different depending on which option is assigned (manually) to this setting. There are five options which are checked for whether or not they can be separated and whether or not they can slide (Syed, 2022).

- bonded contact, elements cannot separate or slide;
- rough contact, elements can separate but not slide;
- no separation, elements cannot separate but slide frictionless;
- frictional, elements can separate and slide frictional;
- frictionless, elements can separate and slide frictionless.

For the discrete elements the frictional contact option is assigned to all contact points. Since there is only fastening with interlocking, there is room to slide (shear), leaving 'no separation', 'frictional', and 'frictionless'. However, upon sliding the elements will experience friction, and therefore the 'frictional' contact option is used.

Solutions run by Ansys
Ansys can run simulations for various factors (called 'solutions' in Ansys) such as deformation and stress. Every solutions can be useful in its own way, for this research the solutions run and reasoning behind them are:

- total deformation, to see the changes that occur in the geometry when loads are applied;
- maximum principal stress, to check where the highest stress in either of the principal directions is;
- normal stress; to check what the normal stresses in the elements are.

6.5 Ansys results

Immediately after running the first simulations, errors occurred. Starting off with the inability to run a convergence, for which two problems happened. The first one being a user face error causing the convergence check not to run properly. This specific error only occurred when running the convergence check on five layers. For the two layers and single element simulations the convergence did run, but after a few runs still saw an increasing graph and eventually there would be enough computing power on the machine used.

Then with regards elements, beginning with using five elements from the middle of the column; this does not work because the top beam is not constrained in the Ansys model whereas it is in the actual system (with another layer). This causes for unreasonably high deformation and forces that would not occur in real life in the same magnitude as in this Ansys model. Figure 6.25 shows the deformation in a set up with five layers. First, in this simulation the maximum deformation is 18,311mm, which is way larger than the acceptable deformation of length/360 (1600/360 = 4,44mm). A second issue is that the top beam bends inward as a result of forces more outward than the support points and the lack of constraint. Additionally, the largest deformation values occur in the ends of these top beams, but the results are not accurate due to the issues explained before.

If just one element is ran, the effect (e.g. the constraint and load transferring) that the elements have on each other cannot be measured, and also the selected joints from Figure 6.18 and Figure 6.19 cannot be compared with each other because they are not 'activated' in a simulation with just one element. Simulating one element does show a better image of stresses in the cut-outs for the joints.

To try and solve this issue with simulating just one element, a model was made with one element and two segments of full beams from the layer above. As it turns out, this also does not work accurately because the contact option (frictional) then causes the elements to separate and Ansys fails to run the simulations. In some cases this can be solved by changing the contact option to, for example, bonded (this is done for the model in Figure 6.26). However, the stress and deformation results do not change much from the simulation with just one element.

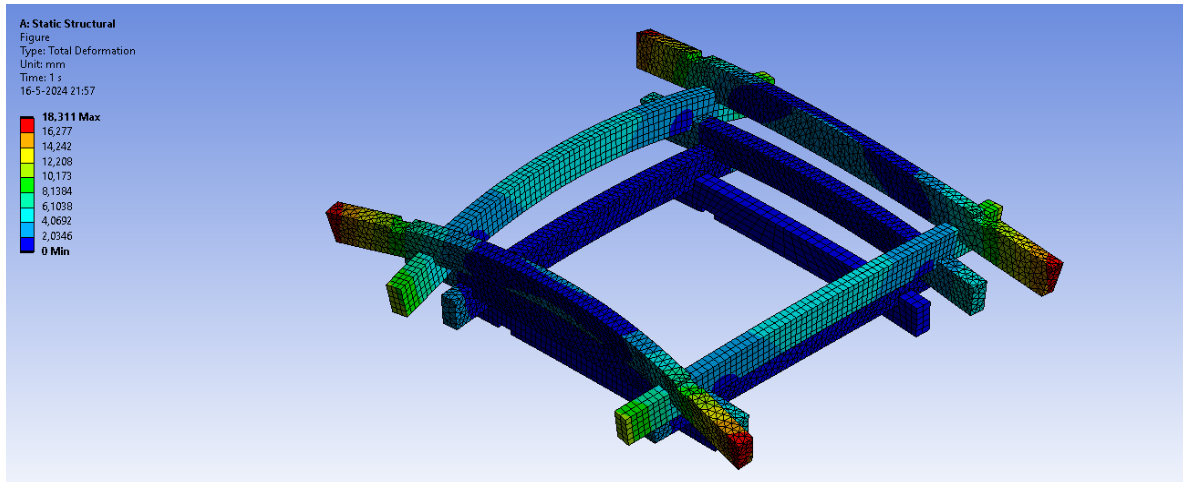


Figure 6.25: Ansys deformation simulation with five layers (own work, 2024)

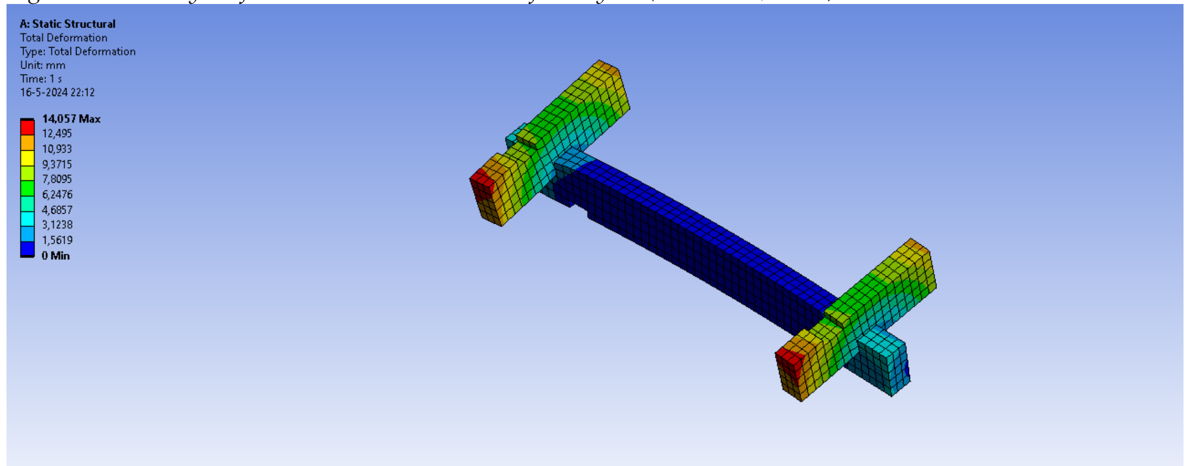


Figure 6.26: Ansys deformation simulation with one layer and two segments on top (own work, 2024)

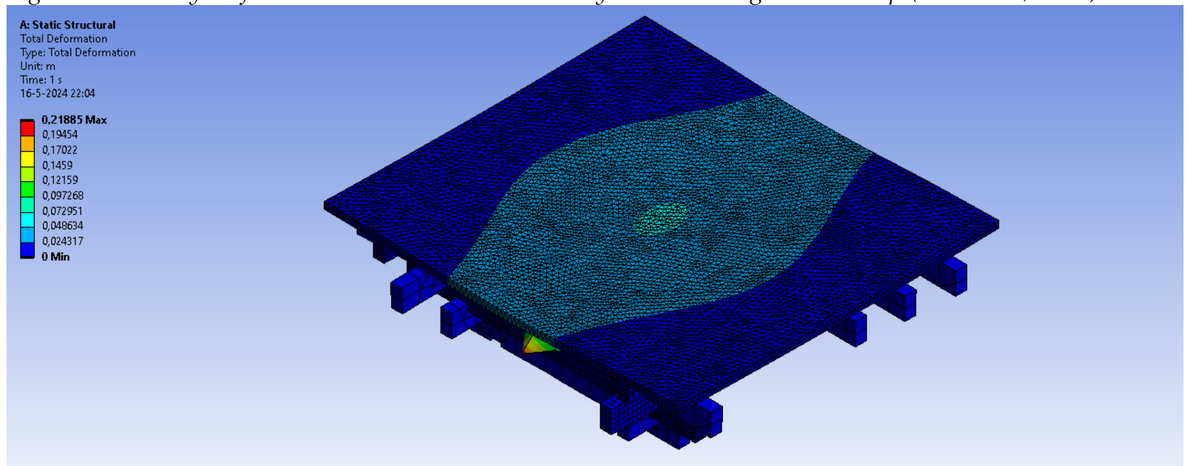


Figure 6.27: Ansys deformation simulation with five layers and top constraint by floor (own work, 2024)

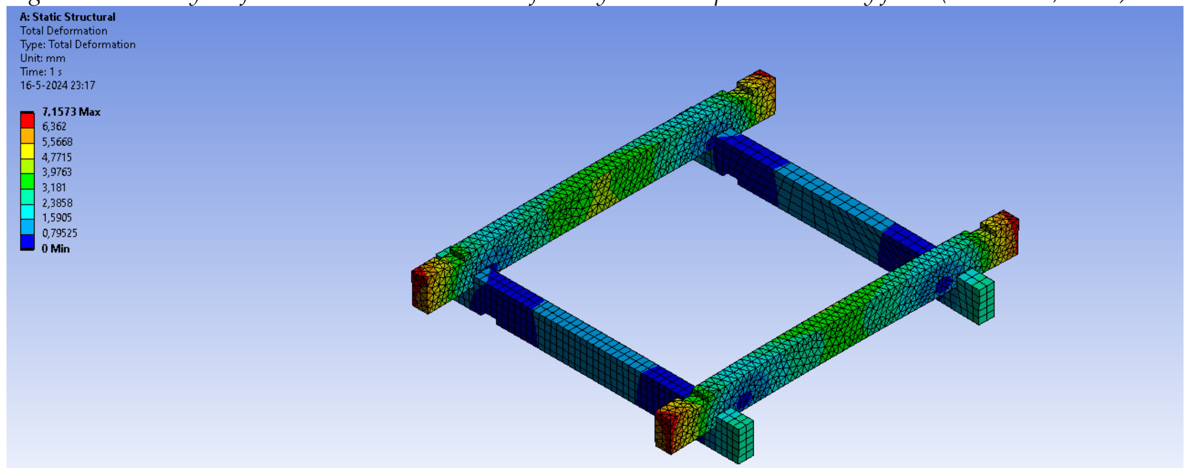


Figure 6.28: Ansys deformation simulation with two layers (own work, 2024)

Then there also is the option to simulate the top five layers (instead of five layers somewhere from the middle), which are then constraint by the floor that would be above it in a building situation, as shown in Figure 6.27. This however resulted in unexplainable peak stresses in the middle of the beams right below the floor.

A last option that was tested, was to run the simulation with two layers instead of five. The result of the deformation simulation is shown in Figure 6.28. Figure 6.25 shows the most deformation in the upper two layers, and this deformation is comparable to what happens in the structure with two layers (see Figure 6.28).

A general issue with all of the options is that it is difficult to see the forces and deformation in the connection itself. This is not the case when running the simulation on just one element, placing the loads and support points then still work the same, but the effect that the elements have on each other cannot be seen here.

For these reasons the joints will be tested in a system of two layers, and for each unique joint also a simulation will be run with just one element to see the stresses and deformation in the connection. There will be images of Ansys simulations in three-dimensional perspective and in two-dimensional view. The three-dimensional perspective is from simulations run on the joints in a two-layered system, the two-dimensional view is from the simulations run on just one element, the image for the latter will be zoomed in to the point on the geometry where the extreme values occur, keep in mind that the element and loads are symmetrical, so what happens on one side happens on the other as well. For both two and three-dimensional images the maximum and minimum of the respective solution will be highlighted.

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Simple square cog

The first simulations are discrete elements with simple square cog joints, for this joint a square cut-out is made in both the lower and upper element which then interlock with each other. This connection constrains translation and rotation on the x-axis and y-axis, movement in de z-direction (up) is still possible, as well as rotation along the z-axis (basically like a seesaw). This is relatively simple joint, also to produce it. Figure 6.29 shows a simple square cog joint for two elements, for the discrete elements of this research the joint is adapted so that

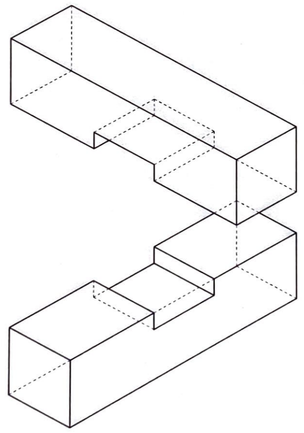


Figure 6.29: Simple square cog joint (Guenoun, 2019)

Total deformation

Minimum
0mm

Maximum
1,2539mm

Average
0,53369mm

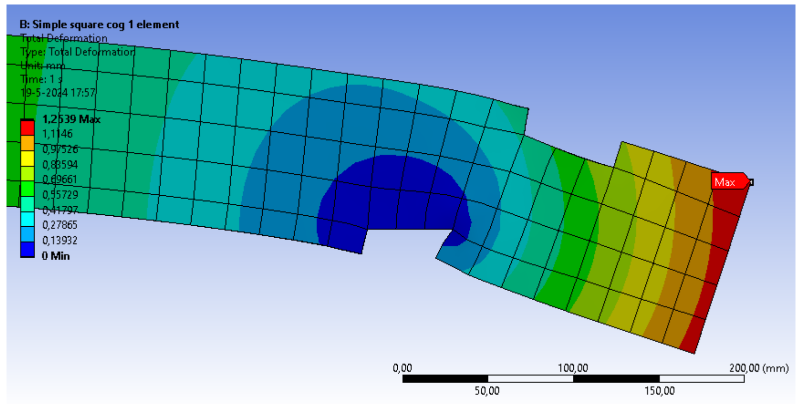


Figure 6.30: Simple square cog Ansys deformation simulation with one element (own work, 2024)

Maximum principal stress

Minimum
-59,915 MPa

Maximum
67,785 MPa

Average
8,2126 MPa

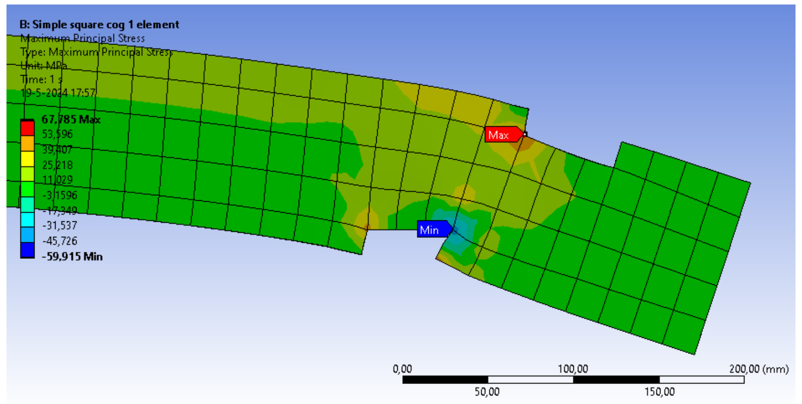


Figure 6.31: Simple square cog Ansys maximum principal stress simulation with one element (own work, 2024)

Normal stress

Minimum
-59,915 MPa

Maximum
18,884 MPa

Average
-0,17311 MPa

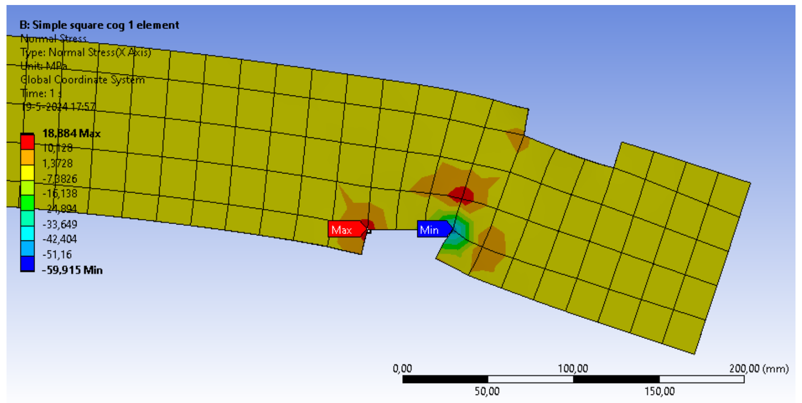


Figure 6.32: Simple square cog Ansys normal stress simulation with one element (own work, 2024)

each element (except the top and bottom ones) have a joint at the bottom and at the top of the element. For both pages, from top to bottom the images show the total deformation (mm), maximum principal stress (MPa) and normal stress (MPa).

Total deformation
The total deformation has it maximum value at the end of the top beam, Figure 6.33 shows clearly the effect that the beams have on each other, as the top beam shows the highest deformation, and the bottom beam in generally more constrained. One of the conclusions from the deformation simulation is the movement the beams tend to make under the applied load cases to where the joints connect - it pushes the part of the beam on the outer side down, but because its connection with the beam below it also pushed the beam to the outside, this is also where the maximum value is in Figure 6.33

Maximum principal stress and normal stress
The maximum (tension) and minimum (compression) values for maximum principal stress and normal stress are localized stresses. Figure 6.31 and Figure 6.32 show that these extremes occur in the joint while the rest of the beam is more towards the average value for the stresses.

Total deformation

Minimum
0mm

Maximum
3,429mm

Average
1,0934mm

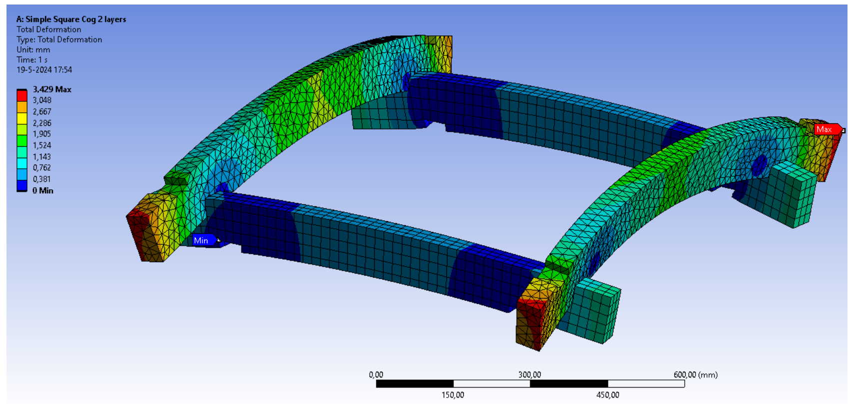


Figure 6.33: Simple square cog Ansys deformation simulation with two layers (own work, 2024)

Maximum principal stress

Minimum
-85,612 MPa

Maximum
189,68 MPa

Average
10,296 MPa

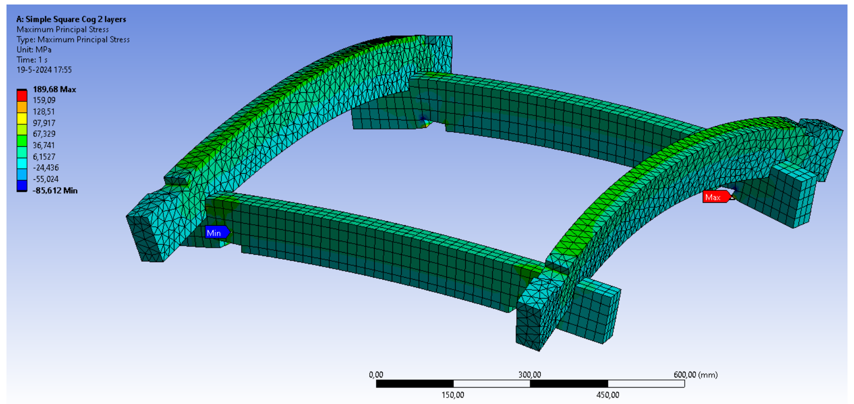


Figure 6.34: Simple square cog Ansys maximum principal stress simulation with two layers (own work, 2024)

Normal stress

Minimum
-238,35 MPa

Maximum
153,13 MPa

Average
2,0934 MPa

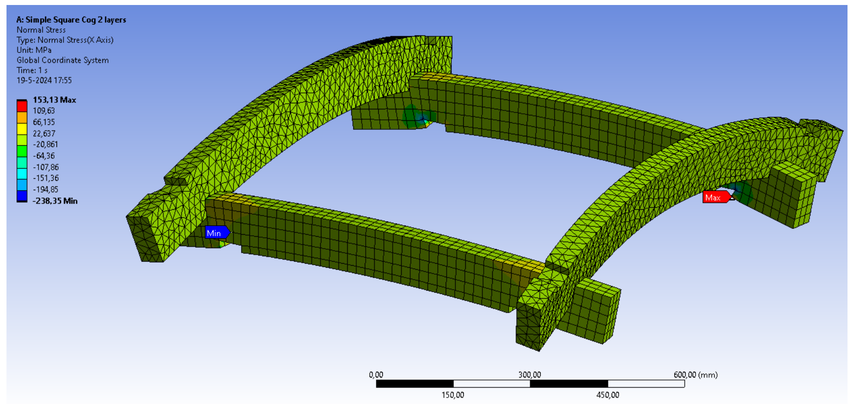


Figure 6.35: Simple square cog Ansys normal stress simulation with two layers (own work, 2024)

Dovetail cog

The next set of images are from simulations run with the dovetail cog joint as shown in Figure 6.36. For this joint, the elements have a small chip carved out on both sides in the top of the element, which the interlocks with the inverse of the shape in the beam above it. The cone shape in one direction and the perpendicular hooks in the other direction prevent translation in the length direction of the lower beam, and also the perpendicular to the length direction, movement and rotation on the z-axis is still free, but can be constrained by adding more elements onto each other. For both pages, from top to bottom the images show the total deformation (mm), maximum principal stress (MPa) and normal stress (MPa).

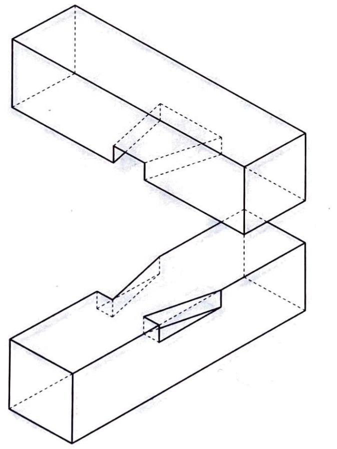


Figure 6.36: Dovetail cog joint (Guenoun, 2019)

Total deformation

Minimum
0mm

Maximum
1,5042mm

Average
0,68594mm

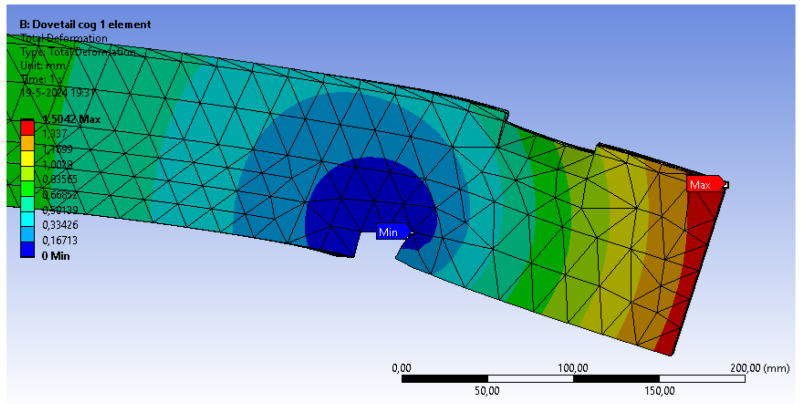


Figure 6.37: Dovetail cog Ansys total deformation simulation with one element (own work, 2024)

Maximum principal stress

Minimum
-89,225 MPa

Maximum
72,453 MPa

Average
9,581 MPa

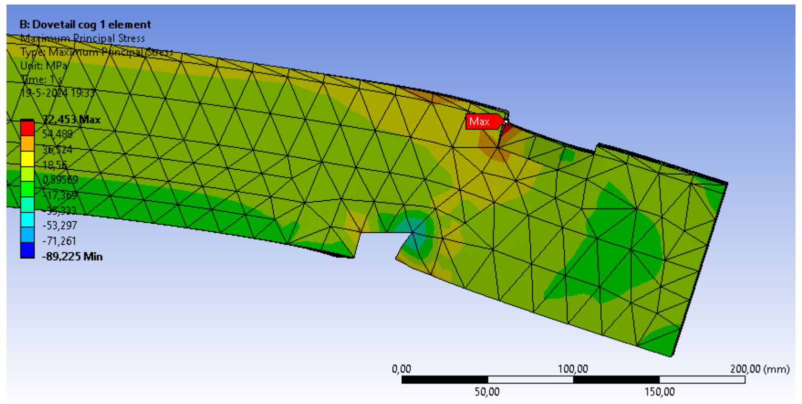


Figure 6.38: Dovetail cog Ansys maximum principal stress simulation with one element (own work, 2024)

Normal stress

Minimum
-91,507 MPa

Maximum
29,006 MPa

Average
-0,21756 MPa

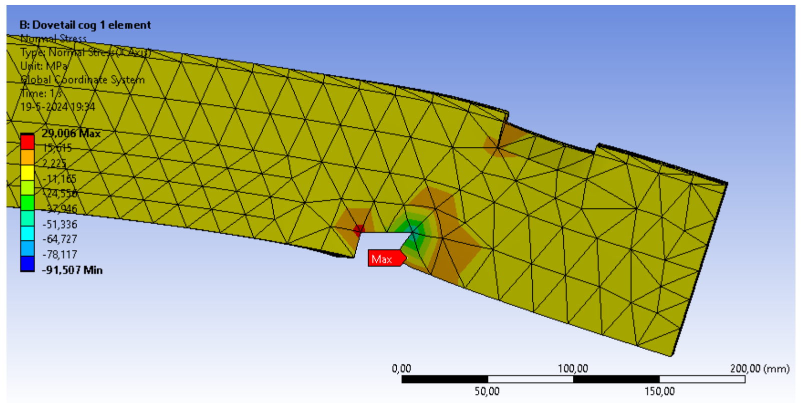


Figure 6.39: Dovetail cog Ansys normalstress simulation with one element (own work, 2024)

Total deformation
The total deformation with this joints is similar to the deformation occuring with the simple square cog joint. In a fully column, where all the beams are constrained by another beam, or a floor, the highest deformations will likely be less (unless a different factor comes into play).

Maximum principal stress and normal stress
The high values for maximum (tension) and minimum (compression) stresses are on highly specific places, which means that the peak stresses occur in a tiny spot on the beam, but even the area around it experiences significantly less stress already. Besides the extremes, the beam shows expected behaviour with in the middle of the beam tension in the upper part and compression in the lower part. The dovetail contains some smaller sized parts in which the high stresses become more crucial because these parts are weaker than thicker parts.

Total deformation

Minimum
0mm

Maximum
3,7813mm

Average
1,1464mm

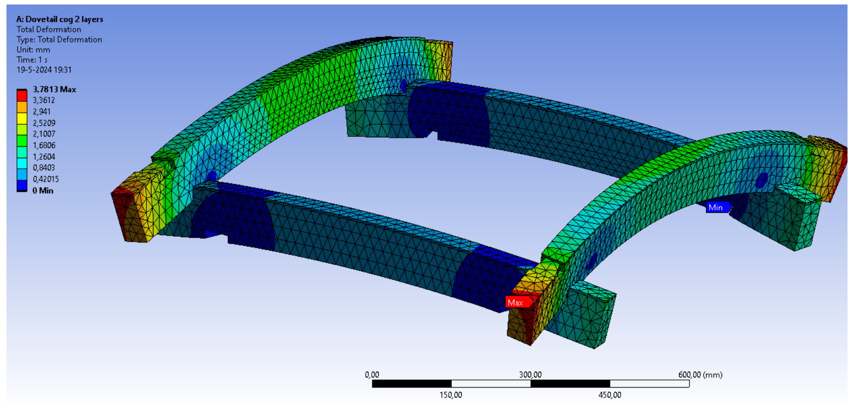


Figure 6.40: Dovetail cog Ansys total deformation simulation with two layers (own work, 2024)

Maximum principal stress

Minimum
-111,29 MPa

Maximum
101,06 MPa

Average
11,003 MPa

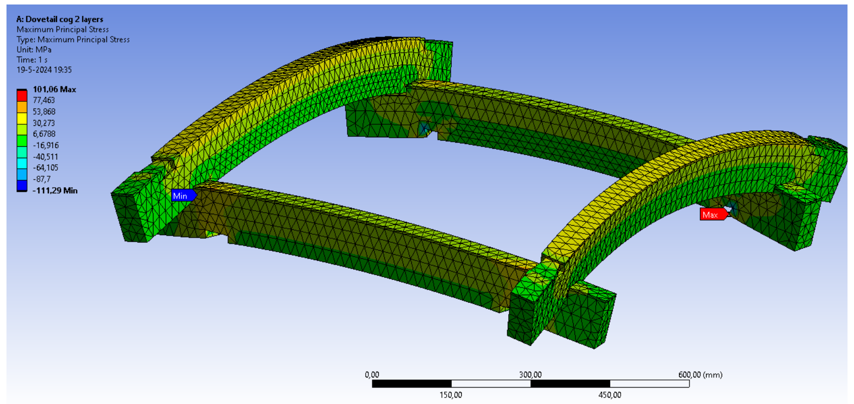


Figure 6.41: Dovetail cog Ansys maximum principal stress simulation with two layers (own work, 2024)

Normal stress

Minimum
-222,64 MPa

Maximum
75,049 MPa

Average
3,1742 MPa

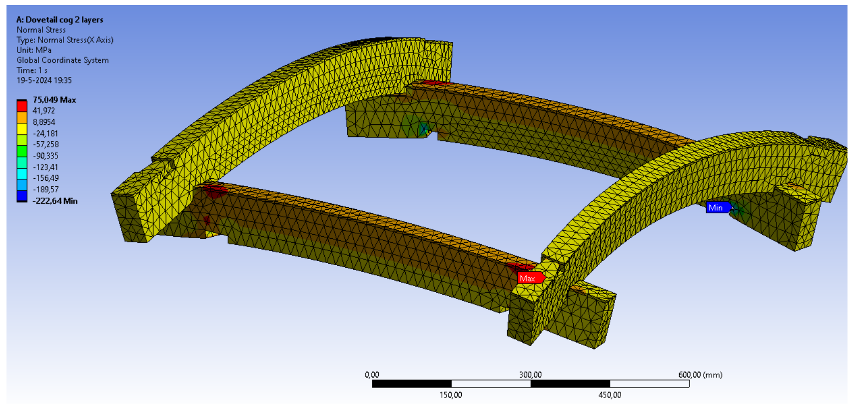


Figure 6.42: Dovetail cog Ansys normal stress simulation with two layers (own work, 2024)

Double cog

The double cog joint (see Figure 6.43) interlocks via a simple rectangular cut-out in the bottom of the upper beam, and the inverse of this in the beam below it. This connections constrains movement in the x and y direction. Because of the interlocking in the top beam in Figure 6.43, this joint is relatively scalable, if this cut out is repeated multiple times the beam can be connected in multiple places. For both pages, from top to bottom the images show the total deformation (mm), maximum principal stress (MPa) and normal stress (MPa).

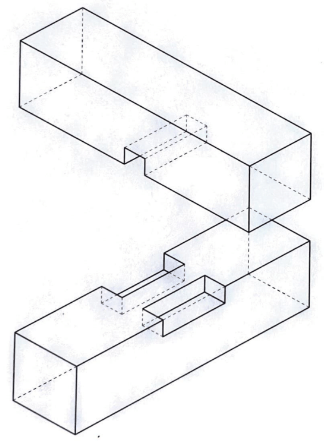


Figure 6.43: Double cog joint (Guenoun, 2019)

Total deformation

Minimum
0mm

Maximum
0,99612mm

Average
0,4169mm

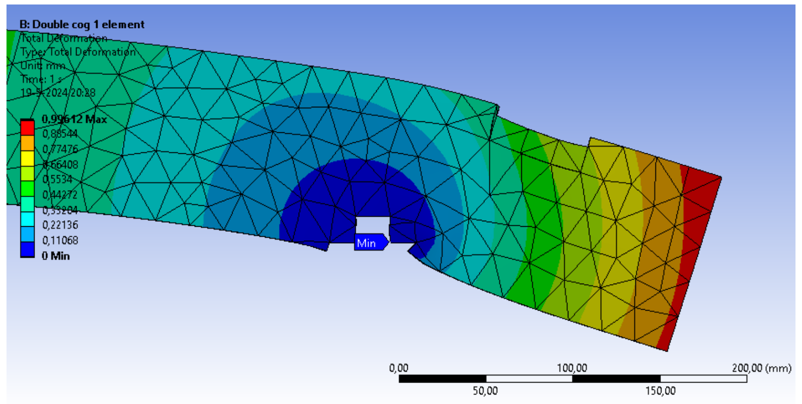


Figure 6.44: Double cog Ansys total deformation simulation with one element (own work, 2024)

Maximum principal stress

Minimum
-72,093 MPa

Maximum
52,587 MPa

Average
7,2063 MPa

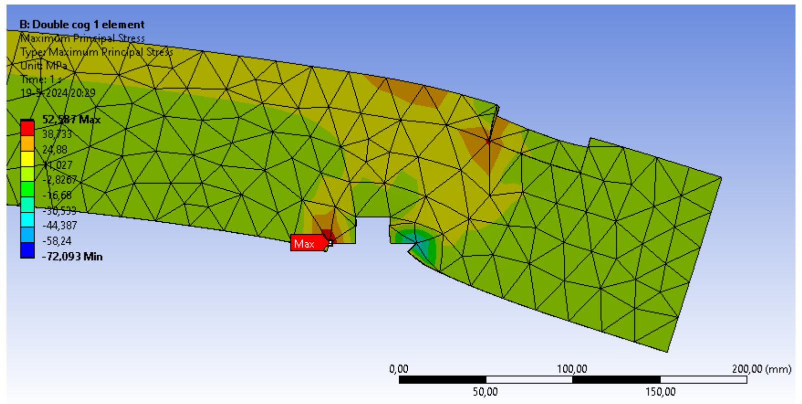


Figure 6.45: Double cog Ansys maximum principal stress simulation with one element (own work, 2024)

Normal stress

Minimum
-78,538 MPa

Maximum
28,87 MPa

Average
-0,28391 MPa

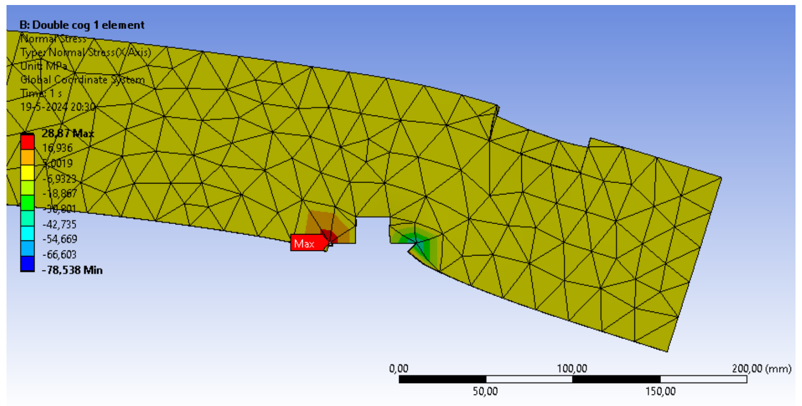


Figure 6.46: Double cog Ansys normal stress simulation with one element (own work, 2024)

Total deformation

The deformation simulation yields values within acceptable bounds, again. Shapewise and also result-wise this joint shares similarities with the simple square cog.

Maximum principal stress and normal stress

Even though this joint shares similarities with a simple square cog, the smaller middle part in the bottom beam in Figure 6.43 can be a critical spot for the stresses, just as it is a flaw in the dovetail lap joint (after this one). Figure 6.48 and Figure 6.49 support this by showing the maximum stress value in the joint.

Total deformation

Minimum
0mm

Maximum
3,1785mm

Average
0,91817mm

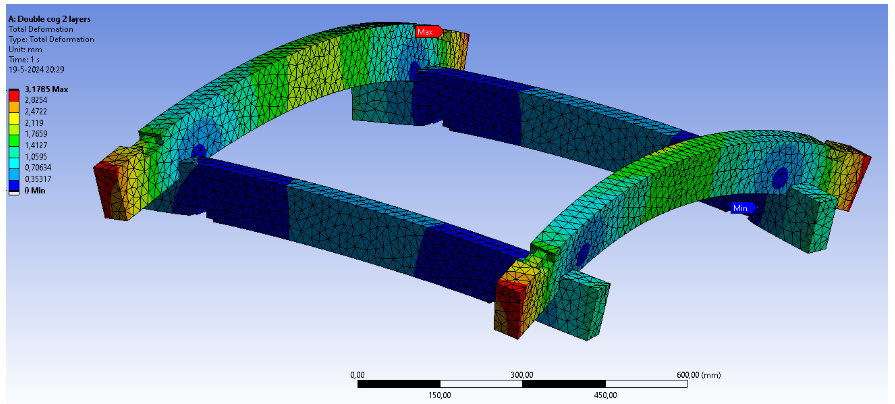


Figure 6.47: Double cog Ansys total deformation simulation with two layers (own work, 2024)

Maximum principal stress

Minimum
-121,03 MPa

Maximum
82,299 MPa

Average
9,3537 MPa

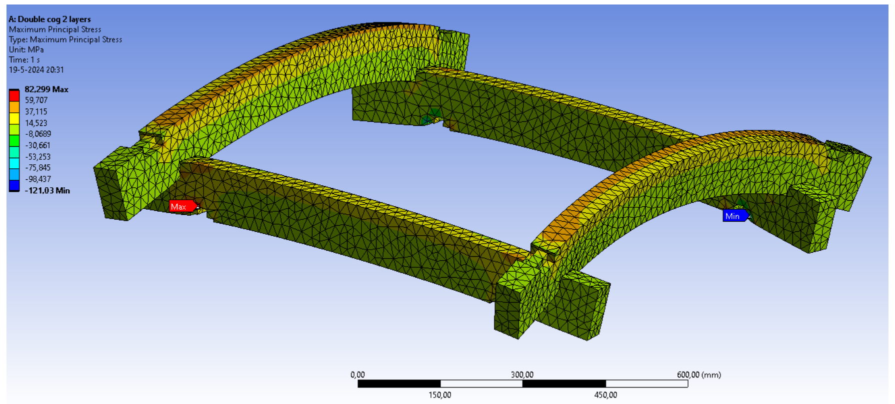


Figure 6.48: Double cog Ansys maximum principal stress simulation with two layers (own work, 2024)

Normal stress

Minimum
-258,45 MPa

Maximum
77,443 MPa

Average
2,0732 MPa

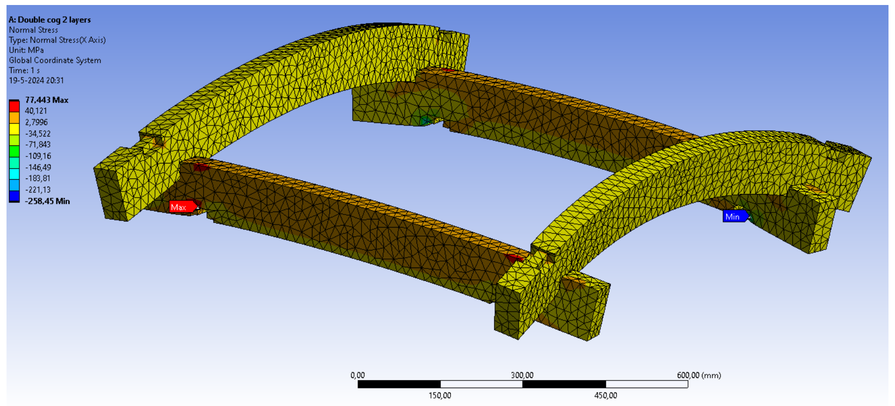


Figure 6.49: Double cog Ansys normal stress simulation with two layers (own work, 2024)

Dovetail lap

The dovetail lap joint is an own adoption as combination between a dovetail cog and an offset lap joint. It makes for an interesting geometry with a large interlocking (see Figure 6.50). This interlocking prevents movement and rotation in the x and y-direction, but not in the z-direction. For both this and the next page, from top to bottom the images show the total deformation (mm), maximum principal stress (MPa) and normal stress (MPa).

Between the simulations run with one element versus with two layers there is a clear and large

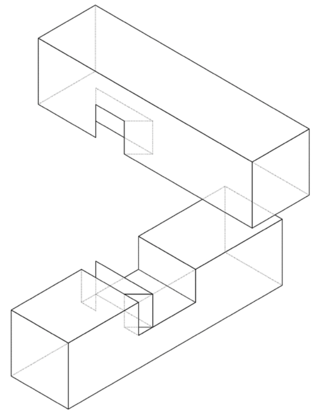


Figure 6.50: Dovetail lap joint (Own work, 2024)

Total deformation

Minimum
0mm

Maximum
0,81609mm

Average
0,34089mm

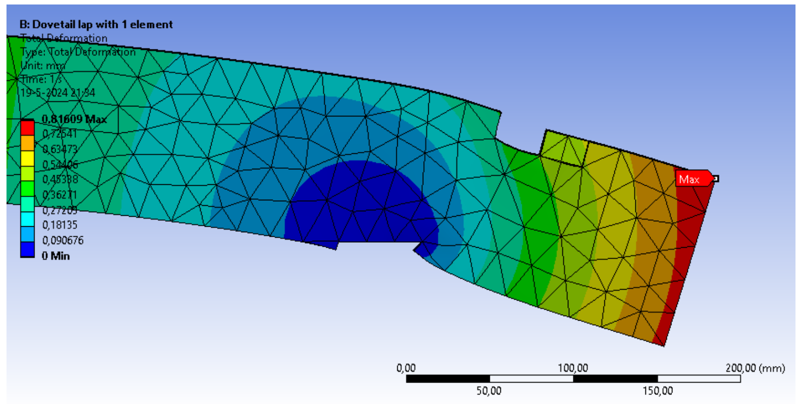


Figure 6.51: Dovetail lap Ansys total deformation simulation with one element (own work, 2024)

Maximum principal stress

Minimum
-82,169 MPa

Maximum
51,107 MPa

Average
5,6949 MPa

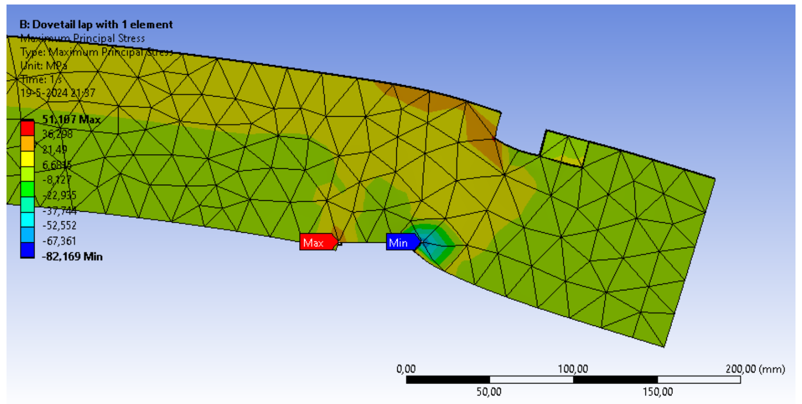


Figure 6.52: Dovetail lap Ansys maximum principal stress simulation with one element (own work, 2024)

Normal stress

Minimum
-90,482 MPa

Maximum
28,61 MPa

Average
-0,35361 MPa

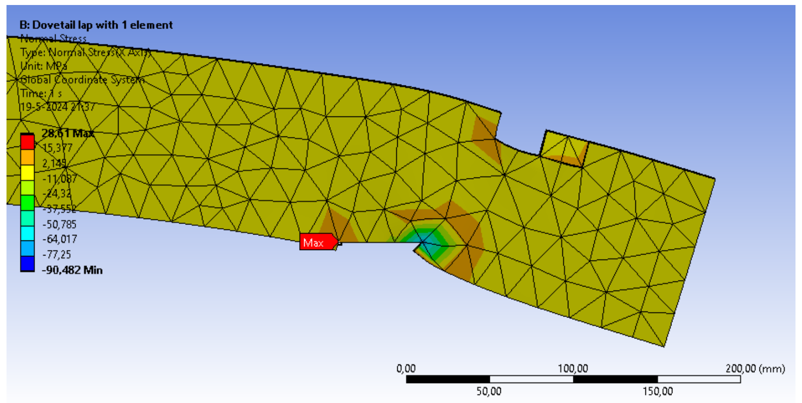


Figure 6.53: Dovetail lap Ansys normal stress simulation with one element (own work, 2024)

difference in the minimum and maximum values. In the simulation with two layers the deformation is factor 10 larger than the deformation calculated with one element, and factor 3 compared to results for previous joints. The stresses are even in the range of factor 50 (or even 70) compared to the stresses calculated in one element, and factor 30-40 compared with previous results.

Also visually, in the images of the simulations with two layers (Figure 6.54, Figure 6.55, and Figure 6.56) show something notable, namely a knot with the largest deformations and stresses in the connection. This also explains the difference between the results from calculating with one element versus two layers, in the simulations with one element there is no stress on the joints from the other layers, whereas in the simulation with two layers this is the case. Which can lead to conclude that the dovetail lap joint is not strong enough for this use case.

Total deformation

Minimum
0mm

Maximum
8,9693mm

Average
1,4244mm

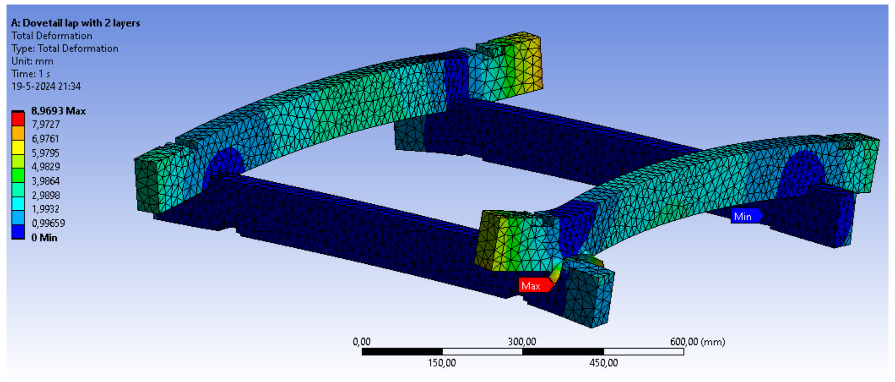


Figure 6.54: Dovetail lap Ansys total deformation simulation with two layers (own work, 2024)

Maximum principal stress

Minimum
-4316,4 MPa

Maximum
4378 MPa

Average
26,097 MPa

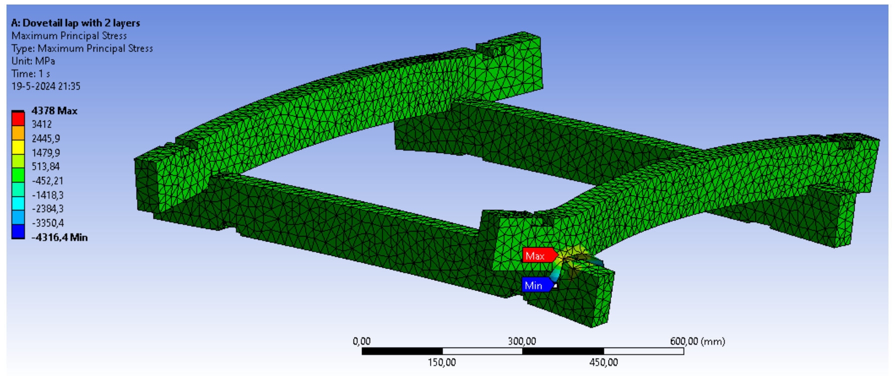


Figure 6.55: Dovetail lap Ansys maximum principal stress simulation with two layers (own work, 2024)

Normal stress

Minimum
-6429,5 MPa

Maximum
4312,4 MPa

Average
-0,28905 MPa

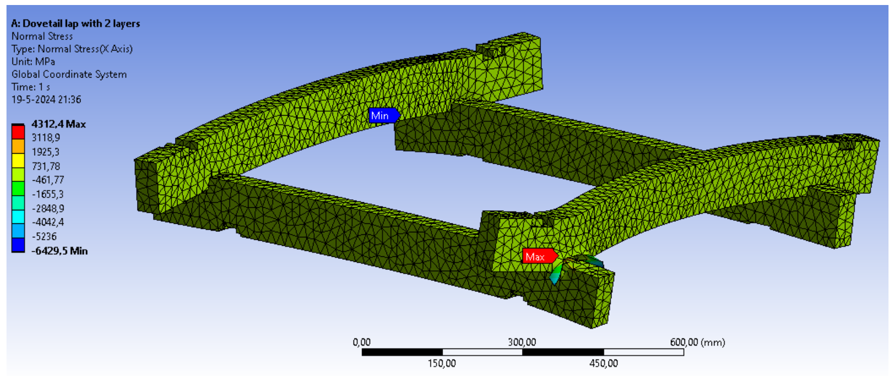


Figure 6.56: Dovetail lap Ansys normal stress simulation with two layers (own work, 2024)

Inverse double cog

The inverse double cog (Figure 6.57) is an own adaptation as well. As a twist to the double cog described and tested earlier. However, this joint has the flexibility to connect the upper beam anywhere on the lower beam because the single slot in the bottom of the beam is continuous over the length of the beam. For this to be possible in this way movement is only prevented in one direction - the element can slide in the other direction, rotation is still prevented in both the x, and y-direction. The slot can also be produced in a discontinued matter, then the beams cannot be connected at any distance along the beam anymore, but rather on predefined locations. For both pages, from top to bottom the images show the total deformation (mm), maximum principal stress (MPa) and normal stress (MPa).

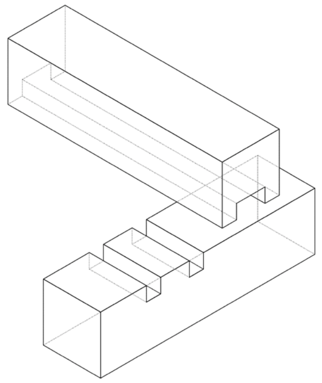


Figure 6.57: Inverse double cog (Own work, 2024)

Total deformation

Minimum
0mm

Maximum
1,6722mm

Average
0,51576mm

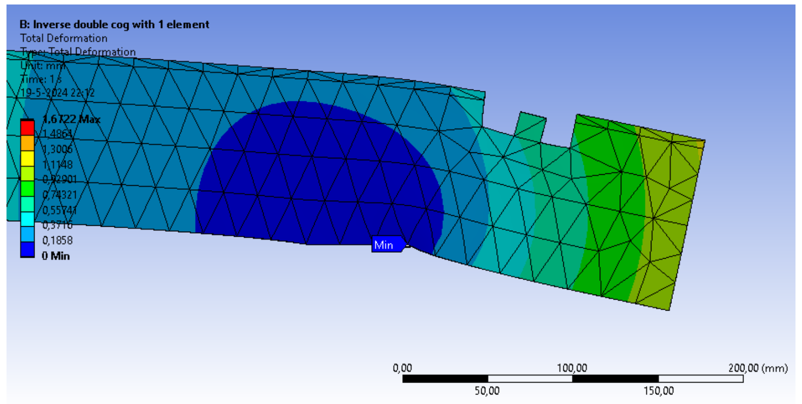


Figure 6.58: Inverse double cog Ansys total deformation simulation with one element (own work, 2024)

Maximum principal stress

Minimum
-65,961 MPa

Maximum
82,493 MPa

Average
8,7665 MPa

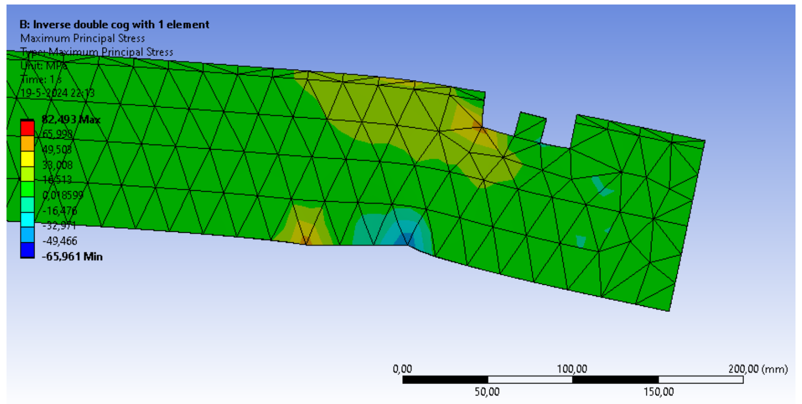


Figure 6.59: Inverse double cog Ansys maximum principal stress simulation with one element (own work, 2024)

Normal stress

Minimum
-84,616MPa

Maximum
32,854 MPa

Average
-0,20431 MPa

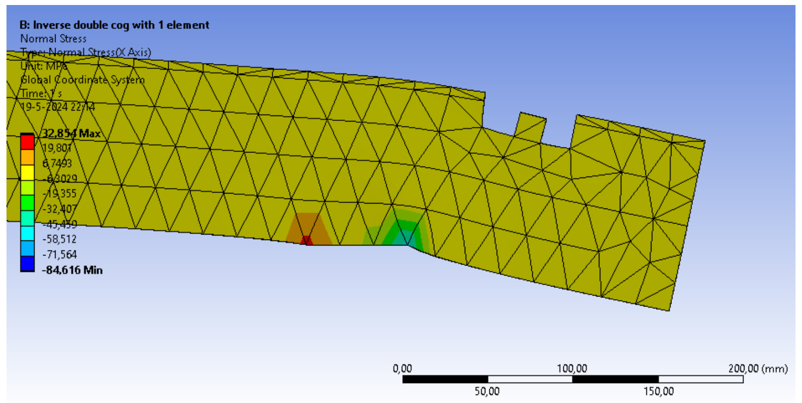


Figure 6.60: Inverse double cog Ansys normal stress simulation with one element (own work, 2024)

Total deformation

Unlike the deformation in the previous four joint options, the deformation for this joint is not uniform. Which means that the maximum deformation that is in the end of the left top beam, is not as high in any of the other ends of the top beam. This is because of the contact option setting in Ansys, which is set to frictional and allows frictional sliding. The loads and support points have been placed using the mesh as a guide which likely caused the supports to be placed asymmetrical and thus the deviation.

Maximum principal stress and normal stress

This asymmetrical issue with the deformation does not occur in the maximum principal stress and normal stress, supporting the logic that the support points are not symmetrical and pulling slightly more to one side. Besides that, there are not any remarkable or unusual occurrence in the stresses. The highest stress occur in the connection, with the compression where the element is pushed into the connection/ support below, and the tension on the opposite side of this compression (which is expected, compression on one side causes tension on the other). Looking at the elements there also is tension and compression where it is expected.

Total deformation

Minimum
0mm

Maximum
6,0444mm

Average
1,8508mm

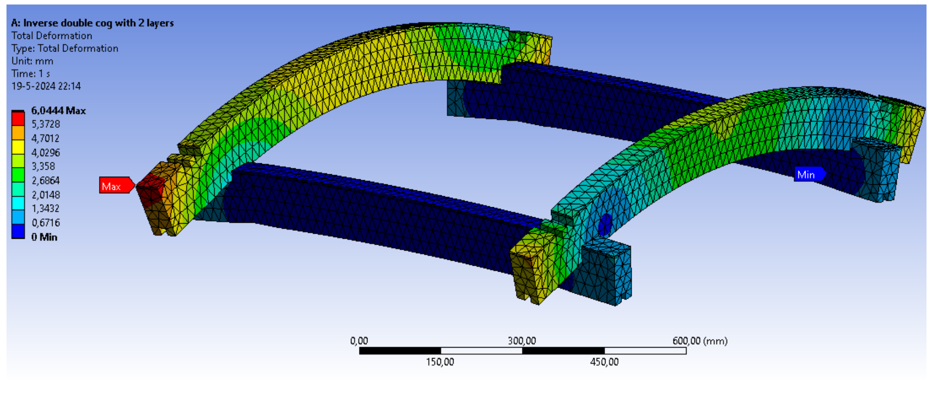


Figure 6.61: Inverse double cog Ansys total deformation simulation with two layers (own work, 2024)

Maximum principal stress

Minimum
-71,707 MPa

Maximum
128,45 MPa

Average
11,62 MPa

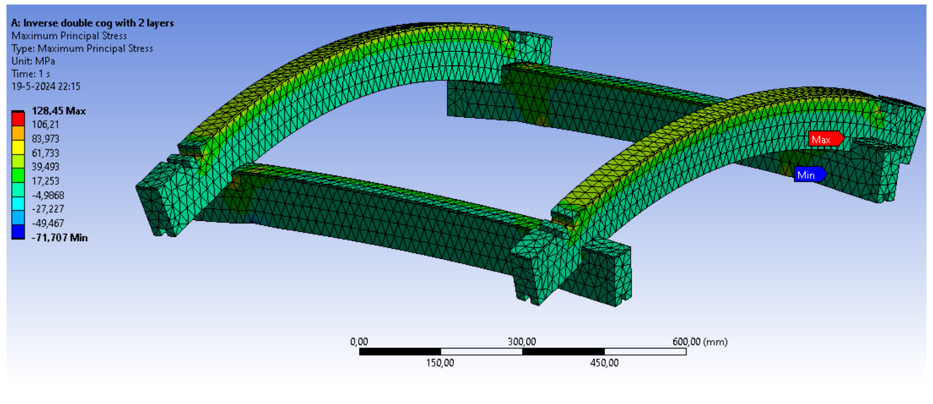


Figure 6.62: Inverse double cog Ansys maximum principal stress simulation with two layers (own work, 2024)

Normal stress

Minimum
-187,12 MPa

Maximum
60,766 MPa

Average
2,508 MPa

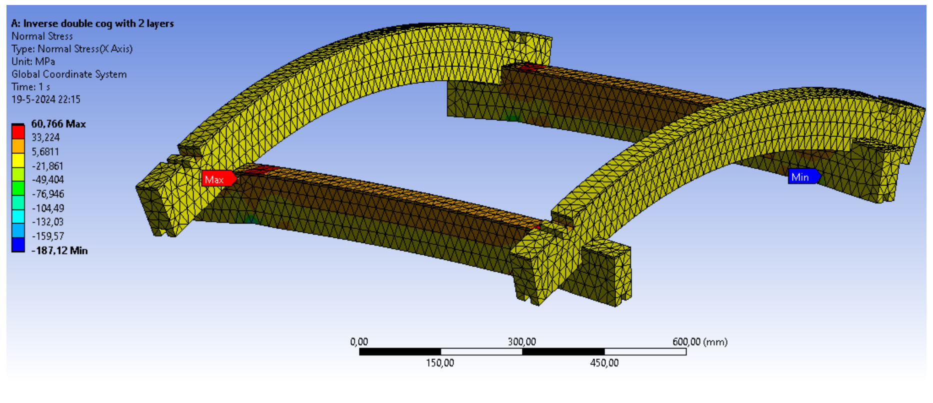
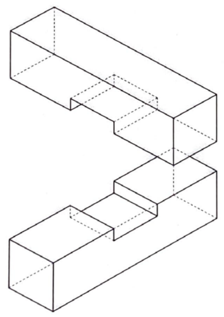


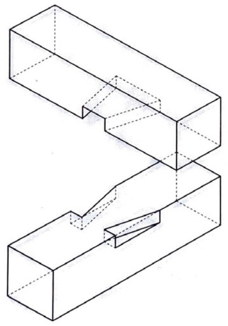
Figure 6.63: Inverse double cog Ansys normal stress simulation with two layers (own work, 2024)

Summarized results single element simulations



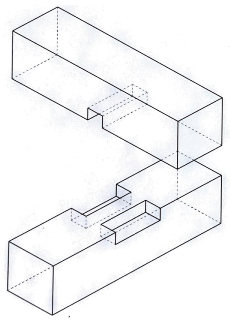
	Simple square cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-59,915 MPa*	*-59,915 MPa*
Maximum	*1,254 mm*	*67,785 MPa*	*18,884 MPa*
Average	*0,554 mm*	*8,213 MPa*	*-0,173 MPa*

Figure 6.64: Simple square cog joint (Guenoun, 2019)



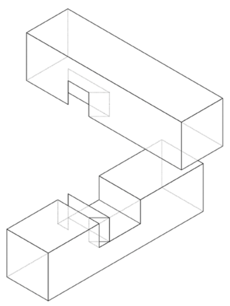
	Dovetail cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-89,225 MPa*	*-91,507 MPa*
Maximum	*1,504 mm*	*72,453 MPa*	*29,006 MPa*
Average	*0,686 mm*	*9,581 MPa*	*-0,2176 MPa*

Figure 6.65: Dovetail cog joint (Guenoun, 2019)



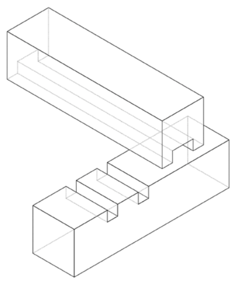
	Double cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-72,093 MPa*	*-78,538 MPa*
Maximum	*0,996 mm*	*52,587 MPa*	*28,870 MPa*
Average	*0,417 mm*	*7,206 MPa*	*-0,284 MPa*

Figure 6.66: Double cog joint (Guenoun, 2019)



	Dovetail lap		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-82,169 MPa*	*-90,482 MPa*
Maximum	*0,816 mm*	*51,107 MPa*	*28,610 MPa*
Average	*0,341 mm*	*5,695 MPa*	*-0,354 MPa*

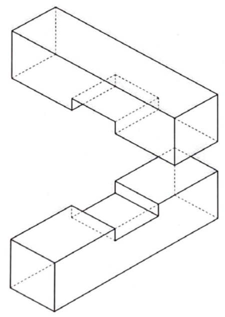
Figure 6.67: Dovetail lap joint (Own work, 2024)



	Inverse double cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-65,961 MPa*	*-84,616 MPa*
Maximum	*1,672 mm*	*82,493 MPa*	*32,854 MPa*
Average	*0,516 mm*	*8,767 MPa*	*-0,204 MPa*

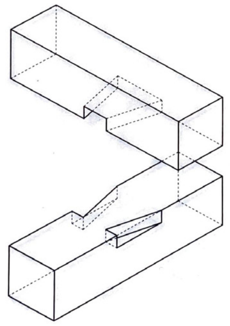
Figure 6.68: Inverse double cog (Own work, 2024)

Summarized results two layers simulations



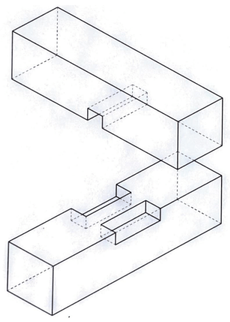
	Simple square cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-85,612 MPa*	*-238,35 MPa*
Maximum	*3,429 mm*	*189,68 MPa*	*153,13 MPa*
Average	*1,093 mm*	*10,296 MPa*	*-2,093 MPa*

Figure 6.69: Simple square cog joint (Guenoun, 2019)



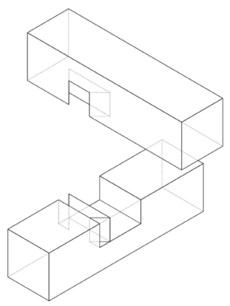
	Dovetail cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-111,29 MPa*	*-222,64 MPa*
Maximum	*3,781 mm*	*101,06 MPa*	*75,049 MPa*
Average	*1,164 mm*	*11,003 MPa*	*3,174 MPa*

Figure 6.70: Dovetail cog joint (Guenoun, 2019)



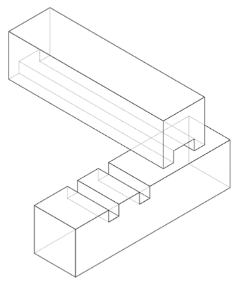
	Double cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-121,03 MPa*	*-258,45 MPa*
Maximum	*3,179 mm*	*82,299 MPa*	*77,443 MPa*
Average	*0,918 mm*	*9,354 MPa*	*2,073 MPa*

Figure 6.71: Double cog joint (Guenoun, 2019)



	Dovetail lap		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-4316,4 MPa*	*-6429,5 MPa*
Maximum	*8,969 mm*	*4378 MPa*	*4312,4 MPa*
Average	*1,424 mm*	*26,097 MPa*	*-0,289 MPa*

Figure 6.72: Dovetail lap joint (Own work, 2024)



	Inverse double cog		
	Total deformation	**Maximum principal stress**	**Normal stress**
Minimum	*0 mm*	*-71,707 MPa*	*-187,12 MPa*
Maximum	*6,044 mm*	*128,45 MPa*	*60,766 MPa*
Average	*1,581 mm*	*11,62 MPa*	*2,508 MPa*

Figure 6.73: Inverse double cog (Own work, 2024)

6.6 Transferring loads from columns to existing structure

The discrete timber system is load bearing, but beside it carrying loads it also needs to transfer these loads, from floor to floor, and from the top-up to the existing load bearing structure. Especially the latter can see issues, as the loads in the bottom of the discrete timber column are more like point loads, and they are transferred onto walls that transfer their loads as uniformly distributed loads, as visualized in Figure 6.74. This means that the forces occurring where the discrete timber system meets the existing load bearing structure meets will be high, and thus need to be designed accordingly to prevent the concrete from crushing under the loads.

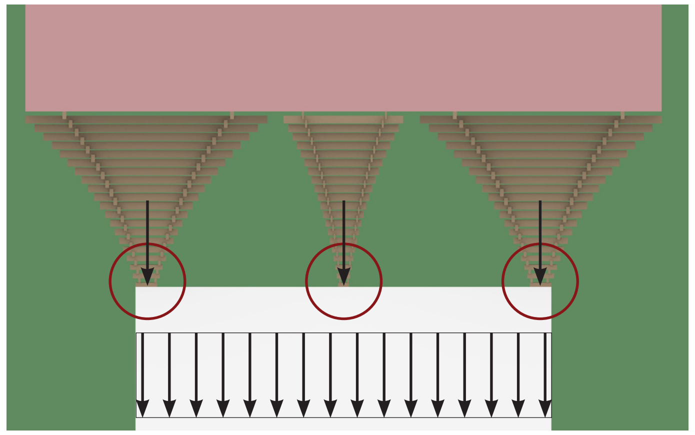


Figure 6.74: Side view of the discrete timber columns on the existing structure (own work, 2024)

Where the aim of the discrete timber system is to create a timber-only system, it seems not possible to have the connection between the discrete timber system and existing load bearing structure made of timber too. The most feasible option for this is to create (custom) steel brackets that are attached to the concrete load bearing structure of the existing building, in which the bottom two discrete timber elements from the discrete timber system are connected, an example solution is shown in Figure 6.75.

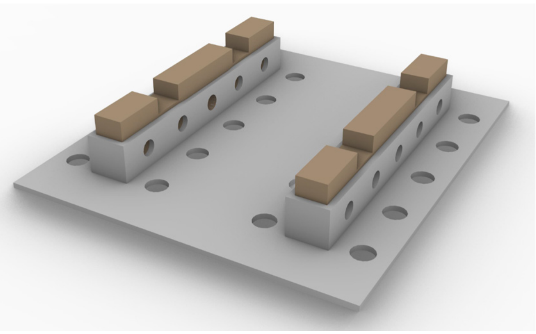


Figure 6.75: Custom steel bracket for discrete timber system (own work, 2024)

6.7 Conclusions

General remarks

The two layers simulation shows clearly what the effect is that the beams have on each other with regards to constraint. The peak stresses occur in the upper beam, while the lower beam (which is constraint by the upper beams) shows results that are within acceptable bounds. Computational limitations obstruct the possibility to see simulate a full column in Ansys and see how this column might work together as one element (such as the two layers show).

The peak stresses are localized stresses, meaning they only occur in this magnitude on specific locations, generally in the joints. From which the conclusion can be drawn that the joints are not strong enough. The rest of the beam shows expected structural behaviour under this loading. The loads are applied on the outsides of the beam, causing the ends to move down, the middle of the beam to move up and thus creating tension in the top of the beam and compression in the bottom of the beam. So the peak stresses can potentially be decreased by redesigning or strengthening the joint.

The support in both simulations were set to be a point support because the elements are assumed to be from somewhere in the middle of the column (and thus only two support points per side). However, for the bottom most beam the support in continuous over the length of the beam, see Figure 6.76

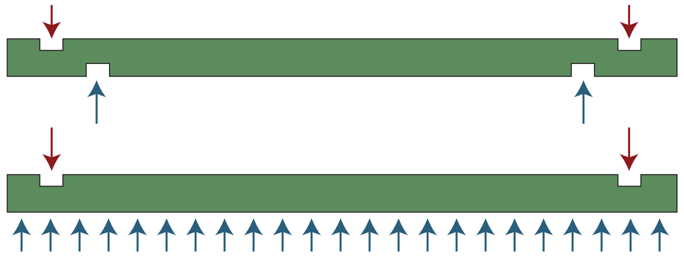


Figure 6.76: Point support vs full beam support (own work, 2024)

The other way around this is also the case for the top most beams and with the loading, where throughout the column it is with point loads, the topmost beam is continuously loaded over the length of the beam. In these situations it could be that the peak stresses are differently divided than how it is in the simulations ran now.

The weakest point is generally in the joint (Blaß & Sandhaas, 2017), this becomes additionally crucial when there are small parts in the joints and the unfavourable result of this we can see for the dovetail lap joint.

If we look at the joint geometry that was used in the Ansys simulation, there is only one joint type that is continuous, meaning that it can be connected to the beam below (and above it) anywhere along the length of the beam. This is the inverse double cog joint, in the bottom of this joint is a cut-out along the middle of the beam which can interlock at any point on this beam, see the figure below.

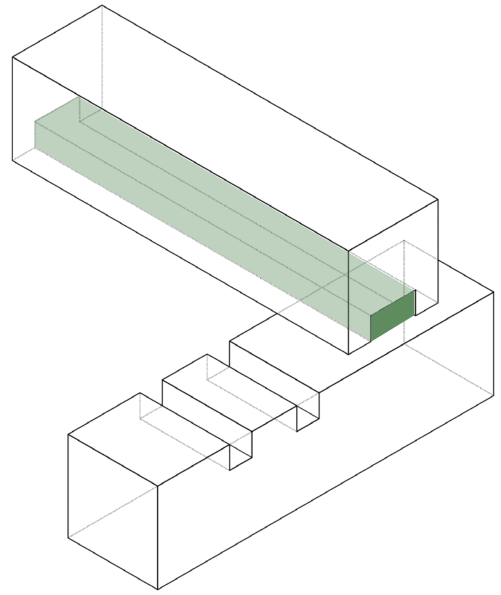


Figure 6.77: Inverse double cog (Own work, 2024)

This is a useful feature for flexibility in the joint, because one beam of 1000mm can be used to make a column of 800mm by 800mm but also 600mm by 600mm for example. However, this joint only constrains the beams in one direction, leaving it the ability to slide which can greatly influence the stability if there are not stability elements applied.

For the other joints to be more flexible the joint needs to be made on different spots over the length of the beam, such as in Figure 6.78. The advantages of this over a continuous joint is that it is constrained in two directions instead of one.

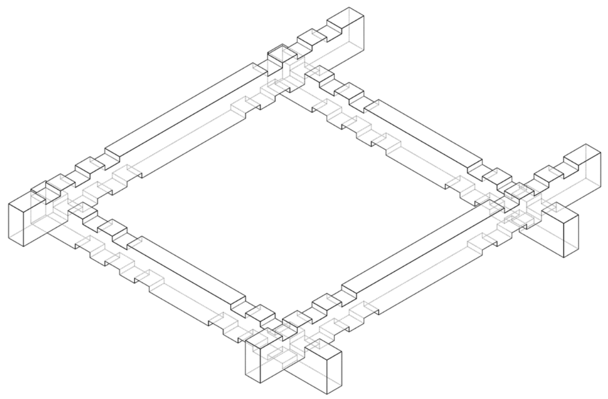


Figure 6.78: Simple square cog joint with multiple connection possibilities (own work, 2024)

Simulation conclusions

On page 66 and page 67 the results from simulating the single element and two layers build ups from each of the five joints have been summarized. For each of the three solutions (total deformation, maximum principal stress, and normal stress), the lowest and second lowest value in each category (minimum, maximum and average value) have been highlighted. The decision to not only mark the lowest but also the second lowest is because for the single element simulations the joint is not loaded and this results in the dovetail lap joint being one of the better options. However, the images from the dovetail lap joint already showed that the joint is the flaw, and by looking at the numeric results from the two layers simulations we can see that the dovetail lap joint is indeed not a suitable option. The second lowest therefore provides an alternative in the situation where the lowest option is not suitable. The lowest value is highlighted in green, the second lowest value in orange.

Eventually the results for both simulations are combined to see which joint option is most suitable from the five that were simulated.

With the dovetail lap joint being ruled out, the simple square cog and double cog joint are the most promising options. The simple square cog scores better in the single element simulations and the double cog in the two layers simulations. For both joints and both simulations the deformation falls within acceptable range of (l/333), so this aspects becomes less important.

Additionally, as stated before, the minimum and maximum stresses are localized stresses, this statement is strengthened by the lower and within-bounds average stress value. While it is important to look at the minimum and maximum values, only the maximum normal stress in the single element simulations is within acceptable bounds for timber with strength class D30, the other values are all too high and would break the joint. The most suitable option would therefore be the simple square cog joint.

7. Aggregating discrete timber elements

Chapter 6 focussed on the element level of the discrete system, so discrete elements. Analysing the various discrete elements and testing joints that can be used to connect these discrete elements to each other, also called aggregation. The aggregation was however limited to, initially, five layers of elements, but eventually only two layers of elements, with the aim to learn about the deformation and stresses on element level. This chapter continuous to build on this knowledge by looking at different ways of aggregating the discrete timber elements into various discrete timber systems.

7.1 Aggregation of discrete timber elements

Discrete elements can be stacked in an infinite amount of ways, each way yielding different results for stress on, and deformation of the elements in the discrete system. What is basically comes down to is how the load flows through the discrete system from top to bottom, as shown in Figure 7.1, where 1 denotes the flow of loads for that discrete system, and 2 denotes a more shallow option (elements more in a straight line), and 3 a wider options for the flow of loads.

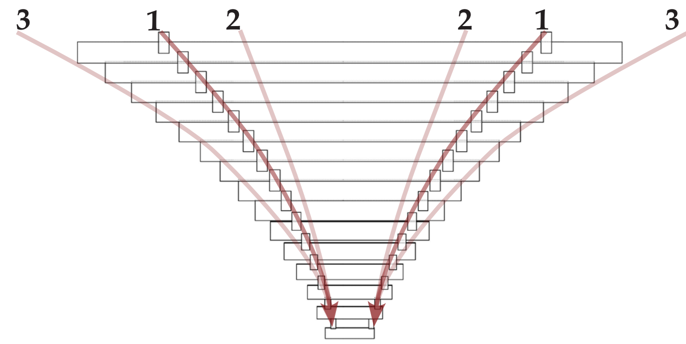


Figure 7.1: Side view of a discrete system column with lines how the load flows (own work, 2024)

If the supports are further away (inwards) from the where the load above it is, the deflection and stresses will be higher. Figure 7.2 and Figure 7.3 show a beam (5m long) with two support points (purple circle) and a load of 1 kN at the endpoints of the beam. Figure 7.2 has the support points closer to where the load is applied which results in a maximum deflection of 0,11 cm. In Figure 7.3 the support points have been moved more inwards, resulting in the applied load being further away from the support points, yielding a deflection of 2,51 cm. These figures show a schematic visualization of what would happen if the flow of loads are like option 3 instead of 1 or 2 (in Figure 7.1). Since there can be infinite aggregations it is not possible to test them all, rather as a first step, the three options shown in Figure 7.1 will be tested so a wide, narrow and in between column.

7.2 Grasshopper script set-up

To get structural results on the different aggregations, Grasshopper, a parametric modelling tool within Rhinoceros (a Computer Aided Design (CAD) software), is used with the help of Karamba3D (a plug-in within Grasshopper). The usefulness in using a parametric modelling tool in this case is that after the script is set-up in Grasshopper, only changes need to be made that tweak the steepness of the load flow, and the deformation and stress results follow 'automatically'.

The Grasshopper script can be divided into five parts (see Appendix B) that each add to a significant feature of the result (a more elaborate description is added when each part of the script is explained in the appendix);

1. generating horizontal lines that can be used to make the discrete elements in the Karamba3D part;
2. a script that divides the lines generated in point 1 at the places on the beam where the connection to the beam below and/or above is;
3. generating vertical lines between the discrete elements that function as the connection between the different layers;
4. the structural analysis in Karamba3D;
5. visualisation of the discrete system column.

Over time quite some changes have been made to the script because the final results it yielded were a lot higher than expected.

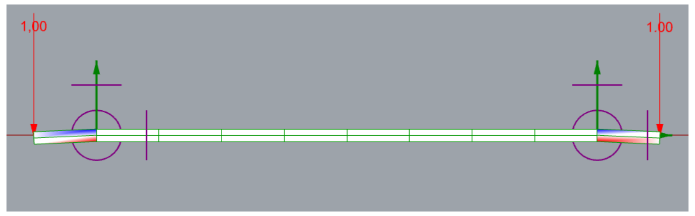


Figure 7.2: Deflection and stress on a beam with supports close to the load (adapted from Karamba3D, n.d.)

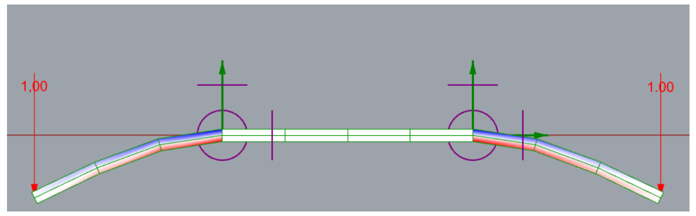


Figure 7.3: Deflection and stress on a beam with supports further from the load (adapted from Karamba3D, n.d.)

First, the model is put together from horizontal lines imitating the beams, and vertical lines imitating the connections between these beams, however the vertical lines can only connect to the horizontal line at a point on this horizontal line (see Figure 7.4). This does not happen when the horizontal beams consist of just one continuous line because then it only has point at both ends. Thus the continuous horizontal line was divided into segments (more thoroughly explained in part 2 of Appendix B).

Figure 7.4: Segmenting horizontal lines to connect vertical lines (own work, 2024)

A second problem occurred with the applied loads on the topmost beam, the total load on the column is calculated to be 296,3 kN, this is divided over two beams meaning 148,15 kN per side. The script takes the length (in m) of the topmost beam and divides 148,15 kN with the length to get the load in kN/m. However, upon applying the load in kN/m to the model, more values appeared than what was expected, e.g. a beam of 6 meters would get 8 values instead of 6. This shows that kN/m is actually not applied per meter. As a workaround method for this, the loads are applied as point loads on top of the connections, just like shown in Figure 6.22.

7.3 Grasshopper simulations

Baseline case

First, a baseline for the column is determined, this will be narrow column (close to straight) with linear scaling from one up to factor 1,5. The Grasshopper settings for this simulation are as follows:

- the base square is 0,5 by 0,5m;
- column height is 3m;
- distance per element (layer) is 0,1m
- cross section is 0,1m high by 0,05m wide;
- the Karamba3D material is set to 'Hardwood'-family with 'D30 parallel'-type;
- the graph mapper scaling factors are shown in Figure 7.5.

For the baseline, but also for the other simulations that will follow after this one, the loads and supports will not change, but the aspects mentioned with the bullet points above will be used to see which aggregation and structure build-up is a more acceptable solution. The figures are named 'cross section view' which is a reference to the respective view setting in the Karamba3D 'Beam View'-component.

1
1.017241
1.034483
1.051724
1.068966
1.086207
1.103448
1.12069
1.137931
1.155172
1.172414
1.189655
1.206897
1.224138
1.241379
1.258621
1.275862
1.293103
1.310345
1.327586
1.344828
1.362069
1.37931
1.396552
1.413793
1.431034
1.448276
1.465517
1.482759
1.5

Figure 7.5: Grasshopper baseline column graph mapper scaling factors per layer (own work, 2024)

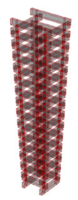


Figure 7.6: Cross section view of the baseline column (own work, 2024)

Figure 7.6 shows the resulting column, it weighs 120 Kg, the resulting maximum displacement is 215,3 cm, and a maximum utilization value of 1,6%. The topmost beams of this column are 0,75m long.

Comparing a narrow, semi-wide, and wide flow of loads aggregations

For comparing the three options: narrow, normal and wide column, the only setting subject to change is the graph mapper and thus the scaling factor for each layer. This way the sole effect from different scaling factors can be seen. The three options are analysed for the following parameters: weight, maximum displacement value (which is measured at the mid-points of the elements), utilization, and length of the topmost beam.

Narrow column
The first option is a narrow column with a highest scaling factor of approximately 3,34, see Figure 7.7 for the full list of scaling factors per layer - the scaling is close to linear.

1
1.075921
1.15232
1.229171
1.306451
1.384138
1.462213
1.54066
1.619463
1.698607
1.77808
1.857868
1.937962
2.01835
2.099023
2.179972
2.261188
2.342663
2.424391
2.506363
2.588574
2.671017
2.753686
2.836575
2.91968
3.002995
3.086516
3.170237
3.254155
3.338265

Figure 7.7: Grasshopper narrow column graph mapper scaling factors per layer (own work, 2024)

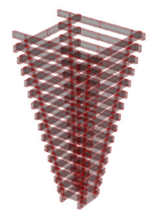


Figure 7.8: Cross section view of the narrow column (own work, 2024)

Figure 7.8 shows the resulting column, this column design weighs 206,4 Kg, the resulting maximum displacement is 218,4 cm, and it has a maximum utilization value of 5,9%. The topmost beams of this column are 1,67m long.

Normal column
The normal column is in between the narrow and wide column with regards to scaling factors The highest scaling factor for this column is approximately 5,2, the full list of scaling factors can be seen in Figure 7.9 The scaling factor is slightly exponential with a steeper growth in the beginning.

1
1.10523
1.213912
1.325942
1.441222
1.559663
1.681182
1.805705
1.933159
2.063479
2.196604
2.332475
2.471039
2.612244
2.756041
2.902385
3.051231
3.20254
3.356271
3.512387
3.670852
3.831631
3.994693
4.160005
4.327536
4.49726
4.669146
4.843169
5.019302
5.197521

Figure 7.9: Grasshopper normal column graph mapper scaling factors per layer (own work, 2024)

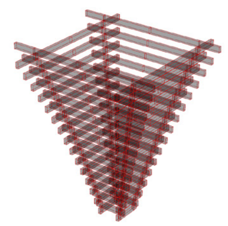


Figure 7.10: Cross section view of the normal column (own work, 2024)

Figure 7.10 shows the resulting column, this column has a weight of 280,7 Kg, the resulting maximum displacement is 225,9 cm, and it has a maximum utilization value of 11.0%. The topmost beam of this column is 2,6m long.

Wide column
For the wide steeper exponential scaling factors are used, the highest scaling factor here is factor 11,7, the

full list of scaling factors can be seen in Figure 7.11. With the scaling factor of 11,7 the topmost beams have a length of 5,8m.

1
1.160938
1.335256
1.523156
1.724827
1.940454
2.170213
2.414271
2.672792
2.945933
3.233846
3.536677
3.85457
4.187663
4.53609
4.899981
5.279465
5.674664
6.0857
6.512692
6.955753
7.414998
7.890536
8.382475
8.890921
9.415977
9.957745
10.516325
11.091814

Figure 7.11: Grasshopper wide column graph mapper scaling factors per layer (own work, 2024)

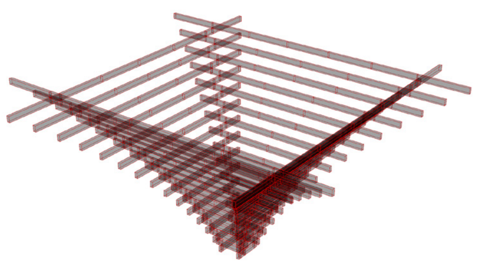


Figure 7.12: Cross section view of the wide column (own work, 2024)

11.684308

Figure 7.12 shows the resulting column, this column weighs 508,4 Kg, the resulting displacement is 361,5 cm, and it has a maximum utilization value of 34,7%.

From these first three simulations some trends are becoming clear. There is a strong positive correlation between the scaling factor and the other parameters (weight, maximum displacement, utilization and length of topmost beams). As the value of the maximum scaling factor increases, the value for the other parameters also increase. Likewise this work the same if the scaling factor decreases, the other parameters decrease as well. Table 7.1 joins all the parameters results from the baseline, narrow, normal and wide column, and Table 7.2 shows the correlation between the maximum scaling factor and each of the parameters, with all correlations near 1 it means that there is a positive correlations between the parameters and maximum scaling factor. This correlation is as expected because, a larger scaling factor means longer beams and thus a heavier column. Longer beams mean that the displacement under the same load is larger too, and an increasing displacement is linked to increasing utilization.

Column	Baseline	Narrow	Normal	Wide
Maximum scaling factor	1,5	3,338265	5,197521	11,674308
Weight	120 Kg	206,4 Kg	280,7 Kg	508,4 Kg
Maximum displacement	215,3 cm	218,4 cm	225,9 cm	361,5 cm
Utilization	1,6%	5,9%	11,0%	34,2%

Table 7.1: Parameters from comparison (own work, 2024)

Correlation between maximum scaling factor and weight (Kg)	0,998170448
Correlation between maximum scaling factor and maximum displacement (cm)	0,959202116
Correlation between maximum scaling factor and utilization (%)	0,996722107

Table 7.2: Correlation between maximum scaling factor and the other parameters (own work, 2024)

From these parameters, the maximum displacement is the most worrisome. The maximum displacement value is measured at the mid-point of the element, for the next part the other influential parameters on the structure will be tested and tweaked to see if a feasible result can be reached.

Additional variations based on findings

Figure 7.13 shows the real-scale displacement of the wide column discrete system, in this figure the topmost column is pushed down the furthest, meaning that with each layer up the displacement of the beam on this layer is increasingly larger. There are some geometry characteristics and material characteristics that can influence the displacement and weight of the column:

- cross section of the elements;
- dimensions of the column base;
- material of the elements

The column height also influences the displacement, however this will not be tested as the assumption is that a column needs to be made at a certain height, and that this is non-negotiable.

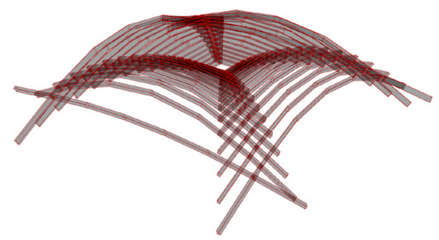


Figure 7.13: Wide column displacement (own work, 2024)

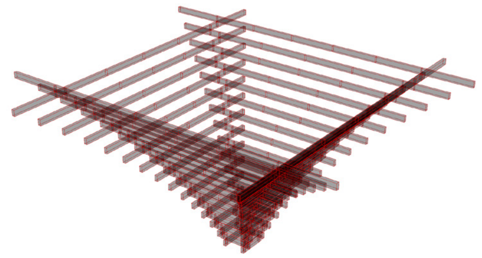


Figure 7.14: Cross section view of the wide column (own work, 2024)

For the following tests the wide column from the previous section will be set as a baseline, first the effect of each influential parameter alone will be checked and eventually they will be added together..

Variable cross sections for the beams
Right now all the beams have the same cross-section, but what if the beams get an increasingly large cross section each layer it goes up. The effect of changing the cross section from 10cm by 5 cm (height x width) to a variable cross section depending on the height is a decrease in maximum displacement from 361,5 cm to 222,01 cm (this is a 39% decrease), this does however come with a weight increase from 508,4 Kg to 2345 Kg (which is a 361% increase).

The variable cross section dimensions were set by taking the base value of 10 cm for the height, and 5 cm for the width. This value is incremented stepwise, with 0,4 for the height, and 0,2 for the width, the ratio between the height and width is always 2 to 1. To see the effect of height increases versus width increase, the variable dimensions are also tested separately.

If the height is set to 10cm, but the width keeps the variable dimensions as explained before, the maximum displacement is 277,9 cm with a weight of 1090,5 Kg.

If the height is set to the variable dimensions as explained before, and the width is set to its base value of 5cm, the total weight is 1090,5 Kg while the maximum displacement is 'only' 229,8 cm (see Figure 7.14 for the column with these cross section dimensions). Meaning that the height has a bigger influence on the maximum displacement, matching expectations because in order to make a beam stronger it is more efficient make the beam larger in the direction of the loading.

Dimensions of the column base
The effect of changing the dimensions of the column base (which is a square, but can also be a rectangle) is linked to what a larger or smaller column base does. The column base basically defines the starting dimensions of the column. If this is a relatively small number, the topmost beam will also be smaller, and vice versa with a larger column base the topmost beam will also be larger. Smaller and larger respectively also result in a smaller maximum displacement and weight or a larger maximum displacement and weight.

Material of the elements
The last parameters for which the influence on the maximum displacement is tested is the material. That the material will be timber is already decided, however there are significant differences in the type and strength class of timber. Within the Karamba3D material library are 28 hardwood classes, and 14 different Glulam classes - for both the strength classes and direction to the grain varies. Appendix C contains the list with 42 hardwood timber and Glulam materials and their characteristics.

In order to check what the better material is (e.g. decreasing both maximum displacement and weight), and optimization plug-in (WallaceiX) is used to quickly run through the 42 options, to find that Glulam GL32c is the best option of these 42 in reducing the weight and maximum displacement the most. Implement this material change to the wide column from before, the weight decreases with 21,9% from 508,4 Kg to 397,2 Kg, and the maximum displacement decreases with 8,4% from 361,5 cm to 331,3cm.

During the study each of the implementations have only been added individually, however to test how a combination of the three implementations can perform they have also been optimized together, however with the assumption that the material would be GL32c. If the material also would be a parameter to optimize there would be a search space of 42.000 possible solutions. Whereas with just the variable cross sections and the dimensions of the column base, the search space has 1.300 possible solutions. The optimization resulted in a column weight of 784Kg (which is an increase of 54%compared to the baseline wide column) and a maximum displacement of 216,7 cm (which is a 40% decrease from the baseline wide column, with the column looking like in Figure 7.15.

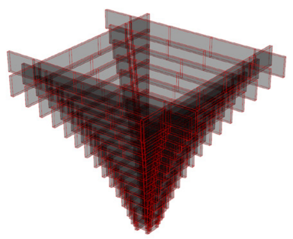


Figure 7.15: Cross section view of the optimized column (own work, 2024)

7.4 Results

After running the simulations in the previous section, there are some things that stood out and are worth mentioning.

Running the simulations yielded some results in how to reduce the maximum displacement, however in the final simulation the maximum displacement was still not within acceptable bounds. Additionally, the utilization factor generally also was very low. The low utilization factor can be deducted from the fact that the load paths went through the joints and never through the middle of the beams, Figure 7.16 zooms in on the elements and places where joints are in the column and only slight utilization is shown around the joints.

Furthermore, the results from the Ansys simulations showed that the elements experienced highly localized peak stresses. Combining this with the high displacement values in Karamba3D, it can be concluded that the joints are the critical points in the system, and with the joints failing as a result of these high peak stresses, the elements undergo high displacement values.

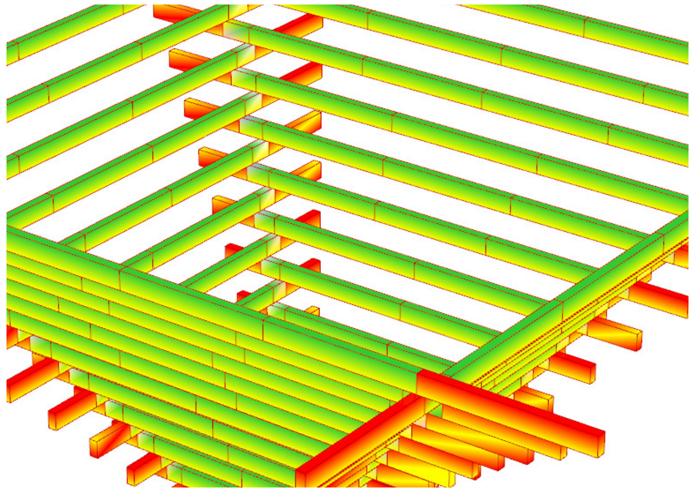


Figure 7.16: Utilization factor (own work, 2024)

7.5 Additional variations of the system's aggregation

Besides the previously shown options of having a narrow, normal or widely stacked column, there are some variations that can be made with the column. These variations can be used to adapt to certain boundary conditions for the column, such as the column being used at the side or a corner of a building without the column crossing the building boundary lines.

To accommodate for this, the column can be made not only symmetrical such as the squared column shown in Figure 7.17, but also asymmetrical such as the rectangular shaped column shown in Figure 7.18. Which then in turn can be applied in a situation with less space in one direction. It is also possible to place the column in a location where one of the sides needs to be flat (Figure 7.19 and Figure 7.20), or even in places where two connecting sides need to be flat (Figure 7.21), think of the side of a building.

Besides the design possibilities that exist in single columns, there is also the possibility to connect the upper beams of two separate columns. What this does is create an arched structure of the two separate columns (see Figure 7.22 and Figure 7.23). The advantage gained with an arched structure is that these help in evenly distributing compressive forces through a structural system (qhplus, 2024).

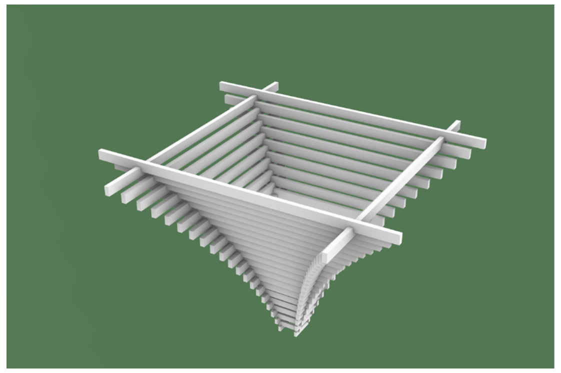
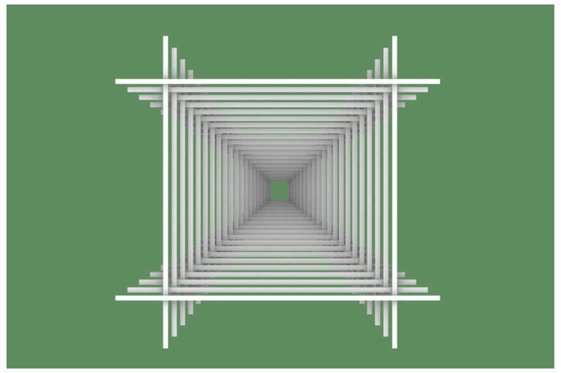


Figure 7.17: Full normal column (own work, 2024)



Top view

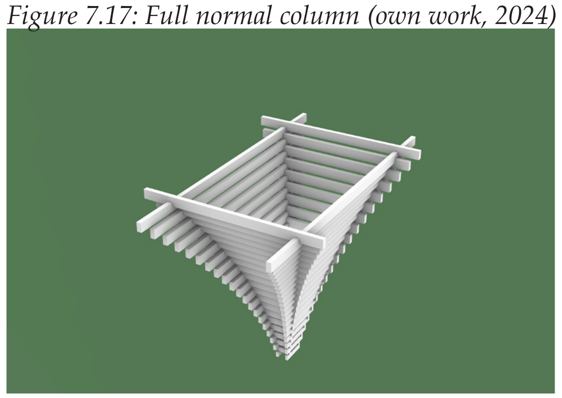
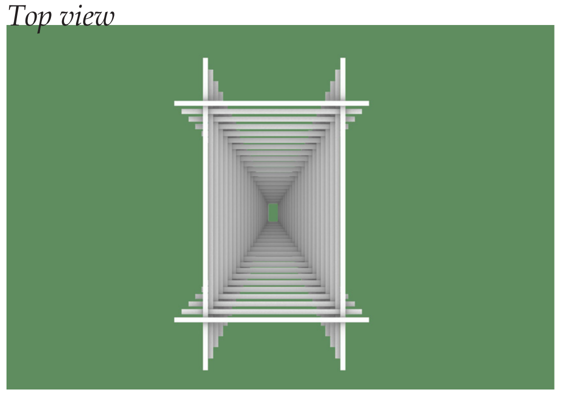


Figure 7.18: Rectangular column (own work, 2024)



Top view

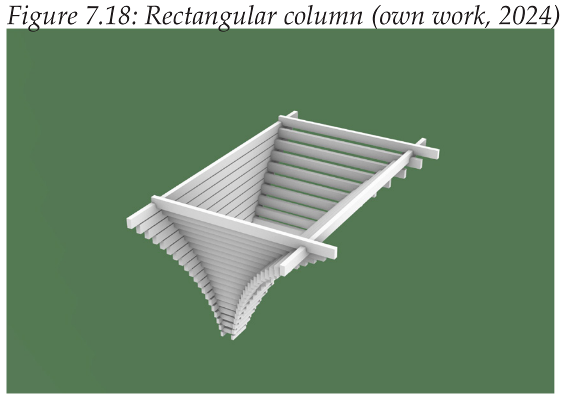
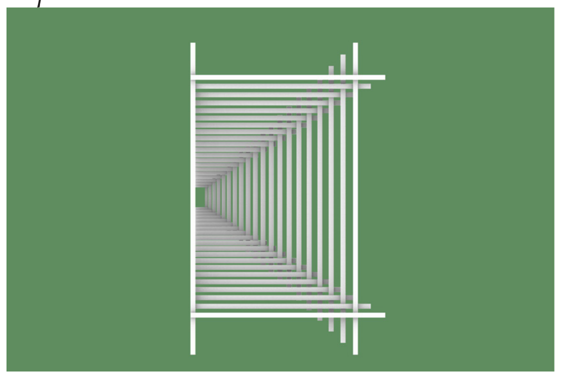


Figure 7.19: One side flush, rectangle (own work, 2024)



Top view

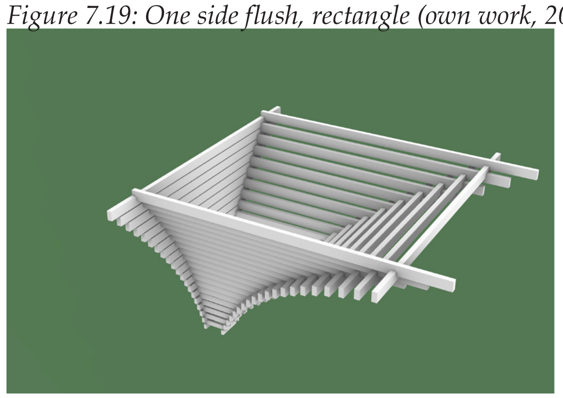
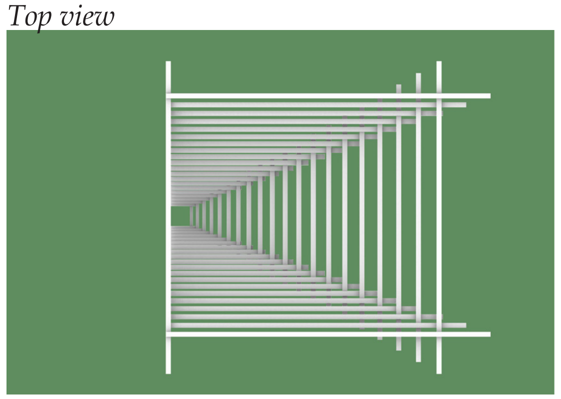


Figure 7.20: One side flush, square (own work, 2024)



Top view

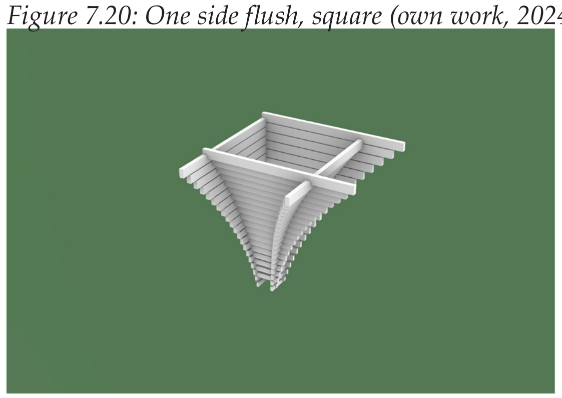
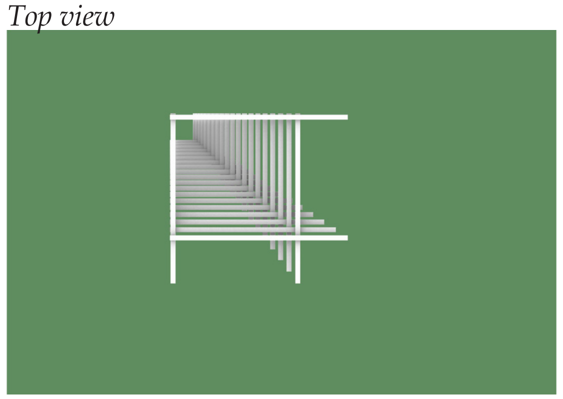


Figure 7.21: Two sides flush (own work, 2024)



Top view

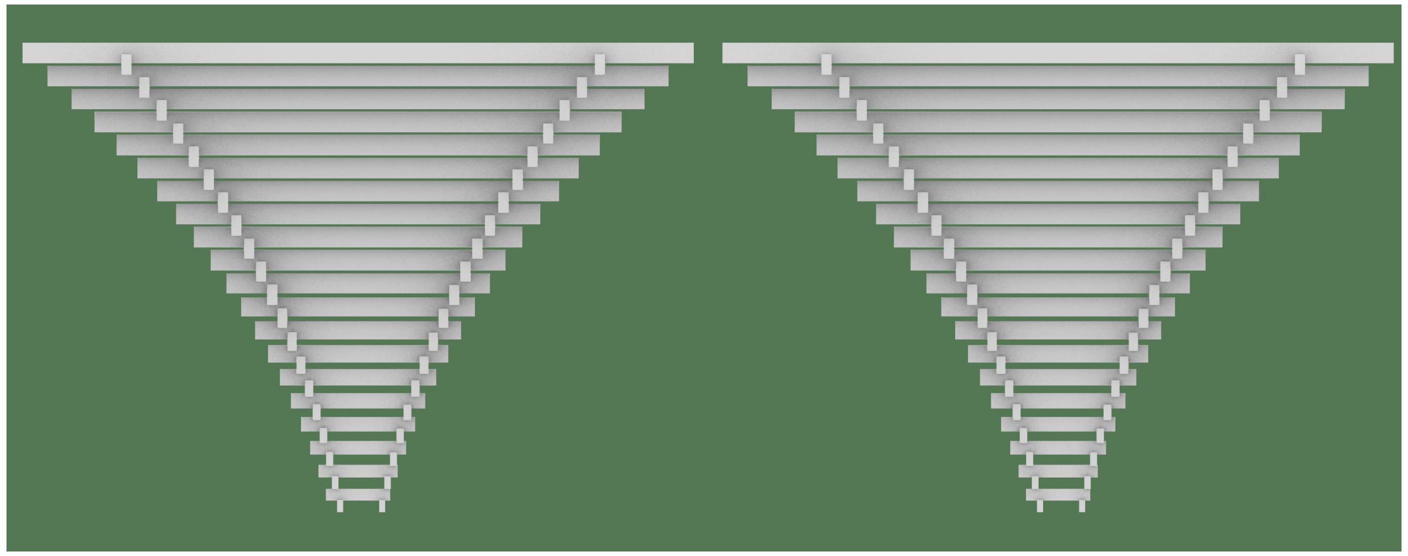


Figure 7.22: Front view double column (own work, 2024)

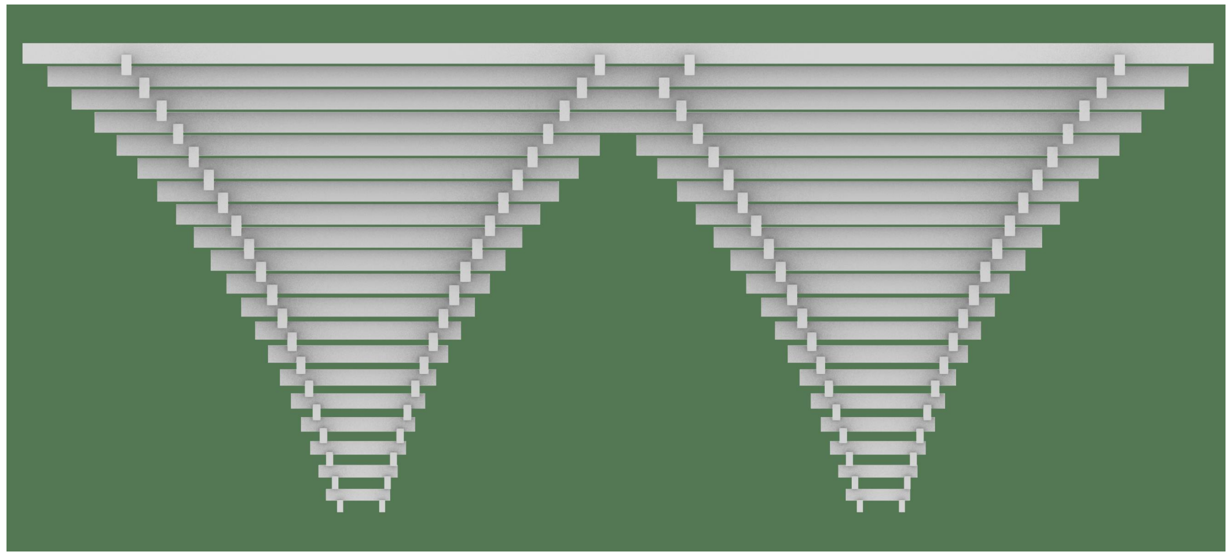


Figure 7.23: Front view double column combined, creating an arch (own work, 2024)

Figure 7.24: Birds eye view of the branching effect of the columns (own work, 2024)

Figure 7.25: Branching effect of columns seen from below (own work, 2024)

7.6 Advantages through application of the system

Advantages on a system level

The previous chapter elaborated on different ways that the discrete timber elements can be aggregated and the resulting discrete timber system - applicable is specific use cases. There are also some advantages that are gained through applying the system, these are additional to the earlier named advantages such as sustainability and reversibility.

By the geometric nature of the discrete timber columns, a top-up can exceed the size (width and length) of the existing block. Meaning that a larger gross floor area can be realized while having a smaller ground bound area, see Figure 7.24 and Figure 7.25.

Another advantage that the system could offer is utilization of the interior space of the column, which as of now can be seen as a void space. The magnitude of its usefulness is however dependent on the size of the column. With larger columns there is some real potential in using the interior of the columns for example for installations, however with the smaller sized columns it is not really feasible or possible to place installations in there. An application that is possible regardless of the size of the columns, is placing a glass roof over the column to make it a multifunctional light tube, this is only possible if the column is on the top floor.

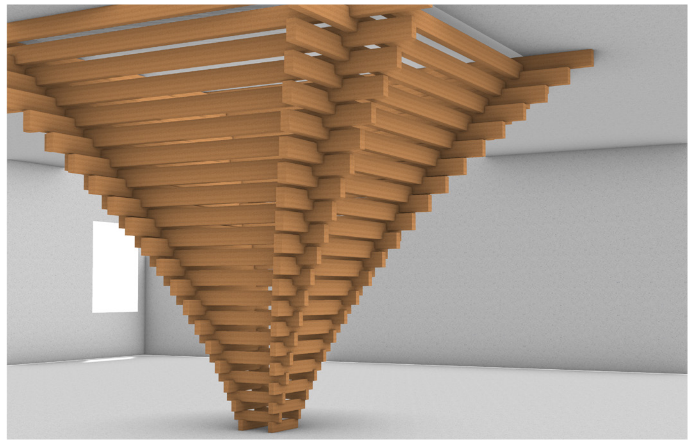


Figure 7.26: Discrete timber column without roof light (own work, 2024)

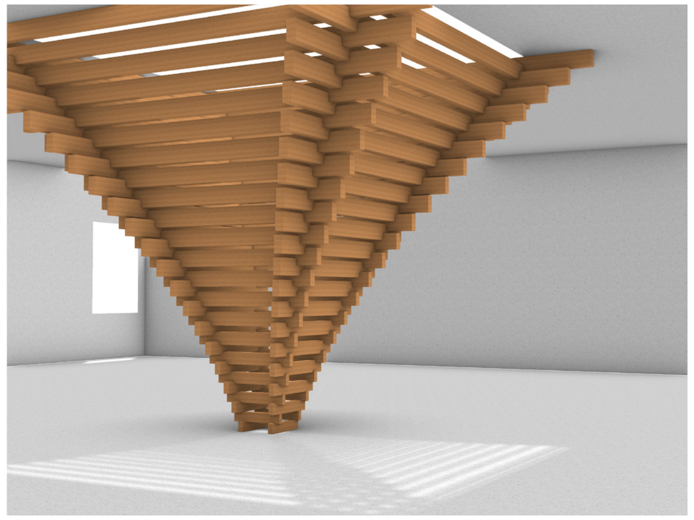


Figure 7.27: Discrete timber column with roof light (own work, 2024)

Advantages on an element level

Where the previously named advantages occur on a system-level, there are also advantages on an element level. For conventional sawn timber, or even engineered timber, larger timber pieces are needed, this means that from one tree only a limited number of useful elements can be gathered. However, the geometry of the discrete timber column clearly shows that the used discrete elements go from smaller at the bottom to larger at the top, which provides the possibility for smaller timber pieces to be used and thus for trees to be used more efficiently.

Another advantage that comes from the fact that the column does not exist from just one element but rather of numerous smaller elements is related to damage to the column. If there is just one column and it would be damaged, this means replacing it by a completely new column even if only part is damaged. In the discrete timber system, if a part is damaged, it is likely in one, two, or a just few elements. Then replacing only the damaged elements would suffice in making the discrete timber system whole again. A potential way to go about this is as follows (see next page for visualization):

Step 1: the discrete timber system

Step 2: one element in this system breaks

Step 3: the elements that this element supports, need to be temporarily supported

Step 4: the broken element can be taken out, if needed it can be sawed apart

Step 5: supports jack up the elements with enough space for a new element to be rotated into place (presumably 4cm, this is not confirmed through testing)

Step 6: a new element is rotated 90 degrees and moved into place

Step 7: new element is rotated 90 degrees into its normal position

Step 8: supports are jacked down and removed

Step 9: the broken element is taken out, and the discrete timber system is whole again.

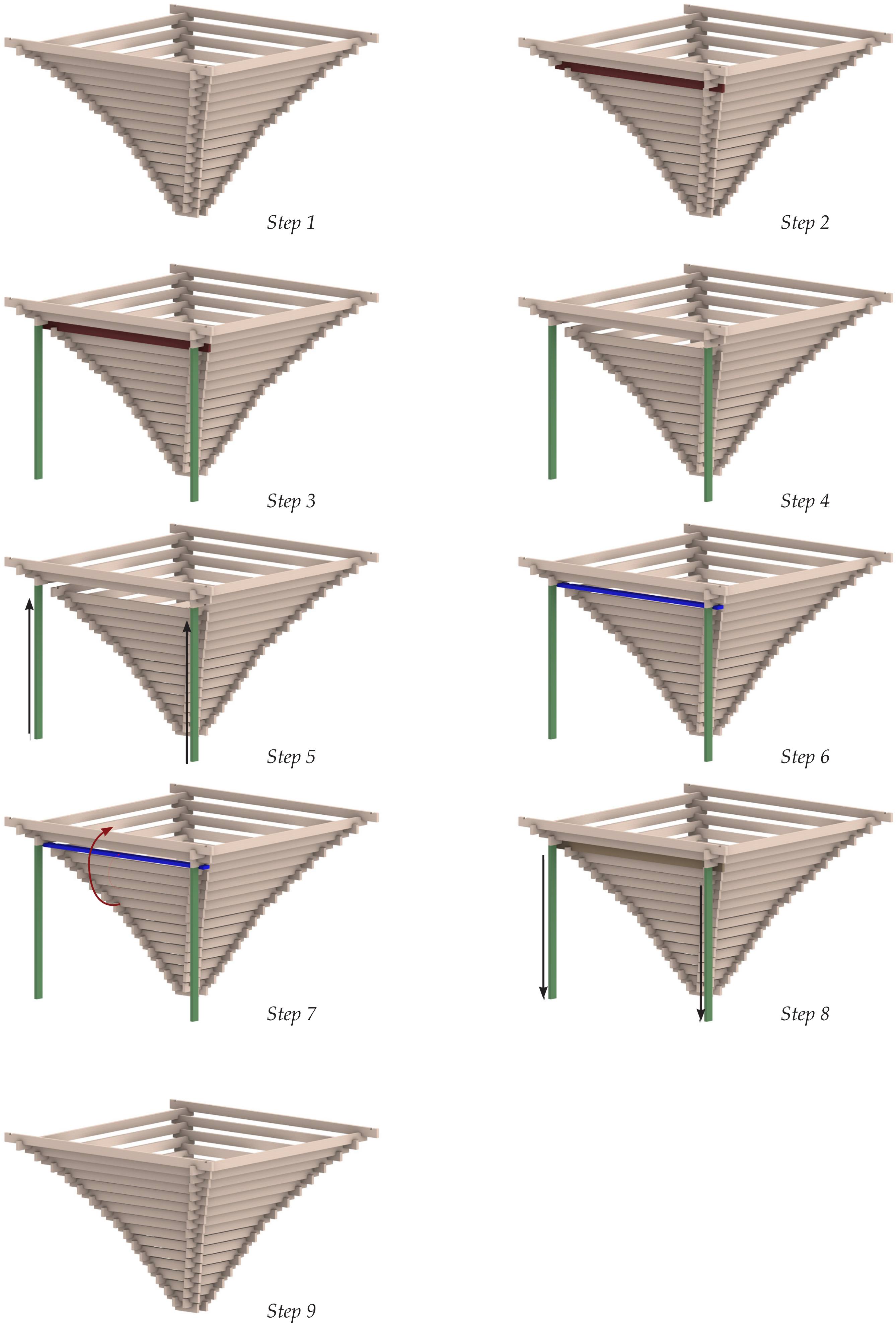


Figure 7.28: Sequence diagram for repairing a broken element in the discrete timber system (own work, 2024)

This method is not (yet) tested, and is an assumption of how a broken element can be replaced. It is possible that it does not work exactly as described or there is a more efficient method to do so. How high the elements need to be jacked up also is assumed to be just a bit more than the height of a joint, to get the exact number this would have to be calculated more thoroughly. With this exact number it can also be checked if it is feasible to move the structure up by this much.

Next to that, due to not using any glue or metal fasteners in the connection, the elements have a better potential of being reused. However, this is strongly related to the number of joints per element. The discrete timber system as shown in Figure 7.28 only has one option to connect beams.

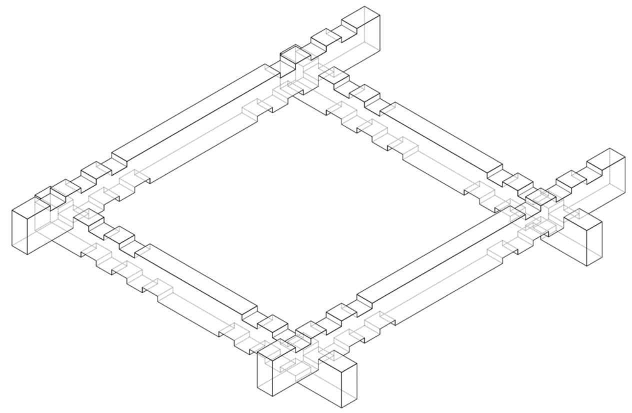


Figure 7.29: Simple square cog joint with multiple connection possibilities (own work, 2024)

If a discrete element is made with more joint options (see Figure 7.29), its potential to be reused and/or reconfigured will also increase, it is however unclear what the effect of this is on the structural performance of the element, especially since the joints are the weakest point in the element.

7.7 Conclusion

This chapter looked at different ways of aggregating the discrete timber elements into a discrete timber system, and input parameters that have influence on the resulting maximum displacement. There are limitless possibilities when it comes to aggregating discrete timber elements into a discrete system. However, in order to limit the limitless, the aggregation is generalized into three categories based on the flow of loads through the discrete column, which are: narrow, normal and wide flow of loads, see Figure 7.1.

Initially, the narrow, normal and wide options have been simulated to see their structural behaviour. From these simulations it became clear that the scaling factor, a multiplier for the length of each beam per layer of the column, influenced many of the other parameters of the column. The influenced parameters were weight, maximum displacement and utilization, and the correlation was positive. So an increasing scaling factor meant increasing weight, maximum displacement and utilization.

Where for the first 3 simulations only the scaling factor was adjusted, for the following ones geometry and material characteristics were defined that also influenced the maximum displacement and weight. Namely, the cross section of the elements, dimensions of the column base and the material of the elements.

With the cross sections of the elements it was found that the height dimensions has a larger influence than the width, which follows the theory of structural design. The dimensions of the column base mimic in a way the scaling factor, because if the column base is larger, the other elements in the column are also larger, therefore it also sees a similar effect as with the scaling factor. The Karamba3D plug-in comes with a set of predefined materials, in which are 28 hardwood and 14 Glulam. Initially the material for the design was hardwood with strength class D30, but after a small optimization plug-in was run it resulted in Glulam GL32c being a better option when minimizing maximum displacement and weight.

The three simulations for the cross sections of the elements, the dimensions of the column base, and the material of the elements were run individually at first, but in the end also combined together resulting in a column with a lower maximum displacement but higher in weight, compared to the baseline wide column.

Additionally to the tested discrete timber aggregations, there are also ways of aggregating that can be used in special situations, or to create stronger columns by connecting two separate ones at the top and by doing so create an arched structure. Using the system in certain ways can provide additional advantages such as generating a larger floor area than the existing building it is placed on, utilizing the interior space of the column for installations or as light shafts.

For a discrete timber system like this wood from a tree can be used more efficiently due to the need for smaller pieces, and if an element breaks not the whole column has to be replaced, as would be the case with a single timber column. Additionally, the system has a good reuse potential because it is connected without glue or metal fasteners, but is more governed by the number of joints. However, if the number of joints is increased it could affect the structural performance of the element.

8. Conclusion

8.1 Conclusions

The findings in this research shed light on the structural capacity in discrete timber systems and (the joints between) discrete timber elements. The research was guided by the following defined main and sub research question.

Conclusions sub research questions

What makes a discrete timber system feasible?
Conventional structural timber and various Engineered Wood Products perform in a certain way, they can do this and that and have a structural capacity of xx.xx MPa. If someone compares steel and concrete alternatives from each other one of the most important aspects is that the alternative material performs structurally about the same. For a discrete timber system to be feasible it must be able to perform structurally in the use case defined for that discrete timber system. However, where someone might choose steel over concrete because of the improved construction time, there are also other aspects that add to a discrete timber system being feasible or not.

Besides the structural capacity and behaviour of the discrete timber system it also should not be ridiculously complex or heavy compared to the conventional timber alternative, this would make it financially not interesting and perhaps too heavy for its use case. It could be a bit heavier (and pricier) than its conventional counterpart because it can be demounted en reconstructed, so after x amount of uses it might be worth it financially.

In the end, discrete timber systems are a feasible alternative to conventional structural timber when using discrete timber is worth it in the use case, but besides it being case-dependent, the discrete timber system also has to be strong enough and light enough, with joints that can be demounted.

What defines the structural performance of discrete timber systems?
Just like with conventional timber, or Engineered Wood Products, there is not one structural performance of discrete timber systems, but it is dependent on various factors.

The factors found in this research that influence the structural performance of discrete timber systems are :

- the scaling factor of each layer of the discrete system, which in the end influences the width of the discrete system and the distance between each element and thus the moments on the other beams;
- the selected material (e.g. hardwood or Glulam) and within a material group the selected material strength, such as D30 or GL28c.
- the cross section of the discrete timber elements, where logically a larger cross section means a stronger element (but also heavier).

In addition to the factors mentioned above, the joints play a crucial role. The various simulations conducted within Ansys showed every time that the peak stresses occur in highly localized places, often within the geometry of the joint. Which in the end causes the joint, and thus the discrete system to fail.

Which reversible joinery techniques and joints are applicable to use in discrete timber systems?
There are hundreds of different joints that can be made reversible. As long as it does not contain permanent influences such as adhesives and glue, or nails and nail plates. Bolts can be used in reversible joints too but it must be made so that the bolt-hole does not wear out. For this research, also with focus on completely take any other material than timber out of the system, dry timber joint techniques were researched and used.

Within this subcategory of joints the focus is on processing timber beams in such a way that two separate elements interlock with each into a continuous element or a stiff joint. Back in the days this would be done by hand but CNC-milling might a good alternative. The research to applicable joints started digitally but ended with a book containing 198 wood joints, which in theory all would have been applicable to use in discrete timber systems. However for the use case of this discrete timber system, the elements needed to be joined perpendicular, which already narrowed down the possibilities from 198 joints to 14 joints. Since it was time-related not doable to simulate all 14 joints in Ansys, a selection was made based on complexity, because if you make a top-up with 50 columns each containing 60 smaller beams it can save a lot of time when the joint is simple rather than complex. The remaining available joints all are applicable to use in discrete timber systems, the way they connect is very similar to the half-lap joints introduced in chapter 4.

What is the structural performance of joints between discrete timber elements?
The joints connecting the discrete timber elements with each other have been simulated in Ansys to see what the structural performance is. The results from this are however limited and slightly biased due to the required computation power needed to get more accurate results. The simulations were valuable to see what the general reaction of the joints and elements

were to point loads and on point supports. This gives a good image of what happens inside the elements for almost all of the elements, except the top and bottom ones because they have (respectively) not point but block loads and not point but continuous supports.

Specifically, the simulation found that the majority of the beam performed exactly as expected under the applied point loads, the parts of the element where the load was applied moved down, causing the midpoint of the beam to move slightly up, resulting in tension in the top of the beam and compression in the bottom of the beam. However, there were also highly localized peak stresses in the joints, likely causing the joint to fail. This was with certainty the case for the dovetail lap joint, this was the joint with the smallest joint, and this could not handle the stresses.

Conclusion main research question

All the part previously used to answer the sub questions all come together in answering the main research question.

How can a reversible discrete timber system be a feasible alternative to conventional structural timber?

In order for a discrete timber system to be feasible it needs to perform in a certain way, not only structurally, but also in simplicity and final weight of the system. Applying the discrete timber system must fit the use case, but more important the system should be not too heavy, simple and reversible, these are the first aspects that make a discrete timber system a feasible alternative to conventional structural timber.

The structural capacity is something that has been covered more in-depth. Both for the discrete timber elements and the discrete timber systems. The discrete timber elements were tested as single element and as aggregation of two layers of elements. The main conclusion found here was that the joints are over utilized compared to the rest of the discrete timber system, the peak stresses all occurred within the joints of the elements, whereas the rest of the elements showcased structural behaviour within acceptable bounds.

The discrete timber elements were aggregated into a few different discrete timber systems. Differing in scaling factor of the elements length and thus width of the column, the cross-sections of the elements and the specific timber material used in the simulations.

In the end it was found that these parameters indeed influence the structural capacity of the discrete timber system. However, there are also some gaps with regards to which joints are suitable, moreover, can the joints be made in timber or should there be resorted to a reinforced joint in a different material than timber? One of the main strengths of a discrete timber system lays in its reversibility, and for the system to be reversible there must be a demountable joint.

Discrete timber systems can be a feasible alternative to conventional structural timber by ensuring that the discrete systems have strong, reversible joints that are simple in production and construction.

8.2 Recommendations

Recommendations for further research

This research has found that there are highly localized peak stresses occurring in the joints connecting the discrete timber elements. However the joints are a crucial aspect in design a reversible discrete timber system. It could be that the joints need to be made from a different material, stronger timber class, or different geometry so that the joint has less weak spots. Here lays a research opportunity to find out how the joint in reversible discrete timber systems can be made stronger.

To not drift the focus away from design a reversible discrete timber system, this research worked with the assumption of freshly produces and manufactured discrete timber elements, however the build up of the column shows there are numerous small elements within the system, providing a research opportunity for using waste wood in constructing discrete timber systems.

The discrete timber system in this research was linked to a case study in topping up on post wart apart blocks, however to start scaling up it is necessary to know where else the discrete timber system can be used.

8.3 Reflection

What is the relation between this graduation project, the Building Technology master track and the master programme MSc Architecture, Urbanism and Building Sciences (AUBS)?

This graduation project has been conducted under the guidance of Dr. Stijn Brancart and Prof. Alex de Rijke. Dr. Stijn Brancart as member of the Chair of Structural Design & Mechanics, and Prof. Alex de Rijke for his extensive knowledge on Timber Products & Innovation in timber.

The relation of this graduation project to the chair of Structural Design & Mechanics is that this project aims to design an innovative, sustainable timber structure, with reversible wood-wood connections. It is doing so by aggregating discrete timber elements with interlocking joints into a discrete timber system functioning as a column used in top-ups on Dutch post war apartment blocks. Key aspects here are life cycle extensions of these post war apartment blocks by adding a top-up with reversible joints so that after the life cycle the discrete systems can be reused, possibly in a reconfigured way.

In relation to timber products & innovation in timber, this graduation project uses timber beams in a novel way, connecting them by adapting existing wood joints to the specific use case of this research. With the eventual goal of reusing the elements in a similar or completely different use case, all with the larger objective to reduce greenhouse gas emissions in the construction industry and consequently reducing climate change effects.

In the larger picture of the MSc AUBS program there are some recurring themes during the bachelor degree and during the masters degree. One of these themes is sustainability and the purpose of sustainability in reducing the effects of climate change. Innovation is another theme that is not only covered in the MSc Building Technology track, but rather MSc AUBS-wide. Besides the relationship between this graduation project and the AUBS program through sustainability and innovation, there is also the more common ones related to the built environment such as structural design with designing a load bearing structure, and the focus on reversibility for the joints.

How did the research influence the design?

Initially it what not clear to me yet what the final discrete system would look like or in which way it could be logical to apply. Then, at some point during the research I came across some examples of stacking orthogonal beams, and then the idea came to look at topping up on existing buildings. It seemed logical then because of the light weight characteristic of the discrete system, but also the variation it could offer to the architectural image of the whole block once the top-up is added.

How did the design influence the research?

The previous part describes how the research influenced the system and case study, but the case study also influenced the research. By knowing that the case study would be in topping-up, a mock version of the column was imagined right on top of an existing apartment block. This made an interesting composition which lead to the idea for mixed-use in the top-up rather than just houses. By having the discrete column directly on the roof of the existing building in an outdoor space.

How do I assess the value of my way of working (my approach, used methods and used methodology)?

If I look back at the graduation period I see a few different ways of working. In the period between P1 and P2 it was very research focussed, a period in which a lot of desk research was conducted to find proper sources. My initial thought with having a timber related research topic was that it would be useful to have an enormous library of available theory in books, scientific papers and articles, reports, previous theses, magazine articles and existing projects. However, at some point I had read (probably) more than 100 of these theory sources, but if I wanted I could spend another afternoon and still find tens of useful sources.

Spitting through, and documenting on the existing research was something that went on a bit longer after P2, until as some point I felt stuck because "I was missing a piece of the puzzle". At this moment a thought came that it was not needed to wait for the exact puzzle piece needed at that moment, but rather that it was more efficient to start working on a different part of the puzzle, and then in the end when all the separate parts of the puzzle where done, they could be connected into the final puzzle. I find a huge value in that thought that I just described, but also that I would have been able to structure my planning better if I approached the research like this earlier on.

After P3 the methodology was like design by research and research by design, alternating and interwoven through each other. Which I found a valuable way to quickly progress on both front. If I was stuck with research I could resort to the design (of the column) which usually resulted in new information and ideas to apply to the research, and if I was stuck with design I could look at sources already consulted before, but now with more knowledge from the design, which at times gave a fresh insight to use in the design part.

How do you assess the academic and societal value, scope and implication of your graduation project, including ethical aspects?

I believe that there is significant academic value provided by this graduation project. It provides a novel insight in a (one of the) method that can be followed for calculating the structural behaviour of a custom discrete timber system aggregated from discrete timber elements with custom, reversible joints. Of course it is not only this specific approach that it provides academic value in, but also for anything along the lines of design a discrete timber system without an available method to calculate its structural behaviour.

On a societal front it provides value in one of the most crucial problems of our time, namely climate change. With the construction industry being a significant polluter in multiple aspects, it is of high importance to come with innovations that helps reducing this pollution. Providing to the circular economy by designing a system that can be demounted an reused might not seem as such a big impact, but if scaled up it could prove very significant.

I do not believe there is an ethics consideration directly linked this graduation project. If it would be further in the development phase there of course is the safety of the users, which of course is something that is always at play, however right now if a structure does not fail it does not cause any harm.

How do I assess the value of the transferability of the project results?

Discrete systems can not only exist with timber as a material, even some of the discrete element typologies from chapter 6 could be named timber bricks very well, just because of its resemblance to brick and mortar structure (but then of course the mortar should be avoided as to improve re-usability). The scripts and methods used however are more case specific and cannot directly be transferred onto other projects, disciplines and used in a broader sense. There is however a certain logic in how the research was conducted, and this logic can be transferred onto other projects and disciplines. To answer the question; I believe that the value of the transferability of the project results is not so high unless the project is regarding discrete timber systems.

What is needed to scale this research up and into the market?

The research is currently still very academic, the results show that there are some flaws in the system (amongst others the joints), my assumption is that the first step is to take a closer look at these flaws and see if there is a way to take these flaws out of the system. If the discrete timber system is flawless, then the material should be tested in real life to verify its structural capacity. After which the potential to scale into the market becomes more reasonable.

How do I assess the value of innovation of this graduation project?

At the start of the graduation process I was on the search for interesting topics, I came across various discrete timber projects and structures, however none of the addressed full scale structural capacity. I believe the graduation project scores relatively high on innovation, it is quite novel, and perhaps that is also what made the decision making throughout the project sometimes so difficult.

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9.2 List of figures

10. Appendices

Appendix A: Calculation of load on a column in the top-up
Appendix B: Grasshopper script
Appendix C: Timber materials in Grasshopper

Appendix A: Calculation of loads on a column in the top-up

In order to set-up the simulations in Ansys and Karamba3D, the heaviest load that a column could experience needed to be known as this is the load that the joints, element and system need carry. What follows is an elaboration on how this load was calculated.

Calculation of the present loads

The floor will be built up from Laminated Veneer Lumber (LVL) with a density of 510 kg/m³, with on top a simple concrete finishing with a density of 2400 kg/m³, and below it the various installation and lighting aspects with a combined weight of 60 kg/m², these components have the following density.

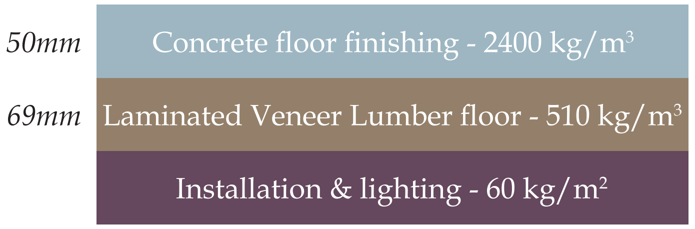


Figure 10.1: Schemati floor set-up with material density in kg/m³ (own work, 2024)

These loads are first transferred to kg/m² by multiplying the density with the thickness, and after that to kN/m². by multiplying the kg/m² times 10^{-2}.

Figure 10.2: Schemati floor set-up with material density in kg/m²(own work, 2024)

When adding all the individual weight of the different floor aspects together we get, (120 + 35,2 + 60) = 215,2 kg/m², which is (215,2 * 10^{-2}) = 2,15 kN/m².

Then there is also the weight of the walls, for this the assumptions has been made that there are relatively light seperation (internal) walls of 0,5 kN/m², and the external walls are 1,0 kN/m².

This brings the total dead load to (2,15 + 0,5 + 1,0) = 3,65 kN/m².

Then there is also the live load, and this depends on the function that is housed on the respective floor. Assuming the 'worst' case in which the top-up is the heaviest, will be a situation with three extra floors. Respectively, (open air) communal social space, could be a roof garden, and two floors with extra houses. Regardless of the function on the first floor (directly on the old roof), the roof will likely need strengthening as the it has been designing with a live load of 1,0 kn/m².

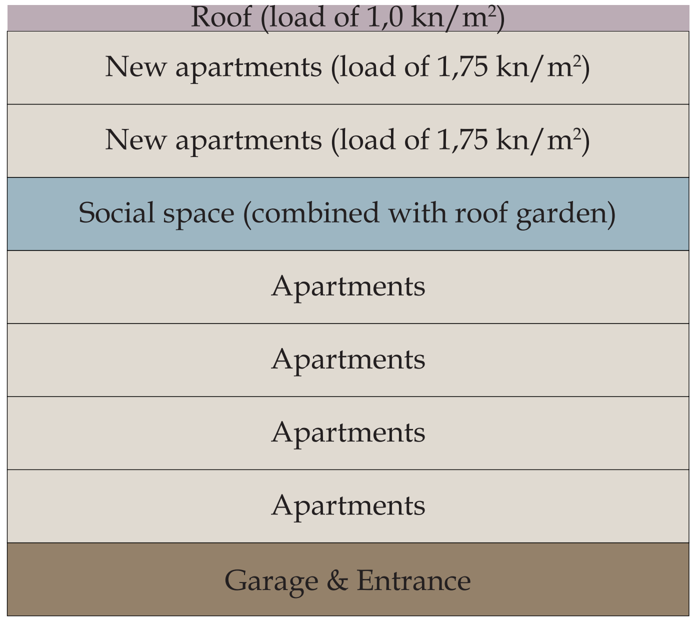


Figure 10.3: Schematic set-up of the functions in the existing block and the top-up with function loads (own work, 2024)

The apartment block on the 'Pirandellostraat' consists of two types of apartments alternating such as shown in Figure 10.4. The segment in Figure 10.5 is the leftmost two apartment blocks from Figure 10.4. On the lines numbered 1 through 6 are load bearing walls in the existing structure, this grid will also be used for the top-up, the lines A, B and C are added, where B is the middle of the building block width.

Without taking the layout of the top-up into consideration, the column location that will carry the most weight will be the column on B2 (and also B2 since it is symmetrical). This is the column for which the loads have been calculated. Figure 10.6 shows the (floor & roof) area that supports on column B2;

- half the length between grid 1 & 2 (1,9m) + half the length between grid 2 & 3 (1,595m), which adds up to 3,495m;
- half the length between grid A & B (4,83m) + half the length between grid B & C (4,83m), which adds up to 4,83m.

This, together with loads per m² information, and certain load factors have been processed in an excel sheet which is shown in Figure 10.7.

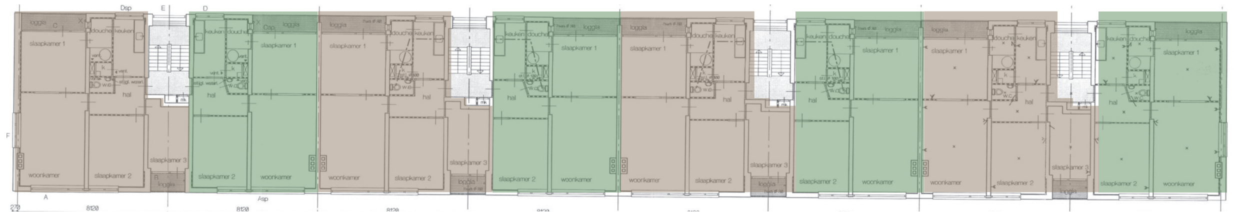


Figure 10.4: Pirandellostraat building floor plan (TU Delft, n.d.)

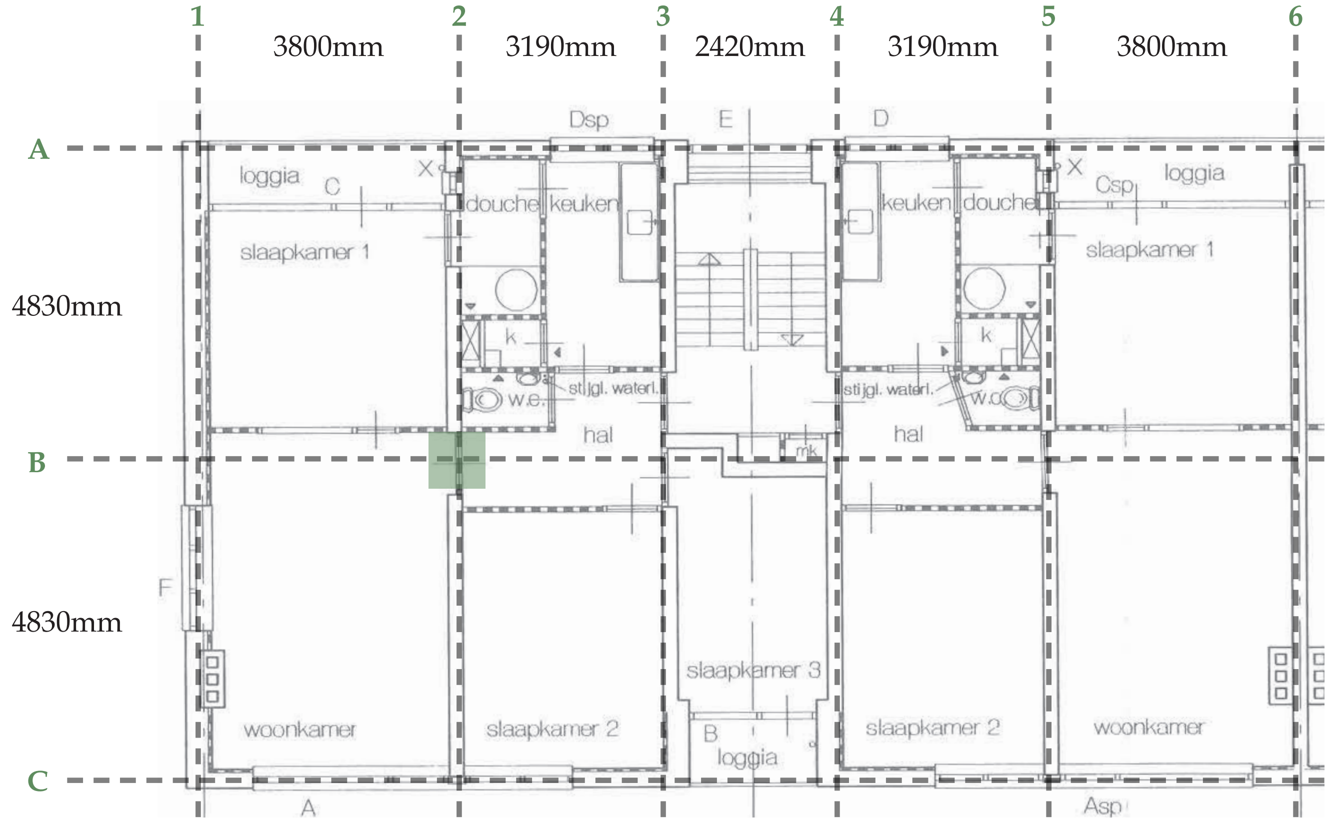


Figure 10.5: Pirandellostraat segment of building floor plan with added gridlines (TU Delft, n.d.)

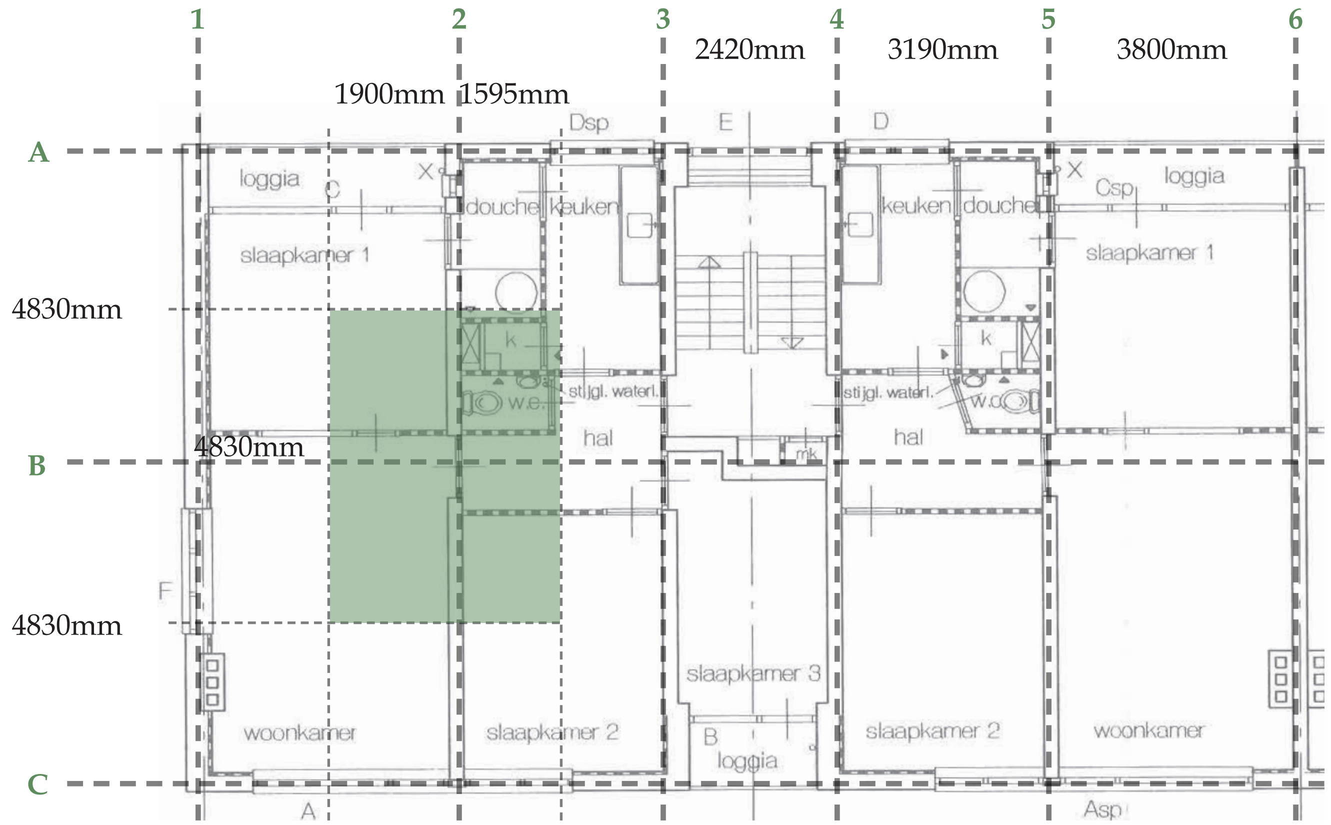


Figure 10.6: Pirandellostraat segment of building floor plan with floor area supporting on column B2 (TU Delft, n.d.)

Load on column table

Weight table	Length [m]		Width [m]		Load/m²		Permanent load [kN]	Total permanent load per floor [kN]	Live load [kN]		Factor ψ		Value for live load with factor [kN]
Roof													
Live load =	3,495	x	4,83	x	1	=	⟶		16,88085	x	0	=	0
Weight roof construction =	3,495	x	4,83	x	1,45	=	24,47723						
Weight wall =	3,495	x	4,83	x	1,5	=	25,32128 +						
							--->	49,79851					
2nd floor													
Live load =	3,495	x	4,83	x	1,75	=	⟶		29,54149	x	1	=	29,54149
Weight floor construction =	3,495	x	4,83	x	2,15	=	36,29383						
Weight wall =	3,495	x	4,83	x	1,5	=	25,32128 +						
							--->	61,6151					
1st floor													
Live load =	3,495	x	4,83	x	1,75	=	⟶		29,54149	x	1	=	29,54149
Weight floor construction =	3,495	x	4,83	x	2,15	=	36,29383						
Weight wall =	3,495	x	4,83	x	1,5	=	25,32128 +						
							--->	61,6151 +					+
Total (kN) =							Permanent load = G:	173,0287			Live load = Q:		59,08298
Partial factor $\gamma_{f,G}$ =	1,2		part.fact. $\gamma_{f,Q}$ =		1,5		$F_{G,d}$ =	207,6345			$F_{Q;d}$ =		88,62446
	F_{cd} = 296,259		x 1000 =		296258,9 N		KN =	296,3			weight per load point (/4) =		74,1

Figure 10.7: Load on column calculation-sheet from TU Delft (own work, 2024)

In the figure above, the length and width (calculated in Figure 10.6) have been entered, the sheet then multiplies this, together with the load per m² to get the total permanent load per floor. The first column of the top-up was selected, meaning there are two floors and a roof supporting on there, see Figure 10.8 for the column being calculated. The live-loads are multiplied by a certain factor, if there are a larger number of floors this reduces the total live load because it is unlikely that all floor with be loaded heavily. This reduction factor counts on all but two floors, the two heaviest floors get a reduction factor of 1, meaning the full live load counts. The reduction factor for the roof is 0 because the column being calculated is not directly under the roof. The partial factors are 1,2 and 1,5 for respectively the permanent load and the live load.

The resulting load on the column B2 is the 296,3 kN, because the column has four joints, the load in each joint is 74,1 kN.

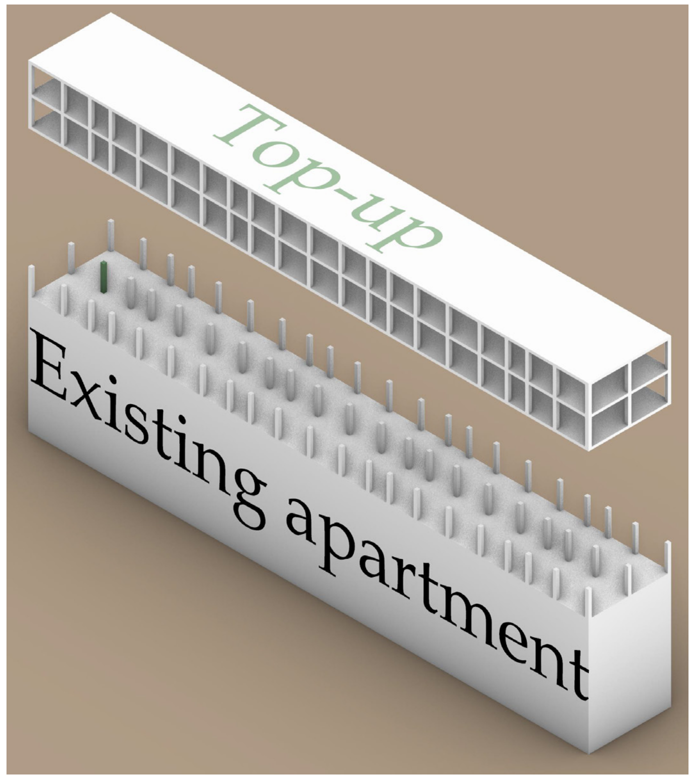


Figure 10.8: Exploded view of schematic set-up for the top-up, highlighted is column B2 (own work, 2024)

Appendix B: Grasshopper script

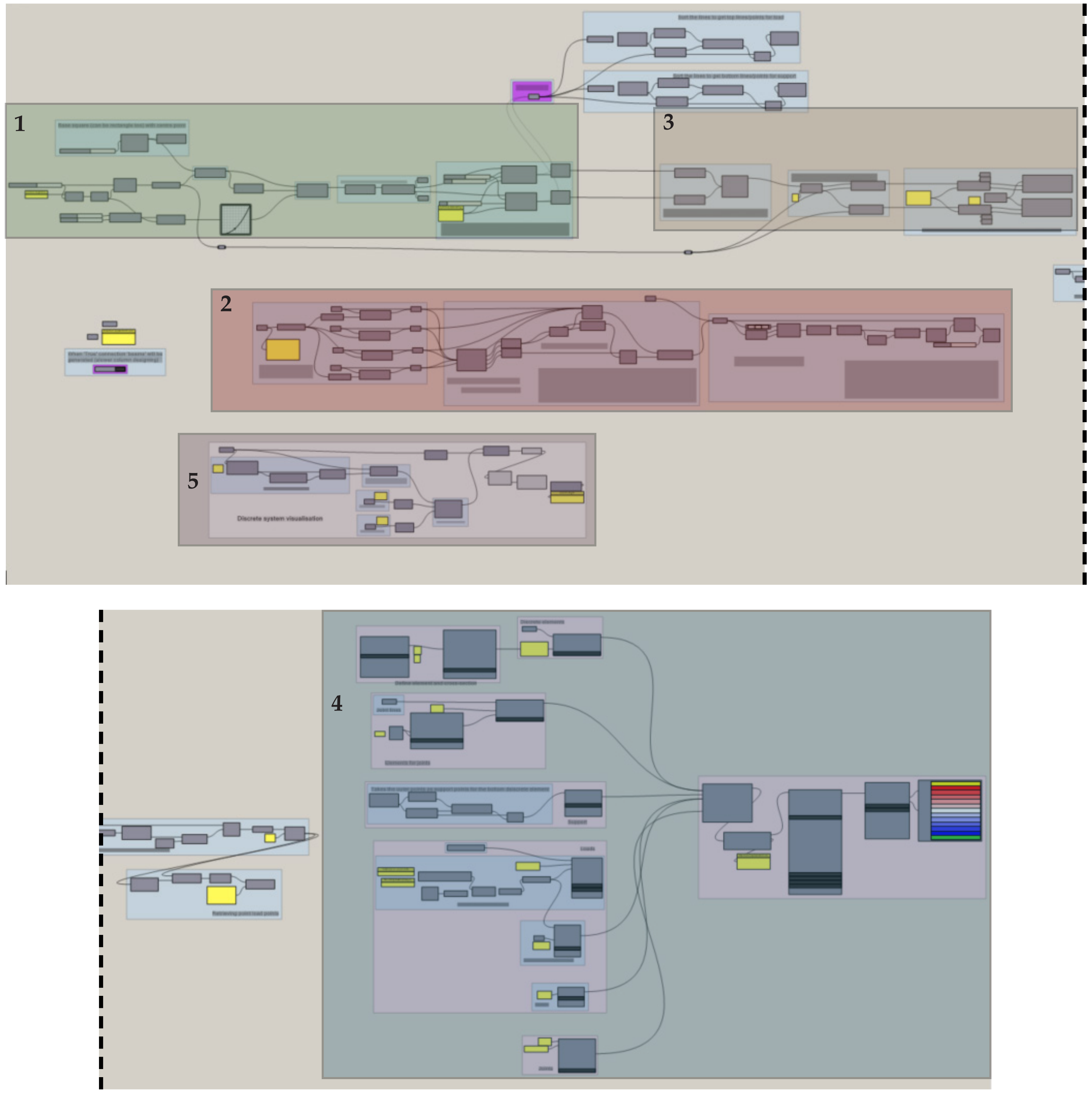


Figure 10.9: Grasshopper for Rhinoceros script for calculating beam deflection and strength (own work, 2024)

1. Generating horizontal lines that can be used to make the discrete elements in the Karamba3D part;

2. A script that divides the lines generated in point 1 at the places on the beam where the connection to the beam below and/or above is

3. Generating vertical lines between the discrete elements that function as the connection between the different layers

4. The structural analysis in Karamba3D;

5. Visualisation of the discrete system column.

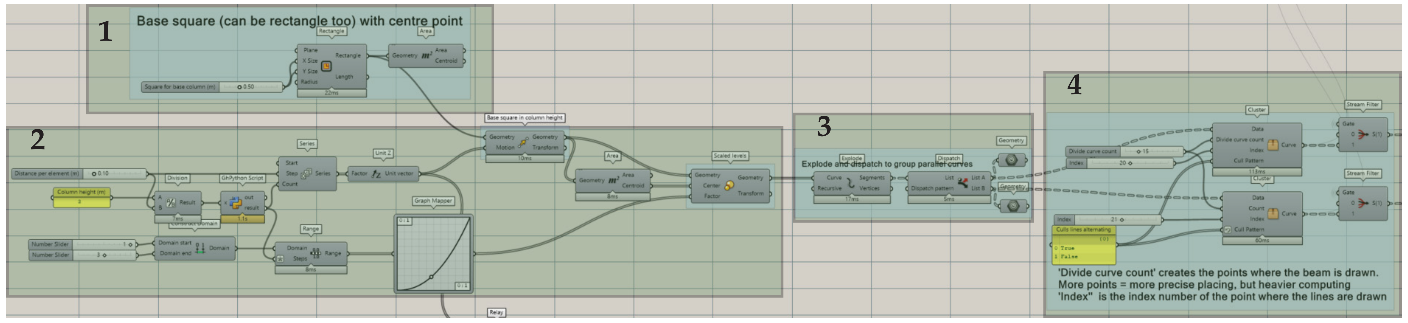
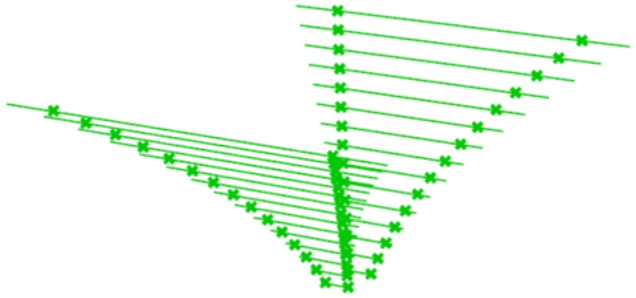


Figure 10.1: Grasshopper script part 1 - horizontal lines for beams (own work, 2024)

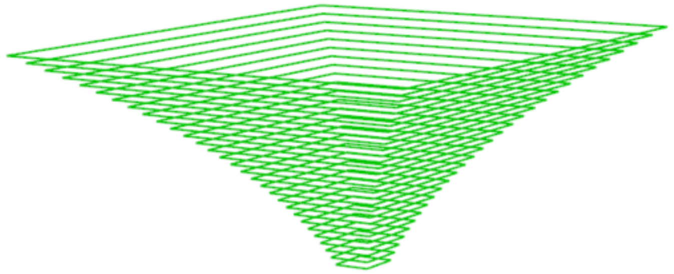
Part 1: Horizontal lines for beams

The script within square number 1 creates a square (or rectangle) that will form the base of the column.

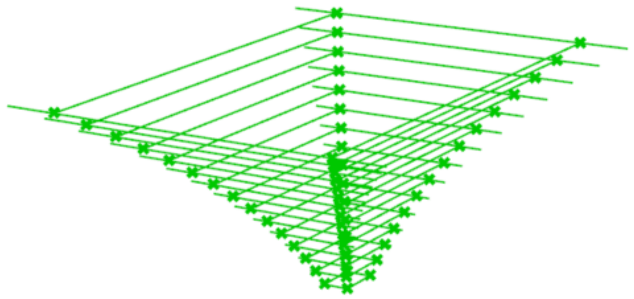
The script in rectangle number 2 calculates the amount of layers based on the column height and distance between each layer. Then the base square/ rectangle is copied over the length of this column and scaled with a graph mapper, this is set to get a more mushroom shaped column at the moment. This resembles the outline of the column.



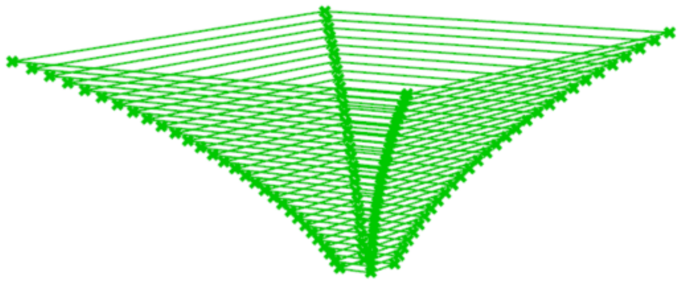
Number four, the last step of this part, takes the outlines that are parallel of each other and plots a point on these parallel curves.

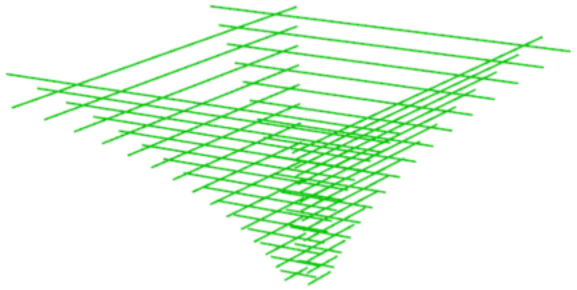


The script labelled 3 take these outline lines (which are closed curves) and explodes them into 4 segments each.



Then a new curve is drawn between the parallel points on the same layer (height). The new curve is a like an offset to the inside.





This is done for both sets of parallel curves which yields the result below. These horizontal lines are eventually made into beams.

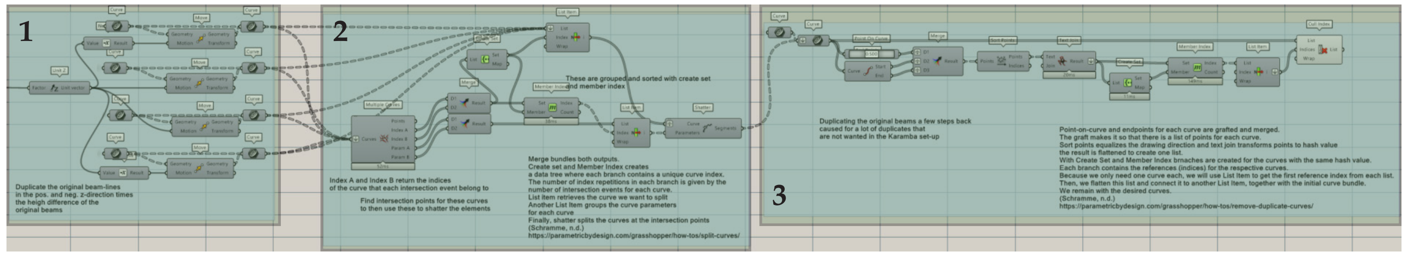
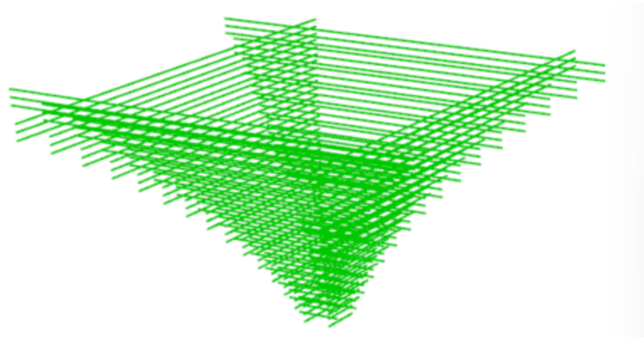


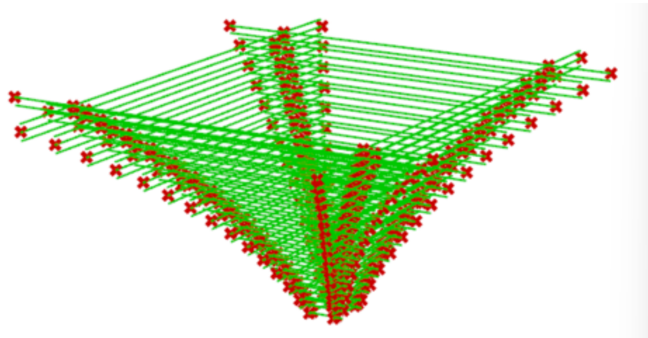
Figure 10.2: Grasshopper script part 2 - dividing the horizontal lines into segments (own work, 2024)

Part 2: Dividing the horizontal lines into segments

The script in rectangle 1 takes the resulting curves from the previous parts and copies these, along the z-axis one layer above and one layer below (i.e. the layer height movement along z-axis positive and negative). The result is on all layers curves intersecting on the points where joints are.



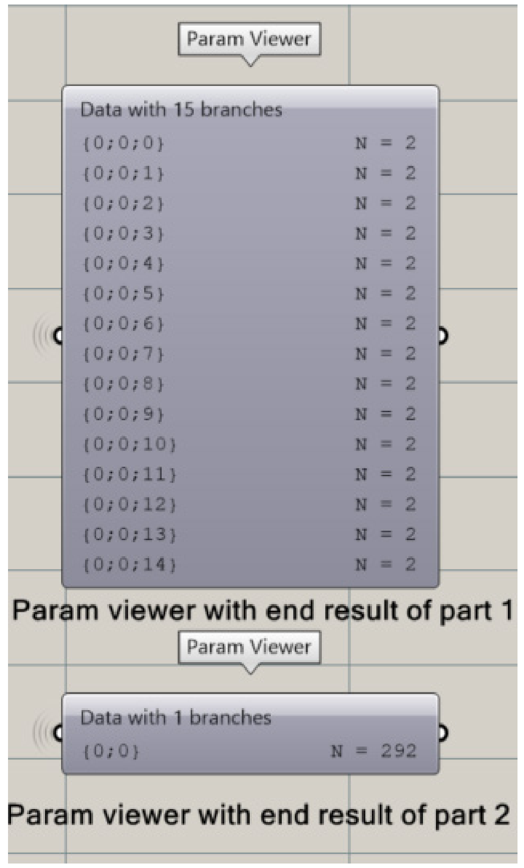
The script in rectangle 2 takes the intersections of the curves and outputs indices for the first and second intersection and parameters on where the intersection is, both on the first and second curve in an intersection event. These points are then used as parameters to split the original curve into segments based on the connection points.



The script in box 3 deletes all the duplicate (continuous) lines that we created in part 1 and box 2 of this part. The result is visually not any different than the end result of part 1, but through the param viewer component it shows that there are almost ten times as much lines in the same structure, segmenting the continuous curves worked.

The top one shows the data structure for the end result of part 1 (keep in mind that this is for one set of parallel curves), which shows a total of 30 curves in 15 branches - each branch contains 2 curves per layer.

The param viewer with the resulting curves from this part shows 292 curves, the data is flattened in the process and therefore in one branch.



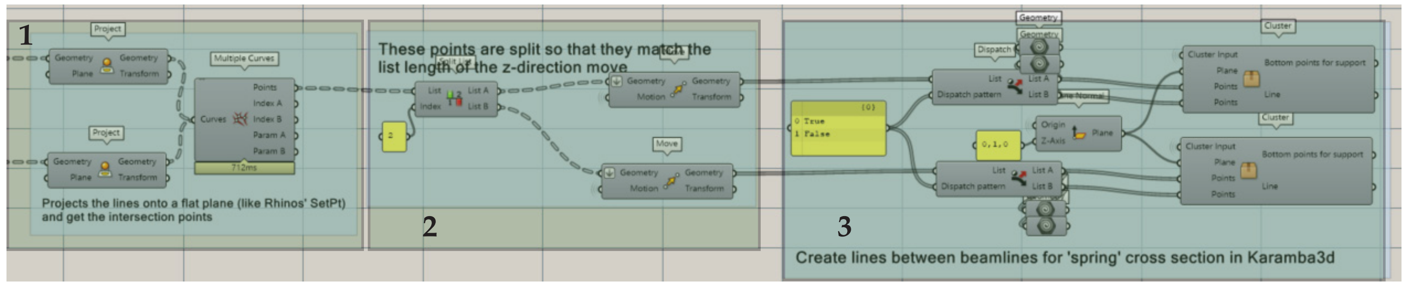
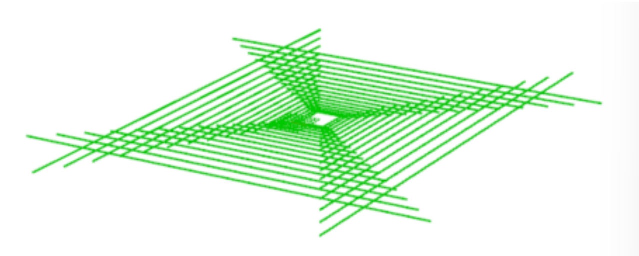


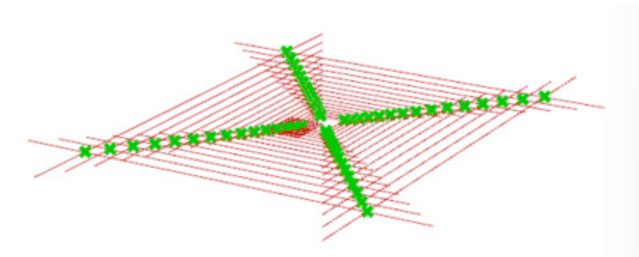
Figure 10.3: Grasshopper script part 3 - generating vertical lines for the joints (own work, 2024)

Part 3: Generating vertical lines for the joints

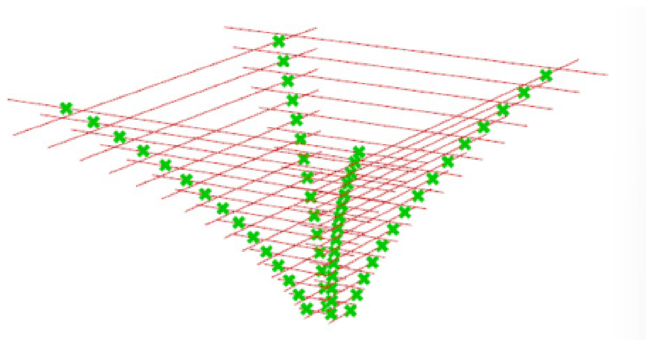
The script in rectangle 1 takes the result curves from part 1 and flattens these onto the z-axis.



To then take the intersection of the outermost curves with each other.



This yield a total of 60 points, which need to be projected, or moved onto the respective curves again. However because there are only 30 layers, the list of 60 points is split into two equally sized lists, and then moved to each layer (red lines are not from this part).



The last step moved the points onto the geometry, but the points are alternating, meaning on one layer the points are on the left, and the other the points are on the right, whereas all points should be on both layers.

The next piece of the script, in rectangle 3, does this. It mirrors the points along the centre of the column to get the points on both sides.

In the cluster short lines are drawn (one for each side) between the points that function as joints.

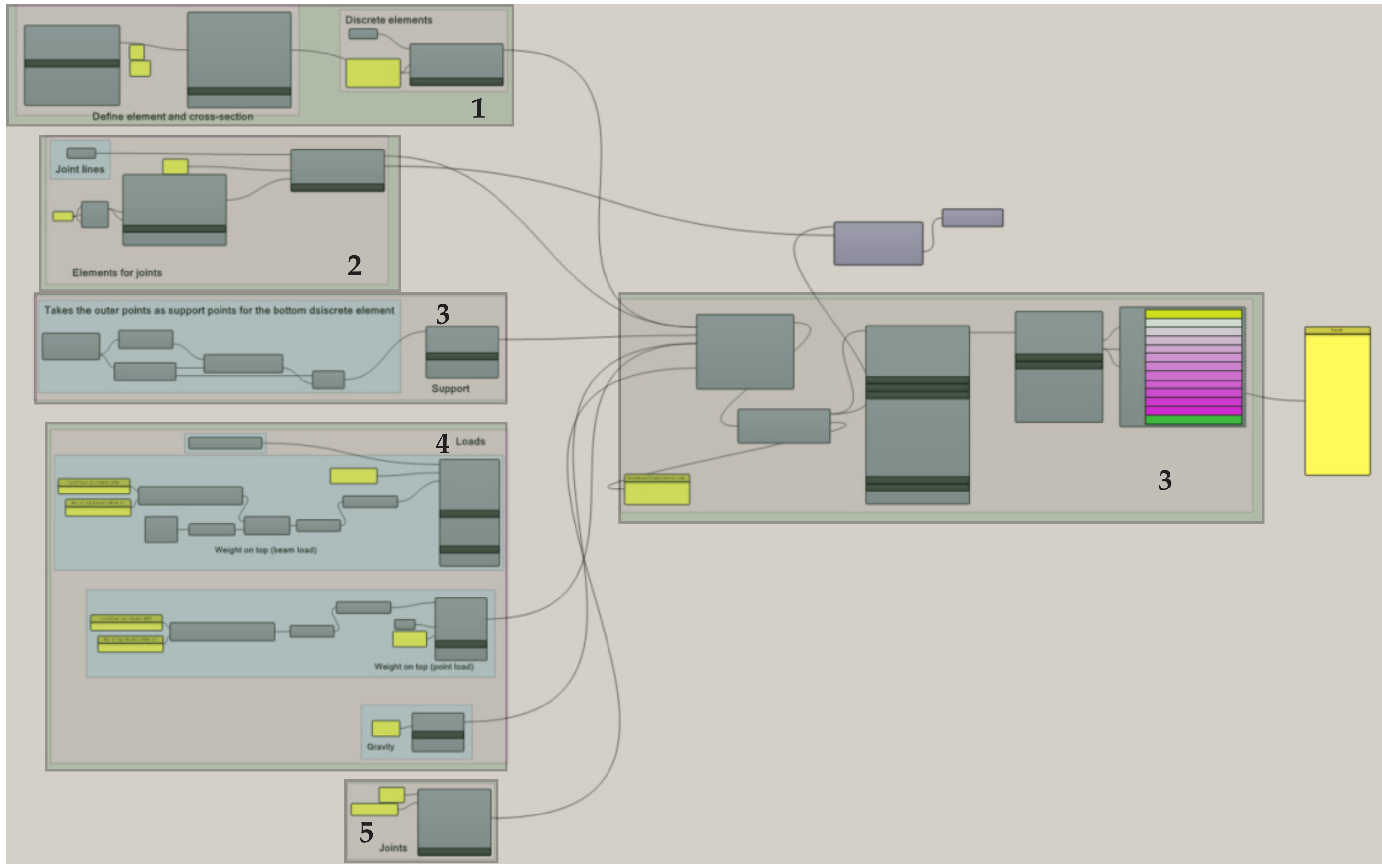


Figure 10.4: Karamba3D script (own work, 2024)

Part 4: Karamba3D

The script in rectangle 1 defines the material and cross section shape and size, and used these, together with the segmented beams from part 2, in Karamba3D's line-to-beam component which translates lines into structural beams.

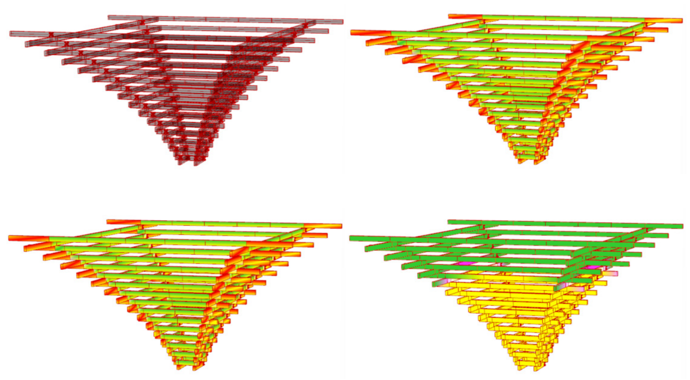
The script in box 2 takes the vertical lines, from part 3, and translates these into so called spring elements, by using the 'spring' cross section for the vertical lines. With the spring element one can define translational and rotational stiffness relations between two nodes.

Rectangle number 3 defines the support points at the bottom of the column. For this all the points in the column are sorted based on their height, and only the points with height (z-value) of 0 are used as input.

In rectangle number 4 the loads are defined. Two load types are used, gravity and the weight of the building above this column. The gravity is automatically applied to all elements in a calculation. The weight of the building that is above this column is defined by point loads, placed directly above the joint. Ideally, a block load (an equally divided line load) is a better representation but the input for block loads take kN/m, but the load would be applied more often than per meter, resulting in a higher load (also explained in chapter 7.2).

Block number 5 is the joint component, that says that there are connections between the spring elements and the discrete elements.

The last block, number 6, calculates the input and shows the resulting structure's cross section, axial stress, utilization, and displacement (shown in the same sequence below).



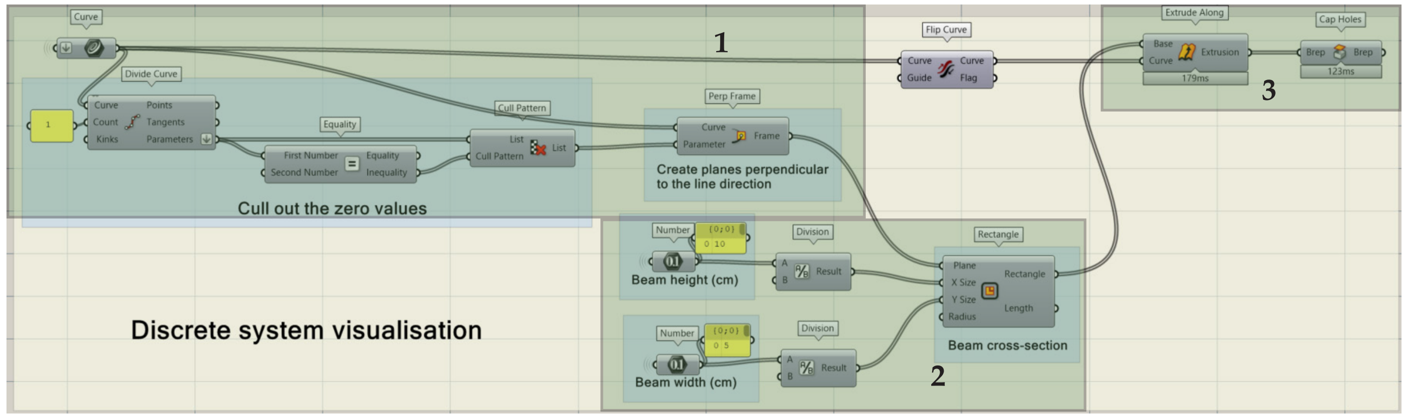
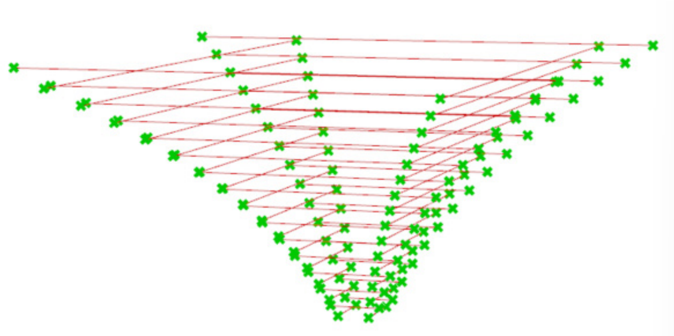


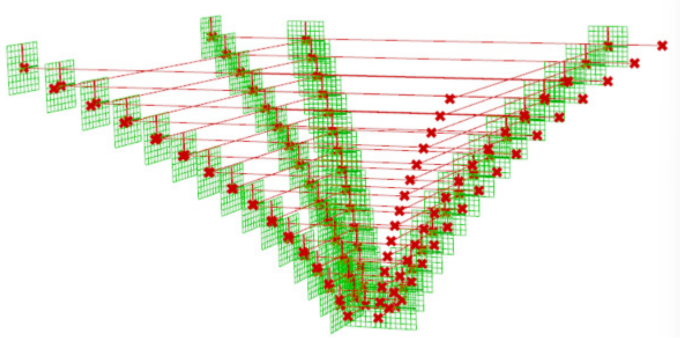
Figure 10.5: Visualisation of the discrete timber column (own work, 2024)

Part 5: Visualisation of the discrete timber column

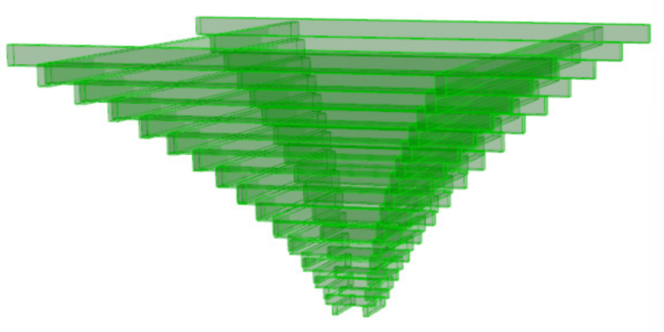
The script in box 1 take the beam outlines (from part 1 and places points on the ends of the beam.



Then (in box 2), on these points, perpendicular to the lines, a frame (or plane) is created, and then using this plane, a rectangle is drawn on the same plane - this rectangle has the dimensions of the cross section of the beams.



As a last step, in rectangle 3, the cross sections are extruded along the horizontal beam lines to get the actual discrete elements, aggregated into a discrete



timber column.

Appendix C: Timber materials in Grasshopper

Material: Hardwood 'D18(parallel)' E:950[kN/cm2] G12:475[kN/cm2] G3:59[kN/cm2] gamma:5.7[kN/m3] alphaT:5.0E-6[1/C°] ft:1.8[kN/cm2] fc:-1.8[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D18(orthogonal)' E:63[kN/cm2] G12:31.5[kN/cm2] G3:59[kN/cm2] gamma:5.7[kN/m3] alphaT:5.0E-6[1/C°] ft:1.8[kN/cm2] fc:-0.48[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D24(parallel)' E:1000[kN/cm2] G12:500[kN/cm2] G3:63[kN/cm2] gamma:5.8[kN/m3] alphaT:5.0E-6[1/C°] ft:2.4[kN/cm2] fc:-2.1[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D24(orthogonal)' E:67[kN/cm2] G12:33.5[kN/cm2] G3:63[kN/cm2] gamma:5.8[kN/m3] alphaT:5.0E-6[1/C°] ft:2.4[kN/cm2] fc:-0.49[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D27(parallel)' E:880[kN/cm2] G12:440[kN/cm2] G3:66[kN/cm2] gamma:6.1[kN/m3] alphaT:5.0E-6[1/C°] ft:2.7[kN/cm2] fc:-2.2[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D27(orthogonal)' E:46.9[kN/cm2] G12:23.45[kN/cm2] G3:66[kN/cm2] gamma:6.1[kN/m3] alphaT:5.0E-6[1/C°] ft:2.7[kN/cm2] fc:-0.51[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D30(parallel)' E:1100[kN/cm2] G12:550[kN/cm2] G3:69[kN/cm2] gamma:6.4[kN/m3] alphaT:5.0E-6[1/C°] ft:3[kN/cm2] fc:-2.4[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D30(orthogonal)' E:73[kN/cm2] G12:36.5[kN/cm2] G3:69[kN/cm2] gamma:6.4[kN/m3] alphaT:5.0E-6[1/C°] ft:3[kN/cm2] fc:-0.53[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D35(parallel)' E:1200[kN/cm2] G12:600[kN/cm2] G3:75[kN/cm2] gamma:6.5[kN/m3] alphaT:5.0E-6[1/C°] ft:3.5[kN/cm2] fc:-2.5[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D35(orthogonal)' E:80[kN/cm2] G12:40[kN/cm2] G3:75[kN/cm2] gamma:6.5[kN/m3] alphaT:5.0E-6[1/C°] ft:3.5[kN/cm2] fc:-0.54[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D40(parallel)' E:1300[kN/cm2] G12:650[kN/cm2] G3:81[kN/cm2] gamma:6.6[kN/m3] alphaT:5.0E-6[1/C°] ft:4[kN/cm2] fc:-2.7[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D40(orthogonal)' E:87[kN/cm2] G12:43.5[kN/cm2] G3:81[kN/cm2] gamma:6.6[kN/m3] alphaT:5.0E-6[1/C°] ft:4[kN/cm2] fc:-0.55[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D45(parallel)' E:1350[kN/cm2] G12:675[kN/cm2] G3:84[kN/cm2] gamma:7[kN/m3] alphaT:5.0E-6[1/C°] ft:4.5[kN/cm2] fc:-2.9[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D45(orthogonal)' E:90[kN/cm2] G12:45[kN/cm2] G3:84[kN/cm2] gamma:7[kN/m3] alphaT:5.0E-6[1/C°] ft:4.5[kN/cm2] fc:-0.58[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D50(parallel)' E:1400[kN/cm2] G12:700[kN/cm2] G3:88[kN/cm2] gamma:7.4[kN/m3] alphaT:5.0E-6[1/C°] ft:5[kN/cm2] fc:-3[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D50(orthogonal)' E:93[kN/cm2] G12:46.5[kN/cm2] G3:88[kN/cm2] gamma:7.4[kN/m3] alphaT:5.0E-6[1/C°] ft:5[kN/cm2] fc:-0.62[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D55(parallel)' E:1550[kN/cm2] G12:775[kN/cm2] G3:97[kN/cm2] gamma:7.9[kN/m3] alphaT:5.0E-6[1/C°] ft:5.5[kN/cm2] fc:-3.2[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D55(orthogonal)' E:103[kN/cm2] G12:51.5[kN/cm2] G3:97[kN/cm2] gamma:7.9[kN/m3] alphaT:5.0E-6[1/C°] ft:5.5[kN/cm2] fc:-0.66[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D60(parallel)' E:1700[kN/cm2] G12:850[kN/cm2] G3:106[kN/cm2] gamma:8.4[kN/m3] alphaT:5.0E-6[1/C°] ft:6[kN/cm2] fc:-3.3[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D60(orthogonal)' E:113[kN/cm2] G12:56.5[kN/cm2] G3:106[kN/cm2] gamma:8.4[kN/

m3] alphaT:5.0E-6[1/C°] ft:6[kN/cm2] fc:-1.05[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D65(parallel)' E:1850[kN/cm2] G12:925[kN/cm2] G3:116[kN/cm2] gamma:9[kN/m3] alphaT:5.0E-6[1/C°] ft:6.5[kN/cm2] fc:-3.5[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D65(orthogonal)' E:123[kN/cm2] G12:61.5[kN/cm2] G3:116[kN/cm2] gamma:9[kN/m3] alphaT:5.0E-6[1/C°] ft:6.5[kN/cm2] fc:-1.13[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D70(parallel)' E:2000[kN/cm2] G12:1000[kN/cm2] G3:125[kN/cm2] gamma:9.6[kN/m3] alphaT:5.0E-6[1/C°] ft:7[kN/cm2] fc:-3.6[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D70(orthogonal)' E:133[kN/cm2] G12:66.5[kN/cm2] G3:125[kN/cm2] gamma:9.6[kN/m3] alphaT:5.0E-6[1/C°] ft:7[kN/cm2] fc:-1.2[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D75(parallel)' E:2200[kN/cm2] G12:1100[kN/cm2] G3:138[kN/cm2] gamma:10.2[kN/m3] alphaT:5.0E-6[1/C°] ft:7.5[kN/cm2] fc:-3.7[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D75(orthogonal)' E:147[kN/cm2] G12:73.5[kN/cm2] G3:138[kN/cm2] gamma:10.2[kN/m3] alphaT:5.0E-6[1/C°] ft:7.5[kN/cm2] fc:-1.28[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D80(parallel)' E:2400[kN/cm2] G12:1200[kN/cm2] G3:150[kN/cm2] gamma:10.8[kN/m3] alphaT:5.0E-6[1/C°] ft:8[kN/cm2] fc:-3.8[kN/cm2] flowHypo: Rankine;

Material: Hardwood 'D80(orthogonal)' E:160[kN/cm2] G12:80[kN/cm2] G3:150[kN/cm2] gamma:10.8[kN/m3] alphaT:5.0E-6[1/C°] ft:8[kN/cm2] fc:-1.35[kN/cm2] flowHypo: Rankine;

Material: GlulamTimber 'GL28h' E1:1260[kN/cm2] E2:42[kN/cm2] G12:78[kN/cm2] nue12:-1 G31:78[kN/cm2] G32:78[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:1.95[kN/cm2] ft2:0.05[kN/cm2] fc1:2.65[kN/cm2] fc2:0.3[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL28c' E1:1260[kN/cm2] E2:39[kN/cm2] G12:72[kN/cm2] nue12:-1 G31:72[kN/cm2] G32:72[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:1.65[kN/cm2] ft2:0.05[kN/cm2] fc1:2.4[kN/cm2] fc2:0.27[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL32h' E1:1370[kN/cm2] E2:46[kN/cm2] G12:85[kN/cm2] nue12:-1 G31:85[kN/cm2] G32:85[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:2.25[kN/cm2] ft2:0.05[kN/cm2] fc1:2.9[kN/cm2] fc2:0.33[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL32c' E1:1370[kN/cm2] E2:42[kN/cm2] G12:85[kN/cm2] nue12:-1 G31:85[kN/cm2] G32:85[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:1.95[kN/cm2] ft2:0.05[kN/cm2] fc1:2.65[kN/cm2] fc2:0.3[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL36h' E1:1470[kN/cm2] E2:49[kN/cm2] G12:91[kN/cm2] nue12:-1 G31:91[kN/cm2] G32:91[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:2.6[kN/cm2] ft2:0.05[kN/cm2] fc1:3.1[kN/cm2] fc2:0.36[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL36c' E1:1470[kN/cm2] E2:46[kN/cm2] G12:85[kN/cm2] nue12:-1 G31:85[kN/cm2] G32:85[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:2.25[kN/cm2] ft2:0.05[kN/cm2] fc1:2.9[kN/cm2] fc2:0.33[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL24h(PerpendicularToGrain)' E1:39[kN/cm2] E2:1160[kN/cm2] G12:72[kN/cm2] nue12:-1 G31:72[kN/cm2] G32:72[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:1.65[kN/cm2] fc1:0.27[kN/cm2] fc2:2.4[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL24c(PerpendicularToGrain)' E1:32[kN/cm2] E2:1160[kN/cm2] G12:59[kN/cm2] nue12:-1 G31:59[kN/cm2] G32:59[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:1.4[kN/cm2] fc1:0.24[kN/cm2] fc2:2.1[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL28h(PerpendicularToGrain)' E1:42[kN/cm2] E2:1260[kN/cm2] G12:78[kN/cm2] nue12:-1 G31:78[kN/cm2] G32:78[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:1.95[kN/cm2] fc1:0.3[kN/cm2] fc2:2.65[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL28c(PerpendicularToGrain)' E1:39[kN/cm2] E2:1260[kN/cm2] G12:72[kN/cm2] nue12:-1 G31:72[kN/cm2] G32:72[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:1.65[kN/cm2] fc1:0.27[kN/cm2] fc2:2.4[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL32h(PerpendicularToGrain)' E1:46[kN/cm2] E2:1370[kN/cm2] G12:85[kN/cm2] nue12:-1 G31:85[kN/cm2] G32:85[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:2.25[kN/cm2] fc1:0.33[kN/cm2] fc2:2.9[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL32c(PerpendicularToGrain)' E1:42[kN/cm2] E2:1370[kN/cm2] G12:85[kN/cm2] nue12:-1 G31:85[kN/cm2] G32:85[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:1.95[kN/cm2] fc1:0.3[kN/cm2] fc2:2.65[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL36h(PerpendicularToGrain)' E1:49[kN/cm2] E2:1470[kN/cm2] G12:91[kN/cm2] nue12:-1 G31:91[kN/cm2] G32:91[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:2.6[kN/cm2] fc1:0.36[kN/cm2] fc2:3.1[kN/cm2] flowHypo: Rankine

Material: GlulamTimber 'GL36c(PerpendicularToGrain)' E1:46[kN/cm2] E2:1470[kN/cm2] G12:85[kN/cm2] nue12:-1 G31:85[kN/cm2] G32:85[kN/cm2] gamma:5[kN/m3] alphaT1:5.0E-6[1/C°] alphaT2:5.0E-6[1/C°] ft1:0.05[kN/cm2] ft2:2.25[kN/cm2] fc1:0.33[kN/cm2] fc2:2.9[kN/cm2] flowHypo: Rankine