Parametric hybrid modular timber construction

Computational design approach for modular construction to discover different building typologies based on global and local structural requirements.

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Building Engineering
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by

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Preface

The basis for this research originally stemmed for my passion for developing and pushing the boundaries of prefabricated systems through computational design. With a focus on multiple building typologies and recent developments in the timber industry.

I want to thank my committee members for their critical view and continuous support over the past 11 months.

M.L.G. Hamelijnck
Delft, August 2021

Abstract

This master thesis' main objective is to develop a parametric design tool which improves the design process and explores the building typologies of a hybrid modular timber construction based on global and local structural requirements. The hybrid modular timber construction is core-less and combines load-bearing and corner-supported modules. The parametric design tool, which is named Habitat21, consists of an automatic placement function (Grasshopper), a manual placement function (Rhinoceros) and a structural analysis program (Karamba).

The hybrid modular timber construction concept is tested by comparing the displacements in the x- and y-direction of a replicate of Hotel Jakarta, generated in Habitat21, with the original displacements in the x- and y-direction of Hotel Jakarta, generated by the case study of Gijzen. It appears that the replicated hybrid modular timber construction of Hotel Jakarta in Habitat 21 is able to approximately reproduce the original displacement in the y-direction of Hotel Jakarta calculated by Gijzen (difference of 0.642 mm). However, the displacement in the x-direction could not be reproduced in the same order of magnitude as the original displacement in the x-direction (difference of 13.081 mm). This could be caused by the fact the assumed inter-module connections, connection 1 and 4, of the hybrid modular timber construction concept were modelled as line elements with two hinges ("pendelstaven"), this could have influenced the displacement behaviour of the CLT shear wall mechanism.

To examine the usability of the manual placement of the modules by Rhinoceros within Habitat21 a workshop with 5 participants (2 architects and 3 engineers) and two cases is performed. In the first case each participant created 3 buildings, each building was generated within 8 minutes, resulting in a total of fifteen different building typologies of the hybrid modular timber construction concept. Ten out of the fifteen building typologies satisfied the global structural requirements, five did not satisfy the global structural requirements. Two buildings were not stable in one or more directions, two buildings did not satisfy the unit check for the displacement in the x-, y- or z-direction and for one building an error occurred. To explore the diversity of the typologies from the fifteen buildings a typology score was given for each building. The highest score was 102 points and the lowest score was 10. This indicates a relatively large diversity in typologies. In the second case each participant worked on two buildings that both did not satisfy the global and local structural requirements but the second building had 2 extra design restrictions. The participants had 5 minutes per building to fix this. For the building with extra design restrictions three participants (all engineers) were not able to meet the local structural requirements. This suggests that the feedback from Habitat21 provided to the user in this case was not adequate.

To examine the automatic placement of the modules by Grasshopper within Habitat21 the calculation time of the automatic placement of one extra CLT shear wall is analysed for 6 different buildings by using an iterative process. These buildings were all stable in the y- and z-direction but unstable in the x-direction. The calculation time for the placement of the extra CLT shear wall at a building with 4 modules and 8 possible positions was 6 seconds and the calculation time for the placement of the extra CLT shear wall at a building with 24 modules and 48 possible positions was 360 seconds. Combining the calculation times of the 6 different buildings it appeared that the calculation time increased exponentially when the amount of modules increased linearly.

In conclusion, it was possible to discretize a hybrid modular timber construction in a parametric design tool. Furthermore, both engineers and architects were able to work with the tool and to independently create diverse typologies of a hybrid modular timber construction. The boundaries of the hybrid modular timber construction concept in Habitat 21 were determined by the inter-module connections, as found in the comparison study with Hotel Jakarta.

Applicability of the parametric design tool and the hybrid modular timber construction concept could be increased by more research into inter-module connections and point supported cross-laminated timber slabs. Also, feedback about how the structural systems works could be implemented in the parametric design tool, where the user gets necessary feedback to fulfill the global or local structural requirements instead of using a trial and error design method.

Contents

1 Introduction

1.1. Challenges construction industry

Currently the construction industry in the Netherlands is facing four challenges: the urban housing shortage, a need to humanize high-density cities, an increasing responsibility towards the environment and a conventional and time-consuming way of working between architects and engineers.

1.1.1. Housing shortage

The housing industry faces challenging times: people can not find a home for themselves due to high selling prices and low availability of houses on the market. The Dutch central government calculated the necessary demand for 2020 to 2030 and concluded that 845 000 extra homes are required to fill the market gap.[53] Various causes contribute to this shortage: population growth, urbanisation, shifting trends of lifestyle, labour shortage and productivity in the construction sector and scarcity of building plots.

The demand for residential buildings is rising due to population growth, urbanization and shifting trends in lifestyle. The expectation is that the population will expand with 1 million to 18,3 million people in 2035. CBS estimated that large cities keep emerging. And due to urbanisation large cities will increase more in size than surrounding areas, an increase between 15 and 20 per cent. [CBS] Single-parent families put further pressure on the housing stock. The result of this is visible in Figure 1.1, where can be seen that the average person per household is decreasing. [81]

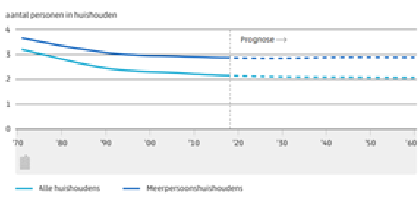


Figure 1.1: The average person per household (vertical axis) is declining per year (horizontal axis). [81]

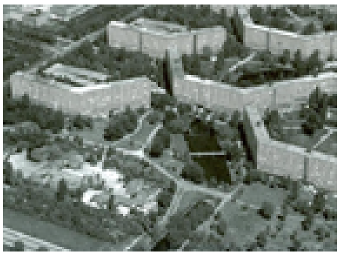
Due to ageing, and a less attractive work environment the labour shortage is rising in the building sector. Furthermore, the combination of a non-automated and non-digitalized industry results in lower labour productivity than other industries. [38] [53]

Building sites are getting scarcer in the Netherlands, as it has one of the world's highest population densities. [20] Especially in urban areas where urbanization strengthens the shortage of building sites.

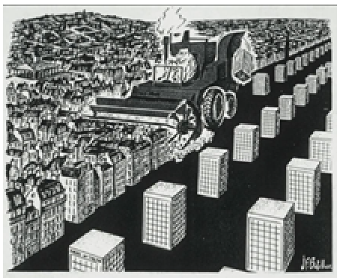
1.1.2. Need to humanize high-density cities

In the 50s and 80s prefabricated concrete residential buildings were a common way to build a large amount of residential buildings. Examples of these buildings are given in figure 1.2a: flats in the Bijlmermeer from the 70s. According to the CBS (2015) 32% of the households are living in such an apartment building, exact number about the height of these apartments' buildings are missing, but a height between 4 and 12 stories is assumed.

Today, with rapid urbanization, population growth and scarcity of building plots efficient high-density structures have risen in urban areas. Instead of creating cities step by step, individual complexes are made (see figure 1.2b). The Zalmhaventoren (see figure 1.2c) is an example of such a complex in Rotterdam. Humanizing these mega-scale high-density complexes is a big challenge according to architects and engineering companies [SOM].



(a) Bijlmermeer



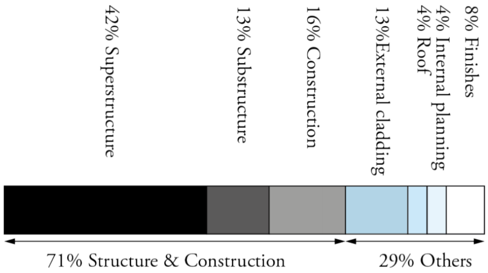
(b) Individual complexes

(c) Zalmhaventoren

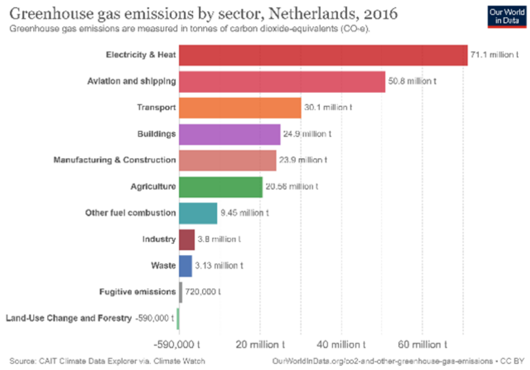
Figure 1.2: Overview high density cities problem

1.1.3. Increasing responsibility towards the environment

Another challenge for the construction industry is the global greenhouse gas emission. The construction industry is responsible for a large part of these global greenhouse gas emissions. In the Netherlands, the Dutch manufacturing and construction industry is responsible for 11% of the total Dutch greenhouse gasses (Figure 1.3b). The reduction of emissions is obtained by sustainable solutions as solar panels, isolation, and triple-glass for the use-phase of buildings. However, when zoomed in on the embodied carbon, a large part of the total emissions can be attributed to the embodied carbon of a building's structure (Figure 1.3a).



(a) Average embodied carbon in offices, hospitals and schools [26]



(b) Greenhouse gas emissions by sector, Netherlands 2016 [66]

Figure 1.3: Overview environmental data

Besides the greenhouse gas emissions, waste is a significant problem in the Dutch building sector. It is responsible for almost half of the total waste in the Netherlands, and 23% of all production in the building sector is waste [CBS].

1.1.4. Conventional way of working between architect and engineer

The conventional way of working in the construction sector can be seen as a barrier to explore more design options. The construction industry still relies on unique projects and conventional design methods [20]. In a conventional design method the structural engineer adapts to the model of the architect. This initial model can be limiting, and small changes can require redraft and time-consuming modifications. It is often an inefficient process which might lead to a disappointing outcome for both. The first draft may have an appealing design but might be too challenging and expensive to construct. So the design process is limited and does not allow further exploration of the design without creating additional work.

However, it might be interesting to research the development of a design process where multiple designs can be assessed on specific parameters. Such a computational, generative, parametric design tool could inform stakeholders in the early stage of a design about the project's feasibility. [46] Nevertheless, an informed decision tool that gives feedback to the structural engineer or architect is not available yet.

1.2. Prefabricated building systems

Often a prefabricated building system is used to construct residential buildings. A small history about prefabricated building systems can be found in appendix A.1. The typology of these building depends largely on the boundaries of the prefabricated structural building system. Two main building typologies can be distinguished: simple stacked and flexible typology (see figure1.4).

(a) Simple stacked building typology (b) Flexible building typology

Figure 1.4: Building typologies

The degree of prefabricated structural elements of a prefabricated building system can be divided in: linear (1D), planar (2D) or spatial (3D) elements. These elements determine the prefabricated building system's characteristic construction properties. A high degree of prefabrication results in less construction time, due to the lower degree of onsite work. However, a higher degree of prefabrication reduces the design flexibility by having restrictions on dimensions (see figure 1.5). Another commonly used word for a prefabricated building system with 3D spatial elements is modular construction, where only 3D elements are used to form a building system. [71] Modular construction gained attention following recent reports, because of the advantages over conventional and other prefabricated construction methods. [47] [53] [60]

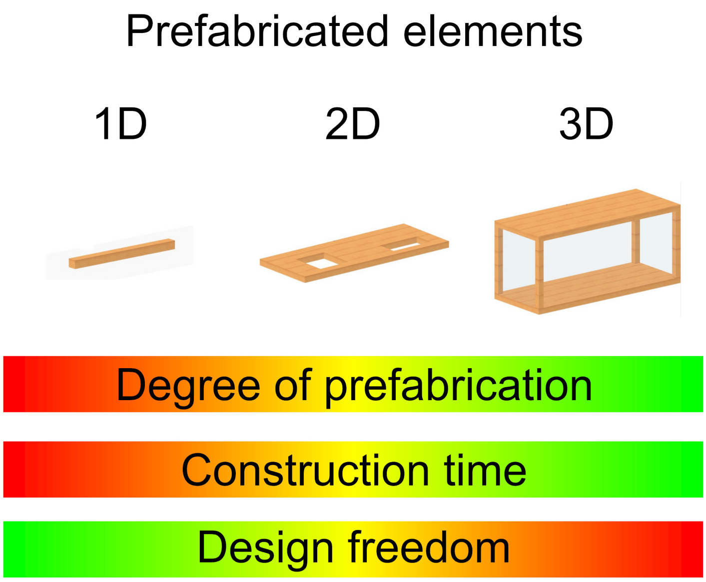
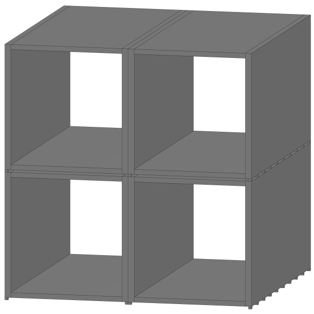


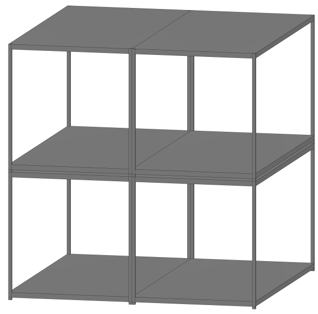
Figure 1.5: Distinction between building systems and influence on construction time and desgn freedom [own]

1.2.1. Modular construction

In literature a distinction is made between load-bearing and corner supported modules for modular construction. Figure 7.18 provides an overview of both geometries.



(a) Load bearing modules [own]



(b) Corner supported modules [own]

Figure 1.6: Timber module configurations

Load-bearing wall modules are often used in concrete and timber buildings. The walls are used to transfer the gravity loads and lateral loads to the foundation. A concrete or steel core is sometimes added to transfer the lateral loads. This typology is often used for hotels and studios, since the design flexibility is low.

Corner supported modules are usually made of steel, where the gravity loads are transferred through the floor to the edge beams and from the corner columns to the foundation. Braced modules or a concrete core transfer the lateral loads to the foundation. Corner supported modules have a better structural performance to weight ratio and have more flexibility in the architectural design. Due to the framing system it is possible to create larger open areas. [47] [50]

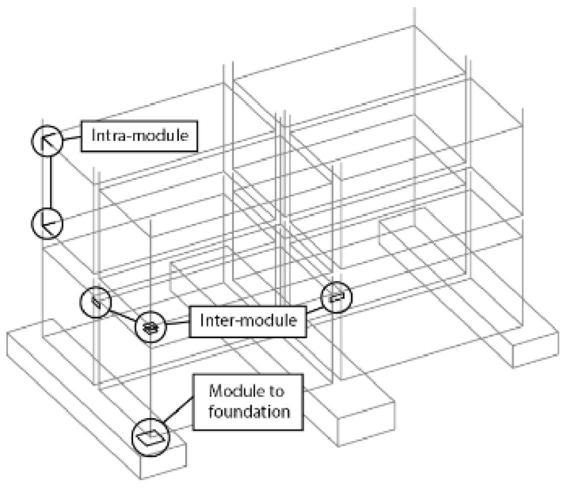


Figure 1.7: Three types of connections in modular construction [45]

Connections for modular systems can be grouped into three different categories: intra-, inter-, and foundation-connections. The intra-module connections assemble the components to form a module, the inter- and foundation-connections assemble all modular units on the building site. The intra-module connections are placed off-site. The inter-module connections assemble all modular units to form a building and are placed on-site. The foundation-connections connect modules with the foundation. The preference is given to bolted or carpentry connections regarding the assembling and the disassembling to give the components of the building system a second life. Apart from having adequate strength, stiffness and ductility, connections are preferred simple for manageable costs and ease in assembly.

The stability systems for concrete and steel modular buildings are presented in table 1.1. The stability system also depend on the material of the building system, this is explained in a further section. The inter-module connection is critical in modular buildings since it has influence on the lateral stiffness. [45] Lawson suggests that moment-resisting connections consisting of steel end-plates or a deep fin plate may provide lateral stiffness for low-rise buildings. For higher buildings often additional lateral stability systems are used. [47]

Storeys	Lateral stiffness
1-4	Moment resisting connections
4-6	Diaphragm action within the modules
6-10	A seperate bracing system in lift or stair shaft
10 <	Additional concrete core

Table 1.1: Stability systems defined by Lawson, for concrete and steel modular buildings. [47]

Modular construction is used in the construction of low-rise and high-rise buildings. There is a difference in the maximum amount of storeys based on the modular systems' principal material (see Table 1.2). Most modular buildings systems are made from steel or concrete, and most modular timber building systems resist lateral loads by having a concrete core (see section 1.2.3). The maximum height for modular timber construction is lower due to the stiffness of engineered timber products compared to concrete and steel. And in combination with the low stiffness of timber connections, large global deformations occur in timber structures. Guidelines for modular connections, especially timber, are limited. [69] This is why the maximum height of modular timber frame buildings is limited to 4 stories [47] and modular load-bearing modules to 8 stories. [30] Although the load-bearing timber modules can be used for higher buildings, this system reduces the design flexibility significant by dividing the building into small fragments. Corner supported modules provide more flexibility and can create open spaces.

Material	Topology	Completed [year]	Floors	Average module weight [kN/m2]
Steel	simple stacked	2012	32	4-6
Concrete	simple stacked	2016	44	9-15
Timber	simple stacked	2009	8	4-6

Table 1.2: Overview modular construction by material [30]

Concrete and steel are often the principal material in modular construction but there are fundamental reasons to favour the use of timber as principal material. Concrete is responsible for 1,5% of the total carbon emissions in the Netherlands and 9-10% worldwide. [77] Dutch concrete is already relatively sustainable, but recent case studies showed better alternatives for concrete, like engineered timber products. A building made of cross-laminated-timber (CLT[1]) has a positive carbon footprint. [15]

[1]See chapter 3 for introduction CLT

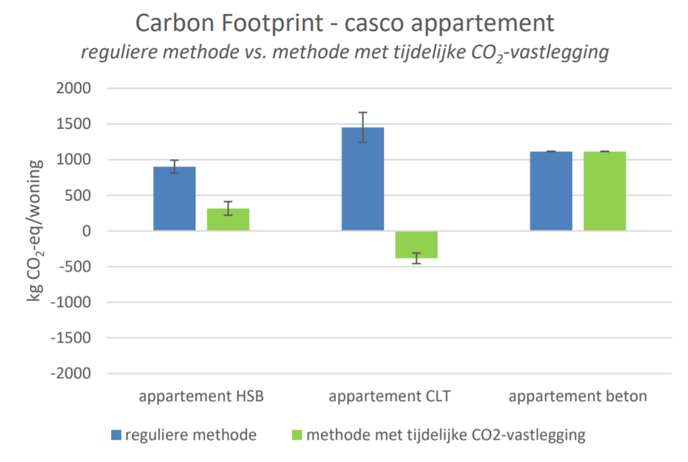


Figure 1.8: The carbon footprint with and without locked carbon. [43]

Timber[2] is a renewable material and does not generally require as much processing as concrete or steel. With good and sustainable forest management it is possible to sequester carbon by using timber in construction. Compared to traditional materials, like steel and concrete, construction with timber can save approximately 45 tons of carbon per dwelling. [19] This can make a difference for the carbon footprint, especially when timber construction is applied at a macro scale. Also recent studies of TNO indicate that the carbon footprint of timber buildings is lower when the locked carbon is taken into account (see figure 1.8). [43] However, the energy needed for drying and adhesives in processed timber products negatively impact the embodied energy. Figure 1.9 shows the amount of energy necessary per timber species; approximately 90% of the total manufacturing energy goes to drying. [64]

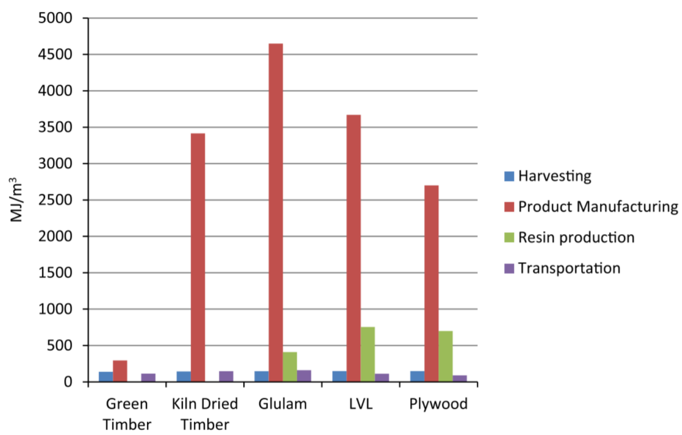


Figure 1.9: Typical embodied energy of construction timber products. [64]

[2]The meaning of timber can relate to processed and unprocessed wood. This research will focus on processed wood.

1.2.2. Observed benefits and downside modular construction

In multiple reports is suggested that modular construction can be a solution for the shortage of dwellings since labour productivity rises. A productivity growth up to 40% can be expected with modular construction as a building method. [53] [30] [60] Lawson et al. researched modular construction and exposed the main benefits. Reducing material, wastage, transport activities, nuisance, embodied energy, accidents, working ours, and embodied carbon and energy can be achieved while productivity and efficiency increase with a modular construction method.[47] Table 1.3 provides an overview of the observed benefits for modular construction.

Financial	Technical	Social & environmental
• Preparing the building site and manufacturing of modular units take place at the same time. This leads to shorter building times, possible earlier return on investment, and lower site management costs. [47] [45] • Reuse of the same building system can create an economy of scale in production, and because of the standardized system, the design costs could be reduced. [47] • Opportunity to dismantle the building while maintaining the asset value of the modular units. [53]	• The lightweight (timber 500 kg/m3, concrete 2450 kg/m3) construction makes it suitable for sites with poor foundation conditions, better handling on-site. [49] • The off-site manufacturing and transporting makes it suitable for narrow sites. [49] • Modular construction can lead to higher quality housing, because the units are built using precise measurements and the indoor process protects units from damage. [37]	• A reduction of carbon dioxide emissions. [62] • There is a more significant opportunity for recycling materials since the production is in a factory. [37] • Modular construction can reduce landfill waste by at least 70%. [48] • Lightweight, less material use, and less wastage compared to site-intensive construction. [48] • Less nuisance for the neighbourhood during construction, reduced from 20% to 50% [48] [30] • Reduction in delivery vehicles up to 70%. [30] • Better work environment for construction workers, almost 80% fewer accidents, relative to intensive site construction. [30]

Table 1.3: Observed benefits during literature research for modular construction

Downside

The observed downside of modular construction can be divided among two topics: financial and technical design. The technical disadvantages are related to the engineering challenges, while the financial is related to the additional costs compared to conventional construction methods. Table 1.4 provides an overview.

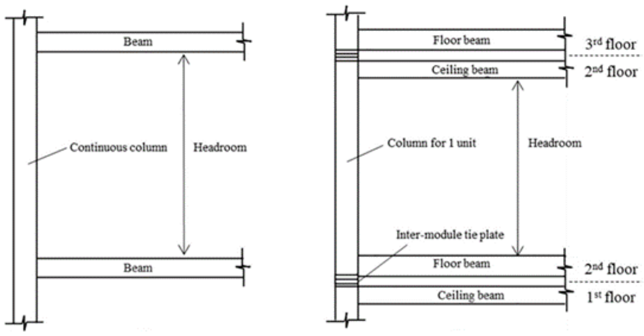


Figure 1.10: Headroom difference with a conventional and modular construction system. [50]

Financial	**Technical Design**
• The payments for traditional buildings can be made in different stages of the building process, while most of the payments should be done at the beginning of the process for prefabricated buildings. In addition to this, financial institutions prefer to invest money on traditional projects. [39] [30] • The weight of the modules is limited by the maximum lifting capacity of the tower cranes. Most used tower cranes have a capacity of 20 000 kilograms. The costs of the crane will increase up to 60% when the modular unit exceeds the weight of 20 000 kilograms. [50] To reduce the construction costs, the weight of the module should be checked in the design stage. • A large number of joints compared to conventional buildings is observed in modular building systems. The number of joints is essential because it can affect the overall costs and the erection time of the building. [30] • From an economic point of view and design, it is desirable to reduce the construction depth and increase the headroom of a module. The modular system consists of a ceiling and floor beam, unlike conventional buildings where a single beam supports the ceiling of the lower and the floor of the upper story. (see figure 1.10) [50]	• Much research into modular steel and concrete structures is done, but recent research into modular timber construction is lacking. This may be one of the barriers to choose for timber construction and the limited application in the construction industry. [45] • The other limitation is the variation of structural plans, due to transportation limits. The maximum width for road transport that does not require a police escort is 3.5 meters with 27 meters in lenght and 4.25 in height. [60] [8] But this can be outweighed by the advantages of modular construction, especially for residential buildings, where a high level of repletion is observed. [53] [45] • Due to the increased number of joints in modular construction the continuity of structural components might have an impact on lateral stability. [30] • Another barrier may be the maximal height of the modular timber construction. With modular steel and concrete construction heights of respectively 32 and 44 stories have been reached, a significant difference to the maximum realized height of 8 stories with load-bearing modular timber construction. With timber framework modules the maximum height is set to 2 or 4. [6] [45] [30] The load bearing capacity and lateral stability are the indicated barriers. • For structural analysis, a notional horizontal force of 1%, which is higher than 0.5% for conventional steel frame building, was proposed for up to 12 storeys for steel modular construction. [50]

Table 1.4: Observed disadvantages during literate research for modular construction

1.2.3. Reference projects prefabricated building systems

A list of prefabricated and modular buildings (with timber) is compiled based on accessible literature. The buildings are analyzed on certain aspects which give insight into current modular timber construction possibilities. In table 1.5 for each building project the following aspects are noted: function, system, material, floors, year of completion, construction duration and typology. A division is made between the system of all reference project since the design differs from each other. Detailed information can be found in appendix A.

Project	**Function**	**System**	**Material**	**Floors**	**Completed [year]**	**Duration [months]**	**Typology**
Hotel Jakarta	Hotel	Modular (3D)	Timber	8	2018	3	Simple
Puukuoka	Residential	Modular (3D) Prefab (2D)	Timber	8	2014	6	Simple
Treet	Residential	Modular (3D) Prefab (1D)	Timber	14 (4)	2015	18 incl. groundwork	Simple
Brock Commons	Residential	Prefab (2D) Prefab (1D) Concrete Core	Timber Concrete	18	2017	2.5	Simple
Residential complex Zurich	Residential	Modular (3D)	Timber	6	2016	6	Simple
Woodie Student Hostel	Hotel	Modular (3D) Concrete Core	Timber Concrete	6	2017	10	Simple
50 Modular Timber Appartments	Residential	Modular (3D) Concrete Core	Timber Concrete	4	2015	9	Simple
Habitat 67	Residential	Modular (3D) Concrete Core	Concrete	12	1967	30	Flexible

Table 1.5: Overview reference prefabricated building projects. (1D = linear, 2D = planar, 3D = spatial)

Review reference projects

In every building multiple prefabricated elements are observed, not one reference project consist out of solely 3D prefabricated modular units. Also, most buildings used a concrete core as stability system and multiple principle materials are used. Only in Puukuoka and Treet timber is the principal material. The stability for Puukuoka is gained from the timber structure and for Treet from the timber diagonals at the perimeter of the building. Another observation is related to the typology of all projects. All modular projects (except Habitat 67), used a simple building typology.

The focus within modular multi-story residential buildings is mostly on mass production and quantity and not on the building's typology. [14] However the quality, design and aesthetics aspects should be considered as important as the quantity and mass production aspects of prefabricated buildings. [58]

1.2.4. Habitat 67

One famous architect that tried to reinvent the urban apartment building typology is Moshe Safdie with Habitat 67 (see figure 1.11a), by creating a flexible, affordable, and modular building system. Habitat 67 is one of the most recognizable landmarks of Canada and famous modular buildings. Concrete modular units form a residential building with own entrances and pedestrian paths between greenery areas leading to open terraces. One of Moshe Safdie design principles was to re-create an urban setting by creating suburban single-family homes with enough privacy. To make his affordable housing project feasible a lower-cost system per dwelling was necessary. Safdie's conceived the idea for Habitat 67 in his final thesis: "A three-dimensional modular building system".

(a) Habitat 67 by architect Moshe Safdie [59]

(b) The concrete modular construction of Habtiat 67 [59]

In 1967, the complex modular structure was delivered, it took 30 months to manufacture and ensemble the modular pieces together. The 12-story high concrete structure consisted of 354 modular units, which formed 158 dwellings in 18 different styles. The dimensions of one modular concrete unit were 5,33 meters by 11,48 meters and 3,2 meters high. Six elevator shafts provided vertical transport, which stopped every 4th floor. The interaction of houses and the monolithic concrete streets provided the overall stability against lateral loads. Reinforced concrete was the only available material in 1967 with a high structural capacity in both tension and compression, had surfaces that were not porous to moisture, and was fireproof.

Habitat 67 was an experiment that has tested the limits of both prefabrication in manufacturing and logistics as well. Transportation and manufacturing of the 3D modular units was one of the flaws, contractors and municipalities were unable to handle the amount of units. The second flaw is related to the financial aspect because the financial costs were higher than any other affordable housing project, due to the high proportion of exterior walls and cantilevered or bridge elements. Besides the fact that the massive amount of concrete does not meet nowadays sustainability standards of reducing the carbon footprint of buildings. Also, the walls were too thick, could have been 3 inch instead of the produced 5 inch. [67] [59]

Safdie's ideology and effort to create affordable housing by reinventing the apartment building's typology was not a success in the end. In 1967 it wasn't possible to produce these houses for a reasonable price. Nowadays Habitat 67 houses the wealthier demographic of Montreal and stands as a tourist attraction for architecture students.

1.3. Hybrid modular timber construction

Just one flexible building typology is observed during the reference project with modular construction, but brings a high carbon footprint. Only simple stacked building typologies for timber are observed during the reference projects. So it might be interesting to research the possibilities to combine the corner supported and load bearing module in one modular building system to form hybrid modular timber construction buildings. Such a system is proposed in Room Modules [40] by Huss et al.. In this system horizontal and vertical timber components (see Figure 1.12) are combined to form a finite number of modular units (see figure 1.13).

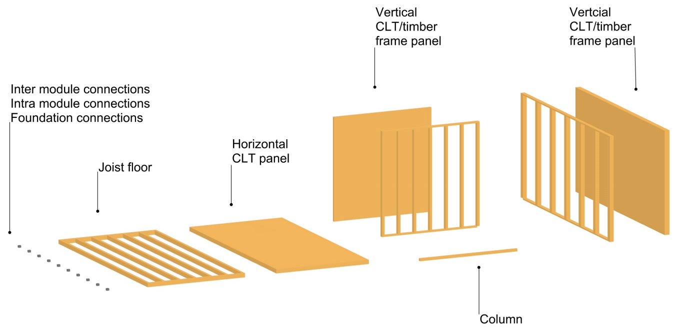


Figure 1.12: Timber components with connection types to form a hybrid modular unit [own]

1.3.1. Hybrid modules

The hybrid modular timber construction contains two types of modules. Open modules only contain a floor and a ceiling. Rigid modules contain a floor, a ceiling and one, two, three or four vertical CLT shear walls which provide the stability of the module. So at every vertical side of the module a CLT shear wall can be placed or not. This results in $4^2 = 16$ different possible module configurations (see figure 1.13).

The difference with traditional modular timber systems is that stiff modules provide the lateral congruity for lateral load transfer. The rigid modules are strategically placed in the main structure to resist the overall lateral loads and transfer them down to the foundation. The central core becomes less critical in this case and can be redundant. The hybrid modular structural system can broaden the design freedom, but to what extent is not known.

Figure 1.13: Module configurations with different wall and column placement, symmetry line in the middle. [own]

1.3.2. Coreless stability system

The stability system can be compared with a coreless stability system (a shear wall system). Which brings additional benefits: the usable floor space with a coreless system could increase following Arcadis from 60% to 80% with a traditional centered core to 80% or 90%. A sketch of a coreless stability system with hybrid modular construction can be seen in figure 1.14. But to what extent a coreless system for modular timber construction can be applied is not known.

(a) Modular construction with core as lateral resisting system [own]

(b) Hybrid modular construction system [own]

Figure 1.14: Resisting lateral load systems

2
Research content

2.1. Research problem

As discussed in the introduction it is clear that building extra houses in the Netherlands is unavoidable in the coming years, see 1.1.1. However, the way how these houses should be build is not clear yet and often a point of debate in politics and in the media. Building with prefabricated modular construction has gained attention in the past years since modular construction can decrease the building time; preparing the building site and manufacturing of the modular units take place at the same time but at a different place. This also leads to a possible earlier return on investment and lower site management costs. On top of that it is possible to dismantle the building while maintaining the asset value of the modular units. If the modular construction is built with timber there are also environmentally related advantages. Timber locks carbon, so a modular timber construction lowers the carbon emissions compared to modular constructions with steel or concrete. However, building with prefabricated modular (timber) units often result in simple-stacked building typologies; which in turn result in efficient but inaccessible high-density cities. Humanizing these high-density cities is a big challenge according to architects and engineering companies. In addition to this, the conventional design process between architect and engineer is time-consuming and does not allow further exploration of the design without creating additional work. Therefore, simple-stacked building typologies are often still the result. As discussed in the last part of the introduction a coreless stability system could increase the usable floor space and therefore the flexibility of the construction, which in turn might increase the creativity for the building typologies. In fact, a hybrid modular timber construction, where load-bearing and corner-supported modules are combined, could broaden the design freedom. But to what extent a coreless stability system for modular timber construction can be applied is unknown. Furthermore, it is also unknown to what extent a hybrid modular timber construction can broaden the design freedom.

A parametric design tool could help (see section 1.1.4) to create new building typologies. Multiple designs of modular systems can be assessed on specific parameters, to inform stakeholders in the early stage of a design and prove the project's feasibility. [46] However, an informed decision tool that gives feedback to the structural engineer or architect to explore the design opportunities for a hybrid modular timber system (with a coreless stability system) is not available yet.

2.2. Main question

The main question for this thesis is:

To what extent is a hybrid modular timber construction structurally feasible with a coreless lateral stability system while exploiting the options of a parametric design tool?

2.3. Sub questions

To support the main question of section 2.2, three sub questions have been formulated:

1. How is a hybrid modular timber construction designed within a parametric design tool?
2. Are architects and engineers able to create hybrid modular timber constructions with a parametric design tool?
3. Do architects and engineers create different building typologies for a hybrid modular timber construction when using a parametric design tool?

2.4. Objective

This master thesis' main objective is to develop a parametric design tool which improves the design process and explores the boundaries of hybrid modular timber constructions that satisfy the global and local structural building requirements. These building requirements will be explained in more detail in chapter 6.

2.5. Methodology

To answer the main- and sub questions a parametric design tool is developed. The development of the parametric design tool consists of the following three phases, as can be seen in figure.

Phase A: Engineering - literature study

Before the parametric design tool can be created it is necessary to study the engineering of a hybrid modular timber construction concept in detail and to obtain an overview of the relevant global and local structural requirements. A literature study on prefabricated timber buildings can be found in the introduction. A further literature study on timber is performed in this phase, considering glued-laminated timber, cross-laminated timber (CLT) and structural design with cross-laminated timber (CLT). A further literature study on hybrid modular timber constructions is also performed in this phase, considering fire safety, acoustics, CLT slab, CLT diaphragm floors, cross-laminated timber (CLT) shear walls and the connections. This phase ends with a preliminary design of the hybrid modular timber construction concept.

Phase A is discussed in chapter 1 (Introduction), chapter 3 (Timber) and chapter 4 (Design).

Phase B: Digitalisation - literature study and programming

Another literature study is performed on parametric modular construction to obtain a better insight into how to develop a script where the user gets instant structural feedback and obtains design freedom. Thereafter, a comparison study is performed on existing software programs relevant for creating the parametric design tool. Two 3D CAD software programs, two parametric plugins and two structural analysis software programs are compared. The hybrid modular timber construction concept is parameterized and combined with a parametric model in this step. This phase ends with a digital version of the hybrid modular timber construction in the parametric design tool, called Habitat21.

Phase B is discussed in chapter 5 (Parametric modular construction) en chapter 6 (Tool development).

Phase C: Observation - workshop and analyzing results
To answer the sub-questions in section 2.3 a verification study, workshop, and feedback optimisation study are performed. Within the verification study the performance of the modular timber construction concept is tested. The workshop verifies whether it is possible to design modular buildings with a parametric design tool for architects and engineers. And in the optimisation study it is investigated if it is possible to provide the user with smart feedback.

Phase C is discussed in chapter 7 (Results), chapter 8 (Discussion) and chapter 9 (Conclusion).

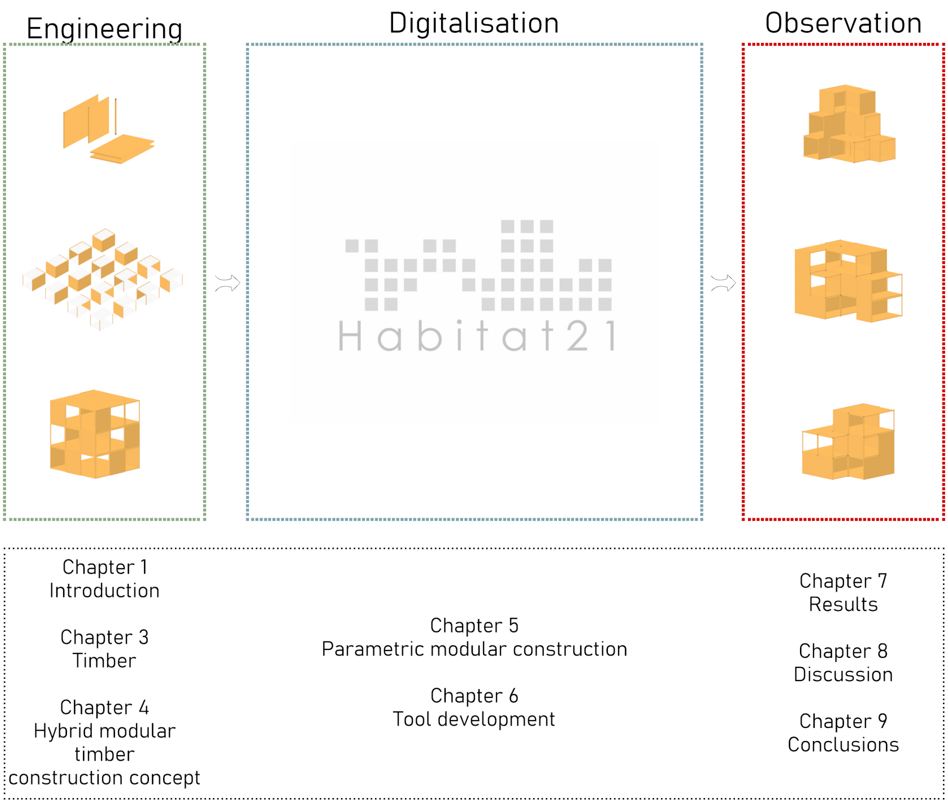


Figure 2.1: Methodology and outline of this thesis. In section **??** is an detailed description of the chapters given. Legend: ■ - Phase A, ■ - Phase B, ■ - Phase C, ■ - Outline

2.6. Scope
To obtains a clear structure in this thesis and keep the focus on the main objective, an overview of the considered and non-considered parameters is given below.

- Height
  To deal with dense urban areas, the focus will be on multi-storey residential buildings from 2 to 10 storeys.
- Foundation
  The design of the foundation is out of the scope of this research. Simplified rules will be applied if parameters of the foundation are necessary. Tension forces are likely to occur due to the lightweight structure and can be expressed with a parameter.

- Fire
  Fire resistance is an important design parameter for high rise residential buildings. Options to protect the structural components against fire will be provided, but will not be implemented in the tool.

- Services
  This research will only focus on the structural parameters concerning the creation of the architect. Parameters for the smart placement of elevators, electrical units, air ducts and plumbing are not within the research scope.

- Sustainability
  In the introduction a positive relation between modular timber construction and sustainability is presented. This was an incentive to choose a modular timber construction system in this thesis. Exact numbers regarding the waste, greenhouse gasses emissions, and labour productivity per building will not be investigated. An additional study should be done on the precise outcome of the sustainability impact on surroundings.

- Aesthetics
  The aesthetic part is not within the scope of this thesis. This thesis will provide information about the hybrid modular timber construction's technical possibilities to the architect and engineer.

- Acoustics
  Solutions to solve acoustic problems will be provided during this thesis, but the acoustics' exact performance is not in the scope of this thesis.

- Building envelope
  Timber can decay in a non controlled environment. The presented system in this thesis needs a closed building envelope to exclude the decay of timber elements.

- Seismicity
  Not taken into account.

- Custom
  Custom objects may be possible in the future, especially with recent developments in the digital CNC timber industry, but will not be in the scope of this research.

3
Timber

Several inherent characteristics make timber an ideal construction material: a high strength to weight ratio, durability, the good insulating properties against heat and sound. Due to available log sizes solid-sawn timber has limitations in maximum cross-sectional dimensions and lengths. The use of engineered timber products can overcome these, due to finger joints or lamination techniques, timbers of uniform and high quality can be in any shape constructed. The research on engineered timber is important in this thesis, because design with engineered timber products like CLT is not included in the European standard for timber structures EN 1995-1-1. [41]

3.1. Introduction timber

Timber is an anisotropic material, which means that the physical properties depend upon grain direction. When timber is used for structural purposes, it is assumed to be orthotropic, meaning to have directional properties in three mutually perpendicular axes. The orthogonal axes are aligned with the grain direction (longitudinal), the radial direction (radial) and tangential direction (tangential) as shown in figure 3.1. The properties along the grain (longitudinal) are referred to as properties to the grain. The tangential and radial direction properties are hardly different from one another and an average value as perpendicular to the grain is taken. [63]

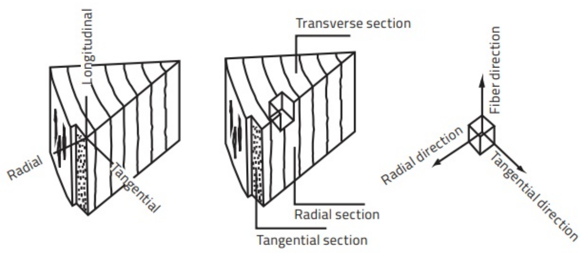


Figure 3.1: general types of connections in modular steel construction [83]

Organic material

Wood is an organic material, and it can degrade over time if it is not treated. If the wood is not treated and unprotected from the weather, it will become grey over time due to deterioration of the outermost lignin layer. However, this does not affect the strength of the timber. It is also essential to take the right precautions against fungi-attack. But this is not the scope of this thesis. [63]

Compression
Compression can act either perpendicular or along the grain. The cellulose fibres take the compression forces along the grain, and when the maximum capacity is reached, it will buckle. For timber perpendicular to the grain, the wood fibres will be crushed, which occurs at a much lower strength than parallel to the grain.

Tension
Although the tensile strength of clear wood is greater than the compression strength, tension failure occurs in a brittle mode, the tensile strength of structural timber is less than compression strength. Two possible failure modes for tension parallel to the grain are possible. The lignin fails, or the fibres are pulled apart. The lignin will fail or lower stresses in the perpendicular direction. Both failures are brittle. A tension member should be checked at the weakest point, and this will generally be at connections.

Shear
The shear strength of timber is normally 10-15% of the tensile strength in the direction of the grain.

3.1.1. Engineered timber

Engineered timbers are tested to predetermined specifications to meet national standards. Multiple engineered timber pieces are developed to overcome the defects in timber and are produced in various forms. The type of layering material or grain orientation differs, to meet application-specific performance requirements.

Material\fiber orientation	**Parallel**	**Perpendicular**
Boards	Glulam	CLT
Thin veneer	LVL	Plywood

Table 3.1: Engineered timber pieces by laminating and grain direction

Table 3.1 provides an overview of a few engineered timber products used in structural timber buildings. The fibers orientation and thickness layer play an important role in the engineered product's properties. Glulam components and CLT panels are made of thick lamella's. Laminated veneer lumber is a timber composite manufactured by laminating wood veneers using exterior-type adhesives. Veneers of 3-4 mm thick are peeled off logs and vertically laminated in a common grain direction, giving orthotropic properties. Plywood is made of cross-laminated thin veneer layers. The veneer layers or lamellas for the engineered timber products are in Europe most of the time made of softwood.

To reduce this thesis's work two engineered timber products are highlighted for the use of columns and panels: glue laminated (glulam) timber and cross-laminated timber (CLT).

3.1.2. Ultimate limit state verification

Timber elements should fulfill the resistance of ultimate limit state verification's. The occurring stresses in ultimate-limit-state (ULS) should be smaller then the design stresses of the elements and connections. In general the design value can be obtained by multiplying the characteristic strength with the modification factor and dividing it by the partial safety factor (see equation 3.1). The modification factor takes the effect of load duration an moisture content into account. While the partial safety factor takes material related uncertainties into account.

$$X_d = k_{mod} * \frac{X_k}{\gamma_m} \tag{3.1}$$

Service class	Permanent	Long	Medium	Short	Instant
1	0.6	0.7	0.8	0.9	1.1

Table 3.3: Modification factor for CLT for service class 1 and 2 [82]

- Where x_k is the characteristic value of a strength property
- Where γ_m is the partial safety factor for a material property
- Where k_{mod} is the modification factor, which takes the load duration and moisture into account

Material	γ_m
Solid timber	1,3
Glued laminated timber	1,25
LVL,plywood, OSB	1,2
CLT	1,25

Table 3.2: Partial safety factors for material properties according to [41] and [82]

Bending and tensile design strength for CLT
In CLT panels different strength of timber boards are found, and the risk of coinciding the weakest cross-sections in the same direction and layer is small. This is why the strength of CLT can be increased with a system factor (k_{sys}).The system factor is used for tensile and bending forces in CLT, where multiple boards can interact. The tensile and bending design strength can be calculated according to equation 3.2. The factor can be determined following equation 3.3. A maximum value of 1.15 is advised by Bergström and Fröbel and will be used in this thesis. [21]

$$X_d = k_{sys} * k_{mod} * \frac{X_k}{\gamma_m} \tag{3.2}$$

$$k_{sys} = \min(1.15; 1 + 0.1 * b) \tag{3.3}$$

- Where b is the effective width of the cross-section in meters

3.2. Glued-laminated timber

Glued-laminated-timber (glulam) is fabricated from two or several small sections of timber boards bonded together with adhesives. The grains of laminates are laid up parallel to the longitudinal axis. Laminates are typically 19-50 mm thick, 1.5-5 m in length, and optionally jointed at the ends by finger joints. Because of the laminations, the impact of imperfections affecting the structural performance is low. The lamination layers make it possible to match the lamination level of design stresses. E.g. beams can be manufactured with higher grade laminations at the outer regions.

3.2.1. Glulam properties

Strength values for glued laminated timber according to the NEN-EN 1408 code can be found in table 3.4.

Property	Symbol	GL 20h	GL 22h	GL 24h	GL 26h	GL 28h	GL 30h	GL 32h
Bending strength	$f_{m,g,k}$	20	22	24	26	28	30	32
Tensile strength	$f_{t,0,g,k}$	16	17,6	19,2	20,8	22,3	24	25,6
	$f_{t,90,g,k}$	0,5						
Compression strength	$f_{c,0,g,k}$	20	22	24	26	28	30	32
	$f_{c,90,g,k}$	2,5						
Shear strength	$f_{v,g,k}$	3,5						
	$f_{r,g,k}$	1,2						
Modulus of elasticity	$E_{0,g,mean}$	8400	10500	11500	12100	12600	13600	14200
	$E_{0,g,05}$	7000	8800	9600	10100	10500	11300	11800
	$E_{90,g,mean}$	300						
	$E_{90,g,05}$	250						
Shear modu-lus	$G_{g,mean}$	650						
	$G_{g,05}$	540						
Rolling shear modulus	$G_{r,g,mean}$	65						
	$G_{r,g,05}$	54						
Density	$\rho_{g,k}$	340	370	385	405	425	430	440
	$\rho_{g,mean}$	470	410	420	445	460	480	490

Table 3.4: Characteristic strength and stiffness properties an N/mm² and densities in kg/m³ for homogeneous glulam

3.2.2. Structural design with Glulam

The grain direction is important for the verification of glulam elements. The glulam timber components grain direction is visualized in figure 3.2 with the number 1 and is parallel to the timber element. Also, the y and z-axis can be determined for figure 3.2.

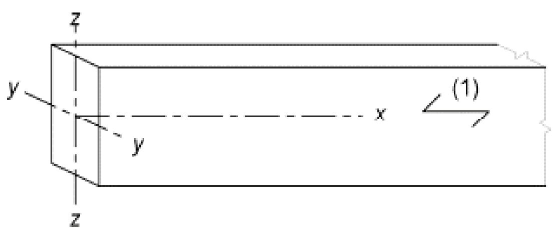


Figure 3.2: Direction glulam timber elemenet for verifications

Tension parallel to the grain
Tension parallel to the grain should fulfill equation 6.1 of the Eurocode:

$$\sigma_{t,0,d} \leq f_{t,0,d} \tag{3.4}$$

Compression parallel to the grain
Compression parallel to the grain should fulfill equation 6.2 of the Eurocode:

$$\sigma_{c,0,d} \leq f_{c,0,d} \tag{3.5}$$

Compression perpendicular to the grain
Compression perpendicular to the grain should fullfill equation 6.3 of the Eurocode:

$$\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d} \tag{3.6}$$

- Where $\sigma_{c,90}$ is the stress based on A_{ref}
- Where A_{ref} is the area where compression zone is increased with 300 mm in the direction along the grain.
- Where $k_{c,90}$ is the for the load conseideration, taken as 1.

Bending
Bending in timber shall fulfill the following conditions (equation 6.11 and 6.12 of the Eurocode):

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{3.7}$$

$$k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{3.8}$$

Shear
Shear stresses should fulfill, where the grains as well as parallel and perpendicular to the grain direction, the following requirements (equation 6.13):

$$\tau_d \leq f_{v,d} \tag{3.9}$$

- Where τ_d is the shear stress based on the effective width
- Where $b_{ef} = k_{cr} * b$ is the effective width
- Where k_{cr} is the factor for the consideration of cracks, for laminate wood products can this be taken as 0,67
- Where b is the width of the element

Bending and tension
The interaction between bending and tension shall fulfill equations 6.17 and 6.18 of the Eurocode:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1 \tag{3.10}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{3.11}$$

bending and compression
The interaction between compression and bending of a timber component shall fulfill conditions 6.19 and 6.20 of the Eurocode:

$$(\frac{\sigma_{c,0,d}}{f_{c,0,d}})^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1 \tag{3.12}$$

$$(\frac{\sigma_{c,0,d}}{f_{c,0,d}})^2 + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{3.13}$$

- Where k_m is the coefficient for the redistribution of bending stresses in a rectangular cross section. Can be taken as $k_{m,rectangular} = 0.7$, otherwise 1.

Buckling
Where there is a risk of buckling, the interaction of compression and bending should fulfill:

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m * \frac{\sigma_{m,z,d}}{f_{m,z,d}} < 1 \tag{3.14}$$

$$\frac{\sigma_{c,0,d}}{k_{c,y} f_{c,0,d}} + k_m * \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{3.15}$$

- Where $k_{c,y}$ is a reduction factor for slender members and can be determined with help of section 6.3 in the Eurocode.

$$k_{c,y} = \frac{1}{k + \sqrt{k^2 - \lambda_{rel}^2}} \tag{3.16}$$

$$k = 0.5 * [1 + \beta_c * (\lambda_{rel} - \lambda_{rel,0} + \lambda_{rel}^2] \tag{3.17}$$

$$\lambda_{rel} = \frac{\lambda}{\pi} * \sqrt{\frac{f_{c,o,k}}{E_{0,mean}}} \tag{3.18}$$

3.3. Cross-laminated timber

Minimal three orthogonally bonder layers form a solid timber panel, known as Cross-laminated timber (CLT) or sometimes X-Lam. The timber panels often have three, five, seven or more layers in odd numbers symmetrically formed around the middle layer. Spruce wood of strength class C24 is predominantly used for the intermediate layers. The layers are stacked perpendicular to one another and are glued together by mechanically pressing or a vacuum bag. Dimensions of the timber boards are ranging from 6 to 45 mm in thickness and 40 to 300 mm in width. [82] Figure 3.3 shows the different orientations of the timber boards in CLT.

3.3.1. Benefits

The CLT panels provide relatively high strength and stiffness properties in longitudinal and transverse directions, enabling a two-way spanning capability. CLT's other structural benefits include enhanced connector strength, splitting resistance, increased dead weight, robustness, high axial load capacity for walls, high thermal and acoustic performance, and a low carbon footprint. [31]

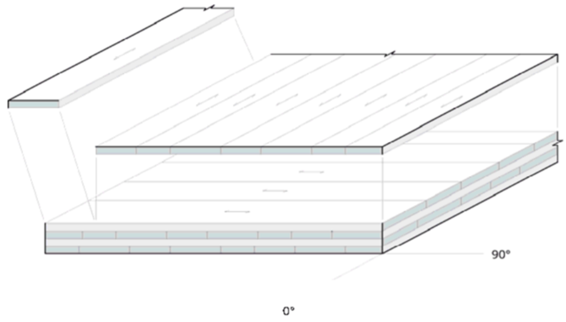


Figure 3.3: Orientation of timber boards in cross-laminated timber (CLT) [82]

3.3.2. Size limitations and suppliers

Various panel sizes are possible and produced by several European manufacturers. Often transport or the crane lifting capacity is the limiting factor for their size. Panel sizes up to 20 m long, 4.8 meters wide and 0.5 meters thick are possible, depending on the manufacturer. (see table 3.5)

Supplier	Width (mm)	Length (mm)	Thickness (mm)	Species used	Country	Product
Metsawood (outdated)	4800	14800	51-300	Spruce	Germany	Leno
Stora Enso*	3450	16000	42-350	Spruce, Larch, Pine	Austria	CLT
Kaufmann*	3000	16500	78-278	Spruce	Austria	BSP Crossplan
Binderholz	3500	20000	60-240	Spruce, Larch, Pine	Austria	BBS XL
KLH	3500	16500	60-500	Spruce, Pine, Fir	Austria	KLH

Table 3.5: Suppliers CLT in Europe and their sizes folowing [63] (sizes are updated by provided information on websites [2020])

3.3.3. Lateral design

CLT walls can be used as an effective lateral load resisting system. [80] Chapter 4 handles this aspect.

3.3.4. Properties

CLT design procedure is not included in the European standard, but several manuals provide design guidelines. The material factor γ_m=1.25 is proposed for CLT as material factor and k_{mod} is equal to the value for solid wood. Table 3.6 provides an overview of the proposed characteristic properties for homogeneous CLT sections made of standard European softwood.

Property	Symbol	CL 24 h	CL 28 h
Bending strength	$f_{m,CLT,k}$	24	28
Tensile strength	$f_{t,0,CLT,net,k}$	16	18
Tensile strength	$f_{t,90,CLT,k}$	0.5	
Compressive strenght	$f_{c,0,CLT,net,k}$	24	28
Compressive strenght	$f_{c,90,CLT,k}$	3.0	
Net shear strength	$f_{v,net,k,ref}$	5.5	
Gross shear strength	$f_{v,gross,k}$	3.5	
Torsional strength	$f_{v,tor,k}$	2.5	
Shear strength parallel	$f_{v,CLT,k}$	3.5	
Rolling shear strength	$f_{r,CLT,k}$	0.8-1.4	
Elastic modulus	$E_{0,CLT,mean}$	11600	
Elastic modulus	$E_{90,CLT,mean}$	300	
Elastic modulus	$E_{c,90,CLT,mean}$	450	
Shear modulus	$G_{CLT,mean}$	450/650	
Rolling shear modulus	$G_{r,CLT,mean}$	65-100	
Density	$\rho_{CLT,mean}$	420	

Table 3.6: Recommended strength [N/mm2], modulus [N/mm2] and density [kg/m3] for CLT base material [41]

3.4. Structural design with cross-laminated timber

This part handles the design of cross-laminated timber. Not everything for designing with CLT is included in the European standard for timber structures EN 1995-1-1 and NEN-EN 16531. Different papers and manufacturing guides are used to analyse the CLT panels.

3.4.1. Load bearing directions

The interlocked build-up of cross-laminated lamellas results in improved swelling and shrinkage behaviour. Horizontally, the panels are predominantly stressed in one-direction (uniaxially), but in some cases in two directions (biaxially) with point supported slabs (see figure 3.4). The main direction of the load-bearing capacity has the higher stiffness and is denoted with 0. The ancillary direction has a lower stiffness and is denoted with 90.

Uniaxial loading

When there is a dominant direction of loading with CLT panels, these can be analysed using a strip approach. Depending on the type of verification, CLT panels can be analysed on the net- or effective-section. Generally the net-section is used, but for the SLS verification for out of plane loading is the effective-section is used.

Biaxial loading

The modular units' CLT components may have a dominant loading direction, but when a floor or ceiling element is point-supported, no clear prevailing direction of loading governs. The CLT

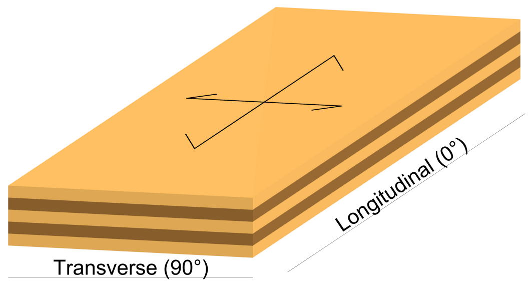


Figure 3.4: Defined directions of CLT [own]

panel should be analysed like a plate instead of the strip method. The method to verify CLT panels without a dominant direction of loading is described in the CLT-handbook by Wallner-Novak et al. and cross-laminated structural design of pro-Holz by Bergström and Fröbel.[82] [21]

3.4.2. Cross-sectional values

To perform calculations for CLT panels the following numbering and dimensions are used, which are visualised in figure 3.5 for a 5 lamina element. The total thickness of a 5 lamella thick CLT panel is given by equation 3.19:

$$h_{CLT} = t_1 + t_2 + t_3 + t_4 + t_5 \tag{3.19}$$

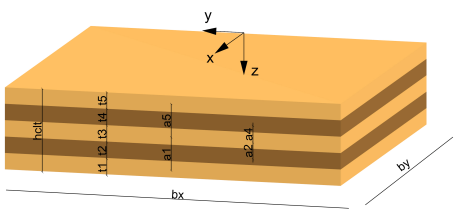


Figure 3.5

Net cross-section

$$A_{0,net} = \sum_{i=0}^{n} b * t_i \tag{3.20}$$

$$W_{0,net} = \frac{I_{0,net}}{max(|z_0|; |z_u|)} \tag{3.21}$$

$$I_{0,net} = \sum_{i=1}^{n} \frac{E_i}{E_c} * \frac{bt_i^3}{12} + \sum_{i=1}^{n} \frac{E_i}{E_c} * b * t_i * a_i \tag{3.22}$$

$$I_{ef} = \sum I_i + \gamma A_i a_i^2 \tag{3.23}$$

Effective cross section
The effective cross-section is used for CLT panels loaded out of the plane. The CLT panel's complex load-carrying behaviour can be reduced to a beam with homogeneous single layers. The behaviour of the normal and shear stresses are shown in figure 3.6.

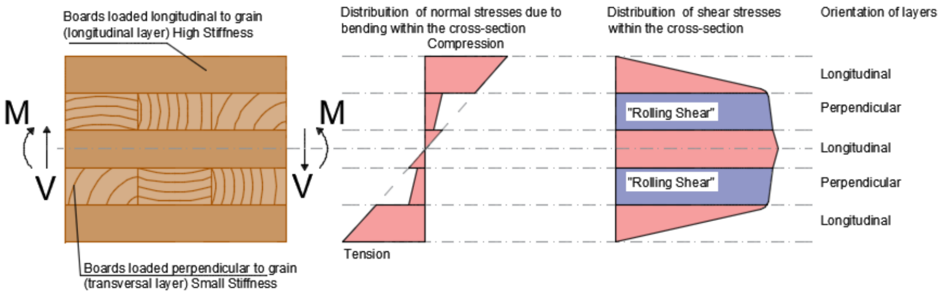


Figure 3.6: Stress distribution of a 5 layered CLT panel [61]

Shear
The shear stresses in the transverse layers (see blue areas in figure 3.6) will result in rolling shear. The rolling shear leads to additional deformation when CLT-panel are loaded out of the plane. Figure 3.7 shows a sketch of the rolling shear between layers. Rolling shear fractures can occur when the wood fibres roll between each other under shear stress across the fibres. Geometry and manufacturing are of importance, and boards with no edge glue, tongue, groove, and a width to thickness ratio of less than four are considered to have a more significant risk of shear failures. [21] Several studies have been done to determine the rolling shear strength and stiffness perpendicular to the grain and pointed it as a critical parameter for the carrying capacity and serviceability of CLT. [41]

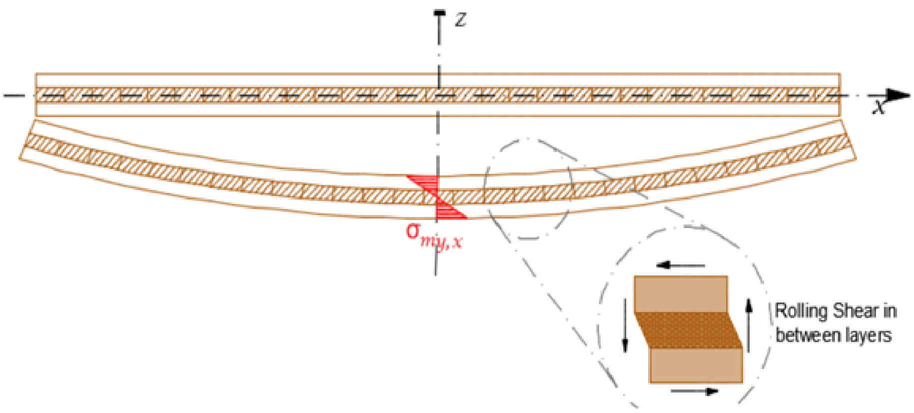


Figure 3.7: Rolling shear effect in a bi-supported CLT panel [61]

Based on experiments in [41], the following conclusions are made on which parameters influence shear strength and stiffness:

- Strength and stiffness depend on the type of wood. The values are much lower for softer types of wood.
- The lamella width to thickness ratio influences the strength and stiffness values. A reduction of the ratio causes a decrease in carrying capacity due to increased tensile and shear stress perpendicular to the grain.
- The greater the distance of the lamella from the heartwood, the lower the shear stiffness of the timber. No evident change in strength is observed.

Determination of effective section
Multiple methods exist to determine the effective section. The most commonly used method for theoretically resistance of CLT panels is the mechanically jointed beam method, also called the gamma-method. The shear analogy method and Timoshenko beam theory are other methods, but wont be handled in this thesis. The gamma method handles the deformations from shear forces in a simplified way. The gamma method is described in annex B of the Eurocode 5. In equation 3.23 the effective section is calculated through the gamma method. This method can be implemented with the Euler-Bernoulli beam element as no shear deformation is considered, but the a correction factor γ reduces the stiffness. This method is only applicable to 3 and 5 layer CLT panels, but can be extended to 7 and 9 layered elements with the extended gamma method.

3.4.3. In-plane loading walls (ULS)

Axial force

$$\sigma_{t,d} = \frac{n_{t,d}}{A_{net}} \leq k_{sys} f_{t,d} \tag{3.24}$$

$$\sigma_{c,d} = \frac{n_{t,d}}{A_{net}} \leq f_{c,d} \tag{3.25}$$

Buckling
For checking CLT walls and panels for buckling two different loads can occur, axial and transverse loads. If the loads are combined equation 3.26 and 3.27 should be met. The method is explained in appendix B.1 to determine the buckling factor $k_{c,y}$.

$$\frac{\sigma_{c,x,d}}{k_{c,y} * f_{c,0,xlay,d}} + \frac{\sigma_{m,y,d}}{f_{m,xlay,d}} \leq 1 \tag{3.26}$$

$$\frac{N_d}{k_{c,y} * A_{x,net} * f_{c,0,xlay,d}} + \frac{M_{y,d}}{W_{x,net} * f_{m,xlay,d}} \leq 1 \tag{3.27}$$

3.4.4. Floors (SLS)

Floor slabs should be checked on deflection, sagging and vibrations in serviceability limit state.

Deflection
The maximum deflection is determined due to short term load and long term loading. The CLT Handbook [21] can be consulted for more information.

Vibrations
Dynamic effects occur when people walk on a floor and affect the floor's serviceability state. In CLT elements with a large span than 4 meters, vibrations may often be a critical design parameter. [41] Multiple parameters affect the CLT floor behaviour: mass, span, stiffness, load distribution and composition of supports. See appendix B.3 for the calculation method. The dynamic vibrations can be split into two categories:

- Vibrations felt by the person causing them, i.e. floor sagging describes the experience of a self-generated vibration or deflection in the floor caused by a single movement.
- Activities of other cause vibrations; i.e. oscillation describes how a person perceives floor vibrations caused by others.

3.5. Cross-laminated timber and Karamba

The material properties used as input for the finite element analysis in Karamba are calculated in this section. Karamba can only account for isotropic and orthotropic properties in one direction. Due to these software restrictions, fictitious values for the elastic and shear modulus need to be calculated for the entire element. This part will handle how will be dealt with the orthotropic CLT elements with different cross-sectional values in x and y-direction. In the software the CLT element will be described as a massive beam with orthotropic material properties.

3.5.1. Input parameters Karamba

The input parameters for Karamba are the elastic modulus in longitudinal and transverse directions (E1 and E2), the shear modulus in longitudinal or transverse direction (G31 and G32), and the in-plane shear modulus (G12). The directions are visualised in figure 1. To calculate the properties regular beam are theories used with orthotropic plate properties. The properties are calculated for 3, 4, and 7 layered CLT panels with equal lamella thickness and wood composition.

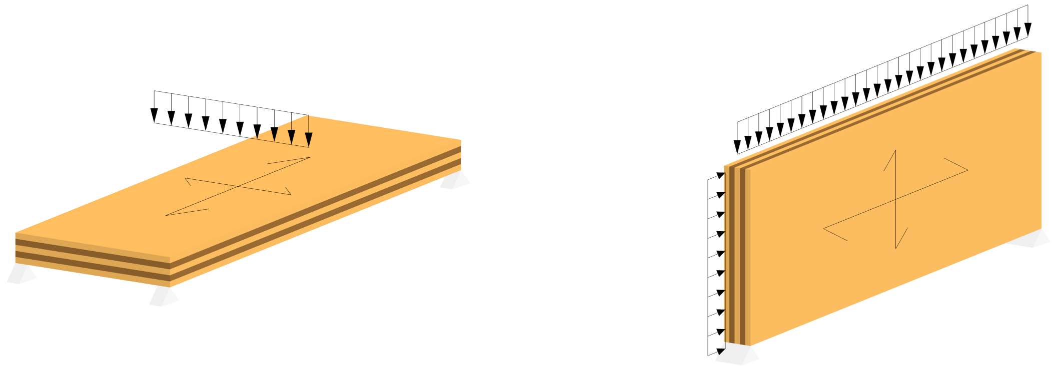
3.5.2. Assumptions

For the calculation of the material properties a number of assumptions are taken:

- The stresses in the transverse layers are not assigned and the modulus of elasticity is assumed 0 ($E_{90} = 0$).
- A symmetrical cross-section of the CLT panels is assumed regarding the lamella thickness and composition of wood.
- Values for the properties of the timber-boards are given in table 3.6 of section 3.3.4, which are valid for homogeneous CLT sections made of standard European softwood.

3.5.3. Direction CLT walls, floor and ceiling

The fictitious elastic modulus derived for the axial and bending stiffness can be used as input for E_1 and E_2 and depend on the load case for the model in question. The CLT shear walls are loaded in-plane and are dominated by in-plane tension and compression forces. Equations 6.1 and 6.2 will be used in this case for E_1 and E_2. The CLT floor and ceiling diaphragm plates are predominantly loaded out of plane, and equation 6.3 and 6.4 are better in this case. It would be possible to put the two axial and bending stiffness's directly in a more advanced finite element program, which is the preferred procedure. In [21] and [82] is explained how to do this. By using the out-of plane properties for the floor and ceiling elements the in-plane stiffness is overestimated in the strong direction (1), and underestimated in the weak direction (2).



(a) Floor (b) Wall

Figure 3.8: Load directions for out of plane and in plane loading CLT wall and floor

3.5.4. Axial stiffness

The axial stiffness is calculated for two principal directions, the longitudinal and transverse. Two limiting cases can be observed for the evaluation of the CLT panel:

1. No contact between the boards within the lamina
2. Complete contact between the boards within the lamina

No contact between the board results in an axial stiffness for the perpendicular lamina that is equal to zero. The axial stiffness for full contact between the boards can be calculated for uniform compression and tension. The contact will not be perfect, the glue will be pushed between the boards, and distribution is never exact, and also cracks will develop over time. No contact is assumed in this case.

$$EA_l = \sum E_{l,i} t_i \tag{3.28}$$

$$EA_t = \sum E_{t,i} t_i \tag{3.29}$$

For the use of Karamba the axial stiffness is translated in a fictious elastic modulus:

$$E_1 = E_{c/t,l} = \frac{EA_l}{h_{clt}} \tag{3.30}$$

$$E_2 = E_{c/t,t} = \frac{EA_t}{h_{clt}} \tag{3.31}$$

3.5.5. Bending stiffness

The effective area should be used for out-of-plane loads. For this thesis not the effective area is used, but the net area. For the use of karamba the fictious elastic modulus is derived for pure bending as follows:

$$E_1 = E_{m,l} = \frac{12EI_l}{t^3} \tag{3.32}$$

$$E_2 = E_{m,t} = \frac{12EI_t}{t^3} \tag{3.33}$$

3.5.6. Shear modulus perpendicular to the plane

The shear modulus perpendicular to the plane gave two different values for the transverse and longitudinal direction. The shear moduli of individual layers are shear along the grain or the rolling shear of each lamina.The shear can be determined with help of the shear correction coefficient (k_z) or factor (k) (se equation 6.7):

$$GA_s = \frac{\sum GA}{k_z} = k * \sum GA \tag{3.34}$$

For a rectangular cross the shear correction factor is equal to equation 6.8, the shear correction factor for CLT should be smaller than 0,83.

$$k_{rectangular} = \frac{5}{6} = 0.83 \tag{3.35}$$

Methods to calculate the shear correction factor are provided in the basic design principles of ProHolz [82] and master project of Richard Eriksson and Maria Karlsson [28]. For this thesis the shear correction factors of multiple methods are compared for different CLT configurations. Table 6.3 provides an overview. For the use of Karamba the fictitious shear modulus perpendicular to the plane is defined as follows:

$$G_{31} = G_{l,fict} = \frac{\kappa_l * G_{r,l}}{h_{clt}} \tag{3.36}$$

$$G_{32} = G_{t,fict} = \frac{\kappa_t * G_{r,l}}{h_{clt}} \tag{3.37}$$

Where $G_{r,l}$ and $G_{r,t}$ are defined for a 3 layer element as:

$$G_{r,l} = [G_0, G_{90}, G_0] \tag{3.38}$$

$$G_{r,t} = [G_{90}, G_0, G_{90}] \tag{3.39}$$

Type G90/G0 ratio	**Value (Brettersperholz)** unknown	**Value (Jobstl)** 1/10	**Value kx** 1/10	**Value ky** 1/10
1 lamella	0.83	0.83	-	-
3 lamellas	0.15 ≤k_{CLT} ≤0.18	0.21	0.2060	0.6944
5 lamellas	0.18 ≤k_{CLT} ≤0.20	0.24	0.2434	0.1881
7 lamellas	0.25 ≤k_{CLT} ≤0.29	0.26	0.2582	0.2291
9 lamellas	0.26 ≤k_{CLT} ≤0.29	0.27	-	-

Table 3.7: Results of the shear correction factor for different CLT configurations from multiple references.

3.5.7. Shear modulus parallel to the plane

The shear modulus parallel to the plane is a complicated topic. The stiffness depends on whether the interfaces are glued and the build up of the CLT panel. [23] A simplified approximation of the in-plane shear stiffness can be calculated with equations‘6.13 and 6.14 determined in [23].

$$G^* = \frac{1}{1 + 6 * \alpha_{FE-FIT,ortho} * (\frac{t}{a})^2} * G_{0,mean} \tag{3.40}$$

$$\alpha_{FE-FIT,ortho} = 0.32 * (\frac{t}{a})^{-0.77} \tag{3.41}$$

- Where G^* is the reduced shear stiffness
- Where $G_{0,mean}$ is the shear stiffness of the material
- Where t is the thickness of the board material
- Where a is the width of the board material, a value of 150 mm is assumed

3.5.8. Results CLT Properties Karamba

This section provides the results for the fictious properties used in Karamba. The properties of the CLT base material is provided in table 6.4. A ratio of $\frac{1}{10}$ is taken for the shear strength perpendicular to the grain.

Material	E0 [N/mm2]	G0 [N/mm2]	G90 [N/mm2]	Density [kg/m3]
CL 24 h	11600	650	65	420

Table 3.8: Base material for calculations CLT

For the configurations of the CLT panels a board thickness of 150 mm is assumed with an average thickness of 40 mm. These values are of importance, because they influence the in plane shear strength. Table 6.5 gives an overview of the CLT configurations with 3,5 and 7 lamellae.

Name	Layer build up [mm]	Board thickness [mm]	Number of layers x direction.	Number of layers y direction	Thickness [mm]
3 CLT H	40-40-40	150	2	1	120
5 CLT H	40-40-40-40-40	150	3	2	200
7 CLT H	40-40-40-40-40-40-40	150	4	3	280
5 CLT V	20-20-20-20-20	150	3	2	100
5 CLT V	30-30-30-30-30	150	3	2	150
5 CLT V	40-40-40-40-40	150	3	2	200
5 CLT V	50-50-50-50-50	150	3	2	250
7 CLT V	43-43-43-43-43-43-43	150	4	3	300
7 CLT V	50-50-50-50-50-50-50	150	4	3	350

Table 3.9: Different CLT configurations for the modular system split in horizontal and vertical directions

Table 6.6 provides an overview of the properties for 3,5 and 7 lammella's as input for Karamba with base material CL 24 h. Table 6.5 provides an overview of the build up of the CLT panels.

Property	3 CLT H	5 CLT H	7 CLT H	5 CLT V	5 CLT V	5 CLT V	5 CLT V	7 CLT V	7 CLT V
h	120	200	280	100	150	200	250	300	350
t	40	40	40	20	30	40	50	43	50
E1 (bending)	11170	9187	8251	9187	9187	9187	9187	8252	8252
E2 (bending)	430	2412	3348	2413	2413	2413	2413	3348	3348
E1 (axial)	7733	6960	6629	6960	6960	6960	6960	6629	6629
E2 (axial)	3866	4640	4971	4640	4640	4640	4640	4971	4971
G12	471	471	471	560	514	471	434	460	434
G31	94	101	103	101	101	101	101	103	103
G32	181	56	72	56	56	56	56	72	72
ft1	10.7	9.6	9.1	9,6	9,6	9,6	9,6	9,1	9,1
ft2	5.3	6.4	6.9	6,4	6,4	6,4	6,4	6,9	6,9
fc1	16	14.4	13.7	14,4	14,4	14,4	14,4	13,7	13,7
fc2	8	9.6	10.3	9,6	9,6	9,6	9,6	10,3	10,3
t12	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5

Table 3.10: Fictious CLT properties used for the cross-laminated timber products in the modular system in N/mm2 or mm

3.5.9. Verification CLT properties

A CLT panel is modeled in Karamba to verify the fictitious CLT properties in table 6.6. The total displacement of a CLT panel in Karamba is compared with the analytical displacement as described in section 4.5.1. The horizontal displacement at the upper right corner is measured, since the upper left corner does not provide accurate results due to peak stresses. In appendix B.4 the geometry of the CLT panel with mesh properties is visualised with stress distribution. The result for a vertical 3 by 5 meters CLT panel can be seen in figure 6.6. The fictitious CLT properties for Karamba give accurate results by having a maximum difference of 2%.

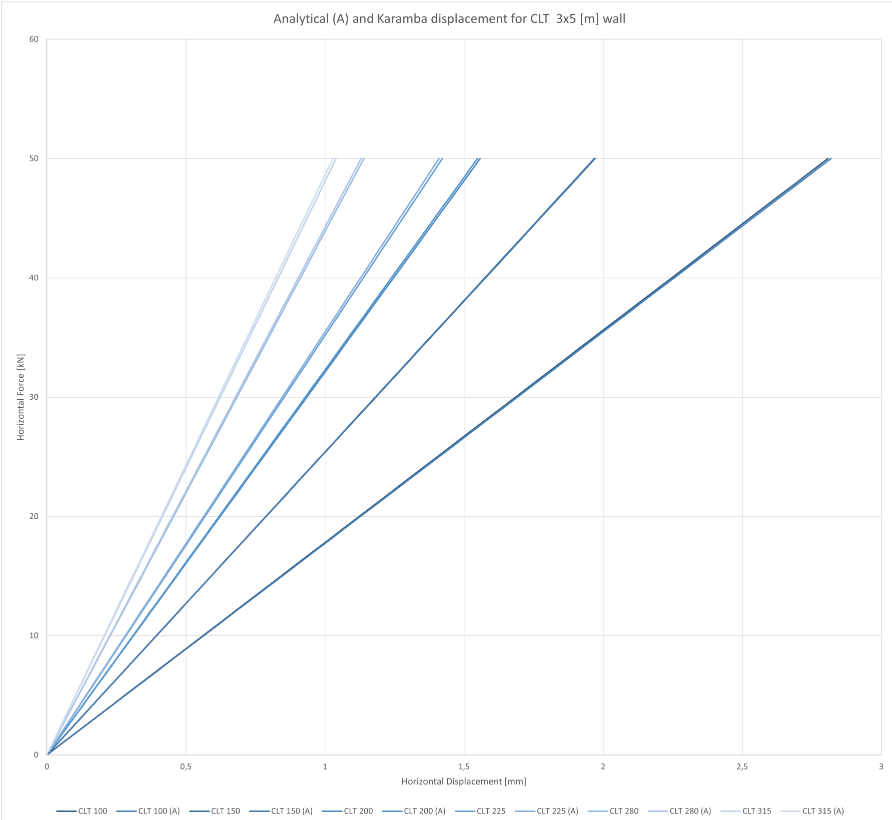


Figure 3.9: Displacments 3x5 meter CLT wall compared, (A) indicates the analytical method.

3.6. Conclusions

The timber components should fulfill the ultimate limit state verification. The glulam columns, connections and CLT-panels can be verified with equation 3.1. But the bending and tensile strength of CLT panels can be increased with a factor k_{sys}, and should fulfil equation 3.2.

4 Hybrid modular timber construction

In reference projects (see section 1.2.3) a concrete core or load-bearing CLT modules provides lateral resistance against wind loads. As discussed before, concrete is not an option in this project since it harms the environment by emitting greenhouse gasses. Cross-laminated timber is an efficient wood product for multi-story timber buildings due to the high bearing capacity and stiffness. It can be used for the assembly of walls and floors due to the load-bearing capacity in in-plane as well as out of plane. Despite these advantages there is little or no guidance on how to design a lateral load-carrying system with timber elements in the Eurocode. [52] [41] And the load-bearing modules made of CLT limit the applicability and design space for modular structures.

A concept of the hybrid modular construction system as suggested in section 1.3 will be explained in this chapter. The design presented here seeks to balance open modular units by integrating rigid modules with CLT shear walls and floor diaphragms to provide lateral resistance (coreless lateral system, see figure 4.1). For the design of a lateral system is explored what the possibilities are with CLT shear walls, floor diaphragms and connections. The connections play an important role in determining the diaphragm behavior, attention should be paid to the wall to floor and panel to panel connections. [24] Before going into a more detailed description of the connections, the CLT slab, diaphragm and shear wall will be addressed. In addition to the structural concept solutions for fire-safety and acoustics are provided.

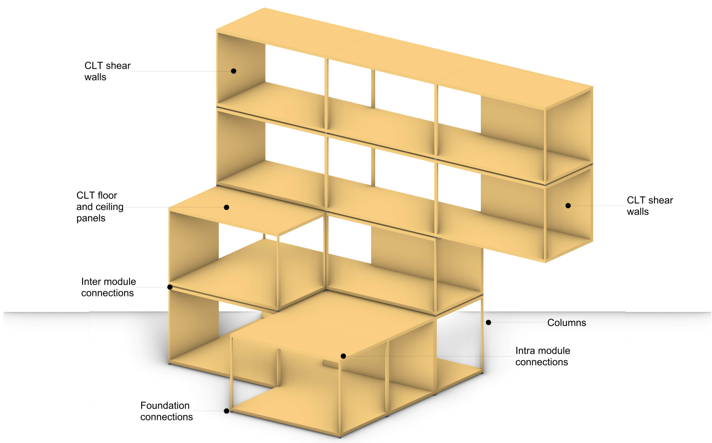


Figure 4.1: Sketch overview of hybrid modular construction system [own]

4.1. Fire safety

Some design options will be provided in this section, that can be implemented in the future.

The event of a fire can be divided into four different stages: pre-ignition, Growth, Burning, decay. The growth phase is tuned into a fully developed fire (burning) through a flashover. The response of a structure during the flashover phase determines the fire resistance. The resistance of a structure to fire is expressed in minutes.

The required resistance of the main load bearing structure depends on the function, height and fire load. The ultimate limit state of the structure must satisfy during this period. Table 4.1 gives an overview of the required resistance for residential buildings. [1]

Height [m]	**Required resistance [min]**
h <7	60
7 <h <13	90
h >13	120

Table 4.1: Required fire resistance main load bearing structure for height of ”verblijfsruimte” for Dutch residential buildings following Dutch code. [1]

Despite timber is a combustible material, it is possible to design timber building that behaves well in fires due to the protective char layer. The resistance of a timber component can be calculated with the charring rate. This charring rate depends on the density of the wood. The fire resistance of timber components can be increased by larger cross-sections or an extra protective layer. This elongates the time that a timber component can fulfill its structural function.

$$\beta = \frac{d_{char}}{t} \tag{4.1}$$

- β is the charring rate in [mm/min]
- d_{char} is the charring depth in [mm]
- t is the fire time in [min]

A full assessment of the behavior of the structure in fire is beyond the scope of this thesis.

4.2. Acoustics

Despite the good acoustic performance of timber there is still sound transmission possible. Following Gertzer 90% of the sound transmission takes place through walls, this can be reduced by either improving the mass or by mechanical insulation layers.

Figure 4.2: Mechanical interruption layer of Rothoblaas to improve the acoustic properties of the building [Rothoblaas]

The soundproofing of a building can be prevented by a very stiff high performance resilient soundproofing profile xylofon of Rothoblaas (See figure 4.2). This material is a mechanical interruption layer between building materials and can block sound transmission pathways. The xylofon profile will be inserted between CLT walls, glulam columns and used in connections. Accurate calculations in line with EN 12354 are hard to perform, since there are no accurate values for the 'relatively' new product CLT. Simplistic approach and publication with solid wood construction can be found at CLT manufacturers. The exact acoustical behaviour of the modular construction system is not researched in this research, since it is out of scope.

4.3. CLT slab

Simply supported CLT slabs are still a topic of research. The design of the CLT floor and ceiling is done with help of experimental tests and information provided by Rothoblaas. The maximum span depends heavily on the used connection and amount of reinforcement. [22] The Pillar connection of Rothoblaas allows a column to column distance up to 3.5 x 7.0 meters with just two screws for reinforcement. This grid can be increased, by having extra reinforcement in the CLT panel to distribute the forces with spider arms of Rothoblaas, to column spacing greater than 7.0 x 7.0 meters. Based on this information it is possible to set the maximum modules dimension to 3.0 x 7.0 meters. A maximum length of 6 meters is chosen, due to grid-size dimensions. All module dimensions are visualised in figure 4.3.

The system allows the CLT floor panels to resists gravity loads using two way slab action, while the columns at each storey transfer axial loads directly to the columns below. The cumulative compression perpendicular to the grain is avoided in this way. Steel threaded rods can be used to connect the CLT slab to the columns.

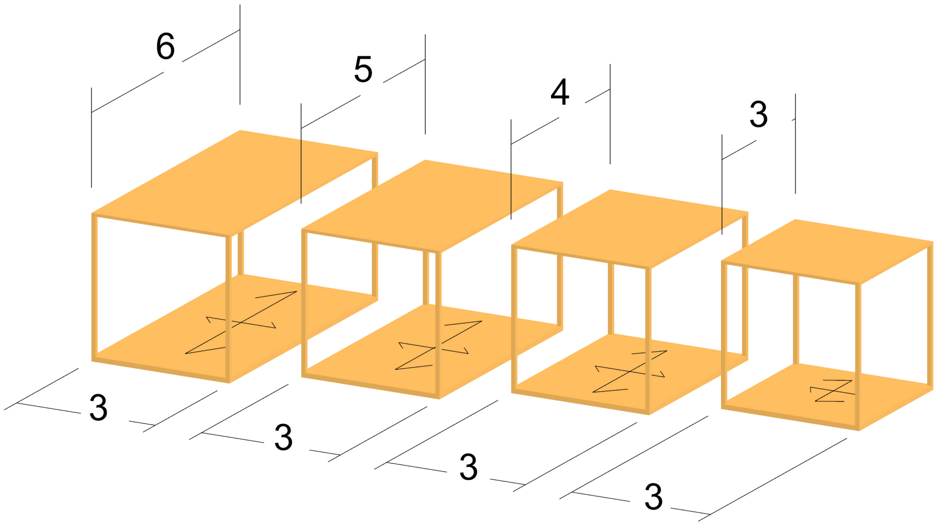
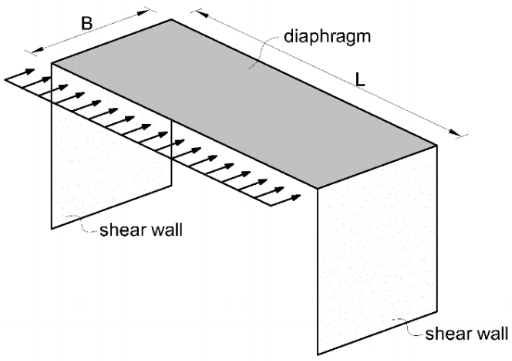


Figure 4.3: Dimensions modules dependent on the maximum simple supported slab size and grid size [own]

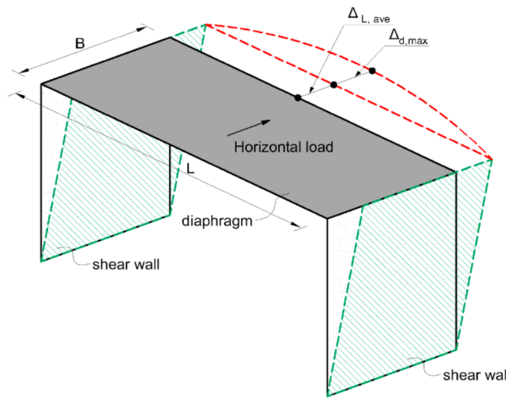
The load-bearing capacity of the point supports depend on the diameter of the cylinder and dimension of the base plate. In appendix C.3 the load-bearing capacities are given for a Pillar support without reinforcement. The orientation of the pillar connection is at the corner of the module. The values for the center can assumed for this thesis, since the CLT panel is supported by the whole surface of the columns. Section 4.6.3 provides a detailed overview of the corner connection.

4.4. CLT diaphragm floor

Figure 4.4a provides a schematic overview of the horizontal wind load on the floor diaphragm, which is transmitted to the foundation by two shear walls. The distribution of horizontal wind loads from the floor to the lower walls depend on the strength and the stiffness of the floor system. A stiff floor allows the transfer of forces to the underlying shear walls with diaphragm behavior. CLT diaphragms are often considered as rigid concerning the stiffness of the shear walls. [52] But modularization can result in the formation of flexible diaphragms, due to the stiffness of the lateral inter-connectivity between modules. [69]



(a) Floor diaphragm with shear wall and wind load [24]



(b) Diaphragm definition based on shear wall deflection [24]

4.4.1. Rigid or flexible

Floor diaphragms are considered as fully flexible or rigid. This depends on the relation between the maximum in-plane deformation of the floor diaphragm ($\Delta_{d,max}$) and the average inter-storey drift ($\Delta_{L,ave}$) (see figure 4.4b). CLT diaphragms are often considered as rigid concerning to the shear walls. But CLT diaphragm are considered flexible if the deflection is 1.1 times the average inter-story drift. (see table 4.2) For the design stage, a stiff diaphragm is much easier to design with if the floor acts rigid or flexible can be checked in the codes.

Diaphragm type	**EN 1998:2010**
Flexible	$\Delta_{d,max} \geq 1.1\ \Delta_{L,ave}$
Rigid	$\Delta_{d,max} < 1.1\ \Delta_{L,ave}$

Table 4.2: Determination of stiffness diaphragm [24]

The panel stiffness (k_{panel}) and the connection stiffness (k_c) influences the total stiffness of the diaphragm and can be calculated as follows:

$$k_{panel} = \frac{F}{\Delta_{d,max}} \tag{4.2}$$

$$k_c = \frac{F}{2 * \Delta_{con}} \tag{4.3}$$

$$k_{diaphragm} = \frac{1}{\frac{1}{k_{panel}} + \frac{1}{k_c}} \tag{4.4}$$

4.4.2. Spring model for the in-plane behavior of a CLT floor

A mechanical spring model can be used to describe the behavior of the diaphragm the coupled CLT panels. The panels can be considered 2D in-plane stress homogeneous elements characterized by orthotropic behavior, defined by the MatProps component of Karamba. The module walls to foundation connections can be assumed as hinges since lateral supporting shear walls are generally not designed to withstand load out of their plane.

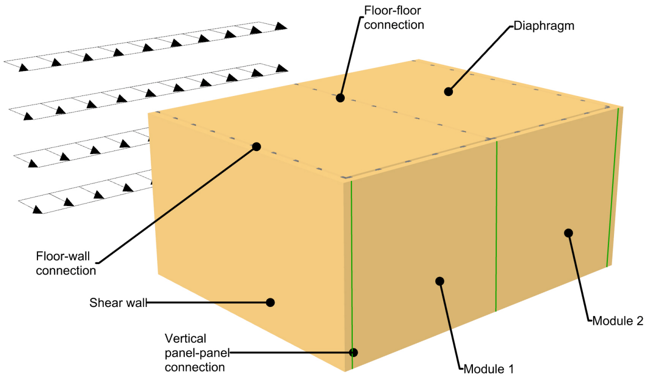


Figure 4.5: Sketch of two modules loaded by a horizontal load. [own]

The most accurate way to analyze diaphragms is with the use of a finite element analysis according to [24]. The diaphragm's global in-plane behavior in this thesis will be described with a spring model in Karamba. Figure 4.5 provides a visualisation of two connected modular units subjected to a lateral load and figure 4.6 gives an analytical representation of the diaphragm system. In figure 4.5 the external load is transferred through the CLT floor panels to the shear walls, with the floor panels acting as a deep beam, the deformation can be split into shear and bending.

$$U_{diaphragm} = u_{shear} + u_{bending} \tag{4.5}$$

The contribution to shear depends on the in-plane shear deformation of the CLT panel, the floor-to-floor connection along the edge of the panel, and the floor-to-wall connection along the wall.

$$u_{shear} = \frac{F}{k_{shear,panel}} + \frac{F}{k_{floor-floor,spring}} + \frac{F}{k_{floor-wall,spring}} \tag{4.6}$$

The contribution due to bending depends on the in-plane bending stiffness of the CLT panels, the floor-to-floor connection perpendicular to the edge of the panel, and the floor to wall connections perpendicular to the load.

$$u_{bending} = \frac{F}{\frac{1}{k_{bending,panel}+k_{floor-floor}}} \tag{4.7}$$

4.4.3. Behaviour connections

The floor to wall spring is represented by line springs that act only parallel to the floor's edges. Two types of connections between the floor panels can be observed: rotational springs and line springs parallel to the floor's edge. The rotational springs' behavior is bi-linear; when the CLT panels contact each other, a rigid contact element is adopted. Otherwise, a linear elastic spring is assumed. The stiffness for the rotational spring is equal to 10% of the floor length, due to experiments with rocking of a single CLT element this value is suggested. [72]. The parallel line spring acts linear elastic. The stiffness of the connections are provided in section 4.6.5.

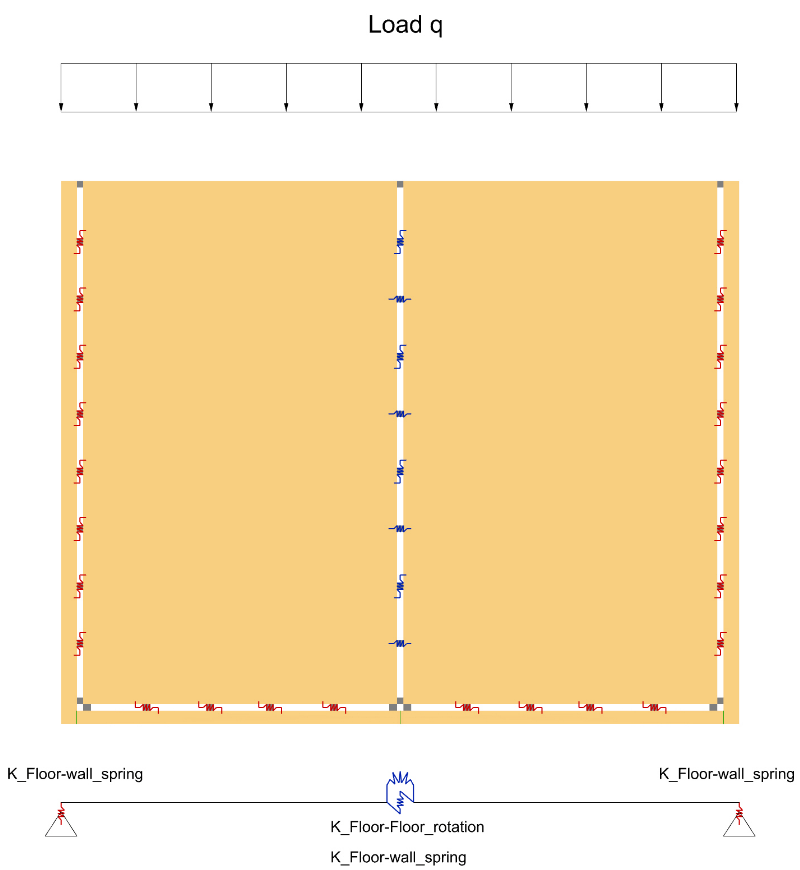


Figure 4.6: Graphical representation of spring model for diaphragms [own]

4.5. CLT shear wall

Before the assessment of a single CLT wall is introduced, it is essential to understand the shear wall system. Four different mechanisms contribute to the shear walls deflection (see figure 4.8):

- Translational deformation due to the connections (slip)
- Rotational deformation due to the overturning moment (rocking)
- Shear deformation of the wall
- Bending deformation of the wall

The interaction between these displacements can largely be ignored. Although there is some interaction, the wall can be idealized as a set of four springs in series. And the total displacement can be calculated as following:

$$u_{total} = u_{shear} + u_{bending} + u_{translation/sliding} + u_{rotation/rocking} \tag{4.8}$$

And the total stiffness of the wall can therefore be calculated as:

$$k_{total} = \frac{1}{\frac{1}{k_{bending}} + \frac{1}{k_{shear}} \frac{1}{k_{sliding}} + \frac{1}{k_{rocking}}} \tag{4.9}$$

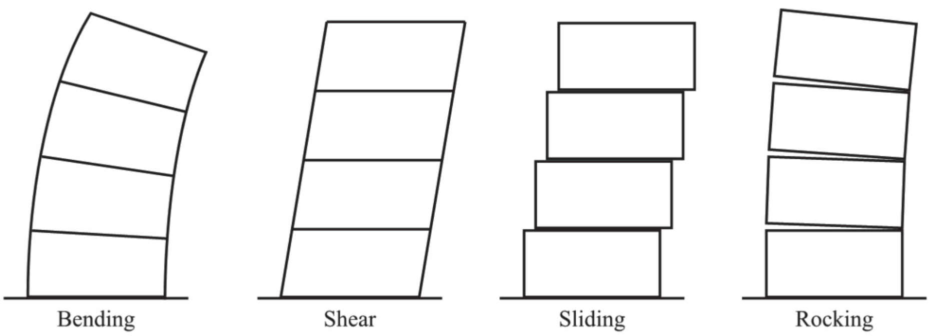


Figure 4.7: Mechanisms deflection CLT shear wall [24]

4.5.1. Single CLT wall

Design of a CLT shear wall is performed by assessing the load-carrying capacity and stiffness. The force-displacement relation determines the stiffness of a shear wall. The wall displacement can be calculated by knowing the maximum force on the shear wall and the connections' stiffness. A mathematical approach to determine the deformation due to bending and shear forces is explained in [21] by equation 4.10 and 4.11.

$$u_{shear} = \frac{F_d * h}{b * t_{tot} * G_{mean}} \tag{4.10}$$

$$u_{bending} = \frac{F_d * h^3}{3 * E_{mean} * I} \tag{4.11}$$

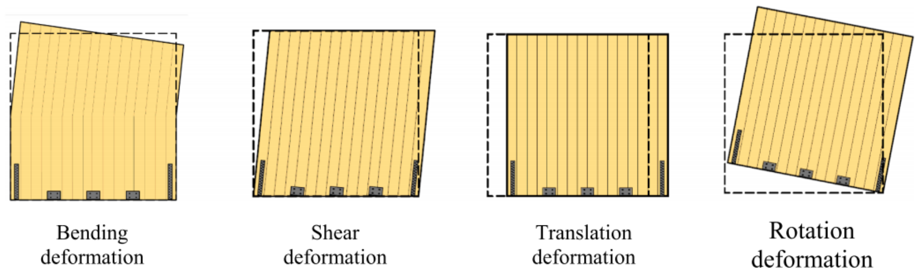
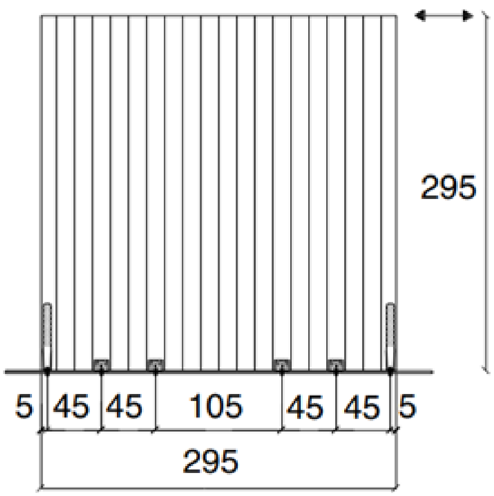


Figure 4.8: Illustration of different contributions to of the CLT shear wall deformation. [52]

Deformation single CLT wall
The contributions of sliding and rocking are often ignored, as they are governed by the steel connections stiffness. But they have a significant impact on the wall its stiffness. [51] [52] [24] Existing wall test observations indicated that, under lateral loading, the shear and bending deformation of CLT wall panel itself is insignificant compared to the deformation of panel connections. The lateral displacement of a CLT wall is mainly caused by rotating as a rigid body about the corners and sliding in a horizontal direction, governed by the steel connections. [32]

Experimental tests were performed by IVALSA Trees and the Timber Institute on single CLT wall with different configuration of connectors. On a typical single CLT wall with 4 angle brackets and 2 hold down connectors is the displacement distribution as following:



Deformation	Deformation percentage
Bending + shear	2.0 %
Rocking	42.5 %
Sliding	55.9 %

Figure 4.9: CLT panel test configuration with deformation [32]

Height to width ration
Figure 4.9 is an indication of the displacement behavior of a single CLT panel. Different parameters have an influence on the displacement distribution. A larger vertical applied load results in less rocking and more sliding deformation. And if the height over the length ration is lowered, sliding deformation will grow and rocking deformation will decrease.

Effect of number of brackets
Also, the effect of number of brackets is tested [32]. A single CLT wall has been tested with 2 and 4 brackets. While the elastic stiffness was equal for both walls, the maximum strength and

ultimate displacement were 57% lower than the CLT wall with four brackets. The translational deformation is dominant with two brackets, while the rotational and translational deformation for four brackets.

4.5.2. Horizontal stacked walls

The vertical joints between adjacent wall panels have a significant influence on the total lateral displacement behavior. The joint strength and stiffness determine the panel's kinematic behavior under later load, which can be divided in three different behaviors (see figure 4.10):

- Coupled wall behavior
- Single-coupled wall behavior
- Single wall behavior

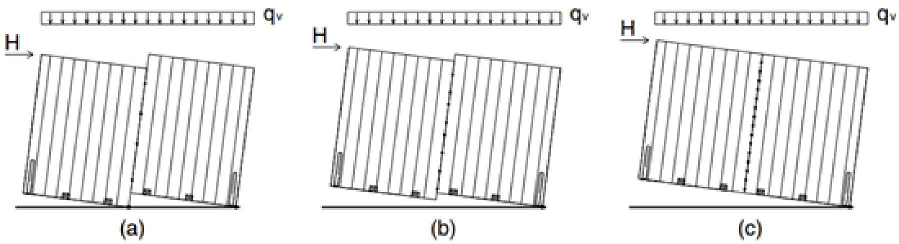


Figure 4.10: Types of the behavior of adjacent wall panels: (a) coupled wall behavior; (b) combined single-coupled wall behavior; (c) single wall behavior [52]

The behavior depends on the number of panels which compose the wall, the type of connection and number of fasteners. A few panels or a lower vertical applied load will result in a monolithic wall. Also, the ratio between the vertical connection and hold down connection's stiffness determine the wall behavior. A high shear stiffness to hold-down tensile strength ratio tends to be like single wall behavior. Due to monolithic or single wall behavior is possible to reduce the hold down brackets at the end of the walls, which result in a cost reduction. And is much easier to design with. [9]

4.5.3. Vertical stacked walls

The rigid rotation due to the shear load must be avoided at each floor. Connections at the corners can limit the rotation. The rigid slip due to the shear load, can be avoided through shear connections at the edges of the panel. Figure 4.11 provides and overview of all acting forces on a double stacked CLT wall.

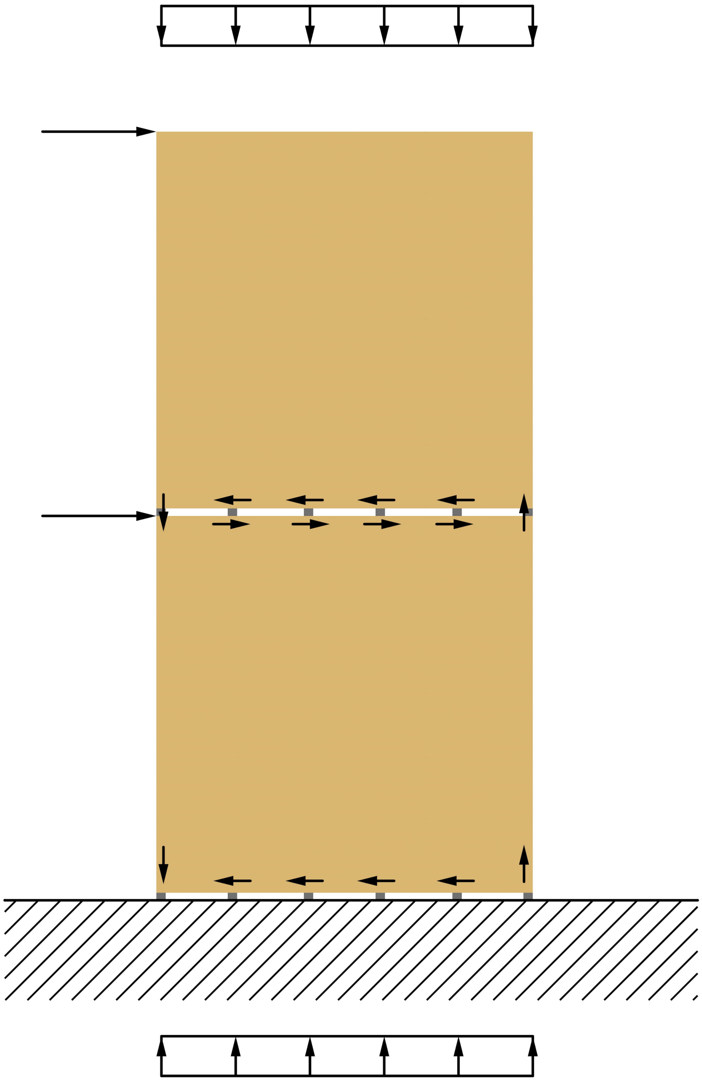


Figure 4.11: Force distribution vertical stacked wall [own]

4.5.4. Spring model for Vertical and horizontal wall interaction

The shear wall will be modeled in Karamba with help of springs. The spring at the edges will prevent rocking and parallel to the edges will prevent sliding. The vertical and horizontal shear connectors try to reach monolithic behavior of the CLT walls. See figure 4.12 and 4.13 for an overview.

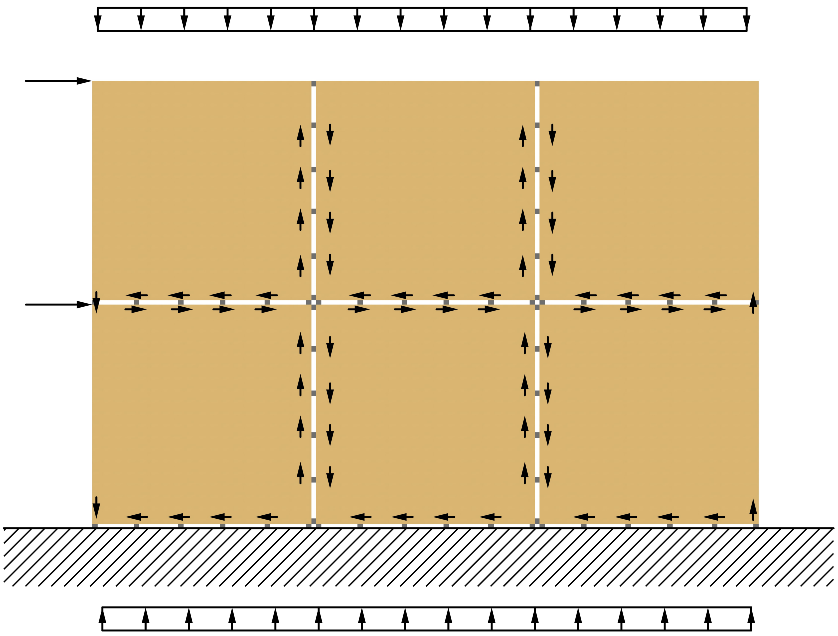


Figure 4.12: Force distribution CLT shear wall [own]

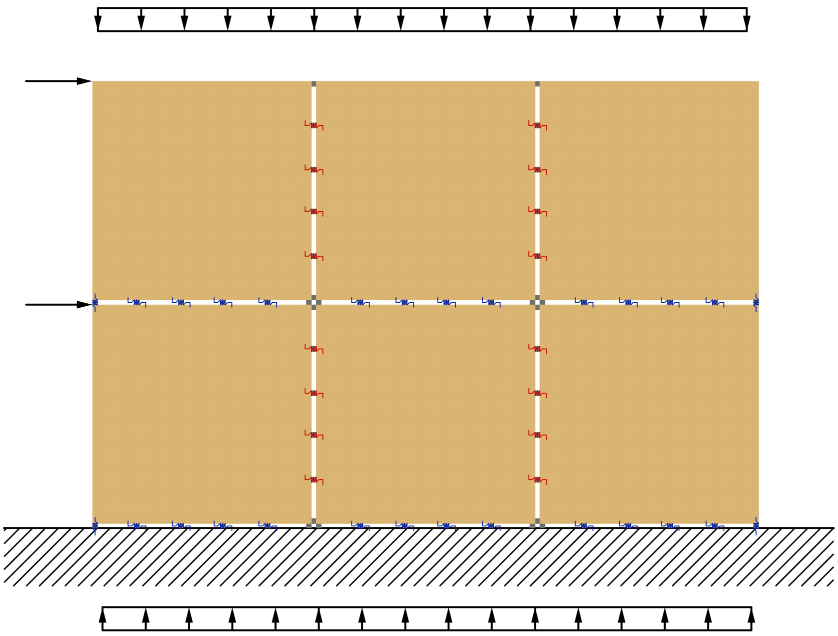


Figure 4.13: Spring distribution CLT shear wall [own]

4.6. Connections

In timber engineering, the joint will be the critical design factor of the structure. The strength of the connections will in most cases be the governing failure mode. Another important design aspect is the displacement behaviour of the connections. As can be read in section 1.2.1 contribute connections large to the total displacement behaviour. Multiple ways are available to connect timbers elements, with each their own displacement behaviour. However the displacement behaviour between the connection types differs. Figure 4.14 provides an overview for the slip of different timber-concrete connection types. From the figure can be concluded that glued connections are more stiff than all other connections. However glued connections wont be the scope of this thesis, since the connections need to be assembled easily. Only mechanical and carpentry joints are examined for an application in modular timber construction.

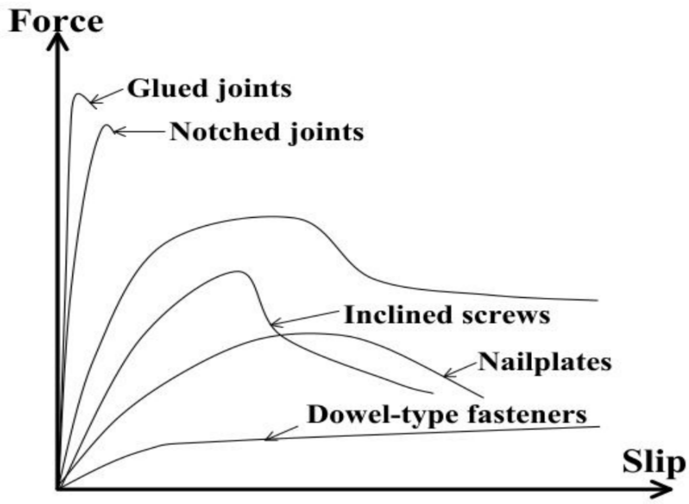


Figure 4.14: Slip of different timber-concrete connection types [27]

4.6.1. Fasteners

Fasteners connections systems focuses on the use of bolts, dowels, screws and nails to connect multiple elements together. The strength and stiffness of the fasteners are of importance since they can be the governing failure mode of the system. The HBS counter sunk screw of Rothoblaas will be used in this thesis (see figure 4.15).

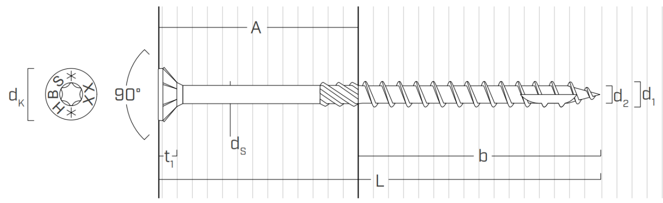


Figure 4.15: Dimensions of HBS counter sunk screw of Rothoblaas

The HBS counter sunk screw with a nominal diameter of 12 mm will be used for calculations. The properties can be found in appendix C.4. The provided strength values are tested, certified and calculated for CLT and glulam products.

Strength
A connection may be laterally, or axially loaded. For the determination of lateral strength of connections with fasteners provides section 8.2 in Eurocode 5 equations for calculating the characteristic load-carrying capacity of fasteners according to the Johansen yield theory. Three main parameters which influence the load-carrying capacity of joints with dowel-type fasteners are:

- The bending capacity of the dowel or yield moment
- The embedding strength of the timber or wood-based material
- The withdrawal strength of the dowel.

Axial Withdrawal capacity
The grain angle influences the withdrawal strength. Figure 4.16 gives an impression of the fastener and the grain direction for CLT and glulam. The characteristic axial withdrawal capacity of Rothoblaas fasteners in solid timber with a maximum density of 590 kg/m3, glulam, CLT, or LVL with can be determined according to EN 1995-1-1:2008:

$$F_{ax,\alpha,Rk} = \frac{n_{ref} * k_{ax} * f_{ax,k} * d * l_{ef}}{k_\beta} * (\frac{\rho_k}{\rho a})^{0.8} \tag{4.12}$$

- $F_{ax,\alpha,Rk}$ is the characteristic withdrawal capacity of the screw at angle α to the grain [kN]
- n_{ef} effective number of screws according to EN 1995-1-1:2008
- k_{ax} is equal to 1 when: $45° \leq \alpha \leq 90°$

(a) Thread withdrawal narrow face CLT panel

(b) Thread withdrawal glulam, parallel to the grain

Figure 4.16: Withdrawal example of screws for CLT and glulam

Stiffness
For a reliable design with fasteners not only the load carrying capacity is governing, but also the load-deformation behavior of the connections are of importance. The design equations for the slip and stiffness for fastener type connections can be determined in section 7,1 of the Eurocode 5. These stiffness equations are very basic and based on a linear calculations. Table 4.3 provides an overview for the calculation of the stiffness based on fastener type and predrilling.

Fastener	K_{ser}
Dowels / Bolts with or without clearance / Nails (with predrilling)	$K_{ser} = \frac{\rho^{1,5} * d}{23}$
Nails (without predrilling)	$K_{ser} = \frac{\rho^{1,5} * d^{0,8}}{30}$

Table 4.3: Equation for estimation of flip for fastener connections following Eurocode 5, with K_{ser} in N/mm

The stiffness of connections with multiple fasteners without predrilling can be calculated according to equation 4.13.

$$K_{v,ser} = \frac{n * \rho^{1,5} * d^{0,8}}{30} \left[\frac{N}{mm}\right] \tag{4.13}$$

- Where d is the diameter of the fastener thread in the secondary beam in mm
- Where ρ_m is the average density of the secondary beam in $kg/m3$
- Where n is the number of fasteners in the secondary beam

If the densities of the two jointed connections are different than ρ_m, the mean density can be calculated according to equation 1. And for timber to steel or concrete connections, K_{ser} should be calculated with the density of the timber element multiplied by 2.

$$\rho_m = \sqrt{\rho_{m1} * \rho_{m2}} \tag{4.14}$$

It should be mentioned that the equations in table 4.3 roughly estimates the real slip behaviour of the fastener connection. Figure 4.17 shows two diagrams of the real behavior of the load deformation of timber connections. A initial slip is followed by linear load-deformation and non-linear load-deformation. Linear load deformation will be assumed in the scope of this thesis, and an additional slip can be accounted for bolts with clearance. [42]

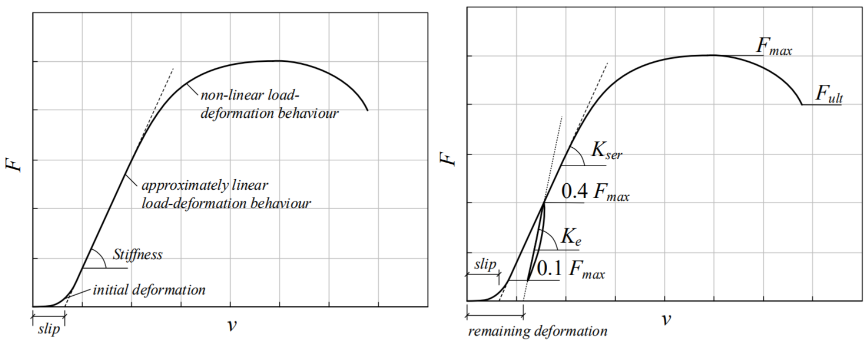


Figure 4.17: Slip of different timber-concrete connection types [42]

Stiffness HBS screws

The lateral slip modulus K_{ser} for HBS screws where the outer thread diameter is between 5 and 10 mm, can be be determined with the following equation [ETA-11/0030]:

$$k_{ser} = 60 * \left(\frac{d * \rho_{mean}}{510}\right)^{1.5} \tag{4.15}$$

- Where d is the outer thread diameter [mm]
- Where ρ_{mean} is the desnity of the softwood [kg/m3]

In equation 4.15 is the lateral slip modulus independent of the angle to the grain.

4.6.2. Connection matrices

The connections for the modular building system are separated into three different classes: intra-, inter-, foundation-connections. Two connection matrices (see table 4.4 and 4.5) are made to connect different components in the system. Based on orientation and location in the system are the connections generated. First is the type of the connection indicated, which can be an intra- inter-module, or foundation connection. Secondly, is the connection selected in the right connection matrix. The intra-module connection matrix contains three connection types, while the inter module connection system contains 5 different connections.

Intra	**Ceiling-wall**	**Wall-wall**	**Floor-wall**	**Column-floor**	**Column-Ceiling**
Type	1	2	1	3	3

Table 4.4: Connection matrix for the intra-module connections

Inter	**Wall Column**	**Wall**	**Column**	**Wall H**	**Floor H**	**Ceiling H**	**Wall Ceiling H**	**Column H**
Wall Column	x	x	1	x	x	x	x	x
Wall	x	2	x	x	x	x	x	x
Column	1	x	1	x	x	x	x	x
Wall H	x	x	x	x	3	x	x	4
Floor H	x	x	x	3	5	x	x	4
Ceiling H	x	x	x	x	x	5	3	4
Wall Ceiling H	x	x	x	x	x	3	x	4
Column H	x	x	x	4	4	4	4	4

Table 4.5: Connection matrix for the inter-module connections

The next sections will focus on the intra- and inter-module connections. The foundation connections will be assumed infinitely stiff hinges and wont be worked out in detail. The intra- and inter-module connections will be handled in the next sections. A physical and analytical representation of will be provided here for every connection.

4.6.3. Intra module connections

Three different intra connections are used in the system, which are indicated with numbers in figure 1. All the connections will be modeled as hinges in the analytical model with different stiffness's in x, y and z direction.

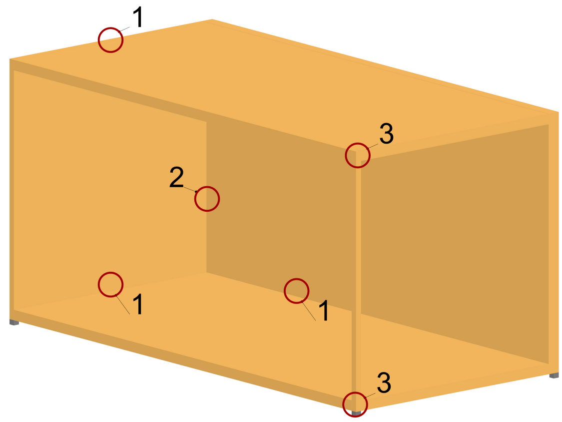


Figure 4.18: Schematic overview of the intra-module connections with numbering [own]

Panel-panel (1)
The T-Lock connector designed by Rothoblaas is easy and quick to install with fasteners and can be disassembled. The connection can be used for CLT panel floors and has high strength values (see table 4.6). The stiffness of the joint can be calculated with equation 4.13.

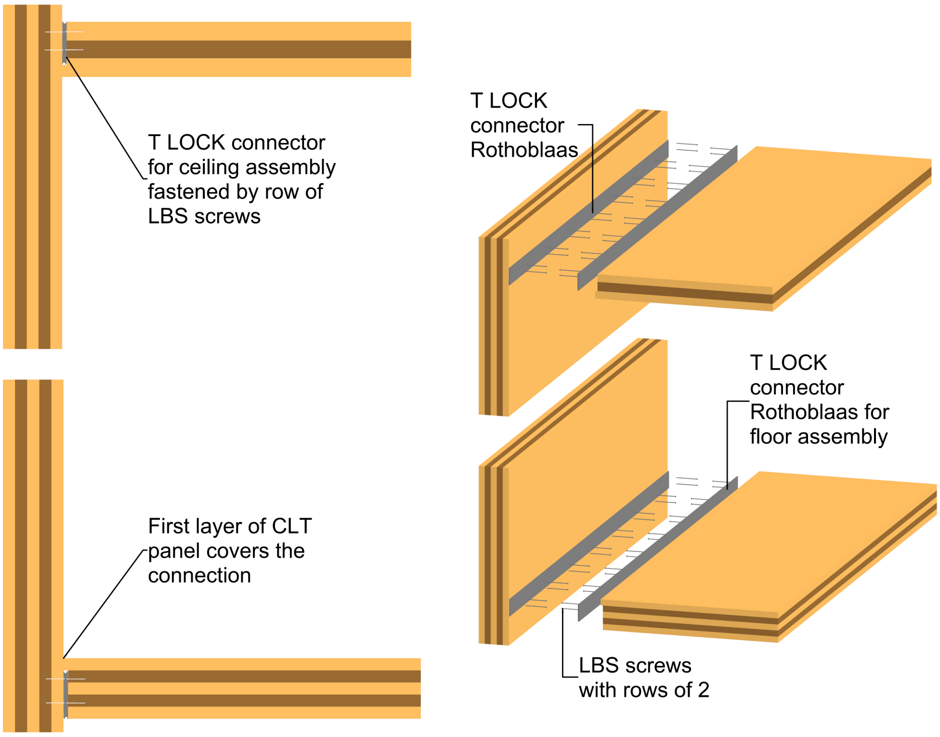


Figure 4.19: Assembly T-LOCK connector of Rothoblaas for CLT panels [own]

Type	**B x H x S [mm]**	**LBS screws [pcs]**	$R_{v,timber,k}$ **[kN]**	$R_{v,steel,k}$ **[kN]**	K_{ser} **[kN/mm]**
LockT135	300 x 135 x 22	8 + 8 - Ø 7 x 80	20.4	240	0.073
LockT135	600 x 135 x 22	16 + 16 - Ø 7 x 80	40.79	480	0.073
LockT135	900 x 135 x 22	24 + 24 - Ø 7 x 80	61.19	720	0.073
LockT135	1200 x 135 x 22	32 + 32 - Ø 7 x 80	81.59	960	0.073

Table 4.6: Optional dimensions with strength values

The assembling of the T-Lock connector contains three steps. First is the aluminium connector placed on the wall and fastened with screws. Secondly, the connector is placed on the horizontal CLT panel with screws. In the final third step is the horizontal CLT panel hooked from top to bottom. Figure C.5 provides an visual overview.

Panel-panel (2)
The connection between two perpendicular wall within the modules will be connected with HBS screws of Rothoblaas (See figure 4.20). Multiple reports conclude that an inclination of the fasteners increases the lateral stiffness significant. The fasteners will be placed in pairs with an 45°angle, but the stiffness will be calculated with equation 4.13, which is on the conservative side. The strength of the connection can be determined with help of section 8.2 in eurocode 5. Table 4.7 provides an overview of the results.

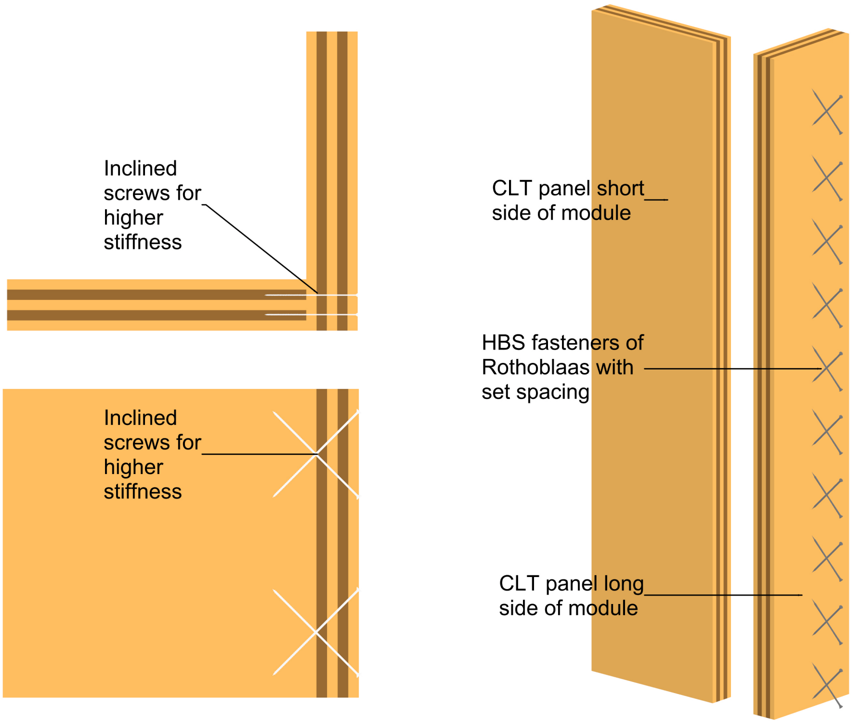


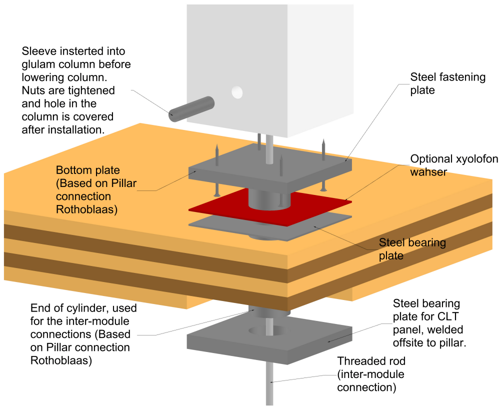
Figure 4.20: Wall- wall connection for perpendicular walls within the module [own]

Type	d	L	$R_{v,k}$ [kN]	$R_{ax,k}$ [kN]	$K_{ser,lat}$ [kN/mm]	$K_{ser,ax}$ [kN/mm]
HBS	12	600	4.65	16.85	0.6	36

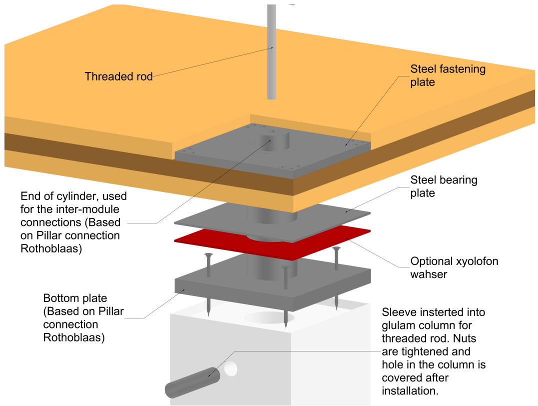
Table 4.7: Values for HBS fastener of Rothoblaas

Column-Panel (3)

The column panel connection of the modular system is based on the pillar connection of Rothoblaas. Figure 4.21 provides an overview. The point supported compression forces can be found in appendix C.1. The connections will be modeled as hinges. In figure 4.22 is also an overview provided of the corner connection when a CLT panel occurs instead of a column. This connection is also based on the pillar connection of Rothoblaas.

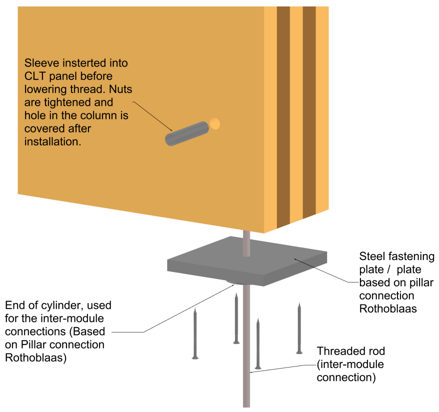


(a) Exploded CLT floor-column connection. The threaded rod is visible, which is part of the vertical inter-module connection. [own]

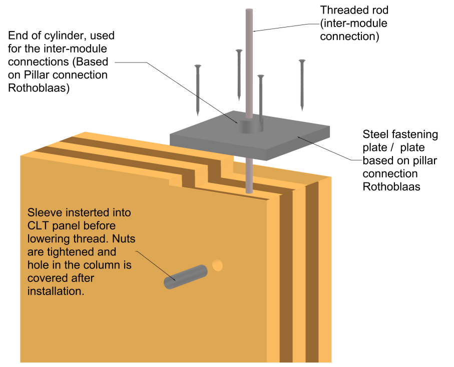


(b) Exploded CLT ceiling-column connection. The threaded rod is visible, which is part of the vertical inter-module connection. [own]

Figure 4.21: Corner connection for columns [own]



(a) Exploded CLT floor column connection. The threaded rod is visible, which is part of the vertical inter-module connection. [own]



(b) Exploded CLT ceiling column connection. The threaded rod is visible, which is part of the vertical inter-module connection. [own]

Figure 4.22: Corner connection for CLT panels [own]

4.6.4. Inter module connections

Five different inter module connections are used for the inter modeled connections to resist tension, compression and shear forces. Just like the intra-module connections are all connections modeled as hinges with stiffness's in three directions. Figure 4.23 provides a schematic overview of where the connections will occur.

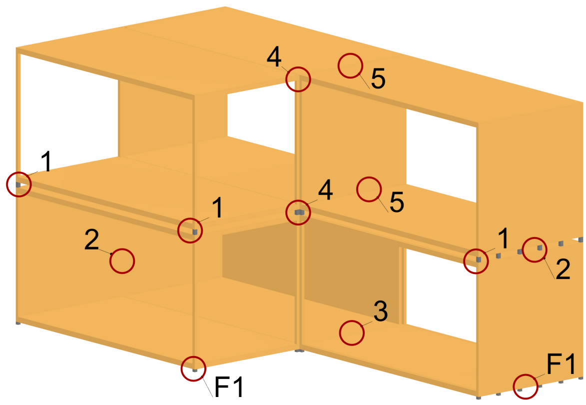


Figure 4.23: Schematic overview of the inter-module connections and foundation connections with numbering [own]

Column-Column (1)

Inter-module connection 1 is the vertical connection between modulus (see figure 4.24). It should resist high compression but also tension forces (see section ?). Since inter module connections for modular timber buildings doesn't exist is the connection based on two Pillar connections of Rhothoblaas, which can resist high compression forces, in combination with a novel inter-modular steel connection proposed by Srisangeerthanan et al. (see appendix **??**). The tension forces are guided through a threaded rod, which is inserted in the bottom column and fastened by a steel sleeve connection. This type of connection is used in glulam column-to-column connections in combination with glued in rods.

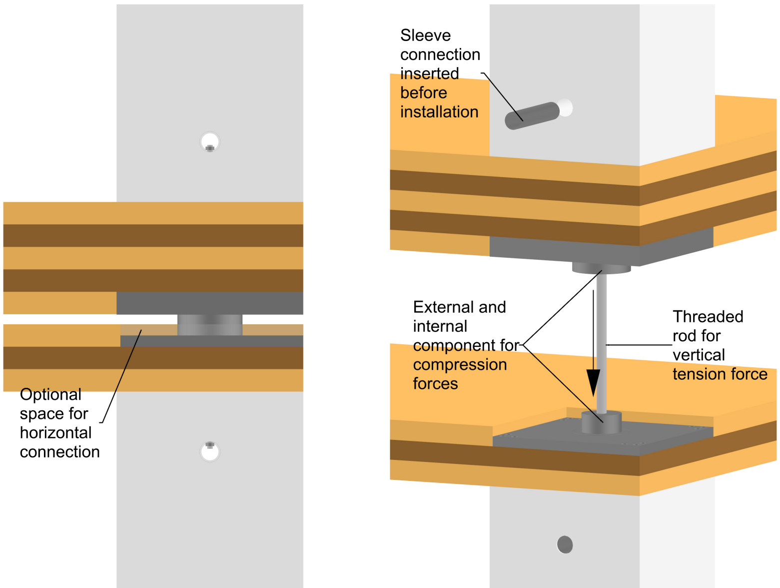


Figure 4.24: Side view and exploded view of the vertical inter-modular connection, based on the proposed inter-modular steel connection [70] and the glued in rods of [75] [own]

The joint is modeled as two hinges with no later stiffness ("pendelstaaf") and infinitely stiff in vertical direction. In figure 4.30 an analytical overview of the connection is illustrated, which also includes the horizontal plate element (Connection 4).

Panel-panel (2)
The traditional connection systems for CLT structures are characterized by many screws and nails, which increases the construction time and costs (see appendix C.2). The slot connector (figure 4.25) engineered by Rothoblaas uses just two optional screws to connect structural panels. The joint allows can transfer exceptional shear stresses between CLT panels to obtain single wall behavior with high lateral stiffness (see table 4.8). The joint is made of aluminium and is easy to handle, with optional inclined screws make tightening between panels easy. The slot can be used for wall assemblies and in floor diaphragms. The stiffness and shear strength can be calculated according to equation 4.16 and 4.17. Calculated properties for the slot connector can be obtained in table 4.9. [3]

Type	Number of connectors	Spacing [mm]	R_{vk} [kN]
Slot	4	967	81.1
Half lap	14	200	42.6
Spline joint	56	100	60.9

Table 4.8: Shear strength values calculated according to ETA-19/0167, ETA-19/0030 and EN 1995-1-1 for traditional joints and the slot connector [Rothoblaas]

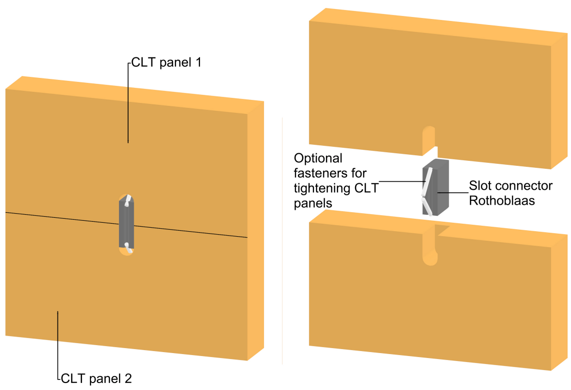


Figure 4.25: Slot connector Rothoblaas with optional inclined screws [own]

Type	Spacing $\sum_{d0}$ [mm]	R_{vk} [kN]	K_{ser} [kN/mm]
Slot	40	34,37	21
Slot	60	48,15	21
Slot	69	54,35	21

Table 4.9: Properties slot connector

$$K_{ser,softwood} = \frac{\rho_k}{20}[kN/mm] \tag{4.16}$$

$$F_{v,Rk} = k_{a1} * b_{ef} * f_{c,0,k} * \left(\sqrt{t_{gap}^2 + 2*(t_e-5)^2 + t_{gap}*(t_e-5)} - t_{gap} - (t_e-5)\right) \tag{4.17}$$

Panel-Panel (3)

Since connection in section 4.6.4 only resist shear forces and no tension forces, is an extra connection added to the system when tension forces occur in the diaphragms or wall assemblies. WHT plates designed by Rothoblaas (figure 4.26) resist tensile forces and can be used in combination with the Slot connector. This plate is generally used in platform-construction, but these can also be used in balloon frame construction. Another option is to use Titan plate T, which can resist shear and tensile forces. The downside of this connection is the amount of nails,the lower resistance against shear forces, and lower stiffness. To reduce the options is chosen is only the WHT plate connector of Rothoblaas added to the system.

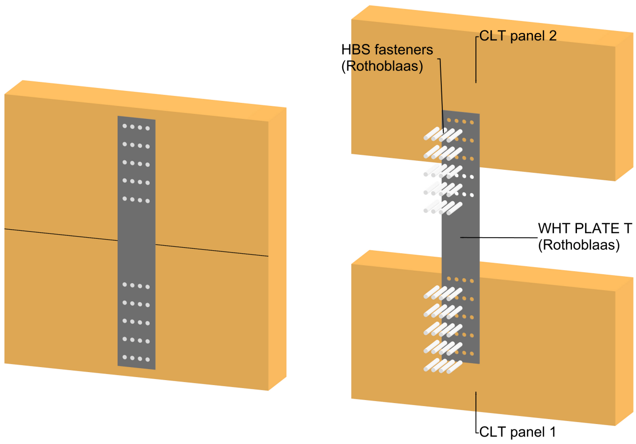


Figure 4.26: WHT plate timber-timber connector Rothoblaas [own]

Strength values can be obtained in table 4.10. For the stiffness an assumption is made, which is related to the hold-down connector of Rothoblaas and based on equation 4.13. The two groups of nails work in series, which means the total stiffness is halved (equation 4.18).

Type	**Type fasteners**	**∅ x L [mm]**	**nv1 [pcs]**	**nv2 [pcs]**	$R_{1,k,timber}$ **[kN]**	$R_{1,k,tsteel}$ **[kN]**	**K-ser [kN/mm]**
WHTPT720	HBS	0 8.0 x 100	28	28	115,8	135,9	21.2
WHTPT820	HBS	0 8.0 x 100	40	40	176,1	206,6	30.3

Table 4.10: Strength and stiffness values for the WHT plate of Rothoblaas

$$K_{v,ser,WHT} = \frac{\frac{n*\rho^{1,5}*d^{0,8}}{30}}{2} \left[\frac{N}{mm}\right] \tag{4.18}$$

Column-Column (4)
The horizontal connectivity between two adjacent modules is based on the connection of Srisangeerthanan et al., where a horizontal plate provides the transfer of shear and axial forces. The connection in figure 4.27 will be modeled as an element with two hinges ("pendelstaaf"), which is capable of transfer axial forces. The system can move freely in lateral direction in this way.

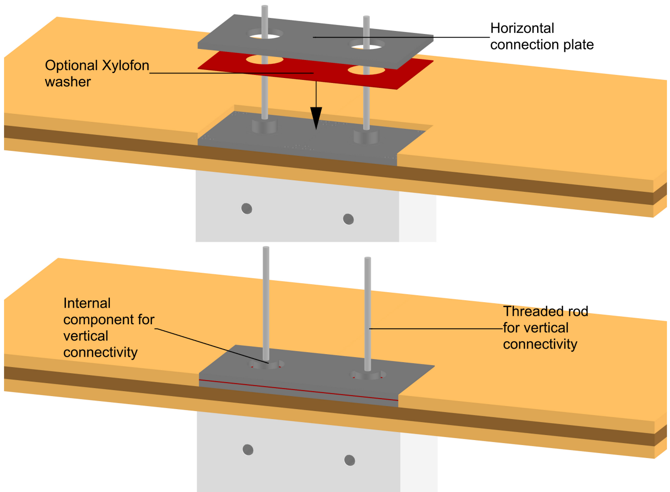


Figure 4.27: Horizontal connection plate for two adjacent modules [own]

Panel-panel (5)
The floor or ceiling of one module and wall can be connected to an adjacent module through an angle bracket of Rothoblaas (see figure 4.28). The vertical stiffness for the Titan joint can be determined with help of equation 4.13. The lateral stiffness of the joint is determined with experimental tests of Rothoblaas and is set to $k_{2/3,ser} = 5600[N/mm]$. The strength in direction 1 depends on whether it is in compression or tension. Table 4.11 provides the lowest resistance values for the lateral direction.

Type	**Type**	**Ø x L [mm]**	**nv [pcs]**	**nh [pcs]**	$R_{2/3,k,Timber}$ **Timber [kN]**	$R_{2/3,Timber}$ **+ Xylofon [kN]**	**Xylofon [mm]**
TTS240	HBS plate	Ø 8,0 x 80	14	14	60	12,5	6,0

Table 4.11: Resistance values for Titan angle bracket of Rothoblaas, direction 4/5 correspond with direction 1; 4 is upward, and 5 is downwards [Values from Rothoblaas]

$$R_d = \min\left(\frac{R_{k,timber} * k_{mod}}{\gamma_m}; \frac{R_{k,steel}}{\gamma_m}\right) \tag{4.19}$$

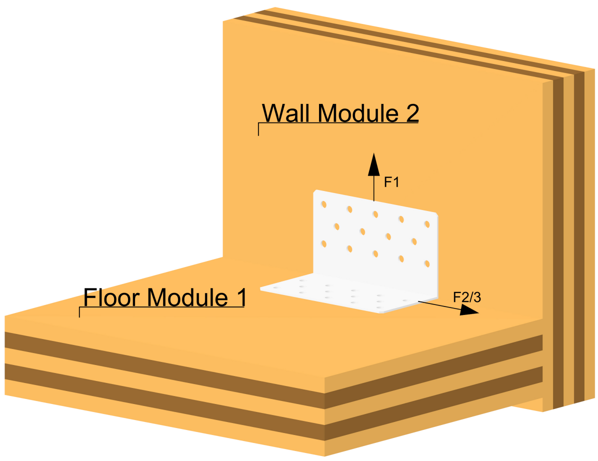


Figure 4.28

4.6.5. Discretization connections

All connections will be modeled as line element with specified stiffness' in earlier sections. Direction K_y represents the in-plane shear stiffness, K_z the out-of-plane shear stiffness and K_x the axial stiffness of the connection. In section 6.1 the directions of an analytical connection are specified.

Conn	kx [kN/mm]	ky [kN/mm]	kz [kN/mm]	Comment
1	∞	0.073	∞	Line Connection
2	36	0.6	∞	Per fastener
3	∞	∞	∞	Global conn 1 and 4

Table 4.12: Determined intra-module connection stiffness'

Conn	kx [kN/mm]	ky [kN/mm]	kz [kN/mm]	Comment
1	∞	∞	∞	Element with two hinges
2	C	28	0	Hinge
3.1	21.2	28	0	Hinge
3.2	30.3	28	0	Hinge
4	∞	∞	∞	Element with two hinges
5	C	5.6	21.6	Hinge

Table 4.13: Determined inter-module connection stiffness'

Table 4.13 provides and overview of all calculated inter-module connection stiffness's. The connection stiffness C is based on the contact between two adjacent CLT panels and can be calculated as is suggested in section 4.4.3. Connection 2 and 3 are often combined, since tensile forces can occur in floor diapragms and shear wall. When tensile forces occur at connection 2, the stiffness in axial direction will be replaced with the stiffness' of connection 3. The third connection is in this way a combination of the shear stiffness of connection 2 and 3. Thereby the behaviour of this joint is bi-linear. Figure 4.29 provides a representation of the linear and bi-linear stiffness of the joints.

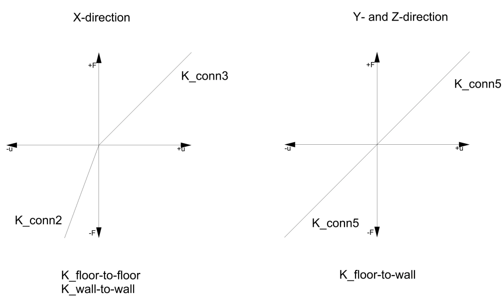
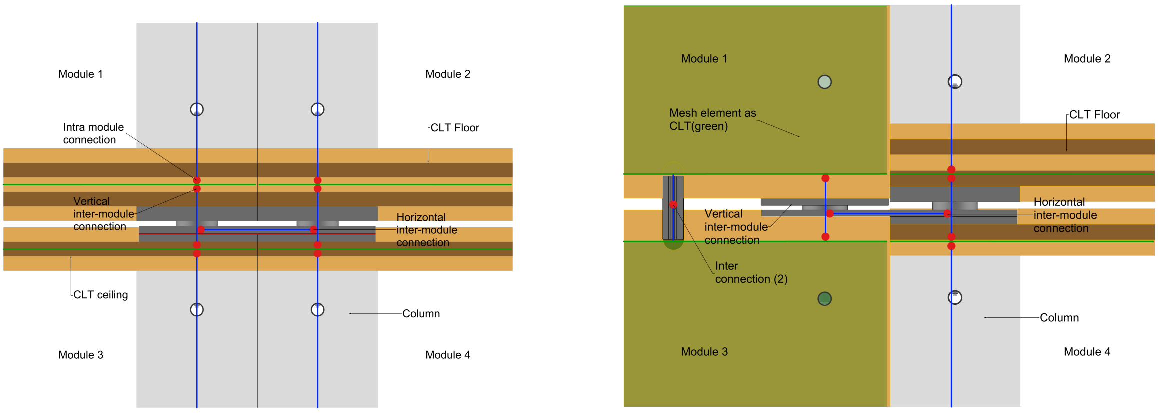


Figure 4.29: Linear and non linear joinstiffness' for inter-module joints [own]

The characteristic strength and stiffness properties of connections 1 and 4 are not known.

These connections are modeled as an element with two hinges. These connections do not provide stability, but the rocking is constrained. An analytical visualisation is presented in figure4.30.



(a) Analytical semi-rigid joint model [own]

(b) Analytical joint model for a CLT panel - panel connection

Figure 4.30: Analytical models for joint configuration

4.7. Conclusions

A corelless stability can be created by stiff CLT floor diaphragms and CLT shear walls. Floor diaphragms transfer the lateral forces to the shear walls. The connections play an important role in the rigidity of the diaphragm (see section 4.4.2). The CLT floor diaphragms can be discretized as a spring system with linear and bi-linear connections.

Rigid modules contain shear walls made of cross-laminated timber. Four mechanisms contribute to the deflections: translational, rotational, shear and bending deformation. The deformation can be idealized as a set of four springs in series. The stiffness of the joints have a huge influence on the maximum horizontal displacement of the wall behaviour (see section 4.5.1 and 4.5.2). The number of panels and type of connection determine the behaviour of multiple shear walls. The continuous shear walls in horizontal and vertical direction can be discretized with the help of linear and bi-linear springs.

Section 4.6 provides an overview of the intra- and inter-module connections. The intra-module connections are: T-Lock, screws and modified pillar connection. The T-lock connector and screws are used to assemble the floor, ceiling and walls of a module. The column connection is based on the Pillar connector of Rothoblaas and connect the column to the floor and ceiling. The slot, WHTPT and TTS240 (angle bracket) connections of Rothoblaas are used for the inter-module connections. The slot connection has a high shear capacity and stiffness, and can be assembled by just two screws. The WHTPT tension plate of Rothoblaas can resist high tension forces. The angle bracket can resist forces in lateral direction for wall-floor assemblies between two modules. Two inter-module connections are based on connections in the mass-timber construction industry and modular steel construction industry, since no existing connections are found in literature.

5

Parametric modular design

The motivation to use parametric design in the construction industry are time and price. Multiple studies in the literature have shown that parametric design is more efficient than the conventional design process. The traditional design process does not require much more time than a parametric design when the algorithm is created for the first time. But when a similar task arises, a parametric design can result in substantial time savings. [36] And the construction costs can be reduced by using a digital design tool. [83]

To understand the problems and possibilities of digital design with prefabricated systems this chapter proceeds as follows: first, the difference between parametric, generative, and computational design is described. Secondly, an overview of reference projects is given to get an initial idea about the possibilities and current barriers. And finally, the concept of the tool created for this thesis is described with the provided software.

5.1. Computational design

Computational, generative and parametric designing methods rely on algorithmic processes. The three terms are often used in collaboration, but there is a difference between the three terminologies. Computational design refers to generating geometries, objects, and architecture with computers in a mathematical approach. Parametric and generative are both examples of computational design. [18]

Parametric design
Parametric modelling enables the architect to create a set of 3D models of buildings with embedded parameters. The parameters' data is changeable. If the designer changes one of the parameters, the form of a geometrical entity changes. [16]

Generative design
The generative design relies on computers and their algorithms. First, the designer puts constraints, goals and provides a ranking for the results. The algorithm of the generative design tool will find the best solution with an iterative process. The algorithm rejects the inadequate answers, and stores the higher solutions. [16]

5.2. Computational design in modular construction

Despite the limited examples of computational design in prefabrication, off-site construction and modular sectors, five reference projects are found. All methods have a common interest in optimizing modular construction design speed with a computer's help. Table 5.1 provides an overview of all projects.

Project	Paper	Author	Elements	Typology	Design approach	Focus	Software	Integrated structural analysis
A	Integrating computational design to improve the design workflow of modular construction (2019)	Greenough et al.	Modular (3D)	Simple	Adapted to modular construction	Efficiency and mass production	Revit + Dynamo + Grasshopper + SAP	An iterative process with SAP and Grasshopper.
B	Generative tool for modular buildings, Gensler research institute (2020)	Gen	Modular (3D)	Simple	Adapted t modular construction	Efficiency and mass production	Dynamo + WPF	No
C	Exhaustive exploration of modular design options to inform decision making (2017)	Mekawy and Petzold	Modular (3D)	Simple	Adapted to modular construction	Efficiency and mass production	Revit + Dynamo	No, maximum of stacking determined on beforehand.
D	Automated design of prefabricated building (1994)	Retik and Warszawski	Prefab (2D)	Simple	Not adapted to modular construction	Preliminary drawings of the architect	Unknown	Element determined on beforehand. No calculations.
E	Development of a design-driven parametric mass timber construction system for modular high-rise urban housing (2019)	Lang et al.	Modular (3D)	Simple	Adapted to modular construction	Efficiency and mass production	Rhinoceros + Grasshopper	No

Table 5.1: Overview software reference projects

All reference projects can be found in appendix D.1. During the literature research are key aspects of the projects noted:

- The elements refer to the dimension of the prefabricated elements that are used (see section 1.2).
- The typology reflects the typology of the constructed building which can be simple or flexible (see section 1.2).
- The design approach reflects if the prefabricated construction system adapts to the design or not.
- The focus is linked to the typology and reflect the goal of the project.
- Which software program's are used.
- If a structural calculation is included in the project.

5.2.1. Review tools

The reference projects show differences, comparisons and limitations. The main design approach branches into two design options:

- Conceptual design not adapted to the modular construction method. In this case, the design is not yet adapted to a particular construction method. The preliminary architectural design is fed into the system and then produces solutions for partitioning the building into modular units.
- Conceptual design adapted to modular construction methods. In the second design method upfront is chosen for a modular construction method. The structural engineer and the architect design a module and a modular system, with certain limitations. During the design the user gets feedback about the design decisions.

Software platform
Most of the reference projects use Revit as central CAD platform. One of the arguments is that the sharing of information and link to other analysing software is relatively easy or Revit was the only reliable solution for developers. Besides Revit, Rhinoceros with Grasshopper is used, due to the better handling of data and visual display of results.

Structural analysis
Currently, the structural analysis and designing of the building shape are performed in separate programs. In an ideal scenario everything is completed and documented in one software package. The number of structural elements in buildings and the lack of interaction between structural and CAD programs result in extra work and low design efficiency.

Workflow reference projects
All tools seem to have a common goal to make better-informed decisions for buildings. Most of the tools' scope is limited by focusing only on efficiency and mass production, which will decrease the customisation in building designs. The traditional parametric software programs, like Grasshopper and Dynamo, are debit to this problem. Slider components can change the model's parameters relatively easy, and with this, a lot of design freedom can be obtained. But this traditional parametric approach cannot present the whole spectrum of solutions for the customised design problems. In other words, the breeding of geometry by parameters (sliders) limits the design possibilities.

Possible workflows
The digital workflow for a modular design distinguishes two workflows:

- Manual approach: this modular design process is based on an iterative process by user choices. The user arranges modules to form a design. The user gets feedback from the structural analysis and can manually change the model based on these results. This model's advantage is to customise a design to the design's needs, not based on optimised parameters. The disadvantage of this workflow is the many manual handling, and it is hard to obtain the optimal structural result.
- Automatic approach: this modular design process is based on an iterative process based on specified parameters stated by the user. A script arranges the modules to form a design. The script gets feedback from the structural analysis and can optimize the design. The advantage of this way of working is the design speed. Only adjustable parameters can create a design. The disadvantage of this workflow is the low customizability of the design.

5.3. Rhinoceros and Revit

The structural analyses were performed in third party software in the reference projects. There are now possibilities to load structural analysis directly into the project's design space. Revit and Rhinoceros are both options to combine structural analysis software with the design space. In figure 5.1 the possibilities are outlined to develop a tool with integrated structural analysis. Before both software programs will be examined a small introduction about Revit and Rhinoceros is given.

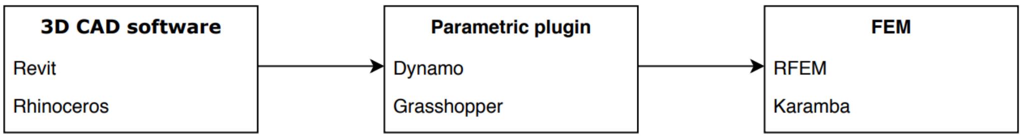


Figure 5.1: Primary software options for design tool [own]

5.3.1. Introduction Revit and Rhinoceros

To create the structural design tool for modular timber construction two standard CAD programs are compared: Revit and Rhinoceros. A test will be performed on both programs to show performance and limitations.

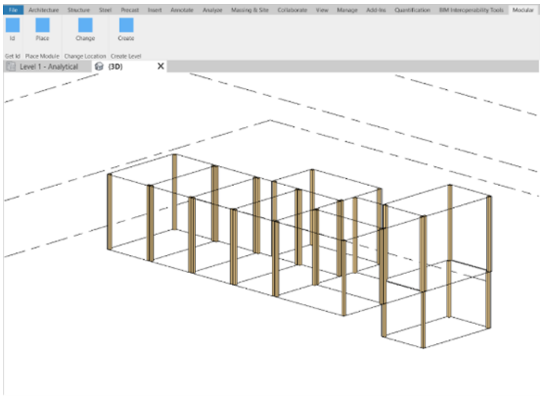
Revit
Revit is a Building Information Modelling (BIM) software program by Autodesk. It can construct a simulated three-dimensional building with parametric objects, which are families. Families are parametrically changeable objects in the model, characterized by adjustable parameters. This means, when an item is updated, it can influence other items, e.g. raising the roof will also change the walls' height.

Rhinoceros
Rhinoceros is a 3D modelling software, which originated as an industrial design software from 1994. Users can exploit NURBS curve algorithms, which make it suitable for all types of complex geometry. This 3D program allows the user to work on parametric modelling, where the additional plugin Grasshopper helps change the parameters, or nonparametric modelling for engineering projects.

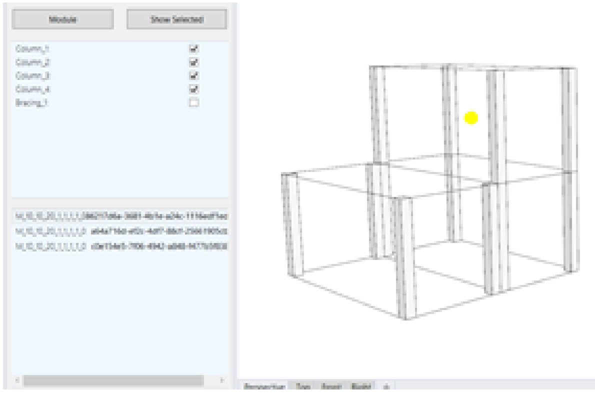
Robert McNeel and Associates constructed the program with Microsofts DotNET framework, which provides a comprehensive framework class, and is open-source. To encourage third party organizations to develop plugins, its Sofware Development Kit (SDK), is made freely available with lots of examples. Rhinoceros distinguishes itself in this way from other parametric software by having an extensive library of free plugins. The external Rhino.Inside library allows Rhinoceros and Grasshopper running inside Autodesk Revit's memory space.

5.3.2. Examination Revit vs Rhinoceros

The first stage consists of examining methods to integrate an intuitive interface for the interaction with the user, within Revit and Rhinoceros. Both programs performed well, through the C-sharp API of Revit, Rhinoceros and Windows Presentation Foundation (WPF) it is possible to add buttons for the placement of the modular units and additional information boxes.



(a) Developed plugin for Revit [own]



(b) Developed plugin for Rhinoceros [own]

Figure 5.2: Modular plugin mockups

Module unit creation
The modules in Revit are based on the parametric family instances while for Rhinoceros instance definitions are used. Modular units can be placed manually or by executed scripts in Dynamo or Grasshopper. The modular buildings can be optimized and manually customized on the user's needs. Instance definitions in Rhinoceros could be created and changed easily by a script, while Revit families' creation was done by hand and took significantly more time.

Structural analysis
In the reference projects the structural analysis is performed in analytical software. The design efficiency can increase up to 80% on specific tasks with relatively simple scripts. [35] Also, the integration of structural results in the model could speed up the design process but is still missing today. [35] Karamba, a plugin of Grasshopper, and RFEM, a plugin of Arcadis, may speed up the design process by giving live feedback on specific design configurations.

Karamba
Karamba, a Finite Elements Analysis (FEA) plugin of Grasshopper, can perform structural analysis for isotropic steel and concrete structures. However, it is also possible to perform structural analysis for anisotropic materials like timber through the MatProps component. [2]

The short computational time of Karamba results in a live response, possible when a change in the geometry occurs and is one of the main advantages. This, combined with the seamless connection between geometry and visual analysis representation, is a significant advantage over other structural analysis programs. [Karamba manual] The following limitations and simplifications of Karamba are observed:

- Karamba does not support load combinations, but they need to be defined as load cases.
- Karamba calculates the buckling length of a beam as the distance between two nodes. With a cantilevered beam the distance is doubled.
- The point load has to be applied through a node at the end of an element.
- Karamba does not support line supports.
- Especially with steel designs some simplifications are made.
- Timber design rules not integrated.

RFEM package by Arcadis
Arcadis created a Dynamo-for-Revit package called DynamoStructural 2.0. It is possible to transfer parametrically generated models to the analytical RFEM and retrieve the results.

Karamba vs RFEM
Karamba has some simplifications that can influence the utilization rates. A case study performed by Jukka verified the results of Karamba with RFEM and showed reliance and potential of Karamba. [54] In 95% of the cases varies the utilization rate between -0.02 and 0.06, meaning that Karamba calculates, in some cases, lower resistance values for structural steel members than RFEM. This research shows that structural design with Karamba gives reliable results and that Karamba is excellent for the use in parametric structural design tools.

A total of 2000 supports, with 1000 line elements, and 1000 line loads has been analysed in Karamba and RFEM by the author of this research. The total calculation time of Karamaba shows significantly better results compared to the traditional software package RFEM. (see table 5.2) A modification in the Arcadis to RFEM package for Dynamo reduces the calculation time substantially, but live graphical feedback is hard to establish.

Type (1000 elements)	**Time RFEM 1**	**Time RFEM 2**	**Time Karamba**
Element creation	7:50 min	9 s	30 ms
Assemble + analyze	3:00 min	9 s	650 ms
Post-processing	3:10 min	3 s	100 ms
Total	14 min	21 s	780 ms

Table 5.2: Calculation results for RFEM and Karamba (own results)

Conclude
Rhinoceros seems to be a better platform to manipulate geometrical modular units. The geometrical entity instance definitions represent the analytical as well as the physical modular unit. These entities can be changed by programming language C-sharp or visual programming platform Grasshopper. The structural analysis can be performed by Karamaba, which gives reliable results and analyses various design configurations quickly.

Program	**Interface**	**Type**	**Module creation**	**Manual placement**	**Algorithm placement**	**Intergrated structural analysis**	**Speed structural analysis**
Revit	Yes (WPF)	Family	Manual	Yes	Yes	Yes	Low
Rhinoceros	Yes (WPF)	Instance definition	Script	Yes	Yes	Yes	High

Table 5.3: Examination Revit vs Rhinoceros (own results)

5.4. Concept modular construction tool

Design decisions for modular construction require a better-informed decision process due to the complexity of inter and intra-modular connections and increasing design possibilities. But no generic process to develop an integrated parametric model for modular timber construction exists. However, to explore and navigate the whole spectrum of design possibilities (custom and automatic designs) and give essential feedback to the designer about his design decisions can be challenging to implement in traditional parametric design tools. In this section will be explained how such a design tool could work and communicate with Rhinoceros and Grasshopper.

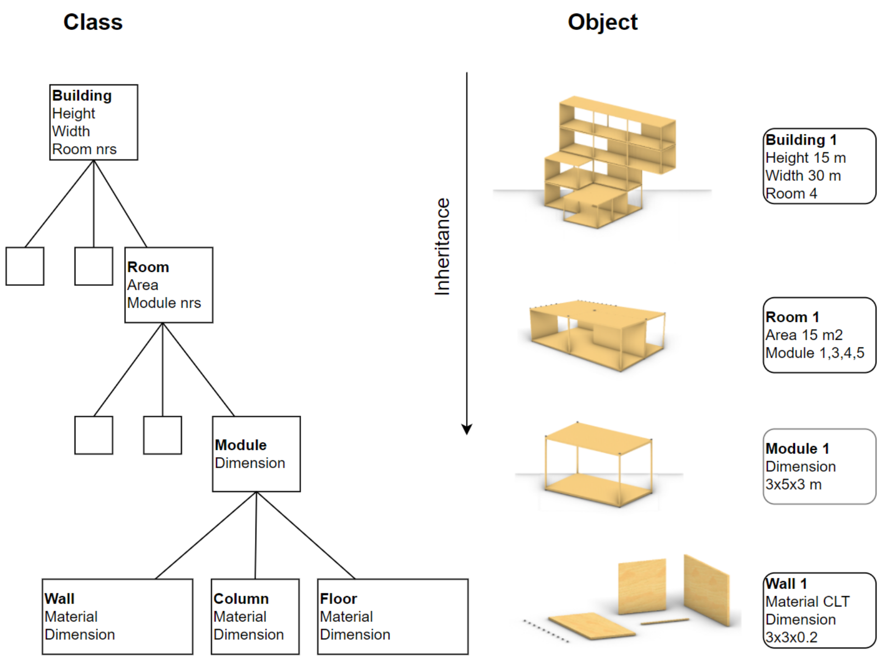


Figure 5.3: Inheritance of a building [own]

5.4.1. Integrated design

Figure 5.4 illustrates a broadly sketched relationship for the used software and steps associated with each modelling process applied in this thesis. An integrated work approach is possible by manual and automatic placement of modules to form a construction for a modular building system. The structural analysis is performed by Karamaba and gives structural feedback for the automatic or manual placement of modules. An iterative design process is possible. The automatic placement can optimize a structure on for example material use. By manual arrangement the user has more control over the placement of modules to customize a design.

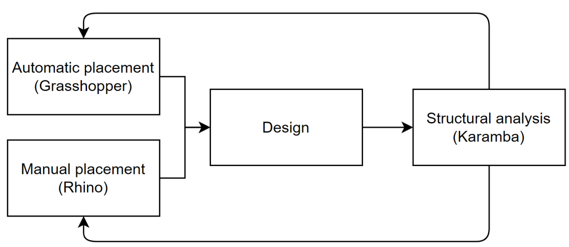


Figure 5.4: Manual and automatic design approach for modular construction [own]

5.4.2. Methodology of parametric modular construction tool

The concept of the created tool in this thesis consists of two phases: phase A and phase B (see figure 2.5). The hybrid modular construction concept (the engineering phase) is discretized in the tool based on three steps:

1. In the first step (A.1): the horizontal and vertical timber components, of a modular unit are discretized in a parametric design tool.
2. In the second step (A.2): the horizontal and vertical timber components form modular units, which are generated by a parametric script and saved in a database.
3. In the third phase (A.3): the intra- and foundation-connections are added to the tool. The hybrid modular timber construction concepts with a coreless stability system can be used to construct buildings.

Phase B enhances the creation of the parametric tool (Habitat21), which can communicate with the hybrid modular timber construction concept from phase A. Four phases distinguish the workflow:

1. In step one (B.1): Rhinoceros recognises modular units and their properties. The user adds modules to the document by help of a plugin (Manual) or by the help of a Grasshopper script (Automatic).
2. In step two (B.2): all geometries are assembled in Grasshopper. The inter-module connections (of step A.3) are added to the system to form building(s). The position of the supports are determined and the load combinations are made automatically.
3. In step three (B.3): a structural calculation is performed by Karamba based on the assembled model from step two (B.2).
4. In step four (B.4): the results are fed to the user by previews and numerical values.

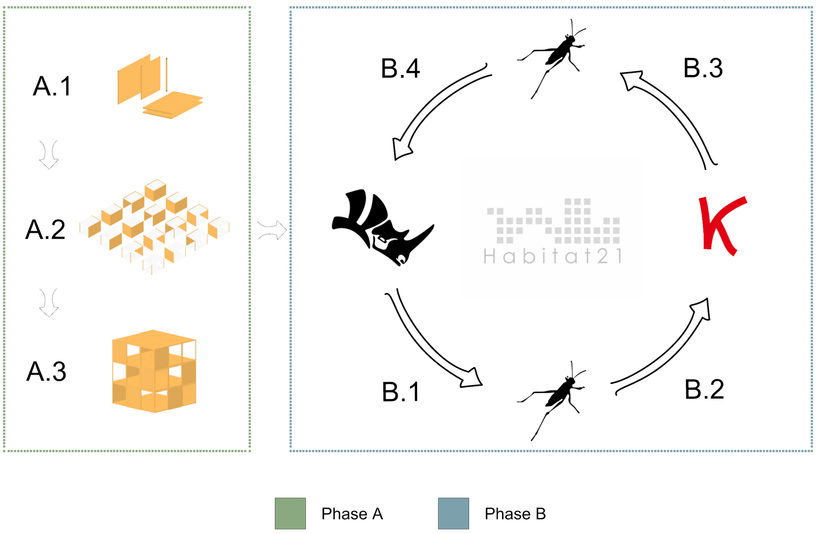


Figure 5.5: The design process of parametric tool in combination with the hybrid modular timber system [own]

5.4.3. Objected-oriented programming and modular system

The parametric tool should communicate with Rhino for manual placement and Grasshopper for automatic placement. To let the tool communicate with Grasshopper and Rhino all objects in a project can be represented by classes created with programming language C-sharp. The building consists of rooms consisting of modules consisting of different structural objects (see figure 5.3)

5.4.4. Analytical model

The modular system communicates with the structural program Karamba. Figure 5.6 provides an overview of the Karamba model. A structural model will be assembled, consisting of nodes, elements, supports, joints, and loads (Yellow boxes). Elements are formed by node ids and can contain loads. Point loads or element loads can be defined based on the element id or node id. Supports can be determined based on the node id. Joints are attached to the start and end of an element. All blue boxes represent the necessary properties.

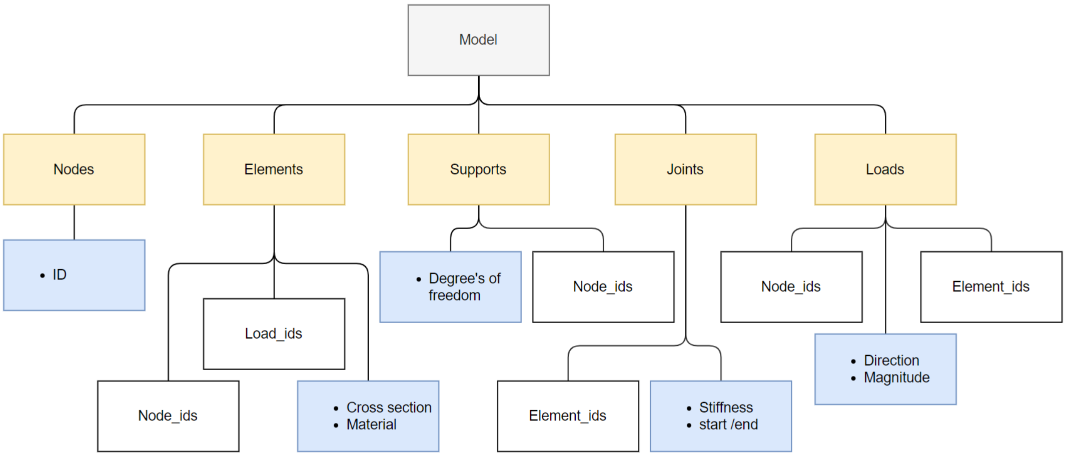


Figure 5.6: The design process of the modular construction system [own]

5.5. Conclusions

Five existing parametric tools, analysed by the author, seem to have a common goal to make better informed decisions for buildings. However, most of the tools' scope is limited by focusing only on efficiency and mass production, which will decrease the customisation in building designs. A parametric tool with an automatic and manual design approach as visualised in figure 5.4 can create different building typologies. The methodology of the parametric modular construction tool is visualised in figure 5.5.

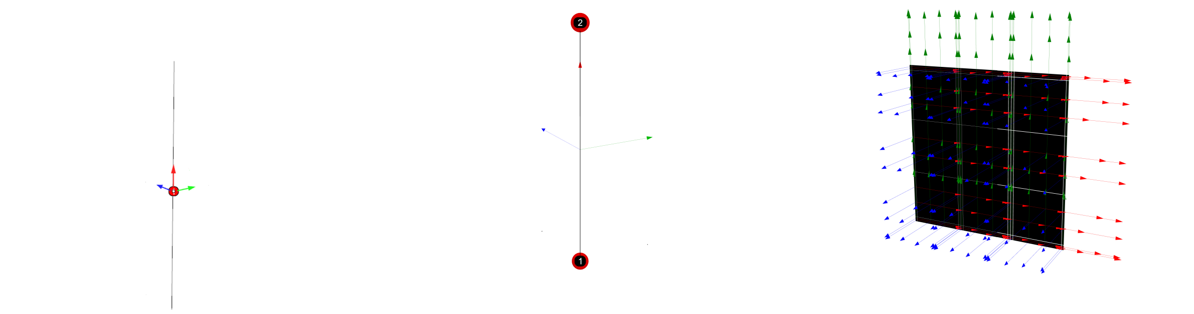
Two software programs have been analysed for a modular parametric design tool in section 5.3. With Karamba and Rhinoceros the user is informed in an early design phase by previews or numerical values. The error in placement of modules is minimized by predefined snapping points with Rhinoceros. The creation of a predefined modular system can be done by a visual programming languages as C#, VB-script or Python instead of Families with Revit. And the structural calculation of Karamba gives reliable and fast results compared to the live feedback of RFEM. Rhinoceros appeared to be more suitable for this thesis to discretize a modular system and inform the user in an early design stage.

6
Tool development

In this section will be explained how the tool is designed based on information supplied in chapter 5 and 4. The interaction between the analytical and physical model is described in the first part. Secondly, the load and load combinations with global and local requirements are defined. And finally, the parametric tool with user interface and included structural feedback is presented.

6.1. Parameterization of system

The modules in the modular system have a physical and analytical representation. The physical representation is related to the architectural visualisation and bears properties as: material, colour, detail. The analytical model is related to the structural schematization of the modules and contains line elements and meshes (figure 6.1 for columns, wall, floor and ceiling elements). The line and meshes have defined orientations to perform calculations with Karamba. The analytical objects bears properties as: cross sectional values, material, and dimensions. The joints for the analytical model are formed by small line elements ($length < 0.005mm$) with a hinge at the start middle or end, with a stiffness' in three different orthogonal (see figure 6.1a).



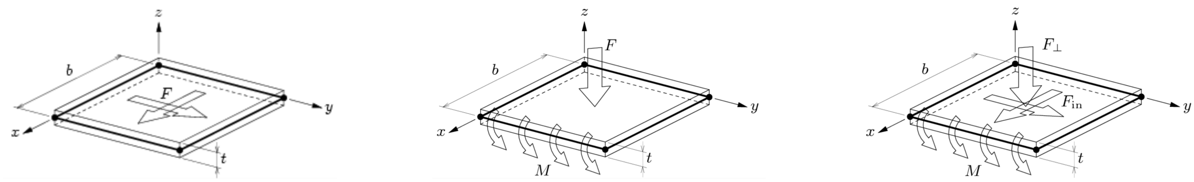
(a) Connection element with stiffness directions, the x-axis (red) longitudinal with the direction

(b) Line element in Karamba and Rhinoceros defined by a start(1) and end(2) point and the x-axis (red) longitudinal with the direction

(c) Mesh element in Karamba and Rhinoceros defined by the orientation of axis; z-axis(blue) defines the front or back of the element

Figure 6.1: Analytical element types in Karamba [own]

The 2D meshes are modeled as quadrilateral shell elements. The shell elements are a combination of plane stress (membrane) and plate elements. This means that shells are able to withstand in-plane forces as well as out of plane forces. Generally are plane stress elements used to model diaphragm action and plate elements to model membrane forces.



(a) Plane/Membrane element (b) Plate element (c) Shell element

Figure 6.2: Type of 2D elements [DIANA FEA]

6.1.1. Parameters

The rules for the modular system are defined in the previous chapters. All these rules are translated into a Grasshopper script which generates a database of all modules. The parameters for the Grasshopper script are defined in table 6.1. The height and width of the modules have constant values, respectively 3 and 3 meters. The minimum length of the module is set to 3 meters with a step size of 1 meters up to a maximum of 6 meters (see section 4.3). Multiple thicknesses are considered for the wall and column thickness. The Wall configurations are explained in section 1.3, and exhibit a total of 16.

Parameter	Type	Parameters
Height	Constant	3
Width	Constant	3
Length	Variable	3,4,5,6
Column/wall thickness’	Variable	100,150,200,225,280,315
Wall configurations	Variable	16 Possible iterations

Table 6.1: Used parameters for creation of analytical modules database

The defined parameters result in a total of 384 different module configurations. Some of these are visualised in figure 6.3. The 384 different modules are saved as instanceDefinitions geometries inside Rhinoceros. This method is chosen, because it doesn’t reduce the calculation speed of Rhinoceros. When adding more than 1000 different instance definitions there is a significant loss in speed in Rhinoceros.

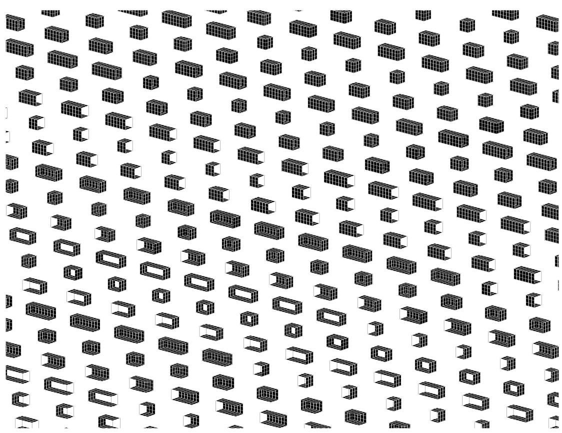
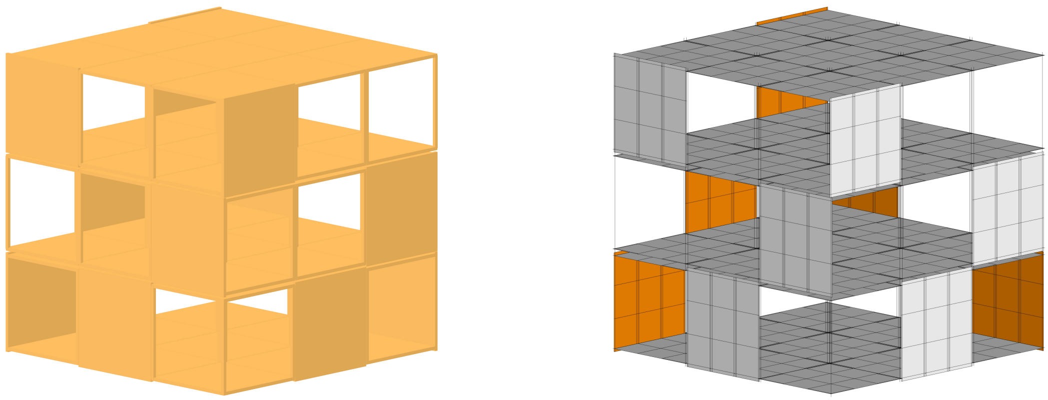


Figure 6.3: Analytical iterations of the modular system [own]

6.1.2. Physical and analytical model

Every module contains a physical and analytical model (see Figure 6.4). A Grasshopper script assembles and categorizes all elements as explained in section 5.4.4. An iterative Karamaba script (see appendix F.1) performs the structural calculations to locate the bi-linear connections and assign the right stiffness' as described in section 4.6.5.



(a) Physical model

(b) Analytical model

Figure 6.4: Model representations [own]

6.2. Glulam and Karamba

Two different methods can be used to discretize properties in Karamaba. The first method is by selecting material from a list of predefined materials through the ”MatSelect” component. The family GlulamTimber can be chosen with type. Or manually specified material properties can be imported through the ”MatProp” component. For this project the properties of GL32h are selected and specified in table 6.2.

GL 32h	Symbol	Value	Unity
Youngs modulus in first material direction	E1	1420	kN/cm2
In plane shear shear modulus	G12	65	kN/cm2
Tensile strength	ft	2.56	kN/cm2
Compression strength	fc	3.2	kN/cm2
Specified weight	gamma	4.8	kN/m3

Table 6.2: overview of the input used as input to define the material GL32h in Karamba

6.3. Cross-laminated timber and Karamba

The material properties used as input for the finite element analysis in Karamba are calculated in this section. Karamba can only account for isotropic and orthotropic properties in one direction. Due to these software restrictions, fictitious values for the elastic and shear modulus need to be calculated for the entire element. This part will handle how will be dealt with the orthotropic CLT elements with different cross-sectional values in x and y-direction. In the software the CLT element will be described as a massive beam with orthotropic material properties.

6.3.1. Input parameters Karamba

The input parameters for Karamba are the elastic modulus in longitudinal and transverse directions (E1 and E2), the shear modulus in longitudinal or transverse direction (G31 and G32), and the in-plane shear modulus (G12). The directions are visualised in figure 1. To calculate the properties regular beam are theories used with orthotropic plate properties. The properties are calculated for 3, 4, and 7 layered CLT panels with equal lamella thickness and wood composition.

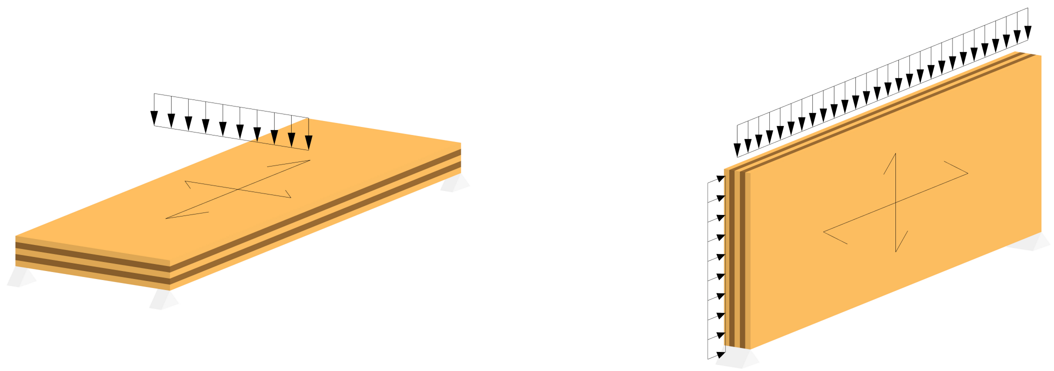
6.3.2. Assumptions

For the calculation of the material properties a number of assumptions are taken:

- The stresses in the transverse layers are not assigned and the modulus of elasticity is assumed 0 ($E_{90} = 0$).
- A symmetrical cross-section of the CLT panels is assumed regarding the lamella thickness and composition of wood.
- Values for the properties of the timber-boards are given in table 3.6 of section 3.3.4, which are valid for homogeneous CLT sections made of standard European softwood.

6.3.3. Direction CLT walls, floor and ceiling

The fictitious elastic modulus derived for the axial and bending stiffness can be used as input for E_1 and E_2 and depend on the load case for the model in question. The CLT shear walls are loaded in-plane and are dominated by in-plane tension and compression forces. Equations 6.1 and 6.2 will be used in this case for E_1 and E_2. The CLT floor and ceiling diaphragm plates are predominantly loaded out of plane, and equation 6.3 and 6.4 are better in this case. It would be possible to put the two axial and bending stiffness's directly in a more advanced finite element program, which is the preferred procedure. In [21] and [82] is explained how to do this. By using the out-of plane properties for the floor and ceiling elements the in-plane stiffness is overestimated in the strong direction (1), and underestimated in the weak direction (2).



(a) Floor (b) Wall

Figure 6.5: Load directions for out of plane and in plane loading CLT wall and floor

6.3.4. Axial stiffness

The axial stiffness is calculated for two principal directions, the longitudinal and transverse. Two limiting cases can be observed for the evaluation of the CLT panel:

1. No contact between the boards within the lamina
2. Complete contact between the boards within the lamina

No contact between the board results in an axial stiffness for the perpendicular lamina that is equal to zero. The axial stiffness for full contact between the boards can be calculated for uniform compression and tension. The contact will not be perfect, the glue will be pushed between the boards, and distribution is never exact, and also cracks will develop over time. No contact is assumed in this case.

$$EA_l = \sum E_{l,i} t_i \tag{6.1}$$

$$EA_t = \sum E_{t,i} t_i \tag{6.2}$$

For the use of Karamba the axial stiffness is translated in a fictious elastic modulus:

$$E_1 = E_{c/t,l} = \frac{EA_l}{h_{clt}} \tag{6.3}$$

$$E_2 = E_{c/t,t} = \frac{EA_t}{h_{clt}} \tag{6.4}$$

6.3.5. Bending stiffness

The effective area should be used for out-of-plane loads. For this thesis not the effective area is used, but the net area. For the use of karamba the fictious elastic modulus is derived for pure bending as follows:

$$E_1 = E_{m,l} = \frac{12EI_l}{t^3} \tag{6.5}$$

$$E_2 = E_{m,t} = \frac{12EI_t}{t^3} \tag{6.6}$$

6.3.6. Shear modulus perpendicular to the plane

The shear modulus perpendicular to the plane gave two different values for the transverse and longitudinal direction. The shear moduli of individual layers are shear along the grain or the rolling shear of each lamina.The shear can be determined with help of the shear correction coefficient (k_z) or factor (k) (se equation 6.7):

$$GA_s = \frac{\sum GA}{k_z} = k * \sum GA \tag{6.7}$$

For a rectangular cross the shear correction factor is equal to equation 6.8, the shear correction factor for CLT should be smaller than 0,83.

$$k_{rectangular} = \frac{5}{6} = 0.83 \tag{6.8}$$

Methods to calculate the shear correction factor are provided in the basic design principles of ProHolz [82] and master project of Richard Eriksson and Maria Karlsson [28]. For this thesis the shear correction factors of multiple methods are compared for different CLT configurations. Table 6.3 provides an overview. For the use of Karamba the fictitious shear modulus perpendicular to the plane is defined as follows:

$$G_{31} = G_{l,fict} = \frac{\kappa_l * G_{r,l}}{h_{clt}} \tag{6.9}$$

$$G_{32} = G_{t,fict} = \frac{\kappa_t * G_{r,l}}{h_{clt}} \tag{6.10}$$

Where $G_{r,l}$ and $G_{r,t}$ are defined for a 3 layer element as:

$$G_{r,l} = [G_0, G_{90}, G_0] \tag{6.11}$$

$$G_{r,t} = [G_{90}, G_0, G_{90}] \tag{6.12}$$

Type G90/G0 ratio	Value (Brettersperholz) unknown	Value (Jobstl) 1/10	Value kx 1/10	Value ky 1/10
1 lamella	0.83	0.83	-	-
3 lamellas	0.15 ≤k_{CLT} ≤0.18	0.21	0.2060	0.6944
5 lamellas	0.18 ≤k_{CLT} ≤0.20	0.24	0.2434	0.1881
7 lamellas	0.25 ≤k_{CLT} ≤0.29	0.26	0.2582	0.2291
9 lamellas	0.26 ≤k_{CLT} ≤0.29	0.27	-	-

Table 6.3: Results of the shear correction factor for different CLT configurations from multiple references.

6.3.7. Shear modulus parallel to the plane

The shear modulus parallel to the plane is a complicated topic. The stiffness depends on whether the interfaces are glued and the build up of the CLT panel. [23] A simplified approximation of the in-plane shear stiffness can be calculated with equations‘6.13 and 6.14 determined in [23].

$$G^* = \frac{1}{1 + 6 * \alpha_{FE-FIT,ortho} * (\frac{t}{a})^2} * G_{0,mean} \tag{6.13}$$

$$\alpha_{FE-FIT,ortho} = 0.32 * (\frac{t}{a})^{-0.77} \tag{6.14}$$

- Where G^* is the reduced shear stiffness
- Where $G_{0,mean}$ is the shear stiffness of the material
- Where t is the thickness of the board material
- Where a is the width of the board material, a value of 150 mm is assumed

6.3.8. Results CLT Properties Karamba

This section provides the results for the fictious properties used in Karamba. The properties of the CLT base material is provided in table 6.4. A ratio of $\frac{1}{10}$ is taken for the shear strength perpendicular to the grain.

Material	E0 [N/mm2]	G0 [N/mm2]	G90 [N/mm2]	Density [kg/m3]
CL 24 h	11600	650	65	420

Table 6.4: Base material for calculations CLT

For the configurations of the CLT panels a board thickness of 150 mm is assumed with an average thickness of 40 mm. These values are of importance, because they influence the in plane shear strength. Table 6.5 gives an overview of the CLT configurations with 3,5 and 7 lamellae.

Name	Layer build up [mm]	Board thickness [mm]	Number of layers x direction.	Number of layers y direction	Thickness [mm]
3 CLT H	40-40-40	150	2	1	120
5 CLT H	40-40-40-40-40	150	3	2	200
7 CLT H	40-40-40-40-40-40-40	150	4	3	280
5 CLT V	20-20-20-20-20	150	3	2	100
5 CLT V	30-30-30-30-30	150	3	2	150
5 CLT V	40-40-40-40-40	150	3	2	200
5 CLT V	50-50-50-50-50	150	3	2	250
7 CLT V	43-43-43-43-43-43-43	150	4	3	300
7 CLT V	50-50-50-50-50-50-50	150	4	3	350

Table 6.5: Different CLT configurations for the modular system split in horizontal and vertical directions

Table 6.6 provides an overview of the properties for 3,5 and 7 lammella's as input for Karamba with base material CL 24 h. Table 6.5 provides an overview of the build up of the CLT panels.

Property	**3 CLT H**	**5 CLT H**	**7 CLT H**	**5 CLT V**	**5 CLT V**	**5 CLT V**	**5 CLT V**	**7 CLT V**	**7 CLT V**
h	120	200	280	100	150	200	250	300	350
t	40	40	40	20	30	40	50	43	50
E1 (bending)	11170	9187	8251	9187	9187	9187	9187	8252	8252
E2 (bending)	430	2412	3348	2413	2413	2413	2413	3348	3348
E1 (axial)	7733	6960	6629	6960	6960	6960	6960	6629	6629
E2 (axial)	3866	4640	4971	4640	4640	4640	4640	4971	4971
G12	471	471	471	560	514	471	434	460	434
G31	94	101	103	101	101	101	101	103	103
G32	181	56	72	56	56	56	56	72	72
ft1	10.7	9.6	9.1	9,6	9,6	9,6	9,6	9,1	9,1
ft2	5.3	6.4	6.9	6,4	6,4	6,4	6,4	6,9	6,9
fc1	16	14.4	13.7	14,4	14,4	14,4	14,4	13,7	13,7
fc2	8	9.6	10.3	9,6	9,6	9,6	9,6	10,3	10,3
t12	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5

Table 6.6: Fictious CLT properties used for the cross-laminated timber products in the modular system in N/mm2 or mm

6.3.9. Verification CLT properties

A CLT panel is modeled in Karamba to verify the fictitious CLT properties in table 6.6. The total displacement of a CLT panel in Karamba is compared with the analytical displacement as described in section 4.5.1. The horizontal displacement at the upper right corner is measured, since the upper left corner does not provide accurate results due to peak stresses. In appendix B.4 the geometry of the CLT panel with mesh properties is visualised with stress distribution. The result for a vertical 3 by 5 meters CLT panel can be seen in figure 6.6. The fictitious CLT properties for Karamba give accurate results by having a maximum difference of 2%.

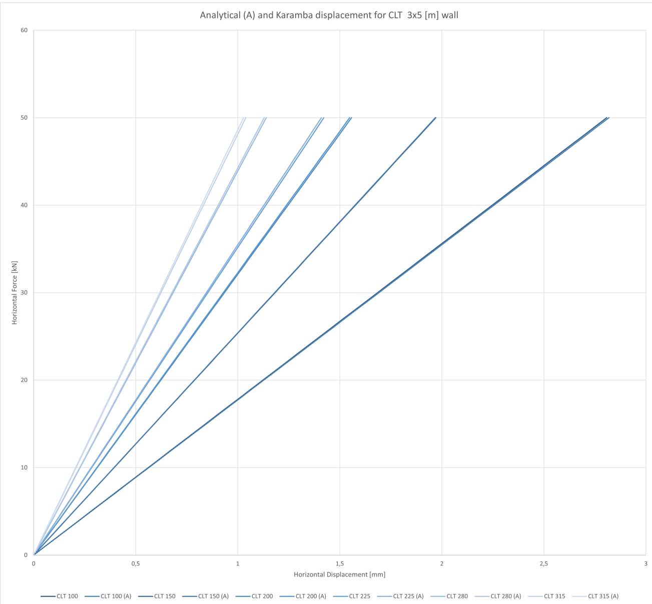


Figure 6.6: Displacments 3x5 meter CLT wall compared, (A) indicates the analytical method.

6.4. Loads

To investigate the building system's structural behaviour within the parametric tool, are different load cases defined in this section. Three load cases will be considered:

- Permanent loads
- Variable loads
- Horizontal loads

6.4.1. Permanent loads

The permanent loads consist of the self-weight of the structural and non-structural components for services. All members' self-weight are given in kg/m3 in table 6.7. The load is determined by Karamaba, which calculate the load based on the self-weight of the material and cross-section. The load for services is given in table 6.8.

Element	Self-weight [kg/m3]
CLT Floor	420
CLT Ceiling	420
CLT wall	420
Glulam column	490
Steel connections	-

Table 6.7: Self-weight in [kg/m3] for used components in the modular building system

Type	Applied	Load [kN/m2]
Services/installations	Ceiling	0.2

Table 6.8: other permanent loads in [kN/m2] for used components in the modular building system

6.4.2. Variable loads

The following variable loads in the table 6.9 can be assigned to the modules. These loads are extreme values and will act in the first case on all floors. But these loads can be reduced with a factor α_n Following section 6.3.1.2 in Eurocode 1. The value for α can be determined with the following equation:

$$\alpha_n = 2 + (n-2) * \frac{\psi_0}{n} \tag{6.15}$$

- Where n is the number of floors ($n > 2$)
- Where ψ_0 can be determined with help of EN 1990 appendix A1

Variable loads and tool

To reduce the workload for this thesis will the extreme variable loads be taken as default. In a late stage is it possible to integrate α_n in the tool, so it is possible to change the load manually. In standard cases is a different load applied for the roofs of the modular units. But to simplify the variable loads is chosen to use the same variable load for all flat floors, and roof apply the permanent load from services' and installations defined in section 6.4.1 at the ceiling elements elements .

Building	Floor load	unit	Category
Dwelling	1,75	kN/m2	A
Stairs	2,00	kN/m2	A
Balconies	2,50	kN/m2	A
Roof	1,5	kN/m2	H

Table 6.9: Loads per type of building following the Eurocode

6.4.3. Horizontal loads

The wind load can be determined following EN 1991-1-4. The wind load depends on the building's height, which is translated to the modular building system per floor. In figure 6.7 is the horizontal wind load per building height determined, the calculation method can be found in appendix E.1. Placement errors result in eccentricities [48], which causes an extra horizontal displacement that increases with the building height are not taken into account.

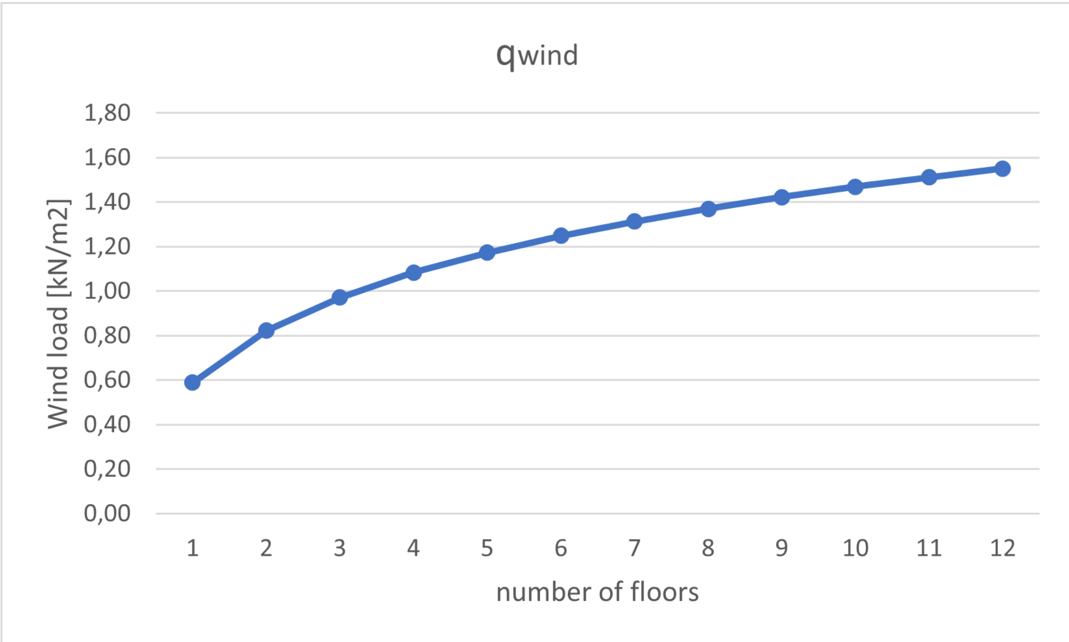


Figure 6.7: Graph of the distributed wind load per building height in [kN/m2]

6.5. Load combinations

The building is considered in consequence class 2, which could lead to medium consequences in terms of loss of human life and/or very significant economic, social or environmental consequences. The load combinations in serviceability limit state (SLS) are used to verify the global structural requirements. The load combinations in the ultimate limit state (ULS) are used to verify the local structural requirements.

6.5.1. Global structural requirements: SLS

The global structural requirements correspond to the maximum displacement at the top of the building. Another word for the global structural requirement is the serviceability limit state. This relates to the prevention of loss of functionality and comfort of the structure. To ensure the functionality and comfort of the building the maximum displacement at the top of the building should not exceed:

$$u_{hor,total} <= \frac{H}{500} \tag{6.16}$$

The maximum displacement in Z-direction is based on equation 6.17 from NA2.23 BS EN 1993-1-1:2005, 7.2.1(1)[B]. The minimum cantilevered length (L) is assumed as a constant in the parametric tool and set to 3 meters, since that is the minimum span of a module in the modular timber construction concept. Therefore, the maximum displacement in the z-direction is a constant value of $3/180$ rounded to zero decimals.

$$u_{z,max} <= \frac{L}{180} = 2[cm] \tag{6.17}$$

Two serviceability limit state load cases are defined to verify the global deformation of the building in x- and y-direction:

1. Permanent loads + horizontal loads in x-direction
2. Permanent loads + horizontal loads in y-direction

6.5.2. Local structural requirements: ULS

The local structural requirement correspond to the ultimate limit state of the components and the connections of the hybrid modular timber construction concept. The ULS relates to the design criteria. The design load is determined with equation 3.1. Three load combinations are calculated to verify the inter-modular joints and components. The permanent loads for ULS 1 are unfavorable. The permanent loads for ULS 2 and 3 are favorable, since the permanent load has a positive influence on the joint verification. The partial factors for ultimate limit state can be found in table 6.10. The load combinations for ULS contain:

1. Unfavorable Permanent loads + variable loads
2. Favorable Permanent loads + wind load x-direction
3. Favorable Permanent loads + wind load y-direction

Design situation	Permanent unfavorable	Permanent favorable	Variable
1	1,35	0,9	-
2	1,2	0,9	1,5

Table 6.10: Ultimate limit state load factors

6.6. Plugin Rhinoceros

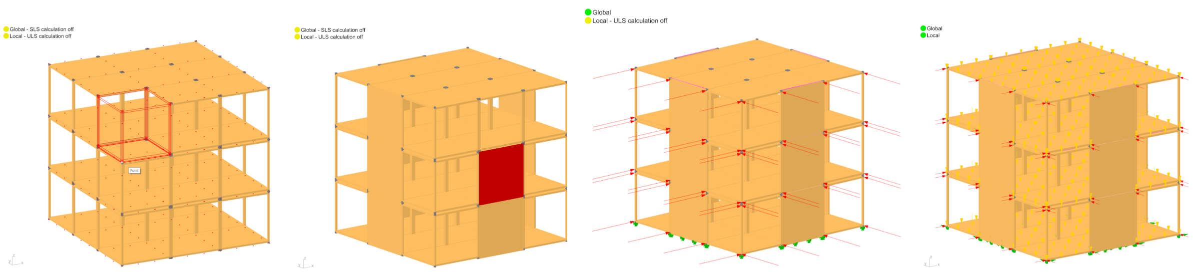
To supply the user with feedback about the structural performance and give the ability to change the model manually a plugin is developed for Rhinoceros with programming languages XAML and C#. The user interface of the plugin contains three tabs where the user has the ability to modify or can analyse the local or global structural performance. A written tutorial of the plugin can be found in appendix G.1.

(a) Design panel (b) Global results (c) Local results

Figure 6.8: User interface Habitat21

6.6.1. User interface

The first tab (figure 6.8a) contains the design parameters. Two toggle buttons can be activated to activate the SLS or ULS calculation. Up to buildings with 20 modules is the calculation speed of Karamba sufficient, for bigger buildings it is recommended to deactivate one or both calculations. If the calculation is activated the circle in the upper corner will turn green or red which represents if the building fulfills the global or local checks.



(a) Step 1 (b) Step 2 (c) Step 3 (d) Step 4

Figure 6.9: Impression workflow tool

6.6.2. Design workflow

One design workflow is visualised in figure 6.9 where a simple building of 9 3x3 modules is generated. In the first step is the geometry of the building generated by selecting a module type and snap it to the right position. In the second phase the modules are modified by placing shear walls. In step 3 the global calculation is performed. Model views for visualisation of the displacements in x and y-direction can be activated in the global tab. In the last phase the local calculations are performed. If a connection does not satisfy the structural requirements, the location of the connection can be indicated via the local results tab.

6.6.3. Feedback

Feedback to the user is provided by numerical results as can be seen in figure 6.8b and 6.8c by the help of magnitude of displacements and unity checks for the connections and components. And in the global and local tabs the user can activate visual feedback.

Global feedback - In the global tab, the user can activate a preview of the displacements. Figure 6.10 provides an illustration of the preview.

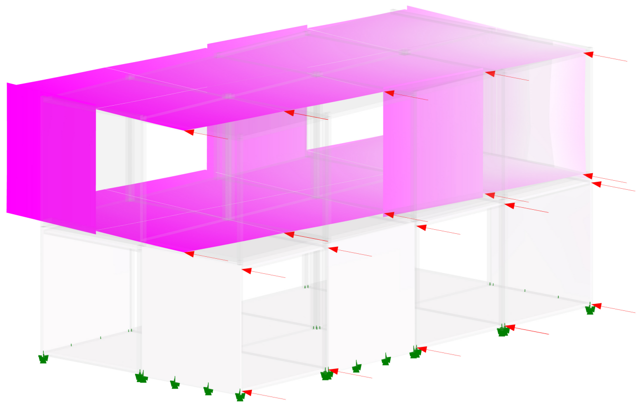


Figure 6.10: Visual feedback of the displacements (global feedback) [own]

Local feedback - In the local tab, the user can activate previews of the governing connections or components. The numerical governing unity check is provided in the left column and the location where the governing unity check occurs can be activated through a button in the right column (see figure 6.8c). Figure 6.11 provides an illustration of the preview where the governing unity check of the slot and WHTPT connection occurs in the walls.

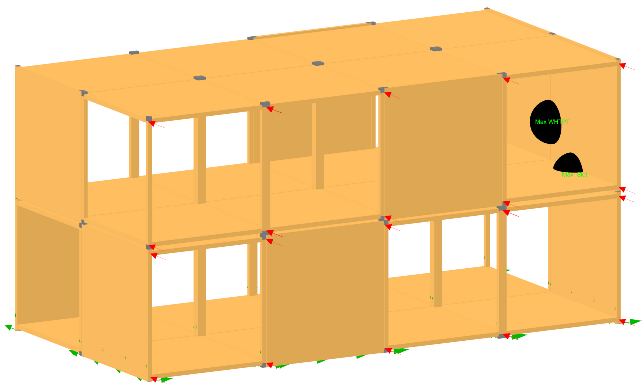


Figure 6.11: Visual feedback of the connections and components (local feedback) [own]

6.7. Conclusions

The intra-module joints and timber components are discretized to instance definitions as described in section 6.1.1, to form a database. For this thesis a Grasshopper script assembles all components and generates the inter-module connections. The shear wall and floor diaphragm mechanism are modeled with linear and bi-linear spring elements as described in section 6.1.2 through an iterative Karamba script. The iterative Karamba script determines the location of the bi-linear joint and assigns the right stiffnesses to the joints.

The properties for glued-laminated timber can be found in NEN-EN 1408. Section 3.2.2 provides all design guidelines for a structural design with glued-laminated timber. The used glued-laminated timber properties for this thesis are discretized via Karamba's ”MatProps” component and can be found in table 6.2.

Design guidelines for cross-laminated timber can be found in section 3.4. Since no European standards for cross-laminated-timber structures exist, papers and manufacturing guides are used for the discretization of cross-laminated panel properties and design guidelines. Due to software restrictions of Karamba, fictitious values for the elastic and shear modulus of cross-laminated-panels are calculated in section 6.3. A cross-laminated-timber panel is described as a massive beam with orthotropic material properties. The properties for various cross-laminated timber cross-sections can be found in table 6.6.

7

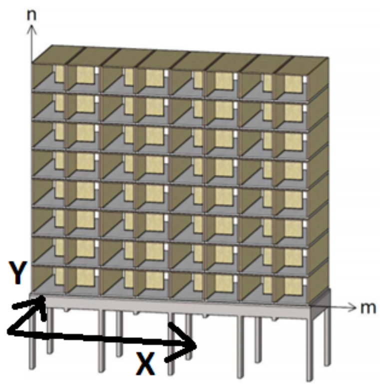
Results

In this chapter the performance of the hybrid modular timber construction concept (provided in chapter 4) is analysed in combination with the parametric design tool (provided in chapter 5). First the results of the examination of the hybrid modular timber construction is done based on a verification study of Hotel Jakarta. The results are presented in 7.1. To examine the manual placement of the modules by Rhino within Habitat21 a workshop with 5 participants (2 architects and 3 engineers) is performed and the results of this workshop are presented in 7.2. At last to examine the automatic placement of the modules by Grasshopper within Habitat21 the calculation time of the automatic placement of one extra shear wall is analysed for 6 buildings. The results are presented in 7.3.

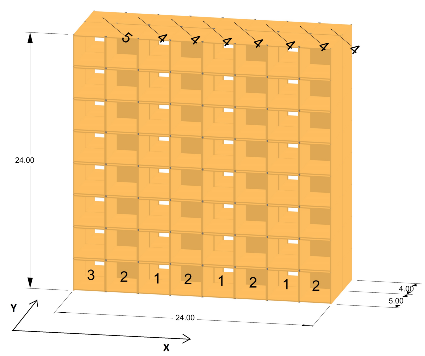
7.1. Examination of the hybrid modular timber construction

The hybrid modular timber construction concept is tested by comparing the displacements in the x- and y-direction of Hotel Jakarta, generated in Habitat21, with the displacements in the x- and y-direction of Hotel Jakarta, generated by the case study of Gijzen. Furthermore the existing connections of the building are tested by unity checks and the magnitude of forces for assumed connections are calculated. At last the components are verified. This considers the maximum stresses in CLT panels due to buckling and the unity checks for glulam columns.

The case study of Gijzen on the modular building Hotel Jakarta in Amsterdam (section A.2.1) is also used for the verification of the structural results of Habitat21, although the modular construction system is different. The differences enhance a thick concrete slab floor instead of a CLT slab, an internal stabilization wall instead of a continuous shear wall (direction m in figure 7.1a) and the connection system. The modular dimensions of Hotel Jakarta and of the hybrid modular timber construction concept also differ in length and width. These are respectively 9 and 3.5 meters for Hotel Jakarta. While the maximum module length and width of the hybrid modular timber construction concept are respectively 6 and 3 meters. To overcome this problem 2 different module sizes are used, which result in a total of 5 module configurations which are visualized in figure 7.2 and indicated in figure 7.1b. The internal stabilization wall withing the modules of hotel Jakarta is created by the hybrid modular construction concept by a continuous shear wall in the x-direction.



(a) Model Hotel Jakarta by Gijzen, width = 28m, length = 9 meters, height = 23,2 meters, m-direction = x-direction



(b) Replicated model of Hotel Jakarta with module numbered modules

Figure 7.1: Comparison two models

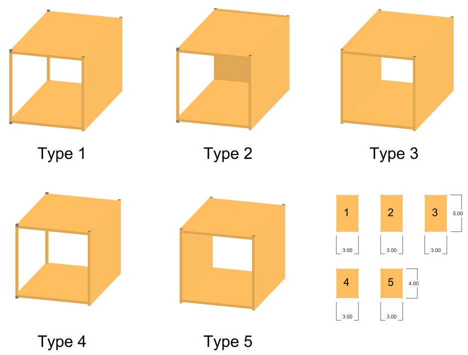


Figure 7.2: Overview of module configurations to replicate Hotel Jakarta

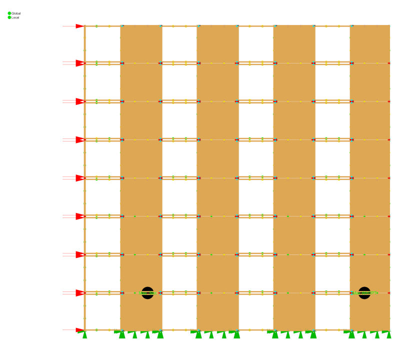
7.1.1. Results displacement x- and y-direction

Table 7.1 provides an overview of the displacements and local verification's of Hotel Jakarta and the replicated buildings with Habitat 21. Three different configurations of Hotel Jakarta are generated using the hybrid modular timber construction concept in Habitat21. As can be seen in Table 7.1 the displacements in y-direction of the replicates in Habitat21 are in the same range as the displacements of the original Hotel Jakarta study. It appears that the replicated hybrid modular timber construction of Hotel Jakarta in Habitat 21 is able to approximately reproduce the original displacement in the y-direction of Hotel Jakarta calculated by Gijzen (difference of 9.829-9.187=0.642 mm). However the displacement in the x-direction could not be reproduced in the same order of magnitude as the original displacement in the x-direction (difference of 41.830-28.299=13.081 mm).

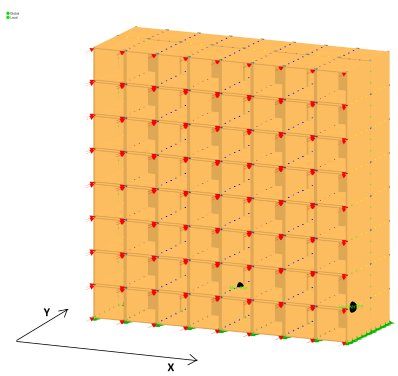
The model with 100 mm CLT wall thickness satisfied the deformation, but the slot joint failed in ultimate limit state. So the local requirements were not satisfied. For the model with a CLT wall thickness of 150 mm the shear capacity of the slot connection increased to 48,15 kN (see table 4.9) and the local requirements were satisfied. The third building of Habitat 21 (*) has stiff connections, which means that the stiffness of the connections is put to infinity in Habitat21. The last row of Table 7.1 shows a displacement in the x-direction of 6.880 mm and in the y-direction of 4.510 mm which suggests that the stiffness of the inter-module connections influences the total displacement in x- and y-direction.

Building	t_{floor} **[mm]**	$t_{ceiling}$ **[mm]**	t_{wall} **[mm]**	u_x **[mm]**	u_y **[mm]**	**Local Requirement**
Hotel Jakarta	140 Concrete	99 CLT	142	41.830	9.187	-
Habitat 21	200 CLT	120 CLT	100	30.855	11.915	Not satisfied
Habitat 21	200 CLT	120 CLT	150	28.299	9.829	Satisfied
Habitat 21*	200 CLT	120 CLT	150	6.880	4.510	-

Table 7.1: Results for case studies (Habitat21* has stiff connections)

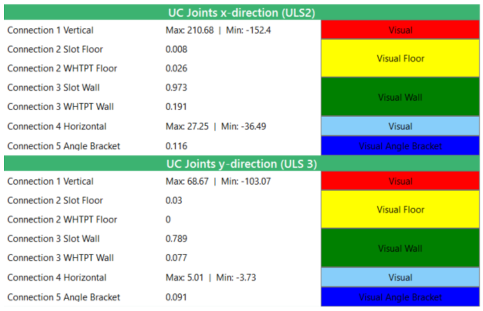


(a) Load combinations ULS 2 (x-direction), governing locations for WHTPT and slot joints in the wall are located



(b) Load combinations ULS 3 (y-direction), governing locations for WHTPT and slot joints in the wall are located

Figure 7.3: Model analyses x and y direction



UC Joints x-direction (ULS2)		
Connection 1 Vertical	Max: 210.68 | Min: -152.4	Visual
Connection 2 Slot Floor	0.008	Visual Floor
Connection 2 WHTPT Floor	0.026	
Connection 3 Slot Wall	0.973	Visual Wall
Connection 3 WHTPT Wall	0.191	
Connection 4 Horizontal	Max: 27.25 | Min: -36.49	Visual
Connection 5 Angle Bracket	0.116	Visual Angle Bracket
UC Joints y-direction (ULS 3)		
Connection 1 Vertical	Max: 68.67 | Min: -103.07	Visual
Connection 2 Slot Floor	0.03	Visual Floor
Connection 2 WHTPT Floor	0	
Connection 3 Slot Wall	0.789	Visual Wall
Connection 3 WHTPT Wall	0.077	
Connection 4 Horizontal	Max: 5.01 | Min: -3.73	Visual
Connection 5 Angle Bracket	0.091	Visual Angle Bracket

Figure 7.4: Local requirements of Habitat model with 150 mm wall thickness. Unity checks for inter module joints of replicated Hotel Jakarta with wall thickness 150 mm. Connection 1 and 4 give maximum (tension) and minimum (compression) forces in kN

7.1.2. Results components

The glulam columns and vertical CLT panels should satisfy multiple unity checks as mentioned in section 3.2.2 and 3.4.3. The columns only resist tension and compression forces, since the joints are modeled as hinges with three rotational degrees of freedom. Table 7.3 gives an overview of the maximum design load for buckling, compression and tension. Glulam properties GL32h are used from table 3.4. Factors for the calculation of the relative slenderness are provided in table 7.2. The unity checks for the Hotel Jakarta building typology are provided in figure 7.5. Figure 7.5 provides an overview of the verification against buckling for multiple cross-sections against the number of floors.

Type	value	unit
β_c	0,1	-
$\lambda_{rel,0}$	0,3	-

Table 7.2: Values for relative slenderness from the Eurocode

Type	G 100	G 150	G 200	G 225	G 280	G 315
Thickness [mm]	100	150	200	225	280	315
A_{net} [mm2]	10000	22500	40000	50625	78400	99225
λ	103,9	69,3	52,0	46,2	37,1	33,0
λ_{rel}	1,57	1,05	0,79	0,70	0,56	0,50
k	1,80	1,10	0,85	0,78	0,69	0,65
k_c	0,37	0,69	0,86	0,89	0,93	0,94
$F_{buckling,d}$ [kN]	57,5	240,2	526,7	692,3	1115,9	1431,0
$F_{c,d}$ [kN]	153,6	345,6	614,4	777,6	1204,2	1524,1
$F_{t,d}$ [kN]	122,9	276,5	491,5	622,1	963,4	1219,3

Table 7.3: Properties of different glulam columns, with tension, compression and buckling design loads for different cross-section

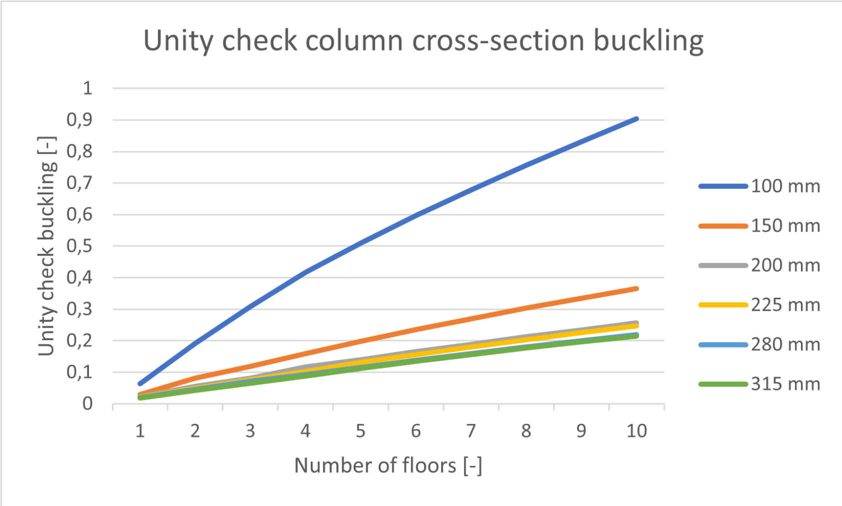


Figure 7.5: Unity check for column with respect to the floors with the following building parameters: Ceiling = 120 H, floor = 120 H, Wall = variable

Glulam column

The glued-laminated column with a dimension of 100x100 mm can be used for this replicated building up to 10 floors (see figure 7.5). However, the unity check for buckling without fire safety is taken into account. Therefore, the glued-laminated column with a dimension of 150x150 or larger could be chosen depending on the fire-safety requirements.

CLT panel

The forces in the shell elements can not be calculated, because the iterative Karamba model (see appendix F.1) produces an error. The maximum stresses for buckling are given in table 7.4, when no bending forces occur. The major direction (x) will be assumed for the transfer of vertical forces. The calculation method is provided in appendix B.2.

Type	**5 CLT V**	**5 CLT V**	**5 CLT V**	**5 CLT V**	**7 CLT V**	**7 CLT V**
$k_{c,x}$	0,412	0,772	0,916	0,942	0,966	0,976
$k_{c,y}$	0,179	0,387	0,635	0,744	0,923	0,946
$\sigma_{b,x}$ [N/mm2]	5,53	10,38	12,32	12,66	12,98	13,12
$\sigma_{b,y}$ [N/mm2]	2,40	5,20	8,54	10,01	12,41	12,71

Table 7.4: Maximum stresses in CLT panel due to buckling with given buckling factors for a 3 meter height CLT panel

7.1.3. Results connections

The connections have a significant influence on the displacement of the building (see table 7.1). Despite large contribution to the displacement in the x-, and y-direction of the building the characteristic strength properties are governing. The unity checks in figure 7.4 for the connections are calculated following equation 3.1 in section 3.1.2. The characteristic values for the connections can be found in section 4.6. There are no unity checks calculated for connections 1 and 4, since these connections have not been tested. These connections are modeled as rods with two hinges and require more attention in a further study, where the connections can be designed as spring elements.

As can be seen in figure 7.6 the slot connection is the governing connection, due to the shear forces between two adjacent shear wall. The shear capacity of this joint can be increased by a thicker CLT wall (see section 4.6.4).

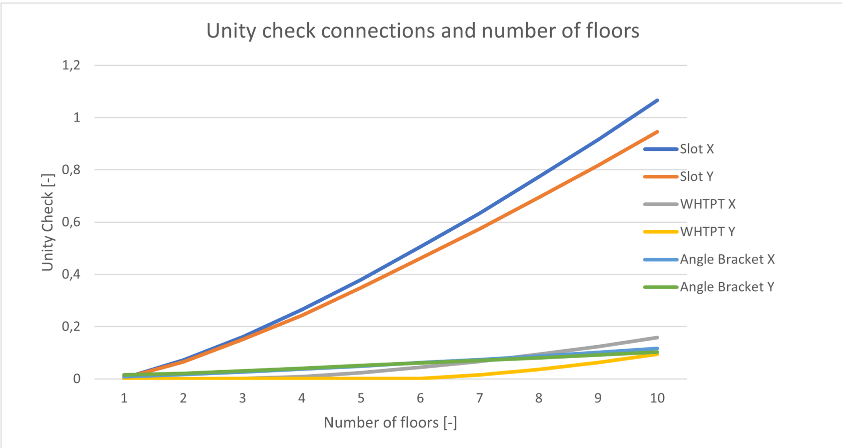


Figure 7.6: Unity check for connection with the following building parameters: Ceiling = 200 H, floor = 200 H, Wall = 200 V

The maximum (tension) and minimum (compression) design forces are calculated for connection 1 and 4. Figure 7.7 provides an overview of the magnitude of forces for Hotel Jakarta with respect to the number of floors.

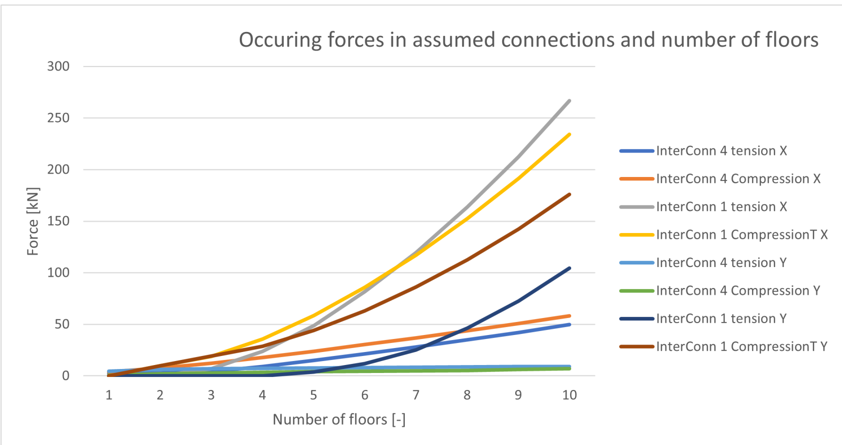


Figure 7.7: Magnitude of force for connection 1 and 4 with the following building parameters: Ceiling = 200 H, floor = 200 H, Wall = 200 V

7.2. Examination manual placement (Rhino): Workshop

To examine the manual placement of the modules by Rhino within Habitat21 a workshop with 5 participants (2 architects and 3 engineers) is performed. This section provides the results of the workshop.

7.2.1. Workshop

The workshop consists of three rounds:

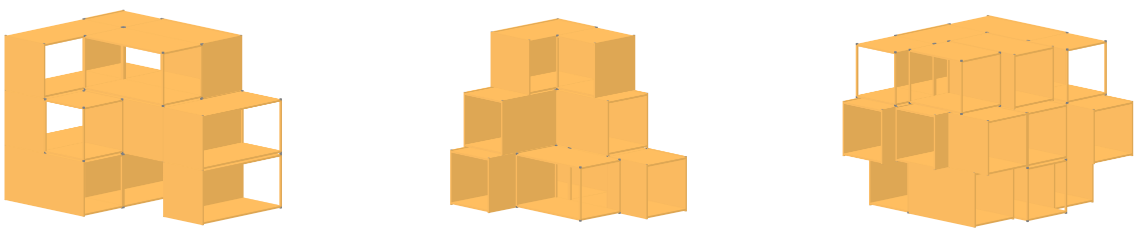
1. The first round contains a tutorial, where participants design two small dwellings. The first dwelling contains a simple stacked modules (figure 7.8a) while the second dwelling (figure 7.8b) covers the boundaries of the system regarding the placement of the modules. Each participant gets a maximum of 15 minutes to fulfill the tutorial.

(a) Model 1 (b) Model 2

Figure 7.8: Two models for the tutorial

2. The second round contains the first case. The participants design multiple building typologies that satisfy the global structural requirements, which are defined in section 6.5.1. Figure 7.9 provides 3 examples of generated building typologies by the author. The participants got 24 minutes to make three designs which comply the following:

   - The design should have a minimum of 10 modules and a maximum of 20 modules[1]
   - The building should have a minimum of 3 levels
   - Unity check of displacement in x- and y-direction (global structural requirement) is < 1.



(a) Example 1 (b) Example 2 (c) Example 3

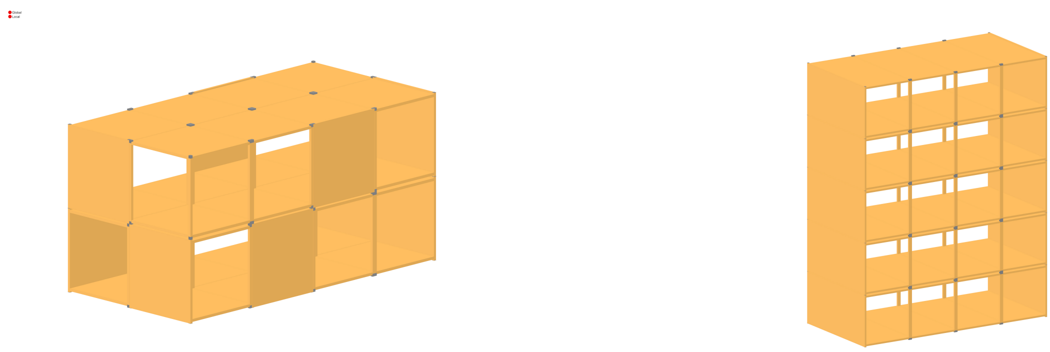
Figure 7.9: Examples generated building case 1 [own]

3. The third round of the workshop contains the second case where the participants finish buildings that do not satisfy the global and local structural requirements, which are

[1]Due to computation time a maximum of 20 modules is set, more modules can be used if necessary

specified in section 6.5.1 and 6.5.2. Each participant gets 5 minutes to fulfill the structural requirements. The two buildings are visualised in figure 7.10. The second building (figure 7.10b) has two extra design challenges:

- No adjacent walls can be placed above each other.
- A minimum of walls should be placed to fulfill the structural requirements.



(a) (b)

Figure 7.10: Buildings for case 3

Table 7.5 provides information about the participants. The participants are classified as an architect or an engineer based on structural knowledge and design experience. The architects have affinity with modular buildings, with a high level of skill in Rhinoceros. The engineers have seldom worked in Rhinoceros.

Nr	**Name**	**Work experience [years]**	**Group**	**Experience Rhinoceros**	**Experience Modular buildings**	**Experience Timber buildings**
1	Michael v Telgen	9	Engineer	Medium	No	Seldom
2	Rogier Schuch	15	Engineer	Low	Yes	Yes
3	Pieter Timmerman	17	Engineer	Very Low	No	Yes
4	Sol v Kempen	6	Architect	High	Yes	Yes
5	Marinus Jongeneel	3	Architect	High	Yes	Yes

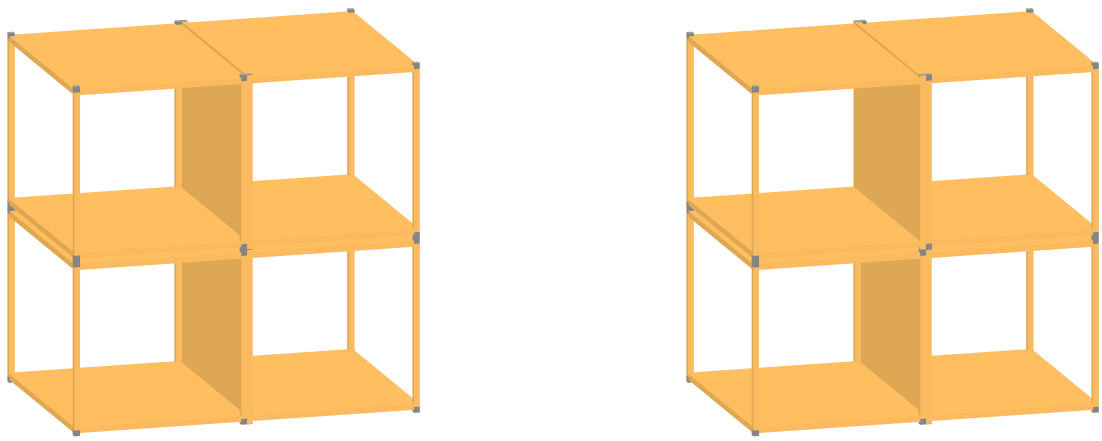
Table 7.5: Participants of the workshop, divided by job function in engineer and architect.

7.2.2. Results tutorial and errors tool

Despite some of the users had no experience with Rhinoceros or modular buildings, all users could duplicate the buildings of the tutorial within the provided 15 minutes. Most of the questions during the tutorial were related to the placement of the modules.

During the workshop one error was observed regarding the missing information about the modular construction concept, these are:

- Errors regarding the placement of modules. The participants wanted to place two adjacent walls on top of each other (see figure 7.11a). However, the second adjacent wall was placed in the wrong module. Due to this misalignment, the forces could not be transferred to the foundation. The author intervened in this situation, and explained that this option will not provide stability.



(a) (b)

Figure 7.11: Placement error, regard the misalignment of continuous walls

Table 7.6 provides information about the error of the tool during the workshop categorized per group.

Error	Architects	Engineers	Total
Modular placement	1	2	3

Table 7.6: Occurred errors during the workshop

7.2.3. Results case 1

Figure 7.12 provides an overview of the produced buildings of the first case. Each participant created 3 buildings and each building was generated within 8 minutes. After 6 minutes the participants were informed to finalize their design and fulfill the global requirements in the remaining two minutes.

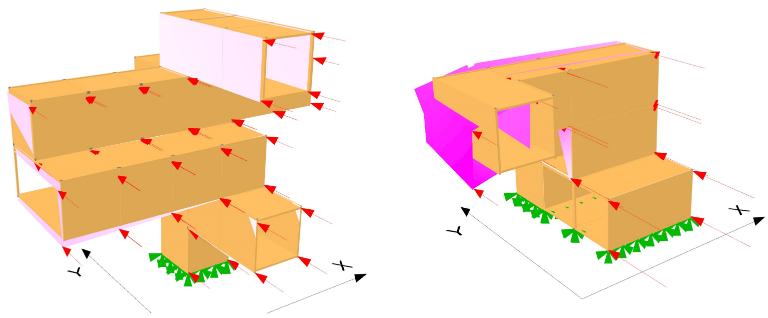
Result / Part. nr.	Building 1	Displacement [cm]	Height [m]	Maximum height / lenght [cm]	Unity check	Building 2	Displacement [cm]	Height [m]	Maximum height / lenght [cm]	Unity check	Building 3	Displacement [cm]	Height [m]	Maximum height / lenght [cm]	Unity check
1 (A)		x = 1.64	12	h = 2.40	0.68		x = 0.95	15	h = 2.40	0.32		x = 2.51	18	h = 2.40	0.7
		y = 1.60			0.67		y = 2.98			0.99		y = 3.48			0.97
		z = 1.51	-	l = 2.00	0.76		z = 4.15	-	l = 2.00	2.075		z = 0.68	-	l = 2.00	0.34
2 (A)		x = 1.43	12	h = 2.40	0.60		x = 1.28	15	h = 2.40	0.43		x = 0.024	9	h = 2.40	0.00
		y = 0.86			0.36		y = 1.46			0.49		y = 0.01			0.00
		z = 1.36	-	l = 2.00	0.68		z = 1.19	-	l = 2.00	0.59		z = 0.375	-	l = 2.00	0.18
3 (E)		x = 1.00	12	h = 2.40	0.42		x = 1e10	15	h = 2.40	X		x = 4.36	9	h = 2.40	1.82
		y = 1.52			0.63		y = 1e10			X		y = 27.77			11.57
		z = 1.92	-	l = 2.00	0.96		z = 1e10	-	l = 2.00	X		z = 14.62	-	l = 2.00	7.31
4 (E)		x = 0.59	9	y = 1.80	0.33		x = 1.43	9	y = 1.80	0.79		x = 2.29	12	y = 1.80	0.95
		y = 1.79			0.99		y = 1.47			0.82		y = 2.08			0.87
		z = 0.38	-	l = 2.00	0.19		z = 1.38	-	l = 2.00	0.69		z = 1.63	-	l = 2.00	0.815
5 (E)		x = 1e10	12	h = 2.40	X		x = 0.27	9	h = 2.40	0.15		x = 2.32	18	h = 2.40	0.64
		y = 1.21			0.50		y = 0.76			0.42		y = 3.13			0.87
		z = 21.58	-	l = 2.00	10.79		z = 1.16	-	l = 2.00	0.58		z = 0.16	-	l = 2.00	0.08

Figure 7.12: Result of the building typologies produced by the architects and engineers

Ten out of the fifteen buildings satisfied the global structural requirements. Five out of the fifteen buildings did not satisfy the global structural requirements. Building one of participant 5 (engineer) and building 2 of participant 3 (engineer) were not stable in one or more directions (see yellow hatched buildings in figure 7.12). Building 2 of participant 1 (architect) and building 3 of participant 3 (engineer) did not satisfy the unit check for the displacement in x-, y- or z-direction (see blue hatched buildings in figure 7.12). For the displacement calculation of building 3 of participant 2 (architect) an error occurred (see red hatched building in figure 7.12). Table 7.7 summarizes these results. The buildings that did not satisfy the global structural requirements are described below in more detail.

Global structural requirements	Architects	Engineers	Total
Stable, but does not satisfy	1	1	2
Unstable in 1 or 2 direction	0	1	1
Unstable in 3 directions	0	1	1
Error	1	0	1
Total does not satisfy	2	3	5
Total satisfy	4	6	10
Total buildings			**15**

Table 7.7: Overview of the buildings that satisfied and did not satisfy the global structural requirements.



Participant 1 (architect), building 2
Unity check displacement z: 2.075

Participant 3 (engineer), building 3
Unity check displacement x: 1.82
Unity check displacement y: 11.57
Unity check displacement z: 7.31

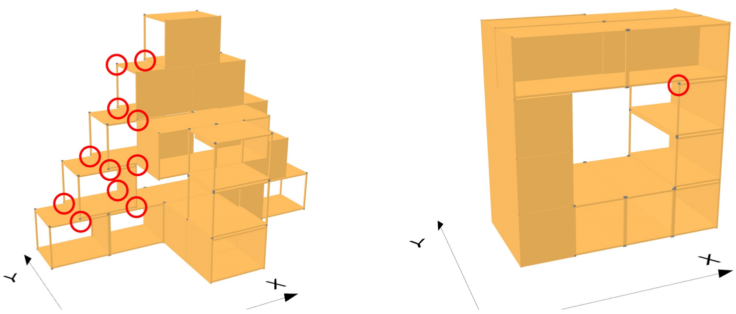
Figure 7.13: Buildings 1 and 3 of participants 1 and 3.

Participant 1 (architect), building 2

Due to long spans in the x-direction large displacements in the z-direction occur. The structural analysis software Karamba calculated a displacement in the z-direction of 4.15 cm and the unity check displacement in the z-direction is 2.075 (>1), as can be seen in figure 7.12. Therefore, the second building of participant 1 (architect) does not satisfy the global structural requirements.

Participant 3 (engineer), building 2

Due to missing shear walls no connections were created on various essential corners of the modules. The missing connections are indicated by the red circles in figure 7.14 from one perspective. The missing connections from other perspectives are not indicated since the building is already unstable. The structural analysis software Karamba calculated a displacement in the x-,y- and z-direction of 1e10 cm as can be seen in figure 7.12. Therefore, the second building of participant 3 (engineer) is unstable in the x-, y- and z-direction, which implies that it does not satisfy the global structural requirements.



Participant 3 (engineer), building 2
Instability: x-, y- and z-direction

Participant 5 (engineer), building 1
Instability: x-direction
Unity check displacement z: 10.79

Figure 7.14: Buildings 2 and 3 of participants 3 and 5.

Participant 3 (engineer), building 3

Due to a cantilevered part in the x- and y-direction large displacements in the x-, y- and z-direction occur. The structural analysis software Karamba calculated a displacement in the x-direction of 4.36 cm (unity check of 1.82), a displacement in the y-direction of 27.77 cm (unity check of 11.57) and a displacement in the z-direction of 14.62 cm (unity check of 7.31) as can be seen in figure 7.12. Therefore, the third building of participant 3 (engineer) does not satisfy the global structural requirements.

Participant 5 (engineer), building 1

Due to missing shear walls no essential connections were created on several essential corners of the modules. One missing connection is indicated by the red circle in figure 7.14. The structural analysis software Karamba calculated a displacement in the z-direction of 21.58 cm (unity check of 10.79) and a displacement in the x-direction of 1e10 cm as can be seen in figure 7.12. Therefore, the first building of participant 5 (engineer) is unstable in the x-direction, which implies that it does not satisfy the global structural requirements.

Participant 2 (architect), building 3

The third building of architect two satisfied the unity checks in the first place. However, due to an error in the Grasshopper script of the parametric tool the wind-load was not generated. Therefore, the displacement in x-, y- and z-direction are not reliable.

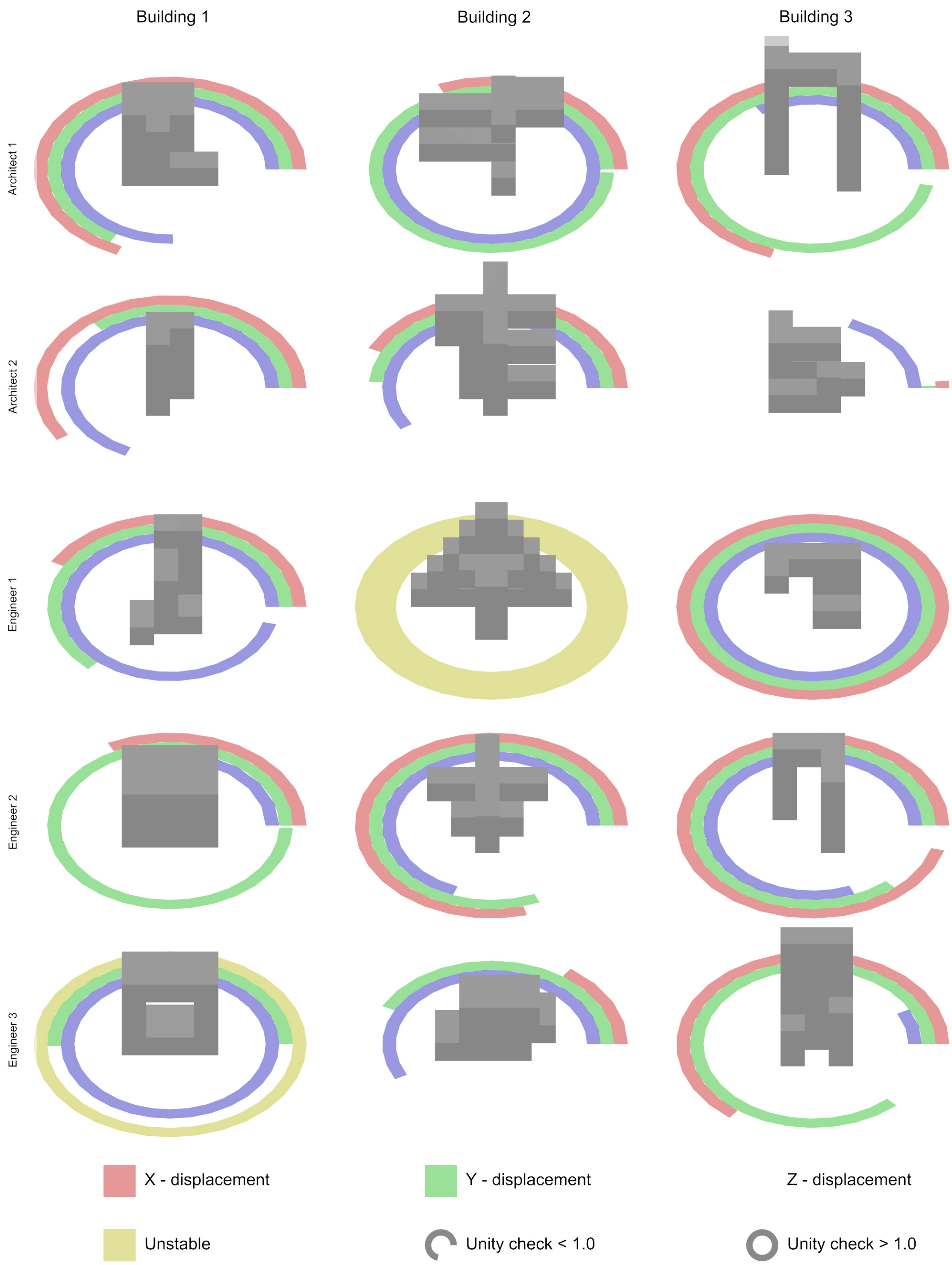


Figure 7.15: Structural performance buildings. The same order as in figure 7.12 is preserved. The top 6 buildings are produced by the architects and the bottom 9 buildings by the engineers.

Structural results The results of case 1 are also presented in a schematic overview in figure 7.15. The unity check for the displacement in x-, y-, and z-direction are presented by respectively red (outer circle), green (middle circle) and blue (inner circle) colors. A full round circle represent a unity check above 1.0, the exact unity check can be found in figure 7.12. The color yellow indicates that the structure is unstable in the x-, y- or z-direction.

Typology score

To compare the building typologies of the buildings generated in case 1, every building is assessed on three different aspects: the amount of corners, the amount of terraces and the amount of overhang terraces (see figure 7.16). One point is rewarded for every corner, while terraces and overhang terraces respectively count for two and three points. So a simple module results in a typology score of 10 points (8*1+1*2+0*3) and is the minimum amount of points that can be rewarded to a design. In figure 7.16 the first building of participant 5 is used as an example. This building obtains $16 * 1 + 2 * 2 + 1 * 3 = 23$ points. Table 7.8 provides an overview of the scores of all generated buildings of case 1. In appendix G.2 an schematic visual overview of all buildings can be found. During the workshop the participants were not informed about this procedure, since it could influence the designs.

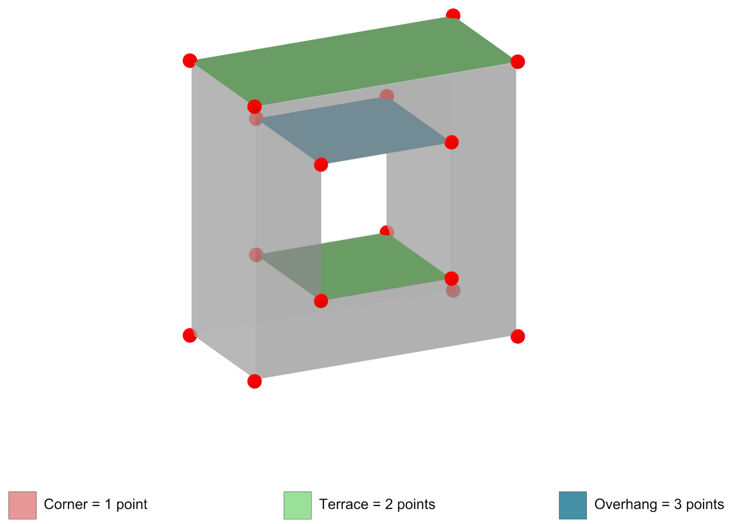


Figure 7.16: Typology score building participant 5 (engineer) building 1, overhang = overhang terraces.

Building	Building 1				Building 2				Building 3			
Participant	Corner	Terrace	Overhang	Total	Corner	Terrace	Overhang	Total	Corner	Terrace	Overhang	Total
1 (A)	26	4	6	36	43	12	15	70	20	2	3	25
2 (A)	24	6	3	33	60	10	18	88	37	10	3	50
3 (E)	37	10	3	50	74	22	6	102	24	6	3	33
4 (E)	8	2	0	10	64	10	24	98	25	2	6	33
5 (E)	16	4	3	23	20	6	3	29	20	6	0	26

Table 7.8: Typology score for every building of case 1.

Global structural requirements and typology score - Figure 7.17 provides a schematic overview of all analysed buildings on global structural requirements and the typology score. The color of the circle indicates whether a building satisfies the global structural requirement (green = unity check < 1, blue = unity check > 1, yellow = unstable in 1 or 2 directions, red

= unstable in 3 directions or error). The building with the lowest typology score of 10 points is produced by participant 4 (engineer, building 1). And the building with the highest typology score of 102 points is produced by participant 3 (engineer, building 2), but this building is not stable in all three directions according to the red color.

The average typology score of the buildings from the architects is 50.3 and the average typology score of the buildings from the engineers is 44.9 as can be seen in table 7.9. Considering only the buildings that satisfy the global structural requirements, this average respectively becomes 30.3 for the architects and 27.3 for the engineers.

Average typology scores	**Architects**	**Engineers**
All buildings	50.3	44.9
Buildings that satisfy the global structural requirements	30.3	27.3

Table 7.9: Average typology scores for the buildings generated in case 1.

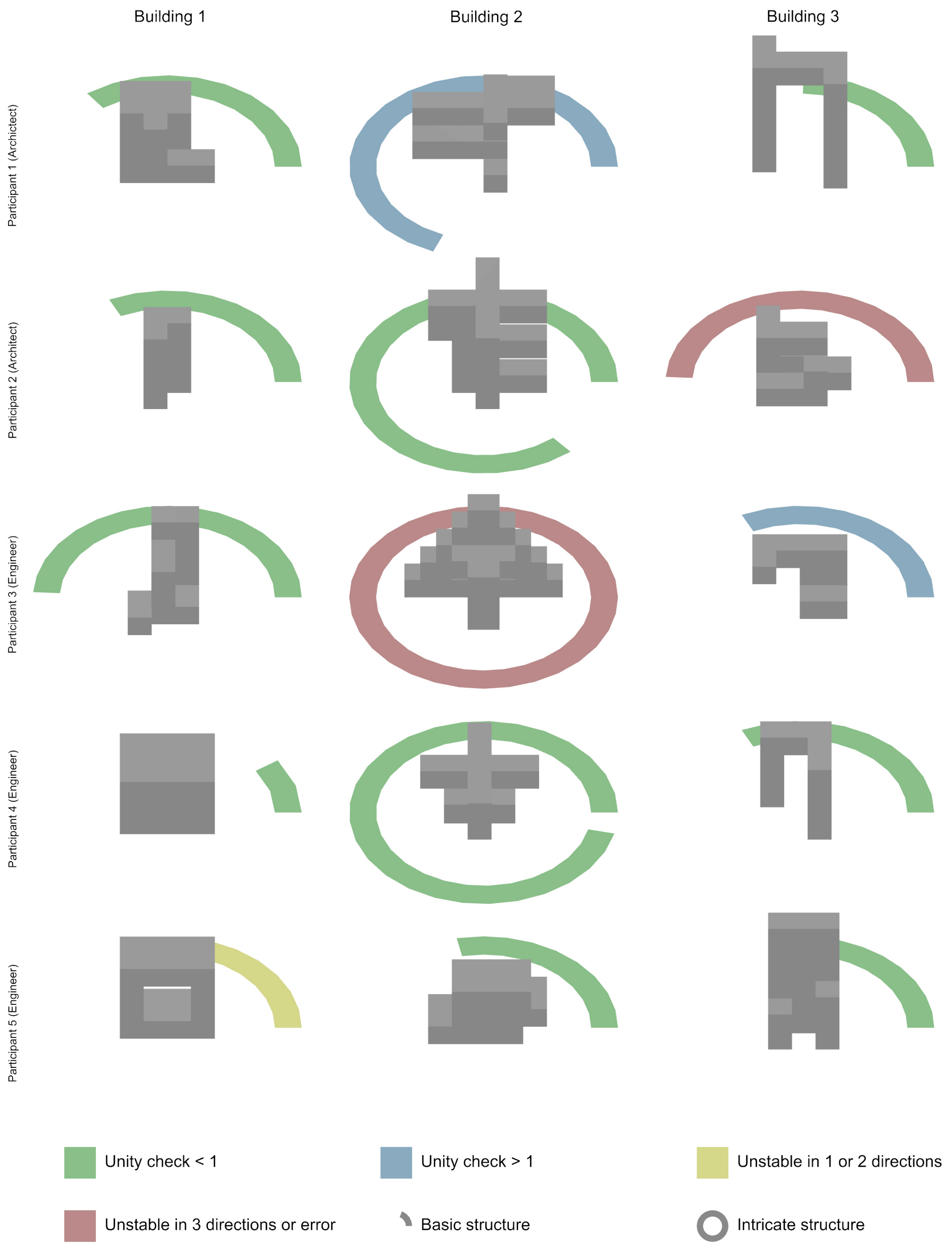


Figure 7.17: Global structural requirements and typology score of buildings.

7.2.4. Results case 2

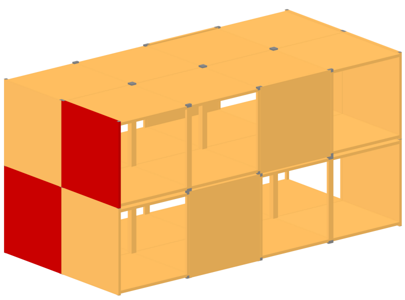
In the second case the participants had to work on buildings that did not satisfy the global and local structural requirements. Each participant got 5 minutes per building to satisfy the global and local structural requirements. For the first building there were no extra restrictions. But for the second building there were two extra restrictions:

- No adjacent walls could be placed above or next to each other.
- A minimum amount of walls should be placed to satisfy the global and local structural requirements.

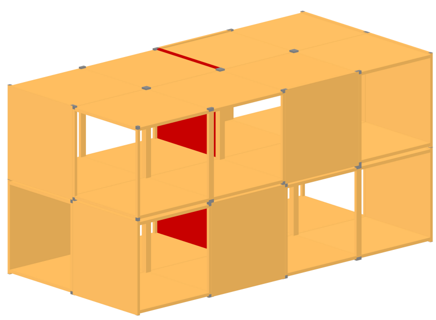
The two generated buildings per participant are visualised in figure 7.19. The colour of the building represents if the building is made by an architect (green) or engineer (blue), and the yellow squares in the building indicate the placed shear walls. The circle around the building represents the required time to fulfill the local and global or global structural requirements. If the participant did not satisfy the local requirements after 5 minutes the needed time to satisfy the global requirements is displayed. A green circle implies that the global and local structural requirements are satisfied, while a yellow circle indicates that only the global structural requirements are satisfied. The results of the first building are represented on the left and for the second building on the right.

First building case 2; two left columns of figure 7.19

- The global and local structural requirements were satisfied for all generated buildings.
- The time spent by the architects is less than the time spent by the engineers engineers (see table 7.10).
- Four participants placed one shear wall at the left side of the building, either on the bottom or the top row (see figure 7.18a). One participant (engineer) placed two shear walls on top of each other in the middle of the building (see figure 7.18b).



(a) The first design of case by participant number 1, 2, 3 and 5. The red surfaces indicate the chosen shear wall. The chosen shear wall can be found by the yellow surface in figure 7.19.



(b) The first design of case 2 by participant number 4, where 2 shear walls are placed.

Figure 7.18: Shear wall placement options, the red surfaces indicate where the shear walls are placed.

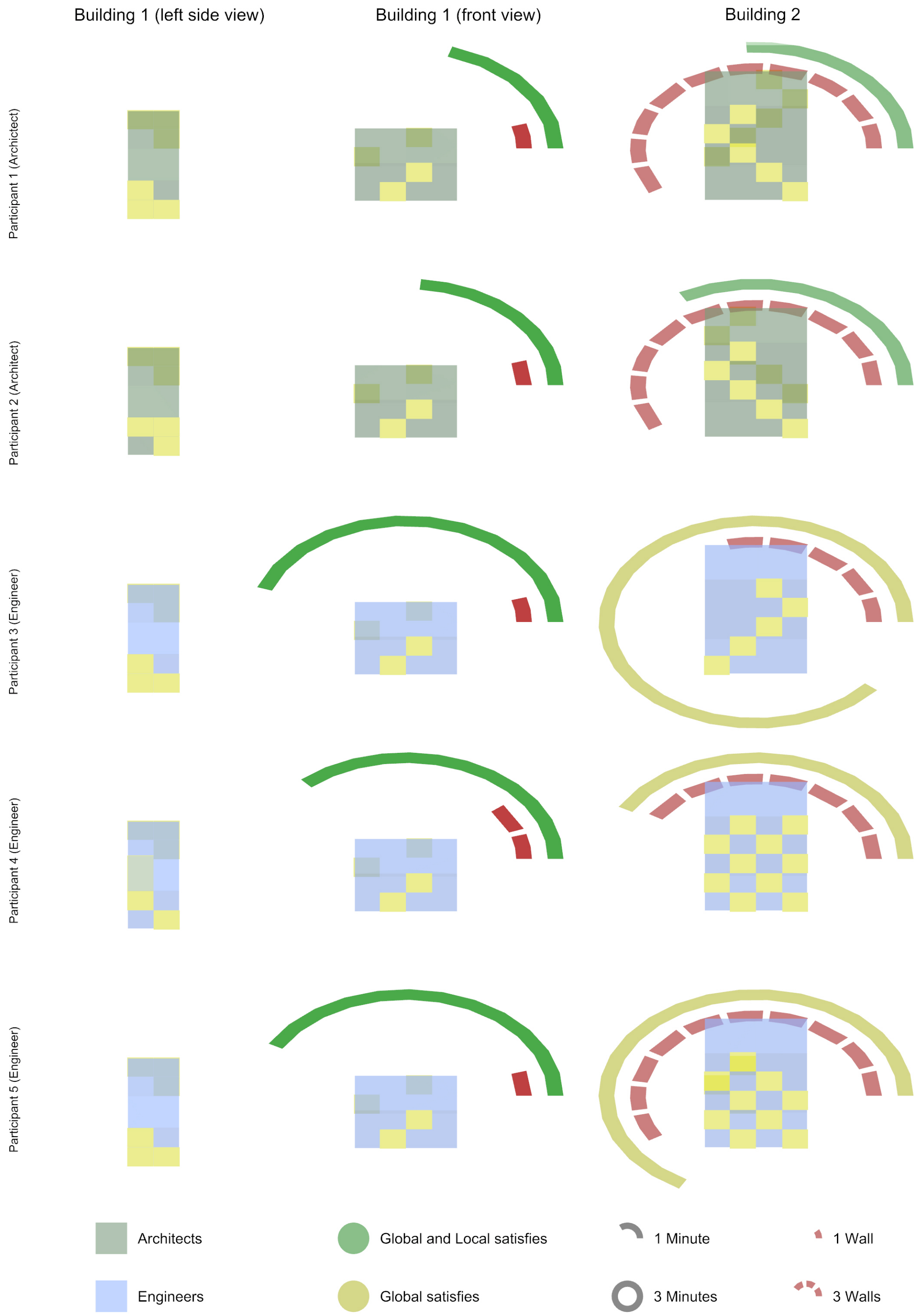


Figure 7.19: Results case 2 of the workshop.

Second building case 2; right column of figure 7.19

- All buildings satisfied the global structural requirements and the two buildings of participant 1 (architect) and of participant 2 (architect) satisfied the global ánd the local structural requirements.

- The slot connection was the governing or failed connection for all participants.
- The time spent by the architects is less than the time spent by the engineers engineers (see table 7.10).
- All buildings satisfied the extra design restrictions.

Results case 2	**Participant 1**		**Participant 2**		**Participant 3**		**Participant 4**		**Participant 5**	
Building	1	2	1	2	1	2	1	2	1	2
Satisfied global structural requirements	yes	yes	yes	yes	yes	yes	yes	yes	yes	yes
Satisfied local structural requirements	yes	yes	yes	yes	yes	no	yes	no	yes	no
Amount of shear walls	1	10	1	10	1	5	2	7	1	10
Time spent to satisfy the local and global structural requirements if satisfied [seconds]	36	47	42	60	80	160	66	75	75	120
Time spent in total [seconds]	36	47	42	60	80	300	66	300	75	300

Table 7.10: Overview of the results of the buildings of case 2. For the sixth row holds: when the local structural requirement is not satisfied, the time to satisfy the global structural requirements is reported.

7.3. Examination automatic placement (Grasshopper)

To examine the automatic placement of the modules by Grasshopper within Habitat21 the calculation time of the automatic placement of one extra shear wall is analysed for 6 buildings. These buildings are stable in the y- and z-direction but unstable in the x-direction. See figure 7.20 below for an overview of the 6 buildings.

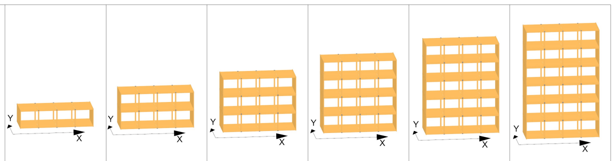
Building						
Amount of modules	4	8	12	16	20	24
Amount of possible positions for placing the CLT shear wall in the x-direction	8	16	24	32	40	48
Calculation time [seconds]	6	23	57	98	175	360

Figure 7.20: Overview of the buildings used for the examination of automatic placement.

Placing one extra vertical CLT shear wall in the x-direction changes the displacement in the x-direction, which could result in obtaining stability in the x-direction. The amount of possible positions for placing the extra vertical CLT shear wall in the x-direction depends on the amount of modules of the building, see figure 7.20.

Important to state here is that for one module the amount of possible *configurations* is $4^2 = 16$, see (see figure 1.13). A configuration considers all possible combinations of vertical CLT shear walls, except the floor and ceiling since these are always included in a module. So for a building with 5 modules the amount of configurations becomes $16^5 = 1048576$. Calculating these configurations in Habitat21 would take a lot of time. To overcome this time issue, the single-step brute force method is created for Habitat21, see appendices F.2 and G.1 for the script and the user interface. Instead of calculating all possible configurations of CLT shear walls within the modules of a building in *one time*, the single step brute force method calculates the placement of CLT shear walls step by step. This means that the CLT shear walls are placed one wall at the time based on the following feedback:

- The number of the possible positions for placing one extra CLT shear wall.
- The updated displacement in the x-direction
- A score between 0 and 1 for placing the extra CLT shear wall at this position compared to placing the extra shear wall in another position (1 is the best position for the CLT shear wall, 0 is the worst position for the CLT shear wall).

Based on the feedback above, the user of Habitat21 decides at which position the extra CLT shear wall needs to be placed. In a possible second round of the iterative process a second CLT shear wall can be placed and so on. The single-step brute force method reduces the amount of calculations for a building with 5 modules from $1048576 (= 16^5)$ to $20 (= 5 * 4)$. However, it should be mentioned that not the full design space is explored, because the next iteration depends on the choice of the user.

The user interface can be found in appendix G.1. The output interface of Habitat21 for the building with four modules is seen in figure 7.22. In this case there is no distinction regarding the score for placing the extra CLT shear wall between the possible positions; every possible position for placing the extra CLT shear wall has a score of 1.

The output interface of Habitat21's single step brute force method for a building with 8 modules and one existing CLT shear wall is also presented to show an example where a distinction between the scores of the possible positions for the extra CLT shear wall is shown. As can be seen in figure 7.21 the third position indicates the highest score (1) and the lowest updated displacement in the x-direction (0.2 cm).

(a) Rendered view

(b) ghosted view

Figure 7.21: The output interface of Habitat21 for the single-step brute force method for a building with 8 modules and one containing shear wall.

(a) Rendered view

(b) ghosted view

Figure 7.22: The output interface of Habitat21 for the single-step brute force method for the building with 4 modules.

As stated above, to examine the performance of the automatic placement by Grasshopper the calculation times of the automatic placement of one extra shear wall is calculated for the 6 buildings in figure 7.20 . As can be seen in the bottom row of figure 7.20 the calculation time for the placement of the extra CLT shear wall at the building with 4 modules and 8 possible positions is 6 seconds and the calculation time for the placement of the extra CLT shear wall at the building with 24 modules and 48 possible positions is 360 seconds. Figure 7.23 indicates the calculation time as a function of the amount of possible positions for the extra CLT shear wall. As can be seen in figure 7.23 the calculation time increases exponentially.

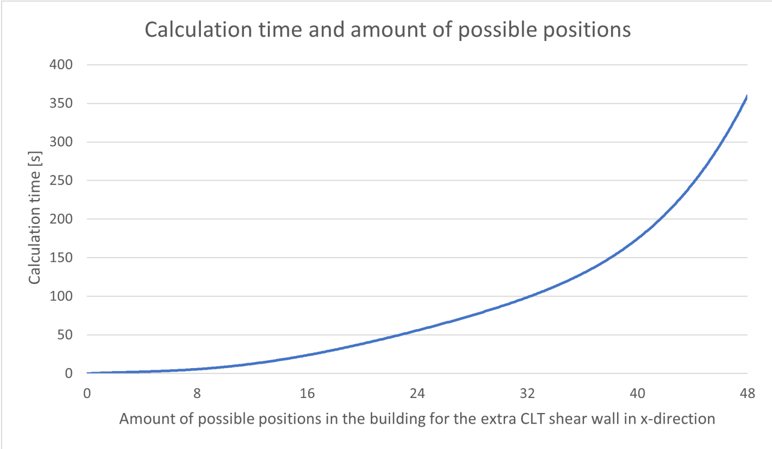


Figure 7.23: Calculations time vs shear wall options for optimisation in one direction

8

Discussion

An examination of the hybrid modular timber construction concept through a verification study, of the manual placement through a workshop and automatic placement of modules through the single-step brute force method is performed in section 7.1, 7.2 and 7. The results in these section are discussed and divided in the following sections: the reliability of the structural result, the hybrid modular timber construction concept, the complexity of the analytical model, the usability of the parametric design tool, the design method and the examination of automatic placement of modules.

8.1. Reliability structural results

The parametric tool (Habitat21) calculated the structural results of the hybrid modular timber construction concept at the case study in section 7.1. This section discusses the reliability of the presented structural results.

8.1.1. Reliability displacement behaviour

The displacement in x- and y-direction of the case study of Gijzen and of the parametric tool (Habitat21) are respectively 41.830 mm and 28.299 mm in the x-direction and 9.187 mm and 9.829 for the y-direction as can be seen in table 7.1. The displacements in y-direction differ but are in order of magnitude in the same range. Due to two reasons the displacements in x-direction are not in the same order of magnitude:

1. The internal stability walls of Hotel Jakarta by Gijzen, which provide stability in the x-direction does not behave as the created continuous shear wall as in the hybrid modular timber construction concept.
2. The discretization of physical to analytical elements. The assumed inter-module connections (connection 1 and 4 of section 4.6.4) of the hybrid modular timber construction concept are modeled as line elements with two hinges ("pendelstaven"), this could influences the displacement behaviour of the shear wall mechanism.

The second reason is more elaborated below:

Discretization of physical connections to analytical connections

The discretization of physical to analytical elements is a subject of debate. Four mechanisms contribute to the displacement of a shear wall: shear, bending, sliding and rocking (see section 4.5). The deformation due to bending and shear of a single CLT wall are verified in section 6.3.9 and are small compared to the displacements due to sliding and rocking (see section 4.8). The inter-module connections have a large influence on the total deformation, which is mentioned by Lacey et al. and verified by stiff connections in the verification study on Hotel Jakarta (see table 7.1).

However, the deformation due to the connections could be under or overestimated. Because the rocking behaviour of the continuous shear walls is restrained, since the vertical connection in section 4.6.4 is modeled as a rod with two hinges with infinite stiffness in vertical direction and no stiffness in horizontal direction. Thereby, the rocking behaviour is underestimated, while the sliding behaviour is overestimated. The actual mechanical behavior is presented in figure 8.1b, where the displacement due to sliding is obtained.

The assumed connections can be modeled as hinges with horizontal and vertical stiffnesses. When the stiffness properties of the assumed connections are determined. In that case all connections are modeled as hinges with different stiffnesses in x-, y- and z-direction. Which lead to more accurate results by having a more complete analytical model. In this manner the rocking and sliding behaviour are taken into account as visualised in figure 8.1a.

(a) Ideal situation: Rocking and sliding [own]

(b) Actual situation: sliding and restrained rocking [own]

8.1.2. Reliability displacements

The reliability of the calculated displacements in table 7.1 and figure 7.12 of the parametric tool (Habitat21) in x- and y-direction are a topic of debate. The parametric tool calculates the displacement based on a predetermined wind load as defined in section 6.4.3 for construction with a stiff foundation (see section **??**). However, both influence the x-, and y-displacement at the top of the building:

The uniform wind-load along the perimeter of the building is assumed as described in section 6.4.3, therefore the wind-load for produced buildings with a slenderness ratio above 5 is underestimated and below 1 is overestimated. The wind-load calculations in appendix E.1 can be included in the tool to obtain an improved result.

The horizontal displacement due to the foundation's rotation are not taken into account. Nevertheless, the rotation of the foundation follows in extra horizontal displacement. The displacement due to the foundation and building can be calculated and tested according to equation 6.16. Or the assumption that half of the total displacements is governed by the rotation of the foundation, as is often the case for high-rise buildings. This implies that the maximum displacement of the building should be calculates as defined in equation 8.1. However, this unity check may be too conservative, because the generated buildings in the workshop do not reach heights over the 20 meter and do not have a single building typology.

$$u_{hor,building,max,assumed} = \frac{1}{1000} * h \tag{8.1}$$

Therefore, it's worth to mention that the unity check for displacement is not complete and could be used for preliminary designs only. The calculated x-, and y-displacements are a indication of the total displacement. Two things could be integrated to have more reliable global results: The wind-load calculations in appendix E.1 can be included in the parametric tool (Habitat21) to obtain an improved result. And the stiff foundation hinges could be modeled as hinges with stiffnesses (springs) to take the extra displacement due to the foundation into account.

8.1.3. Reliability connections

As can be seen in figure 7.6 9 floors could be build with satisfying the unity checks of the connections. The slot connection of section 4.6.4 is determined to be the normative connection. However, the reliability of the unity checks presented figure 7.6 are point of discussion.

Slot connections - The forces through the slot connector may be overestimated. The assumed vertical connection (connection 1 of section 4.6.4) does not transfer any shear forces, since it is modeled as a rod with two hinges. However, the assumed connections in appendix C.1 does transfer shear forces. Which indicates that the amount of shear force through the slot connection may be overestimated.

WHTPT and tension forces assumed connections - Figure 8.1b illustrated that there is no rocking behaviour, due to the modeled assumed connections (connection 1 and 4 of section 4.6.4). Due to the restrained rocking there is a low displacement in the z-direction, which result in lower forces in the adjacent WHTPT connection plates (connection 3 of section 4.6.4) and higher forces in the assumed vertical connection (connection 1 of section 4.6.4). Therefore, the unity checks of the WHTPT plates in figure may be underestimated and magnitude of forces for the assumed vertical connections overestimated.

8.1.4. Reliability unity checks glulam

The unity check for buckling of the glulam columns is presented in figure 7.5. The glulam column with a dimension of 100x100 mm can be used for up to 10 floors. However, the unity check for buckling without fire safety is taken into account. As described in section 4.1, depends the required resistance of the main load bearing structure on the fire load. and is the resistance to fire expressed in minutes that is satisfy the ultimate limit state of the structure. The required amount of minutes can be found in table 4.1. The fire resistance of timber components can be increased by larger cross-sections or an extra protective layer. Depending on the precautions, the glulam cross-section can increase in size. Therefore, the calculated unity check for the glulam cross-section by the parametric tool (Habitat21) is an indication and should be determined in a later design stage.

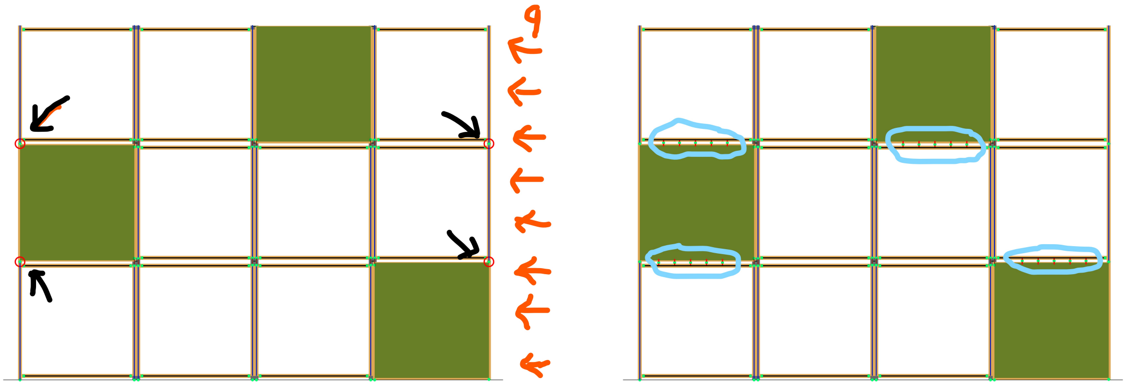
8.1.5. Reliability unity checks CLT

The maximum buckling and tension stress values for the CLT components are provided in section 3.4.3. Due to software restrictions the unity checks are a work in progress. Also unity check regarding the CLT slab are not calculated. As stated in 4.3 simply supported CLT slabs are still a topic of research.

8.2. Hybrid modular timber construction concept

The structural engineers were not able to satisfy the second building of the second case of the workshop (see section 7.2.4), the unity check for the slot connection appeared to be the governing connection in the buildings that failed the local structural requirements. This can be explained as follows:

The horizontal forces through the building are transferred by floor diaphragms and shear walls to the foundation. When the shear walls are not placed on top of another, the forces are transferred through the perpendicular shear wall to the floor below. Thereby, the path of forces is long and magnitude of forces through the connections in the perpendicular-wall connection is high (see concentrated load at highlighted corners with black arrows in figure 8.2a). A shear connection between the floor diaphragm and ceiling diaphragm (see red elements with green dot indicated by blue circles in figure 8.2b) reduces the force path distance and could broaden the design opportunities of hybrid modular timber construction from structural perspective.



(a) Actual hybrid modular construction concept; the wind-force (red arrows) is transferred through the red indicated points.

(b) New hybrid modular construction concept with shear connections between floor and ceiling indicated in the blue circles.

Figure 8.2: Actual and new hybrid modular construction concept with possible new inter-module connection.

8.3. Complexity analytical model Karamba

An examination of the differences between RFEM and Karamba is performed in section 5.3. The use of Karamba and Rhinoceros is preferred based on the calculation speed, ease of generating a prefabricated modular construction concept and visual feedback. In section 6.1 the model used in Karamba has been constructed. However, by utilizing Karamba, the complexity of the analytical model is reduced. The indicated barriers during this research enhanced:

- Bi-linear joint stiffness
- Non-linear joint stiffness
- Peak stresses
- Stress distribution
- Cross-laminated timber

These topics are elaborated below:

Bi-linear joint stiffness - Karamba only allows for linear connections stiffnesses. Therefore, through an iterative process, a connection with bi-linear stiffness is calculated (see appendix F.1).

The use of this script is still in early development, since it produces an error in the stress distribution for meshes.

Non-linear joint stiffness - The connection and screw stiffnessess are based on reference projects or section 7,1 the Eurocode. But as discussed in section 4.6.1, the linear calculation method of section 7,1 of the Eurocode roughly estimates the real slip behaviour of the fastener connection. The real slip behaviour as illustrated in figure 4.17, shows non-linear behaviour. The accuracy of the model can be improved by acquiring more exact stiffness values through testing instead of applying the simplified formulas (equations in table 4.3). Additionally, these non-linear stiffness's could be used in the analytical model. However, non-linear joints can not be created in Karamba.

Peak stresses - The compression forces between two meshes (CLT-panels) are transferred through line elements, which results in peak-stresses. In an ideal situation the forces are transferred to an adjacent mesh by a line joint, as such distributing the compression forces more equally. However, the line-joint is still in development by Karamba. Another option would be to add more line joints, which spread the compression forces equally over the adjacent mesh.

Stress distribution - Due to the iterative calculation process performed in appendix F.1, the stress distributions in the mesh elements are not determined. The visualisation of the stress distribution in meshes needs extra attention in future developments. A possible method to fulfill this goal is by giving the Karamba C# script in appendix F.1 extra attention.

Cross-laminated timber - It is possible to use the stiffnesses matrix of CLT in more advanced programs as suggested by Bergström and Fröbel and Wallner-Novak et al.. But Karamba only accounts for isotropic and orthotropic properties in one direction, therefore fictitious elastic modulus are derived for the axial and bending stiffness of CLT, which is elaborated on in section 6.3. The CLT elements are discretizised as a massive beam with orthotropic material properties. The in-plane stiffness is overestimated in the strong direction, and underestimated in the weak direction for the floor and ceiling elements by using the out-of-plane properties. The in-plane properties are used for the walls and are verified in section 6.3.9.

8.4. Usability of the parametric tool

As illustrated in figure 7.17 the participants made during the first case of the workshop 15 building typologies. The average typology score of the architects is 50.3 and of the engineers 44.9. However, considering only the buildings that satisfied the global structural requirements the average respectively became 30.3 for the architects and 27.3 for the engineers (see table 7.9). These scores could indicate multiple things:

- There was not enough time, to satisfy the global structural requirements.
- Due to lack of feedback or information about the structural system the global structural requirements were not met.
- The boundaries of the modular timber construction concept were met for buildings with a high typology score.

These points are elaborated below:

Time - Due to time, because the participants were interrupted after 8 minutes during every design during the first case. With more time they could have satisfied the global structural requirements. The third case showed that, when the participants had sufficient time, they were able to satisfy the global requirements.

Feedback and information - As illustrated in figure 7.14, did two buildings not satisfy the global structural requirements, due to missing shear walls no connections were created on various essential corners of the modules. This could be caused by a lack of feedback or information about the structural system provided at the tutorial. The parametric tool provided information with regards to the stability and displacement, however, information about where connections are or are not not created could help the user. Or this could be due to a lack of provided information about the hybrid modular construction concept at the tutorial. A more elaborated tutorial could help the user with placement of modules and placement of shear walls.

Boundaries of modular timber construction concept - As illustrated in figure 7.13 the displacements of the created buildings were to large in the x-, y- or z-direction. This could indicate that the boundaries of the hybrid modular timber construction concept were met.

8.4.1. Second case workshop

Just 2 out of the 5 designs at case 2 fulfilled the local requirements. Furthermore, the architects fulfilled the local requirements by accident. The following reasons can be the cause why the engineers did not fulfill the local requirement at the second case:

- Due to the lack of feedback and understanding of the hybrid modular timber construction concept.

Feedback and understanding - Due to the lack of feedback and understanding of the hybrid modular timber construction concept. The distribution of forces is not visualized with the parametric tool (Habitat21), only the locations of the governing unity checks. Therefore, a stress distribution could be the missing feedback to provide a better understanding of the structural system. By implementing a stress distribution, the path of forces could become more clear for the user. In figure 8.3 a preview can be seen of the possible visual feedback regarding the stress distribution. On top of that, information about the force distribution through the connections is missing. A zoom option for the components and connections, could provide the user with more detailed information about the connections and components on how the forces are transferred through a connection or component and could help the user in certain design situations.

8.5. Design method

The results of the first case of the workshop (see figure 7.17) illustrated that architects and engineers made various building typologies. However, the applied design method can be a subject of discussion. The used design method by the participants could be described as a trial and error design method. And therefore the parametric tool as a serious game, in essence a game where behavioral change is put central. The highlighted matter of debates regarding this topic are:

- Product and degree of acceptation
- Engineer and degree of acceptation
- Architect and degree of acceptation

These points are elaborated below:

Product degree of acceptation - The parametric tool can be used as a product in combination with a prefabricated building system. Beside the structural information it is possible to implement additional information regarding the sustainability, costs and duration of the building project. This information can be used as a second opinion on existing projects or provide information in the preliminary design phase. And with serious gaming it is possible to create more awareness or expand the knowledge about a specific subject. The structural engineer parametrizes a structural system with a set of connections, 1D, 2D and 3D components, while architects, constructors, project developers and structural engineers can use the construction system in combination with a tool to discover the possibilities. This design method may broaden the awareness and expand the knowledge, but the degree of acceptation within the conservative construction industry can be a matter of debate. The reason for this is that the design method does not match how contractors, architectural and engineering companies are working.

Engineer and degree of acceptation - The engineers were enthusiastic during the workshop, however the workshop revealed that the degree of acceptation among the engineers can be low in early stages, since they were sceptic about the outcomes. Detailed information about where connections are and are not created was lacking and they did not know what to do to satisfy the local requirements at the second building of the second workshop (see figure 7.19). Therefore, based on the results of the second building of the second case of the workshop, the tool can be described as a black box. Through time, realisation of existing projects or parametric tool development it may be possible to obtain a higher degree of acceptance.

Architect and degree of acceptation - The architects already had affinity with modular construction and Rhinoceros and satisfied the all the global requirements at the second case of the workshop. And it is remarkable that the time spent by the architects is less than the time spent by the engineers to satisfy the global or local requirements during the second case of the workshop (see table 7.10). This could be due to the experience of the architects with Rhinoceros and modular buildings as clarified in table 7.5). It would be interesting to determine if architects without any affinity with modular construction will prefer such a method above their own design method and can satisfy the global and local structural requirements. To give an answer to this question could more architects be asked to use the parametric tool and fulfill the cases of the workshop. However, the architects were interested in a tool which gives structural feedback with regards to the stability of a building without the inclusion of a structural engineer.

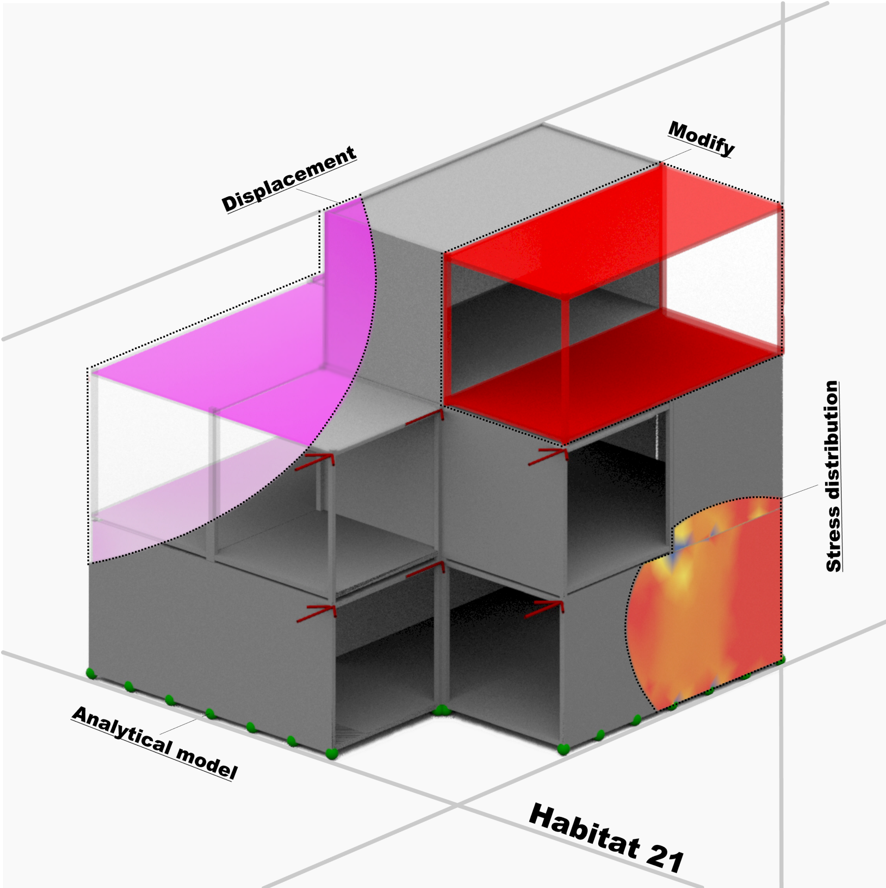


Figure 8.3: Visual representation of feedback parametric tool Habitat21

8.6. Examination automatic placement

Smart feedback - A single-brute force method as described in section 7.3 could be used to provide feedback to the user through created script. However, the single-brute force method requires a significant increase in calculation time for larger buildings. The parametric tool should be optimized to reduce the calculation speed of one building configuration. Or smarter algorithms should be used to reduce the calculated design space.

9 Conclusions

This chapter provides an answer on the main question based on the results, see chapter 7, and the discussion, see the previous chapter. The main question, stated in 2.2, is repeated below:

To what extent is hybrid modular timber construction structurally feasible with a coreless lateral stability system while exploiting the options of a parametric design tool?

Both for the hybrid modular timer construction and for the parametric tool is examined which aspects are feasible or unclear. First, all sub-question of section 2.3 are answered.

9.1. Sub questions

To answer the sub-questions a parametric tool in combination with the hybrid modular timber construction concept has been developed. The following sub-questions regarding the tool and the concept are answered to give an answer to the main question.

1. **How is a hybrid modular timber construction designed within a parametric design tool?**

   Multiple prefabricated projects in combination with parametric tools in the literature research in chapter 5 showed already that it is possible to design modular buildings. However, most of these tools' scope was limited by focusing only on efficiency and mass production, which will decrease the customisation in building designs. Furthermore, a structural analysis was not included or performed in secondary software. Therefore, a parametric tool with integrated structural analyses is developed in this thesis. The parametric tool enhances a plugin with user interface for Rhinoceros in combination with structural analysis software Karamba. This method appeared to be more suitable compared to Revit and RFEM, considering the structural calculation speed and discretization of modular construction elements (see section 5.3).

2. **Are architects and engineers able to create hybrid modular timber constructions with a parametric design tool?**

   Yes, the architects and the engineers created 10 out 15 buildings that satisfied the global structural structural requirements during the first workshop in section 7.2.3. The architects satisfied 4 buildings and the engineers 6 buildings (see table 7.7). Four of the five remaining buildings of the first case of the workshop did not satisfy the global structural requirements due to time, lack of feedback or information about the structural system or boundaries of the hybrid modular construction concept as discussed in section 8.4. The last building satisfied the unity checks in the first place, however the displacement in x-, y-, and z-direction were not reliable due to an error in the Grasshopper script.

In the second workshop (see section 7.2.4), the architects and the engineers satisfied the global requirements of both buildings. However, the local requirements of the engineers were not satisfied for the second building of case 2 of the workshop. This could be due to the lack of feedback and understanding of the hybrid modular construction concept as discussed in section 8.4.1.

3. **Do architects and engineers create different building typologies for a hybrid modular timber construction when using a parametric design tool?**

   To explore the diversity of the typologies from the fifteen buildings of the first case in the workshop (section 7.2.3) a typology score was given for each building. The highest score was 102 points and the lowest score was 10. This indicates a relatively large diversity in typologies.

9.2. Main question

Based on the discussion and answers to the sub-questions above, the answer to the main question is divided for the hybrid modular construction concept and parametric tool into: feasible or unclear and recommendations.

1. **Hybrid modular Timber construction concept**

   Feasible - The verification study in section 7.1 showed that hybrid modular construction can provide horizontal stability in the right order of magnitude. Furthermore, during the first case of the workshop were 10 out of 15 different building typologies created with the created hybrid modular construction concept that satisfied the global structural requirements (see figure 7.17). And as can be seen in table 7.1 the inter-module connections have a significant influence on the total displacement.

   Unclear and recommendations - It is unclear what the exact boundaries of the hybrid modular timber construction concept are. For the reason that the concept has been worked out to a certain extent and there are still topics that need to be investigated. As discussed in chapter 8 the reliability of the structural results could be improved by:

   - Determining the stiffness properties of the assumed inter-module connections (connection 1 and 4 of section 4.6.4). (see section 8.1.1 and 8.1.3)
   - Including an exact wind-calculation model and model the stiff foundation hinges as spring elements. (see section 8.1.2)
   - Taking fire safety into account. (see section 8.1.4)
   - Tool development to determine the occurring stresses in CLT and research into point supported CLT slabs. (see section 8.1.5)

Furthermore, the path of forces is long and the magnitude of forces through the perpendicular-wall connection is high for the building displayed in figure 8.2a. More research into a new connection between the floor and ceiling as discussed in section 8.2 could reduce the force path and broaden the design opportunities from structural perspective. However, to what extent is unknown. And by utilizing Karamba, the complexity of the analytical model is reduced as discussed in section 8.3.

However, not all generated buildings during the first case of the workshop (see section 7.2) satisfied the global structural requirements. As discussed in section 8.4 the typology score could indicate that the boundaries of the modular construction concept were met for buildings with a high typology score.

2. **Parametric tool**

   Feasible - It is possible to design hybrid modular timber construction with a parametric design tool as answered in sub-question 1 of section 9.1. And yes, architects can create different hybrid modular timber constructions with the parametric design tool as answered in sub-question 2 of section 9.1. And on top of that, create architects and engineers different building typologies for a hybrid modular timber construction when using a parametric design tool as answered in sub-question 1 of section 9.1.

   The single-step brute force method, created for this thesis, in section 7.3 showed the automatic placement and modification of modules by a script.

   Unclear and recommendations - The architects and engineers were not able to satisfy all global and local structural requirements as answered in sub-question 2 in section 9.1. Multiple recommendations on tool developments are made in section 8.4 to improve the usability of the parametric tool.

   Moreover, the parametric tool could be seen as a serious game. But the degree of acceptation of this design method as explained in section 8.5 could be low:

   - Since it does not match the conservative way of working in the construction industry.
   - Since the parametric design tool can be described as a black box.
   - Since it is not known if architects without any affinity with hybrid modular timber construction will prefer such a method above their own design method.

   As can be seen in figure 7.23 the single-brute force method requires a significant increase in calculation time to provide feedback to an user for larger buildings. Therefore, it is recommended to reduce the calculation speed of the parametric tool (Habitat21) or develop smarter algorithms.

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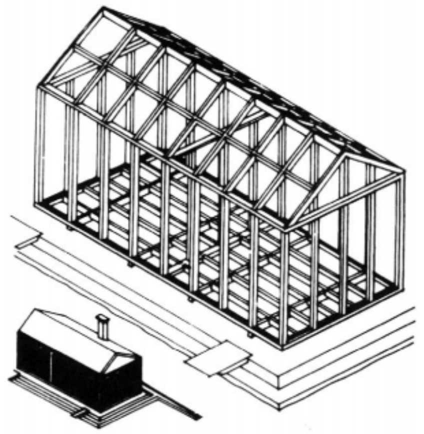
A

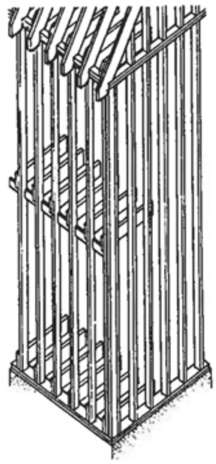
Modular construction

A.1. Small history prefabricated building system

1600/1700 Prefabrication is often referred to as an off-site manufacturing method and can be traced back to western cultures in the sixteenth and seventeenth century. Settlements of British colonization required a fast building method. Timber components were manufactured in England and shipped by boat to the different colonies worldwide.



(a) Manning portable cottage developed by Manning: a quickly deploy able solution, which could be assembled in one day, to rapidly expanding the British colonies [68]



(b) Example of balloon framing system with timber studs. [71]

Figure A.1: Prefabrication in the west

1800 The timber architecture in England is seen as the introduction of the timber-balloon frame construction method in America. The wooden skeleton was referred to as balloon frame since it seemed as if a strong wind-flaw could blow the house away. The balloon frame resulted from two available factors: a plentiful supply of wood and mass-produced iron nails. [68] [34] George W. Snow created the balloon frame in 1832. Compared to timber framing construction, where posts and beams are used, timber studs with close spacing were applied in balloon framing (see Figure A.1b). [71]

1800 In the eighteenth century the use of iron manufacturing for building construction started. The development of systems based on identical prefabricated elements resulted from the cast and rolled iron products readily available as semi-finished products in the factory. One of the most outstanding prefabricated iron buildings in the 19th century is the crystal palace (1851). Gardener Joseph Paxton, with engineers Fox and Henderson, designed a repetitive system with a minimum number of standardized elements which formed a skeleton (see Figure A.2). [71]

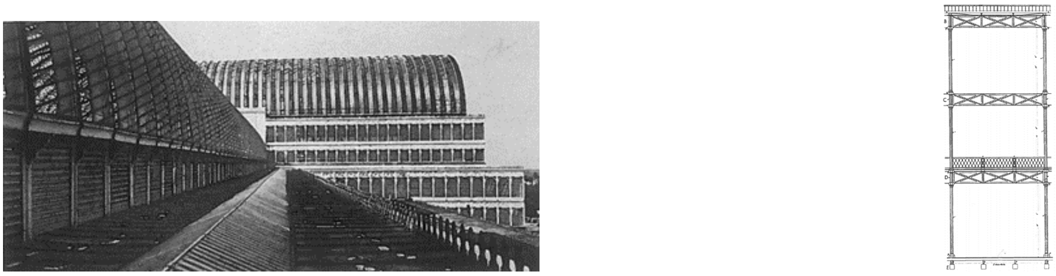
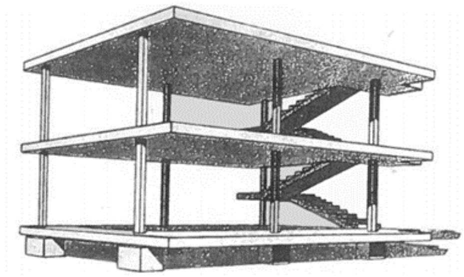


Figure A.2: Crystal Palace and the iron frame structure were only two different forms of columns throughout the whole building. [71]

1900 Around the 20th century, Joseph Monier experimented with wires in flowerpots and developed the first reinforced concrete. This new building material was suitable for monolithic construction and offered excellent stability, which resulted in the first modular concrete unit by Francois Hennebique. (see Figure A.3a) [71] [40]

(a) A concrete modular unit, by Francois Hennebique, 1896 [40]



(b) Dom-ino standardized housing project, a system based on concrete columns and flat slabs. [71]

Figure A.3: Standardisation

The mass production in the automotive industry, realized by Henry Ford in 1913, has also influence the construction industry. Producing houses in the same way as cars became a possibility. Le Corbusier developed the Dom-ino House project based on standardization (see figure A.3b). Prefabricated building components were arranged to form houses.

1920 Richard Buckminster Fuller was fascinated by the transportation of prefabricated buildings. He became one of the driving forces behind prefabricated houses in 1920, with the Dymaxion House. A hexagonal construction suspended from a mast, made with steel and aluminium to resist tension forces (see figure A.4). A maximum amount of surface area was created with minimum a use of material. But due to technical limitations, the idea could not be fully applied in the building industry. [71]

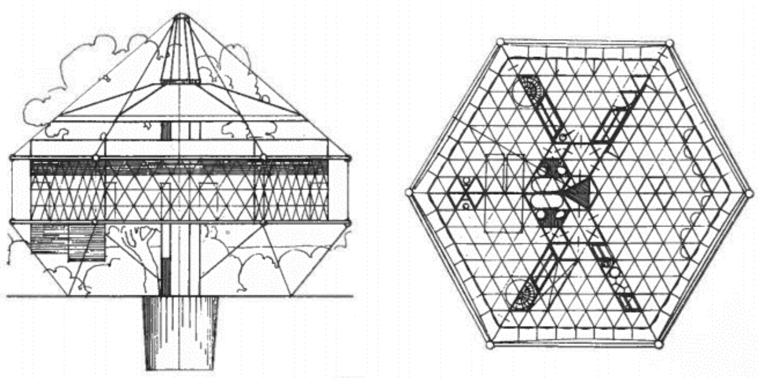


Figure A.4: Dymaxion house project by Buckminster Fuller [71]

1960 Another remarkable project is the Metastadt project, by Richard Dietrich at the end of 1960s (see figure A.5). Dietrich developed a building system for flexible multi-storey buildings, in which all prefabricated two-dimensional elements components could vary. It was an attempt to offer new urban solutions. A range of new technical construction instruments were invented and developed to realize this project. The structure was intended to provide the residents with multiple options, but making structural alterations appeared to be too complicated. The building was demolished 13 years after realization, due to constructional deficiencies and vacancy.

Figure A.5: Metastadt housing project by Richard Dietrich. [71]

A.2. Reference projects modular construction

In table A.1 the main properties of the observed prefabricated reference projects are summarized.

Project	**Function**	**System**	**Material**	**Floors [lvls]**	**Completed [year]**	**Duration [months]**
Hotel Jakarta	Hotel	Modular (3D) Concrete core	Timber Concrete	8	2018	3
Puukuoka	Residential	Modular (3D) Prefab (2D)	Timber	8	2014	6
Treet	Residential	Modular (3D) Prefab (1D)	Timber	14 (4)	2015	18 incl. groundwork
Brock Commons	Residential	Prefab (2D) Prefab (1D) Concrete Core	Timber Concrete	18	2017	2.5
Residential complex Zurich	Residential	Modular (3D) Prefab (2D) Concrete Core	Timber Concrete	6	2016	6
Woodie Student Hostel	Hotel	Modular (3D) Concrete Core	Timber Concrete	6	2017	10
50 Modular Timber Appartments	Residential	Modular (3D) Concrete Core	Timber Concrete	4	2015	9
Habitat 67	Residential	Modular (3D) Concrete Core	Concrete	12	1967	30

Table A.1: Overview reference prefabricated building projects

A.2.1. Hotel Jakarta

[1] At Java island in Amsterdam is Hotel Jakarta located. A hotel which residents 200 hotel rooms, restaurants, and a garage. A part of the hotel rooms in Hotel Jakarta is built with the modular system of Ursem. It took two and a half months to manufacture 176 timber-concrete modules and just 13 days of on-site work to place all the modules.

Location	Amsterdam, Netherlands
Year	2018
Height and Stories	32 m / 12 storeys (8 modular storeys)
Type	Hotel


(a) Hotel Jakarta, Amsterdam [25]


(b) Modular construction Hotel Jakarta [44]


(c) Modular unit Ursem [44]

Figure A.6: Hotel Jakarta

System
A hybrid structure of concrete and timber is used for the Hotel Jakarta. Concrete is used for the foundation, garage, and the ground and first floor. The first concrete floors give a lot of design flexibility to the architect and create a stable plateau for the stacked modules. A maximum of 8 modules is stacked on top of the table. A reinforced concrete elevator shaft gives the building its required lateral stability. The plateau stabilises with prestressed reinforced concrete columns. The structural integrity of the modules is gain by the placement of strips perpendicular to the modules. Fire safety requirements are reduced to 90 minutes by introducing a sprinkler system.

Modular system
Ursem modular components (Ursem Crosscon) contain a thick concrete slab (150 mm) with cross-laminated walls (140 mm) and a cross-laminated ceiling (90 mm) with an additional acoustic barrier. A perpendicular CLT wall guarantees the required stability of the module. The load-bearing CLT walls limit the design flexibility, which results in the inability to construct large open areas with this system. The maximum number of staking is limited to eight levels.

Manufacturer	Ursem
Maximum dimension	12500 x 4000 x ?? mm
Maximum stacking	8
Main support structure	Concrete floor / CLT load-bearing walls
Timber properties	XLAM - Derix Gelijmde Houtconstructies

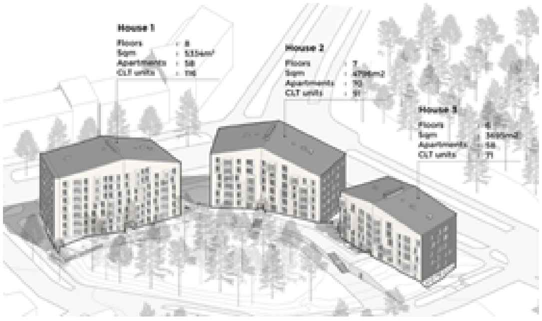
Table A.2: Overview Ursem Crosscon modular system

[1]Information Hotel Jakarta [25] [44] [78] [79]

A.2.2. Puukuoka

[2] The opportunities of cross-laminated timber are explored with the multi-residential buildings in Jyväskylä. Three residential buildings with respectively eight, seven and six stories rise at the horizon in Jyväskylä. The buildings contain 58 prefabricated apartments, based on the Urban multi-storey concept of Stora Enso. The use of the prefabricated modules made it possible to cut the construction time down to six months.

Location	Jyväskylä, Finland
Year	2014 (highest apartment ready)
Storey's	8 stories
Type	Residential



(a) Puukuoka project 3 modular buildings [10]

(b) Modular construction Puukuoka [10]

Figure A.7: Puukuoka

System

The foundation and first floor are made of concrete and serve as a garage and public spaces. The primary structure and frame are made of cross-laminated timber and laminated veneer lumber. The timber structure provides lateral stability as well as resists the gravity loads. Modular units, prefabricated façade elements and roof elements are all used to construct the three residential buildings. The facade elements close the building envelope and are mounted on-site. Some walls are covered with gypsum board for fire safety regulations.

Modular system

The urban multi-storey concept of Stora Enso is used for the apartments. The modular units' building process consists of three phases: manufacturing, assembly, and installation of additional equipment. Elements are manufactured and assembled in the factory to make a modular 3d unit. The second phase consists of the modular units' placement on-site, and edges and openings are covered in this stage. In the third phase are secondary elements installed along with HVAC installations. The multi-storey concept design has fewer limitations because of different vertical and horizontal components and bracing types. Larger open areas are possible in this case.

[2] Information Puukuoka [10] [76]

Manufacturer	Stora Enso
Maximum dimension	Transport limitations
Maximum stacking	8
Main support structure	Massive timber construction
Timber properties	CLT or LVL

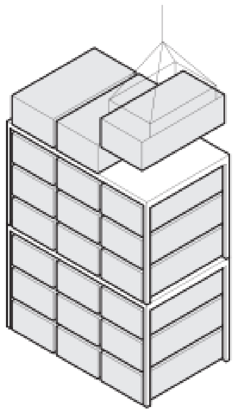
Table A.3: Overview Stora Enso modular system

A.2.3. Treet

[3] The Treet in Bergen became the tallest timber building in the world when finished. A load carrying superstructure in combination with prefabricated timber modules holds 62 apartments. The modular units are manufactured by the wood company Kodumaja and comprise the apartments. Modular construction reduced the construction time, include groundworks took the construction time a total of 18 months. Information about the placement of modules is missing.

Location	Bergen, Norway
Year	2015
Height and Stories	52.8 m and 14 stories
Type	Residential

(a) Treet, Bergen [17]



(b) Combined modular construction system [40]

(c) Kodumaja modular units [6]

Figure A.8: Puukuoka

System

The primary structural system consists of glulam perimeter trusses for lateral stability as well as the gravity loads. Due to the low self-weight of the building do tension forces occur in the foundation. These forces transfer through the glulam beams to the concrete foundation. Modular units comprise the main volume of the building and are stacked up to a maximum of four. The CLT walls inside the building do not contribute to the lateral stability of the structure. The stacked modular units rest on a reinforced deck with extra glulam perimeter trusses and carry the modular units above.

[3]Information Treet [17] [55] [40] [6]

Modular system

The timber modules are produced and delivered by the company Kodumaja. The manufacturer completes up to 95% of the interior work and most of the factory's exterior work. The modules consist of a timber frame which is the load-bearing structure of the module. Due to transportation and hoisting is a more robust structure required for the prefab modules. Certain limitations restrict the architectural design; the designer must think in terms of modules when designing a building.

Manufacturer	Kodumaja
Maximum dimension	5300 x 14500 x 4500 (w x l x h)
Maximum stacking	4
Main support structure	Timber frame
Timber properties	Certified dried timber

Table A.4: Overview Kodumaya modular system

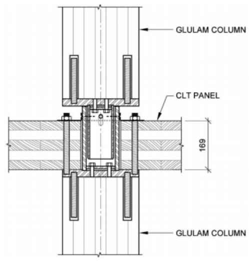
A.2.4. Brock Commons

[4] A prefabricated tall timber building located near Vancouver rises in 2017. When all prefab components arrived on site, is the Brock Commons in less than 70 days constructed, approximately four months faster than a conventional building method. The 18 storeys high residential building has a capacity for 400 students and acts as an academic and a recreational hub for the students of the University of British Columbia.

Location	Vancouver, Canada
Year	2017
Height and stories	54 m and 18 stories
Type	Residential

(a) Brock Commons, Vancouver [73]

(b) Schematic structural render [73]



(c) Detail connection CLT floor - Glulam column [73]

Figure A.9: Brock Commons

System
The structure is a hybrid mass timber structure. The foundation and ground floor are made of concrete, which forms a stable podium for the mass timber construction above. The 17 stories above are comprised of mass timber construction. Two reinforced concrete cores stabilise the mass timber construction against lateral loads. Timber-based lateral force systems, such as CLT walls/cores, were feasible design options, but testing, costs, and time which would have extended the completion date and increased the clients budget were barriers. The mass timber construction consists of glulam columns with steel connectors and five-layer CLT panels for the floor structure. By spaning the CLT panels in two directions, the design team was able to eliminate beams. Eight meters wide by almost three meters high prefabricated façade elements closes the building envelope on-site.

Prefab system
Instead of modular construction is a prefab system used, which holds a specific building sequence. The first involved erecting all columns on one level. The second phase consists of the installation of CLT panels. The third phase consists of installing steel plates at the concrete cores and perimeter to support the façade system. The fourth phase was the installation of the façade elements. The mass timber construction with columns (Glulam) and cross-laminated floor panels (CLT) in a grid of 6 meters by six meters creates a lot of design flexibility. The concrete cores are necessary for enough stiffness, which limits the design. The building envelope is closed by prefabricated façade elements.

[4]Information Brock Commons [73] [74] [29]

Brock common Mass timber prefab	
Maximum dimension	Limitation depend on CLT panels
Maximum stacking	-
Main support structure	Mass timber and reinforced concrete
Timber properties	CLT, Glulam, PSL

A.2.5. Residential complex Zurich by Rolf Mühlethaler

[5] Three residential buildings with a height of 20 meters, width of 18 meters and respectively 70, 90 and 100 meters in length are constructed at the Freilager Zurich site. The architects ground plan typology with its orthogonal planning and repetition of elements suited timber construction. The construction time for three buildings was six months.

Location	Zurich, Zwitserland
Year	2016
Height and stories	20 m and 6 stories
Type	Residential

Figure A.10: Residential complex Zurich by Rolf Mühlethaler [7]

system
A prefab construction method is used for the three cellular residential buildings. The system can be described as a hybrid structure with a reinforced concrete core to provide lateral stability and resist gravity loads and timber panels and columns for the extra gravity loads.

Modular or prefab system
Construction company Renggli delivered the prefabricated components. A variety of possibilities are possible and depend on project-specific cases. Prefabricated two-dimensional elements can be combined with three-dimensional modular units. Timber framework or CLT panels are the possibilities in off-site timber manufacturing.

[5]Information Residential complex Zurich [7] [11]

Manufacturer	Renggli
Maximum dimension	Prefab/modular transportation limits
Maximum stacking	6 (project-specific)
Main support structure	Project-specific, timber
Timber properties	CLT, Glulam

A.2.6. 50 Modular Timber Apartments / Residential complex Toulouse

[6] Fifty modular timber apartments have been build on the west portion of the ADOMA site in Toulouse. It took nine months to construct the building with all exterior and interior finishes. The timber modules were mounted following the concrete core's completion and in module groups across all floors within ten days. The modular units are offset and turned in different directions to compromise between privacy and sunlight, resulting in a compact, stacked construction.

Location	Toulouse, France
Year	2015
Stories	4 stories
Type	Residential

(a) 50 apartments, Toulouse [40]

(b) Construction, concrete core surrounded by CLT modules [12]

(c) CLT modules from Pyrenees charpentes [4]

Figure A.11: Residential complex Toulouse

System
The four high construction is stabilised by a reinforced concrete core, which functions as an elevator and staircase shaft. The CLT modular units contain load-bearing walls and are self-supported. Thermal insulation was mounted in timber-frame panels and applied on site, the prefab aluminium façade sheets close the building perimeter and protect the modular units from weather conditions.

Modular system
The modular units are engineered by the company Prenees Charpentes. The system is not generally used; the dimensions and properties are project-specific for this modular system. The CLT modular units are linked to each other by steel panels with welded pegs. Large open areas are not observed in this building due to the modular units' load-bearing CLT walls.

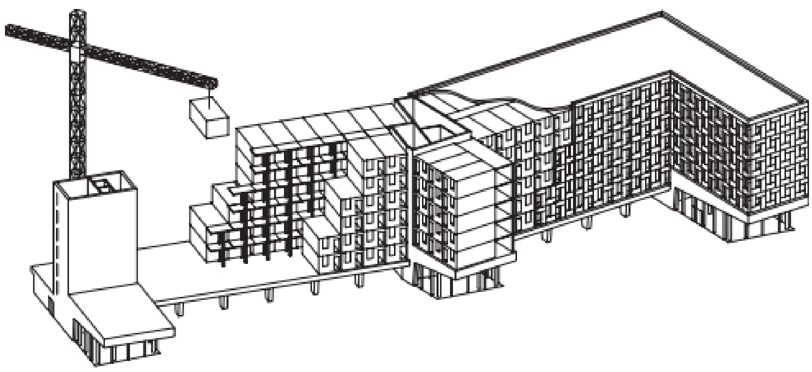
[6] Information Residential complex Toulouse [40] [12] [4]

Manufacturer	Pyrenees charpentes
Maximum dimension	3.5 m (wide) x 6.55-7.275 m (length) x ??
Maximum stacking	4 (project-specific)
Main support structure	Concrete core + CLT Modules
Timber properties	CLT

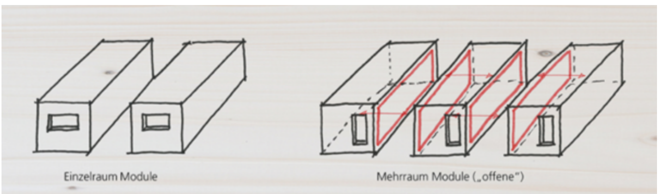
A.2.7. Woodie student hostel Hamburg

[7] Prefabricated stacked modular units on top of a concrete plateau form the 370 apartments in the woody student hostel in Hamburg. It is the largest residential building made of modular timber units (Room Modules). The corridor areas contain concrete elements to full fill fire safety requirements. A maximum of twelve modules a day could be installed. The total construction time enhances ten months.

Location	Hamburg, Germany
Year	2017
Stories	6 stories
Type	Residential



(a) Schematic overview Woodie Student hostel Hamburg [40]



(b) Schematic overview modular system Kaufmann Bausysteme [13]

(c) CLT modules from Kaufmann Bausysteme [13]

Figure A.12: Woodie Student Hostel Hamburg

System

A Reinforced concrete table structure and core fulfil the lateral stability of the design. Modular CLT units resist the gravity loads and are fireproof dimensioned for 90 minutes. The prefab timber façade panels are mounted on site.

[7]Information Student Hostel Hamburg [40] [13]

Modular system

Kaufmann Bausysteme manufactures the modular system. The system contains ”Einzelraum” or ”Mehrraum” modules to make small or large open spaces (See figure A.12b). The load-bearing walls are made of CLT.

Manufacturer	Kaufmann Bausysteme
Maximum dimension	-
Maximum stacking	6 (project-specific)
Main support structure	Concrete core + CLT modules (project-specific)
Timber properties	CLT

B

Cross-laminated timber

This appendix provides information about:

- buckling factor CLT (B.1)
- Calculation buckling factor CLT (B.2)
- Vibrations CLT (B.3)
- Verification CLT Karamba (B.4)

B.1. Buckling factor CLT

Calculation of the buckling factor with verification for CLT elements.

$$\frac{\sigma_{c,x,d}}{k_{c,y} * f_{c,0,xlay,d}} + \frac{\sigma_{m,y,d}}{f_{m,xlay,d}} \leq 1 \tag{B.1}$$

$$\frac{N_d}{k_{c,y} * A_{x,net} * f_{c,0,xlay,d}} + \frac{M_{y,d}}{W_{x,net} * f_{m,xlay,d}} \leq 1 \tag{B.2}$$

$$k_{c,y} = \frac{1}{k_y \sqrt{k_y^2 - \lambda_{rel,y}^2}} \leq 1 \tag{B.3}$$

$$k_y = 0.5 \left(1 + 0.1 \left(\lambda_{rel,y} - 0.3\right) + \lambda_{rel,y}^2\right) \tag{B.4}$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi} \sqrt{\frac{f_{c,0,xlay,d}}{E_{0,x,05}}} \tag{B.5}$$

$$\lambda_y = \frac{L}{i_{x,ef}} \tag{B.6}$$

$$E_{0,x,05} = k * E_{0,x,mean} \tag{B.7}$$

$$k = 1 - \frac{0.328}{\sqrt{\frac{2 * b_x}{0.15} - 1}} \tag{B.8}$$

For cased when $\lambda_{rel,y} < 0.3$ the risk of buckling is almost non-existent and it is necessary to verify the following equation:

$$\left(\frac{\sigma_{c,x,d}}{f_{c,0,xlay,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,xlay,d}} \leq 1 \tag{B.9}$$

Type	5 CLT V	5 CLT V	5 CLT V	5 CLT V	7 CLT V	7 CLT V
Length [m]	3	3	3	3	3	3
Width [m]	3	3	3	3	3	3
Layers x [-]	3	3	3	3	4	4
Layers y [-]	2	2	2	2	3	3
h [mm]	100	150	200	225	280	315
t [mm]	20	30	40	45	40	45
A_{xnet} [mm2]	1,80E+05	2,70E+05	3,60E+05	4,05E+05	4,80E+05	5,40E+05
A_{ynet} [mm2]	1,20E+05	1,80E+05	2,40E+05	2,70E+05	3,60E+05	4,05E+05
I_{xef} [mm3]	6,60E+04	2,23E+05	5,28E+05	7,52E+05	1,30E+06	1,85E+06
I_{yef} [mm3]	1,73E+04	5,85E+04	1,39E+05	1,97E+05	5,28E+05	7,52E+05
I_{xef} [mm4]	1,98E+08	6,68E+08	1,58E+09	2,26E+09	3,90E+09	5,56E+09
I_{yef} [mm4]	5,20E+07	1,76E+08	4,16E+08	5,92E+08	1,58E+09	2,26E+09
$I_{x,ef}$	33,17	49,75	66,33	74,62	90,18	101,46
$I_{y,ef}$	20,82	31,22	41,63	46,84	66,33	74,62
λ_x	90,45	60,30	45,23	40,20	33,26	29,57
λ_y	144,12	96,08	72,06	64,05	45,23	40,20
$\lambda_{rel,x}$	1,492	0,995	0,746	0,663	0,549	0,488
$\lambda_{rel,y}$	2,314	1,543	1,157	1,029	0,726	0,646
k_x	1,673	1,030	0,801	0,738	0,663	0,628
k_y	3,279	1,752	1,212	1,065	0,785	0,726
$k_{c,x}$	0,412	0,772	0,916	0,942	0,966	0,976
$k_{c,y}$	0,179	0,387	0,635	0,744	0,923	0,946
$\sigma_{b,x}$ [N/mm2]	5,53	10,38	12,32	12,66	12,98	13,12
$\sigma_{b,y}$ [N/mm2]	2,40	5,20	8,54	10,01	12,41	12,71

B.2. Calculation buckling factor CLT

Calculation of the buckling factor with verification for CLT elements.

$$\frac{\sigma_{c,x,d}}{k_{c,y} * f_{c,0,xlay,d}} + \frac{\sigma_{m,y,d}}{f_{m,xlay,d}} \leq 1 \tag{B.10}$$

$$\frac{N_d}{k_{c,y} * A_{x,net} * f_{c,0,xlay,d}} + \frac{M_{y,d}}{W_{x,net} * f_{m,xlay,d}} \leq 1 \tag{B.11}$$

$$k_{c,y} = \frac{1}{k_y \sqrt{k_y^2 - \lambda_{rel,y}^2}} \leq 1 \tag{B.12}$$

$$k_y = 0.5\left(1 + 0.1\left(\lambda_{rel,y} - 0.3\right) + \lambda_{rel,y}^2\right) \tag{B.13}$$

$$\lambda_{rel,y} = \frac{\lambda_y}{\pi}\sqrt{\frac{f_{c,0,xlay,d}}{E_{0,x,05}}} \tag{B.14}$$

$$\lambda_y = \frac{L}{i_{x,ef}} \tag{B.15}$$

$$E_{0,x,05} = k * E_{0,x,mean} \tag{B.16}$$

$$k = 1 - \frac{0.328}{\sqrt{\frac{2*b_x}{0.15} - 1}} \tag{B.17}$$

For cased when $\lambda_{rel,y} < 0.3$ the risk of buckling is almost non-existent and it is necessary to verify the following equation:

$$\left(\frac{\sigma_{c,x,d}}{f_{c,0,xlay,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,xlay,d}} \leq 1 \tag{B.18}$$

Type	**5 CLT V**	**5 CLT V**	**5 CLT V**	**5 CLT V**	**7 CLT V**	**7 CLT V**
Length [m]	3	3	3	3	3	3
Width [m]	3	3	3	3	3	3
Layers x [-]	3	3	3	3	4	4
Layers y [-]	2	2	2	2	3	3
h [mm]	100	150	200	225	280	315
t [mm]	20	30	40	45	40	45
A_{xnet} [mm2]	1,80E+05	2,70E+05	3,60E+05	4,05E+05	4,80E+05	5,40E+05
A_{ynet} [mm2]	1,20E+05	1,80E+05	2,40E+05	2,70E+05	3,60E+05	4,05E+05
I_{xef} [mm3]	6,60E+04	2,23E+05	5,28E+05	7,52E+05	1,30E+06	1,85E+06
I_{yef} [mm3]	1,73E+04	5,85E+04	1,39E+05	1,97E+05	5,28E+05	7,52E+05
I_{xef} [mm4]	1,98E+08	6,68E+08	1,58E+09	2,26E+09	3,90E+09	5,56E+09
I_{yef} [mm4]	5,20E+07	1,76E+08	4,16E+08	5,92E+08	1,58E+09	2,26E+09
$I_{x,ef}$	33,17	49,75	66,33	74,62	90,18	101,46
$I_{y,ef}$	20,82	31,22	41,63	46,84	66,33	74,62
λ_x	90,45	60,30	45,23	40,20	33,26	29,57
λ_y	144,12	96,08	72,06	64,05	45,23	40,20
$\lambda_{rel,x}$	1,492	0,995	0,746	0,663	0,549	0,488
$\lambda_{rel,y}$	2,314	1,543	1,157	1,029	0,726	0,646
k_x	1,673	1,030	0,801	0,738	0,663	0,628
k_y	3,279	1,752	1,212	1,065	0,785	0,726
$k_{c,x}$	0,412	0,772	0,916	0,942	0,966	0,976
$k_{c,y}$	0,179	0,387	0,635	0,744	0,923	0,946
$\sigma_{b,x}$ [N/mm2]	5,53	10,38	12,32	12,66	12,98	13,12
$\sigma_{b,y}$ [N/mm2]	2,40	5,20	8,54	10,01	12,41	12,71

B.3. Vibrations CLT

The CLT floor slabs for residential buildings can be designed schematically according to Eurocode 5 on the four aspects following:

1. Determine the fundamental frequency. If this is lower than 8Hz, a special assessment is required.

2. Determine the floor structure's required quality by determining threshold values for parameter a and b (see figure B.1).

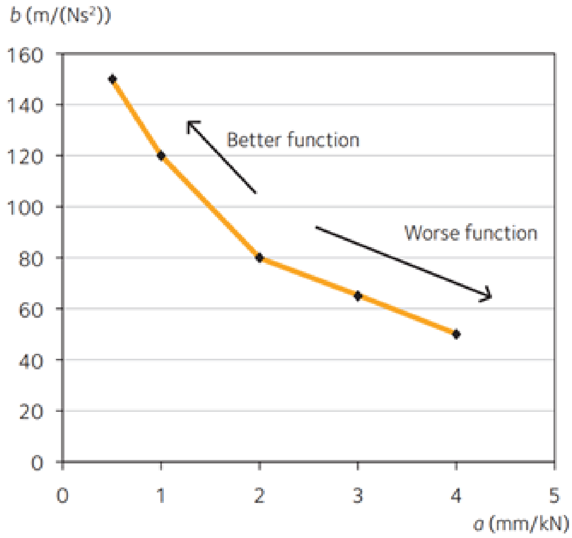


Figure B.1: Threshold values a and b for CLT floor. [21]

3. Check the stiffness of the floor structure by calculating the deflection of one footstep. This is equal to a point load of 1 kN at the centre of the slab.

$$\frac{w}{F} \leq a[\frac{mm}{kN}] \tag{B.19}$$

When a CLT-slab has two-load bearing directions, the floor's stiffness in both directions can therefore be used. The deflection can be determined with the following formula:

$$w = \frac{PL^3}{48 * (EI)_L * B_{ef}} \tag{B.20}$$

$$B_{ef} = \frac{L}{1.1}\sqrt{\frac{(EI)_B}{(EI)_L}} \tag{B.21}$$

- Where $(EI)_L$ is the bending stiffness in the stiffest direction in Nm^2/m
- Where $(EI)_B$ is the bending stiffness perpendicular to the stiffest direction in Nm^2/m
- Where L is the length is the stiffest direction in $[m]$

4. The impulse velocity response v against the chosen floor structure quality, with equation:

$$v \leq b^{f_1 \zeta - 1} \tag{B.22}$$

$$f_1 = \frac{\pi}{2L^2}\sqrt{\frac{(EI)_L}{m}} \tag{B.23}$$

$$v = \frac{4(0.4 + 0.6n_{40})}{mBL + 200} \tag{B.24}$$

$$n_{40} = \left[\left(\left(\frac{40}{f_1}\right)^2 - 1\right)\left(\frac{B}{L}\right)^4\left(\frac{(EI)_L}{(EI)_B}\right)\right]^0 .25 \tag{B.25}$$

- Where m is the floor structure mass per meter in $[kg/m^2]$
- Where ζ is the relative damping, which is assumed to be 1 percent for a point supported CLT slab.
- Where $(EI)_L$ is the bending stiffness in the stiffest direction in Nm^2/m
- Where $(EI)_B$ is the bending stiffness perpendicular to the stiffest direction in Nm^2/m
- Where L is the floor span in $[m]$
- Where B is the floor width in $[m]$
- Where v is the impulse velocity response in $[m/Ns^2]$
- Where n_{40} is the number of first order modes with fundamental frequencies of up to 40 Hz

B.4. Verification CLT

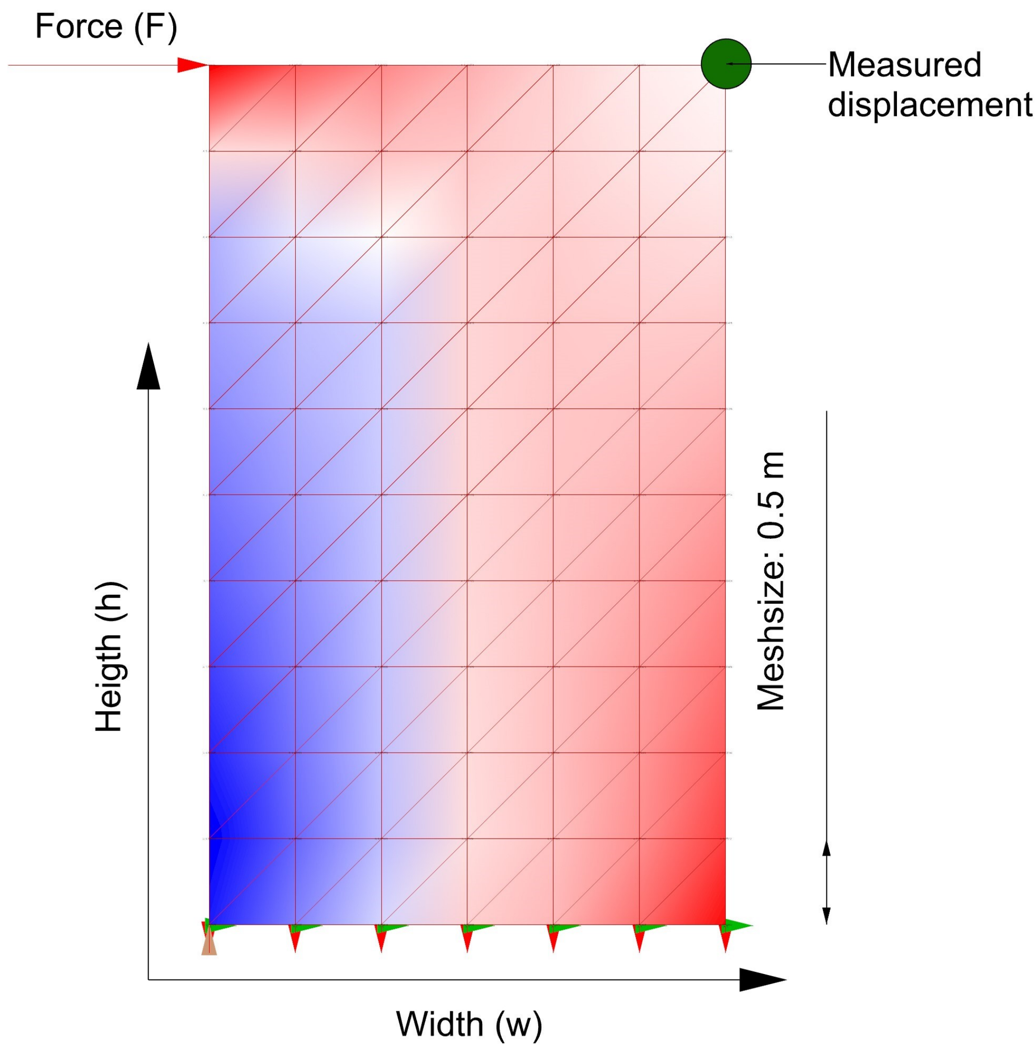


Figure B.2: Tested CLT panel in Karamba

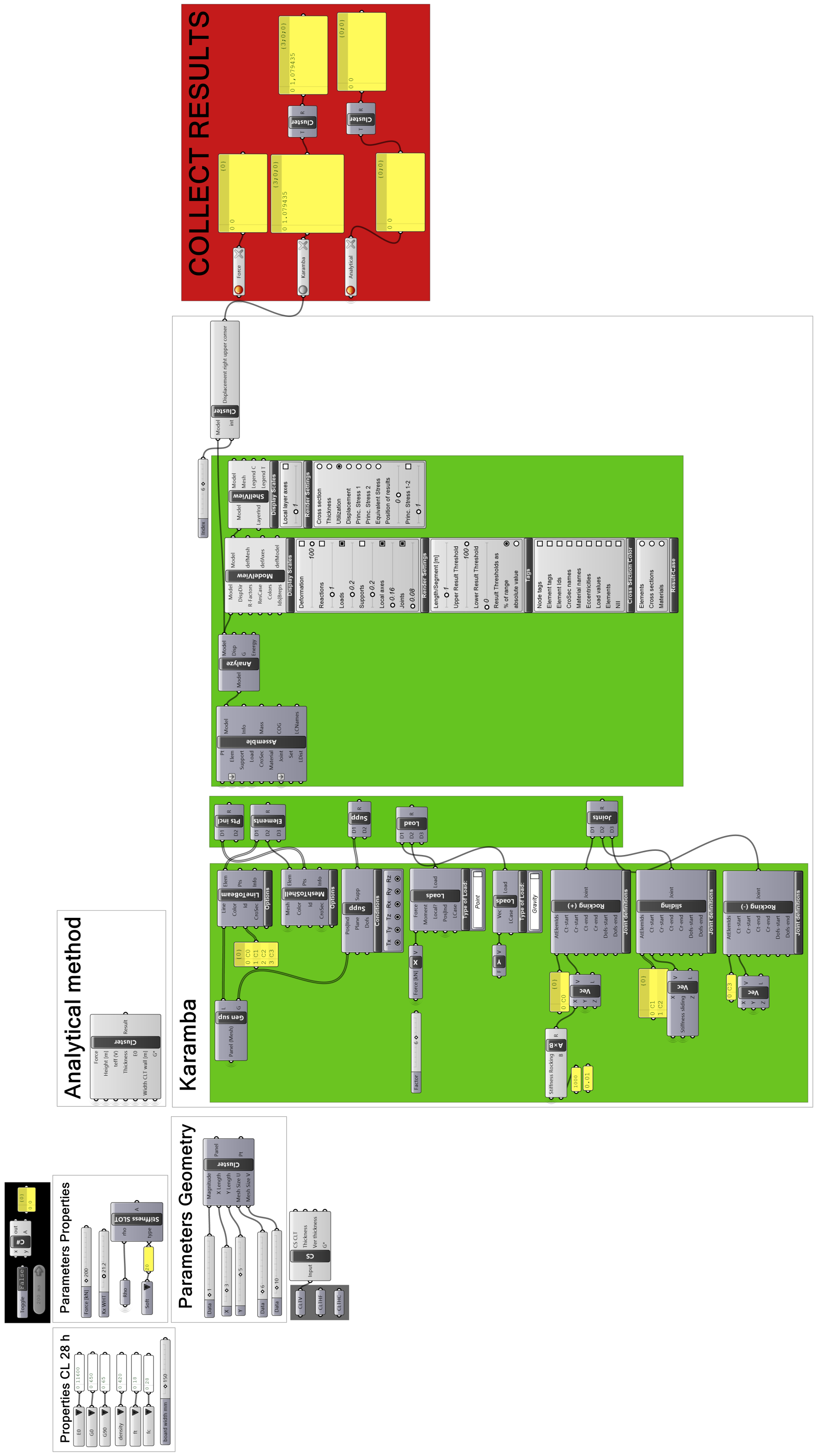


Figure B.3: Grasshopper script to verify Fictitious CLT properties

C

Connections modular construction

C.1. Inter-module connection

In [70] a inter-modular connection is proposed for modular steel buildings, which has high structural and work-ability standards. The connection in figure C.1 does not require space to provide accessibility and facilitate design requirements and can be tightened and fastened from a distance (see figure C.2). The connection is capable of achieving 112% of the designed column member's compression capacity in compression, 139% in shear and about 40% in tension. Nevertheless, despite the high structural advantages, the greatest limitation is the precise manufacturing and therefore may the proposed connection not capable of handling large construction tolerances.

Figure C.1: Novel inter-module connection for modular steel buildings consisting of three components: the internal component, external component, and horizontal transfer plate.

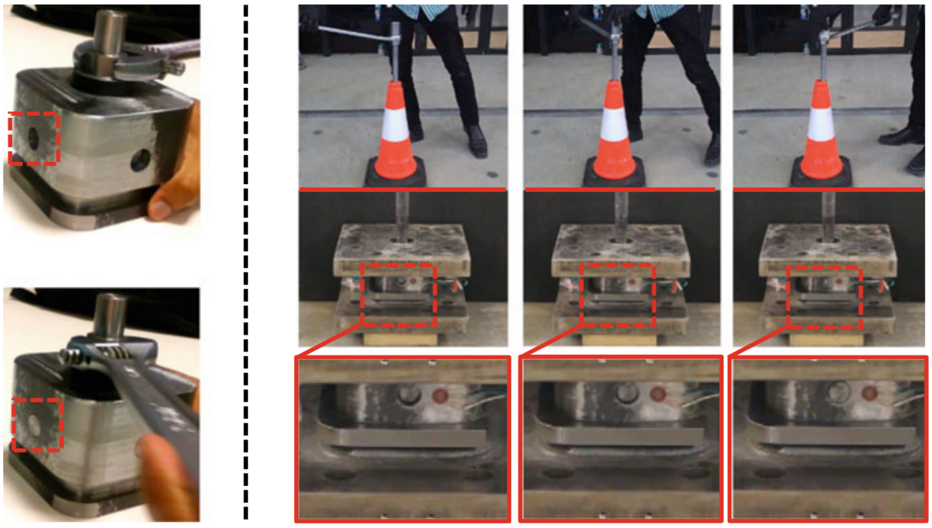


Figure C.2: Demonstrating how the connection ca be locked by torque

In the technical case study of the Prince George a typical column to column connection is proposed, which can transfer tension forces through threaded rods and a sleeve connection. The hold-down nuts, which are inserted in the sleeve connection are tightened using a special tool made from magnet and a ratcheting box wrench. At completion, the access holes are filled with timber to conceal the bolts. [75]

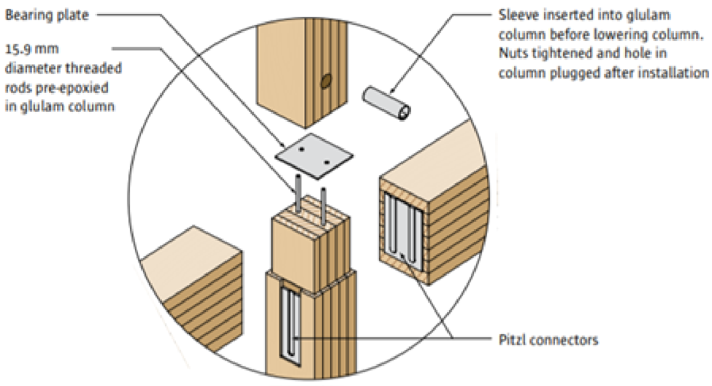


Figure C.3: Exploded view of a typical view of a column- to column connection with beam connectors [75]

C.2. Traditional Cross-laminated timber connections

Three typical vertical panel-to-panel connection types are presented in figure C.4. The number of screws in the vertical joint is critical because the number of screws determines the kinematic behavior. The half-lap joint shows and LVL spline joint with a sufficient number of screws show single-coupled wall behavior or coupled wall behavior with fewer screws. [32]

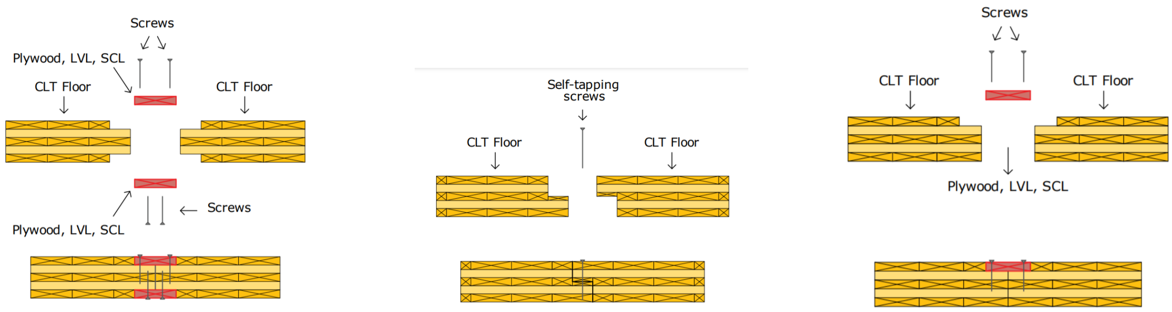


Figure C.4: Overview traditional panel to panel connections for CLT [57]

C.3. Load bearing Pillar connection

D_{cyl} / D_{bp}	$F_{PILLAR,Rk}$ [kN] without reinforcement											
	CENTRAL				EDGE				CORNER			
	CLT 5l		CLT 7l		CLT 5l		CLT 7l		CLT 5l		CLT 7l	
	○	□	○	□	○	□	○	□	○	□	○	□
120/240	140	183	179	235	61	80	75	99	26	34	30	40
120/280	216	270	278	346	95	118	116	145	40	50	47	59
100/240	159	203	204	260	70	89	85	109	30	38	35	44
100/280	236	289	302	370	103	127	127	155	44	54	51	63
80/200	110	141	142	180	48	62	59	76	21	26	24	31
80/240	175	219	225	281	77	96	94	118	33	41	38	48
80/280	252	305	323	391	110	134	135	164	47	57	55	66
60/200	124	154	159	197	54	68	66	83	23	29	27	33
60/240	188	232	242	298	83	102	101	125	35	43	41	50
60/280	265	318	340	408	116	140	142	171	49	59	58	69

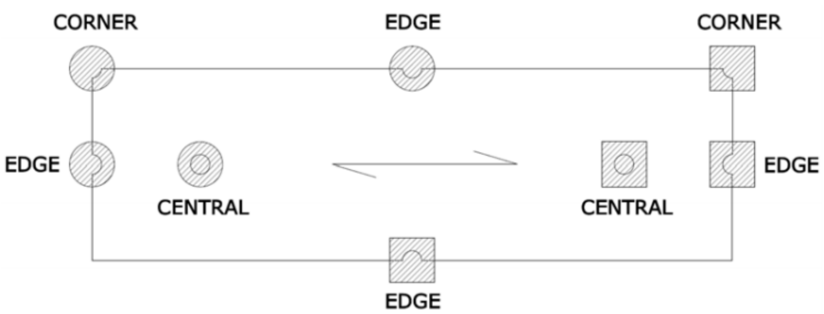


Figure C.5: The characteristic load-bearing capacity of the Pillar connector on the CLT panel, with orientation of placement of the Pillar connector [ETA-19/0700]

C.4. HBS properties

Nominal diameter	$d_{1)}$	**[mm]**	**12**
Head diameter	d_k	[mm]	20.75
Tip diameter	d_2	[mm]	6.8
Shank diameter	d_s	[mm]	8
Head thickness	t_1	[mm]	7.2
Pre-drilling hole diameter	d_v	[mm]	7
Characteristic yield moment	$M_{y,k}$	[Nm]	48
Characteristic withdrawal-resistance capacity (softwood)	$f_{ax,k}$	[N/mm2]	11.7
Associated density (softwood)	ρ_a	[kg/m3]	350
Characteristic withdrawal-resistance capacity (softwood LVL)	$f_{ax,k}$	[N/mm2]	15
Associated density (softwood LVL)	rho_a	[kg/m3]	500
Characteristic head-pull through paramater (softwood)	$f_{head,k}$	[N/mm2]	10.5
Associated density (softwood)	ρ_a	[kg/m3]	350
Characteristic head-pull through paramater (softwood LVL)	$f_{head,k}$	[N/mm2]	20
Associated density (softwood LVL)	ρ_a	[kg/m3]	500
Characteristic tensile strength	$f_{tens,k}$	[kN]	33.9

D

Parametric modular construction

D.1. Reference projects parametric modular construction

This appendix has an overview of all reviewed computational design in combination with prefabrication projects. Tabel D.1 provides an overview.

Project	Paper	Author	Elements	Typology	Design approach	Focus	Software	Integrated structural analysis
A	Integrating computational design to improve the design workflow of modular construction (2019)	Greenough et al.	Modular (3D)	Simple	Adapted to modular construction	Efficiency and mass production	Revit + Dynamo + Grasshopper + SAP	An iterative process with SAP and Grasshopper.
B	Generative tool for modular buildings, Gensler research institute (2020)	Gen	Modular (3D)	Simple	Adapted t modular construction	Efficiency and mass production	Dynamo + WPF	No
C	Exhaustive exploration of modular design options to inform decision making (2017)	Mekawy and Petzold	Modular (3D)	Simple	Adapted to modular construction	Efficiency and mass production	Revit + Dynamo	No, maximum of stacking determined on beforehand.
D	Automated design of prefabricated building (1994)	Retik and Warszawski	Prefab (2D)	Simple	Not adapted to modular construction	Preliminary drawings of the architect	Unknown	Element determined on beforehand. No calculations.
E	Development of a design-driven parametric mass timber construction system for modular high-rise urban housing (2019)	Lang et al.	Modular (3D)	Simple	Adapted to modular construction	Efficiency and mass production	Rhinoceros + Grasshopper	No

Table D.1: Overview software reference projects

D.1.1. Integrating computational design

A computational design tool for modular steel construction is made and presented by Greenough et al.. [35] A parametric Dynamo script generated the modular steel units with a given set of parameters and assembled them. Two main steps divide the workflow:

1. Geometry creation.
2. Geometry modification and structural member assignment and placement of modules inside the Revit BIM[1] environment with Dynamo.

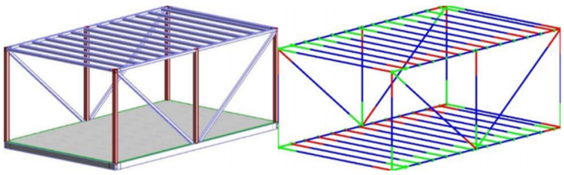


Figure D.1: Modular unit and analytical view of the modular unit [35]

Parameters	**Type**
Module dimension	Sliders
Member assignment	Dropdown box
Connections	Selection options
Placing of module	Dynamo, sliders

Structural analysis
The analytical geometry is linked to an analytical program (SAP) to complete the structural analysis. The finished analysis can address necessary changes in the model that impact the design and result in an iterative process. The changes need to be made in both the analytical software and the BIM model. Changes are made in Revit and can be exported again to SAP. Loads and load combinations are defined in the structural program. This reduces the workflow efficiency, but a dynamo tool that reduces time in updating analytical models is still missing.

Figure D.2: Assembled model in Revit after an iterative design process [35]

Ideal scenario
The spacing between members and connections between modules results in a higher number of elements than the conventional structure of similar size. It is challenging to display all the elements' results, which can be done graphically or in tabular form in structural programs. A combination of Grasshopper and Python is used to display all the results in graphical form. This workflow reduces the amount of time spent by engineers to design the foundation elements by 50%. And for checking connections on uplift resulted in time-saving of 80 up to 90 per cent. In an ideal scenario everything is completed and documented in one software package.

[1]BIM = building information modeling

D.1.2. Generative tool for modular buildings

The Generative design tool for modular buildings by Gensler Research Institute [5] helps the designer to generate multiple hotel configurations quickly in the early stage of a design process. The workflow comprehends two steps:

1. First, all the modular systems' requirements and constraints are determined and translated into a visual programming language.
2. Secondly, a series of inputs is established corresponding to the needs and limitations, with a user interface.

Parameters	**Type**
Module dimension	Sliders
Member assignment	-
Connections	-
Placing of module	Dynamo, sliders

Early stage design tool

The generative tool focuses on residential and hotel buildings. Alphabetical building layouts can be produced with the tool and visualized in Revit. Through a custom interface, the user is able to generate and compare designs on the objectives rapidly. Analytical geometry is lacking in this tool; it only focuses on the placement of modules in an organized and ordered way, making this tool an early stage design tool for mass studying.

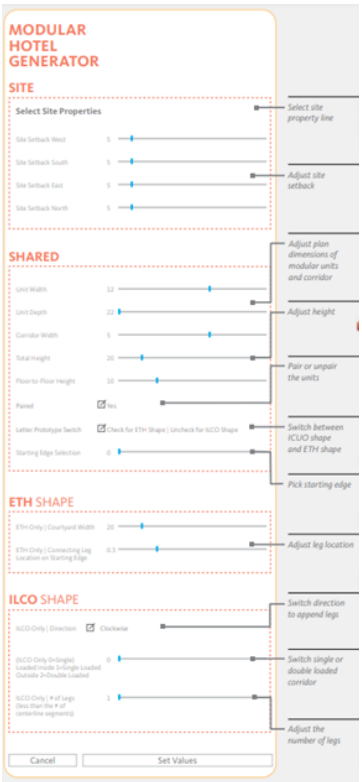


Figure D.3: The interface of the tool to set specific parameters. [5]

D.1.3. Exhaustive exploration of modular design options

Mekawy and Petzold present an approach realized as a Dynamo for Revit plugin called Box Module Generator(BMG), consisting of a collection of custom nodes to help the designer generate modular construction solutions. [56] The tool contains windows forms for a graphical interface. The workflow of this tool consists of two steps (see Figure D.4):

1. The process starts with the input from architects, owners and manufacturers, regarding the project requirements, building regulations and construction regulations. The architect defines design rules, the manufacturer supplies the maximum stacking of modules, and the owner has specific lighting requirements. The input, which consists of adjustable parameters by sliders or code blocks, is then passed to the solution generator, producing all possible combinations of the input parameters.

2. The user can slide through all the solutions and sort with optioneering, and select a model to generate in Revit.

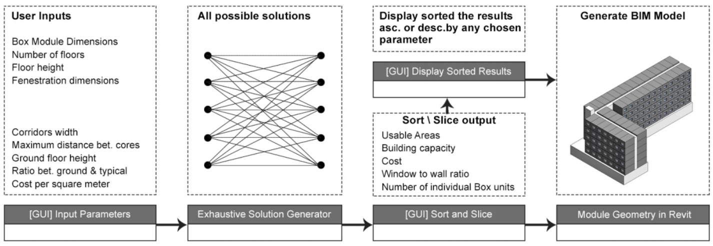


Figure D.4: The design process [56]

Parameters	**Type**
Module dimension	Sliders
Member assignment	-
Connections	-
Placing of module	Dynamo, sliders

Configurations

This research aims to generate a list of possible configurations for a modular building design using an algorithm. Project stakeholders can make informed decisions early in the process and efficiently use the preliminary BIM model to perform further analysis. The generated models are arranged in stacked configurations to achieve maximum efficiency (See figure D.5). It is not possible to make more custom designs, and another flaw is the missing structural analysis.

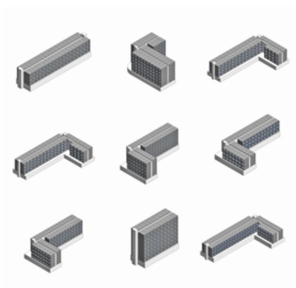
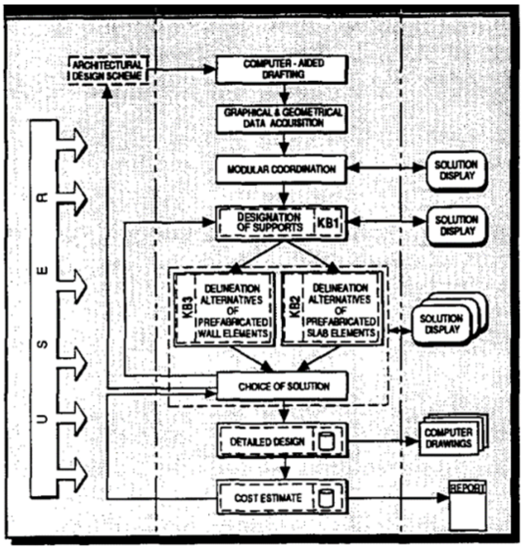


Figure D.5: Some of the models generated by the BMG tool [56]

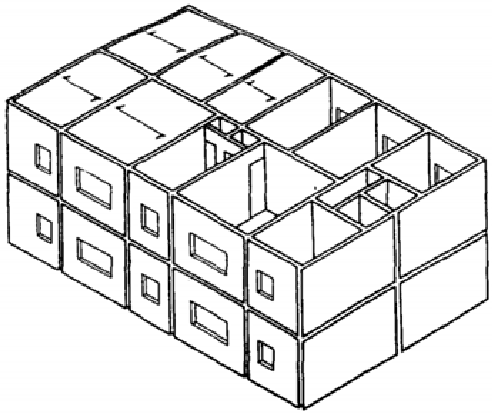
D.1.4. Automated design of prefabricated building

This is one of the oldest approach found about automated designs for prefabricated buildings. [65] Retik and Warszawski did research where a tool translates preliminary architectural drawings into prefabricated concrete slab elements. The design process consists of seven stages:

1. The input of preliminary architectural drawings.
2. The geometries with properties are extracted in the system.
3. The preliminary design adapts to the modular construction system. The adaptation may result in some changes in the location of walls and partitions.
4. Designation of supports. The user selects the preferred option in this stage.
5. This stage follows an iterative process. The layout of each prefabricated component in architectural design and the location of the prefabrication method's supports and constraints are being checked. If the user is not satisfied with the results, the system will advise the user about possible changes.
6. A detailed design of components is made.
7. Cost estimation of the components is made.



(a) Workflow



(b) Prefabricated building result

Figure D.6: Short overview of result and workflow of: Automated design of prefabricated building. [65]

Custom design

The possible manual adaptions make the tool suitable for a custom design. Also, the approach is different compared to other reference prefabrication tool projects. Because the geometry created by the tool is not generated by adjustable parameters but based on decisions of the architect and engineer.

D.1.5. Design-driven parametric mass timber construction system

Lang et al. designed a digital tool to create preliminary designs for high-rise urban housing. A parametric platform shares the design's fundamental aspects, such as its use and modules' topology. Within the tool a design space is created, which defines the range of possibilities for a constrained process. The design space is determined by each module's size, maximum module stacking, overall building height and other structural parameters.

Design process

The total workflow consists of 4 main steps:

1. Based on the site constraints the parametric model fills the available space with boxes, representing each module's boundaries. At the end of this stage, it is possible to extract the envelope area, footprint, and floor space ratio. Also it is possible to estimate the amount of wood and steel needed at this stage.
2. A variety of home types is generated in this phase, based on stage one's resulting modules arrangement. Based on the requirements, this process can be manually controlled through a table like input.
3. Construction elements are defined and form the modules.
4. Information for manufacturing of individual parts can be exported, by CNC drawings.

Parameters	**Type**
Module dimension	Sliders
Member assignment	-
Connections	-
Placing of module	Parametric model (Grasshopper?)

Collaboration

The tool described in this source allows for an early collaboration in the construction process, leading to an increase in sustainability, efficiency and quality. However a structural analysis is missing in the design process. Parameters of the building define the member assignment, like the overall height or position concerning the overall grid. The design for custom modular buildings is not possible due to the use of parameters and methodical infill scripts instead of drawings and due to the focus on high-efficiency buildings.

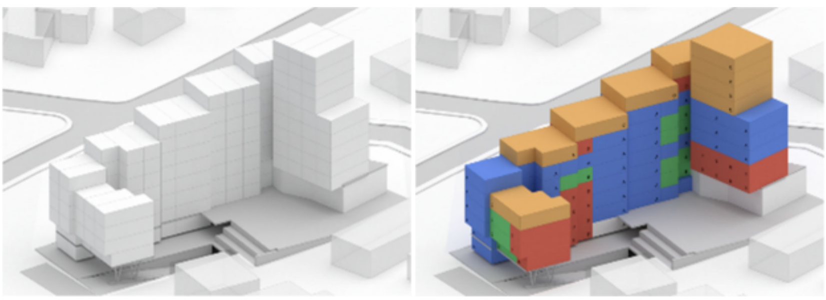


Figure D.7: The design process: First, the available space is filled with boxes to site constraints, then the created work gets populated by different home typologies. [46]

E

Loads

E.1. Wind-force

The wind force can be calculated with equation 5.4 in section 5.3 of the national annex (NEN-EN 1991-1-4):

$$F_w = c_s c_d * c_f * q_p(z) * A_{ref} \tag{E.1}$$

- Where $c_s c_d$ is the structural factor
- Where c_f is the force coefficient
- Where $q_p(z)$ is the peak velocity wind pressure for height z in meters
- Where A_{ref} is the reference area on the structure

The distribution of wind force depends on the height and dimension of the building. Three cases divide the calculation of wind forces (see figure E.1):

1. For a building where the building's height is smaller or equal to the building's width: The wind pressure is uniform along with the building.
2. For a building where the height is between one and two times the building width: The Wind pressure is divided into two blocks.
3. For buildings taller than two times the width: the wind pressure is divided into three blocks.

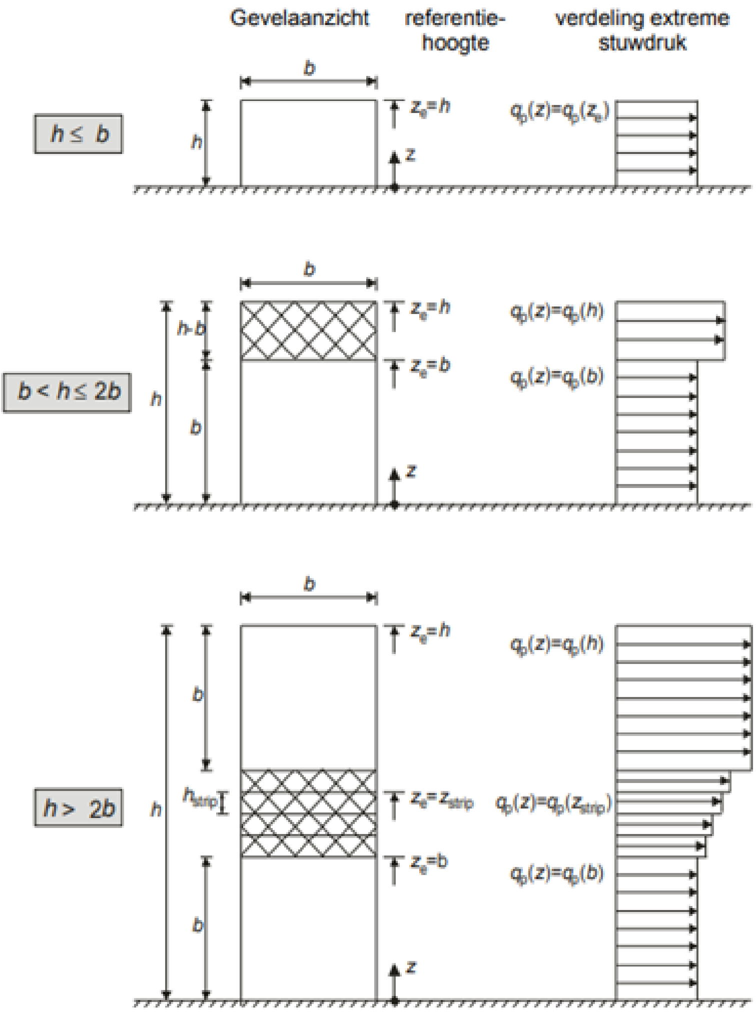


Figure E.1: Wind force distribution per dimension of building following the Eurocode 1991-1-1-4

The peak velocity wind pressure (equation 4.8 in Eurocode 1991-1-4) is calculated as follows:

$$q_p(z) = (1 + 7 * l_v(z)) * \frac{1}{2} * \rho * v_m^2(z) \tag{E.2}$$

- Where ρ is the density of air, taken as 1.25 kg/m^3

Number of stories	Buidling height	qp(z)
1	3	0,45
2	6	0,63
3	9	0,75
4	12	0,83
5	15	0,90
6	18	0,96
7	21	1,01
8	24	1,05
9	27	1,09
10	30	1,13
11	33	1,16
12	36	1,19

Table E.1: Results of the peak velocity for wind force distribution 1 of figure E.1

The factor $c_s c_d$ takes the effect of the occurrence of the peak wind pressure won the surface with the turbulence of the structure. This structural factor can be determined in section 6.2 of the Eurocode 1991-1-4. As simplification for this thesis is a value of 1.0 taken to full fill all calculations.

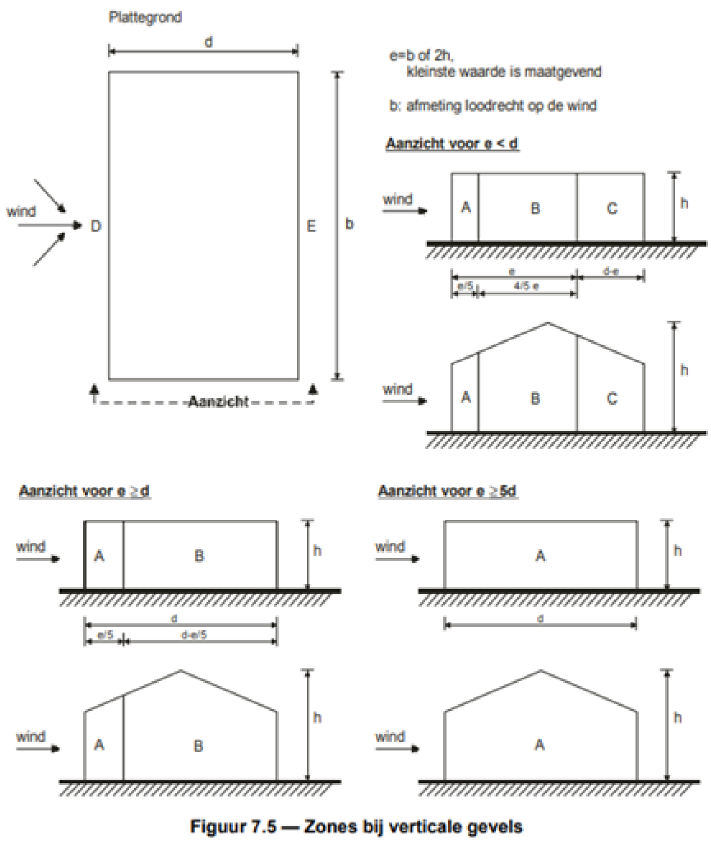


Figure E.2: Wind direction and zones of a building following the Eurocode 1991-1-1-4

For the calculation of the wind force, should the coefficients determined for suction and pressure. Table E.2 provides an overview of all values. The pressure coefficient $c_{pe,10}$ is given in table E.2 , which is for surfaces of 10 square meters and larger. This value represents the orthogonal wind directions, and is unfavorable for wind directions of 45 degrees. Only the surfaces D and E will be evaluated in the tool, and the pressure on the sides wont be taken into account. A value of $0.25 < h/d < 5$ is assumed for this thesis, since the aim of the modular tool is not to create slender buildings.

Zone	**A**	**B**	**C**	**D**	**E**
h/d	Cpe_{10}	Cpe_{10}	Cpe_{10}	Cpe_{10}	Cpe_{10}
5	-1.2	-0.8	-0.5	0.8	-0.7
1	-1.2	-0.8	-0.5	0.8	-0.5
≤ 0,25	-1.2	-0.8	-0.5	0.7	-0.3

Table E.2: Coefficients for suction and pressure

For buildings with an height over width ratio smaller than one, may the resulting wind force multiplied by 0.85 (see equation E.3). This factor is not implemented in the tool, to reduce the workload. A conservative factor of 1 is considered.

$$q_{wind}(z) = c_s c_d * c_f * q_p(z) * k \tag{E.3}$$

h/d	**k**
5	1
1	0.85

Table E.3: Determination of factor k for reduced wind load

Number of stories	**Building height**	q_{wind}
1	3	0,59
2	6	0,82
3	9	0,97
4	12	1,08
5	15	1,17
6	18	1,25
7	21	1,31
8	24	1,37
9	27	1,42
10	30	1,47
11	33	1,51
12	36	1,55

Table E.4: Wind load q_{wind} in [kN/m2] following wind distribution case 1

F

Developed C# scripts

F.1. Itterative Karamba script

This appendix contains the C# Karamba methods. The AssembleModel method assembles all elements of the model. The ItterationModel returns the final model, where all tension and compression joints are determined with a maxiumum itteration of 15.

```
public Model AssembleModel(List<System.> elements,
List<System.> supports,
List<System.> loads,
List<System.> joints)
{
  List<BuilderElement> Elements = new List<BuilderElement>();
  for (int i = 0; i < elements.Count; i++)
  {
    var Element = elements[i] as BuilderElement;
    Elements.Add(Element);
  }

  List<Support> Supports = new List<Support>();
  for (int i = 0; i < supports.Count; i++)
  {
    var Supp = supports[i] as Support;
    Supports.Add(Supp);
  }

  List<Load> Loads = new List<Load>();
  for (int i = 0; i < loads.Count; i++)
  {
    var Load1 = loads[i] as Load;
    Loads.Add(Load1);
  }


  List<Joint> Joints = new List<Joint>();
  for (int i = 0; i < joints.Count; i++)
  {
    var Joint1 = joints[i] as Joint;
    Joints.Add(Joint1);
```

```
    }


    var k3d = new Toolkit();

    double mass;
    Point3 cog;
    bool flag;
    string info;
    Model model = k3d.Model.AssembleModel(Elements, Supports,
    Loads, out info,
    out mass, out cog,
    out info, out flag,
    Joints);

    return model;
  }


  public Model AnalyzeModelHabitat(Model model)
  {
    var k3d = new Toolkit();
    List<double> max_displ;
    List<double> out_g;
    List<double> out_comp;
    string message;
    Model calcModel = k3d.Algorithms.AnalyzeThI(model,
    out max_displ, out out_g,
    out out_comp, out message);

    return calcModel;
  }

  public Tuple<List<string>, List<string>> ResultModelHabitat(Model model,
  List<string> idsToCheck)
  {
    List<List<double>> nList;
    List<List<double>> mList;
    List<List<double>> vList;
    BeamResultantForces.solve(model, idsToCheck, "0", 1, 1, out nList, out vList, out mList);

    List<string> newCompressionJoints = new List<string>();
    List<string> newTensionJoints = new List<string>();

    for (int i = 0; i < nList[0].Count; i++)
    {
      if (nList[0][i] <= 0)
      {
        newCompressionJoints.Add(idsToCheck[i]);
      }
      else
      {
        newTensionJoints.Add(idsToCheck[i]);
      }
    }
```

```

    return Tuple.Create(newCompressionJoints, newTensionJoints);
  }


  public Model AssembleModel(List<System.> elements,
    List<System.> supports,
    List<System.> loads,
    List<System.> constantJoints,
    List<string> idsJointCF,
    List<string> idsJointTF,
    List<string> idsJointCW,
    List<string> idsJointTW,
    double? kx, double? ky, double? kz)
  {
    List<BuilderElement> Elements = new List<BuilderElement>();
    for (int i = 0; i < elements.Count; i++)
    {
      var Element = elements[i] as BuilderElement;
      Elements.Add(Element);
    }

    List<Support> Supports = new List<Support>();
    for (int i = 0; i < supports.Count; i++)
    {
      var Supp = supports[i] as Support;
      Supports.Add(Supp);
    }

    List<Load> Loads = new List<Load>();
    for (int i = 0; i < loads.Count; i++)
    {
      var Load1 = loads[i] as Load;
      Loads.Add(Load1);
    }

    List<Joint> Joints = new List<Joint>();
    for (int i = 0; i < constantJoints.Count; i++)
    {
      var Joint1 = constantJoints[i] as Joint;
      Joints.Add(Joint1);
    }

    Joint jointCompressionFloor = JointHabitat1("J23CF", kx, ky, null, idsJointCF);
    Joint jointTensionFloor = JointHabitat1("J23TF", kx, ky, null, idsJointTF);

    Joint jointCompressionWall = JointHabitat1("J23CW", null, ky, kz, idsJointCW);
    Joint jointTensionWall = JointHabitat1("J23TW", kx, ky, kz, idsJointTW);

    Joints.Add(jointCompressionFloor);
    Joints.Add(jointTensionFloor);
    Joints.Add(jointCompressionWall);
    Joints.Add(jointTensionWall);

    var k3d = new Toolkit();
```

```

    double mass;
    Point3 cog;
    bool flag;
    string info;
    Model model = k3d.Model.AssembleModel(Elements,
    Supports, Loads,
    out info, out mass,
    out cog, out info,
    out flag, Joints);
    return model;
  }

  public Joint JointHabitat1(string name,
  double? kx,
  double? ky,
  double? kz,
  List<string> elemIds)
  {
    Joint jointAgent = new Joint();
    jointAgent.props
    = new double?[] { null, null, null, null, null, null, kx, ky, kz, null, 0, 0 };
    jointAgent.name = name;
    jointAgent.elemIds = elemIds;

    return jointAgent;
  }

  public Tuple<Model,
  List<string>,
  List<string>,
  List<string>,
  List<string>> IttertionModel(
    List<System.> elements,
    List<System.> supports,
    List<System.> loads,
    List<System.> joints,
    List<System.> constantJoints,
    List<string> idsNonLinearJointsFloor,
    List<string> idsNonLinearJointsWall,
    double? kx,
    double? ky,
    double? kz)
  {
    // First analysis
    Model modelIn = AssembleModel(elements, supports, loads, joints);
    Model modelAnalyze = AnalyzeModelHabitat(modelIn);
    Tuple<List<string>, List<string>> resultsF_1
    = ResultModelHabitat(modelAnalyze, idsNonLinearJointsFloor);
    Tuple<List<string>, List<string>> resultsW_1
    = ResultModelHabitat(modelAnalyze, idsNonLinearJointsWall);

    bool run = true;
    int maxItteration = 15;
    int itteration = 0;
```

```

    Model finalModel = null;
    Tuple<List<string>, List<string>> finalTupleF = null;
    Tuple<List<string>, List<string>> finalTupleW = null;

    while (run & itteration <= maxItteration)
    {
      // first result compression, second result tension
      Model modelItteration = AssembleModel(elements,
        supports,
        loads,
        constantJoints,
        resultsF_1.Item1, resultsF_1.Item2,
        resultsW_1.Item1, resultsW_1.Item2,
        kx, ky, kz);
      Model modelItterationAnalyze = AnalyzeModelHabitat(modelItteration);
      Tuple<List<string>, List<string>> resultsF_2
      = ResultModelHabitat(modelItterationAnalyze, idsNonLinearJointsFloor);
      Tuple<List<string>, List<string>> resultsW_2
      = ResultModelHabitat(modelItterationAnalyze, idsNonLinearJointsWall);

      if (resultsF_1.Item1.Except(resultsF_2.Item1).ToList().Count > 0
        |
        resultsW_1.Item1.Except(resultsW_2.Item1).ToList().Count > 0)
      {
        run = true;
        itteration += 1;
        resultsF_1 = resultsF_2;
        resultsW_1 = resultsW_2;

      }
      else
      {
        run = false;
        finalModel = modelItterationAnalyze;
        finalTupleF = resultsF_2;
        finalTupleW = resultsW_2;
        Rhino.RhinoApp.WriteLine("Calculation SLS 1 X finished");
      }
      if (itteration == maxItteration)
      {
        finalModel = modelItterationAnalyze;
        finalTupleF = resultsF_2;
        finalTupleW = resultsW_2;
        Rhino.RhinoApp.WriteLine(
        "Maximum itteration reached SLS 1, change maximum itteration or rebuild model");
      }
    }

    return Tuple.Create(finalModel,
    finalTupleF.Item1,
    finalTupleF.Item2,
    finalTupleW.Item1,
    finalTupleW.Item2);
  }
```

F.2. Single step brute force method

This appendix contains the C# code for generating options for the brute force method, which is described in section 7.3. The code contains multiple objects. HabitatBuilding, which represents one habitatbuilding with properties. HabitatBuildingOpt, which represents all possible buildings, the user can choose one option with the function ReplaceModule(int habitatBuilding).

```
using Rhino.DocObjects;
using Rhino.RhinoDoc;

 public class HabitatBuilding
  {
    public double Displacement { get; set; }
    public Mesh AddedWall { get; set; }
    public string NumberWall { get; set; }
    public string idReplaced { get; set; }
    public int integerReplaced { get; set; }

    // Contains all elements of the building,
    // Each branch is inherent to a module
    public DataTree<RhinoObject> rhinoObjectsCeiling
    = new DataTree<RhinoObject>();
    public DataTree<RhinoObject> rhinoObjectsFloor
    = new DataTree<RhinoObject>();
    public DataTree<RhinoObject> rhinoObjectsColumn
    = new DataTree<RhinoObject>();
    public DataTree<RhinoObject> rhinoObjectsWall
    = new DataTree<RhinoObject>();
    public DataTree<RhinoObject> rhinoObjectsInter
    = new DataTree<RhinoObject>();
    public DataTree<RhinoObject> rhinoObjectsIntra
    = new DataTree<RhinoObject>();
    public DataTree<Point3d> rhinoObjectsSnapPoints
    = new DataTree<Point3d>();

    // Constructor
    public HabitatBuilding(List<string> geomIDS,
    List<Transform> transModules,
    string number)
    {
      string[] codeAdded = number.Split('_');
      int intAdded = Convert.ToInt16(codeAdded[0]);
      string numberWall = codeAdded[1];

      for(int i = 0; i < geomIDS.Count; i++)
      {
        bool run = false;
        if(intAdded == i)
        {
          this.idReplaced = geomIDS[i];
          this.integerReplaced = intAdded;
          run = true;
        }

```

```
        Tuple<List<RhinoObject>,
          List<RhinoObject>,
          List<RhinoObject>,
          List<RhinoObject>,
          List<RhinoObject>,
          List<RhinoObject>,
          Tuple<List<Point3d>,RhinoObject>> tuple
          = ExtractRhinoObjects(geomIDS[i], transModules[i]);

        GH_Path path = new GH_Path(i);
        rhinoObjectsCeiling.AddRange(tuple.Item1, path);
        rhinoObjectsFloor.AddRange(tuple.Item2, path);
        rhinoObjectsColumn.AddRange(tuple.Item3, path);
        rhinoObjectsWall.AddRange(tuple.Item4, path);
        rhinoObjectsInter.AddRange(tuple.Item5, path);
        rhinoObjectsIntra.AddRange(tuple.Item6, path);
        rhinoObjectsSnapPoints.AddRange(tuple.Item7.Item1, path);

        if (run)
        {
          for (int k = 0; k < tuple.Item4.Count;k++)
          {
            if (tuple.Item4[k].Name == "Wall_Wall_" + numberWall)
            {
              this.AddedWall = tuple.Item4[k].Geometry as Mesh;
              this.NumberWall = numberWall;
            }
          }
        }
      }
    }

    private RhinoObject[] GetInstanceGeom(string name, Transform trans)
    {
      InstanceDefinition instanceDefinition
      = ActiveDoc.InstanceDefinitions.Find(name);
      RhinoObject[] objs = instanceDefinition.GetObjects();
      return objs;
    }

    private Tuple<List<RhinoObject>,List<RhinoObject>,
      List<RhinoObject>,List<RhinoObject>,
      List<RhinoObject>,List<RhinoObject>,
      Tuple<List<Point3d>,RhinoObject>> ExtractRhinoObjects(
      string idInstanceDefinition, Transform trans)
    {
      RhinoObject[] objs = GetInstanceGeom(idInstanceDefinition, trans);
      List<RhinoObject> CeilingList = new List<.RhinoObject>();
      List<RhinoObject> FloorList = new List<RhinoObject>();
      List<RhinoObject> ColumnList = new List<RhinoObject>();
      List<RhinoObject> WallList = new List<RhinoObject>();
      List<RhinoObject> IntraList = new List<RhinoObject>();
      List<RhinoObject> InterList = new List<RhinoObject>();
      List<Point3d> PointList = new List<Point3d>();
      RhinoObject wallAdded = null;
```

```

      for(int i = 0; i < objs.Length; i++)
      {
        objs[i].Geometry.Transform(trans);
        if (objs[i].Name.Contains("Snap"))
        {
          Point point = (Rhino.Geometry.Point) objs[i].Geometry;
          Point3d point3d = point.Location;
          PointList.Add(point3d);
        }
        else if (objs[i].Name.Contains("Inter"))
          InterList.Add(objs[i]);
        else if (objs[i].Name.Contains("Intra"))
          IntraList.Add(objs[i]);
        else if (objs[i].Name.Contains("Ceiling"))
          CeilingList.Add(objs[i]);
        else if (objs[i].Name.Contains("Floor"))
          FloorList.Add(objs[i]);
        else if (objs[i].Name.Contains("Column_"))
          ColumnList.Add(objs[i]);
        else if (objs[i].Name.Contains("Wall_"))
          WallList.Add(objs[i]);
      }
      return Tuple.Create(
        CeilingList,
        FloorList,
        ColumnList,
        WallList,
        InterList,
        IntraList,
        Tuple.Create(PointList, wallAdded));
    }
  }
```

```
public class HabitatBuildingOpt
{
  // field properties
  public List<InstanceObject> InstanceObjects = new List<InstanceObject>();
  public List<Transform> Transforms = new List<Transform>();

  // one of the options to model
  public List<HabitatBuilding> HabitatBuildings = new List<HabitatBuilding>();

  // Constructor start of the calculation
  public HabitatBuildingOpt(List<object> instanceObjects)
  {
    ClearOpt();
    for (int i = 0; i < instanceObjects.Count; i++)
    {
      this.InstanceObjects.Add(instanceObjects[i] as .InstanceObject);
    }

    for (int i = 0; i < this.InstanceObjects.Count;i++)
    {
```

```
            Transforms.Add(this.InstanceObjects[i].InstanceXform);
          }
        }

        public void GenerateHabitatBuildings()
        {
          Tuple<DataTree<string>,DataTree<string>> options = GenerateOptions(true, true);

          for (int i = 0; i < options.Item1.BranchCount; i++)
          {
            string code = options.Item2.Branch(i)[0];
            HabitatBuilding building = new HabitatBuilding(
            options.Item1.Branch(i),
            Transforms, code);
            HabitatBuildings.Add(building);
          }
        }

        private void ClearOpt()
        {
          InstanceObjects.Clear();
          Transforms.Clear();
          HabitatBuildings.Clear();
        }

        public Tuple<DataTree<string>,DataTree<string>> GenerateOptions(
        bool w2,
        bool w4)
        {
          List<InstanceObject> currentBuilding
          = this.InstanceObjects;
          DataTree<string> outputBuildingsIds = new DataTree<string>();
          DataTree<string> addedWall = new DataTree<string>();

          for (int i = 0; i < currentBuilding.Count; i++)
          {
            string[] IDs = currentBuilding[i].InstanceDefinition.Name.Split('_');
            string start = IDs[0] + "_"
            + IDs[1] + "_"
            + IDs[2] + "_"
            + IDs[3] + "_";
            string w1s = IDs[4];
            string w2s = IDs[5];
            string w3s = IDs[6];
            string w4s = IDs[7];
            string end = "_" + IDs[8];

            if (w2 & w2s == "0")
            {
              GH_Path path = new GH_Path(outputBuildingsIds.BranchCount);

              for(int k = 0; k < currentBuilding.Count ;k++)
              {
                if (k != i)
                {
```

```
              outputBuildingsIds.Add(currentBuilding[k].InstanceDefinition.Name, path);
            }
            else
            {
              outputBuildingsIds.Add(start
              + w1s + "_"
              + "1" + "_"
              + w3s + "_"
              + w4s + end, path);
              addedWall.Add(k.ToString() + "_" + "2", path);
            }
          }
        }

        if (w4 & w4s == "0")
        {
          GH_Path path = new GH_Path(outputBuildingsIds.BranchCount);
          for(int k = 0; k < currentBuilding.Count ;k++)
          {
            if (k != i)
            {
              outputBuildingsIds.Add(currentBuilding[k].InstanceDefinition.Name, path);
            }
            else
            {
              outputBuildingsIds.Add(start
              + w1s + "_"
              + w2s + "_"
              + w3s + "_"
              + "1" + end, path);
              addedWall.Add(k.ToString() + "_" + "4", path);
            }
          }
        }
      }
      return Tuple.Create(outputBuildingsIds, addedWall);
    }

    // Choose itteration
    public void ReplaceModule(int habitatBuilding)
    {
      HabitatBuilding hBuilding = this.HabitatBuildings[habitatBuilding];
      int replaceInt = hBuilding.integerReplaced;
      string replaceId = hBuilding.idReplaced;

      InstanceDefinition instanceReplace = ActiveDoc.InstanceDefinitions.Find(replaceId);
      this.InstanceObjects[replaceInt].Attributes.SetUserString("Wall_"
      + hBuilding.NumberWall, "True");
      bool done = ActiveDoc.Objects.ReplaceInstanceObject(
      this.InstanceObjects[replaceInt].Id,
      instanceReplace.Index);
      ClearOpt();
    }
    }
```

G
Results

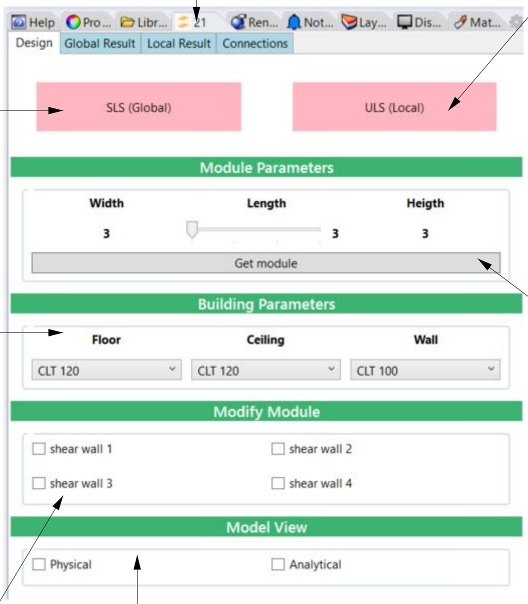
G.1. Tutorial Rhinoceros plugin

The create rhinoceros contains 3 tabs and 1 tab in development. The first tab contains all functions to modify the created design and activate the global or local structural calculation. In the second and third tab visual and numerical feedback is provided about the global and local structural requirements. The final tab contains the optimisation method for the single-step brute force method.

Load Habitat21 plugin
by **Habitat21Command**

Activate **SLS(global)** calculation by activate button, check of calculation in the left top corner. (see figure 4 and 5)

Activate **ULS(Local)** calculation by activate button, check of calculation in the left top corner. (see figure 4 and 5)



Set the **length** (slider) and click **get module** to get a module preview (figure 1. Bake/Add a module by clicking left mouse button, the module will snap to the closest the red point (figure 3). Delete a module by selecting a module en press delete. Unselect a module by pressing key escape.

Change the cross sections for the floor/ ceiling/wall components

Modify modules by adding or removing shear walls. First select a module, secondly check/uncheck a checkbox to add or remove a wall. The selected module is indiacted with a yellow sphere, hovering over a checkbox give a preview of the new module (figure 2)

Activate the physical preview or analytical preview

Controls:
Rotate the module by key: r
Select another corner by keys: 1, 2, 3 ,4
Add module: Left mouse click
Delete selected module by key: Delete
Deselect module by key: Escape

Figure 5

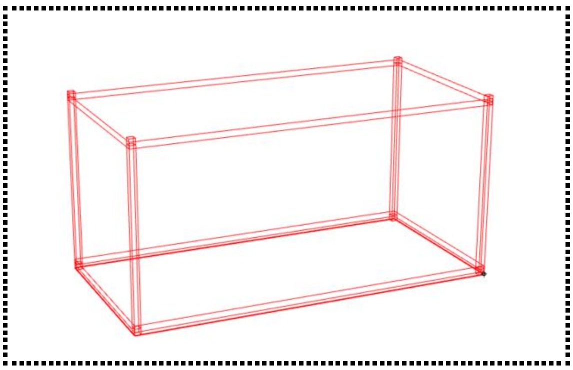


Figure 1

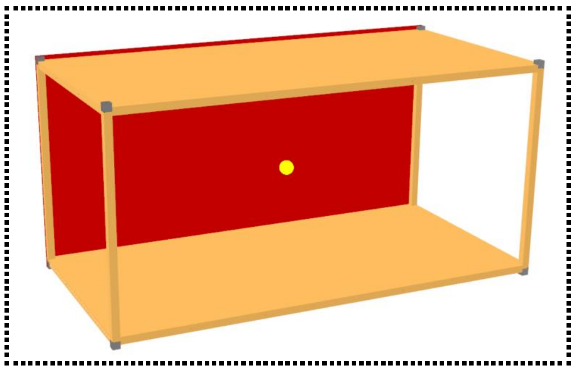


Figure 2

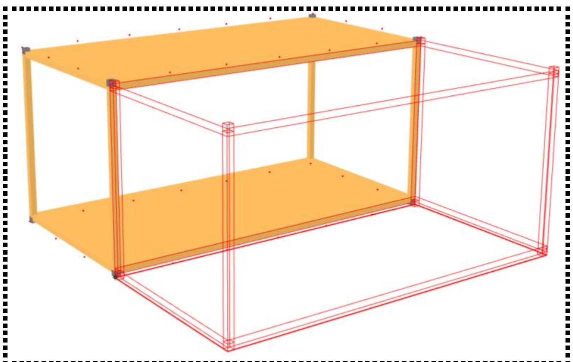


Figure 3

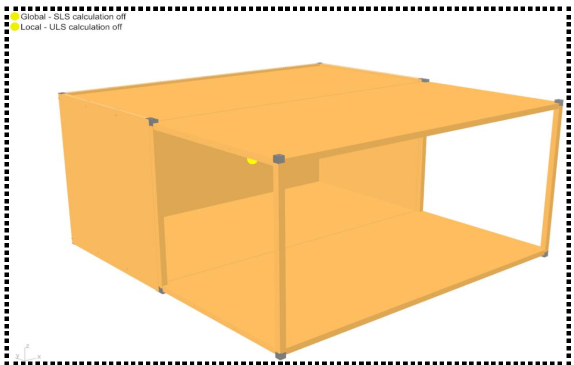
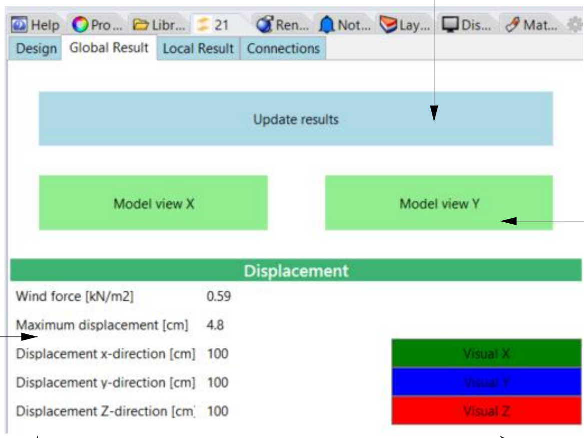


Figure 4

First click **update results** to refresh results.



Active displacment view for X and Y direction, displacements of the building are visualised with a scaling factor 10. (figure 1)

Check the displacement in x and y direction with the maximum horizontal displacement. (figure 1)

Press togglebutton to indicicate location of maximum displacement:
Green sphere: Maximum horizontal displacment in x direction
Blue sphere: Maximum horizontal displacment in y direction.
Red sphere: Maximum vertical displacment in z direction. (figure 1)

Check the maximum displacement in z direction to check if modules are supported (governing in this case). In normal situation is displacement of floor and ceiling governing.

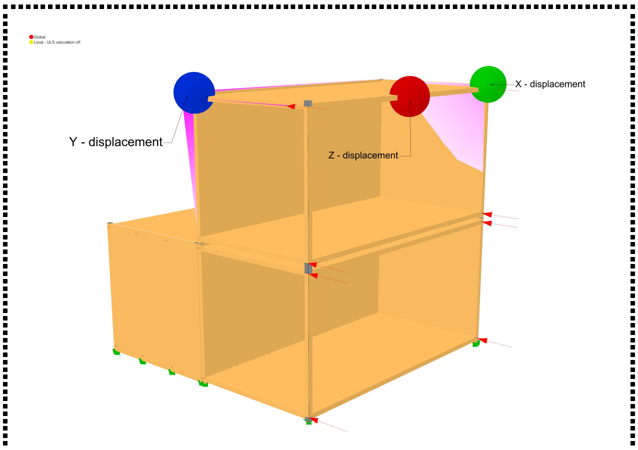
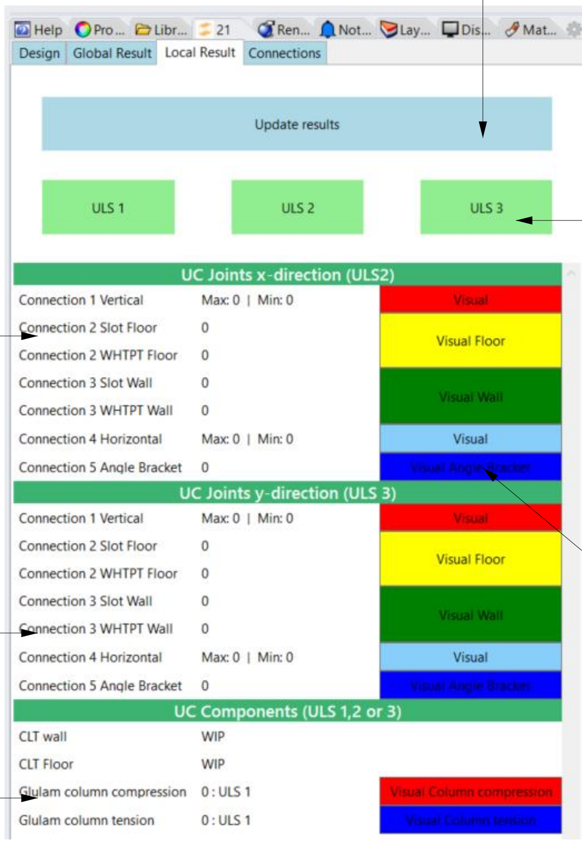


Figure 1

First click **update results** to refresh results.



View with forces and supports for ULS 1, 2 and 3. Displacements of the building are visualised with a scaling factor 10. (figure 1)

Unity checks for connections for ULS 1. Connection 1 and 4 have no unity checks, but maximum (tension) and minimum is given.

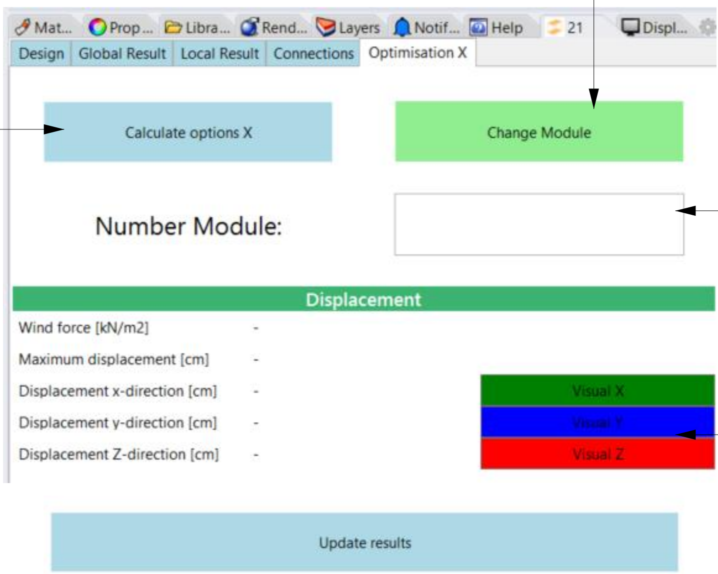
Unity checks for connections for ULS 2. Connection 1 and 4 have no unity checks, but maximum (tension) and minimum is given.

Press togglebutton to indicicate location of maximum unity check for connections.

Unity checks of timber components are given. CLT wall and floor are missing due to an error.

3) Finally, when the integer from step 2 is entered click this button to modify the model. When structural requirement is not fulfilled yet, go to step 1.

1) First click calculate options, the results of this can be seen in figure 1.



2) Secondly, when decided which module wall should be placed, enter the first number (figure 1) in the textbox.

Same Panel as global results

Click **update results** to refresh results, global result unity check is also visualised in working document.

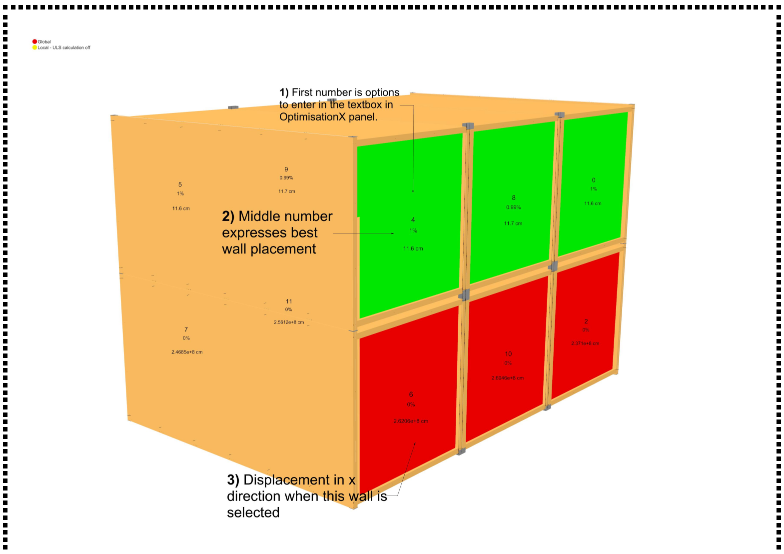


Figure 1

G.2. Results case 1

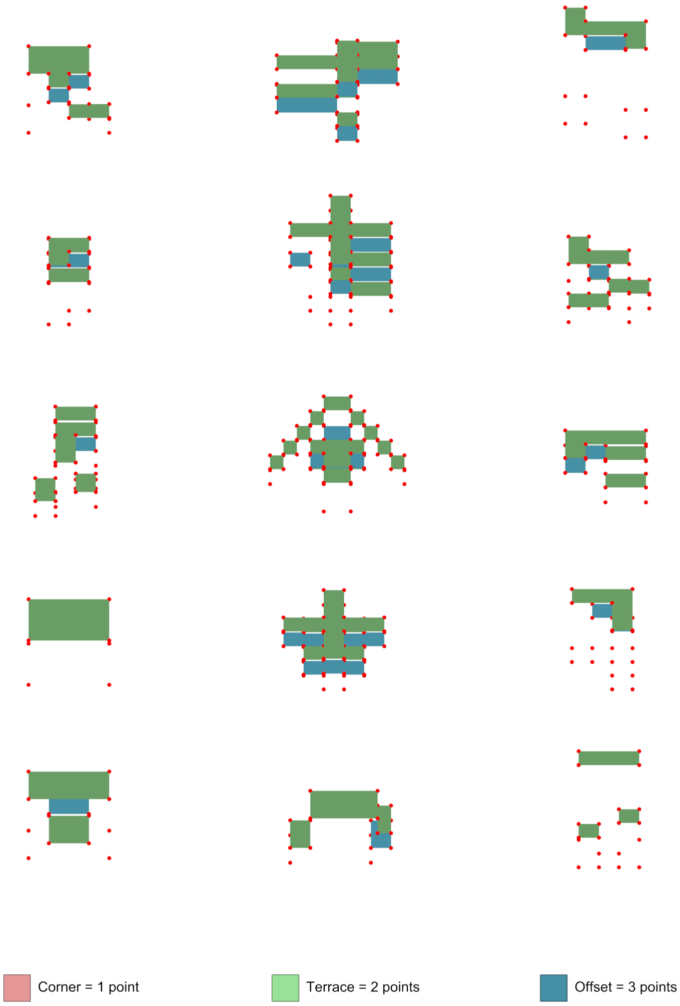


Figure G.1: Overview point count differences building typologies