

T T E

TALL TIMBER EXTENSION

*Design study for a new construction method
in the city of Rotterdam*

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ABSTRACT

In most of city centres in the Netherlands, there is a growing demand for residential housing. However, as most of the land is occupied by existing buildings, developers recur to demolish and build higher buildings, in order to fulfil the urban need. Even though demolition, may be the simplest option in technical and economic terms, it can also destroy architectural heritage, cause noise disturbances and damage the environment.

On the other hand, refurbishment can be a sustainable option in building scale, but does not provide extra capacity for the densification demands of the city.

Structural extensions can be an attractive alternative that combines the benefits of the last two options, gaining more residential capacity in city centres and respecting the architectural heritage, and at the same time, reduce environmental impact by re-using old structures.

Although, an ambitious extension can pose important technical constraints, many successful precedents have been realized. This can be an indicator that some of the complexities will partly overcome in the near future. This work aims at increasing the awareness of structural extension possibilities in existing buildings, as an alternative for ambitious architectural and structural projects, with sustainable considerations.

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PART 0

RESEARCH FRAMEWORK

1.1 BACKGROUND

In the recent years, a few master thesis in TU Delft and TU Eindhoven have been elaborated analysing the possibilities for multi-storeys additions on top of existing buildings. These works focus exclusively on the structural design from an engineering perspective, resulting in technically and economically efficient solutions. However, they do not provide insights into other architectural qualities, for instance, building expression or functionality.

In the last decade, precedents of multi-storeys building extensions have been constructed, especially in The Netherlands. These projects highlight the potentials of building extensions, by exploring technical complexities in combination with architectural design, and at the same time, achieving financially-feasible results. Nevertheless, up to date, there is still an absence of information concerning structural and architectural principles, for building extensions.

Furthermore, there are new demands for sustainability in construction products (ref), these requirements are especially important for the main structure of the building, because of the large quantities of material needed. It has been documented in the literature (ref), that engineered timber is one the most sustainable construction material given that is renewable, it requires low embodied energy for manufacturing and it can be recycled. At the same time, it can be very suited for structural extensions and other complex structures, given its lightweight and dry construction process (ref). On the other hand, there are also other technical questions, such as durability, absolute strength or fire resistance, that up to date, have restricted the use the material in technically-driven projects. With recent research in engineered timber, and their potential in high-rise structures, many of these issues have been clarified, enabling a new rediscovered use of timber in construction.

In conclusion, we can say that there is need for more research about possibilities for integrated design of building extensions using engineered timber products.

1.2 PROBLEM STATEMENT

Nowadays, there is a growing need for construction methods that are able to cope with urban growth in a sustainable way. Tall Timber buildings can be an interesting alternative, however, in congested city centers they required ample space for the building base and often the demolition of an existing building. Needless to say, this is highly unsustainable from an environmental and cultural perspective. On the other hand, extensions of buildings are often executed in a small scale, with light structural interventions. Tall Timber Extensions, can be a new construction typology that combines the sustainable benefits of timber structures, with the advantages of building high on an existing structure.

1.3 OBJECTIVES

Aim

Understanding possibilities of new construction methods with timber structures, and to what extent can they be utilised for extending existing buildings in dense cities.

Literature and case studies objectives

- Analyse precedents for building extension projects, in order to understand and illustrate "the-state-of-art" of construction and architectural principles.
- Describe lateral stability systems for tall building structures, and suitable principles for engineered timber products.
- Highlight sustainability potentials of using engineered timber structures.

- Describe principles and construction methods for engineered timber structures, and suitable applications in building extensions.

Design outline

Location - Ter Meulen, Rotterdam, The Netherlands

The rationale behind the selection of this building, was derived from the literature study of precedents and state-of-art building extensions. The new building called Karel Doorman, represents the highest precedent of building extensions using extra structural capacity of an existing structure. In the new extension, 16 lightweight extra storeys were added on top. After physical assessments and structural re-calculations of the original structure with new standards, the Ter Meulen existing structure proved to be in good condition and showed a significant undiscovered extra load-bearing capacity (Hermens, Visscher & Kraus, 2014). This will allow the exploration of ambitious structural and architectural designs.

Programme - Residential

The future programme demand of the architectural design were based on current market demands in major dutch city centers.

Boundary conditions

The design case studies will not focus on the technical assessment of the existing structure. It will consider that the existing structure is in the original condition, based on the technical data extracted during the construction of Karel Doorman.

Design objectives

- Understand urban regulations for a tall building in the specific context
- Understand and calculate composite action, fire and acoustic behaviour of CLT timber products, and comparison with conventional construction methods
- Research engineering and architectural integration possibilities in a building extension project
- Extract guidelines for architects and engineers for sustainable integration in a complex building extension

1.4 RESEARCH QUESTIONS

To what extent is a Tall Timber Extension (TTE) technically feasible?

- What is the state-of-the art of tall timber structures?
- What are the main technical considerations for a tall timber structures?
- To what extent is a Tall Extension technically feasible?

To what extent can we build a residential TTE in the intended location?

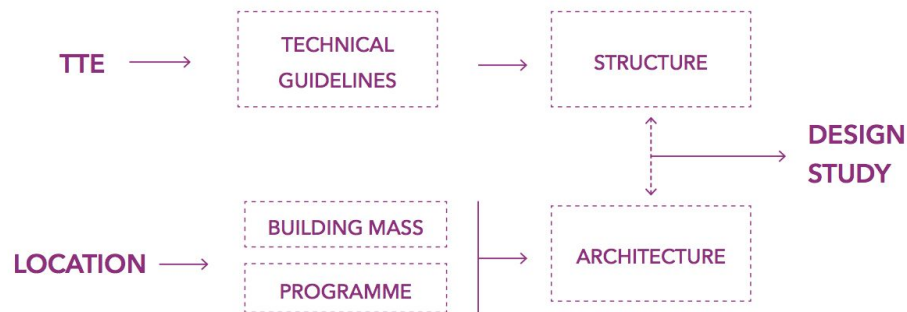
- What is a tall building and how does it apply to TTE and context?
- What is Rotterdam urban policy regarding tall buildings in the intended location?
- What are the current market demands for a tall residential building in Rotterdam?

How can we design an integrated TTE in the intended location?

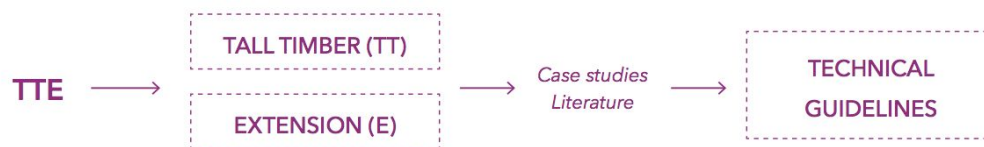
- What is the solar impact of a Tall building in the existing context?
- What are the most important structural design principles for designing timber structures?
- What are the most suitable structural systems for the TTE in the intended location?

1.5 METHODOLOGY

Understanding and designing a building can be a difficult task due to the multiple parameters simultaneously involved, especially if it is considered from a strict research perspective. In that context, an ambitious research framework is proposed with three main parts that try to integrate case studies, literature review and research by design, in three parts.



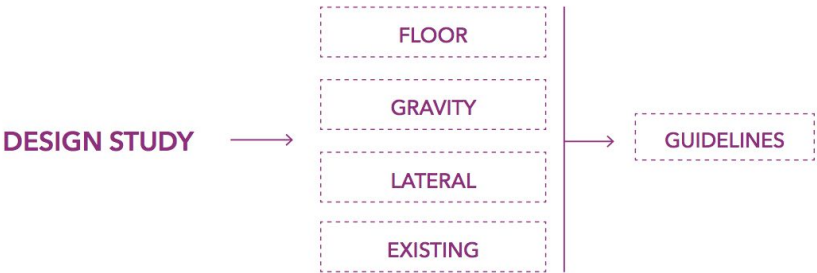
- Part 1 researches the technical considerations for Tall Timber buildings (TT) and Extensions in existing buildings (E), through an extensive literature review and case studies, in order to understand the state-of-the-art of these construction methods and extract general guidelines for the architectural design.



- Part 2, focuses on the specific urban policy and architectural demands that a tall building in the selected location has to meet. From these considerations a general building mass and programme guidelines can be used in the later architectural design.



- Part 3, intends to apply the obtained research into an specific project with both architectural and structural ambitions.



PART 1

TALL TIMBER EXTENSION (TTE)

2. TALL TIMBER (TT)

- Timber as a construction material
 - Cross-Laminated Timber
- Tall Timber today
 - Composite tall timber structures
- Built precedents
 - The Treet (Norway)
 - UBC Brock Commons (Canada)
 - Murray Grove (United Kingdom)
 - Tamedia (Switzerland)
 - Esmarchstrasse 3 (Germany)
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- Concept studies
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 - Finding the Forest Through The Trees (Michael Green)

Wood is a warm and natural material that it is commonly used in interior finishings because of the relax and softening properties. It creates a welcoming feeling inside the building. At the same time, studies have demonstrated that it can also have health benefits towards the users of the space, as it emits zero electrostatic charges, and helps in soothing and relaxing the nervous system. Regarding building physics properties, it contributes to regulating humidity inside the room, storing heat through its thermal mass and dampening sound.



Interior spaces with exposed wood (Source: Google images)

The two most common structural materials, reinforced concrete and steel, have been the preferred structural choice for tall buildings in the last centuries. Some of the reasons are their durability, stiffness and strength. The engineering advantages of those materials, contrast with their environmental and architectural perception. Whereas wood is socially perceived as sustainable and pleasant, concrete and steel are regarded as unsustainable and cold.

Needless to say, the most important difference between timber and other structural materials is its natural character. That quality defines the properties of the material, and consequently the perception of it. Trees are alive organisms that grow with sun and water, its internal structure is a complex molecular system of micro-cells that adapt according to soil and wind conditions. As a consequence, every tree is a magnificent natural assemblage of millions of micro-tubes and cells, that define the unique texture and hue of each tree.

In a way, every tree is a piece of art. As the architect Michael Green 2013 mentions, “if you cut a tree, honour that tree’s life by making something beautiful”. The russian-french artists, Marc Chagall, 1977 remarks “Art is the unceasing effort to compete with the beauty of flowers – and never succeeding”



Light wood textures. The flow of growth rings and imperfections can be compared to pieces of art (Source: Google images)

In comparison, reinforced concrete and steel are industrially produced, usually with rough finishings, and they are experienced as cold and hard materials. As a consequence, in many cases they are clad with higher quality finishings, e.g. plaster.

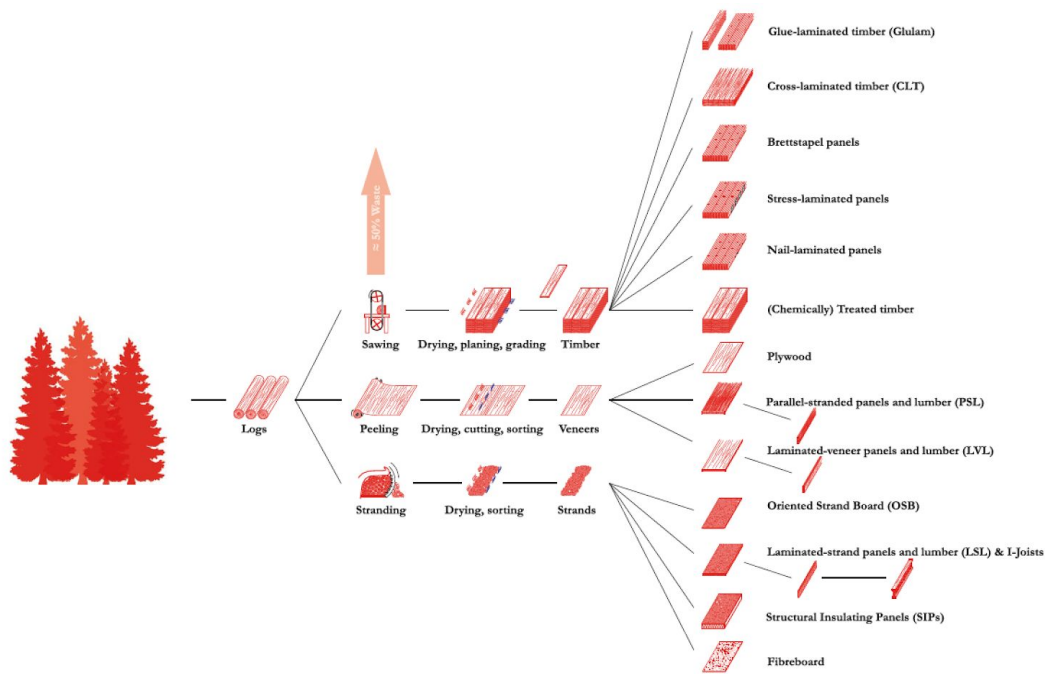
These profound differences in perception between timber and other traditional materials, should determine a different treatment regarding tall building structures. In a way, living inside a timber structure can be compared to living inside the art of nature, which can be an important quality in highly urban cities.

The timber structure of the building should evidence the “natural-soft character” of wood by making a structure that can be contemplated from inside. This means the structure should be maximally exposed to the perception of the users, reducing cladding.

The building should clearly showcase the maximum structural challenges that timber is capable to defy, including high-rise construction,. This means that a distinctive structure will have to be expressed in the architecture of the building.

2.1 TIMBER AS A CONSTRUCTION MATERIAL

In the last decades various engineered timber products (ETP) have entered the market as primary structural members. The main advantages of these products are the possibility of using greater dimensions, compared to typical sawn timber, and increased homogeneity within the material properties. This is achieved in the manufacturing process by improving material classification, separation and bonding of the smaller components. At the same time, the products are fabricated in a controlled factory and can achieve great precision. The lamination technique enables the production of large structural members from smaller pieces selected for strength and quality, from various derivative wood products strands, veneer or boards of wood. Sizes are typically limited by transportation and constructability constraints.



The processing chain of engineered timber products (Source: P.H. Fleming)

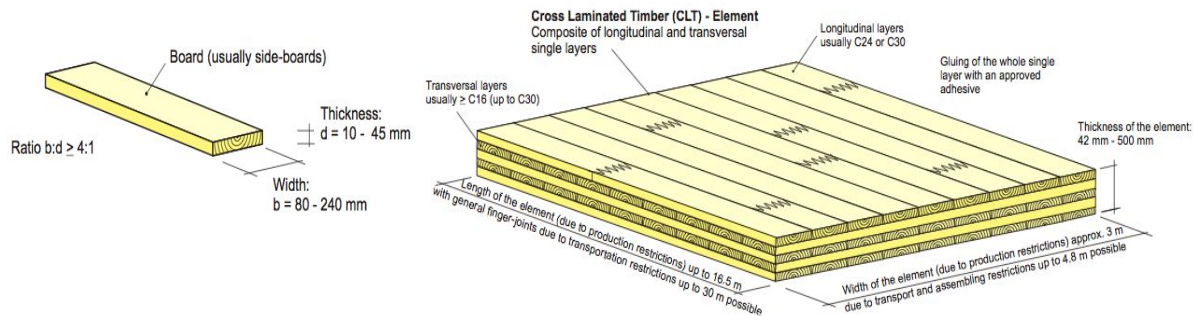
Some of the most common engineered timber products are:

- Glue Laminated Timber (GLT)
- Cross Laminated Timber (CLT)
- Parallel Strand Lumber (PSL)
- Laminated Veneer Lumber (LVL)
- Laminated Strand Lumber (LSL)
- Oriented Strand Lumber (OSL)

These products can be extremely versatile and can be used in diverse building typologies. They are made of renewable resources and manufactured with lower carbon emissions than steel or concrete. At the same time, the material can be easily customised for different functions, shapes, structural capacity, aesthetic qualities and finishing treatments. These products have been traditionally used in large halls and bridge engineering, commonly in linear structural elements such as truss systems or solid-web girders.

Cross-Laminated Timber

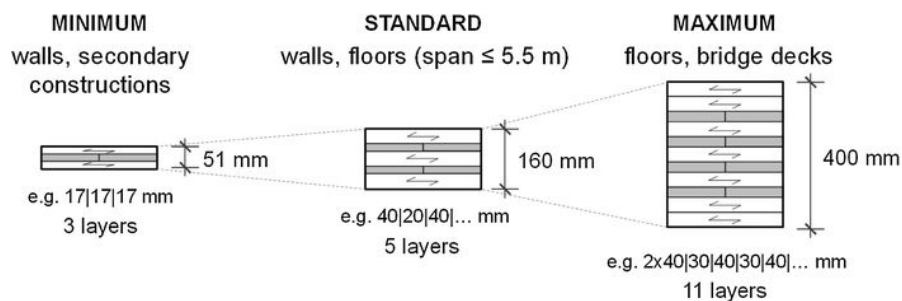
A new type of engineered product was developed in Graz, Austria, in 1981 by Dröge and Stoy, under the German term "Brettsper Holz (BSP)". Later in 2000, the term was translated into English under the name Cross Laminated Timber (CLT), by Schickhofer and Hasewend. CLT is a stand-alone laminar large-sized plate, which is commonly composed of an uneven number of timber layers (3, 5, 7 or 9). Each of the timber boards is placed side-by-side, and arranged crosswise to each other at an angle of 90°. The width of the boards are usually 80-240mm, with thickness from 10-45mm (depending on the producer this can be up to 100mm). The width to thickness ratio should be defined as $w/t = 4:1$. Commonly, coniferous wood species (spruce, pine and fir) are used, but other deciduous species (ash and beech) may be more popular in the future. For the purpose of this report, Norway spruce (*Picea abies*) will be considered. CLT-elements can bear in and out-of plane loading. Typical maximum sizes can range up to 18-30 meters, and widths up to 3-4.8 meters. Typical maximum thickness can range up to 300-400mm.



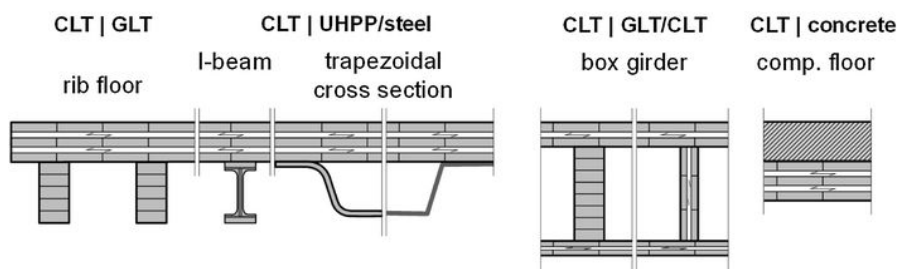
Basic board for CLT production and configuration of CLT element with 5 layers (CIE5124 Handbook)

Apart from the natural sustainable characteristics of its base material, there are many important advantages of CLT, in comparison with other timber products. The main innovative aspect behind CLT relies on its thickness which allows high dimensional stability in-plane due to the cross-wise layering, and as a result greater strength and stiffness properties. CLT panels can be used as load bearing floors, walls, roofs or function as diaphragms or shear walls.

If used as a laminar product, it allows load bearing in and out-of plane. If used as a linear structural element it shows promising resistance against shear in-plane and tension perpendicular to grain. Openings can be pre-cut in the factory or on-site, ranging from smaller holes for piping and ducts to opening for doors and windows.



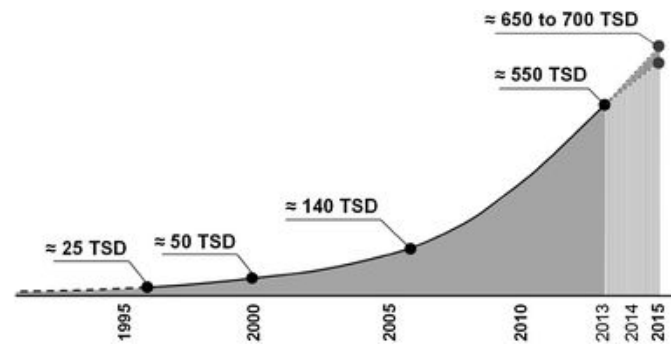
Layout examples of CLT. (Source: Brandner, 2016)



Examples of composite structures with CLT. (Source: Brandner, 2016)

The first European product standard for CLT was EN 16351 (2014). Further standardisation, including regulations for testing, design and execution are needed. For instance, CLT is going to be included in later versions of European timber design code Eurocode 5.

Current construction activities suggest a worldwide interest on CLT construction, with a constant rise of production volume and double-digit annual growth rates within the next decade. With the recent trends a continuous shift towards tall timber buildings is being expected.



Development of worldwide production volume of CLT until 2013, and forecast until 2015 (m3). (Source: Brandner, 2016)

2.2. TALL TIMBER TODAY

The following chapter summarises the precedents “the claimed tall” timber buildings constructed in the past decade. This study is carried out with the intention of understanding structural systems adopted for tall timber design. In order to provide light for future possibilities of structural design of tall building using timber, 7 built case studies and 2 concept studies have been studied.

Building			Structural system		
Name	Location	Stories	Vertical	Lateral	Floor system
Esmarchstrasse 3 (E3)	Berlin, Germany	7	Post+Beam GL Wall <i>Brettstapel</i> Mini-Core RC	Post + beam Timber infills Steel X-bracing ties. Mini-Core RC	TCC (RC100)
Kingsgate House	London, United Kingdom	7	Wall CLT Ground RC	Core + wall CLT	CLT + SC
Strandparken	Stockholm, Sweden	7	Wall CLT TF	CLT+ TF Steel tension rod	CLT + GL beam
WIDC	British columbia, Canada	7	Column GL/LVL Wall CLT/TF	Core CLT	GL + CLT
T3	Minneapolis, United States	7	Column GL Core RC Ground RC	Core RC	Beam GL + NLT + SC
UEA	Norwich, United Kingdom	8	CLT wall	Core + wall CLT	CLT + SC55
Limnologen	Växjö, Sweden	8	Wall CLT TF Column GL Ground RC	CLT + TF Steel tension rod	CLT + GL beam
LCT One	Donbim, Austria	8	Column GL Wall RC Ground RC	Core RC	HBV (RC80)
Holz8	Bad Aibling, Germany	8	Wall TF	CLT+ TF Steel tension rod	CLT + SC
Butler Square	Minneapolis, United States	9	Wall masonry Column timber	Wall masonry	Timber beams

Cenni di Cambiamento	Milan, Italy	9	Wall CLT	Core + wall CLT	CLT
Murray Grove	London, United Kingdom	9	Wall CLT Ground RC	Core + wall CLT	CLT + SC55
Dalston Lane	London, United Kingdom	10	Wall CLT Ground RC	Core + wall CLT	CLT + SC55
Forté Building	Melbourne, Australia	10	Wall CLT Ground RC	Core + wall CLT	CLT + SC70
Wenlocke Road	London, United Kingdom	10	Column steel Wall CLT Core RC Ground RC	Core RC	CLT
* 457 West 18th (Robinson et al. 2016)	New York, United States	10	Core + Wall CLT Column GL	Core + wall CLT	CLT + SC
* FTT 12 (Green and Karsh 2012)	Vancouver, Canada	12	Column GL Wall CLT	Core CLT	CLT + SC40
* Framework (Robinson et al. 2016)	Portland, United States	12	Core PT CLT Column GL	Core PT CLT	Beam GL + CLT + SC
Treet	Bergen, Norway	14	Column GL Modules TF/CLT Ground RC	Truss frame GL	Stacked modules 5-10th levels Power storeys. Truss GL + RC200
UBC Brock Building	Vancouver, Canada	18	GL Core RC Ground RC	Core RC Edge beam steel	CLT
* NEWBuildS (NEWBuildS 2015)	Vancouver, Canada	20	Column GL Wall CLT Ground RC	Core-wall CLT - link steel	HBV (125mm RC)
* FTT 30 (Green and Karsh 2012)	Vancouver, Canada	30	Wall CLT	Core-wall CLT Link beams Steel	CLT + SC40
*Post-tensioned CLT walls with concrete outriggers (Xia & van de Kuilen, 2010)	Shanghai, China	40	Wall CLT	Core-wall CLT Post-tensioned bars Concrete outriggers	250 CLT
* CJTF - SOM (SOM 2013)	Chicago, United States	42	Column GL Wall CLT Ground+1 RC	Core+wall CLT Link beams RC	Floor CLT Spandrel RC
* All Timber - SOM (SOM 2013)	Chicago, United States	42	Column GL Wall CLT Ground+1 RC	Core-wall CLT - link GL	CLT

Where, CLT: Cross-laminated timber, CLT-ST: Cross-laminated timber with steel tension rods inside, RC#: Reinforced concrete of thickness #mm SC#: Non-structural concrete screed of thickness #mm SC: Non-structural concrete screed of unknown thickness TF: Timber-frame panel construction GL: Glue-laminated timber HBV: RC-GL: Composite structure of Reinforced Concrete + Glulam timber Ground RC: Concrete ground floor + transfer slab Ground+1 RC: Concrete ground and first floor + transfer slab NLT: Nail-laminated timber PT: Post-tensioning *Unrealised buildings at the time of writing

Composite tall timber buildings

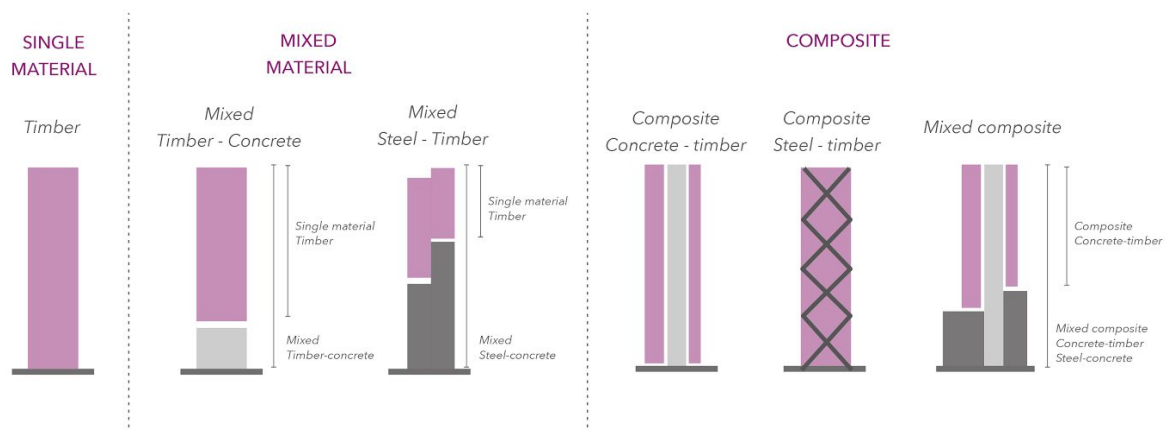
All present tall buildings with any recognised claim to tallness, are constructed using steel, concrete or a combination of the two as structural materials. However, in a strict sense, very few building are constructed purely with a single material, reinforced concrete contains steel rebars, and steel structures typically contain concrete flooring decks. Nevertheless, these structures are commonly considered single material, concrete and steel respectively as those materials provide the main structural elements. The same can be said for single tall-structure built entirely out of timber, as foundations and usually ground floor are constructed with concrete. For that reason, it is important to define what it is a composite-timber building, and to what extent that differs with the so-called tall timber building. According to CTBUH 2015, the definition of tall building regarding structural materials is at follow.:

“Single material”, when the main structural elements are constructed with that material.

“Composite material”, when different materials respond to different structural functions, for example, typical concrete core and steel frame, concrete resists lateral loading, and steel, vertical loading. In the case of timber, a concrete-timber building would be a large concrete core resisting lateral loading with timber vertical element, walls, columns and floors resisting gravity loading. A timber-concrete building, would use less amount of concrete, for instance, CLT shear walls and core connected by concrete linking beams. In this case, concrete only absorbs the shear stresses between the timber elements, and couple their behaviour.

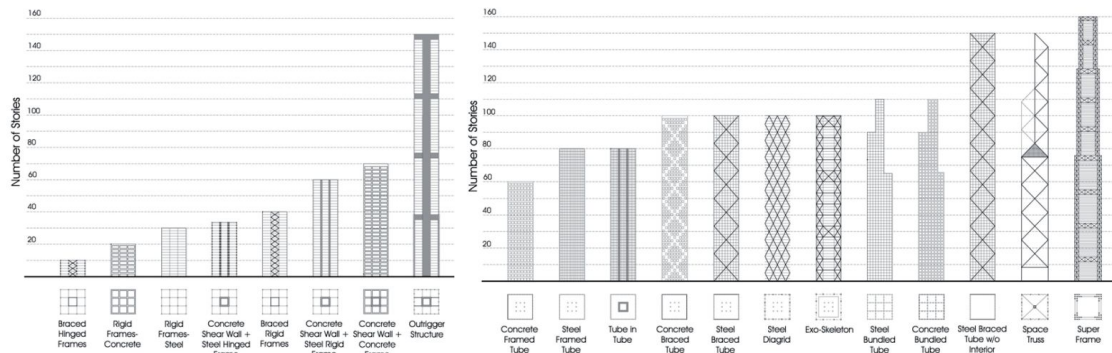
“Mixed-structure”, when there are different structural systems above/below each other. In this case, upper structural system can be thought as a separate structure founded on the lower.

As mentioned before, It is typical in any timber construction to use concrete in the ground floor, in order to avoid penetration of water through end-grain of timber elements, and thus enhance durability. Concrete may also be more suitable for transfer structures required to accommodate a more open structural grid at ground level (for retail spaces and entrances) (IstrucE & TRADA, 2007). A building constructed in concrete to the first floor and 20 stories of timber above, would be considered 21-story mixed timber/concrete building according to the above definition.



Examples of building typology by structural material

Khan (1969) proposed a schematic relationship between typologies of structural systems and characteristic height ranges for tall buildings, based on structural efficiency. In addition, other investigators (Falconer 1981; Iyengar 2000; Ali and Moon 2007; Gunel and Ilgin 2007) continuously updated the original structural systems charts during the last decades, incorporating newly developed systems:



Classification of lateral force-resisting structural typologies for tall buildings, interior (left) and exterior (right)
(Source: Ali and Moon 2007)

Those indicative heights have only been established for steel and concrete structures. Differences in density, strength and stiffness of the structural material will vary the building heights at which the structural system is economic. Indicative heights for economic tall buildings structural systems have yet to be established.

It may be expected that the relatively low stiffness and mass of timber will lead to lower slenderness ratios for a tall timber structure. An example of this can be "The Treet" building in Norway, a 14 storeys construction with braced frames across multiple storeys, in comparison with a single storey bracing required in a steel structure.

Consequently, it may be suggested that tall buildings using structural timber may be feasible at lesser heights than their counterparts in steel or concrete structures.

Malo et al. (2016) argued that the comparable specific stiffness and strength between steel and glulam timber would result in similar stiffness and mass with braced steel-structures. The author estimates that the bulk density of "The Treet" building is around 140Kg/m³. Previously researchers had estimated that the bulk densities for typical steel and concrete buildings may be around 160Kg/m³ and 300kg/m³ respectively. (Cho et al. 2004, Yang et al. 2004, Huang et al 2007). Malo et al. (2016) also calculated the fundamental frequency of The Treet building to be slightly greater than 1Hz, establishing a preliminary indication that a braced glue-laminated building may be designed to have similar mass and exhibit similar dynamic behaviour to a braced steel-frame building.

Structural material	Estimated bulk density	References
Timber	140Kg/m ³ (The Treet)	Malo et al. 2016
Steel	160Kg/m ³	Cho et al. 2004, Yang et al. 2004, Huang et al 2007
Concrete	300kg/m ³	

Estimated bulk density for a typical tall building, calculated as dead load divided by gross building volume

Braced-frames systems in glulam timber and steel, have similar mass and exhibit similar dynamic behaviour, but require bigger-sized structural elements.

2.3 BUILT PRECEDENTS

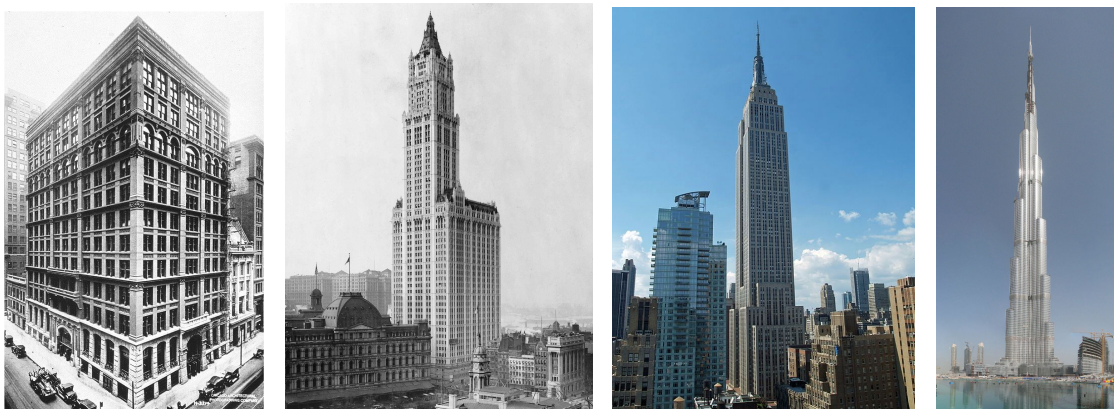
Timber was an important structural material for churches and spires that would have been regarded as tall structures until twentieth century. Horyuji Pagoda (32.5m), Yingxian Pagoda (67m) and Barsana Monastery (56m) are notorious examples of the use of timber as a structural material in the past. With some of those structures being older than 1000 years, there are a great proof of the durability of timber if preserved and detailed correctly. The tallest timber structure ever made up to date, was the 190m Ismaning radio tower in Germany, which unfortunately became in disuse and collapsed in 1983.

According to CTBUH 50% of the building height must be occupied by usable floor area to be considered tall building. According to that definition the aforementioned structures can not qualify as tall buildings, (i.e., Yingxian pagoda contains only 5 stories because of the dense and intricate floor system of stacked joints)



From left to right: Horyuji and Yinxian pagodas; Barsana monastery and Ismaning tower (Source: Google images)

The first tall structure 10-storys Home Insurance Building in Chicago (1885). Immediately after, a competitive race started, in order to have the highest building in the world, 60-storys Woolworth building in New York (1913) or 102-storys Empire State Building (1931) (Gottmann 1966). The engineering developments in the last centuries lead to structures never seen before. Today, the tallest building structure in the world is 163-storys Burj Khalifa in Dubai (2010).



From left to right. Home Insurance (1885), Woolworth (1913), Empire State(1931), Burj Khalifa (2010). (Source: Wikipedia)

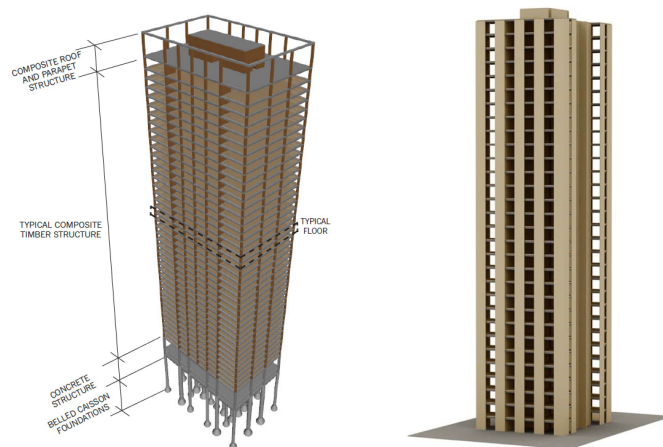
Even though, we cannot say that the skyscraper competition is over, it is certain that today, the technological developments are shifting towards sustainability, using less resources and minimising the impact in the environment. In this crucial aspect is where timber material can play a vital role for tall structures, as it requires less energy for production and is able to sequester CO₂ from the environment.

In the recent years, several buildings have been built claiming the title of “highest timber building in the world” 9-storys Murray Groove in London (2008), 10-storys Forté in Melbourne (2012) or 14-storys The Treet in Bergen, Norway (2015), 18-storys timber-concrete composite UBC Brock Commons in Vancouver, Canada (2017).



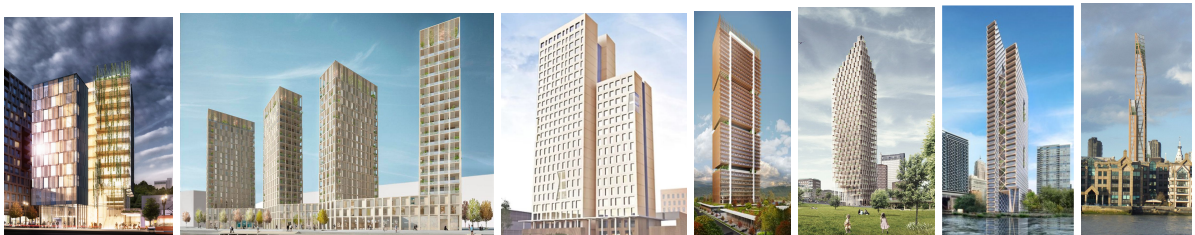
From left to right. Murray Grove (2008), Forté (2012), The "tree" (2015), UBC Brock Commons (2017)
(Source: Google images)

Needless to say, these structures are far from the heights achieved with concrete or steel. Other feasibility studies such as 42-storeys Timber-Concrete Tower Research by SOM (2013) or 30-40 storeys FTTT by Michael Green (2012), suggest that very significant increases in height of timber buildings may be possible in the coming years.



Tall timber feasibility research projects. SOM, 2013 (Left) Michael Green, 2012 (right)

In the last 2-3 years, several conceptual designs for tall timber towers phase have been released online. Even though, many of them may not be feasible from a structural point of view, they reflect the ambition of architects and users to create taller timber buildings, and the marketing associated with it.



Recent conceptual designs for timber towers worldwide. (Sources: Various architectural magazines)

The rapid development of tall timber structures worldwide, the amount of online references, (Michael Green, SOM, ARUP...), and initiatives from governmental programmes (Canada, Austria...) suggest that there is a growing interest in using timber as a structural material for high-rises, and achieving the title of "the highest timber building". In this chapter, some of the most representative timber mid/high-rise buildings in the world have been analysed, together with unrealised technical studies. In the process, a deep understanding of systems and construction methods for tall timber structures have been achieved. At the same time, critical issues such as fire protection, acoustic considerations and structural connections between elements have been researched.

The Treet (Norway)



Render of the building (Source: Wood skyscrapers) and building during construction (Source: Global timber forum)

General information

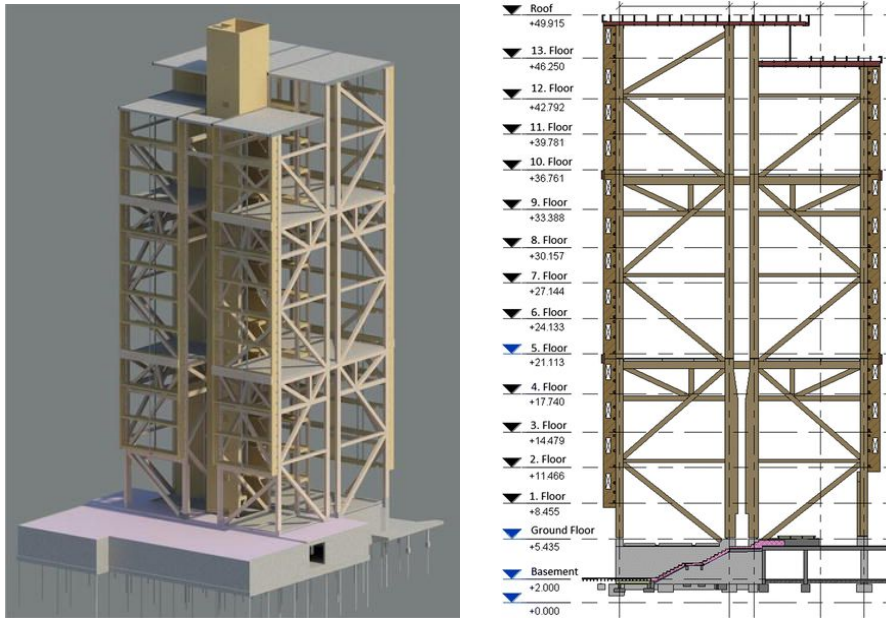
Location	Bergen, Norway	Construction	2014-2015
Architect	Artec	Height	52.8m - 14 storeys
Client	BOB	GFA	5830m ²
Engineer	Sweco	Basement	920m ²
Glulam and CLT structures	Moelven	Glulam	550m ³
Building modules	Kodumaja	CLT	385m ³
Design	2011-2013	Apartments	62 units

The project started with the ambition to make a high-rise timber building, in a cost efficient and sustainable way. Secondly, it was desired to achieve maximum degree of prefabrication, and industrial manufacturing. Each prefabricated unit is made with double walls and floors, in order to comply with acoustic and passive house regulations. The building contains 62 residential units a gym on the 9th floor and a roof terrace for the residents

Structure

The structural system can be explained as a cabinet rack with filled drawers. (Abrahamsen and Malo, 2014). The "cabinet rack" is formed out glulam trusses frames, that provide the lateral stability and main load-bearing system, and the "drawers" prefabricated CLT modules that form the load-bearing secondary structure.

The structural design of the glulam trusses is based on previous experiences from modern timber bridge structures. The base of the building is a rectangle of 23 x 21 m. The height is 45 m, and the maximum vertical distance between the timber components is 49 m.



Left, 3D view of structural model showing timber glulam frames and concrete slabs. Right, vertical section of load bearing structure.

Glulam trusses

The lateral stiffness is provided with glulam trusses along the façades. The 66 residential units are formed with prefabricated modules made with CLT panels, and are supported by the load bearing structure. The modules have no significant contribution to the global stiffness of the building.

Power storeys

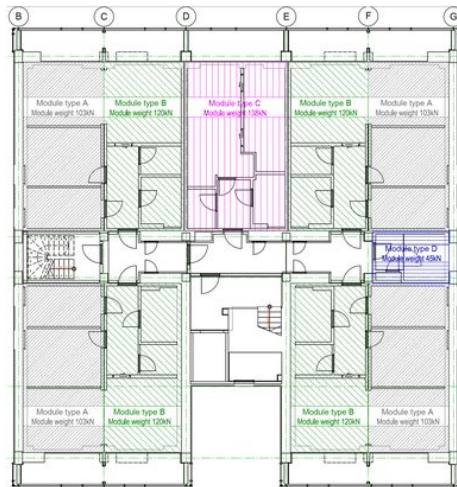
The glulam trusses are strengthened at 5th and 10th levels, with extra glulam elements and a prefabricated concrete slab. The concrete slab is incorporated to connect the trusses and at the same time increase the mass of the building and hence to improve the dynamic behaviour. (Bjertnæs and Malo, 2014). An additional function of “the power storey” is to serve as a base for the modules stacking.

Stacked modules

The modules are stacked vertically up to four levels on top of the “power storeys”, ground floor, 5th and 10th levels. As they are not participant on the lateral resisting system they are only connected to the top of the concrete slab, and there is no connection at any point with the glulam load bearing structure.

The building is created in a U-shape for architectural and aesthetic considerations.

The modules have two main dimensions 4 x 8.7m and 5.3 x 8.7m.



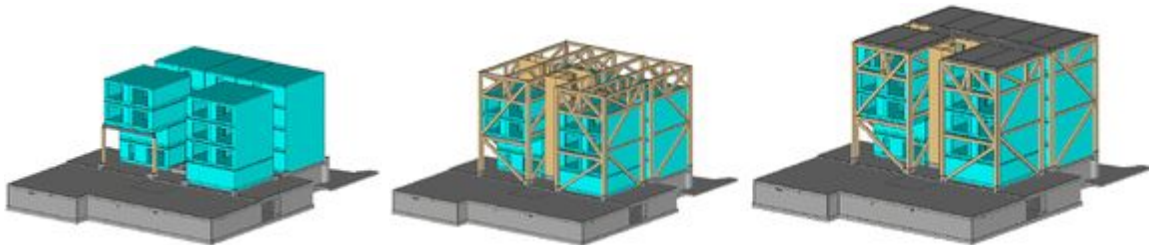
Typical plan of the building

Erection

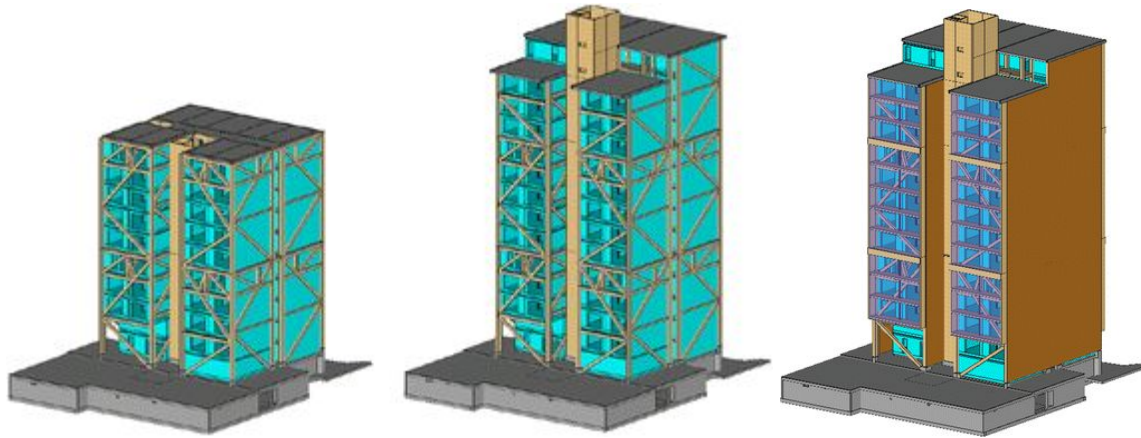
The building is composed out of relatively large prefabricated elements that are connected on site. A tower crane and a climbing scaffolding systems are used during the building erection. The glulam frames are fabricated in as large parts as possible, limited by transportation. Temporary roofs are used to protect the building modules and glulam elements from moisture.

Figure 5. shows the building erection sequence. From left to right:

1. Construction of concrete foundation and parking garage
2. Stacking of 4 levels prefabricated housing modules
3. Glulam frames are lifted and placed in between the modules. (The frames contain pre-installed slotted-in steel plates, for fast dowels connections on site)
4. Installation of modules in level 5, strengthened storey
5. "Power storey" or finalisation of concrete deck on top of level 5 (This is the new base level for the next 4 levels of stacked modules)



From left to right: Step 1-2, foundation and 4 levels of modules. Step 3, Installation glulam trusses. Step 4, Installation module 5th level and concrete deck



The steps 2-5 are repeated, using level 5th as the new base for the building process. In the last step, the external weather skin (glazing, balconies) is attached to the glulam frames and the building is finalised.

Fire design

90 minutes resistance for main load bearing system. Glulam trusses and CLT prefabricated modules

60 minutes resistance for secondary load bearing systems. Corridors and balconies

The structural fire design is performed according to Eurocode 5 (CEN 1995-1-2 2004, CEN 1995 2004), using the effective residual cross-section after charring. A charring rate of 0.7mm/min results in a charring depth of 63mm after 90 min. This has implications for the steel-timber connections, that must have a minimum distance from any exterior glulam surface. The steel dowels ends and steel plates are placed 65 and 108mm from the timber outer surfaces. These measures are on the safe side, as it is not likely that the steel dowels will contribute to an increased heat flux towards the steel plates. Furthermore, all gaps between connected timber members and slots for steel plates are protected with intumescent fire seals.

A collection of fire protection measures are incorporated into the building:

- Fire stops in the façade every second storey, using horizontal glulam elements of the external trusses
- Fire resistant lacquer type Teknosafe 2407 and 2467 for the timber elements in escape routes
- Sprinkler systems for early suppression of fire
- Pressure system in escape stair shafts for safe evacuation.

Detailing

Typical column cross-section dimensions are 405x650 and 495x495mm, and the typical diagonal cross-section is 405x405mm. All glulam elements are connected by slotted-in steel plates and dowels. This is a high capacity connection used in bridges and large buildings in Norway. In most cases, 3 steel plates of 8mm thickness and 12mm dowels are used. Figure 5 shows the splicing of the columns, in order to fulfill tolerances mounting gaps are introduced between the column elements. The gaps are filled with a high-strength expanding acrylic mortar after installation. The three steel-plates are located in 10mm wide slots, with in-between distance of 80mm centrally located in the cross section. The length of the dowels is in most of the cases 275, and hence, they do not extend to the glulam surface.

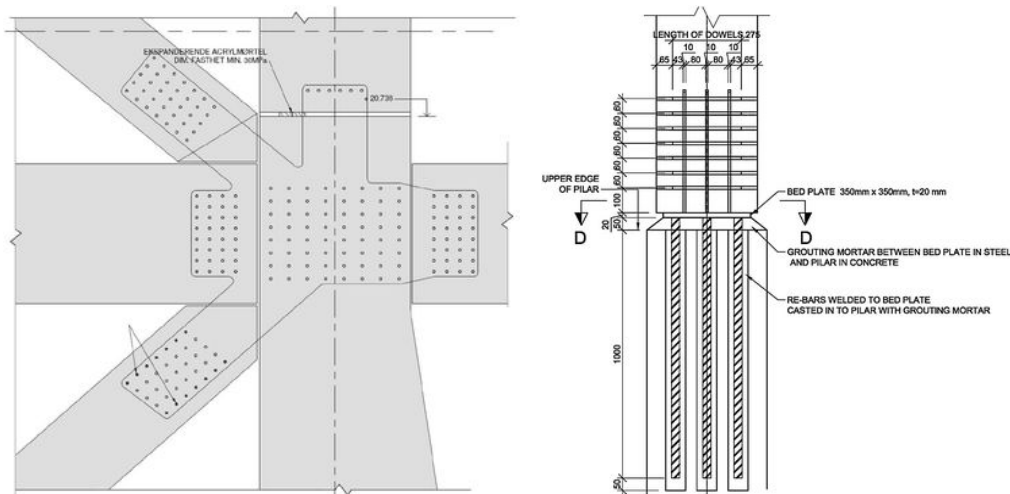


Fig. 1. Left, side view of connection with slotted-in steel plates and dowels, right, cross-section anchoring of glulam to concrete (Source: Malo, Abrahamsen & Bjertnes, 2016)

More than 100 vertical and tilted steel core piles are driven into the bedrock, 5 meters below the garage. Some of the foundation piles must also handle tension forces. When the building is exposed to wind loading, some of the glulam members receive tensile forces. For that reason the building is anchored to concrete pillars by the use of bed steel plates and joints based on slotted-in-connections as shown in figure 5.

There is a theoretical clearance of 34mm between prefabricated modules and glulam trusses, in order to avoid possible horizontal movements of the modules and consequently the development of interface forces. The modules are stacked into the interior of the building in an ordinary way, placing body sills on prepared small concrete foundation walls. The contact stresses because of the deadweight ensures the connection.

Durability and safety

External cladding and glazing of the building is attached to the load bearing trusses and balconies. The wind load will not affect the residential modules directly, except during the erection phase.

In order to increase durability and reduce maintenance all timber elements are protected from rain and sun.

Service class 1 and 2 are used of indoor members and members on cold sides.

The structure is designed in order to withstand the failure of some members without collapsing. For instance, the corridor structure is dimensioned in order to resist the failing impact of the higher corridor. The glulam truss members are designed to take extra force in case of truss member is removed.

Materials

All the main load-bearing elements are made out of timber. Glulam is used for truss and cross-laminated timber (CLT) for elevator shafts, staircases and internal walls.

For the structural design, the properties for glulam classes GL30c and GL30h according to EN 14080:2013 (CEN 14080 2013) have been used. The CLT specifications are bending strength, $f_{mk} = 24 \text{ Mpa}$, and properties similar to C24 structural timber. Protected timber elements and CLT modules are made with untreated Norway spruce. Elements subject to weathering are made out of Nordic pine treated with chromated copper arsenate.

The steel plates have a steel grade of S355 hot dip galvanised. The dowels are A4-80, acid proof stainless steel. The galvanised steel ensures that rust water will not discolour timber during the erection.

Structural modelling

The software Robot Structural Analysis Professional 2013 was used for the global analysis of the building according to design codes of Eurocode 5 (CEN 1995 2004).

For the dynamic analysis, the concrete weight of the basement was set to zero and the basement was fixed in the horizontal direction at the bottom level. The dynamic analysis includes only the axial stiffness of the piles and vertical elements of the basement structure.

The first four levels of stacked modules are not included in the FEM-model as they are not connected to the trusswork. The modules in the "power storeys" were only modelled by added mass to the truss work. All modules should carry a live load of 2KN/m², and 30% of this load was added as additional mass for the dynamic analysis. ULS check was decisive for most structural dimensions. A few elements were governed by fire design. The highest compression force in a column is computed as 4287KN. The highest tensile force in a column is 296KN. A highest tensile force in a diagonal is calculated as 930KN.

The effect of possible slip at the joints is not included in the design, but the sensitivity to joint slips was concluded to have minor impact on the force distribution, as well as on the fundamental frequencies and displacements.

Wind-induced vibrations

The building lightweight leads to wind-induced vibrations, and are one of the most important structural considerations. Calculated maximum acceleration for 13th floor is slightly higher than recommended by ISO 10137 (2007), but it is considered acceptable. The 12th floor will have accelerations below recommended value. Based on Boggs 1995 research, some of the residents on the top floors in rare cases may feel some vibrations, but it is unlikely that they will become uncomfortable.

Special structural typology with glulam braced frames for main load-bearing structure, and stacked prefabricated modules for housing units and secondary structure. The structural effects of the modules installed solely to the concrete slabs at certain levels, do not lead to problematic dynamic behaviour, as module stacks behave as rigid bodies. A tolerance of 34mm between modules and glulam frames is considered in order to avoid interferences or damages on the modules. Experimental tests and simulations have proven that 4 storeys stacked prefabricated modules are much stiffer and have considerably higher fundamental frequencies than the overall structural system with glulam frames.

UBC Brock Commons (Canada)



3D drawing of the building and perspective section. From bottom to top: Concrete structure with CLT canopy, encapsulated mass wood structure and exposed mass wood structure. (Source: Acton Ostry Architects)

Location	Vancouver, Canada	Construction	2017
General Architect	Acton Ostry	Height	53 meters - 18 storeys
Timber Architects	Hermann Kaufmann	GFA	14000m ²
Structural engineers	Fast + Epp	Programme	404 student housing
Fire consultant	GHL	CLT panels	1973m ³
Client	University of British Columbia	Glulam columns	260m ³

At the time of writing the building was the tallest hybrid-timber construction in the world. Founded by University of British Columbia, the building aims at demonstrating the viability of mass timber structures for the construction and architecture industries. One of the design intentions was to keep the building as simple as possible, emphasizing the use of economic and sustainable timber construction. The combination of timber and concrete was used for achieving a more economical structural system.

The reductions of carbon emissions are estimated as 2563 tonnes of CO₂, or the equivalent of 490 cars of the road per year. This robust system is able to meet the new seismic design requirements for National Building Code of Canada.

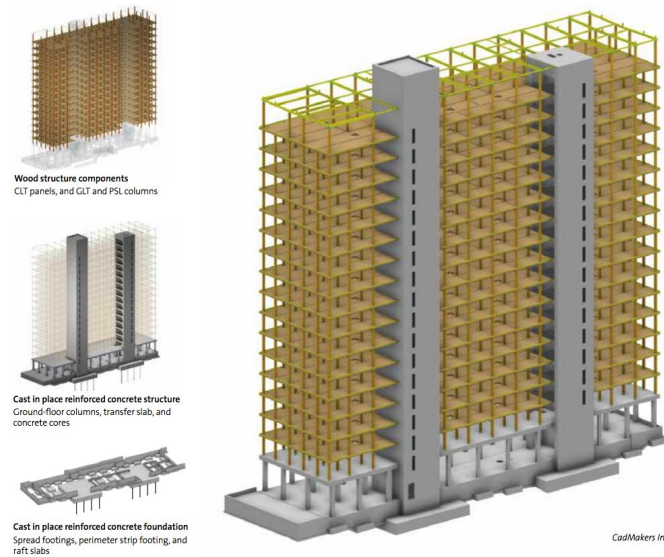
Structure

The building structure is composed of reinforced concrete columns on the ground floor and concrete transfer slab on the second floor, two reinforced concrete cores, mass-timber slabs and columns the upper floors, and a steel perimeter beam at each floor. The beam stiffens the edge of the CLT panels and supports the building envelope. The concrete column grid is 5x5 meters while the timber upper floors is 4x2.85m.

The concrete second-floor-slab acts as transfer slab between concrete and timber structures, and allows the ground-floor structural grid to be different from the above timber grid.

In the first 2-5 storeys due to the high loading areas, glulam columns are replaced with PSL (Parallel strand lumber) with high compression strength. Typical cross sections are 265x265mm from 2-9 storeys and 265x215mm from 10-19 storeys. Point connections between the column and slab consists of hollow structural section steel assemblies.

- Vertical loading is carried down by the timber structure
- Lateral loading is resisted by the two concrete cores



3D schemes of the structural system of the building. From left to right, timber structural elements, concrete cores, and concrete foundation. (Source: Cadmakers Inc)

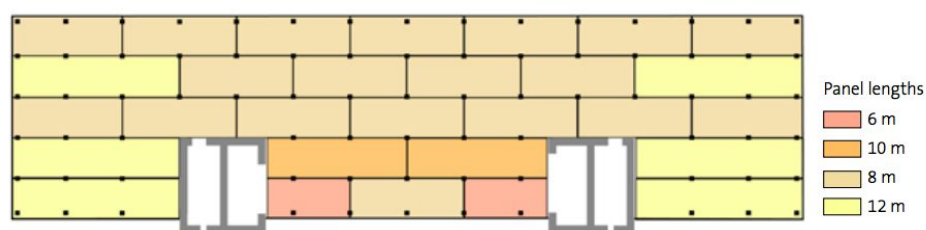
The building foundation is composed of 2.8 x 2.8 x 0.7m reinforced spread footings and 250mm thick wall on a 600x300mm thick strip footing at the perimeter of the building.

Below each core, there is a 1.5m raft slab, that includes soil anchors with 1250 KN tension capacity.

The floor is composed out of CLT panels, 169mm thick, made of five layers of timber. The panels act as two-way slab diaphragm, similar to concrete flat-plate slab. The panels are joined together with 150x25mm plywood splined screwed to each panel.

The panels are four different lengths: 2x6m, 2x10m, 19x8m and 6x12m.

A 40mm concrete topping completes the assembly for acoustic insulation and extra fire protection. At the same time, concrete increases the weight and stiffness of the assembly, and helps reducing vibrations.



Erection

The complete structure was erected in 9 weeks. A 4D virtual software and a full-scale 2-storeys mock-up was constructed in order to predict the construction problems and detect clashes with installations. Erection and assembly proved to be faster than originally projected.



Building in construction (left) and assembly of the floor system through steel connectors (right) (Source: Hermann Kaufmann)

Fire design

In order to facilitate approval process, strict fire protection measurements were put into place, arguably making the building safer in the event of fire than traditional concrete or steel structures.

The mass timber elements are encapsulated with multiple layers of gypsum boards to provide the fire resistance rating. Internal demising walls are designed to provide 2-hour fire resistance rating between units, and 1-hour fire resistance rating between units and corridor.

Automatic sprinkler system with back-up water supply offers additional fire protection.



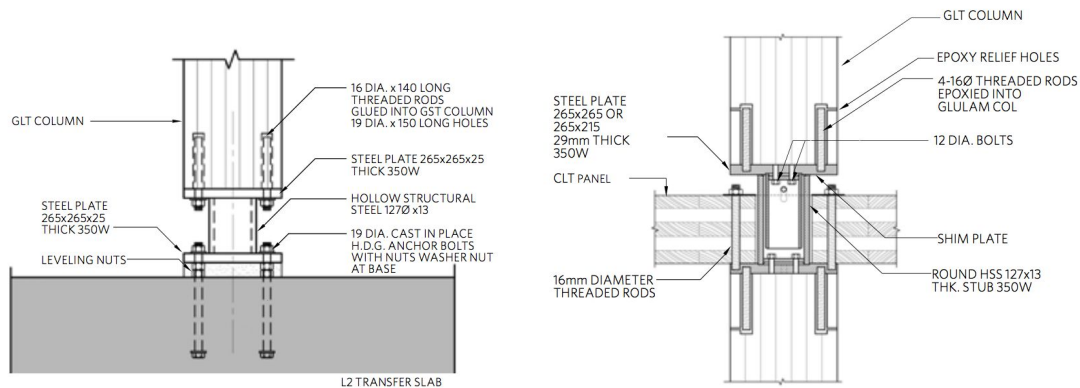
From left to right. 1. CLT floor slabs with glulam columns and steel connectors. 2. Partial encapsulation during construction. 3. Completed construction. (Source: Hermann Kaufmann)

Detailing



Rounded steel connector detail. (Source: University of British Columbia)

Glulam columns with steel connectors provide direct load transmission between columns and support the floor panels. A round hollow structural section is welded to steel plate embedded at the top and bottom of each column, by means of threaded rods epoxied into the column. The connection assemblies at the base of each column have a smaller-diameter hollow structural section that fits into the one at the top of the lower column. The CLT panels rest on top of the lower columns and are bolted to the steel plates by four threaded rods. This type of connection allows vertical load to be transferred directly from column to columns, while supporting the vertical and shear loads of CLT panels.



Detail timber column to concrete transfer slab (left) and column-to-columns (right). (Source: University of British Columbia)

Conclusions

- The structure of the building is a composite of three different structural materials. Cast-in-place concrete is used for stability core and egress route, because of its continuous and non-combustible materiality. Steel is used for its absolute strength, coupling the timber elements in local connections. And timber is used in the majority of structural elements, floors, walls and columns, because of its lightweight, easy of erection and warm feeling.

Murray Grove (United Kingdom)



Image of the building in the context. (Source: Waugh Thistleton Architects)

Location	London, United Kingdom	Construction	2009
Architect	Waugh Thistleton Architects	Height	29.75m - 9 stories
Client	Telford Homes PLC and Metropolitan Housing Trust	CLT	300 tonnes
Engineer	Techniker	Apartments	29 apartments
CLT supplier	KLH UK		

At the time of the construction, concrete and steel were expensive and in high demand because of the construction of 2008 Olympic Games in Beijing. For that reason, the architects decided to impulse the creation of the first modern timber high-rise in 2009. The architects considered solid timber construction as an option of similar cost with other building structures and demonstrated sustainable potentials. To the date of construction there were examples made of 5 storeys buildings with similar characteristics (Thompson, 2009)

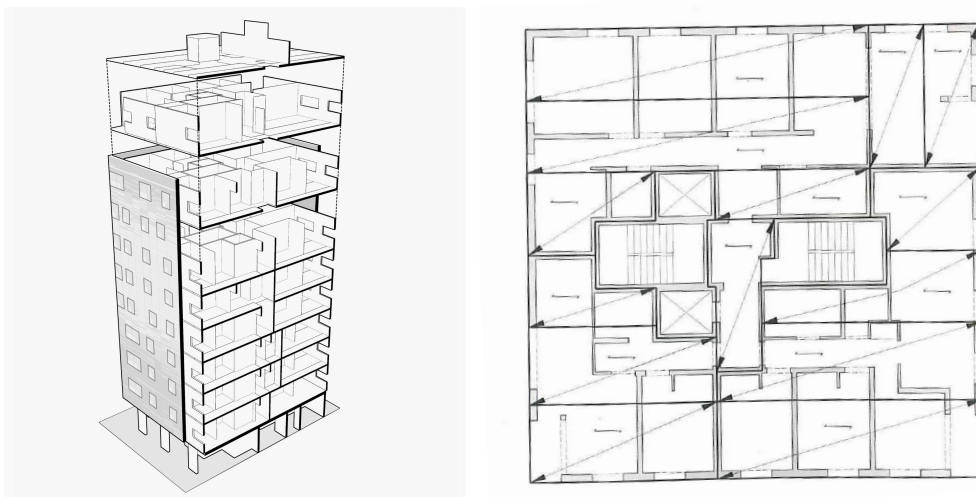
The base of the building is 17x17m, and its nine-story structure includes a ground-floor retail, three stories of social housing and five stories of private housing, adding a total of 29 residential units, each with private balconies.

Structure

The structure is designed as a cellular construction, involving structural perimeter walls and inner partition walls. By extending the structural core towards the edges of the building, it is possible to maximise lateral and torsional stiffness of the building against lateral wind loading.

On the ground floor a ground storey of in situ reinforced concrete provides more open space at street level. By lifting the timber structure above the plinth, there is an improved performance against hazardous events, such as fire risk, impacts and general durability. The foundations consists of 20 bored piles of 600mm diameter and 15m depth, bearing on the clay soil layer.

The structural design provides redundancy of the load-bearing walls, which means that one vertical wall can collapse without sacrificing the integrity of the global structure. Most of the floor panels are designed to double span or cantilever if a support is removed. At the same time, vertical walls can act as high beams spanning from two walls below.



Section showing honeycomb structure and floor panels (Source: Waugh Thistleton Architects)

Due to the change in layout at the connection with the concrete base, some wall-floor junctions may take excessive bearing stresses. This issue was solved by inserting steel shear plates at the base of the CLT panels instead of the alternative of increasing the size panel. This connection has a longitudinal shear capacity of 130KN.

Erection

Each of the panels was prefabricated including cut-outs for windows and doors. The CLT supplier provided panels with maximum dimensions for transportation. 110mm and 190mm thick panels were used for load bearing walls and floors respectively. The maximum panel weight is around 9 tonnes, within the weight limitations range of standard construction cranes. Although, the structure is more expensive than an equivalent reinforced concrete frame, CLT brought savings in the building construction time. While the construction of an equivalent building in concrete is estimated to take 72 weeks, the CLT solution required only 49 weeks. The erection was accomplished with a large mobile crane, eliminating the usual tower crane in a building structure. Scaffolding was required to fix the cladding but not necessary for the timber structure. A four-man crew was on site three days a week and accomplished the entire superstructure in 27 working days, over nine weeks. Temporary barriers were easily fixed once each floor was installed.

Untreated timber was used for the building structure, as the building relies on the envelope for rain and wind protection. If the installation occurs in wet weather, while it is inconvenient, it has no effect for the structural performance, as the panels are capable of releasing moisture once installed.



Erection sequence. From left to right, core, shear walls, and floor panel installation (Source: KLH-UK)

On top of that the structure tolerances were ± 2 mm, compared to 10 mm normally expected in concrete structures. This has important consequences for ease of structural assembly cladding fixing and good air tightness.

Installation of building services has proven easier, and it is expected than in future projects, this feature will lower the cost of solid timber constructions. Cables and pipes were generally surface mounted with screw-fixed straps.

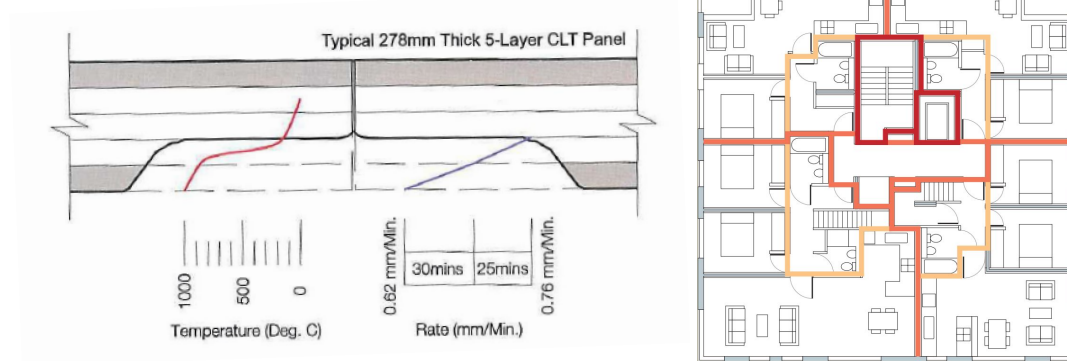


Dry installation with lightweight power tools. From left to right. Self-drill wood screws, angle brackets and services fittings on ceiling.. (Source: Trada)

Fire design

Structural fire resistance relies on the charring of the solid timber elements. Because of the solid mass of the CLT panels, they start to reduce the section progressively until the fire is controlled. For that reason, walls and floors were thickened up according to the fire requirements, resulting in an approximate 35% of extra superstructure weight.

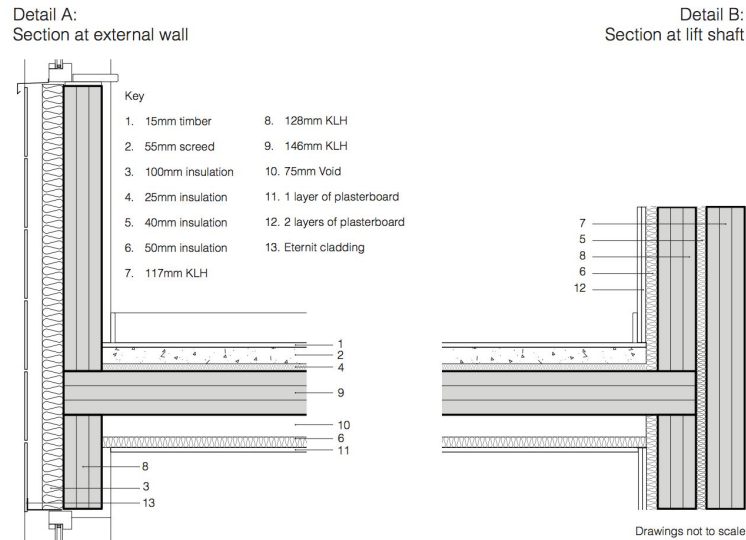
In the image below a floor plan with the fire compartmentation is shown. In red, the walls with 120 minutes fire resistance (apartments-vertical cores). In orange, 60 minutes (apartment-apartment, or public corridor-apartment). In yellow, 30 minutes fire resistance between internal corridors and living spaces



Left, Temperature gradient (left) and charring rates (right) for a typical 278 mm thick, five-layered cross-laminated panel. Right compartmentation of floor plan

Detailing

Due to the low cost of the system, the so called platform construction method was used in the building, First, cross-walls are placed perpendicular to one another and on top of them, floor panels are laid over. However, this system may hamper the structural performance of timber, as the upper vertical loading rest on the side grain of the horizontal floor panels. The end-grain strength of the wall panels is 24N/mm^2 , whereas the cross-grain crushing capacity of the floor plate is 10 times less, $2.7\text{N/m}^2\text{m}$. In order to solve that problem long screws arrays were placed in the connections. Buckling considerations were studied as the walls have to carry vertical loading from upper floors. A limit height of 5 meters, was established, before buckling would govern. In order to meet the UK acoustic regulations, two layers of gypsum board were applied on each side of pantry walls. An acoustic ceiling is required between floors, and double CLT panels with 40 mm air gap are applied around lift and stair cores.



Loading and analysis

The building was calculated according to loading and deflection criteria for Eurocode 1 (CEN 1991-1-4 (2002)). An elastic analysis using Robot Millennium software provided forces and moments that were used for the sizing of the elements, in accordance to the normative for timber structures Eurocode 5 (CEN 1995-1-2 (2004)).

Panelised buildings are susceptible to progressive collapse, the failure of one component may lead to sequential failure of others. At the time of the construction, there were no EU regulations for the issue. After discussions with Timber Research and Development Association (Trada) and UKTFA, resulted in a design criteria for progressive collapse of a notional shock load of 7.5kN/m², and the removal of any single wall, either full length or over a distance of four times its height.

Floor panels are designed to double-span or cantilever if support is removed. By providing ties between floors and walls with simple brackets and screws, vertical elements can act as deep beams spanning across compromised elements below.

Moisture levels changes can have an important effect in any timber construction, because of the associated dimensional changes of the elements. For example, a 10% increase in London climate, results in 2% dimensional change perpendicular to grain and 0.5% longitudinal to grain. Long-term creep movement can be considered negligible as it is less than 0.01% and 1%, longitudinal and perpendicular to grain, respectively.

Panels were fabricated with 12% moisture levels. At the installation this level rose to 14%, and after rain periods to 18%. Nevertheless, the levels returned to 12-14% when cladding was placed

Conclusions

- The structural system is a composition of core and shear walls in a platform style, resulting in a cellular construction. Both interior and perimeter load-bearing walls provides resistance to lateral stability and vertical loading.
- The ground floor and foundation levels are constructed with concrete allowing improved durability and more flexibility for the entrance of the building.
- The simplicity of details and advance prefabrication of CLT, allowed a quick erection of the building requiring only 4 people in 27 days.
- CLT timber construction is very suitable for high-dense areas, because it is lightweight (it does not require a heavy crane for erection) and only lightweight power tools are required for installation.
- Panelised construction systems are susceptible to progressive collapse, thus the structural system is designed with redundant internal walls and multi-supported floors slabs.

Tamedia (Switzerland)

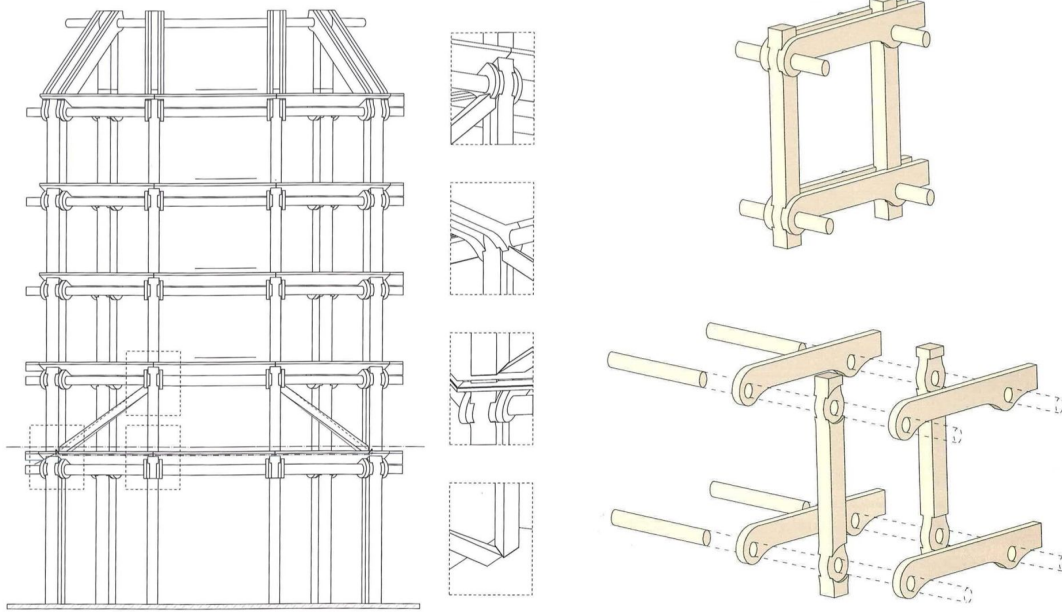
Left, photograph of the building during construction. Right, interior space. (Source: Shigeru Ban)

Location	Zurich, Switzerland	Timber supplier	Blumer-Lehmann AG
Architect	Shigeru Ban	Construction	2011 - 2013
Client	Tamedia AG	Height	23m - 7 storeys
Contractor	HRS Real Estate AG	GFA new building	8900m ²
Timber engineer	Holz GmbH SJB Kempter Fitze AG	GFA existing building	1350m ²
Structural engineer	Urech Bärtschi Maurer Bauingenieure	Glulam	2000m ³

The building located in the urban heart of Zurich, demonstrates an innovative use of timber in construction. The Pritzker winner architect Shigeru Ban, aimed at creating a calm working environment, by leaving the timber structure exposed. The material was selected for its natural warmth feeling. The seven-storey building, 8900m², sits on a two-story underground basement, and adds two-storeys extension, 1350m² atop an adjoining existing building, also part of the Tamedia company headquarters.

Structure

The building most important innovation is the post and beam interlocking structure, with spans of 5.45x3.2m and 5.45x10.98m in short and long directions respectively.



Left, timber frame details at first floor truss in order to allow column free ground floor. (Source: Blumer Lehmann AG) Right, axonometric diagram of timber frame (Source: Shigeru ban)

The structure is a series of timber frames comprised of 4 columns and generally 5 double beams. Each frame has 3 bays (3.2/10.98/3.2m). The columns are 23m tall, running the full height of the building. The double beams are made with two 120 mm thick block glue-laminated timber, and run the full 18m total width of the building, intersecting the columns at four points. Finally large horizontal oval beams connect the columns laterally and complete the frame. These intricate fitting prefabricated assembly demonstrates the accuracy capable with timber manufacturing. The entire 23m column has a tolerance of 4mm. While the timber structure takes the vertical gravity loading, the stability is provided by two concrete cores, containing vertical circulation, restrooms, and technical services.

Erection

There is a scarcity of space in the centre of Zurich, and the whole building had to be erected on-site are on 20x20m, and deliveries scheduled just-in-time. The structure could not be assembled horizontally and then lifted into position because of the lack of available space. For that reason, the concrete core serve as temporary support for the assembly of the elements into bigger frames. The structure was assembled using a mobile crane and following the next steps:

1. Completion of first frame attached to concrete core. The first sub-part of beams are fixed to the concrete cores, followed by the columns, and the second sub-part that completes the double beam, and consequently the first frame. This serve as a template for the rest of the structure
2. The same logic is followed for the assembly of the next frame. Sub-part of beams attached to first frame, followed by the columns, and then completed with second sub-part of double beam.
3. Completed second frame is moved into its position with a mobile crane
4. Second frame is stabilised with oval bracing beams and temporary shoring.
5. Prefabricated floor and ceilings are craned into place
6. An additional prefabricated roof truss is completes the first pair of frames

Each pair of frames took approximately three days to complete. 60-70% of elements were fabricated in the factory in a period of approximately 6-7 months. The erection took 3-4 months to erect all of them with a high level of logistical coordination. A big benefit of the construction with timber is a great deal of information can be imbedded into the manufacturing facility enabling a clear and quick construction sequence



From left to right. Second frame placed into position and braced with oval elements. Bracing of roof trusses. Transport of prefabricated roof truss elements to site. (Source: Blumer Lehmann AG)

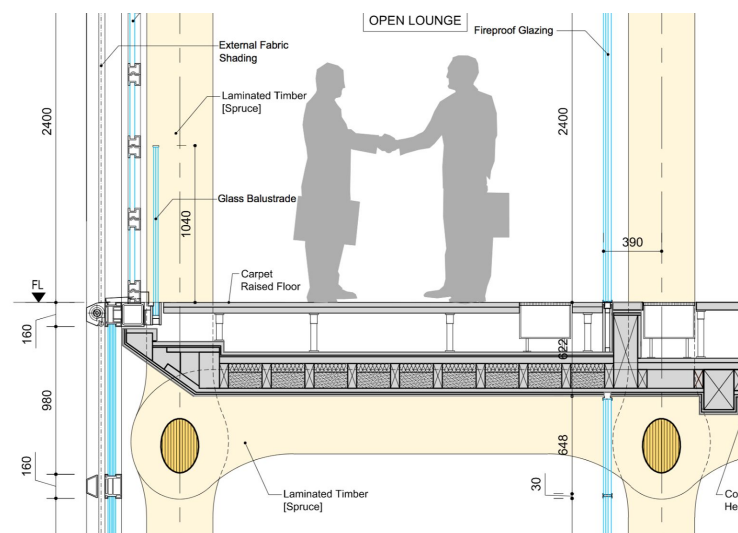
Fire design

Because of the innovative structural system and open spaces, the fire concept was lengthily discussed with the building authorities. The glulam structural elements are over-dimensioned according to 60 minutes fire protection. In addition there is a sprinkler system installed in the building. Regarding the floor system, two gypsum boards on the underside, and one cement-wood composite chipboard on the top, provide 60 minutes fire resistance.

In order to separate open lounge and office spaces, a fire rated glazing was chosen, with solid timber blocking fire stops embedded in the floor system. When the structural elements penetrate the fire-protected glazing, a special fire-sealant is applied.

Due to the fact that building services run through the floor system, over 3000 pre-drilled openings were made in factory, and later fire-sealed on site, once the installations were placed.

Detailing



Floor system

Prefabricated floor elements contained additional layers to meet fire and acoustic requirements, from top to bottom the floor layers composition is the following: Carpet; Raised floor; Cement wood chipboard t20, Rubber Mat t10; Mineral wool t140, Sand t80; 3x Timber boarding t27 and 2x Gypsum fiberboard t15

As above mentioned, the gypsum and cement boards assist with the fire protection. The raised floor and sand layer helps in reducing sound transmission and provide space for services. With the floor modules are ceiling-mounted radiant heating and cooling panels to condition the office spaces.

Hardwood dowel connection

The transmission of forces from the double beams to the columns required a hardwood reinforced special connection. In order to avoid or reduce stresses perpendicular to the grain an extra plate and an oval dowel was made out of beech hardwood, interlocking the oval bracing beams and double beams together horizontally.

A close collaboration between the timber manufacturer Blumer-Lehmann AG, and the architect was needed in order to avoid construction conflicts. The creation of a 3D model for production, and a 1:1 scale mock-up was critical for refinement of the details.



From left to right. Manufacturing of timber elements. Detail of column, beam, and ceiling interface. Detail of column and beam joint with hardwood connector dowel. (Source: Blumer Lehmann AG)

Conclusions

- The exposed timber structure creates a warm feeling inside the building.
- The design intention of not using metal connections resulted in an intricate and complicated structure that demanded a high-level of coordination in the construction sequence.

Esmarchstrasse 3 (E3)

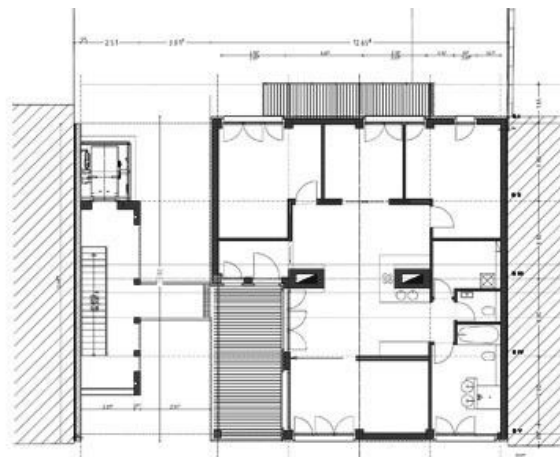


Photograph of the façade (left) and image during construction (right) of the building. (Source: Kaden Architekten)

Location	Berlin, Germany	Fire engineer	Dehne, Kruse & Partner
Architect	Kaden + Partner	Design	2005
Client	Baugruppe e3 Gbr	Construction	2007-2008
Engineer	Julius Natterer, Bois	Height	22.85m - 7 stories

	Consult Natterer SA		
Testing	Heinrich Kreuzinger	GFA	941m2

The project is a close collaboration between the architects and a co-housing corporation. As an alternative to commercially driven housing, there is no need of a developer, and in this way, more budget can be dedicated to the construction and architecture. The result is a high-quality building for working families at a lower cost. The building generated a great interest in Berlin, over 1000 inhabitants came to the site on the opening day. The project is conceived as a vertical stack of single family houses, in each floor there is a single residential unit (except in the third floor with two smaller units). The vertical circulation is placed on the exterior of the building, and connected by concrete bridges, in order to liberate the floor plan and create an additional "third façade", which allows more daylight and ventilation. There is a high-quality treatment of the balconies and exterior spaces, with recessed corner loggias on the front façade, steel-suspended balconies on the back and side backyard spaces.



Plan (left) and section (right) of the building (Source: Kaden Architekten)

Structure

The structure is a series of post and beam glulam timber elements connected by steel knife-plate connectors. The steel connections ensure that the glulam elements are not loaded perpendicular to the grain (the weakest behaviour of timber). Between the main structural posts and beams, dowel-laminated infill panels and steel cross-bracing ties are placed in an alternating way with full height windows. This creates a playful effect on the façade.



Left, image of post and beams glulam elements with steel connectors. Right, Timber infill panels for compression and steel cross-bracing ties for tension. (Source: Bernd Borchardt)

The lateral stability of the building is provided with three different structural elements, that work in a composite way.

- Dowel-laminated infill panels that act as diagonals in compression inside the post-beam frame. There are at least three infill panels per floor, in an alternate way
- Steel cross-bracing ties that act as diagonals in tension inside the post-beam frame.
- Small interior concrete cores, that act as shear walls. They are also used for encasing the vertical shafts for the installations

The vertical load-bearing system is supported by the glulam post and beams frame, with infill dowel-laminated infill walls and floors. The building has a ground level constructed from cast-in-place concrete

Erection

Located in a dense urban site, there was little available space to store the construction materials, and therefore a just-in-time construction schedule was chosen. Four workers on site, were able to construct one floor per week, following the next steps:

1. Post and beam structure, Infill panels, exterior insulation and concrete service chasing
2. Timber floors, and surface treatment with water-repellant cement
3. Reinforcement of screed slab
4. Pouring screed slab, and allowed to dry over the weekend



Insertion of infill timber panels (left) and placement of timber floor (right). (Source: Bernd Borchardt)

The total completion of the building was 9 months of construction. The industrial prefabrication of wood elements resulted in high level quality and short construction time. The structural timber elements were CNC off-site so that they can be inserted seamlessly into steel knife-plate connections that join the structure together.

Fire design

The building was the first timber structure to be categorised as Building Class 5, exceeding the city's building codes. Twelve specific steps were required before granting the building permit. The Technical University of Munich performed tests of the structural components, in order to ensure that the building was safe in the event of a fire. After testing, it was concluded that the structure of the building could resist the 90-minutes fire resistance. The following measures were required regarding fire design:

- The timber components were encapsulated with fire-resistant material
- Smoke detectors installed in each apartment
- Short escape routes with exterior concrete stair
- Transparent fire resistant coating on the underside of wooden ceilings
- The steel connections are recessed into the glulam elements, so as the mass of timber assures the fire protection

Offsetting the exterior windows also assists in fire prevention, reducing the risk of fire spreading directly from one window to the next.

Detailing

The post and beams glue-laminated timber elements are approximately 320x360mm. They are connected by a three plates steel knife connector, that ensure that the transmission of stresses perpendicular to grain direction in the timber elements.



Structural details of steel connector between post and beam timber posts. From left to right, detail normal connection, detail at corner of the building, and technical detail drawings.

The infill timber panels are 160mm thick. Because of the fire regulations, in the interior side two layers of 18mm gypsum board are placed. On the exterior side, 12mm of sheathing, 100mm non-combustible mineral wool insulation and 8mm mineral plaster complete the assembly with 90-minutes fire resistance.

Floor system. Timber-concrete composite (TCC)

The floor system is a composite between concrete and timber (TCC), resulting in an efficient system that can span longer than timber alone, and lighter than a normal concrete slab. In addition, the construction is simplified because the timber provides the formwork for concrete, and the later adds mass to the system reducing vibrations. The floor system spans 6.5 meters, from exterior walls to the inner concrete mini core used for services. This arrangement removes the need for any vertical columns and increases flexibility of the floor plan. The composition of the floor includes 160mm thick dowel-laminated timber floors + 100 mm reinforced concrete slab.

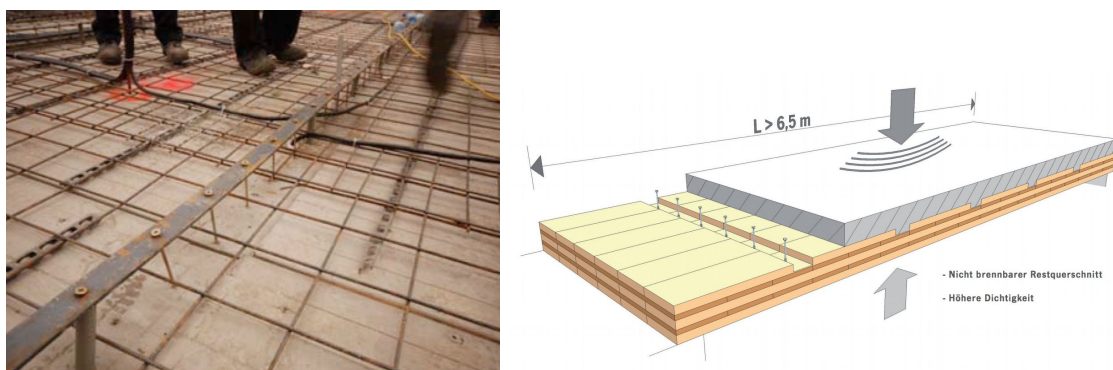


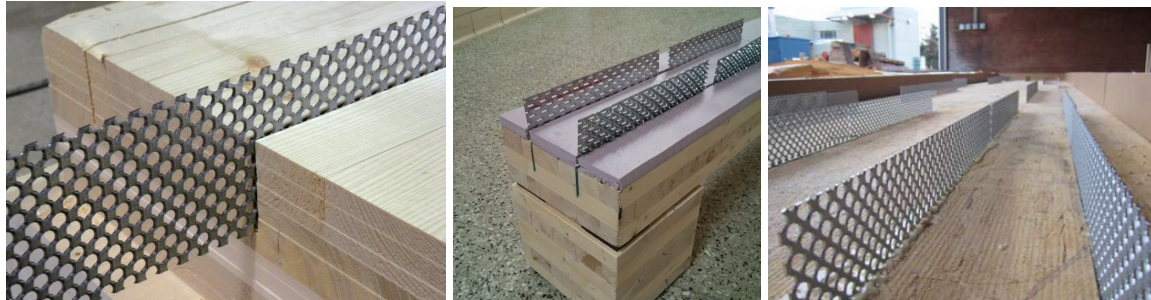
Image of the construction of the floor system in E3 building (left) and with grooves into timber element and steel shear screws (right)

Series of 75mm screws were installed into the timber element in, 20 mm deep grooves are cut out into the timber floor for interlocking the structural behaviour between timber and concrete. The concrete slab is completed with impact sound insulation, radiant heating and cement screed. By using the 100mm thick

concrete slab, it was possible to leave the wooden ceilings exposed. In total the combination of the elements have a 90 minutes fire resistance.

Another method for transferring shear between timber and concrete was developed by Holz-Beton-Verbund (HBV), consisting in perforated steel strips that are epoxy-glued to the timber element. One example of this system was used in UB Earth Science Building, where 89mm thick LSL panels were completed with 100mm reinforced concrete.

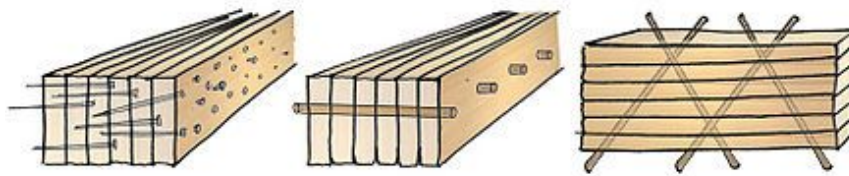
One advantage of the system is that insulation material can be inserted within the floor slab, reducing acoustic vibrations.



Perforated steel strip connectors similar to TCC floor system in UBC Earth Sciences Building. (Source: FTP Innovations)

Brettstapel- Dowel Laminated Wall

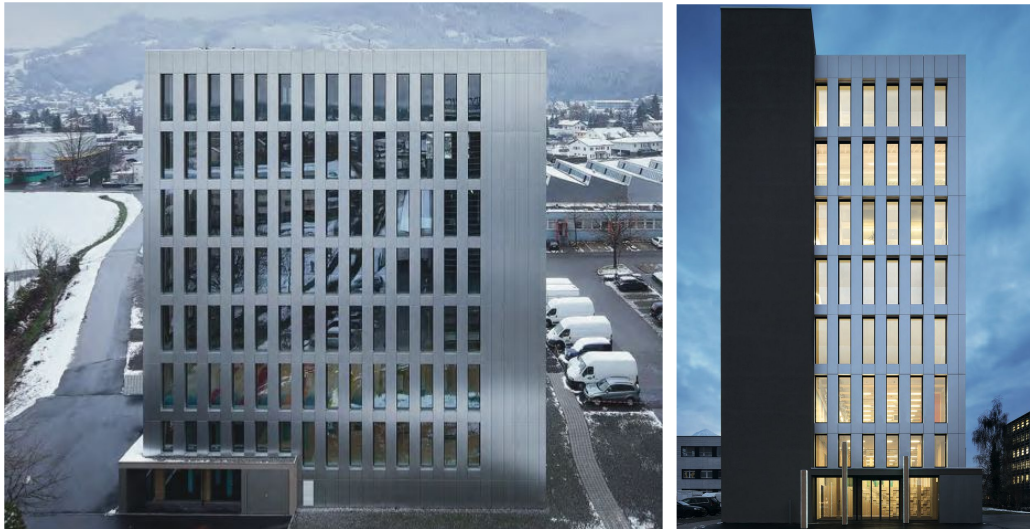
The alternate timber infill walls between in the beam+post system, are made using a type of glue-free laminated panel. "Brettstapel" uses hardwood timber dowels, in order to connect softwood lower grade laminations. This is one of the few construction methods that can be completely fabricated without the need of glue or nails, resulting in a healthier indoor air quality.



Types of dowel-laminated panel "Brettstapel" configuration

Conclusions

- The lateral stability of the structure is composed out of several systems that partly contribute to the global stiffness, i.e. glulam post and beam, steel ties, infill CLT panels and mini concrete cores.
- The use of composite concrete-timber floor systems can help reducing issues regarding vibrations, and fire resistance, and at the same time can improve the structural performance achieving longer spans.

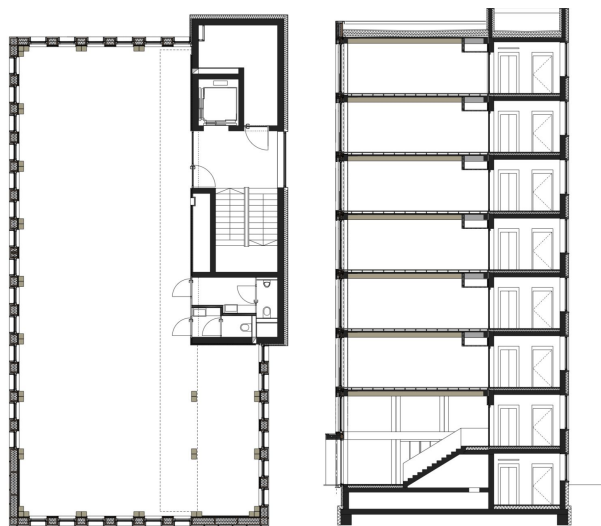
LCT One

Exterior photographs of LCT One Tower. (Source: CREE, GMBH)

Location	Dornbirn, Austria	Fire consultant	IBS
Architect	Hermann ZT GMBH	Construction	2012
Structural engineers	Merz Kley Partner GMBH	Height	27 meters - 8 storeys
Developer	CREE GMBH (Rhombert Group)	GFA	1600m2

The LifeCycle Tower One (LCT ONE) is a 8 storey commercial office developed by CREE GMBH. The building serves as an educational laboratory for knowledge of mass timber construction. In this sense the whole building, was designed as a prototype to develop and test prefabricated mass timber components.

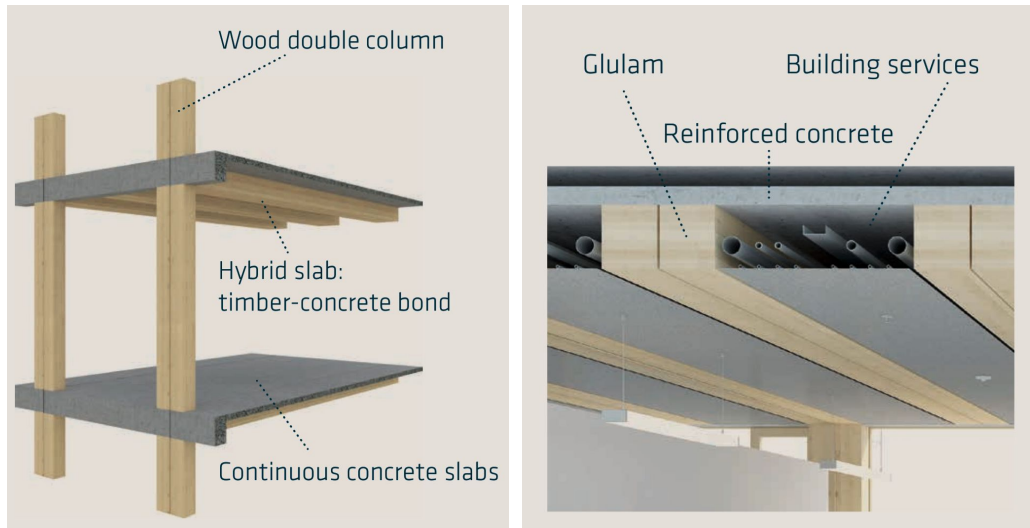
The CREE research team also developed a theoretical study for a 20 storeys timber building with CLT stability core and composite floor slab.



Plan and section of the building (Source: CREE)

Structure

The structural system has been patented by CREE (Creative Resource and Energy Efficiency) and Rhomberg Group. It is a structural assembly composed of double glulam columns and glulam-concrete floor system. By using concrete material, multiple benefits can be achieved: long spans (9.45m), improved fire and acoustic performances, and service integration between the beams.



3D visualisations of timber structure (left) and system integration between glulam beams (right). (Source: CREE)

Load bearing system

The gravity loads are transferred through the columns (hinged columns) directly into the hybrid slabs and out again into the double columns below. Two columns of typical dimensions 240x490mm, are placed next to each other with 10mm gap. The structural grid from centre-to-centre is approximately 8.1 meters, and at the corners columns are tripled. Theoretically, this system can reach height of 30 storeys even when using a timber core. The floor-to-floor height is 3.30m with a clear story height of 2.80m. The building uses a typical concrete structure for basement and ground floor.

Lateral resisting system

The building levels are accessed via a unique concrete core, which also serves as the main lateral resisting system of the building. For this purpose, cast-in-place concrete was used.

Floor system

The hybrid timber-concrete floor slab is one of the main innovations of the structure. With the use of timber, acoustic, thermal and fire safety requirements are more easily met. Using a prefabricated system rather than site-cast allows for greater speed of construction by removing the need of formwork and wet trades in the construction process.

The assembly is composed out of a timber-concrete composite rib slab of approximately 9 x 3 meters. This floor system can span up to 9.45 meters from load bearing glulam posts in the perimeter of the building, removing the need for intermediate columns. Technical services can be installed flush with the slab, and no suspended ceilings are necessary, therefore reducing floor-to-floor height to a minimum. The pull out or lateral forces, between the hinged columns and the hybrid slabs, are prevented from separating through the use of simple mortise and tenon joints. Connections between concrete core and hybrid slabs are made with angle steel brackets.

Erection

The structural solution was developed as a kit of parts, facilitating a prefabricated modular systems and therefore minimising the complexity of the construction. Both walls and floor slabs are prefabricated offsite and brought to the construction site for final installation. First, timber frame walls are attached to the double columns to create one façade component that can be installed onsite. After the walls are erected, the composite floor slabs are placed on top, in a platform-style construction, and construction workers can continue with the next level. Building progresses much faster than is the case with conventional systems thanks to the connection of the primary and secondary structures.

5 façade and 9 floor slab elements per floor took approximately 5 hours to complete, including sealing and grouting. The entire structural frame was installed in only 8 days. On the contrary, the cast-in-place concrete core took about 3.5 months to be completed.



Pictures of the assembly process. From left to right, Lift of façade component, installation façade component, installation floor component. (Source: Architekten Hermann Kaufmann)

Fire design

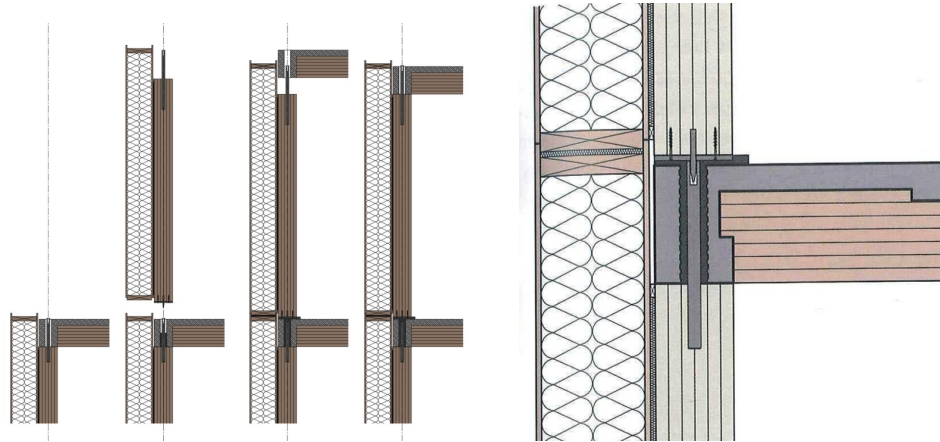
Fire authorities were involved from the beginning in the design process and demanded the following requirements:

- Fire separation between stories, by using a composite concrete slab, and no timber-to-timber contact
- Elimination of cavities and penetrations within walls and floors to reduce potential risk of fire spread
- Non-combustible egress route, by using a concrete core.
- As most of the timber elements are exposed, the fire department demanded the use of an automatic sprinkler system. This was considered redundant according to later authorities.
- Extensive testing of the structure in a full-size chamber. Initial floor slab (only timber) achieved a fire rating of 30 minutes. However, with the addition of concrete, a fire rating of 120 minutes was demonstrated.
- All timber members are oversized in order to provide the protective charring layer.

Detailing

Each of the composite slabs is supported on each of the four corners with 76mm diameter steel tubes that are threaded into the precast concrete in the corner of the floor slabs.

At the bottom of the columns steel pins are inserted into the hollow steel tubes, thus creating a pinned connection that is capable of supporting vertical forces during the erection. The building core was casted with steel angle brackets that support the other side of the floor slab. Finally, a sealant and non-shrink grout is used at the junction between steel plates and floor slabs. A series of notches are created within the timber elements in order to ensure composite action between concrete slab and double-glulam beams.



Left: Assembly sequence, from left to right: Platform-style, façade component and floor component. Right: Vertical detail section of floor and exterior wall (Source: Architekten Hermann Kaufmann)

Conclusions

- The use of a Timber-Concrete composite floor systems can be a good alternative for achieving long spans between columns (<9.45m), and meeting fire and acoustic demands. Steel tubes inserted in precast concrete can provide good structural behaviour for CLT connections absorbing internal shear stresses.
- The building can be quickly erected using large prefabricated modules (façade and floors) in a platform style, reducing the number of on-site operations and improving the overall construction quality.

Limnologen



Photograph of the building in surroundings and façade close up (Source: Bolaget architect)

Location	Växjö, Sweden	Constructor	NCC and JSB
Architect	Ola Malm, Arkitektbolaget Kronoberg AB	Construction	2007-2009
Client	Midroc Property	Height	8 storeys
Timber engineer	Martinsons byggsystem (timber)	GFA	10700m ²

Concrete engineer	Tyréns (Concrete)	Programme	4 building blocks 134 residential units
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The buildings were partly a showcase for high-tech construction with timber and highly prefabricated methods, impelled by Swedish national timber promotion strategy. As the buildings are located next to a lake, the architect wanted to create the "wooden house feeling". 4 eight-storeys blocks create the residential complex that in total contain 134 units, with apartments between 37-114m², for 1 to 5 people. Each apartment provides generous outdoor spaces.



Typical floor plan of the building

Structure

The load-bearing system consists of load-bearing exterior CLT walls in combination with some CLT interior partitions. Glulam columns are also used at certain locations in order to reduce the deformations. In addition, traditional timber frame walls are used in non-structural partitions (separation of apartments). The ground floor is made of concrete, increasing the weight of the building and facilitating anchoring of the above storeys. Inner structural walls were used for the stabilisation of the building and vertical loading, requiring a deep dialogue between the architect's ambitions of open floor plan and the structural requirements. At the same time, the building's relatively complex geometry imposes further demands. Structurally, CLT floors act as stiff plates to transfer wind loads to CLT shear walls. 48 vertical tension rods are concealed inside the shear walls, connecting the concrete foundation with the top floor level. These steel cables are used to resist wind-loading and lift-up forces, anchoring the building to the concrete base.



CLT interior wall lifted into place over steel tension rod (left) and interconnection of tension rods between floors (right) (Source: Kirsi Jarnerö)

At the same time, this means that load-transferring connectors between the wall elements are not necessary. The tension rods must be re-tightened after some time due to relaxation in the steel, creep deformation or drying of the timber elements.

Erection

Weather protection system

It is of special importance that the building process was moisture proof, from manufacturing, transportation and erection on the construction site. For that reasons, a tent structure with an overhead crane was erected in order to protect the structural elements from the weather. This tent is capable of rising vertically following the construction of each additional storey.

The wall and floor prefabricated elements are manufactured indoors, wrapped in plastic film, and transported covered by a tarpaulin.

Once on site, the elements are unloaded using a forklift and stored on the lifting zone under the large tent. Next, the tent crane lift each of the elements into place in the building.

Erection sequence

Each complete storey was constructed in about two weeks. The sequence of installation was generally the following:

- Installation of external and internal walls, including columns and beams during the first week
- Construction of floor, balconies and vertical shafts during the following week.



Weather protection system with integrated overhead crane (left) and lifting of prefabricated floor element inside the tent (Source: Johan Vessby)

Fire design

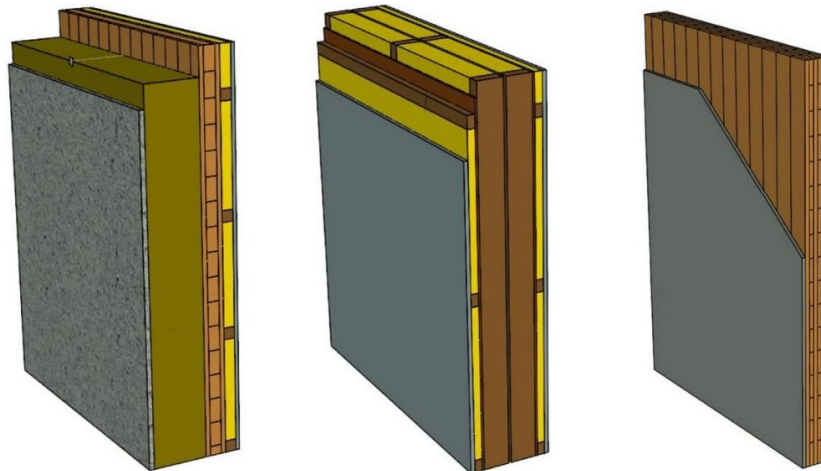
The following measures were used regarding fire protection:

- Interior apartment separated as fire cells, with special attention to detailing.
- Timber structural elements rated 90 minutes fire-resistance
- Wall and ceilings are clad with gypsum boards
- For additional safety, sprinklers system was used.
- Balconies are constructed with CLT and glulam beams, and have a 30-60 minutes fire resistance, to stop the spread of flames between floors.

Detailing

Walls

The structure of the building uses three main types of prefabricated walls elements. 3ly-CLT exterior walls, traditional double-stud timber framed walls providing extra acoustic insulation between apartments and 3ly-CLT interior walls. All those elements are part of the stability system. The façades are plastered or covered by wood panels, whereas the interior walls are cladded with gypsum boards.



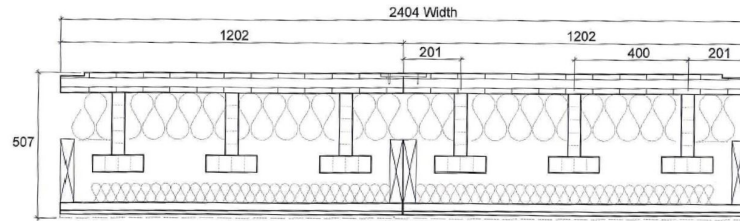
From left to right: Exterior wall, apartment separation wall and interior partition inside apartment (also contributes to stability) (Source: Erik Serrano)

5ly-CLT panels with total thickness of 85mm are used at the elevator shafts walls, with an interior layer of gypsum board. Other vertical chases and service cores are similarly constructed.

Floor system

Every storey is composed out of 30 prefabricated floor elements, made up of 3ly-CLT 70mm and T-shaped glulam beams placed at 600mm centres. The floor is completed with 170mm mineral wool insulation, 45x220mm timber framing, 70mm mineral wool insulation and 2 layers of 13mm gypsum board.

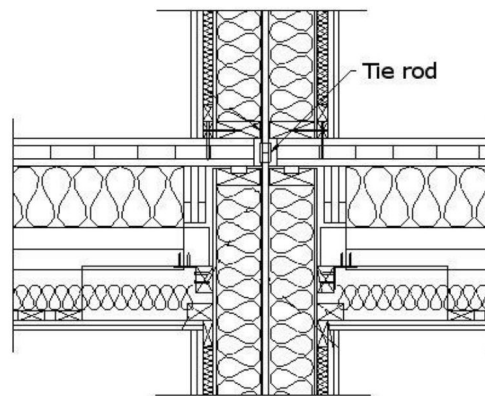
The composite behaviour between CLT panel and glulam beams allows for 8m spans. Services such as ventilation, water and sprinklers run along the length of the panel and were previously installed in the factory. At the edge, each floor panel has a flange that rests on a load-distributing timber beam, screwed into the CLT wall beneath.



Longitudinal section of prefabricated timber floor deck. (Source: martinsons Byggsystem AB)

Acoustics

The building should meet class B acoustic criteria, or at least 52dB sound insulation between apartments. In order to meet this requirement, a polyurethane sealant is used between vertically discontinuous walls and the flange of floor elements. At the same time, the concealed tension rod and screws for wall-floor connection are also sound-insulated.



Typical connection detail between interior wall and floor (Source: Martinsons)

Note in the image above that both walls and floor elements are discontinuous through the join area. The figure also shows the vertical tension rod, used for the overall stability of the building.



Examples of acoustic detail connections used in Limnologen building. From left to right: Detail at exterior wall; structural tension rod inside wall cavity; and insulation strips on top of pantry wall. (Source: Kirs Järnerö)

2.4 CONCEPT STUDIES

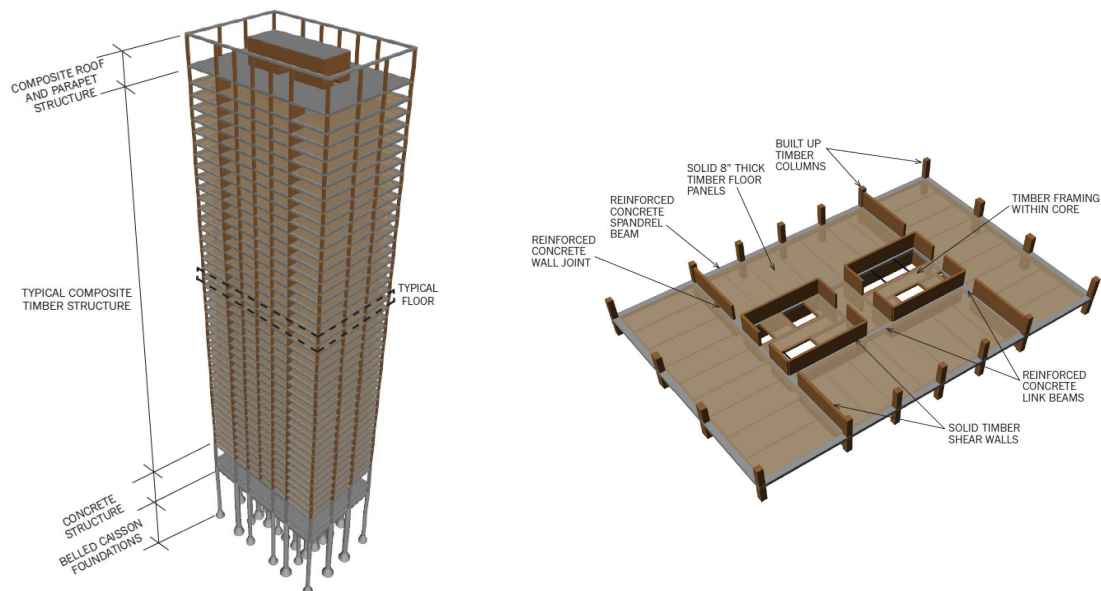
Concrete Jointed Timber Frame (SOM 2013)



Dewitt-Chestnut Apartments original structure, punched concrete tube (left).
Timber frame research tower (right). (Source: SOM)

In 2013, the architecture and engineering firm, Skidmore, Owings & Merrill LLP (SOM), designed a research project using Dewitt-Chestnut Apartments in Chicago as a benchmark. The building was chosen because of the simple shape, and efficient structure which would give a lower-bound for comparison with a timber structure. The original building was designed in 1966, with a structure of reinforced concrete flat plate floor, interior gravity columns and a perimeter “frame tube”.

The benchmark structure with base dimensions of 24x37m and 120 meters height (42 storeys), used an efficient structure with bulk densities of 250Kg/m³ concrete and 100kg/m³ steel rebars.



Overall structure (left) and typical floor structure (right). (Source: SOM)

Structure

The structural system proposed mainly consist of mass timber elements (CLT panels or glulam linear elements) and structural reinforcement with concrete joints.

Timber is used for the primary structural elements such as floors, columns and shear walls. Steel rebar reinforcement is connected to the primary elements by drilling holes in the timber and bonding reinforcement with epoxy adhesive inside the perforation.

The connection between timber members is done with lap splicing reinforcement through the concrete joints. This results in a band of concrete at the perimeter of the building and wall-floor intersections. Supplementary reinforcement is provided in the concrete perimeter beams in order to achieve long spans, as well as in the concrete link beams that couple the behaviour of wall panels. Additional structural steel elements are used for connecting timber elements during erection, and later embedded into the concrete joints.

The system is approximately 80% timber and 20% concrete by volume of a typical floor.

The entire building (considering foundations and ground floor) is 7% timber and 30% concrete.

	Superstructure		Foundation	
Material	Benchmark	Prototype	Benchmark	Prototype
Concrete (kg/m ²)	717	183	102	66
Steel reinforcement (Kg/m ²)	29	8,3	0,5	0,5
Timber (Kg/m ²)	-	97,6	-	-
Structural steel (kg/m ²)	-	1,5	-	-

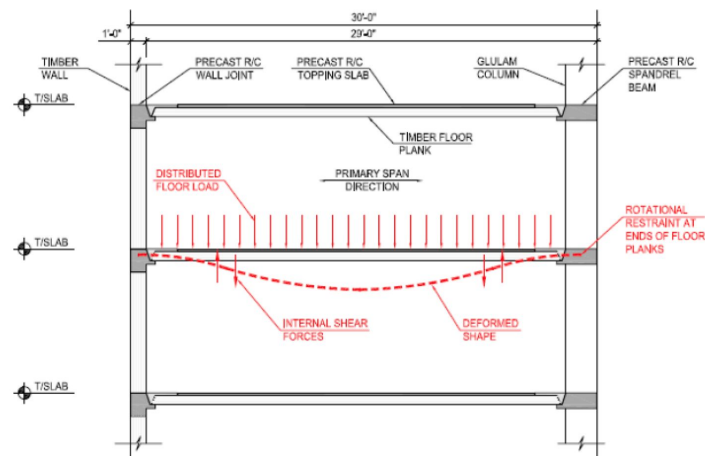
The volume of the required structural materials for the “Concrete Jointed Timber building” suggest that the prototype building can be competitive against a conventional construction method with reinforced concrete

Gravity load resisting system

The floor systems consists of a CLT panel or similar, that spans between inner shear walls in the centre of the building and perimetral concrete spandrel beam. The edge of the floor panels are restrained from rotation with the reinforcement connections. This scheme stiffen the systems reducing deflection and vibrations.

The columns at the perimeter are spaced 7.3m at centers. Consequently, the spandrel beam has to be robust enough to support the floor loads between columns and restrain the floor edge movement and rotation.

Concrete material was a better candidate for this element, since timber is weaker in torsion and steel may imply some fire complexities. The spandrel beams represent approximately 65% of total concrete used on a typical floor.



Floor structural behaviour and loading assumptions. (Source: SOM)

The columns and walls carry the load below ultimately until the foundations.

The top level floor is designed to support high-loading from technical services. Therefore a composite CLT panel + normal weight concrete topping slab is used for a better distribution of loads, and enhancement of acoustic insulation.

Lateral load resisting system

The lateral-resisting system consists mainly out of solid mass timber CLT panels, that behave as shear walls.

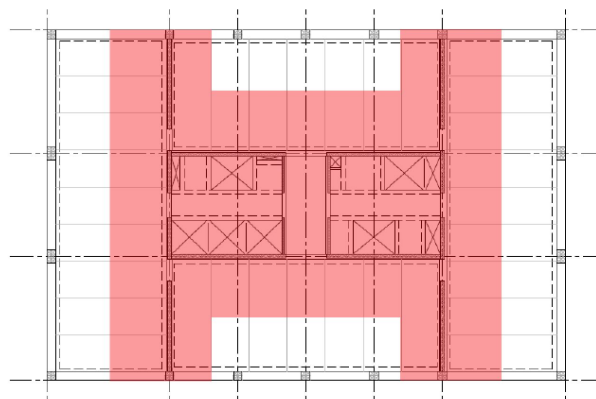
The shear walls are located around the vertical transportation and service core at the centre of the building, and resist wind in both directions, as well as the overall building torsion.

Supplementary shear walls extend from the central core to the perimeter in the shortest side of the building.

These walls are critical to resist net building uplift due to wind forces. The shear walls are coupled by reinforced concrete link beams, that are reinforced accordingly. The extended shear walls reduce in length along the height of the building, as the overturning demands from lateral loading decrease.

The design approach follows similar strategies that are commonly used in couple concrete shear walls.

In the design of a tall timber building the combination between gravity and lateral loading is of utmost importance. Maximising the amount of gravity loading supported by the lateral-resisting system minimises the potential net uplift.



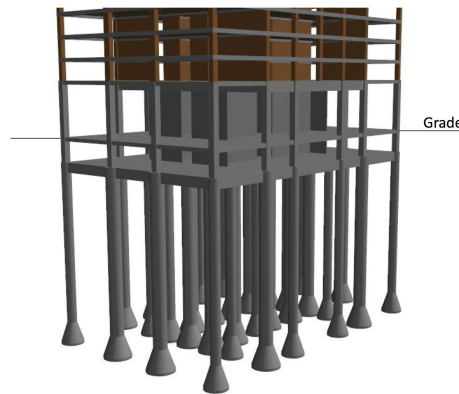
65% distribution of gravity loading to lateral-resisting system. (Source:SOM)

The figure above shows the distribution of gravity load supported by the lateral-resisting system, corresponding to 65% of the total floor area. Even though, this distribution is relatively high, it could also be optimised by engaging the perimeter columns in the lateral-resisting system, e.g. by using outriggers and belt walls, or braced frame external tube.

Substructure and foundations

The structure of the building is concrete framing (shear walls, columns, beams and slabs) from the foundation level to the second level. The increased strength of the concrete system allows for more flexibility, openness and durability in the ground floor.

The foundations of the benchmark building consist of belled caissons, with bearing point at 22.86 meters. The same foundation type is used for the new prototypical building. Because of the lighter weight of timber, only 65% of the original foundations are required for the new proposal. In order to control net uplift, tension piles/caissons, grade beams distributing the tension forces and vertical reinforcement at lower depths is required in the foundations.

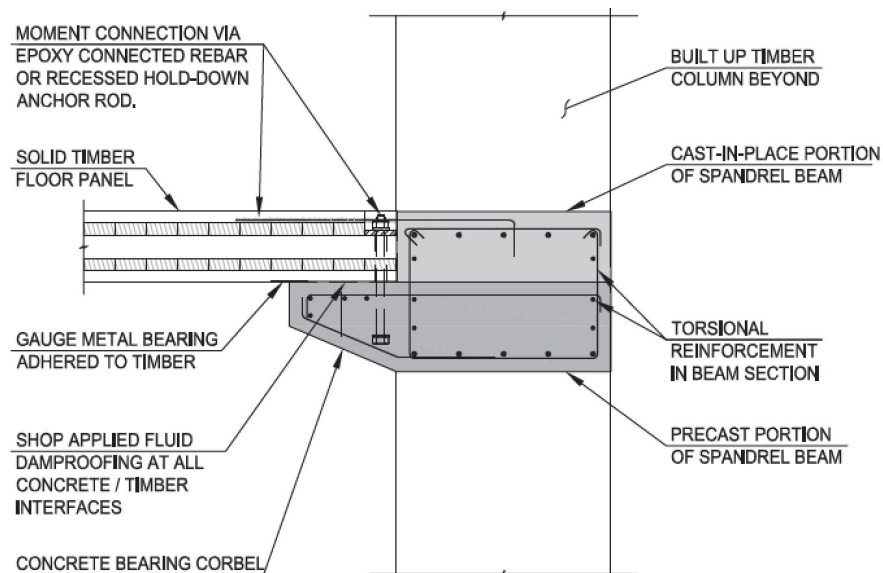


3D scheme of belled caissons and grade beams in foundations system. (Source: SOM)

Floor system

A rigid moment connection provide rotational end restraint to the floor, which reduces the peak bending moment demands and stiffens the floor, therefore reducing deflections and vibrations.

The proposed structural system is governed by vibrations and deflections similar to a concrete one way slab system. The timber floor system can have similar behaviour as a concrete system by providing fixed end connections to the vertical structure. The "Concrete Jointed Timber Frame" utilize the reinforcement through concrete to provide a moment rigid connection between the CLT floor panel and the superstructure, resulting in a more optimised floor system.



Typical spandrel detail, moment-rigid connection between timber floor and spandrel beam. (Source: SOM)

Erection

The building sequence is designed similar to a steel structure with metal deck slabs. The vertical elements are connected to the corresponding vertical elements on the storey above with steel fittings. This allows a quick erection and avoids intermediate concreting of the joints.

The formwork of the concrete joints is supported by the vertical structure, and lower portions of the spandrel beams are prefabricated, in order to avoid re-shoring the concrete elements.

The common construction sequence for each floor follows the next steps:

1. Vertical elements, glulam columns and CLT panels. The structural elements have steel fittings that are used for temporary connection with the member below, until the concrete is poured.
2. Precast concrete beam, that spans between perimeter columns. Formwork for shear wall cores and link beams.
3. Floor panel. Temporarily supported on precast concrete beam at the perimeter, formwork to core and fastened to one another
4. The timber construction continues vertically repeating steps 1-3
5. Reinforcement placement and concrete pouring
6. Removal of formwork

Fire design

Fire regulations in United States for a 42-storey building demand a "Non-combustible" structure. This results in the following schematisation of the structural components:

- Interior bearing walls: 4 hours
- Exterior columns: 4 hours
- Beams, girders and trusses: 3 hours
- Floors system: 3 hours

While it is potentially feasible to calculate and test the systems for fire resistance, it is not possible to classify the structure as non-combustible. Therefore the structure itself can not fall within the existing framework of the prescriptive code.

Conclusions

The volume of the required structural materials for the "Concrete Jointed Timber building" suggest that the prototype building can be competitive against a conventional construction method with reinforced concrete.

In a Core + Wall CLT timber system, the most difficult elements to design are the link beams coupling the movements of different elements. The composite behaviour of different CLT panels against lateral loading is of utmost importance. In this project the solution of concrete beams was used because of their higher shear capacity and additional weight.

Due to the low weight of timber buildings, often uplifting forces occur. In order to solve this problem, a combination of solutions was chosen:

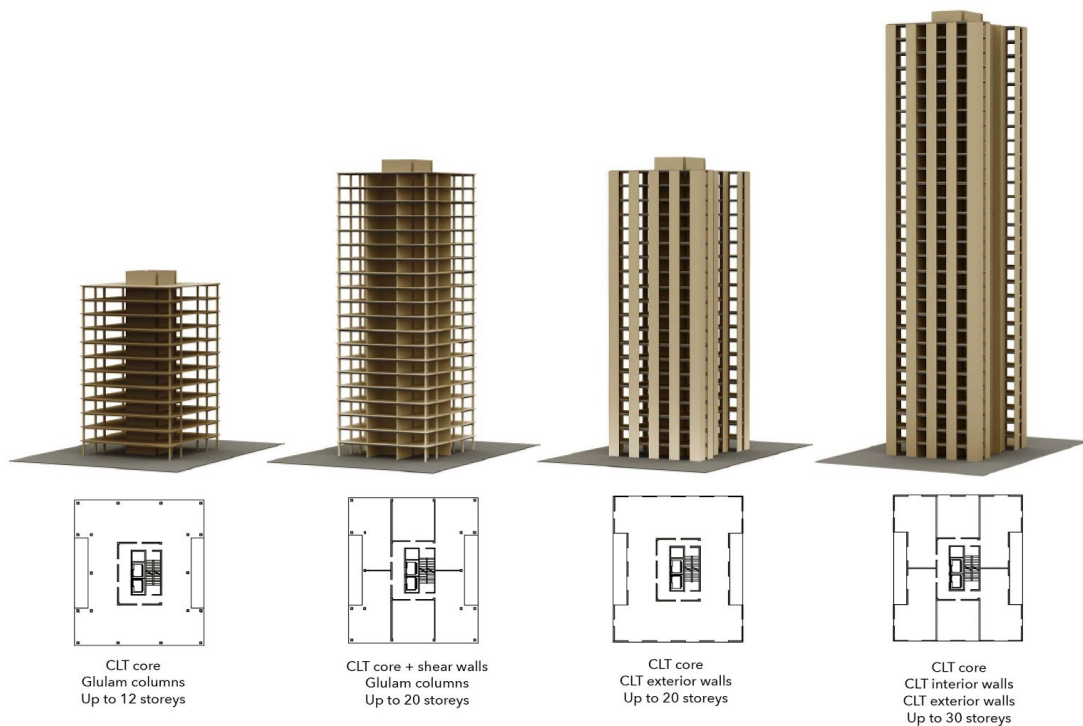
- Increase the effective base of the building, by using extended CLT wing walls
- Adding extra concrete in controlled locations (spandrel and link beams)
- Use tension piles with bell at the end bearing
- Continuous vertical lamination of steel plates inside bottom glulam columns.

The limiting factor for a timber high-rise is the stiffness, and not the strength. The material elastic stiffness for concrete is 3 times larger than the one for timber. This means that in order to design a lateral-resisting system with timber, the structure will have to either use more material, or distribute it more efficiently than in a normal concrete system.

Finding the Forest Through the Trees (FFTT)

Project	Concept study, 2012	Structural Engineers	Equilibrium Consulting Eric Karsh Robert Malczyk
Architects	Michael Green Mgb architects	Client	Wood enterprise coalition

The technical study illustrates the possibilities of Mass Timber products for the construction of Mid-Rise and High-Rise buildings. The report shows that buildings from 10 to 30 storeys can be realised with the use of Mass Timber techniques, and discuss the implications in architecture, market and climate change. It aims at providing a universal structural system, with different configurations, according to flexibility of floor plan and façade.



Summary of structural systems and estimated reference heights. (Source: mgb architects). From left to right, we can see an increase in feasible structural height but less flexibility in floor plan and façade.

Structure

The structure is a composition of CLT walls in two directions connected by steel link beams. CLT panels are used for floors, walls and vertical circulation core, supporting stability and gravity loading. In structures above 12 storeys, beams and ledgers made out of steel are used, coupling the shear walls movements in a "strong column-weak beam" approach, in order to provide some ductility in case of earthquakes and assure an easy construction.

The scope of the structural study was analysed with 4 cases studies:

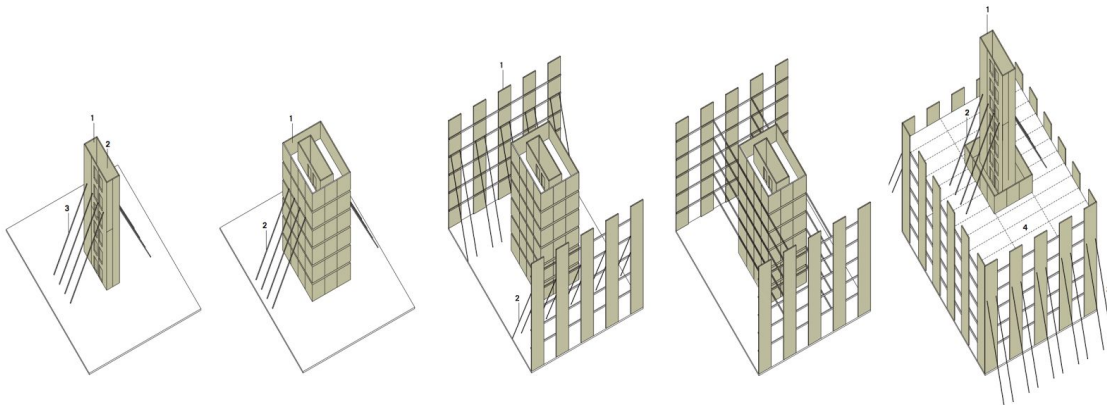
- 12 storeys building with core only
- 20 storeys building with core and interior shear walls
- 20 storeys building with core and perimeter moment frames
- 30 storeys building with core and perimeter moment frames and interior walls

The FFTT systems uses predominantly 255mm thick CLT, or 267mm LSL panels for floor and roof, with or without concrete topping.

Erection

All timber elements are prefabricated off-site. On-site, the timber panels are connected together on the ground, and then “tilted up” several stories at a time. The core will be erected first, in order to brace the later columns and walls, which can be up to 12m high (CLT) and 19.5m high (LSL/LVL).

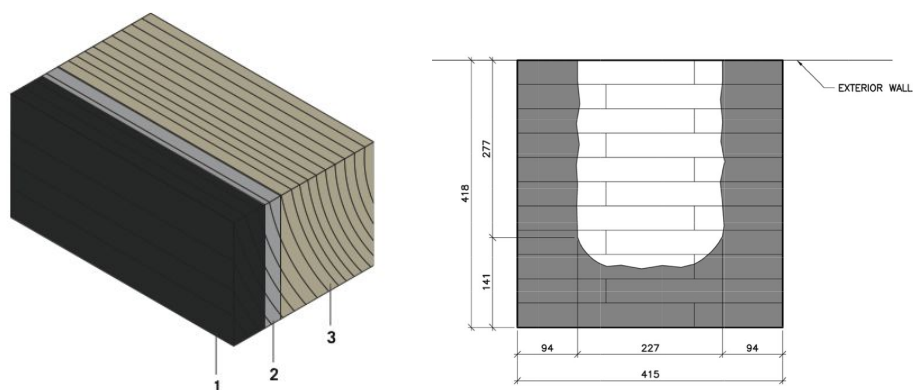
1. Installation inner core
2. Installation outer core
3. Exterior walls
4. Steel beams connecting core with outer walls
5. Floor panels
6. Repeat step 1-5



Sequence of erection. (Source: mgb architects)

Fire design

The structural elements are over-dimensioned according to the fire charring rate. Charring is a process in which the outer layers of wood reach its ignition temperature and burns continuously. In that process, there is a chemical reaction that leaves a layer of carbon. This layer has low conductivity, that isolate the inner element from fire effect. Previously, fire tests performed by FPInnovations, demonstrate 1-2 hours fire resistance capacity of solid timber elements when charring calculation method is applied (FPInnovations, 2011; Eurocode 5). Additional fire design implied modern fire suppression systems and compartmentalization of the building. At the same time, especial attention must be put into the detail to assure that the structural elements are fully protected with the layer of mass timber.



Left, charring diagram. Char layer (1), Pyrolysis zone (2), Normal wood (3), and design charring for glulam column exposed on 3 sides (right). (Source: mgb architects)

Additionally the following fire protection systems can be used in the building:

- Compartmentalisation of the floor layout, in separate fire zones

- Flame spread rating of the exposed materials that may contribute to fire temperature
- Encapsulation of structural elements with non-combustible materials, such as gypsum or cement boards
- Scissor stair design and protection of connection hardware
- Automatic sprinkler system
- Enhanced fire detection systems

Conclusions

CLT timber structures can be coupled using steel link beams using “strong column - weak beam” approach, and allowing high ductility of the structure. This has advantages for earthquakes-induced movements.

Up to 5 storeys vertical elements can be prefabricated for quicker erection process.

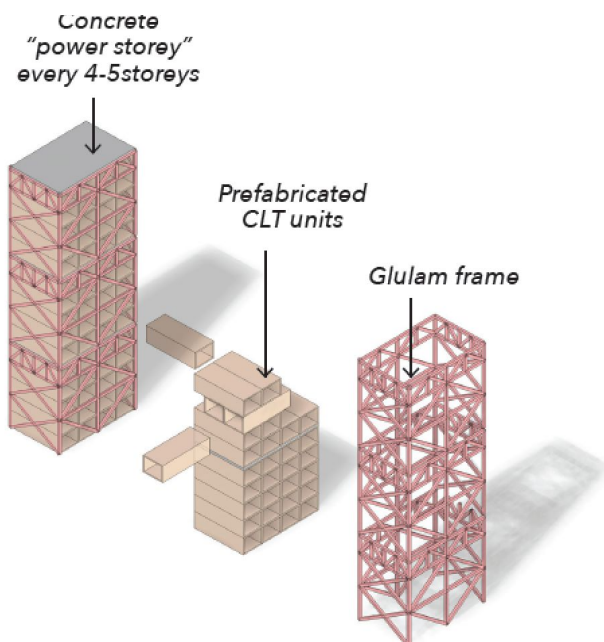
3. STRUCTURAL CONSIDERATIONS FOR TALL TIMBER (TT)

- Typologies
- Uplifting forces
 - Shape of the building
 - Distribution of weight
 - Foundations
 - Reinforcement structural elements
- Horizontal sway
 - CLT coupled with concrete link beams
 - CLT coupled with steel link beams
 - Post-tensioned CLT with concrete outriggers
- Differential shortening
- Floor systems
- Detailing
- Conclusions

3.1 TYPOLOGIES

Trussed glulam frame with stacked prefabricated modules

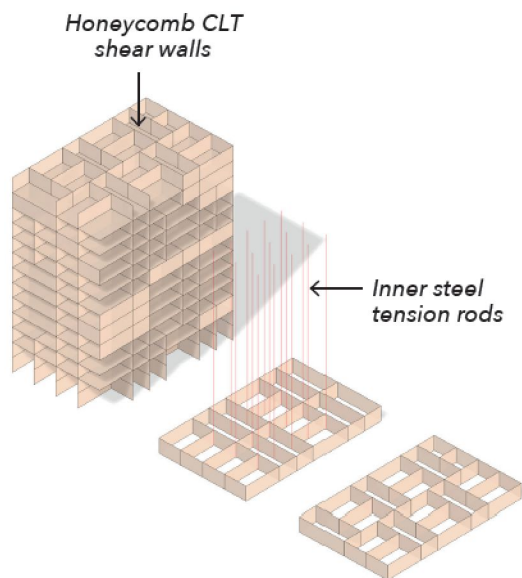
A perimetral trussed-frame made out with glulam elements supports lateral loading. The systems is complemented with prefabricated modules, that can serve as housing units and come with finishings and services integrated. They are stacked inside the glulam frames. Intermediate concrete slabs and smaller glulam trusses distribute gravity loading from the modules to the glulam frames and separate them from the lateral resisting system. Because of the lightweight system, the main limitation is related to wind-induced accelerations. Some solutions to mitigate the problem can be addition of extra mass, or using alternative damping systems. The Treet building in Norway with a height of 53 meters is the only built precedent that uses this typology.



*Precedent height: 53 meters
 Lateral stability: Braced glulam frame
 Vertical loading: CLT modules
 Reinforcement: Concrete power storey
 Example: The Treet (Norway)*

Core and shear CLT walls

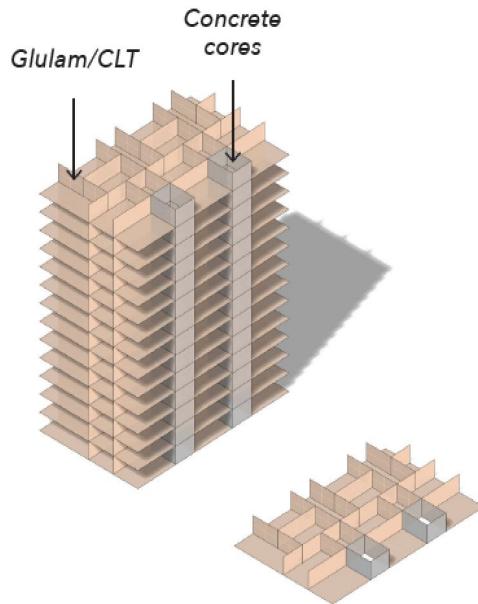
One of the most conventional structural system of tall timber construction. The structure relies on more or less intricate compositions of CLT panels, distributed along the height of the building and around vertical circulations. Structurally these elements, behave as shear walls providing lateral stability to the building. The main weak point of the system is the coupling of the elements, vertically and horizontally, as contrary to concrete, timber is prefabricated in discrete elements. Some solutions such a coupling of CLT panels with steel or concrete beams, or introducing steel tension rods inside the panels, may mitigate this problem. At the time of writing the highest precedent using this typology was Forte building in Australia with a height of 32.2 meters.



Precedent height: 32.2 meters
Lateral stability: CLT shear walls
Vertical loading: CLT walls
Reinforcement: Steel vertical tension rods or shear plates
Examples: Forte (Australia), Murray Groove (UK)

Concrete core (stability) and CLT panels (gravity)

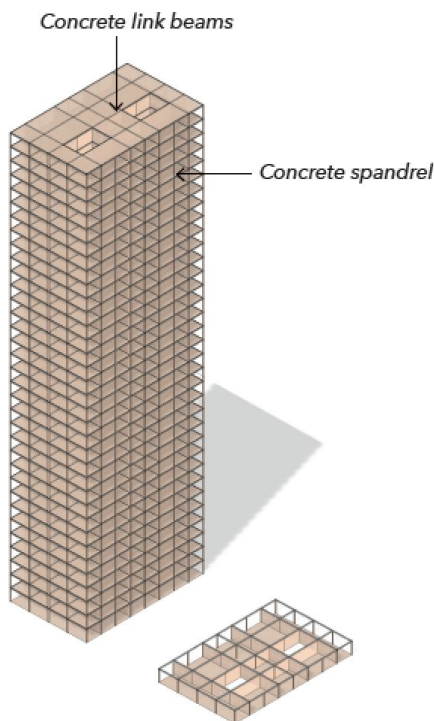
This typology can be considered a composite version of the previous one. The addition of concrete for vertical circulation cores, provides enhancement towards fire, (concrete is traditional non-combustible material accepted by fire authorities), and stability, (concrete is continuous monolithic material with higher shear stress capacity than timber.) CLT panels or glulam columns with additional steel connections, complete the structural system for gravity loading. Because of economical considerations, this is may be the preferred option for many developers, that want to introduce timber in high-rise structures without the assuming too many risks (fire authorities approval and extra fire dynamic vibration and connections research). The highest built precedent up to date with this systems is UBC Brock Commons in Canada with a height of 53 meters.



Precedent height: 52.8 meters
Lateral stability: Concrete cores
Vertical loading: CLT walls/columns
Example: UBC Brock Common (Canada)

CLT shear walls coupled with link beams

The structure is composed out of multiple CLT shear walls in multiple directions, similar to previous examples. The system is strengthened by steel and concrete link beams, that ensure the coupling of movements, and moments and shear transmittance. Therefore, the connection between beams and panel is very important for the reliance of the typology, and special attention to the detail must be considered. SOM 2013, and Michael Green 2012, both rely on this methodology, using concrete and steel beams respectively. At the time of writing, there are not built precedents using this system.



Precedent height: 120 meters
Lateral stability: CLT shear walls and outrigger glulam
Vertical loading: CLT walls/columns
Reinforcement: Concrete spandrel and link beams
Example: Concept study by SOM (unbuilt)

3.2 UPLIFTING FORCES

Net uplift due to lateral loading occurs when lateral loading overcomes gravity loading, causing the building to lift up and placing tension in vertical elements and foundations. Members in tension are more difficult to design and construct. Net uplift is usually not a problem in concrete structures because of the heavy material weight. However timber is a very lightweight material, and consequently net uplift due to lateral wind loading is a major issue in the structural design of tall timber buildings. As a result, tension forces may occur in vertical elements and foundations, leading to a more complicated construction. The following table compares overall densities for concrete and timber buildings, using the technical information in the case study for a Timber-Composite building (SOM 2013)

Type of structure	Total Weight/Total Volume	Dead Weight/Total Volume
Benchmark building. Dewitt-Chestnuts Apartments (Concrete)	336kg/m ³	288kg/m ³
Prototype building. (SOM 2013) (Composite Timber-Concrete)	160kg/m ³	112kg/m ³

Comparison of densities of concrete benchmark and timber prototype buildings (Source: SOM).

For reference, density of balsa wood is 128kg/m³

The following strategies can be used to mitigate net uplift:

Shape of the building

Decrease height

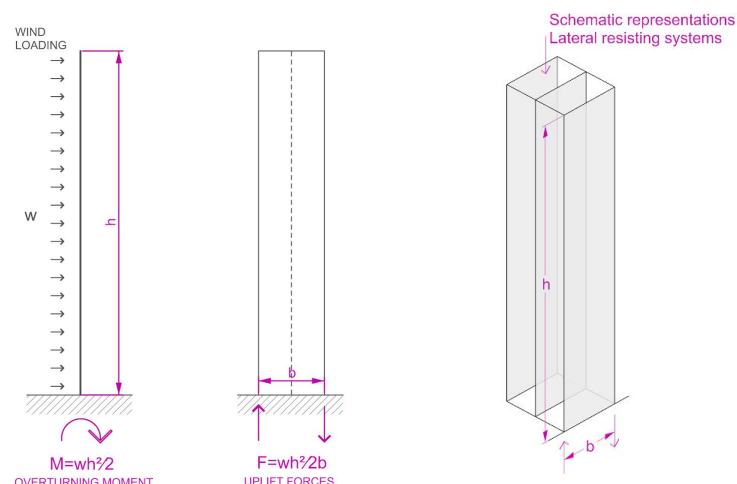
Because the magnitude of the overturning bending moment is dependant on the height of the building to the power of two, this will have the most significant contribution to the uplifting forces. Needless to say, the height of the building has also an impact in architectural and economical factors of the building.

Increase effective width or base of the lateral loading resisting system

Increasing the distance between the central axis of the stability system and the perimeter, leads to a linear reduction of the uplift forces. As above-mentioned, the base of the stability systems has an impact on overall architectural and economical demands.

Addition to more lateral resisting systems

This results in a proportionally linear reduction of uplift forces according to the number of lateral loading resisting systems.



The total gravity loading (beneficial for mitigating uplift) increases with the number of storey and thus height of the building in a proportional way. At the same time, the uplift forces increase to the power of two. This means

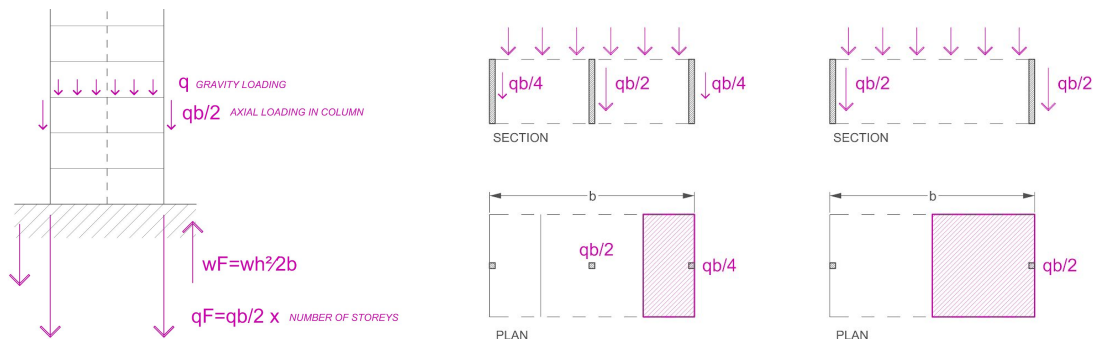
that, making the building heavier by adding more storeys will not mitigate uplifting forces, as height will also increase, and therefore overturning moments at the bottom.

Distribution of weight

It is probably the easiest method in order to reduce uplift forces. A common solution is to add concrete on the top of floor slabs, or by creating 1-2 storeys concrete plinth. An increase in the gravity loading leads to higher compression forces, thus reducing tension forces in the windward part of the structure and anchoring the building down. If the effect is repeated in every storey small modifications can have significant contributions to the total overall behaviour.

Maximising the amount of gravity loading that is supported by lateral-resisting system

By concentrating maximum weight in the lateral resisting system, it minimises potential net uplift at the bottom of the building. This can be done, by creating floor spans from core to perimeter, avoiding interior columns that may reduce gravity loading in the perimeter of lateral resisting system. The repetition of this effect is particularly important for high-rises with multiple storeys.



Wing shear walls reaching perimeter

By using the structure at the perimeter for lateral loading, the effective width of the building increases, and consequently the uplift forces are reduced. Alternative systems such as outrigger with CLT panels or glulam trusses, belt trusses or perimeter braced frames can also be used.

SOM carried a small study of three types of lateral load resisting system, in order to analyse the capacity to minimise net uplift forces. The conclusion was that the efficiency of the system is highly dependent on the “effective width” of the building:

- 90% improved efficiency by using, “wing” shear walls extending up to the perimeters
- +10% improved efficiency by thickening primary walls (20%) and secondary walls (40%)

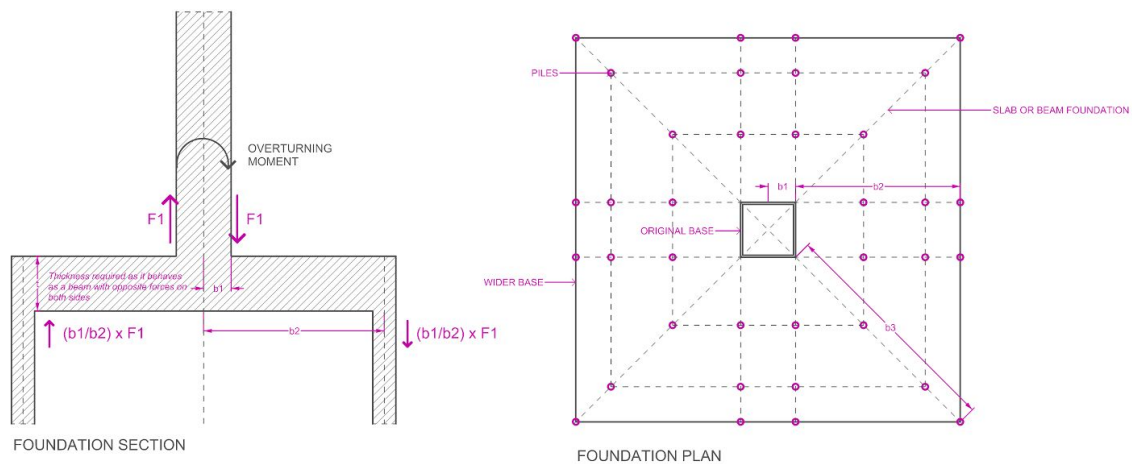
Central core	Core + Wings	Core + Wings, refined thickness
Beff = 6,1 meters	Beff = 11,58 meters	Beff = 12,8 meters

Comparison lateral system effective widths (Source: SOM)

Foundations

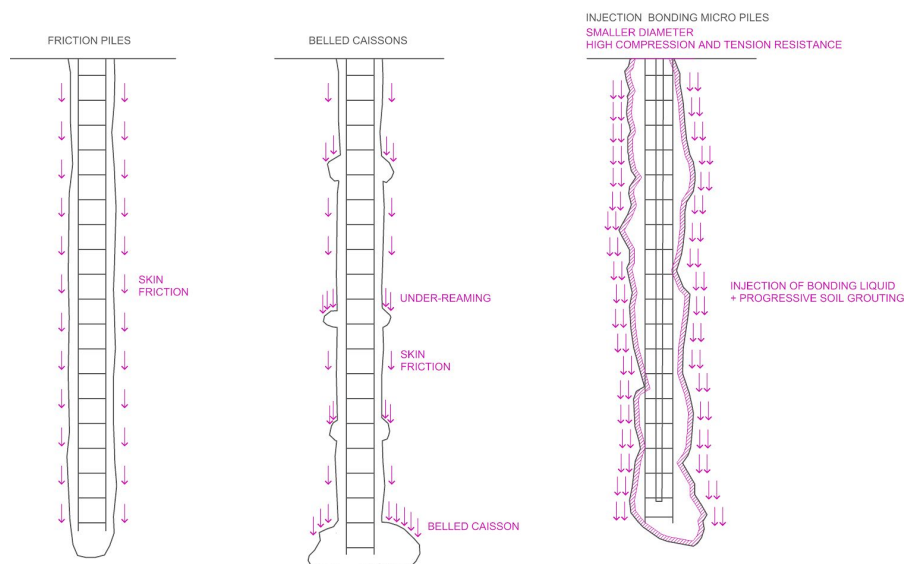
Increase foundation base

The overturning moment at the base of the structure can be reduced with a proportional factor equal to the increment of the foundation width. In the above image, a schematic explanation can be seen, where F_1 is original uplifting force and F_2 is the reduced uplifting force by the factor b_1/b_2 . Additional foundations grade beams or continuous foundation slabs for distribution of uplift forces to multiple piles, and continuous vertical reinforcement at lower depths must be required in the foundations.



Use tension piles

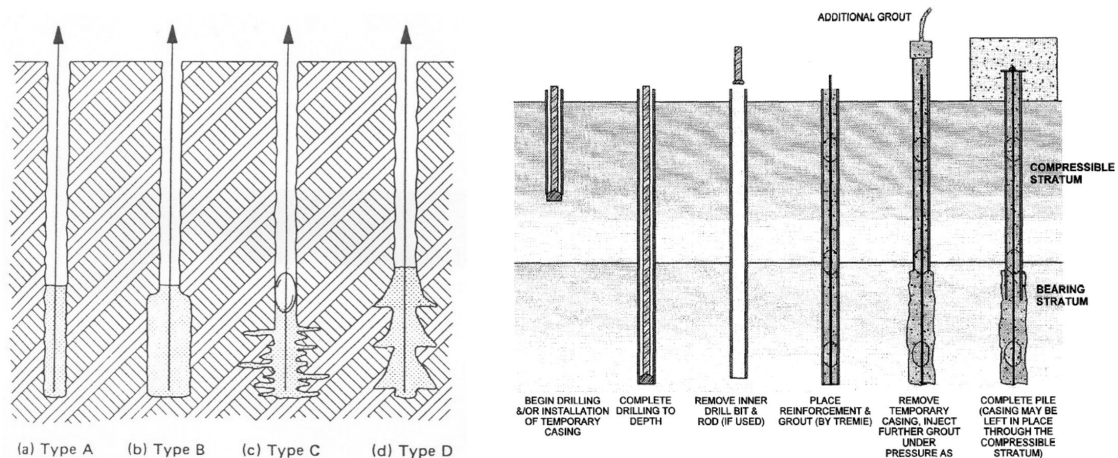
Tension piles or anchor piles resist uplift forces with only the action of friction along their length, in contrast to conventional piles that also use the end bearing. Tension piles transfer the load in three possible ways, friction with the ground (conventional piles), under-reaming or enlarging of the pile (bored piles with bells) and by grouting/bonding with the soil (micro piles or anchoring piles). Continuous vertical reinforcement should also be used along the length of the concrete piles.



Use small diameter bored injection piles or micro piles

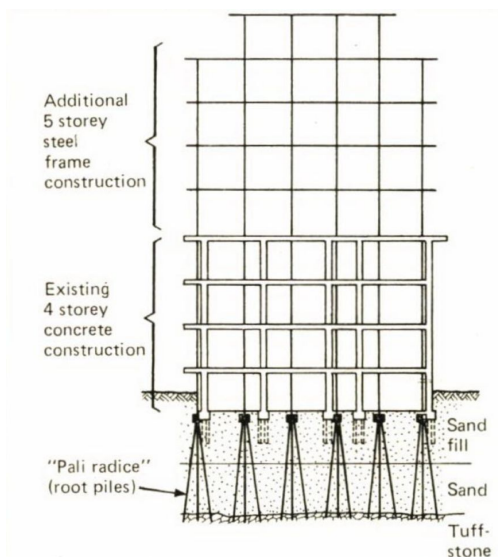
These piles offer a number of advantages relative to traditional piling: they have a high carrying capacity, the equipment of the pile installation is small, roughly the size of a pickup truck, the size of the pile is smaller (specially in height) and they are constructed without vibration and noise. The only disadvantage of micropiles is

the relatively high cost compare to traditional piling systems. Micropiles can also be known under names such as root-piles, needle-piles, pali radice or mini-piles.



Left, types of micropiles according to grouting method. Right, typical construction sequence (Shong et. al. 2003) In the above image, four main types of micropiles can be distinguished. A, grout is placed only under gravity head; B, grout is placed under pressure (0.5-1MPa), C, grout is placed under pressure (1-2 MPa) causing hydrofracturing of the surrounding ground, D, similar to process C, but modifications in the secondary grouting

Because of the mentioned advantages, these piles are the ideal choice for restricted job sites, even inside pre-existing structures One main application for these piles is for preservation of buildings, where they can also be used for strengthening existing foundations. Another application is for construction on top of existing buildings. In the image below, we can see a five storey building in Naples that was founded on new columns that were constructed from inside the existing building and supported by Pali radice (root piles).



Building on top of an existing building with "pali radice" (Source: Fondedile, 1964)

The main reason for the high carrying capacity is the special method of construction, during which concrete or mortar is forced into the soil. Micropiles can be constructed in all types of soil, and they can be drilled in any direction, the base of the pile is not enlarge so the load is transferred into the soil only through skin friction. Axial loads in both directions, compression and tension forces can be applied, but nevertheless only small bending moments can be accommodated. The diameter of a conventional bored pile is generally 300mm, while in the case of a "micro piles" pile is between 120-250mm. This said, the load carrying capacity is comparatively higher:

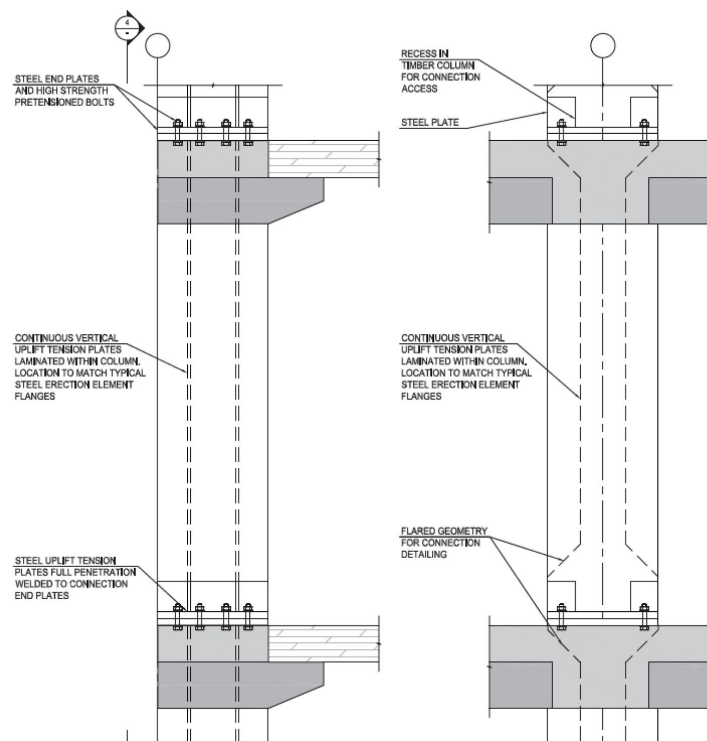
Type of pile	Diameter (mm)	Compression/Tension load (MN)
Conventional bored pile	400	0.30-0.37
Micropile RC 8x16mm	150	0.46/0.3
Micropile RC 8x22mm	250	1.02/0.73
Micropile Gewi Steel bar 50mm	125	0.6/0.6

Comparison of conventional and micro-piles dimensions and load capacity.

Reinforcement structural elements

Continuous vertical steel plates reinforcement inside glulam elements

The peak ultimate uplift tension, for SOM case study is approximately 4450KN, located at the four pilaster columns directly connected to the shear wall system. The uplift effect occurs from foundations to the 6th level. Continuous vertical steel plates are laminated within the mentioned columns in order to reinforce the glulam elements for tension. These steel plates are also bolted through the joints of the columns for distribution of the load between storeys. In the ground and basement level, made with concrete, the uplift is resisted by mechanically spliced vertical reinforcement.



Steel lamination inside timber for net uplift forces. (Source: SOM 2013)

Steel tension rods inside CLT panels

The use of steel tension rods inside CLT shear walls connecting the elements vertically from foundation to top can help in resisting wind-loading and uplift forces and anchoring the building to the concrete base, by absorbing tension forces. In addition, the use of these vertical rods eliminates the need of more complex load-transferring connectors between wall elements.

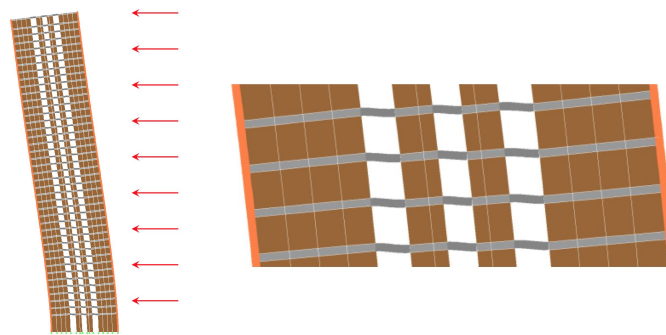
3.3 HORIZONTAL SWAY

It is common in timber buildings, to have excessive horizontal displacements due to wind-loading, this is due to the weaker mechanical properties of timber, compared to steel or concrete. In order to counteract that behaviour, increasing the base of the building, reducing the height, or adding more stability systems are the

most conventional solutions. Nevertheless, using multiple vertical elements for the same stability system, e.g. multiple CLT shear wall, may be another effective option, that in principle increments the lateral capacity of the structure. However, each of the lateral systems must be structurally connected with the others, to behave appropriately and act in a global composite way. In comparison with cast-in-place concrete, timber structures cannot be erected in a continuous monolithic way. Another innovative solution developed by (Xia and van de Kuilen, 2010), is to use post-tensioning cables for achieving a more efficient vertical coupling of CLT panels. In conclusion, special attention must be placed into systems that are capable of achieving more efficient couplings between CLT structural elements, both vertically and horizontally.

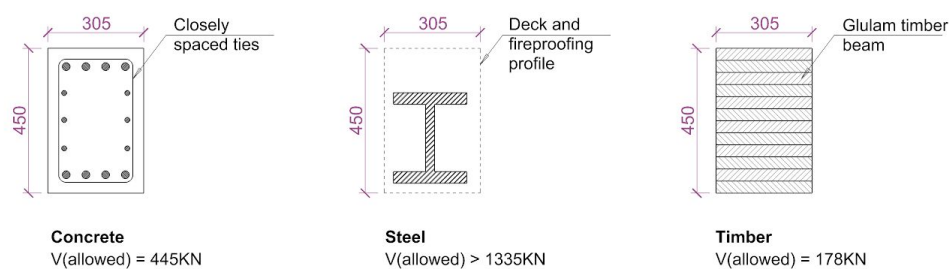
CLT coupled with concrete link beams

In SOM 2013 technical study, concrete beams are used to connect the different CLT panels, and as a result, high stresses in those elements are derived. However, as it is mentioned in SOM report, an increase in the size of the elements will influence the floor-to-height of the building, resulting in higher overturning moments. It is therefore decided to restrict the volume of the beams to a maximum of 457x406mm. A small study is performed with link beams in concrete, steel and timber, in order to evaluate the structural feasibility of each material.



Lateral loading resisting system and "link beams" deformations. (Source: SOM)

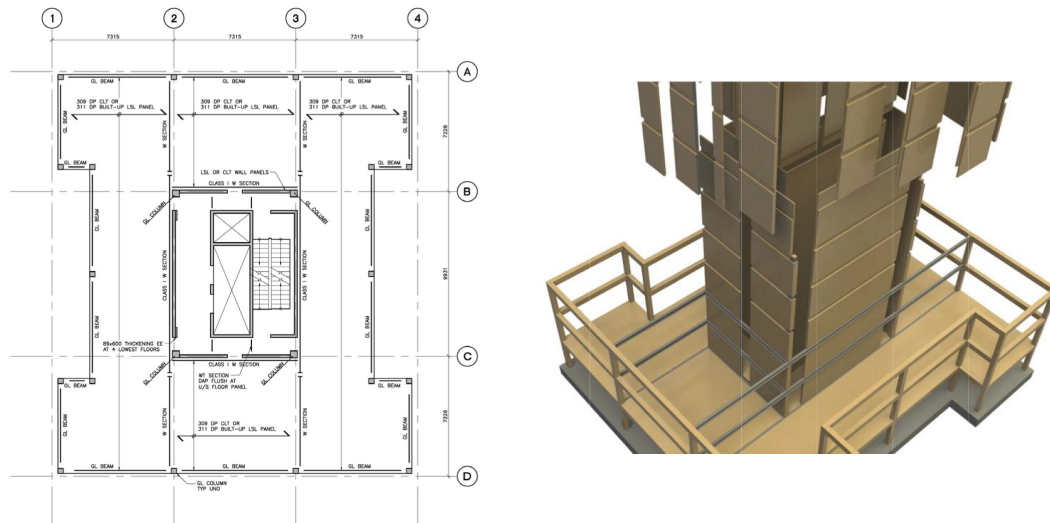
The coupling shear forces that these members must resist is the range between 445-225KN, in a maximum of 305x450mm space. For reference, a similar size timber beam, would be able to resist 178KN. It is clear that timber beam has low shear capacity compared to other structural materials, and it is one of the primary reasons why concrete joints and beams have been used for this conceptual project.



Comparison between different structural materials for 305x450mm maximum available space. (Source: own elaboration based on data from SOM 2013)

CLT coupled with steel link beams

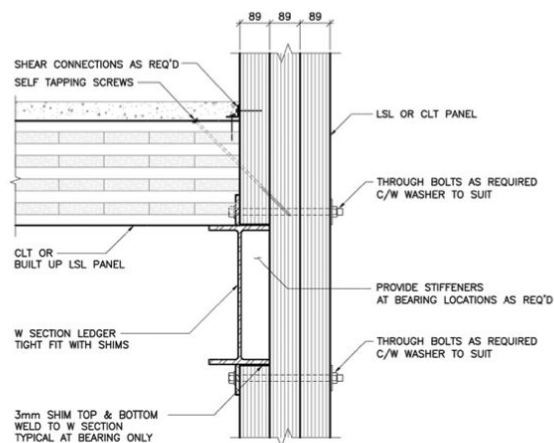
In the concept study FFTT by Michael Green in 2012, steel link beams were used to couple the CLT shear walls. The structural system is based on the "strong column - weak beam" approach. The vertical CLT/LVL panels acts as "strong" columns coupled by "weak" ductile steel beams. These beams have been designed as wide flange steel headers of class 1. The headers are proportioned to yield near the edge of the wall panels at near design loads, contributing to the overall ductility of the system. This will provide flexible and reliable high-ductility moment frames with simple connections, without the risk of brittle failures.



Floor plan layout (left) and 3D scheme (right). In the image, we can see the steel coupling walls inserted into the CLT vertical panels. (Source: mgb).

The vertical joints between adjacent panels ends consists of lapped joints (+/- 150mm wide) connected with a large number of self-tapping mechanical fasteners over the full height of the core. The horizontal joints can also consists of lapped joints (+/- 600mm wide) connected with mechanical fasteners and reinforced with shear keys for higher shear loading.

The “weak” ductile steel beams coupling the walls are partially embedded into the timber panel, connecting individual the shear wall panel together, and ensuring a composite action. These elements are designed to develop plastic hinges near design loads. Alternative link beams and associated connection arrangements can also be considered, such as back-to-back ductile channels connected directly to the face rather than being embedded.

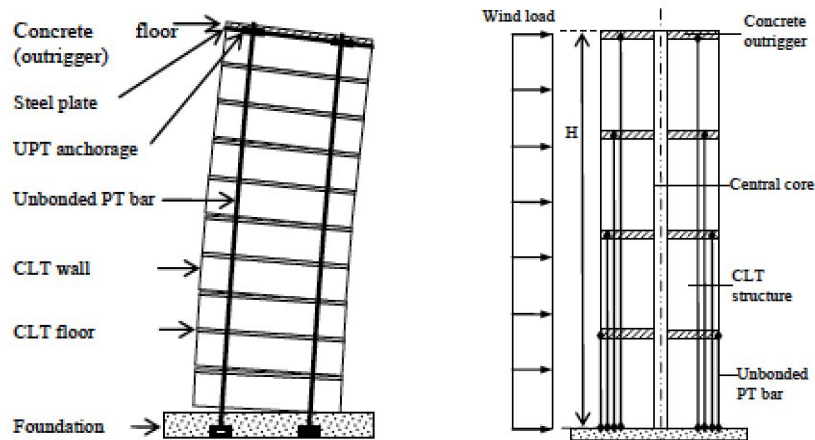


Detail of coupling steel beam inserted into CLT vertical walls (Source: mqb architects)

Post-tensioned CLT with concrete outriggers

In 2010, Zhouyan Xia and Jan Willem van de Kuilen, from Technical University of Delft, proposed an innovative system for a 40 storeys building using high-strength post-tensioned steel bars inside CLT panels. The study is carried out for a typical residential high-rise in Shanghai, with three apartments of approximately 100m² per story, 132 meters total height and 3.3 storey height. Overall dimensions are 33.7x 15.7 meters. The structural system uses unbonded post-tensioning steel bars inside the vertical CLT walls, in order to restrain horizontal

sway in the building. The system is based on the experience learned from precast concrete systems. An example of this solution was used in the Limnologen building in Sweden.

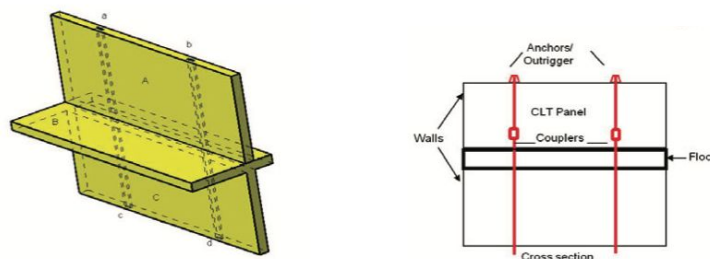


Unbonded post-tensioned CLT wall under lateral loads (left), and example of tall CLT building with four concrete outriggers (right) (Source: Xia, et. al. 2010)

Traditional concrete constructions are usually composed of monolithic continuous concrete systems. On the other hand, with CLT, the overall structure is composed out of discrete elements. Most of the CLT tall buildings built up to date, use conventional fasteners, such as hold-downs, angle brackets and screws in order to connect the elements together. Even though, this connection methods are simple and fast, the connections stiffness have to be assessed accordingly in order to predict the overall building behaviour. This may be one of the limiting factors for CLT tall buildings up to date, with maximum height of 32.2 meters.

In the first scheme above (left), the steel bars run along the full height of the building and are placed inside oversize ducts embedded inside the CLT panels. The post-tensioning system is only connected to the CLT panels with end-bearing panels at the top of the structure, and at the foundations. In this way, a large number of brackets can be avoided.

In the second scheme (right), a 40 storeys CLT building is separated every 10 storeys by concrete outriggers. Four groups of post-tensioned bars with outriggers are installed within the walls introducing extra compressive forces. The unbonded PT bars are fixed to the foundation and anchored at corresponding outriggers. The main aim is to increase the horizontal stiffness of the building by counteracting tensile stresses caused by wind-loading with the stresses induced by the post-tensioning of the steel bars. Consequently, a reduction in global horizontal displacements of the building and a decrease in the large number of conventional connections (brackets, angles) can be achieved.



Unbonded PT steel embedded inside CLT wall. (Source: Van de Kuilen, et. al. 2010)

As it is shown in the above images, high strength PT bars are placed inside oversize ducts that are embedded inside CLT panels. The thickness of core walls is varied along the height of the building, in groups of 10 storeys, 350, 300, 250 and 200mm from the base to the top. The thickness of CLT side walls and floors is consistent, being 300 and 250 respectively. The post-tensioning bars are made with high strength steel with minimum ultimate strength of 1035Mpa.

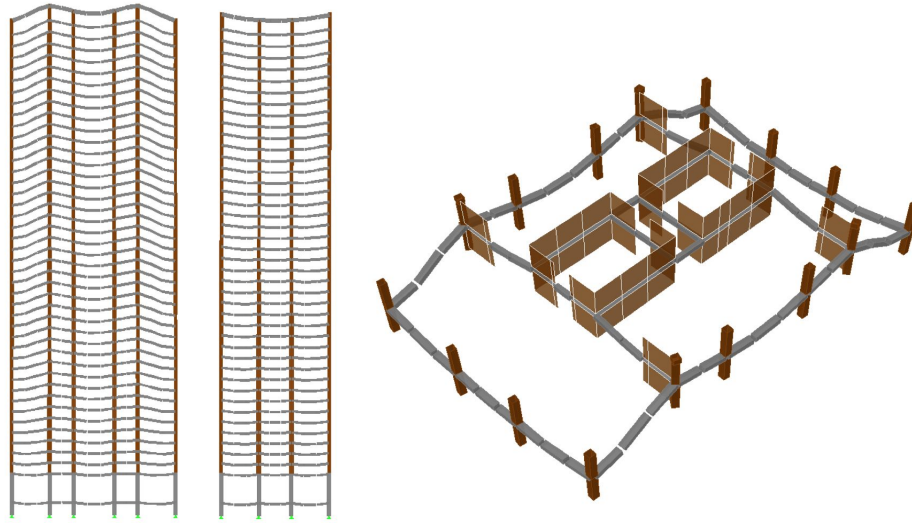
At the same time, with the combination of the concrete outrigger system, the following benefits can be achieved: more stability to the core, a lever arm for global bending moments, fire compartmentation of the building along its height and shelter spaces in case of fire emergencies.

(Xia & Van de Kuilen et al. 2010), also indicated that further studies should be performed where stronger outriggers with steel, concrete or even CLT can be used as mega structures together with smaller timber sections infilled in.

3.4 DIFFERENTIAL SHORTENING

In high-rise buildings, shortening of the elements may occur due to the great accumulation of loading. At the same time, in wood and timber structures long-term shortening may also happen because of shrinkage or creep of the materials. In general, differential shortening between elements must be prevented or controlled.

The case study of Timber Prototype building developed by SOM will be studied in order to understand common features of differential shortening in tall timber buildings:



Schemes of differential shortening in SOM prototype timber building.
Expected differential shortening from 500-750 mm (Source: SOM 2013)

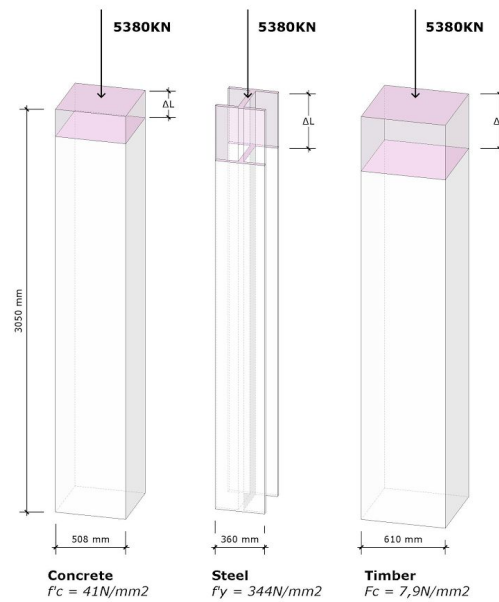
The shortening of an element can be defined with the following two equations:

$$k = \frac{F}{\delta}$$

where, k is the axial stiffness of the element, F is the force applied to the body and δ is the displacement produced by the force along the same degree of freedom (for instance, the change in length of a stretched spring).

$$k = A \cdot \frac{E}{L}$$

Where, A is the cross-sectional area, E is the Young's Modulus of the structural material and L is the length of the element



Scheme for loading assumption with loading of 5380kN, and height 3050mm. The table below summarises the cross-sectional properties, axial stiffness of the element and the resulting differential shortening, according to columns size for strength:

Material	Cross section (mm)	Axial stiffness (kN/mm)	Differential shortening (mm)
Concrete (f'_c 41N/mm ²)	508x508	2730	2,03
Steel (F_y 344N/mm ²)	W14x99	1225	4,31
Timber (F_c 7,9N/mm ²)	610x610	1173	4,57

Axial stiffness comparison for columns height 3050mm, loaded with 5380kN (Source: SOM)

The material elastic stiffness for concrete is 3 times larger than the one for timber. This means that in order to design a lateral-resisting system with timber, the structure will have to either use more material, or distribute it more efficiently than in a normal concrete system. Timber and steel have similar differential shortening, for element sized according to strength

According to SOM, the total shortening for the timber prototype building is expected as 800mm, 50% higher than in a similar structure made with concrete. Further research must be done in order to partially compensate this issue.

3.5 FLOOR SYSTEMS

Needless to say, the assembly of the floor system has a great impact in high-rise constructions, due to the high recurrence, having an important effect in the amount of materials for the overall structure. Some of the common issues that govern the design of timber floor systems for tall structures are fire protection, span/depth ratio, acoustic considerations and facility of erection.

In addition the thickness of the slab has a direct effect on the floor-to floor height of the building. I.e, increasing the overturning moments, and consequently net uplift at the foundations, and increasing the area of façade.

Simple supported

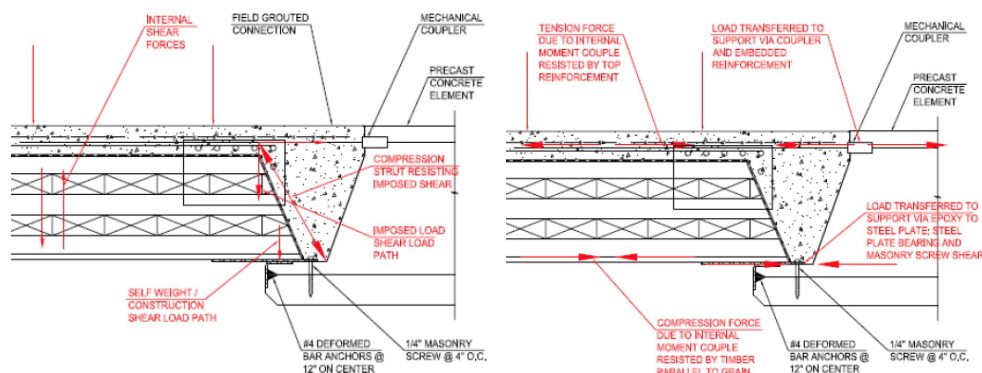
Michael Green, 2012, using the concept study FFTT system, compared alternative configurations for floor assemblies, regarding layer composition and span/depth ratios. The panels are assumed to span one way over interior steel beams which also act as link beams. The following alternative floor systems were compared in the study:

Floor type	Floor layer composition	Span (mm)	Depth (mm)
CLT	55mm concrete topping 25mm rigid insulation 190 CLT (5 layers)	8000	270
CLT - Concrete (composite)	75mm concrete topping 19mm rigid insulation 114 mm CLT (3 layers)	12000	208
LSL - Single layer	38mm concrete topping 19mm rigid insulation 89mm LSL panel	2400	146
LSL - Double layer	38mm concrete topping 19mm rigid insulation 178 LSL panel	6000	235
LSL - Triple layer	38mm concrete topping 19mm rigid insulation 267 LSL panel	6000	324
LSL - Concrete (composite)	75mm concrete topping 19mm rigid insulation 89mm LSL panel	6000	183

Comparative table with different timber-composite floor systems (Source:mgb)

Fixed connection

"The Concrete Jointed Timber Frame" by SOM 2013, propose a rigid moment-tight connection at the floor edge in order to reduce peak bending moment and stiffen the floor. As a result, less deflections and vibrations can be achieved with the same floor depth, in comparison with a simple supported option. The floor is structurally connected along the edge to a concrete spandrel beam, that is designed to resist torsion. The reinforcement is similar to top bars provided in a concrete slab. At the same time, the timber floor edges rest on 30 cm prefabricated concrete corbel, that gives additional rotational restraint to the floor panel and provides passive fire protection.



Explanation of forces transmission for shear (left) and moment (right) in floor connection. (Source: SOM)

If a fixed connection is used, a 200mm thick CLT panel can be used, a substantial reduction in the floor thickness. If the end floor edge is pinned supported, this would require a 320mm thick CLT panel. Alternatively, a ribbed panel system can be used with 355mm deep ribs. The last two options, mean more use of material and an increase in the overall building height, resulting in a more expensive building.

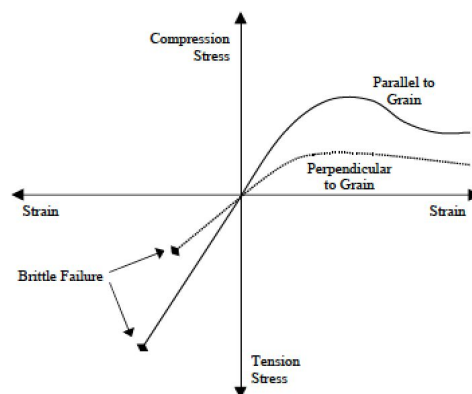
Floor type	End support condition	Equivalent slab thickness (mm)	% change in total amount timber used
Slab type. CLT or similar timber panel	Simple supported	317	38
	Partly fixed	244	14
	Fully fixed	203	Baseline
Ribbed type. CLT panel + glulam beams	Simple supported	241	13
	Partly fixed	205	1
	Fully fixed	185	-6

Table comparing different design options for floor system. (Source: SOM)

Note that the table indicates the “Equivalent slab thickness” and not the total overall dimension for the floor systems. In addition, other economical considerations, such as the more elaborate construction of ribbed elements opposite to simple panels, or the epoxy-bonded connection are not considered.

According to SOM 2013, the effect of the moment-rigid connection of the floor edge can restrict the total materials of the building by 20-25%. As the structural behaviour of the connection is critical for the performance of the structural system, physical tests (structural and fire) and further research must be performed. At the same time, the difference between fixed and simple support condition of the floor systems, results in 5% additional building height, 10% increase in overturning bending moments, and 20% net uplift due to wind.

3.6 DETAILING



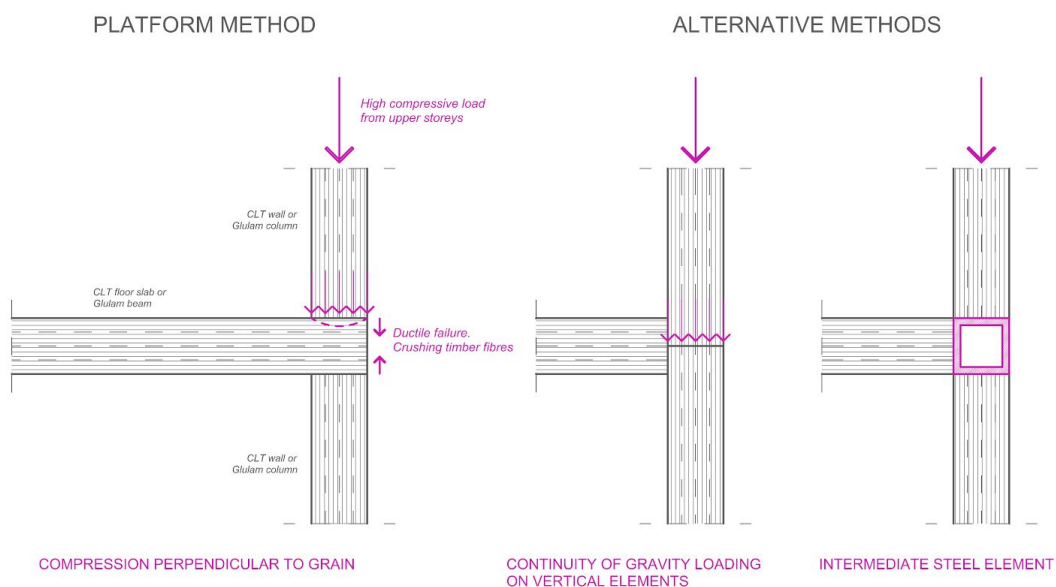
Typical load-deformation relationships for wood parallel and perpendicular to grain.
(Source:FPIInnovations, 2013)

Wood has different strength properties in different directions relative to the grain, and it is therefore categorised as an anisotropic material. In the image above, we can see that mechanical properties of timber (compression and tension), are significantly lower perpendicular to grain. In addition, when loaded in tension perpendicular to grain, timber elements experience a brittle failure.

Consequently, when designing structural members and connections, tension perpendicular to grain should be avoided as it can cause sudden catastrophic failures endangering human lives. On the other hand, compression failure experiences a more ductile effect that is more favourable.

In the structural design of tall buildings great accumulation of gravity loading occurs at the lowest storeys, causing great compression forces. In traditional construction methods with timber, it is common that vertical loading from columns rest on top of lower beams or slabs. This is usually called, platform method. One example of this system was used in the 9 storeys of Murray Groove building in London.

However, due to the mentioned inherent anisotropy of timber material, compression perpendicular to grain is significantly lower than parallel to grain, a limitation that can govern the structural design if a platform construction is used. Therefore, a tall timber structure should minimise potential compression perpendicular to grain in connection interfaces.



One possible solution is to allow continuity between vertical elements. For example, by creating continuous columns, and hanging beams with metal hanger or concealed plates, or design larger sections that can partially rest on the posts. For ease of assembly and simplicity of connections it is recommended to leave some distance between column-column and column-beam connections.

For CLT systems, floor and walls panels edges can be profiled to fit together like a tongue and groove. This would provide partial bearing of the floor slab on the wall, and at the same time, allow direct transfer of gravity loading through the walls. According to Techniker, engineers of Murray Groove building, using this method, up to 25 storeys can be erected with economical wall thicknesses. (Techniker engineers, 2009)

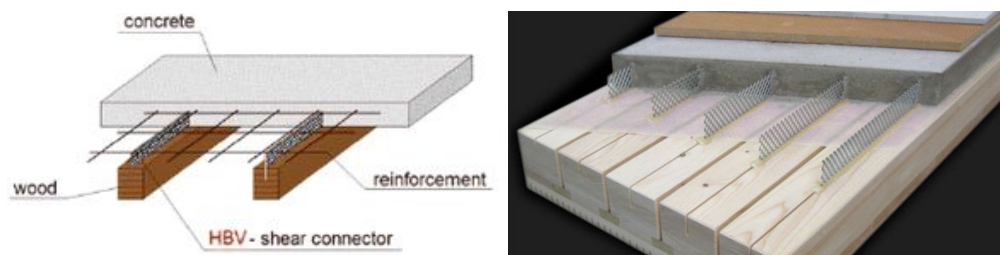


Examples of continuous column connections (left) and interleaved CLT walls and floor panels (right).
(Source:FPIInnovations, 2013)

Shear connector Composite Concrete Timber

The choice of an effective shear connection is the key to achieve strong and stiff composite systems. Some systems use wet concrete poured in-situ on top of timber. However, these usually have lower stiffness and higher creep during concrete curing, higher cost and problems with quality control. The prefabrication off-site of a concrete slab with already inserted shear connectors, can reduce the aforementioned issues.

One type is the toothed shear connection. They are usually 55x55x250mm metal plates folded at an angle of 90°, and moulded into the concrete slab. The pre-cast concrete slab with the toothed plate is then pressed into. Another recently developed type is the HBV connector, longitudinal perforated steel plates that create the interface between concrete and timber.



Example of prefabricated timber-concrete composite with HBV shear connection (Structure craft)

Kanócz 2015, investigated 4 possible configurations of floor systems bonded by adhesive Sikadur T35 LVP. The main advantage of this adhesive is that is moisture tolerand and thereby possibility of bonding wet concrete with timber. Several authors have proved that glued shear connections can provide almost full composite action

In the case of all timber lamellas oriented in span direction, the timber-concrete composite connection can be assumed ideally rigid. This is not the case for CLT when crosswise layers have different grain orientation and rolling shear stiffness occurs, causing stiffness reduction. The glue composite connection between CLT and concrete is rigid connection with $\gamma = 1.0$. Experimental tests showed higher bending stiffness with adhesive connection compared to mechanical fasteners.

For the purpose of this study, an adhesive connection between concrete and timber with ideally rigid behaviour will be assumed.

Stiffness of connections

In Eurocode 5 (CEN 1995 2004), simple design guidelines are given in order to evaluate the stiffness of timber connections according to the type. For slotted-in steel plates with multiple dowels, it implies that the stiffness is proportional with the number of dowels and shear planes. For a dowel fastener connecting steel and timber the stiffness modulus is given per dowel and shear plane by:

$$K_{ser} = 2\rho_{mean}^{1.5} \frac{d}{23}$$

where K_{ser} is stiffness modulus, ρ_{mean} mean density of timber, d diameter of dowels in mm.

In the literature (Siem 2014, Malo 1999 and Reynolds et al. 2012), performed tests in order to research the relationships between number of dowels, steel plates and stiffness of the connection. They also tested the effect of initial slips and stiffness for reloading cycles, obtaining large variation of results. Plausible causes can be nonlinear contact stiffness between the wood and steel surfaces, unequal embedment stiffness distribution along the dowels and/or possible elastic bending of dowels. In general, initial slips in the connections are caused by drilling inaccuracy, misalignments and possible damage to the wood surface during installation.

For the purpose of this report, values obtained according to Eurocode 5 simplified formulas will be used.

The case scenario of “The Treet” building was investigated to test the effect of connection in the overall global behaviour. First the structure was modelled with all nodes tied (maximum connection stiffness), i.e. all members in a joint are forced to have the same rotations. Next, the nodes were allowed to have separate rotations (minimum connection stiffness), i.e. the nodes were modelled as hinges. Both scenarios resulted in insignificant differences in deflections and natural frequencies. Concluding that the structural response of glulam truss frame is very similar to pinned truss-work, and there is no need for evaluation of the rotational rigidity of the connections. The main load bearing glulam frame can be both modelled with pinned and tied joints in the structural model.

3.7 CONCLUSIONS

Highest built precedents

The Treet and UBC Brock Commons, both at 53 meters height, define the highest built precedent of tall timber structure, at the time of writing. Their height is situated slightly above the definition of high-rise according to CTBUH. Many of the claimed tall timber buildings, may not enter standard high-rise definitions. As soon in “The Treet”, and its characteristic structural system technical developments in bridge engineering can be an inspiring source for new ideas regarding lateral load-resisting systems of tall timber buildings

Technical and sustainable marketing

The main attractive point of timber high-rise constructions is the fact that there are not many precedents up to date. A common perception of timber, partly true, is that it is weaker than concrete and steel, and it burns in case of fire, which makes the material unsuitable for high-rise buildings. However, the aforementioned projects challenge that technical argument, and consequently become “attractive” from a technical point of view. In conclusion, a tall timber building is more attractive, and consequently more marketable, in the way that it is able to defy preconceived structural definitions. The fact that a common material is used in a very unconventional manner, with high technical demands, creates the main selling point of the building. The construction of tall-timber buildings between 20-50 meters has grown exponentially in the last years, showing a growing interest worldwide, from private to public sectors, to showcase the potentials of engineered timber for tall building structures.

High-quality prefabricated construction

Engineered timber is lightweight and often highly prefabricated, allowing the possibility of details with very low tolerances. This results in an improvement of the construction quality, quicker, and safer erection sequence, and minimum disturbances in the surroundings. Structural elements can be manufactured in large prefabricated components containing structure, façade and installations, improving overall quality and integration of trades within the building.

Fire considerations

Fire authorities are decisive for the construction of a tall timber building, often demanding bespoke testing and research for each specific situation. Often it is required the use of non-combustible encapsulation, e.g. gypsum or cement boards, smoke detector and automatic sprinklers. The special charring properties of mass timber, glulam and CLT, prove that the material may be suitable for safe high-rise construction, considering over-dimensioning of the elements for fire protection. This can be predicted with a charring rate of 0.7mm/min. However, further research in collaboration with authorities is needed for establishing quick design/approval methods for a tall timber structures, and recent studies suggest that strict fire regulations may become more flexible in the future.

Composite timber structures

One alternative is to use each structural materials where they best perform, creating a three-materials composite structure. For example, cast-in-place concrete is used for stability core and egress route, because of its continuous and non-combustible materiality. Steel is used for its absolute strength, coupling the timber elements in local connections. And timber is used in the majority of structural elements, floors, walls and columns, because of its lightweight, prefabrication/erection and warm feeling. Other alternative entail the use of steel and concrete for coupling lateral resisting CLT shear walls. In addition Timber-Concrete composite floors can also enhance the structural behaviour of the systems by improving the structural behaviour, and helping meeting fire and acoustic demands.

“Concrete jointed timber framework” research project developed by SOM, developed of a 42-storeys composite structure, surpassing current limit height for timber structures. SOM also performed a study for a same-height tower all-timber, concluding that although it is be structurally feasible, there are significant economical disadvantages compared to composite structures. They suggest that the composite system CJTF is cost-comparable with steel and reinforced concrete solutions saving up to 60-75% carbon emissions.

The “economic height” of a all-timber structure is lower than those for composite structural systems. (Baker et al. 2014, SOM 2013). At the same time, particular demands in a tall building means that the economy of the structure may govern the economy of the whole building. From this, we can conclude that the appropriate choice of structural systems and materials is critical to the viability of the building. The choice of composite structure (steel-timber, concrete-timber) takes full advantages of the properties of several structural materials creating a competitive structural design.

Concrete plinth

In almost every case, concrete is used in the ground level and 1st storey. This is usually done for three reasons:

- Add more flexibility for access, retail stores and connection to public space in street level
- Provide more durability against moisture and other hazards, such as car impact
- Contribute to likely uplift tension forces, by adding extra mass and reinforcement for tension forces.

Coupling CLT walls

Built precedents using only CLT panels for stability, (shear walls, or cellular base systems), are limited to 8-10 stories. Technical limitations such as cross-grain crushing of floor panels, and limited coupling between timber core and shear walls restrict the lateral load capacity of the overall structure. Consequently, in order surpass the height barrier using only CLT, structural coupling with steel and concrete may be needed.

Attention to connections

Special attention must be put in order to prevent perpendicular to grain or shear stresses at the connections between prefabricated discrete elements. In the above case studies different methods are used. A solution with screwed or nailed steel brackets was used in Murray Groove and Forté and buildings. Cenni di Cambiamento uses long self-tapping screws with plate-in-groove and dowel. In The Treet, dowelled steel-plate of similar sizes those used in bridge construction were chosen. In Dalston Lane discrete grout pockets were used in order to enhance the compressive capacity of timber. And finally Limnologen buildings uses CLT panels and vertical steel rods running the full height of the building. Even though, it is not the scope of this study, the author recognises the ductility and stiffness of the connections are crucial factors for the overall behaviour of a timber structure, particularly under seismic loading.

4. FIRE PROTECTION IN TALL TIMBER BUILDINGS (TT)

- Fire regulations in The Netherlands
 - Compartmentalisation
 - Automatic sprinklers
- Encapsulated tall timber (TT)
 - Complete encapsulation
 - Limited encapsulation
- Exposed tall timber (TT)
 - Effective charring depth
 - Delamination CLT

Most building codes limit the probability that construction materials will contribute to growth and spread of fire, which could lead to significant damage and unacceptable harm to persons. At the same time, regulations also indicate that the structural materials should provide enough fire resistance in order to allow a safe exit from the occupants before the collapse of the building.

Often, many national codes require “non-combustible” construction with specific fire-resistance ratings for varying heights and areas of the building. As timber is a “fire-combustible” material most buildings codes has traditionally prevented the use of timber in tall buildings . However, in the recent years, it has been proven that unlike traditional light frame construction, mass timber (glulam and CLT) can provide a high level of resistance against fire. During fire timber burns at a predictable rate, creating a char layer that isolate the material beneath the charring layer. Consequently, a sacrificial layer of timber can be extra-dimensioned, in order to protect the required dimensions of the structural element.

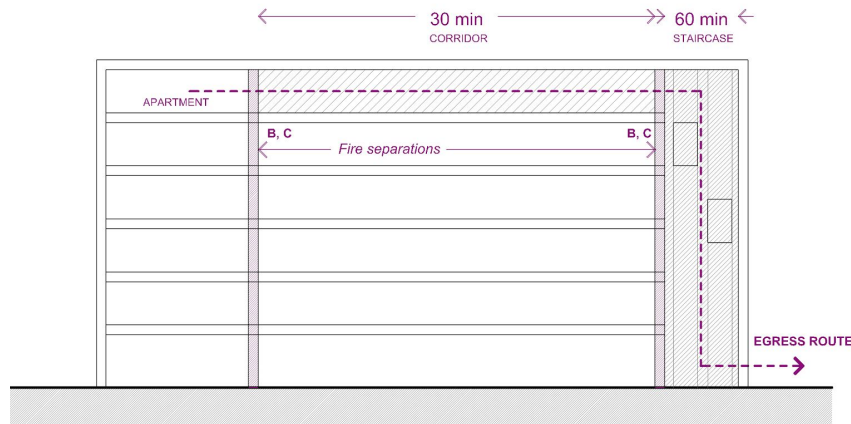
At the same time, key questions relative to fire ignition/development within timber structures needs to be answered, such as the potential contribution of timber elements to fire, impact of smoke spread in tall building, as well as flame spread characteristics of combustible materials, compared to non-combustible. Many researchers have demonstrated that mass timber structures can provide the required 2-hours fire resistances for a tall building, achieving a high level of safety.

4.1 FIRE REGULATIONS IN THE NETHERLANDS

According to dutch regulations the structure of a residential building higher than 13 meters should provide 120 m fire resistance before collapsing, giving time for escape of the occupants and enabling the fire brigade to investigate the building.

Another consideration, is the risk of progressive collapse. This means that when one local structural element fails, the overall structure should not collapse. In the structural analysis, this can be simulated by removing the specified element and performing the required calculation.

The load bearing structure should resist a given time, determined by the highest storey of the building. At the same time fire requirements in residential buildings, are stricter than in other types of buildings, due to the fact that there is both day and night use.



Escape route fire protection for residential buildings (Verdiepingbouw met staal, redrawn by the author)

In the national fire safety regulations (Bouwbesluit 2003), it is indicated that buildings of more than 3 storeys should provide a fire and smoke protected escape from the building to the outside.

- Flames and hot gasses should not penetrate the separated compartment
- The temperature on the unheated side is not allowed to surpass an average increment of 140°C
- The radiation on the unheated side ins not allow to surpass 1kW/m2 on one meter distance.

Horizontal circulation (corridors) should be smoke free (60 min)

Vertical circulations (Staircases) should be smoke and fire free (30 min)

Contrary to concrete and steel timber structures are combustible against fire, and therefore contribute to the fire load. If it can be demonstrated that the fire load is lower than 500MJ/m² (for example, by using sprinkler system or non-combustible materials), fire resistance requirements can be reduced 30 minutes. 500MJ/m² are equivalent to 25kg of spruce representing 60mm of CLT per square meter. In the Netherlands regulations, the effect of sprinkler system can lead to a reduction of 30-60 minutes fire resistance, depending on the fire authorities. European standards divides 7 classes of fire resistance for materials, according to the contribution of fire, and smoke and flames production.

Class	Flashover propagation of fire	Contribution to fire	Smoke production	Flame droplets
A1	Not possible	No contribution	No contribution	No flame droplets
A2		Very little	S1 (slight)	D0 (No flame droplets)
B		Limited	S1 (slight)	D0 (No flame droplets)
C	After 10 minutes	Average	S2 (average)	D1 (less than 10 secs)
D	Between 2-10 minutes	High	S3 (large)	D2 (more than 10 secs)
E	Less than 3 minutes	Very high	-	-
F	Not tested	Not determined	-	-

European standard fire classes

CLT wall panels are classified in European class D, with smoke production S2, and flame droplet class D0. CLT floor panels are considered D, and S1 classes. This means, that they cannot be used in fire separation compartments (e.g. escape routes) without additional protection. In order to improve superficial fire resistance of timber two methods can be applied:

- Up to class B, timber fire retardant treatments.
- Up to class A2, fire retardant materials, such as gypsum fibre board.

Compartmentalisation

In the case of non-load bearing assemblies, the fire resistance is based on its fire separating function, and its ability to maintain integrity (self supporting). By containing the spread of fire, occupants can have enough time to exit the building and fire-fighters can prevent further property loss without the risk of building collapse. This is special importance for egress routes, such as corridors or vertical shafts (elevator and services), that need to be protected accordingly. Typically, these areas of the building have to be protected with non-combustible materials, in addition to sprinkler systems.

In the case of CLT structures, they usually contain a high degree of sub-compartments which can be highly beneficial for fire compartmentalisation of the building. In conjunction with an automatic sprinklers "fast-response" the likelihood of a potential fire developing beyond the compartment or origin can be minimised.

Another fire safety measure is to limit fire and smoke spread within the designed fire compartment. This can be done by using fire separations. In order to limit that one-hour rated fire separation between fire compartments is required.

Automatic sprinklers

For the above mentioned reasons, it is recommended to install fast-response residential sprinklers in all fire compartments. This feature will mitigate the possibility of a fire developing in a small non-sprinklered spaced and spreading to other areas of the building. The exterior spaces such as balconies, patios or ground levels, will also be sprinkler protected, in order to mitigate potential fire spread and ignition through the exterior façade.

Researches have demonstrated the reliability of modern automatic sprinklers systems in preventing fires in the range between 96-99% (Richardson, 1985).

It must be also said, that the lower bound of reliability corresponds to older systems, and a few measures can be taken to reasonably improve. This can be done using a secondary water supply in the building, ensuring that if one part becomes impaired, the fire department can still have the ability to pump into the system and boost the pressure in the sprinkler system. It is also advisable to install redundant power sources, and piping systems in strategic locations, and use smoke detectors devices, initiating a fire alarm and an immediate fire department response

- Secondary water supply
- Redundant power sources and piping systems
- Automatic signals for fire department

The potential of water damage may be a concern in timber buildings, providing that sprinklers systems operate inadvertently, releasing significant quantities of water over the timber structure. However, in practice this is a very unlikely scenario. By comparison, sprinklered buildings are expected to use only 4-5% of the water that unprotected buildings require from the fire department operations, in order to extinguish the fire. This is because automatic sprinklers help in localising the fire origin, and reducing the smoke levels, enhancing visibility in the event of a fire.

4.2 ENCAPSULATED TALL TIMBER (TT)

Complete encapsulation

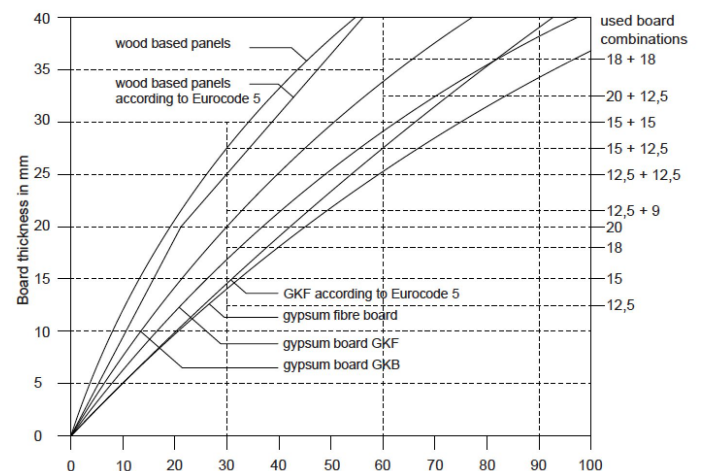
The fundamental approach is to protect all timber members with non-combustible material such as gypsum boards or cement boards. If this is the case, the fire performance of a tall timber structure would be comparable to a non-combustible structural materials like concrete. The recommended design approach is to consider all structural elements as fully encapsulated and work systematically through the building in order to decide which elements can be left exposed.

As an initial conservative approach the duration of delay for ignition should be taken as two-hours fire resistance for tall buildings.

Protecting walls, beams, columns and floor elements with multiple layers of Type X gypsum board has been demonstrated to provide "complete" protection. Therefore, a building with the timber elements fully encapsulated should not contribute to the fire intensity or spread of fire or smoke..

Laboratory research has demonstrated that 2 layers of 12.7mm X gypsum board can provide fire resistance for more than 2 hours (Taber, Loughheed, Su & Bénichou, 2013)

A consideration is that, protection must be applied directly to the mass timber elements, using fastening methods with sufficient depth to resist deterioration or pull-out during fire exposure. As with typical drywall construction techniques, gypsum board layers should incorporate overlapping joints to maintain a continuous thermal protection of the underlying timber surfaces. In addition, it is often required to design special designs with fire stops and a continuous fire protected layer, at certain building joints.



Fire resistant various lining materials (Staalframebouw handboek)

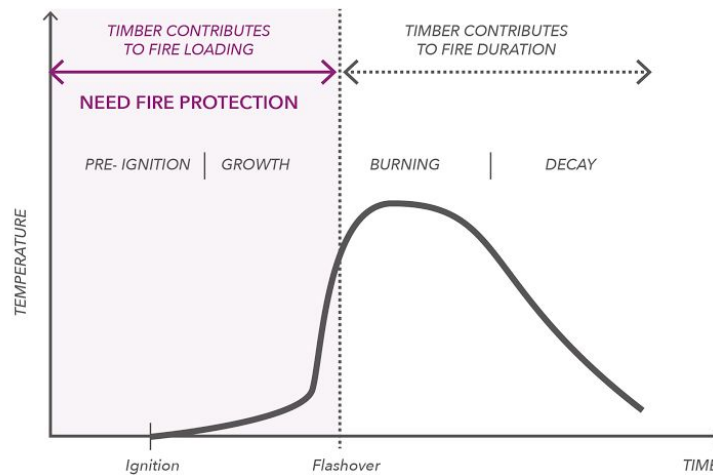
Limited encapsulation

The development of a fire can be divided in two fundamental stages, ignition and growth (pre-flashover) and fully-developed (post-flashover).

After flashover stage, the presence of timber element, or other combustible materials does not contribute to an increase in the fire loading. Providing encapsulation until the flashover stage, has been demonstrated to provide equivalent performance to a steel or concrete structure.

This is because after flashover, the burning rate is controlled by the available oxygen and contents along with the rest of combustible finishes in a conventional building

In this sense, by providing one 15.9mm layer of Type X gypsum board directly attached to timber, fire requirements can be met.

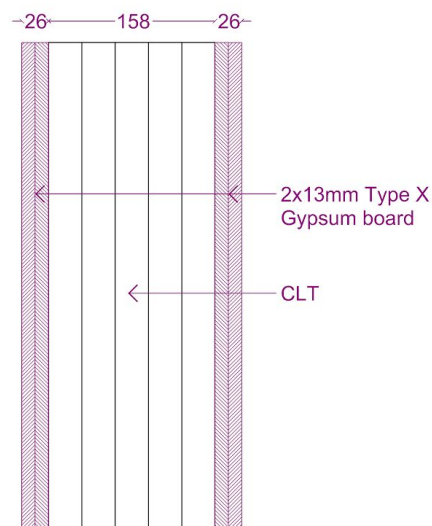


Typical stage of fire development and limited encapsulation approach (Own elaboration)

If this approach is used, it is necessary to address the extended duration of fire. This can be done by using enhanced protection systems, i.e. on-site or gravity fed water supply for automatic sprinklers, and improved provisions for fire-fighters.

Complete encapsulation **Conventional fire protection systems**

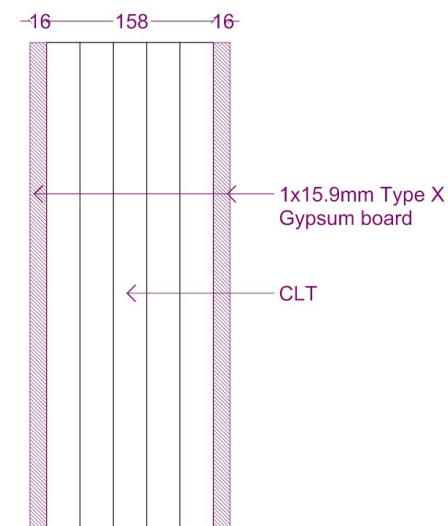
After flashover stage
120min



Limited encapsulation **Enhanced fire protection systems**

Gravity-fed water supply automatic sprinklers
Additional provisions fire-fighters

Before flashover stage
60min

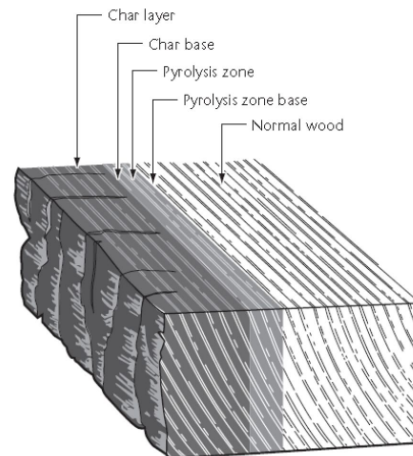


Encapsulation options for CLT panels after and before flashover stage (Own elaboration)

4.3 EXPOSED TALL TIMBER (TT)

Effective charring depth

When exposed to elevated temperatures, wood material undergo thermal degradation. This is a material-specific property, called pyrolysis, which begins at approximately 200°C and slowly convert the wood into char. This charring rate depend on numerous factors such as timber type, density, tree species, adhesives, moisture content, structural behaviour and the characteristics of the fire.



Char layer, and pyrolysis zone formed by wood (CSA 0177, 2011)

The charred layer formed around the exposed surface acts as a thermal protection to the inner core, therefore reducing the rate of burning. Buchanan A.H, 2002, researched that the effective thermal protection provided by the char layer requires a minimum char depth of approximately 25mm.

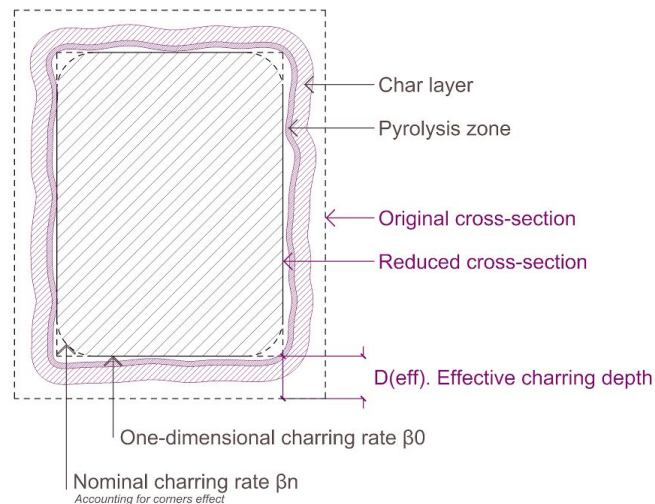
For simplicity, most structural design codes specify a constant charring rate throughout the fire exposure, depending on the wood density. The one-dimensional charring rate, β_0 for thick slabs, and nominal charring rate β_n , accounting for the loss of cross section at the corners and fissures. The rounding at the corner is equal to the depth of charred layer.

Characteristic density	One-dimensional charring rate, β_0	Nominal charring rate, β_n
Softwood glulam >290 kg/m ³	0.65mm/min	0.7mm/min
Softwood solid timber >290 kg/m ³	0.65mm/min	0.8mm/min
Hardwood >290 Kg/m ³	0.65mm/min	0.7mm/min
Hardwood >450 kg/m ³	0.50mm/min	0.55mm/min
LVL >480 kg/m ³	0.65mm/min	0.7mm/min

Charring rate values as given by the Eurocode 5

In addition, the charring layer can be seen as more reliable since it is inherent with the material properties of timber, and do not have be maintained. (Schaffer 1984) also concludes that in cases where there is no automatic fire suppression systems, mass timber elements will perform as well under fire conditions as protected steel, or even better if steel coating is damaged.

It can be seen that for any given fire exposure duration (t), the reduced cross section can be calculated based on the charring rate. Charred wood is assumed to provide no strength or rigidity, therefore the remain (reduced) cross-section must be capable of carrying the required structural requirements.



Reduction section for fire protection in timber elements (Own elaboration)

The rates are measured according to “worst-case” scenarios. In most cases, if automatic sprinkler systems are installed, they will operate to control temperatures and fire development within the location of the fire, minimising the impact on the timber structure. Nonetheless, in case of the sprinklers malfunctioning problem, the structural elements must be dimensioned with enough fire-resistance.

Because of their massive solid constitution, mass timber surface is also expected to have more resistance to surface ignition compared to other wood interior finishes, such as typical veneers in furniture elements. It is also to be noted, that when mass timber elements are expected to be left exposed, surface burning characteristics can be augmented with the use of “fire-retardant treatments” or other chemical applications.

Delamination in CLT

Full-scale fire-resistance tests on CLT assemblies conducted by FPInnovations and National Research Council Canada, have demonstrated close to three hours fire-resistance in unprotected CLT floor elements under loading conditions.

Engineered timber is usually manufactured with adhesives. When exposed to fire, the glued product has to perform similarly to a solid timber element. The behaviour of the adhesive in case of fire is dependant on the type of adhesive, and the moisture conditions, and must be consulted with the manufacturer

In CLT panels delamination of layers may occur, leading to adhesive failure. This was observed in full-scale fire tests in CLT manufactured with structural polyurethane (PUR), (Osborne, Dagenais & Bénichou, 2012).

In the master thesis, Roy Crielaard, made several experiments in order to test the reliance on the structural adhesive, contribution to fire and charring rate for CLT panels. He concluded that delamination can be prevented with thicker outer lamellas, providing additional charring depth before fire action can reach the adhesive layer.

The modified char depth model for CLT products made with adhesives that may suffer delamination, is a step-wise approach that resets the time in the charring rate equation to zero whenever the calculated char depth reaches the glue line of adjacent laminates. It can be seen from conducted tests in Canada, that the charring rate in CLT is influenced by the thickness of the layers (thinner layers heat up quickly and reach adhesive line faster, causing delamination).

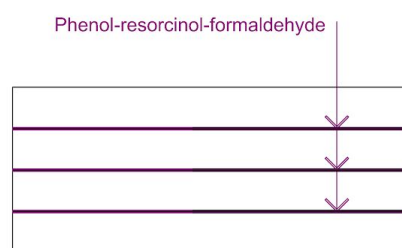
The one-dimensional rate β_0 , is influenced by the thickness of the laminates, d . According to Dagenais, Osborne and Benichou, 2013, the charring rate for CLT can be determined according to the following expression:

$$\beta_0(\text{mm/min}) = \begin{cases} 0.65 & \text{when } d \geq 35 \text{ mm} \\ 0.65^{(d/35)} & \text{when } d < 35 \text{ mm} \end{cases}$$

Thickness of laminations	Charring rate (taking into account delamination)
35mm	0.65mm/min
30mm	0.69mm/min
25mm	0.73mm/min
20mm	0.78mm/min

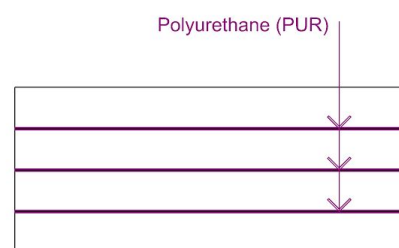
Linear charring rate of CLT, taking into account delamination effects (CLT Handbook, Canada)

Extensive testing made in Canada made with phenol-resorcinol-formaldehyde adhesive has shown that charring does not result in delamination when this adhesive is used.



NO DELAMINATION

0.65mm/min



DELAMINATION

35mm - 0.65mm/min
 30mm - 0.69mm/min
 25mm - 0.73mm/min
 20mm - 0.78mm/min

Consult manufacturer for chemical composition and specific fire behaviour of adhesive

Charring rates for CLT panels depending on adhesive effect (Own elaboration)

5. SOUND PROOFING IN TALL TIMBER BUILDINGS (TT)

- Basic acoustic notions
- Netherlands requirements
- CLT acoustic assemblies

Sound proofing is one of the most important issues when building with timber structures. Sound is defined as energy in form of vibrations. It can be longitudinal sound waves that create slight pressures on the building construction. For single-leaf constructions, it can be deducted from theoretical Rayleigh's mass law that to obtain sufficient sound insulation of a construction sufficient weight per unit area kg/m^2 is necessary. For instance, a 250mm thick CLT (C24) panel results in 105kg/m^2 , compared to 600kg/m^2 with the same thickness of concrete. This does not mean that achieving good acoustic insulation with lightweight materials is not possible, nevertheless it is a complex topic that requires advance acoustic knowledge, and craftsmanship.

The main parameters for sound insulation in buildings are the following:

- *Total weight per unit area.* Higher weight, better sound insulation (especially in low frequency sound)
- *Sound absorbing material* in the cavity between layers
- *Decoupling building components.* Less contact between components, better sound insulation.
- *Airspace in cavity wall.* Larger air space, better sound insulation
- *Floor surface hardness.* Harder surface poorer impact sound insulation

Four main strategies can be used in Tall Timber Extensions are:

- Breaking structure-borne sound transmission, by separating floor systems from different uses.
- Provide relative high mass or topping
- Provide soft materials for floor covering or between assemblies for sound dampening

5.1 BASIC SOUND NOTIONS

Sound can be imagined as a periodically moving wave, that consecutively compress and stretches air particles between the wall and sound source. Sound insulation is defined as the amount of energy that is blocked by a certain construction. When dealing with sound insulation, we can distinguish between airborne sound insulation and structure-borne sound insulation or impact sound.

Airborne and impact sound insulation

Airborne sound insulation related to the insulation against sound wave in air, generated by an acoustic source
Structure-borne insulation deals with the insulation against vibrational waves inside materials directly cause by a mechanical source.

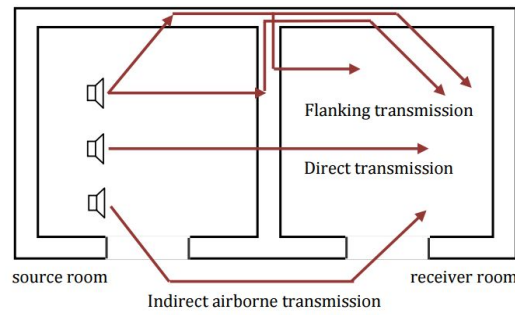
Direct, flanking and indirect sound transmission

Sound can travel from one room to another via three different transmission paths.

Direct sound transmission involves the sound transmission directly through the partition wall between two rooms

Flanking transmission, is the transmission of sound waves via the junctions towards the other room

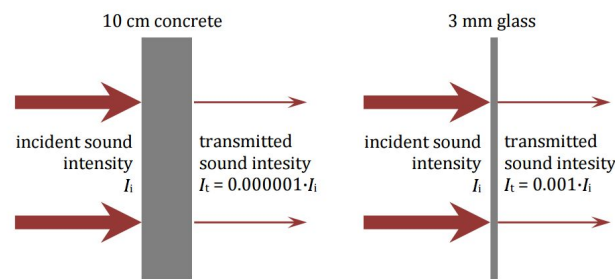
Indirect airborne sound transmission is the transmission of sound via air cavities suspended ceilings and hallways.



Direct, flanking and indirect transmission paths (Tenpierik)

Surface mass parameter

Mass is an important parameter that influences airborne sound insulation. This is explained because sound is a longitudinal wave impacting a certain wall with a certain frequency, back and forth, and moving the air on the other side. Newton's second law, $F=ma$ describes that the change in motion of a mass is proportional to the mass. This means that the higher the mass is the smaller the acceleration and thus the vibration of the partition. Doubling the mass of a partition wall results in an increase in sound insulation of 6dB.



Surface mass and sound insulation (Tenpierik)

Lab and In-Situ measurements

It is very difficult to measure sound intensities or sound levels. Therefore, a method has been developed for a measuring normalised airborne sound insulation of building elements in the laboratory. In this procedure, the test laboratory has high requirements for avoiding all kinds of flanking transmission paths. R , is the airborne sound insulation of a construction, and does not depend on contextual factors, if S (tested surface) or A_r (area of absorption in receiver room change, $L_{p,r}$, sound level in receiving room should also be affected)

$$R = L_{p;s} - L_{p;r} + 10 \log \left(\frac{S}{A_r} \right)$$

In practice, contrary to laboratory measurements, there are more transmission paths than just through the partition walls, flanking sound transmission, indirect airborne transmission and transmission through leaks and seams. All these transmission paths affect the final result of the measurement. The normalised sound level difference between source and receiver room, D_{nT} , can be deducted with the following formula, where T_r is the average and T_0 is a reference reverberation time (s) for the receiver room (0.5s dwellings) :

$$D_{nT} = L_{p;s} - L_{p;r} + 10 \log \left(\frac{T_r}{T_0} \right)$$

Contrary to airborne sound insulation in a laboratory setting, D_{nT} , depends on contextual factors. In the past, the laboratory based equation has been used to measure sound insulation between two rooms in practice. This value was denoted as R' (practice based', to make a distinction between R (laboratory based).

This value can be recalculated into D_{nT} with the following expression:

In conclusion, if there is only direct sound transmission through a partition wall, then R' equals R , and then a high sound level difference between source and receiver room, D_{nT} can be obtained with:

- High sound insulation of partition (airborne, flanking and indirect airborne transmission)

- High total absorption in the receiver room

Flanking sound transmission

Flanking sound transmission is the transmission through walls. This acoustic property is difficult to evaluate, and advance laboratory measurements are usually required. For a rule of thumb practical method a decrease in 6-10dB in the sound insulation properties of the constructions must be considered.

There are basically two types of flanking transmissions:

- Sound leaking through the openings
- Vibration transfer between the coupled surfaces or continuous elements

The basic principle to control flanking transmission is to seal openings, decouple surfaces and discontinue structural elements. However, this is sometimes not possible for structural obvious reasons. The manual elaborated by FPIInnovations in Canada, establishes a quick checklist for appropriate detailing in order to prevent excessive flanking sound transmission.

- Attach gypsum board on resilient framing channels
- Leave gap between floor and walls and fill it with perimeter isolation board or acoustical caulking
- Discontinue floor as much as possible, and if it is not possible use floating topping
- Seal partitions edges with gaskets or caulking

5.2 NETHERLANDS REQUIREMENTS

Sound insulation requirements in the Dutch code relate to the sound insulation between two adjacent dwellings, not the sound insulation of walls or floor as such. This means that flanking transmission has to be taken into account.

- Global sound insulation between two dwellings (Not only sound insulation of partitions)
- Flanking sound transmission (depends on the detailing)
- Sound insulation of the construction (depends on cross-section and layers of construction)

Apart from sound insulation, requirements are formulated for a number of other possible noise sources, as for example, the reverberation in staircases halls, sanitary installations, slamming doors.

Airborne (R')	>55dB
Impact (L')	>54-61dB

Sound insulation requirements between dwellings in The Netherlands

5.3 CLT ASSEMBLIES

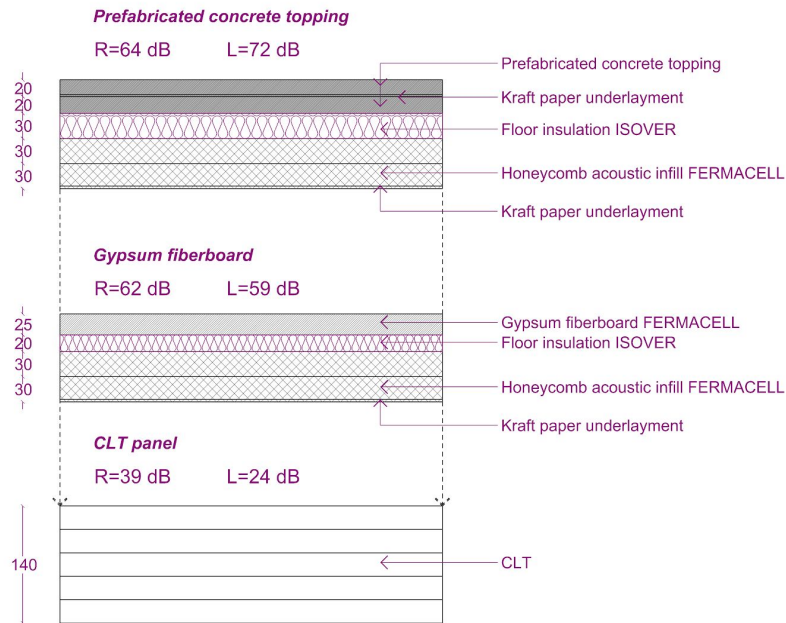
CLT handbook in Canada (Gagnon and Kouyoumji, 2011) provides CLT wall and floor assemblies with associated sound transmission ratings.

CLT	Thickness	Airborne sound insulation (R)	Impact sound insulation (L)
3 layers	95-115mm	32-24 dB	-
5 layers	135mm	39 dB	23 dB
5 layers	146mm	39 dB	24 dB

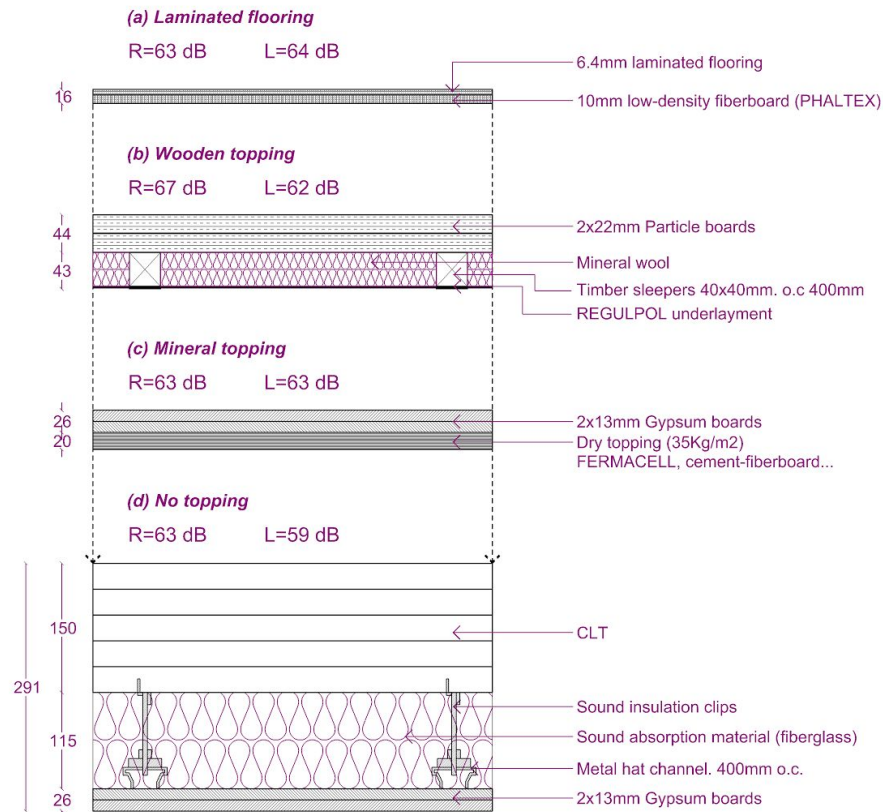
Examples of bare CLT assemblies and sound insulation, assuming density 500kg/m³ (CLT handbook)

The above table indicates that due to the low mass of timber and specifically CLT panels, wall and floor constructions require additional materials in order to meet strict sound insulation requirements in the Netherlands. The images below show a different assemblies with CLT panels that meet Netherlands acoustic requirements, R, represents airborne-sound insulation, and L impact sound insulation measured in laboratory.

FLOOR ASSEMBLIES WITH FLOOR COVERING



FLOOR ASSEMBLIES WITH CEILING

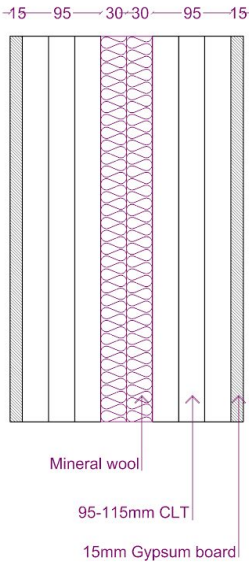


Section details and sound insulation for floor assemblies with CLT panels, ceiling and flooring (Own elaboration)

WALL ASSEMBLIES

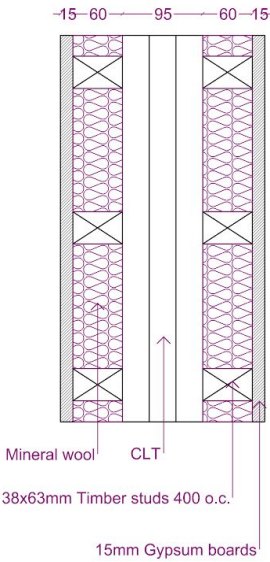
Double CLT panel

30mm Mineral wool
R=55 dB
60mm Mineral wool
R=60 dB



CLT panel + studs

120mm Mineral wool
R=58 dB



Section details and sound insulation for wall assemblies with CLT panels (Own elaboration)

6. EXTENSIONS ON EXISTING BUILDINGS (E)

- Case studies
- Structural typologies
- Guidelines

Nowadays, many people (especially young) want to live in city centres. Some of the reasons may be increased urban activity, (such as retail stores, restaurants), better public transport connection (closer location to train central stations) or better amenities (parks, libraries). This trend is especially noticeable in Dutch city centres according to Raven 2016. In addition, there are not many empty spaces in the city centres for creating a new construction.

Confronted with that situation developers have three options, demolition, refurbishment or extension of an existing building. (Wilkinson, Remoy & Langston, 2014).



Demolition and construction of a new building is the most common construction practice, as it is the most effective regarding economical and technical grounds (Pearce, 2014). At the same time, a higher or bigger building can replace the existing construction increasing urban densification with the same amount of building space. Nevertheless, as it is mentioned several authors (Remoy & Voord, 2014; Ellison, Sayce & Smith, 2007; Bullen & Love, 2010) this option created noise disturbances, waste material and pollution and it is harmful for the environment. In addition, it destroys existing building, that may become architectural heritage of the city and urban fabric in the future.

Refurbishment is the most sustainable of of construction as it re-uses existing structures (Ellison, Sayce & Smith, 2007; Yung & Chang, 2012), and has the capacity to preserve old buildings for the future. However, this solution is not always financially feasible (Wilkinson, Remoy & Langston, 2014), and does not provide a solution for a growing needs of extra space in city centres.

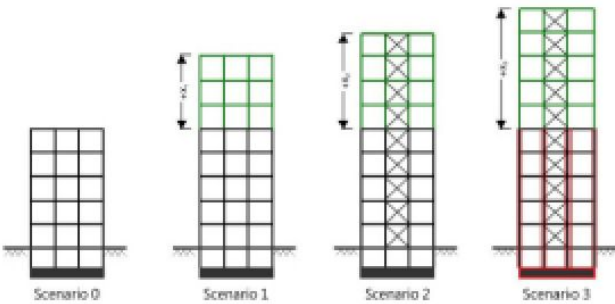
For that reason, extensions on existing building can be an alternative solution method, as they combine the benefits of demolition and refurbishment, as it is mentioned by multiple authors (Ellison, Sayce & Smith, 2007; Ball, 1999; Brand, 1994; Douglas, 2002; Kohler & Hassler, 2002). Extensions can provide social, sustainable and economical benefits, and a solution for urban densification (Yung & Chang, 2012). They can reduce environmental impact of conventional construction methods, as they avoid demolition of existing structures (Ellison, Sayce & Smith, 2007).

Moreover, in the case of refurbishment combined with extension, the extra value provided by the new added programme can compensate the energy upgrading financial costs, making the system very interesting from a financial perspective (Cukovic-Ignjatovic & Ignjatovic, 2006). Finally, they have the potential to preserve

existing architecture and urban fabric of the today's cities, which is especially important for european cities with low ratios of new constructions (Penttilä, Rajala & Freese, 2007; Mill, Alt & Liias, 2013).

On the other hand, there are multiple technical complexities related to the existing building structure that may difficult the structural design of this type of construction. In many cases, the existing structure has been deteriorated due to the time passage, or may not be strong enough to accommodate extra loading. It is also common not to have enough technical data of the existing building, that can assure a smooth design process.

In 2016, Maria Papageorgiou, research the optimal vertical extension height from a technical and economical standpoint. Considering a case study, she concluded that below 4 storeys, there is no need increase the structural provisions of the existing building, e.g. additional stability core or strengthening methods. According to LCA assessments and costs considerations, the optimal vertical extension is 4 extra storeys.



Four different scenarios for structural extension considered by Maria Papageorgiou, 2016

Uwimana in 2011, proposed a design study for 9 extra storeys added on top of an existing 6 storey building . His main structural approach was to connect the two structures (new and existing) so that the stability of the building can be enhanced by their composite behaviour.

Because of the lack of information regarding structural extension typologies, and what implications on existing buildings several case studies have been analysed in order to comprehend the potential and limiting factors for this type of construction system.

6.1 CASE STUDIES

The cube

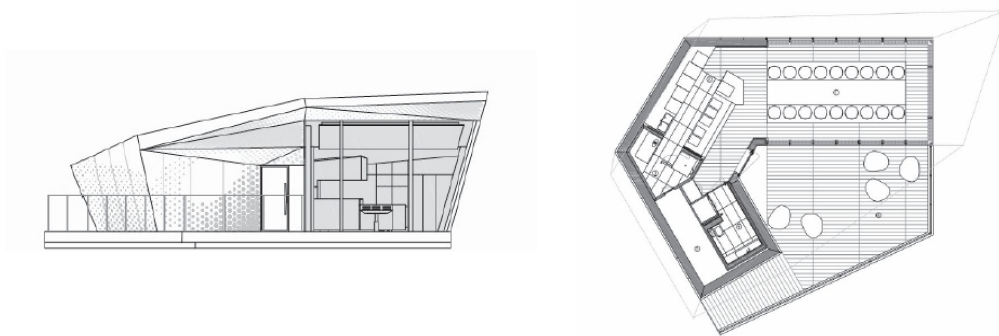


Images of the parasite extension in London (left) and Milan (right)
(Source: Park associati)

Architect	Park associati	Stockholm	2012
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Client	Electrolux home appliances Enea N.V.	London	2012
		Milan	2011
GFA	140m ²	Brussels	2011

The Cube is a pavilion designed to house an itinerant restaurant commissioned by Electrolux. Conceived to be placed in unexpected and dramatic European locations, the restaurant will be active simultaneously in twin structures and will stay in each location for a period of between four and twelve weeks. The interior of the building has been designed to suit different arrangements; the pavilion consists of a large open-planned space with a visible kitchen and a single large table that can be made to disappear by raising it up to the ceiling to form a lounge area for use after eating. The total floor area of 140 sqm is divided between the open-planned space and a 50 sqm terrace. The building has been conceived as a module-stand that can be put up and taken down relatively easily,



Cross section (left) and floor plan (right) of The Cube parasite extension (Source: Park associati)

Las Palmas Parasite

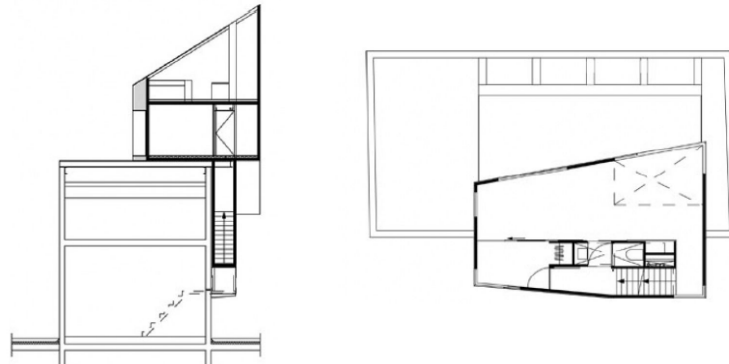


Photography of extension under construction (left) and completed building (right)
(Source: Korteknie Stuhlmacher)

Architect	Korteknie Stuhlmacher	Location	Rotterdam
Client	Parasite foundation	Completed	2001-2006

The Las Palmas Parasite was a prototypical house aiming at combining the advantages of prefabricated technology and the unique qualities of tailor-made design. The limitations imposed by the size of the elevator shaft and the strength of its walls demanded a compact plan and volume. Despite its temporary character, the building remained in its location until the summer of 2005 and was used for numerous activities. Due to the planned renovation of Las Palmas the Parasite had to be removed and is currently in storage. Walls, floors and roof of the new extension are made with CLT timber panels made from European softwood. The elements were prefabricated, pre-cut to size and delivered on site as a complete building package. The assemblage on site took

just a few days - despite the difficult and exceptionally windy location. Services like water supply, sewage and the electric installation had been linked to the existing installations. The interior surfaces had been left untreated and uncovered, the exterior had been clad with painted plywood in large sheets. Openings were cut out as simple holes.



Cross section (left) and floor plan (right) of Las Palmas parasite (Source: Korteknie Stuhlmacher)

Studio East Dining

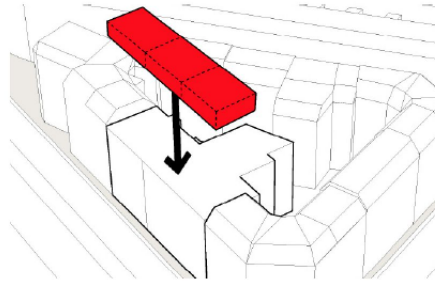


Aerial view (left) and schematic section (right) of the building extension. (Source: Carmody Groarke)

Architect	Carmody Groarke	Completed	2010
Client	Bistrotheque/Westfield	GFA	490m2
Location	London	Programme	Commercial

Studio East Dining was built on top of a 35m high multi-storey car park within the live construction site of Westfield Stratford City development in East London. The temporary pavilion provides elevated views across London's 2012 Olympic site. A cluster of interlocking timber-lined spaces formed the translucent dining room, which framed key rooftop views and allowed up to one hundred and forty guests to enjoy an intimate dining experience. The project was designed in collaboration with Restaurateurs Bistrotheque and built within ten weeks from initial briefing to opening night. With a lifespan of only three weeks, the 800m2 lightweight structure was constructed with hired materials borrowed from the existing construction site including scaffolding boards, scaffolding poles and an industrial grade heat retractable polyethylene roof membrane. All building materials selected for their ability to be 100% recycled following closure of the restaurant.

Birkegade rooftop



Exterior view (right) and concept scheme (left) for building extension
(Source: JDS architects)

Architect	JDS	Location	Copenhagen
Client	Andelsforeningen	Completed	2010
Engineer	EKJ	GFA	490m2
Contractor	Logik & Co	Programme	Residential

The project is located in one of the most densely populated areas of Copenhagen. The architects designed three penthouses on top of a residential building, and use the rooftop of the newly built penthouses for communal spaces of the building, garden, and playground. The top storey and roof was demolished in order to accommodate the new extension.

The hedonistic rooftop is reflected in a playground with shock-absorbing surface and a playful suspension bridge, a green hill with varying accommodation backed by real grass and durant vegetation, a viewing platform, an outdoor kitchen and barbecue, and a more quiet wooden deck.

Hanover house



Construction of CLT folded plate (left) and completed extension on top of the existing Victorian building (right)
(Source: Kraus & Schönberg)

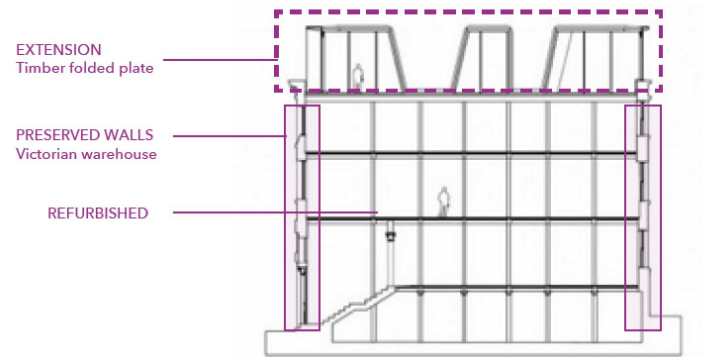
Location	Bradford, United Kingdom	Contractor	Eastwood construction
Architect	Kraus & Schönberg	Completed	2008
Client	Garbe group	GFA	1300m2

Engineer	EDA	Programme	Residential
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Hanover House was a Victorian warehouse graded II for historical preservation. The architectural project involves the refurbishment, and roof extension, converting the abandoned existing building program into a residential block. Together with the surrounding building it is a reminiscence of the early industrial age in Bradford. The sculptural aspect of the roof silhouette reflects the roofscape of the surrounding buildings and has been used to form the living areas.

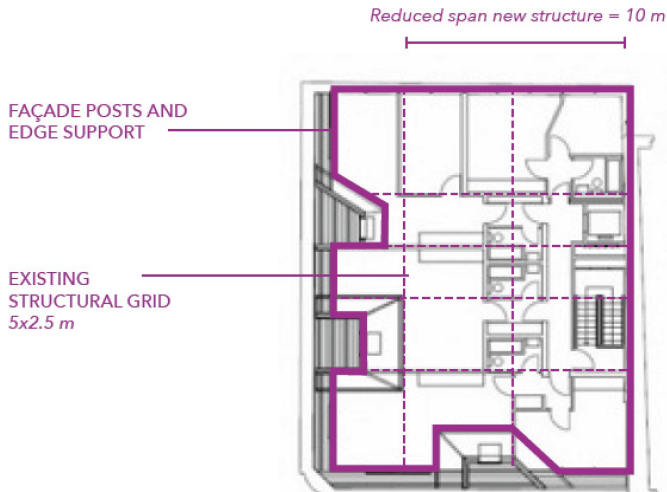
Structural system

Type of building extension	Construction on top of existing structure Existing building was heavily renovated. 1 extra storeys
Gravity loading system	Existing concrete cores and limestone walls
Lateral loading system	Concrete frame with rigid connections and adjoining building
New structure	Folded CLT load bearing roof/walls



Section of existing victorian house and structural extension. (Source: Own elaboration based on Kraus & Schönberg documentation)

Aiming to create a highly efficient structure, the roof has been designed as a self-supporting system to avoid the use of columns, and consequently reducing the loading onto the existing structure. The structure of Cross-Laminated Timber panels, conforms a roof that folds up and down, and bears at the edge of the building. At the same timber, the use of engineering wood, implies a highly efficient and sustainable structure.



Floor plan of the structural extension. (Source: Own elaboration based on Kraus & Schönberg documentation)

Halle 12 - Rheinauhafen



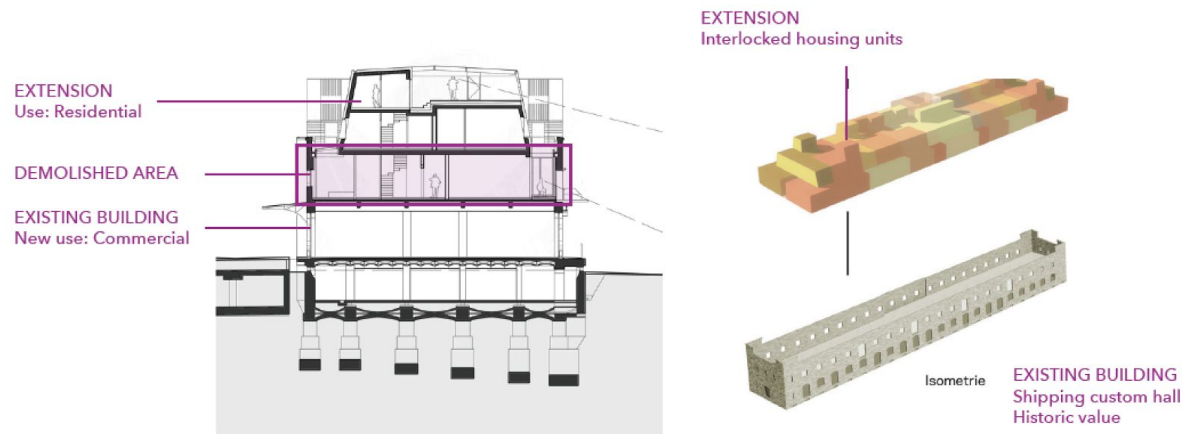
Photographs during construction (left) and completed building (right) (Source: Molestina)

Location	Cologne, Germany	Contractor	Ars Factum GmbH
Architect	Molestina	Completed	2009
Client	Portus Pristinus GmbH	GFA	6100m2

The original building was built in 1920 as a custom hall for the shipping port of Cologne. Molestina architects designed a proposal for extension + renovation, by reinterpreting the old building windows pattern in the new design of the extension. The new building interconnects 18 apartment units from 80m2 to 120m2, stacking them in multiple configurations in order to optimise the distribution of daylight, and at the same time create inner exterior spaces between the apartments. The modern permeable façade creates a sharp contrast with the materiality of the existing building. The ground floor is refurbished and the program converted to commercial.

Structural system

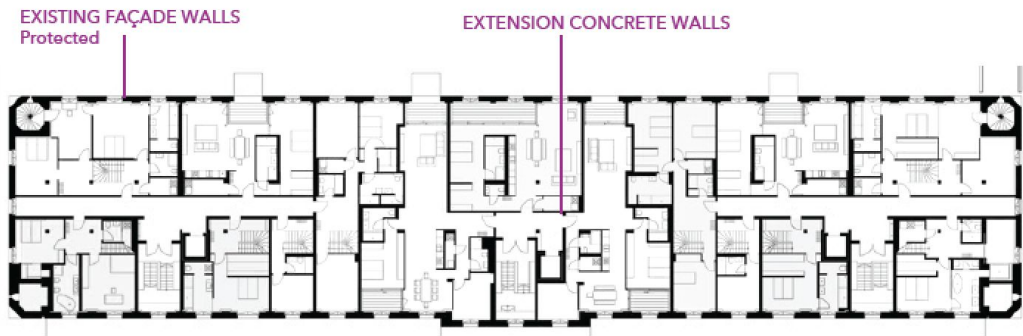
Type of building extension	Construction on top of existing building Demolition of top storey. 3 extra storeys
Gravity loading system	Existing concrete structure
Lateral loading system	Concrete frame with rigid connections
New structure	Prefabricated concrete walls



Cross section of the building (left) and scheme (right)

(Source: Own elaboration based on Molestina documentation)

The existing building sandstone façade walls are preserved because of their historical value. For the connection with the extension, top floor columns and ceiling of the original building are removed. The structural extension is anchored to the original building, with both lateral stability and vertical loading supported by the existing concrete structure. In the new building the walls of the housing units are made with prefabricated concrete for an easy erection and floor and sub-structure are made out of wood. Finally, at the top light weight steel structures, conform the modern gold-lined steel sheets of the façade and covering for exterior spaces.



Floor layout. (Source: Own elaboration based on Molestina documentation)

Fahle house



Historic photograph of existing building and construction on top (Source: KOKO architects)

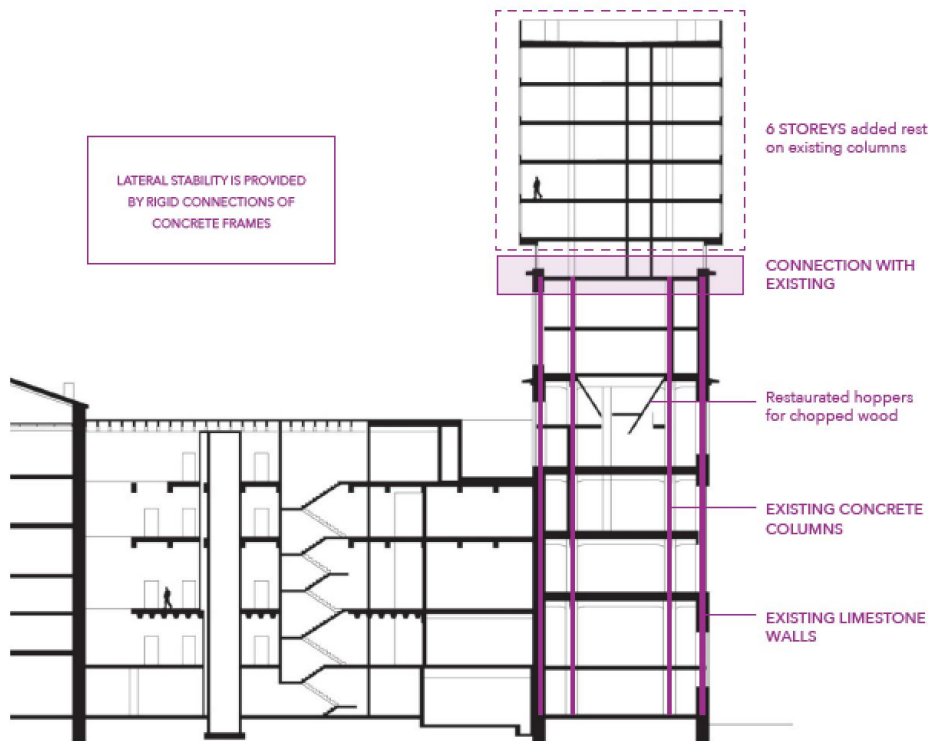
Architect	KOKO architects	Completed	2004-2007
Client	Koger & Partners	GFA	35000m2

Fahle house by KOKO architects, is one of the characteristic examples of architecture during the recent economic boom in Estonia. The building is part of the complex of a former cellulose and paper factory where the most outstanding building is the tall and voluminous boiler house (1926) built from limestone and designed by architect Erich Jacoby. The reconstructed Fahle house complex also carries several service and business functions, from beauty salons to a restaurant.

The architects tried to preserve and display the historic interior details and the wall and floor surfaces where possible. For example, the hoppers, which were partially preserved, are part of the fourth floor offices and apartments. These hoppers were funnels made from reinforced concrete, which were used to direct wood mass into the gigantic boilers. The boilers were destroyed but their locations are marked using circles on the ceilings and floors.

Structural system

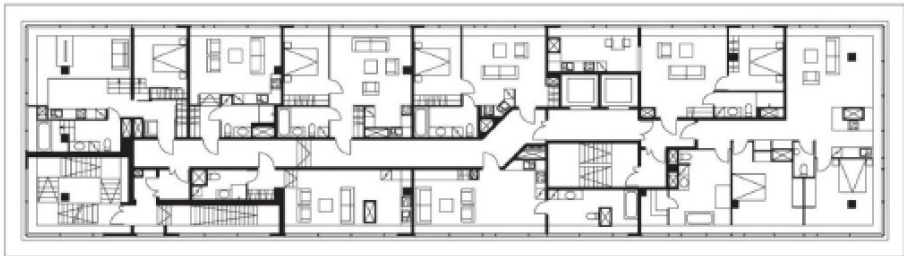
Type of building extension	Construction on top of existing building, 6 extra storeys
Gravity loading system	Existing concrete cores and limestone walls
Lateral loading system	Concrete frame with rigid connections and adjoining building

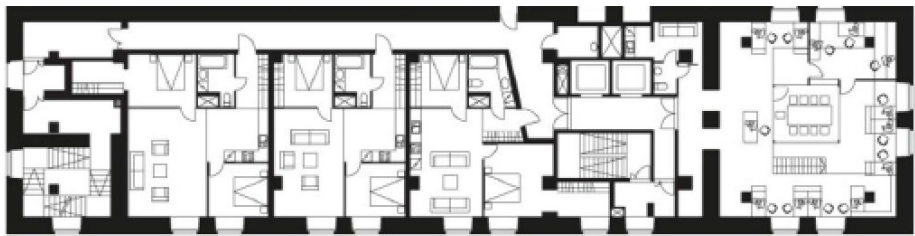


Cross section of the building. (Source: Own elaboration based on documentation from KOKO architects)

Six extra storeys with apartments were added on top of the old boiler house, on the limestone walls. By the time reconstruction started, the interior of the plant had been destroyed and this made it possible to reorganize the internal layout and room division. Lightweight construction systems were used in the new construction, the extra part of the building is supported by reinforced concrete beams, which have been hidden between the walls of the boiler house and reach down into the subsoil.

Offices and service spaces are mainly located in the historical rooms of the plant. Different sized apartments are located inside the new section with a glass facade, on the roof of the former boiler house.





Comparison floor layouts and structure between new extension (above) and existing building (below). (Source: KOKO architects)

Rottermann carpenter’s workshop



Perspective of completed building (left) and aerial view during construction of concrete cores (right) (Source: KOKO architects)

Architect	KOKO architects	Completed	2006-2009
Client	Rottermann City OÜ	GFA	2700m2

The reconstruction of the historic carpenter’s workshop is one of the boldest architectural undertakings in the modernising Rottermann quarter. The building faces the central square of the quarter and is one of its most spectacular sights – the three techno-futurist towers make reference to 20th century industrial architecture and are also visible from outside the quarter.

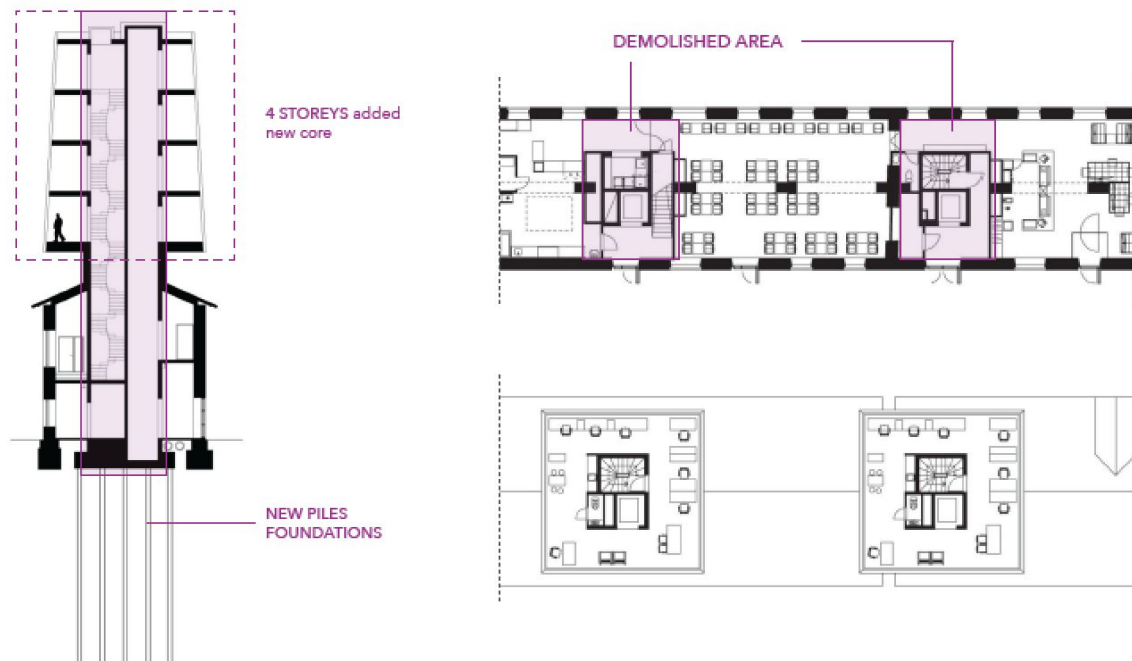
The two lower floors of the old carpenter’s workshop, the limestone volume of the building, house commercial and service facilities. The three new vertical volumes placed on the central axis of the building accommodate compact office spaces. The reconstruction of the Rottermann carpenter’s workshop – a limestone building under national heritage protection – is a great example of architecture during the economic boom, when private clients had the desire and courage to commission more extravagant architecture. On the other hand, the reconstructed carpenter’s workshop also reflects the desire of the heritage protection institutions to reconsider their approach to historic and modern architecture.

Structural system

Type of building extension	Insertion inside existing building, 4 extra storeys
Gravity loading system	Supported by inserted concrete core and new foundations
Lateral loading system	Supported by inserted concrete core and new foundations

The 'towers' were designed in order to avoid harming the historic walls of the building. The reinforced concrete cores placed on localised piles ensure the autonomy of the three volumes. The console ceilings, angular facade elements and windows are all attached to the core and lit up during the night, conveying a sense of modern and self consciously vigorous architecture.

The vertical loading and lateral stability are supported by the newly added reinforced concrete core, that has been inserted within the existing building perimeter. For that purpose, also additional pile foundations were created. In this sense, it can be discussed whether this type of construction is in reality an extension or a new full construction.



Cross-section of the building (left) and ground floor, existing building and 4th floor, new building (right) (Source: Own elaboration based on documentation from KOKO architects)

Groot Willemsplein



Photograph of the building extension during construction of the cores (left) and completed building (right) (Source: DAM & Partners)

Location	Rotterdam	Engineer	Pieters Bouwtechniek
Architect	DAM & Partners	Contractor	Slavenburg
Client	LSI project investment N.V.	Completed	2013

The structural information for this building was extracted from the Master Thesis of Papageorgiou. In that work, she specifies that the structural engineers Pieters Bouwtechniek had at their disposal reports of vertical and stability calculations, details, and foundations plans from the existing structure. The reinforced concrete structure (Strength class B15) was designed for a vertical load of 15Kn/m². This is the main reason why adding three storeys was structurally feasible.

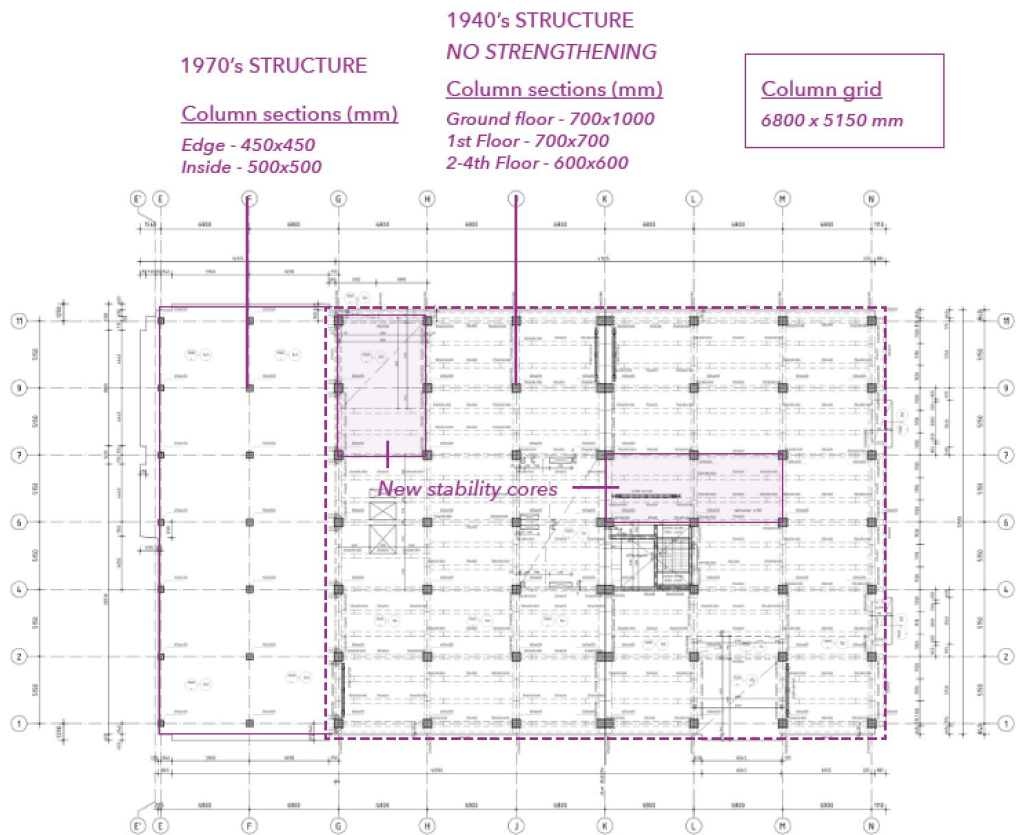
Structural system

Type of building extension	Additional stability cores inside existing building Intensive demolition required. 3 extra storeys
Gravity loading system	Supported by existing structure
Lateral loading system	Supported by additional concrete cores and new foundations
New structure	Steel beams and columns + hollow core slab
New foundations	Injection piles and heavy cap piles

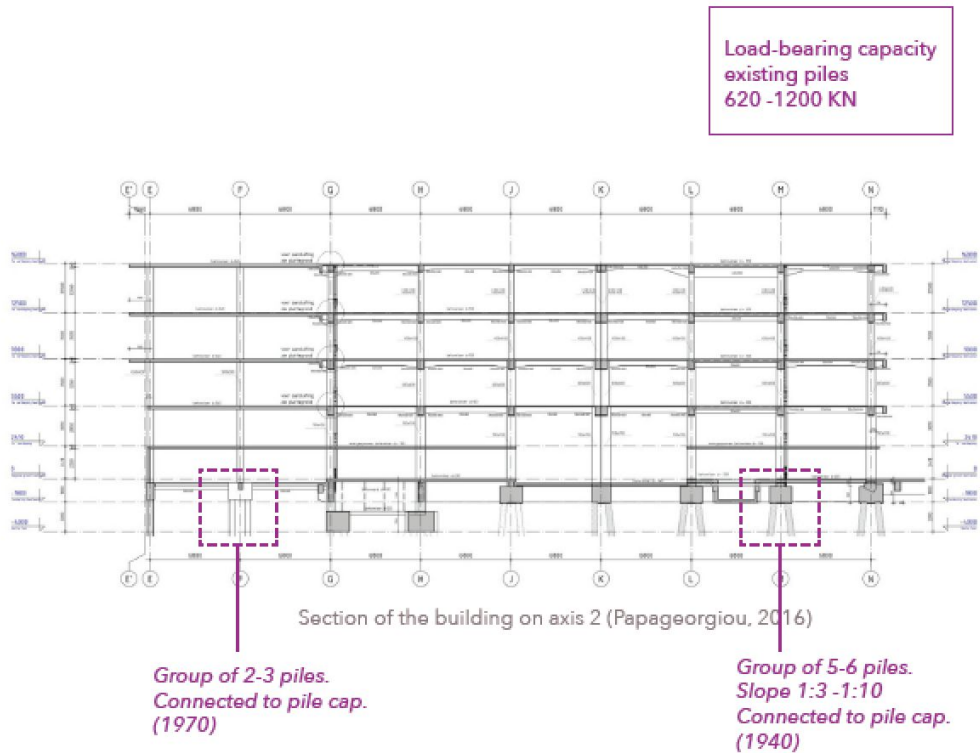
The moment-resisting rigid connections between beams and columns provide the lateral stability. Two new concrete cores provide the stability for both, existing structure and extension. The former core was previously disconnected from original beams and floors. After that, the two new concrete cores were connected to the floors, this way, the previous structural scheme for resisting lateral loading was transformed from concrete rigid frames to shear core. The vertical loading of the new extension is carried by the existing structure.

In order to proceed with the vertical extension, the building was stripped down almost to its structural skeleton. Elements and later additions such like, separation walls, technical rooms and a later added floor of the office building were totally removed. The only element left untouched was the prefabricated concrete façade, a typical sample of the façades constructed in the end of the 1970's. A large part of the first floor has also been removed in order to create an atrium in the middle of the office building.

In order to provide structural stability and, at the same time, different access to the extension, two new concrete cores were built. Parts of beams and floors connected to the old core were demolished, and later connected to the new stability cores. This way, the previous structural scheme for resisting lateral loading was transformed from concrete rigid frames to core system. The foundations under the new cores are injection piles and heavy cap pile. The new structural extension is composed out of lightweight steel beams and columns and hollow core slabs of 6,8m length.



Structural drawing of second floor (Own elaboration based on documentation from Papageorgiou, 2016)



Section of the building on axis 2 (Own elaboration based on documentation from Papageorgiou, 2016)

Giesshübel



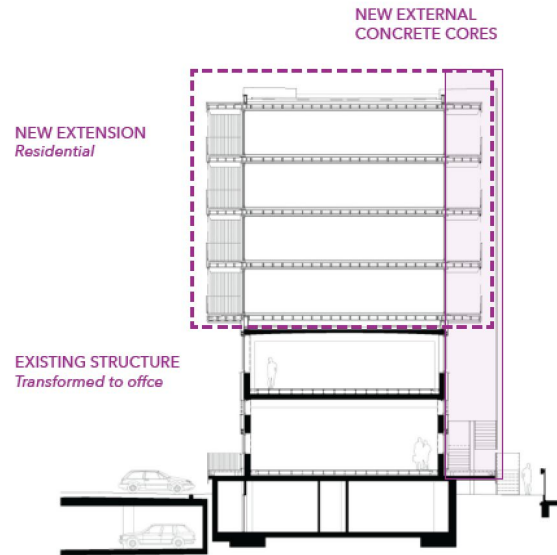
Construction of the concrete stability cores and CLT walls (left) completed building (right). (Source: Burkhalter Sumi)

Location	Zürich, Switzerland	Timber construction	Hector Egger
Architect	Burkhalter Sumi	Contractor	Unirenova
Client	SZU AG	Completed	2013
Engineer	Lüchinger and Meyer	GFA	6200m2

The old two storeys industrial building is transformed into a hybrid of office and residential program. The existing building is converted to host SZU Headquarters, and the extension provides the program for 24 new apartments, occupying little extra space in an urban context. The architects decided to use cross-laminated timber because of the lightweight and prefabricated benefits, and at the same time, the construction details were simpler, and the costs were reduced. The building was erected in 5 weeks. The floor depth is 27.5cm, and it is supported on both long edges of the building. The acoustic isolation is provided inside the timber panel, and on both sides, with a suspended ceiling and floating floors. It was also the intention of the architects to cover timber elements except the floor with gypsum boards, to emphasize the mineral character of the context.

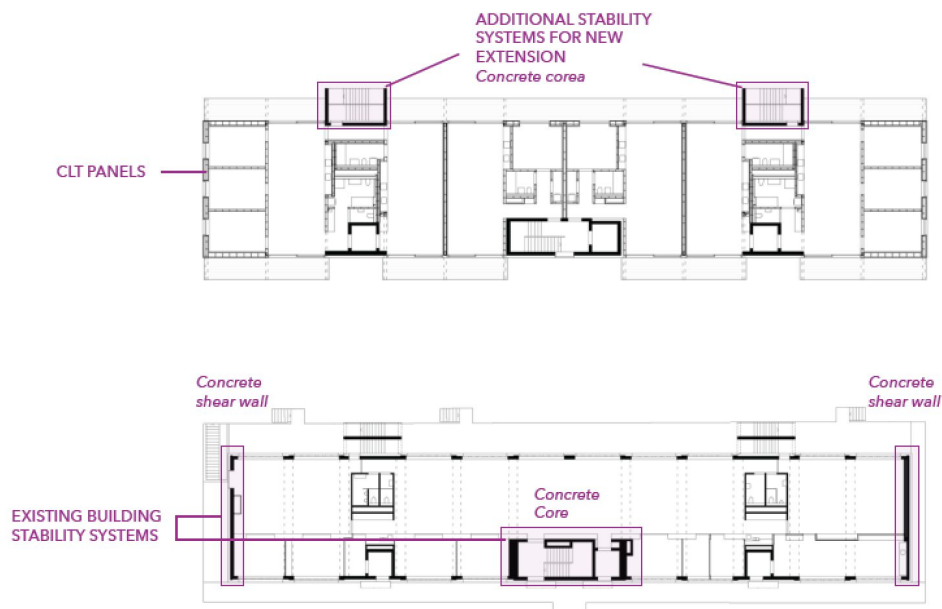
Structural system

Type of building extension	Construction on top of existing structure + additional stability cores 4 extra storeys
Gravity loading system	Existing concrete frames
Lateral loading system	New concrete stability cores
New structure	Concrete stability cores + CLT walls



Section of existing building and structural extension. (Source: Own elaboration based on Kraus & Schönberg documentation)

4 new storeys are piled-up on top of the existing structure that carries the vertical loading to the ground. The lateral stability for the extension is provided by two additional concrete cores that are placed next to the original building.



Typical floor of structural extension (above) and ground floor of existing building (below). (Source: Own elaboration based on Kraus & Schönberg documentation)

Westerlaantoren

Construction of the extension and reinforcement existing structure (left) and completed building (right)

Location	Rotterdam	Engineer	Aronsohn ingenieurs
Architect	Ector Hoogstad	Contractor	Dura Vermeer
Client	Maarsen groep	Completed	2012

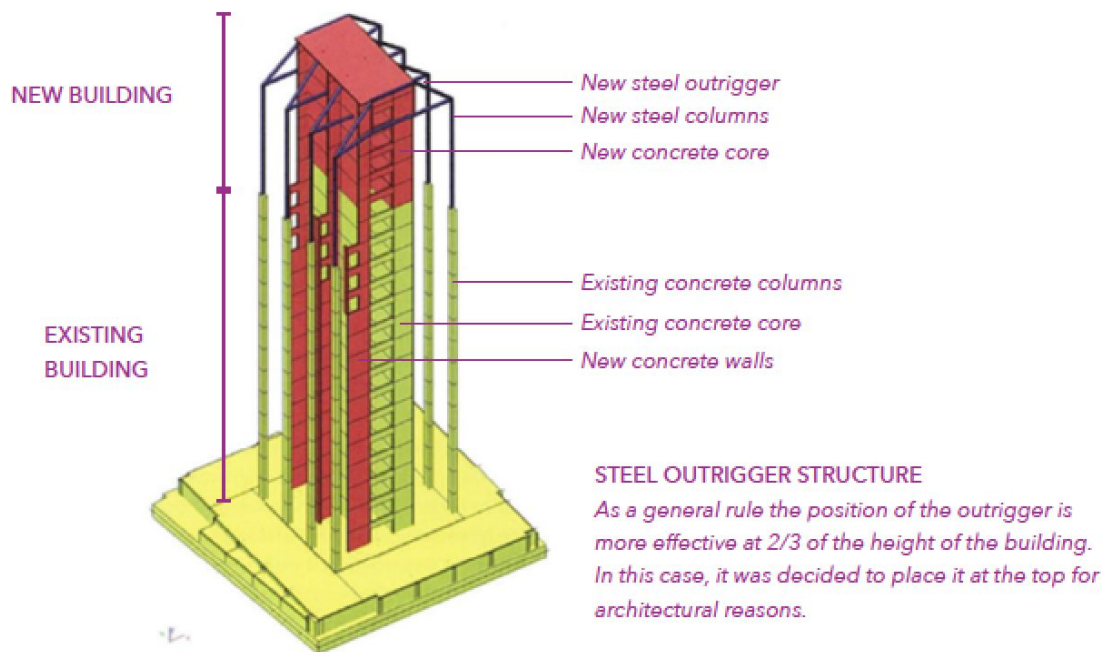
The structural information for this building was extracted from the Master Thesis of Papageorgiou. The program of the building is composed of a commercial level on the ground floor, offices from the 1st -10th and residential from 11th-19th floors. The roof is used for installations and maintenance of the façade. In the basement, there is a parking garage, storages for residences and HVAC systems for the offices. The structural design was done by Aronsohn Constructies Raadgevende Ingenieurs, consequently, reports and structural drawings were directly available. The existing structure was made with in situ casted concrete floors, beams, columns and cores. The existing tower has an almost square floor plan of 32,5 x 32,5 m², with extra dimensions for cantilevers 1-4th floors. The structural extension increases the height of the building from 61 to 76 metres, demolishing two storeys from the original building and adding extra 5 storeys. The original building is stripped to its structural skeleton.

A foundation engineering consultancy, Tjaden Ground mechanics, recalculated the allowable load bearing capacity of the piles in line with the current standards. The research showed that the foundation piles had a lot of spare capacity. The original load bearing capacity of the piles was calculated about 1000 kN, with the recalculation and new CPTs, Cone Penetration Tests for the soils, the allowable load bearing capacity of one pile could increase up to 2000 kN. This is mostly due to the more conservative old standards of calculations. However, centre to centre distance was 1450mm, approx 2,7D. As compaction of the soil, or making more CPTs tests were not economically feasible, finally, 1450KN (per pile) were used for the new design.

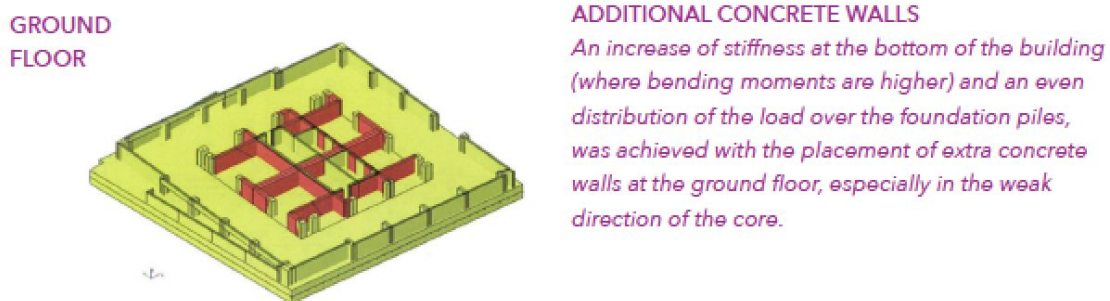
Structural system

Type of building extension	Construction on top of existing structure + additional steel outrigger and reinforcement existing concrete core Demolition 2 top storeys and addition 5 extra storeys
Gravity loading system	Existing in-situ concrete frames
Lateral loading system	New steel outrigger + existing concrete core and perimetral columns

New structure	Steel structure and extended concrete core
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Structural scheme of existing and new structural approach (Source: Aronsohn engineers)



Structural for central core reinforcement at ground floor (Source: Aronsohn engineers)

The existing building structure had the following structural limitations that unable great increases in the load-bearing capacity:

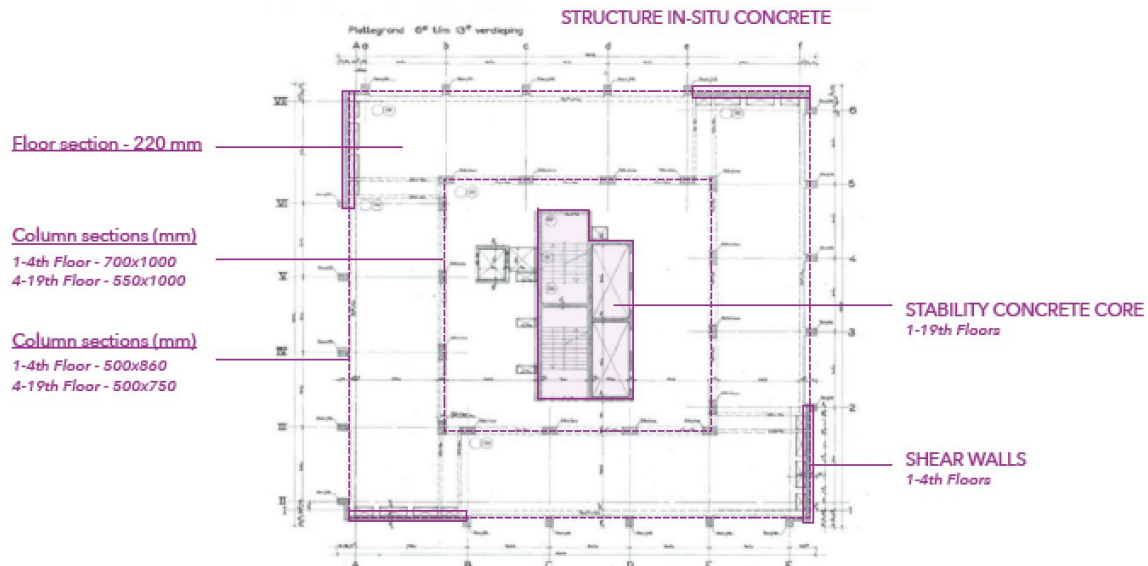
- The existing concrete core had a weaker direction against lateral loading.
- Thick foundation slab (2000 mm), unable the option of placing extra piles.

After recalculating the load-capacity of the foundation piles, and analysing the current state of the super structure, new structural design modifications were considered:

- Increase core inertia in the weak direction. A new honeycomb structure (red) occupying the inner ring of columns.
- Extend shear walls to the total height of the building.
- Provide an outrigger structure connecting outer columns with the core, and consequently increasing overall stiffness of concrete core.

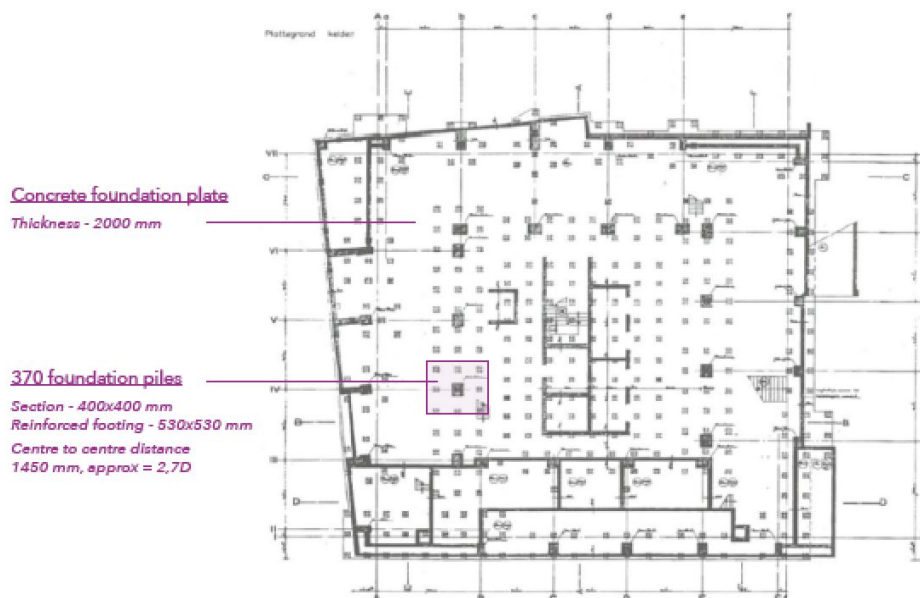
The final solution was driven by architectural requirements that lead to the structural combination of the first and third options. A provision of steel outrigger structure was place at the top, in order to make the inner columns participate from the lateral resisting system (only in the weak direction of the existing concrete core). This

structural mechanism creates a counteracting moment at the top of the building, which results in a reduction of the bending moment at the bottom of the building. At the ground floor, a honeycomb structure of 4500 mm height was created with the additional concrete wall. This has two structural effects, a further increase of the stiffness of the concrete core, and an even distribution of vertical loading over the foundation piles.



Typical structural floor plan (Own elaboration based on documentation from Aronsohn engineers)

Load-bearing capacity of the piles was increased from 1000kN (old standards) to 2000kN (new standards + new CPTs). However, centre to centre distance was 1450mm, approx 2,7D. As compaction of the soil, or making more CPTs tests were not economically feasible, finally, 1450kN (per pile) were used for the new design.



Typical foundation plan (Own elaboration based on documentation from Aronsohn engineers)

The increase in height of the tower translates to an addition of 14% in vertical loading and 60% in the bending moment at the foundation, considering ultimate limit state (ULS). The original moment (61 meters building) was 97000kNm, whereas the new moment (76 meters building) is 15700kNm, according to current standards.

Zeemanshuis



Façade of the building (left) and aerial view of the building (right). (Source: Google maps)

Location	Rotterdam	Engineer	Zonneveld
Architect	Bias Architecten	Contractor	Hijbeek B.V. Zwijndrecht
Client	Foundation Zeemanshuis	Completed	2004, 2007

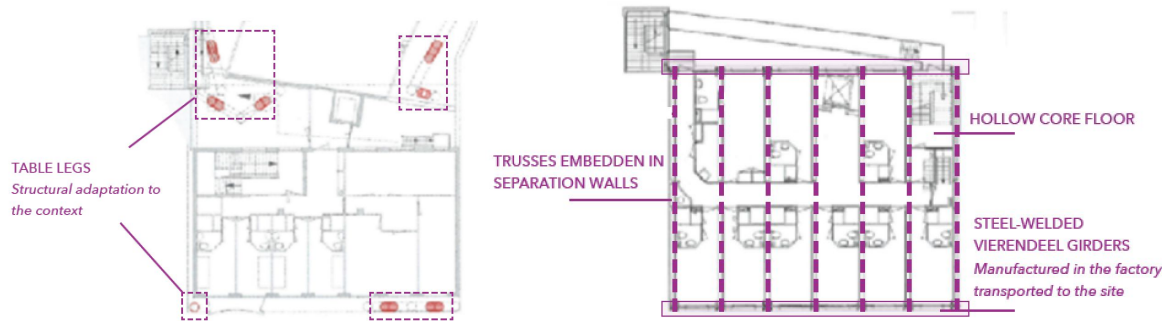
The structural information for this building was extracted from the Master Thesis of Papageorgiou. The Zeemanshuis was built, in two stages. The building from 1950s, faces Willemskade street, and the building from 1960 around Westerstraat. It is a typical example of a buildings constructed after the Second World War in Rotterdam. The former function was a low-budget accommodation stay for seafarers that wanted to stay a few days in the city. It was decided to vertically extend the part of the building on the Willemskade side with three extra floors that would accommodate thirty hotel rooms. Three years after the completion of the first extension, the clients decided to proceed with one more vertical extension of three floors, at the Westerstraat side.

Structural system

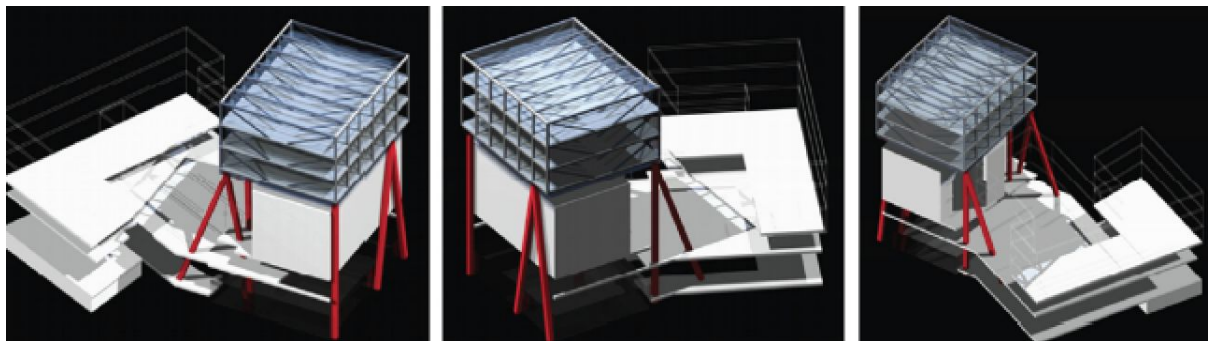
Type of building extension	Hovering above supported by external table structure Addition 3 extra storeys
Gravity loading system	Storey-height trusses and steel vierendeel girders in façade
Lateral loading system	8 oblique columns that support lateral and vertical loading
New structure	Steel structure, with 8 table legs, and storey-height trusses that span between them

There was an absence of structural drawings or calculations and consequently the load-bearing capacity of the foundations was difficult to determine. At the same time, the existing building was declared monument by the municipality of Rotterdam, with special value to the existence of the artwork "Zeeman aan stuurrad met Madonna" in the façade.

Because of the mentioned constraints, the engineers and designers decide to completely separate the structure of the new block, a table structure hovering above the existing building and with independent foundations. Fortunately, the building was slightly recessed from the plot line allowing the creation of "structural legs" in front of the preserved façade. The structural design was inspired by a drilling platform which is standing on high legs. For the columns of these structures, circular hollow sections (CHS) were chosen, filled with concrete for fire safety reasons. To ensure the stability of the structure, seven out of the eight columns are set oblique in order for them to assure both stability and vertical loading.

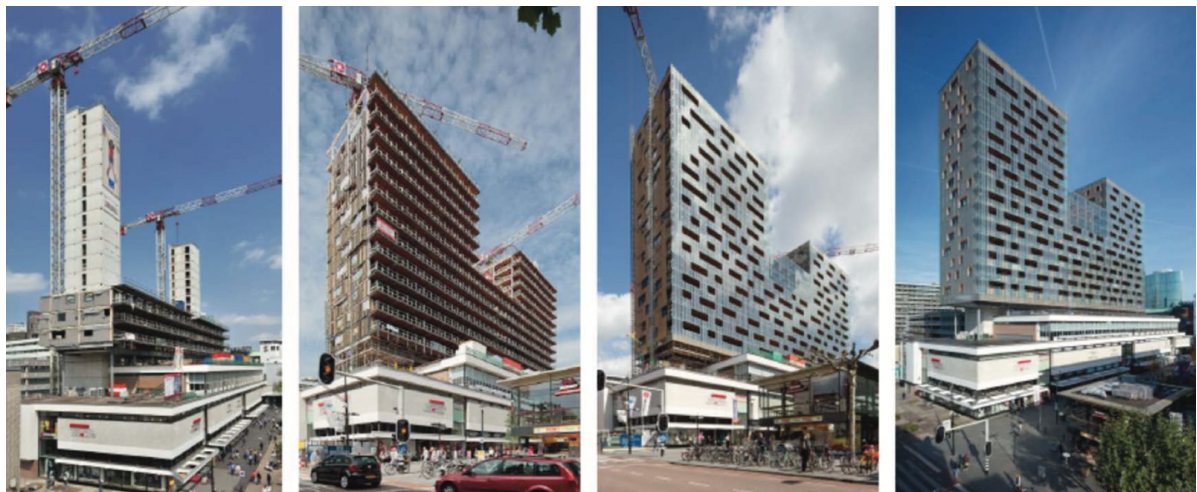


Ground floor (left) and typical floor plan of the extensions (right) (Own elaboration based on documentation from Papageorgiou, 2016)



Schematic 3D of table structure and extension above (Source: Papageorgiou, 2016)

Karel Doorman



Sequence of construction of the new structural extension with two concrete stability cores. From left to right, June 2010, June 2011, June 2012 and October 2012. (Source: Ibelings van Tilburg Architecten)

The technical information was extracted from the conference article "Ultra-Light Weight Solutions for Sustainable Urban Densification" (Hermens, Visscher & Kraus, 2014) and the Master Thesis, Optimal Vertical Extension (Maria Papageorgiou, 2016).

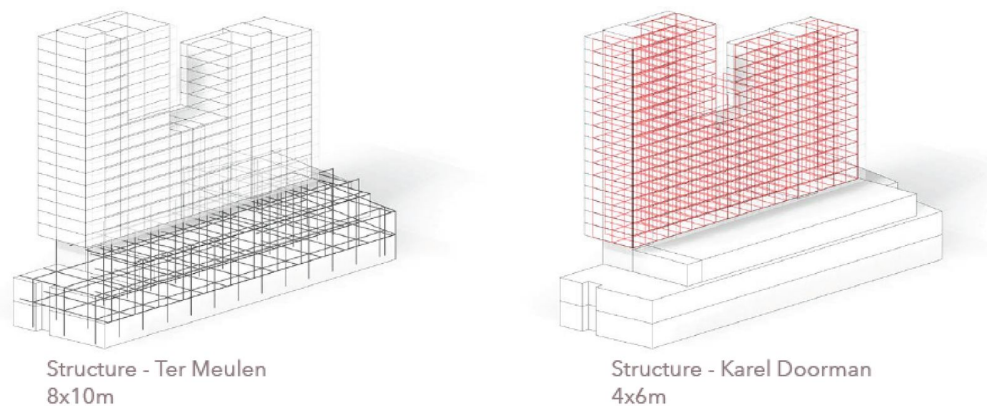
The existing building was well documented: gravity load calculations and stability calculations, concrete dimension and reinforcement calculations and drawings of reinforcement were available. Also the pile plan, the geotechnical survey and advice and a report on the installation and testing of a test pile were available, together with a calendaring drawing of the installation of the piles.

Structural system

Type of building extension	Additional stability cores. Rigid frames are liberated from lateral loading, and can carry extra vertical loading.
Gravity loading system	Rigid concrete frames
Lateral loading system	Two additional stability cores

The structural system of Ter Meulen building was completely cast-in-situ concrete. 8x10 meters grid of rigid concrete frames provide the lateral stability of the building through rigid frame action. Consequently the columns have similar dimension along the height, as they have to resist bending moments due to wind. (850 diameter mm in basement, ground floor, and 800mm, second floor). Typical dimensions of beams are 600x850mm. The existing structure was checked and designed according to the national codes for buildings of the time. The designed compressive strength of the concrete structure was 200-250 kgf/ cm² (compared to a C14/17).

Modification of existing structure

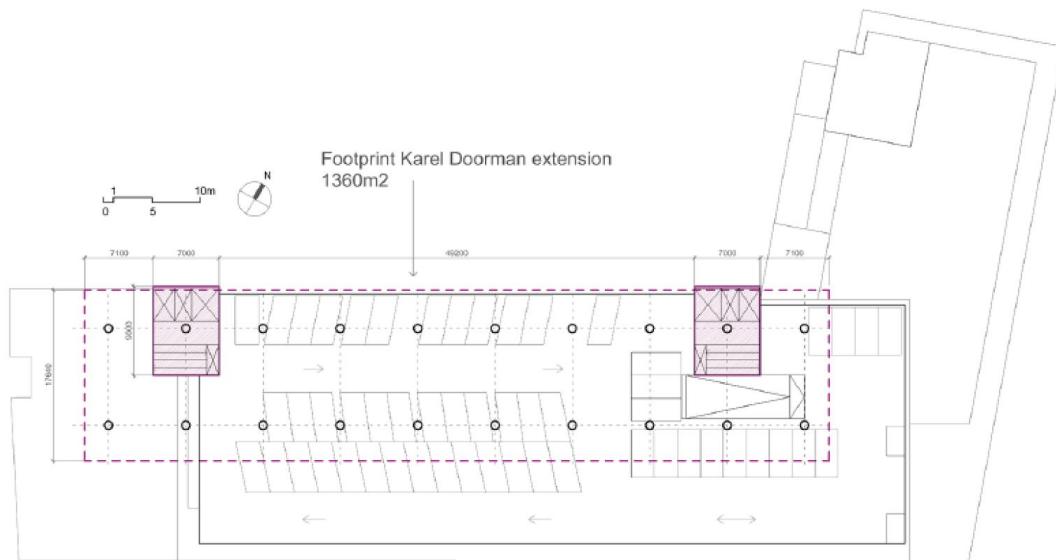


Schemes of existing and new structure (Own elaboration)

Type	Concrete rigid frames
Structural grid	8x10 meters
Columns	850mm (basement, ground floor), 800mm (second floor)
Beams	600x850mm
Extra capacity (Doubled)	20N/mm ² (1951) - 40N/mm ² (2001)

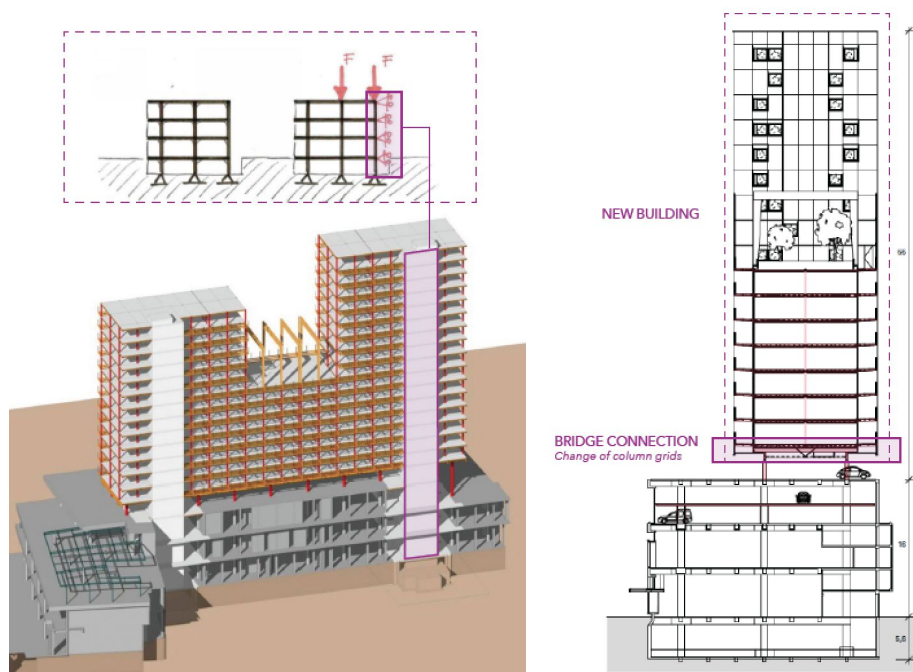
In 2012, a new extension of 16 storeys was completed on top of Ter Meulen building, making it the highest precedent up to date. The name of the new structural extension is Karel Doorman. The most important structural consideration was separating lateral and gravity loading systems, for the new expansion and existing building. With this purpose, two additional concrete stability cores of 7x 9 meters and wall thickness 400mm were added next to the original building. The new stability cores were rigidly connected to the existing building rigid frames, converting the them from a system with rigid frame action, with bending moments in the beams and columns, to a system where columns, only carry vertical loads. With this modifications, the load bearing capacity of the columns increased from about 5.000 kN to about 10.000 kN.

By liberating the existing structure from lateral loading, an extra 5000KN of axial loading can be gained in the columns.



New structural extension (Karel Doorman) on top of existing building (Ter Meulen) (Own elaboration)

The column grid for the structural extension is 4 x 6 meters. At the connection between new and existing structures, a transfer system with steel beams is used in order to accommodate the different structural grids 4x6m (new), 8x10m (existing).



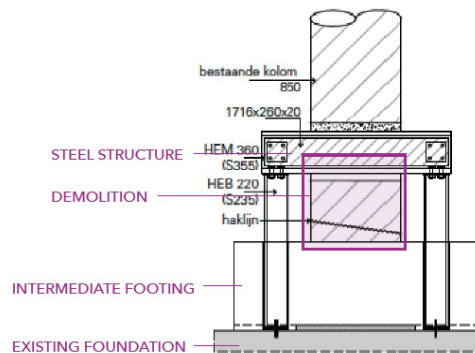
Structural scheme with additional stability cores (left) and bridge connection for transferring different structural grids (right) (Source: Ibelings van Tilburg Architecten)

Foundations

The existing foundations were prefabricated concrete piles of 380x380mm dimensions, and 760mm tip. Cone penetration tests of the soil proved two time more capacity. At the same time, when construction started in 2001, piles were inserted into the ground, and more soil resistance proved to be higher than calculated with the theoretical approach. Consequently, two times extra capacity of the existing piles foundations from 700KN (1951) to 1600/2000KN (2001) was considered. Column axial loading was distributed over 8 foundation piles, resulting in 12800KN maximum capacity.

A foundation plate of 10x16 meters was added under the stability cores, in order to prevent uplifting forces, the cores had small footprint (minimal demolition of existing structure), and lightweight building. 25 meters long tension new piles were added under the perimeter of foundation slab. In the Ultimate Limit State tension forces in the piles are up to 600KN.

The footing of the columns was not sufficiently reinforced to evenly spread the increased loading over the eight piles. In order to solve that issue, the lowest part of the column was demolished, with the assistance of a temporary support steel structure, and substituted with an intermediate footing with 4 short posts, achieving a more even distribution of the loading over the 8 foundation piles. Demolition, construction and repair low part of the column are carried out with the utmost care to prevent movements and retrieve a solid connection between the column and foundation.



Section of column and foundation footing (Source: Hermens, Visscher & Kraus 2014)

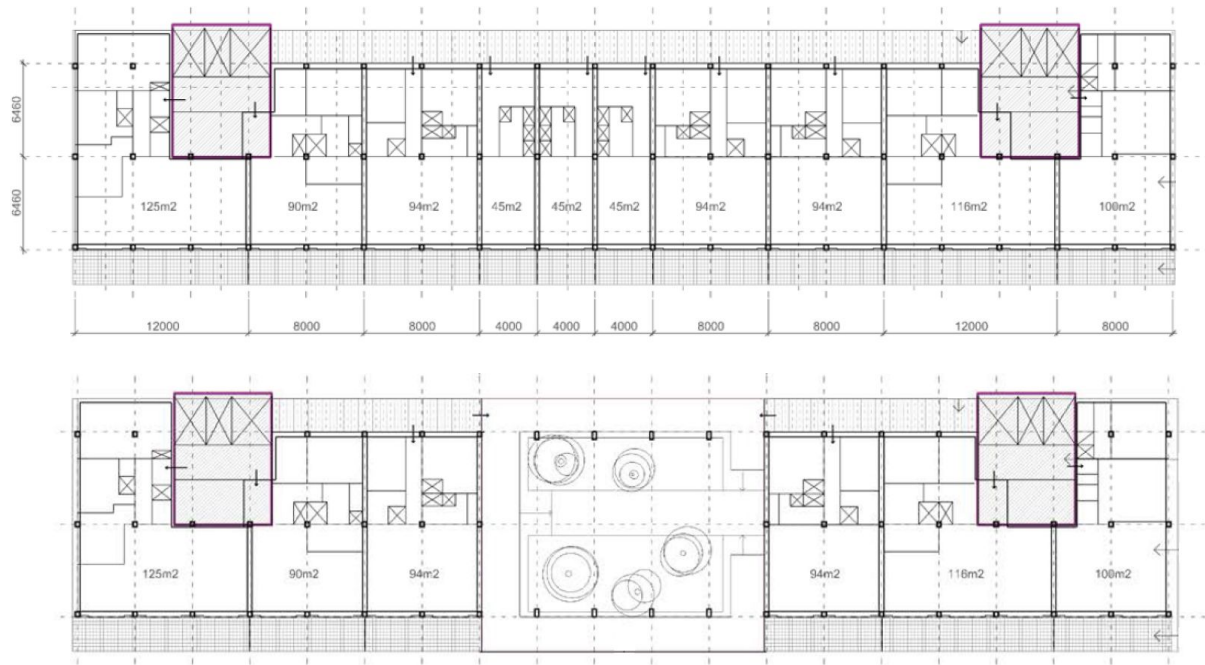
The new building extension inflicts differential loading on existing building, because it is placed on 2 of the 3 existing column lines, creating settlement difference of up to 25mm.

Lightweight new structure

Type	Ultralight steel columns and beams + Timber floor
Structural grid	4x6 meters
Stability concrete cores	7x9 meters (400mm thickness)
Dead load	250kg/m ²
Live load	175kg/m ² (one storey) 70kg/m ² (other storeys)

An ultralight column-beams structure was used for the new extension, with timber floor systems and 55 concrete topping. The interior walls were constructed with metal studs and gypsum boards, and the façade consist out of timber and glass panels. All inclusive a total dead weight 250kg/m² was considered for every stores, which is approximately one fifth of the weight of a standard apartment.

A design live load of 175 kg/m² on one floor and 70 kg/m² on all other floors (load combination factor 0,4 according to Dutch Code) was used for the calculations.

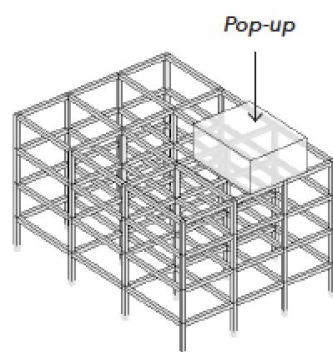


Typical floor plans 1-7th (above) 7-13th (below) of new structural extension, Karel Doorman (Own elaboration)

We can conclude that the most important structural limitations were located on the foundations. The lack of necessary reinforcement in the column footing for extra loading vertical loading, required the demolition of the lowest part of the column and adding an intermediate footing with 4 steel mini-columns. Uplifting forces, and foundation piles under the additional stability cores were an important issue. The solution required the use of long tension piles.

6.2 STRUCTURAL TYPOLOGIES

Pop-up

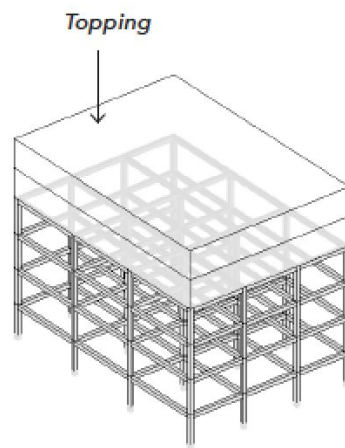


Lateral stability:
- Existing structure
Vertical loading:
- Existing structure
No demolition
Small scale

These extensions are characterised as small interventions with no demolitions. They can be considered small pavilions, "pop-up", or "parasites". They are made out of lightweight, small-scale structures that are placed on top of existing building, sometimes for a short period of time, as a temporary pavilion. Because of their small scale, they can easily be anchored to the existing building without any intervention on the original structure. Typically, the exterior is made out with bright plastic that creates a sharp contrast with the existing building. In many cases, the ultimate aim is to emphasize the existing building, because of its historical character, or because of its abandoned use. Restaurants or exhibition spaces are usual uses of the new parasites.

The structure is 1-2 storeys high, made with lightweight materials, such as aluminium or timber/CLT. The new extension is anchored onto the existing structure, in which relies entirely for both lateral stability, and gravity loading. The cube, Las Palmas Parasite (The Netherlands), Studio East Dining (United Kingdom) are some examples of this extension.

Topping



Lateral stability:

- Existing structure

Vertical loading:

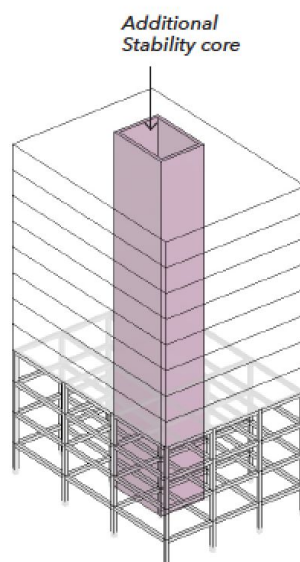
- Existing structure

Reduced demolition

Small/medium scale

These structural extensions consist typically of 1-3 storeys that are added on top of the existing buildings. They usually required small refurbishments and demolition works on the existing building. They have a permanent character that integrates within the existing construction, commonly, respecting the same structural grid and vertical circulation. Birkegade rooftop (Denmark), Hanover house (United Kingdom), Halle 12 (Germany) and Fahle house (Estonia) are some of the examples for this type of extension.

Stability core



Lateral stability:

- Additional stability core

- Composite action with existing

Vertical loading:

- Existing structure

- Additional stability core

Demolition required

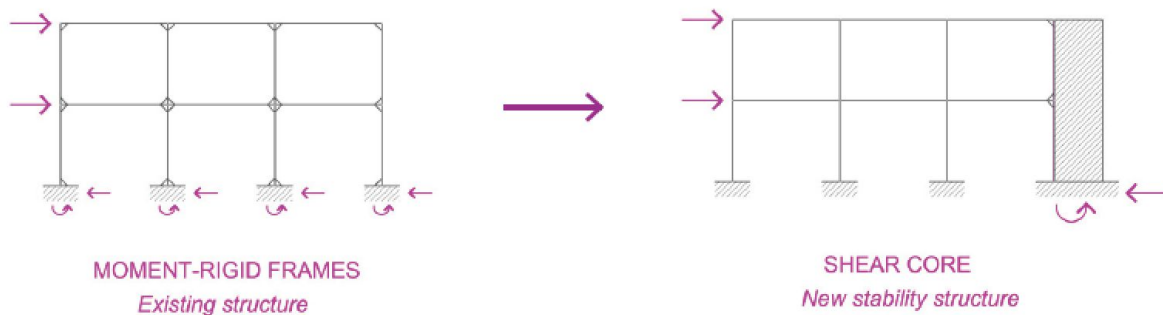
Medium/Large-scale

These extensions typically consists of 3-5 additional storeys atop of the existing structure. Because of the height of the new extension, additional stability systems are required, namely concrete cores. Two possible configurations can be possible:

- Core is externally adjoined to the original structure without the need of demolition
- Core is internally placed within the original structure by demolishing partly the original structure.

Giesshübel (Switzerland), Groot Willemsplein (Netherlands) and Karel Doorman (Netherlands), are some examples where gravity loading is supported by existing structure.

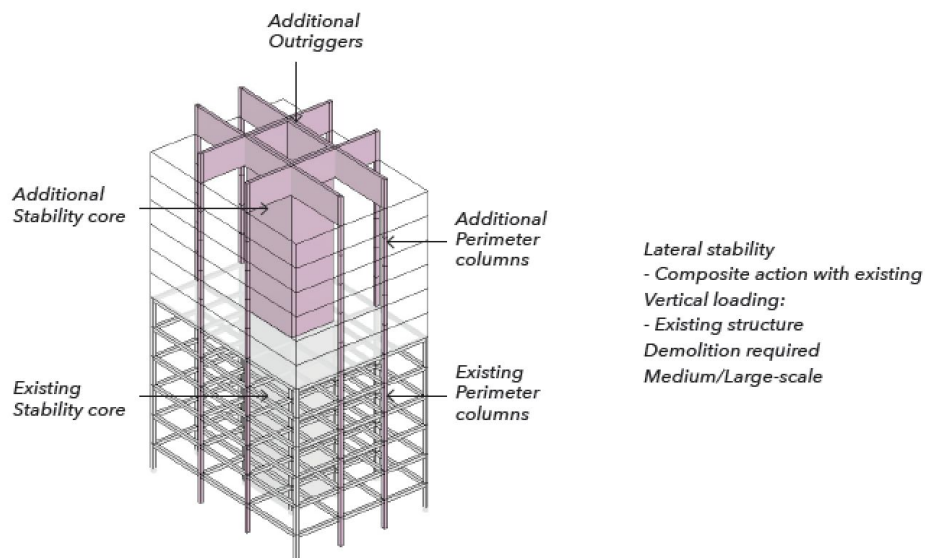
In other cases, such as Rottermann carpenter's workshop (Estonia) both gravity and lateral loading are supported by the added core..



A great number of multi-storey building from the past are constructed with concrete frames and rigid connections. This structural system is capable of transmitting gravity loading, mostly with the area of concrete columns, and lateral loading with the moment-resistant connections in the concrete frame.

If additional concrete cores are added, and connected laterally to the existing structure, the original concrete frames can be liberated from resisting lateral loading, and consequently, the vertical loading capacity is enlarged.

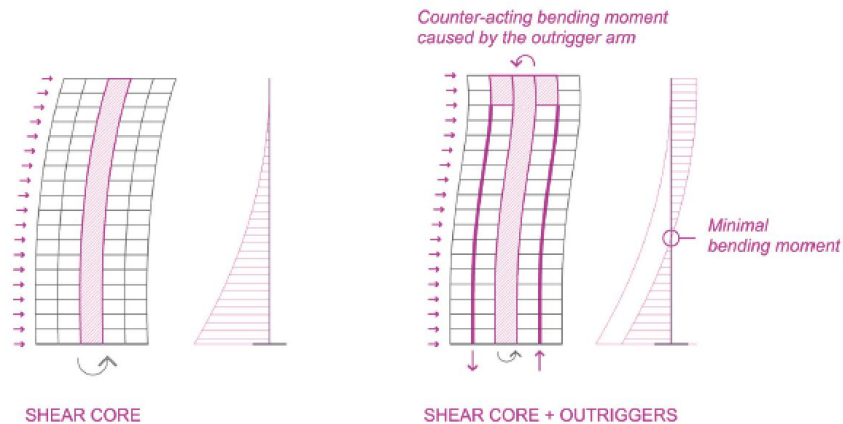
Outrigger



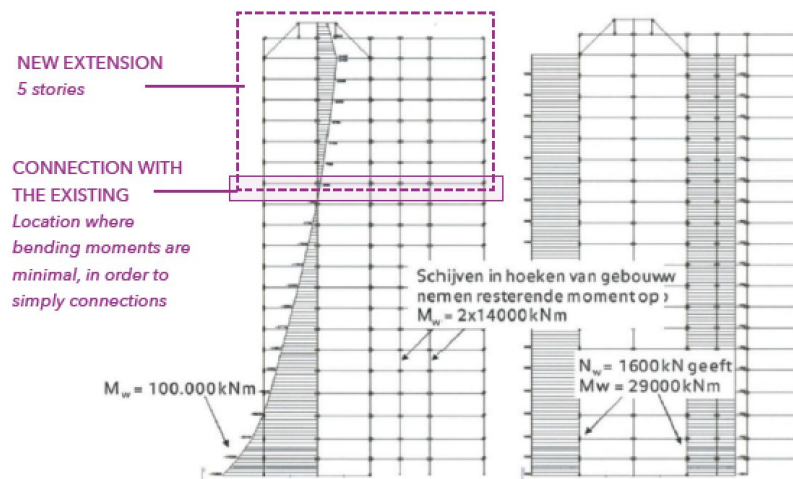
Using an additional outrigger steel structure, connected to the existing core and the inner-ring columns, the existing columns at the perimeter can react to the horizontal loads. In such a way, there will be axial forces in the columns, which will generate an adverse moment compared to the moment generated from the horizontal loads. This counteracting moment will have as a result not only the reduction of the moment acting on the lower part of the core but also the significant reduction of the horizontal deformations.

Moreover, an interesting point is that the stiffer the outrigger and the columns become with respect to the core, the larger the counteracting moment resulting from the outrigger, and hence the reduction of the moment on the foot of the core. Given the fact that the outrigger and the new columns are made out of steel, one of the most important parameters is the cross-sectional area of the steel profiles. In most of the cases, stiffness was

the normative parameter and larger profiles will need to be used, which were not necessary from the strength point of view.



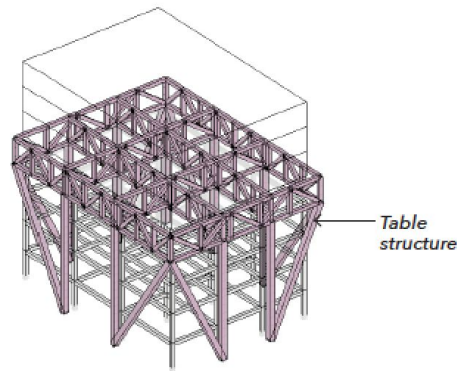
Scheme with differences between conventional stability core and outriggers provision



Example of Westerlaantoren case study with bending moments in the existing core after additional outrigger (left) and axial loading on the perimeter columns due to lateral wind (right). (Source: Aronsohn engineers)

Table structure

This structural system is composed of a strong steel framework, like a table structure, that literally lift the new building above the existing one. In this sense, it would be much questionable whether it is a real structural extension, even though it uses the same plot area. Nevertheless, the new structure can also be connected to the existing one, so that new table structure and existing structure behave in a composite way. Vertical circulation, grids and façade can be connected or disconnected with the original building. One example of this systems is Zeemanshuis in Netherlands.



Lateral stability:
 - Table structure
 - Composite action with existing

Vertical loading:
 - Table structure
 - Composite action with existing

No demolition
Medium scale

6.3 GUIDELINES

The construction of Karel Doorman with 16 storeys on top of an existing building is the highest at the time of writing, demonstrating an innovative ambitious approach towards structural design.

Other recommendations for achieving a simpler, and more efficient structural design is to respect existing structural grid, and spread evenly the new vertical loading on all existing columns. In this way, the use of complex grid-transfer systems, and settlements in the foundations can be avoided.

We can conclude that building extensions are complex projects that need an integrated approach from the engineer and the architect. It must be said that practice research is difficult to achieve, as they are dependant on multiple factors, such as context or economical constraints. At that same time due to the unusual technical complexity, the overall impact of structural considerations is higher than in a conventional building.

One of the objective of extensions is to avoid demolition of the existing structure. However, extensions of more than 4 storeys required the addition of an extra stability system. In many the construction of new structure is only possible inside the existing plot boundaries, and therefore a partial demolition is required.

Most of the building extensions are constructed on industrial buildings, or protected buildings. In many cases, together with the extension, refurbishment works are undertaken.

In large-scale extensions, such as Westerlaantoren and Karel Doorman, strengthening methods for the existing structures have been used, especially towards the bottom part of the building and existing foundations. These interventions can be extraordinarily complicated. In order to minimise these issues, some recommendations can be considered:

- Respect structural grid of existing building (in order to avoid transfer structures)
- Spread new loading evenly on existing columns (in order to avoid different displacements in the foundations)

The architectural concept aims at the following criteria:

- Maximising the use of the existing structure (more efficient use of material and sustainability)
- Minimise demolition (preserve existing urban fabric and reuse structures)
- Create a tall building (intensification of use of the city and consequently less resources)

According to that the following table can be extracted:

Type of extension	Extra storeys	Use of existing structure	Demolition	Tall building
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Pop-up	1-2	+	-	-
Topping	2-4	+	-	-
Stability cores	4-16	+/-	+/-	+/-
Outriggers	2-4	+/-	+/-	+/-
Table structure	2-4	-	-	-

+ Likely; +/- Partially; - Unlikely

The following scheme schematised the different building extensions typologies regarding the construction of tall building and the application of the system in the location of the design study.



PART 2

LOCATION

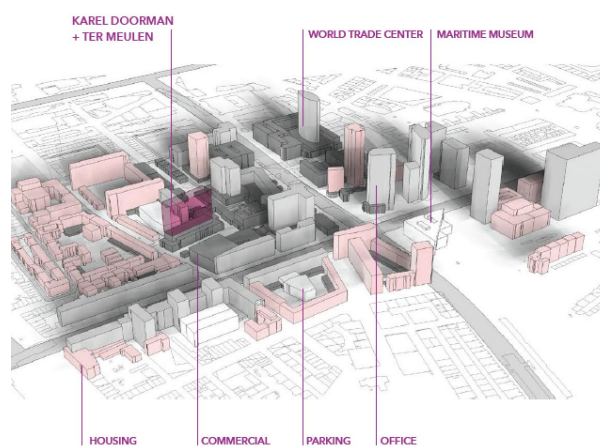
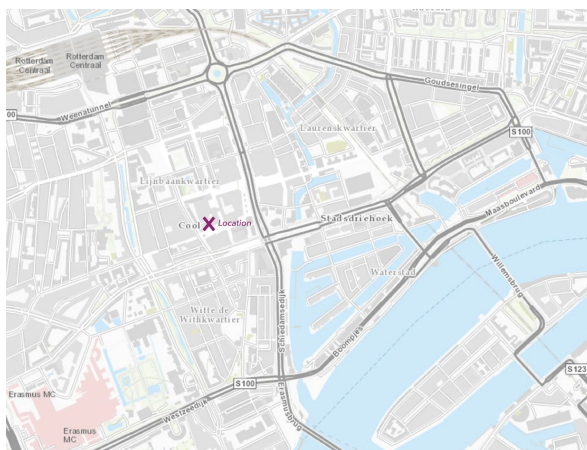
7. TALL HOUSING IN ROTTERDAM CENTER

- Selected location
- High-rise in The Netherlands
 - Historical background
 - High-rise today
 - The city of Rotterdam
- Definition of tall building
 - Height
 - Tallness
 - Height canvas
- Rotterdam high-rise policy
 - Urban zoning requirements
 - Public space
 - Building mass
 - Architectural requirements
- Market demands
 - Case studies
 - Conclusions

This chapter provides an overview of residential tall buildings in Rotterdam with the intention of finding the most relevant factors that can influence the design of successful quality housing.

7.1 SELECTED LOCATION

In order to evaluate the architectural and structural feasibility of the proposed concept, Tall Timber Extension TTE, a location in the centre of Rotterdam, namely Cool District was chosen. This specific location was selected inspired by the construction of the highest precedent of a building extension up to date. The zone is very active, with pedestrian streets and retail stores. The building plot is at the intersection between Lijnbaan and Binnenwegplein street. These two streets are very symbolic for the city of Rotterdam, as they were two of the first pedestrian streets in Europe.



Rotterdam city centre (Source: Actueel Hoogtebestand Nederland) and study of building programmes in the surroundings (Own elaboration)

7.2 HIGH-RISE IN THE NETHERLANDS

Historical background

The first era of high-rise housing was an explosive development after the Second World War, with the intention of solving the housing shortage of the new urban working class. Influenced by international opinion leaders, and professional architects such as Gropius, Le Corbusier and Hoobouw-Laagbouw institution, about 300.000 new dwellings were constructed during 1964-1974. Most of the high-rise in that period were built by housing associations by means of prefabricated rapid construction methods. The rapid development was paired with favourable government policies in order to swiftly meet the labour and housing demands in contemporary cities.



Image of typical high-rise construction and housing unit from 1960-70s. (Source: TNO Bouw)

The main typology is isolated blocks, or linked blocks with gallery-access, comprising two and three bedrooms apartments. The apartments are small, with no outdoor private spaces. The private domain is limited only inside dwellings, with no intermediate private communal areas inside the building. They new districts of high-rise are characterised by large scale and uniform block, based on rapid construction methods and efficient functionality.

The CIAM ideas of large scale, equality and function separation of the urban planning, resulted in an urban context as a dormitory for the working class, that work during the day in the city and commute to the surroundings at night. The housing high-rise were clustered in residential districts in the surroundings of the cities, with big public open spaces between the housing blocks, and dangerous feeling associated. There was no resident attachment to the area with high mobility and little social bonding.

In conclusion, this type of building never became popular among the residents, resulting in a poor living condition. The vast uniformity of the buildings lead to unhappiness of the users, as they felt unattached to urban environment. High-rise areas became unpopular leading to social deprivation. An example of this is Bijlmer district in Amsterdam.

From 1985 to 2005, there was a great impulse in the restructuring of depressed high-rise districts build after the World War 2. One approach was to improve the quality of life of existing housing, by using unconventional approaches such as partial demolition, building additional storeys, or combining dwellings.

Another urban strategy for the creation of new high-rise was the combination of different typologies, high-rise, low-rise, and other mixed typologies in extensive clusters outside of the cities. There was a focus on variation, comfortable living and sufficient amenities.

The main characteristics of high-rise housing developments built before 1995 are small apartments without private terraces, no communal social spaces inside the buildings, poor construction quality, uniform shape of the building and separation of programme in urban area. In conclusion, this lead to residents insatisfaction poor perception of high-rise housing typology. Therefore, differentiation of the building is important for creating an attractive environment, where inhabitants are offered a more diverse living style.

High-rise today

Nowadays, there is an increasing focus for urban regeneration in existing inner city districts using residential high-rise. The main approach impulse is the creation of key-projects that would act as a catalyst for the regeneration of several city locations. In general, those buildings are constructed by the private sector, and are dedicated to people that prefer city life style, and could afford higher living expectations.

This new optic for residential high-rise in active urban contexts has gained high popularity. The constructions are usually located in city centres, with mix of functions, residential, work and leisure, and the housing units and building appearance capture the differentiation of citizens. There are fewer public spaces between blocks (also a consequence of the city location), and security perception is augmented. At the same time, there is an increasing trend to use high-rise as landmark for the city.

High-rise in the Netherlands occupies a special position in the housing market. The high-rise blocks dating from the nineteen-sixties and seventies are by far the most uniform, the most direct and the most visible result of post-war spatial planning. Highrise has been making a comeback since the start of the nineteen-nineties. New high-rise housing is more luxurious, is located in a sought-after location, offers prospects for other groups in the population and, probably most important of all, over the years it has acquired a different image.



New high-rise developments in Rotterdam. (Source: Roel Dijkstra)

A study performed by TNO Bouw, indicates that the majority of current high-rise buildings were built in the period 1960-1970s, and most of them are concentrated in big cities and especially in South Holland. The most common high-rise dwelling covers an area between 60-100m². Usually the quality in a high-rise buildings is higher than in other types of buildings, better materials, construction details, bigger apartments, balconies...

Regarding the occupancy of the building, 90% of high-rise units are occupied by 1-2 people, with very few families living in tall buildings. From this, it can be extracted that this type of architecture can be more suitable for elderly people, young couples or students. According to TNO Bouw, the most crucial factors for residents in a high-rise are the size of the flat, outdoor areas, daylighting, appropriate soundproofing and good context location near public amenities, schools, public spaces, etc.

In conclusion, new high-rise developments must be able to accommodate private outdoor spaces, terraces and balconies, and high-quality communal spaces, rooftop garden, gym or swimming pool. Regarding the urban area, there should be a mix of different types of programmes, residential, office, leisure activities... especially at ground level, in order to activate the living environment, and impulse bonding to the district.

The city of Rotterdam

Rotterdam is the second most populous city in the Netherlands and the most striking example for high-rise construction. Nowadays it is home to Europe's largest port and has a population of around 630.000 inhabitants. The city has always had a strong reputation for architecture.

The history of the city dates back to 1340, and since then, it has slowly grown into a major logistic and economic center in The Netherlands. The White house skyscraper built in 1898 in French Chateau-style, is one of the evidence of Rotterdam's rapid growth and success. When completed it was the tallest office building in Europe, with a height of 45 meters. In World War II, the city was almost completely destroyed, known as Rotterdam Blitz, in 1940.



Witte Huis (left) and aerial bombardment of Rotterdam in 1940. (Source: Wikipedia)

From 1950s to 1970s the city was gradually rebuilt. In the following decades until now, the city has developed an ambitious policy, resulting in a new skyline with varied architectural landscape. In the 1990s, Kop van Zuid was built on the south bank of the river as a new business centre. In the first decades of 20th century, some influential architecture in modern style was built, such as Van Nelle Fabriek or Feyenoord's football stadium. Rotterdam is also famous for its hosting one of the first pedestrian streets in Europe, Lijnbaan constructed in 1952 by architects Broek en Bakema. Today, the city is home to some of the world-famous architecture firms, such as OMA, MVRDV, or Neutelings & Riedijk.

Today, Rotterdam is home to 352 high-rise buildings, situating as one of the top skylines cities in Europe, together with Frankfurt, London, Paris, Warsaw and Moscow. The city is also the location of some of the tallest structures in the Netherlands. The Erasmusbrug, built in 1996, with a height of 138 meters, New Orleans tower, the tallest residential high-rise at 158 meters high, or Maastoren, the tallest office building in the Netherlands with 165 metres height. At the moment there are over 30 new high-rise projects being developed.

#	Name of the project	Completed	Height	#	Name of the project	Completed	Height
1	Maastoren	2010	165m	6	Millennium Tower	2000	149m
2	New Orleans	2010	158m	7	World Port Center	2001	134m
3	Montevideo	2005	152m	8	First Rotterdam	2015	128m
4	Gebouw Delftse Poort	1991	151m	9	The Red Apple	2008	124m
5	De Rotterdam	2013	151m	10	Nieuwbouw Erasmus MC	2012	120m

Current highest buildings in the city of Rotterdam. (Source: Skyscrapercenter)

The municipality of Rotterdam is expected to add ten thousand new dwellings by year 2030. (Rotterdam City Vision 2030). This is equivalent 100 times a normal city block. At the same time, apart from the residential units,

office programme is also expected to increase in the future, and other activities such as shopping, culture and leisure must be also included into a compact and lively inner city centre. However, the space is limited. The aforementioned arguments suggest that the construction of new high-rises in the city centre is inevitable.

7.3 DEFINITION OF TALL BUILDING

Moon et al. (2007), indicates that regarding of location and construction type, building heights between 50-70 stories are likely to be the most economic. However, timber high-rise structures are far below this range of heights. It is therefore needed to clarify the term tall timber building, in order to understand and produce a meaningful comparison between buildings using different structural materials.

Height

Building height is a straightforward parameter, provided there is a common understanding of the start and end point to be measured. The Council for Tall Buildings and Urban Habitats (CTBUH 2013, 2015) recognises three ways for measuring tall building height:

- Height to the architectural top or gross height, provides the basis for World's Tallest Buildings and is measured to the permanent top of the building, including features such as spire but excluding antennae. Supertall and megatall buildings are those heights exceeding 300 and 600 meters
- Highest occupied floor or net height, is the number of occupied floors, and thus likely to be of greater practical interest for measuring meaningful tall buildings.
- Height to tip includes projections such as antennae that may not be permanent features of the building.

This report intends to compare tall timber buildings regarding meaningful architectural parameters, thus the term term height will only refer to the highest occupied floor or net height.

In general, tallness and height are not the same thing. Height is objective; it is a measurable property of a physical object. Tallness is subjective, it is a description of a physical object alluding to contextual references.

- *Height is objective, number of meters*

According to Council of Tall Building and Urban Habitats, CTBUH, tall building is a building with net height (highest occupied storey) of 50 meters. Considering national dutch standards, the National Covenant Hoogbouw defines a tall building as any structure higher than 70 meters with special use of tall building technologies, i.e. fire escape, vertical circulations, installations.

We can conclude that an objective building height above 50-70 meters can be considered as a starting point for the definition of tall building.

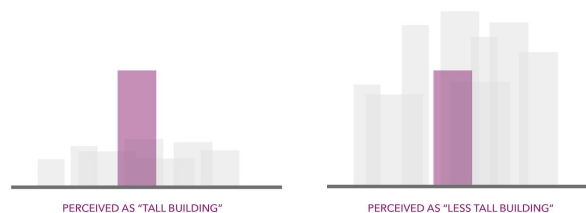
Tallness

With regards to the definition of "tallness", a building can be considered tall with respect to a number of different considerations, i.e. the use of a particular structural material, the context where the building is located, proportion of the shape, etc. Tallness is relevant for engineering science as the design of buildings that exceed the height of precedents using similar materials or systems places an additional burden upon the structural engineer (Foster et al. 2016). Another consideration that historically played an important role for the definition of a tall building is the resistance to fire. A tall building in this regard, is a construction whose height is such that the fire cannot be fought from equipment base in ground level (Calder et al. 2014). The performance of structural timber in fire is the subject of ongoing research (Fragiacomo et al. 2013, Klippel et al. 2014 and Frangi et al. 2008). Historical precedents with steel and concrete suggest that this perception of tall timber building will diminish with more research and experience regarding fire protection.

- *Tallness is subjective, in relation to references*

The Council for Tall Buildings and Urban Habitats (CTBUH) identifies three categories of qualities that contribute to the definition of tallness (CTBU 2015): Height relative to context, proportion and use of tall building technologies.

"Height relative to context" refers to the built environment in which the building is situated. For example, a 14-story building located in a suburban neighbourhood may be locally described as tall building, while the same construction in a high-rise area may be considered "less tall"



In the design study, the intended location is Rotterdam city centre, where the height of the surrounding buildings is very varied. In the following images we can see that a high-rise areas with buildings up to 100-140m height are located towards the east, whereas 2-3 storeys row houses with height between 5-20 meters are located in the west.

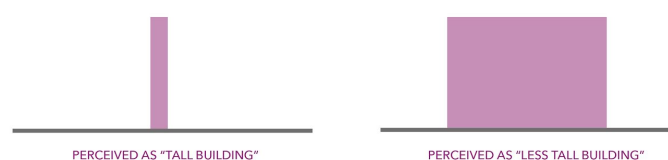


Left. Infographic with heights of representative buildings in the surrounding area (Own elaboration)

Right. Heights map of the surrounding area. (Source: Actueel Hoogtebestand Nederland)

From this, it can be concluded, that is it difficult to define the concept of tall building related to the context. A rough estimation of building height between 50-100m can be estimated as a starting point.

"Proportion", indicates the ratio between geometry and massing of the building. For example, a 14-story building on a small footprint may appear slender, and hence taller, than a similar building with a bigger base. Building proportion can be approximated with slenderness or aspect ratio, the ratio between structural height to smaller dimension of the structure.



It is considered that slenderness ratios of lower than 1:6 will not present a particular challenge for lateral load-resisting systems. On the other hand, ratios higher than 1:8, will impose great technical demands on the structure and the dynamic behaviour of the building due to wind and seismic actions.

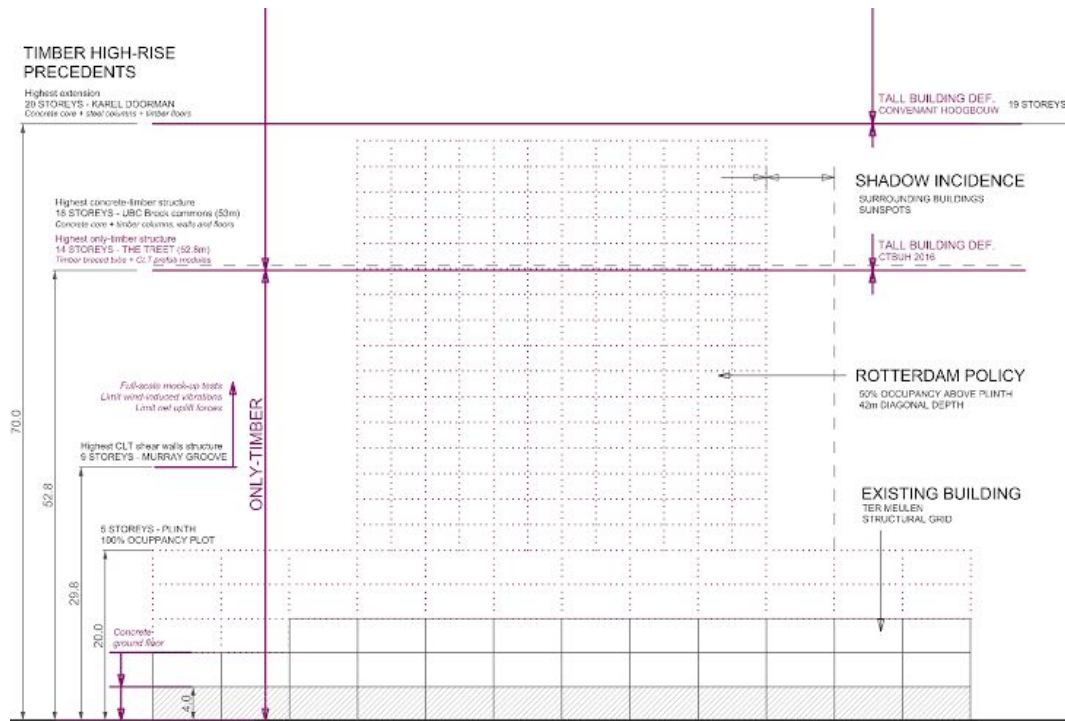
Thus, it can be concluded that a building can be considered tall if the slenderness ratio is higher than 1:6

“Tall-building technologies” refers to advanced vertical transportation, enhanced lateral force-resisting systems and damping systems that are particularly relevant for tall building design. A definition of tallness regarding technologies is of special relevance with respect to timber structures, because of its unconventional use in tall-buildings constructions. Both the lateral resisting and damping systems are closely related to the slenderness of the construction. Khan (1969) indicates two types of buildings structures, lateral loading and gravity loading governed structural design. While the forces carried by the vertical load-resisting system increase linearly with the height of the building, the moments caused by lateral forces increase to the power of two. At the same time, the greater wind velocities above the Earth surface exacerbate this effect. Each additional storey that the building extends results in an enlargement of the structure, simultaneously adding to the costs and reducing usable floor area, and with the addition of vertical transportation there is even a further decrease in the usable area. From an engineering perspective, the most important considerations are lateral forces due to wind and seismic actions, actual lateral sway, perceived lateral sway and differential vertical movements due to thermal effects and axial shortening. Foster et al. 2013 defines a tall building as any height that exceeds current precedents. At the time of writing, the highest timber building built up to date are “The Treet” and UBC Brock Commons, both reaching heights of approximately 53 meters.

According to Foster et al. 2013, a tall building requires the use of special technologies regarding, lateral resisting system, damping for wind-induced accelerations, fire protection and vertical transportation.



Height canvas



7.4 ROTTERDAM HIGH-RISE POLICY

In 2011, the municipality of Rotterdam released “Hoogbouwvisie”, the main policy rules for the city regarding high-rise buildings. The document is based on 4 main components, namely, high-rise zone and maximum height, continuous city, climate and volume and slenderness rules.

Urban zoning requirements

In the past Laurens church stood as the main visual element for the skyline of Rotterdam. In today's skyline there is an increasing concentration of towers in the inner centre that determines the hierarchy of the city. This image has been reinforced with the new developments and municipal policies. At the same time, this means that new high-rise constructions must reconcile in scale, size and consider implications in their surroundings.

High-rise zone

The first spatial planning regarding tall buildings in Rotterdam was the “Binnenstadsplan 1993-2000” that designated an specific area for high-rise buildings in order to impulse the development of the city. The area runs roughly from Central District to the Southbank of Maas river, clustering tall buildings according to economic functions. In the new city vision, “Hoogbouwvisie 2011” the line from Hofplein via Coolsingel and ending in Wilhelminaplein defines the backbone for tall buildings in Rotterdam. Two clear zones are defined, one with no height restrictions (dark purple) and one transition zone (light purple), with height restrictions depending on surroundings 70-150 meters.



Height map of Rotterdam based on Hoogbouwvisie 2011 (Own elaboration)

Solar rights

Micro-urban climate can also be highly affected by the construction of tall buildings that may block the sun incidence in the urban space and surrounding buildings, consequently increasing the discomfort levels. The municipality has designated one main area with intensive use of public space, as “Climate Comfort Zone”. This means that when a new high-rise building is erected in the area, the effect on the surrounding environment must be carefully studied. The other high-rise areas, Wilhelminapier and Central District Rotterdam, have larger buffer zones towards the north, which results in less affection to the immediate surrounding climate.



Potential zones for high-rises and regarding climate impact in urban activity.

When a new constructed is erected in a high density area, it should be clear the amount of shadow that it will created in the immediate surroundings. It is also important to consider the time of the day where the shadow will occur, for instance, there are places that are intensively used in the afternoon, and less in the morning. Then occasional shade at early hours may not be permitted.

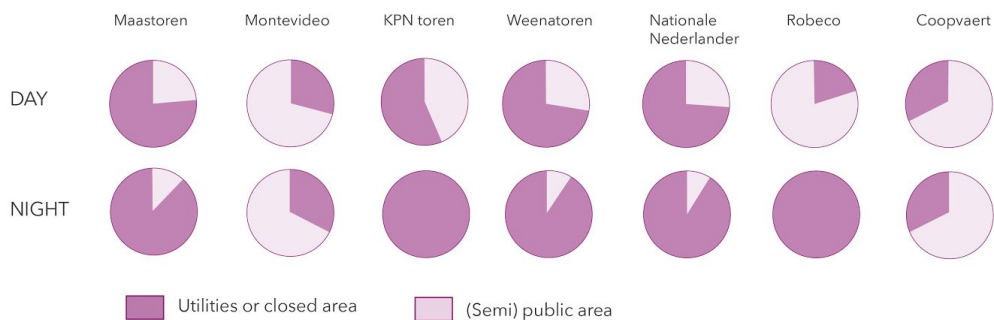
In the “Comfort Zone” or inner city centre of Rotterdam, there are restriction of shadows depending on the location of the site. There is a distinction between occasional and complete shading restrictions. The later ones, are called “sunpots”, places where no deterioration may occur in designated times of use. Those are places that have a specific quality and are representative for the city. Other areas, can receive up to 2 hours occasional extra shadows, this times apply within the normative period, March 21 to September 21 at ground floor level.



In some of the existing high-rise projects, there are wind nuisance at ground levels. This is commonly because of later consideration in the design process, and it is usually solved with additional elements such as canopies at entrances. With new regulations, earlier investigations of wind effects in surroundings have to be investigated in early stages of the design

Public space

The plinth of the building is composed out of the first storeys of the high-rise and it provides the perception of the tower at street level. A survey of existing high-rise buildings in Rotterdam shows that 30-60% of the plinth GFA is used for utilitarian purposes, such as storage, parking, technical rooms or emergency exits. This percentage is even higher for office towers, with 40-80%. Furthermore, it also appears that from the little remaining space, 60% is dedicated to the entrance lobby. This means that there is little space for an attractive urban program, such as retail, cultural or leisure in the lower layers of the building. It is also noticeable, that because of the relative small size of the base compared to the overall volume, the entrance lacks representativeness.



Plinth programme in high-rise in Rotterdam.

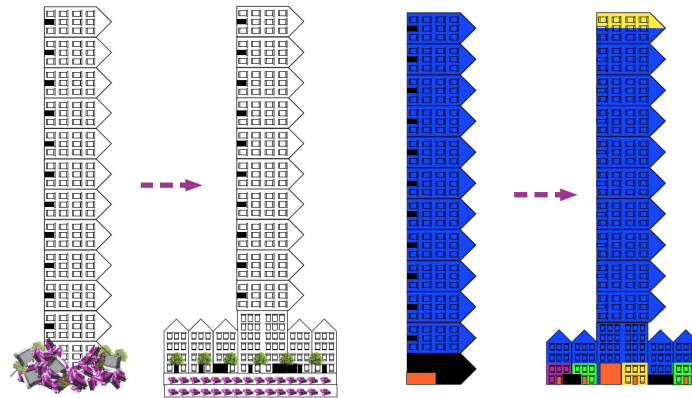
(Source: Own elaboration, based on data from Hoogbouwvisie 2011)

The quality of public or semi public functions in the towers is very diverse. Towers such as Montevideo and the Coopvaert have an important part of the plinth filled with urban-attractive programme, creating liveliness throughout the day and night. On the other hand, there are several high-rises, with little urban connection at the

plinth level, and activities that are non-functional during night-time, for instance a lobby entrance of an office building that usually closes at 18.00 and during the weekends.

City vision

The main city vision is to create a largen urban plinth, as high-quality programme base for the 'Groundscraper' and 'City Lounge'. The new high-rise should result in a richer program and an integrated urban layer, a network of streets, squares and public spaces formed by interesting and attractive building bases. It is therefore essential to create good plinth in high-rise buildings, where private and public life are not mutually exclusive.



Rotterdam vision for high-rise buildings (Source: Hoogbouwvisie 2011)

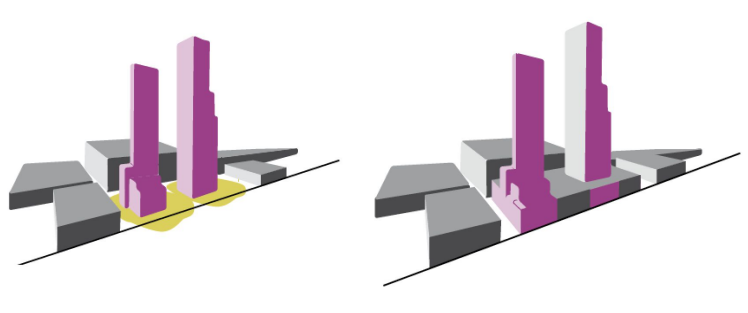
The policy defines two dimensions of high-rise, "streetscape" and "cityscape". The perception from street level, and the image it creates in the city skyline. New high-rise should actively contribute to the urban atmosphere and enhance micro-climate. In conclusion, situating functions in the base of the building that can be attractive for the by-passers, and putting more attention in the human dimension and differentiation is paramount.



Street level and skyline experience of Montevideo high-rise. (Source: Mecanoo architects)

Urban continuity

The inner city centre is formed by continuous paths of street walls and blocks with an average height of 25-30 meters. It is therefore important to ensure a special attention to the first 4-5 storeys of any new high-rise construction. The building should be part of a larger urban block, and must contain an urban plinth. The programme of the base should contribute to the urban dynamics and continuity of the cityscape, for example retail, cultural and leisure functions. The base should also be carefully detailed and as transparent as possible, and should have an impact in the overall volume of the high-rise.



“Stand alone” and urban continuity high-rise typology. (Source: Hoogbouwvisie 2011)

The size of the plot determines the dimensions of the building. The total plot must be constructed to a height between 20-25 meters, creating the plinth for connection with the urban space. This may vary depending on the height of the buildings in the surrounding area.

Respect existing fabric

The turbulent recent history of Rotterdam, it is still readable from the combination of urban structures and buildings in the inner city centre. These constructions represent the collective consciousness of the city, and create a continuity in the cultural heritage of the past. It is therefore important, that the existing fabric is carefully respected. New high-rise buildings in the inner city centre of Rotterdam must respect the composition, articulation, materialisation and position of the existing buildings in the city.

Utilitarian functions

Various high-rise buildings in Rotterdam contain plinths with low quality regarding the connection with the urban space. As it was aforementioned, this is due to the relative small area on the ground floor and the required utilitarian facilities. Because of those two factors, there is little extra space for more urban-attractive functions.

Utilitarian functions should be concentrated towards the most inner part of the building, apart from the public sides. This allows the creation of public programme for the connection to the urban space.

Building mass

In order to achieve maximum daylighting and at the same time, avoid great shadow impacts in the surroundings, restrictions regarding size and volume have been established, mostly maximum occupancy of the plot and maximum depth.

As a general rule, a maximum of 50% volume of the total plot can be constructed above the plinth.

In order to avoid great wind disturbances in the city, the building should avoid large continuous vertical planes in contact with the street level. The overall shape of the building should follow a stacked pyramid concept, open-voids, or multiple volumes that decrease in size towards the top.

Maximum floor area

The maximum floor area is specific to the main programme of the building, considering a maximum of:

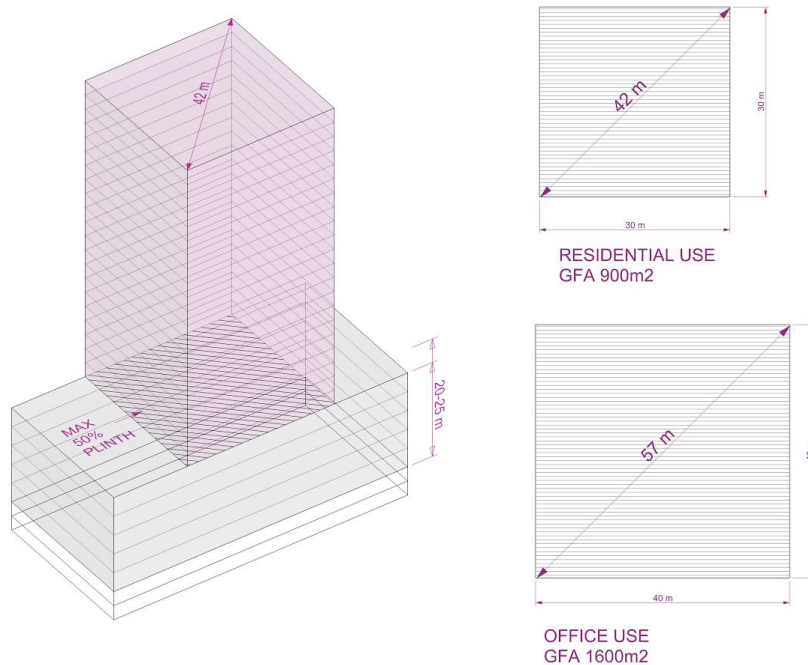
1600m² for office and 900m² for residential towers.

Smaller gross floor areas are advised for residential high-rises because higher restrictions in order to maintain quality of the dwelling units, adequate light and outdoor spaces. The second consideration is related to the floor depth, and adequate daylighting. The main restrictions are:

Below 70m height, a maximum diagonal of 56 meters (Equivalent to 40x40 meters, 1.600m²)

Above 70m height, a maximum diagonal of 42 meters (Equivalent to 30x30 meters area of 900m²)

The use of the diagonals for measuring building depth ensures the construction of towers instead of other building forms, such as disks that can have high shadow incidence in surroundings.



Maximum occupancy and floor depth

In the case of a high-rise building higher than 70 meters and a plot of less than 3200m² a GFA of more than 900m² may be possible, providing that the new constructions strongly contributes to the urban space at ground level. This means that at least 60% of the plot must have a public or semi-public character. It is possible to in a plot of less than 3200m².

Architectural requirements

High-quality rooftop

In a high-rise building, both the roof and other external spaces can have an enormous potential. Terraces and rooftops should have special attention, and the sustainability should be an integrated part of the building. They can enhance the image of the city, in contrast with hard materials of pavements, improve quality of human experience and benefit the climate of the city and the building itself. For that reason, the municipality wants to encourage the use of green in rooftops and terraces in high-rise buildings.

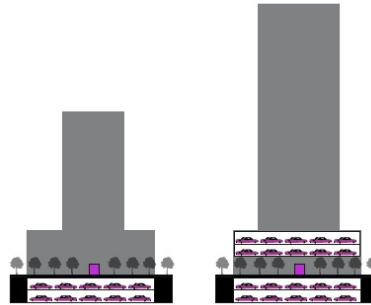
According to Hoogbouwvisie green roofs can provide longer life of the rooftop, as it protects against UV radiation and temperature fluctuations and insulating climate effect, warmer in winter, and cooler in summer,, enhancing energy savings and reducing CO₂ emissions. At the same time, they can be an absorption of sound and buffer against pollution/air dust, and slow down rainwater and reduce floods during heavy rains.

In conclusion, the building is required to have higher quality at the top, as it adds to the amenities of the building and help achieving better energy management. In addition the building must provide good outdoor spaces, by adding green spaces on the façade, terraces and rooftop. Technical installations should be hidden from view.

Parking and car accessibility

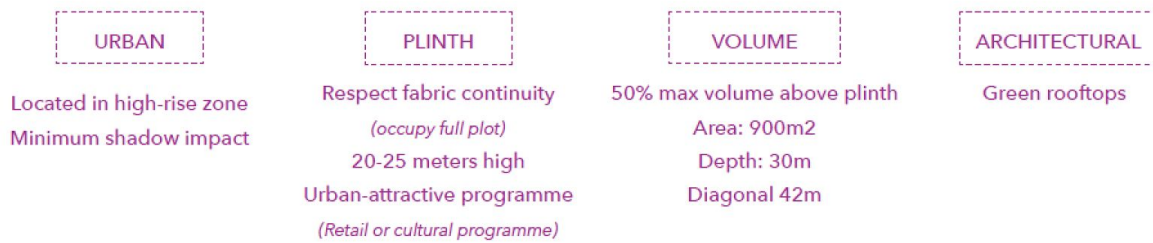
New urban densification policies in the centre of Rotterdam, result in higher demands for parking spaces. Because of that, many high-rise projects create additional parking spaces above the underground level of the building. Policies encourage the placement of parking on underground levels. In case it is above ground levels, it should be above first two storeys, and should not be visible in any case. The use of external parking facilities

in other buildings can also be studies. Finally, when possible, programmes should encourage a reduction in car usage, by impulsing bicycles and public transport and bicycles.



Parking typologies in high-rise in Rotterdam. (Source: Source: Hoogbouwvisie 2011)

The following image summarises the most important municipal requirements for a high-rise development in Rotterdam, grouped in four categories, urban pint, volume and architectural demands.

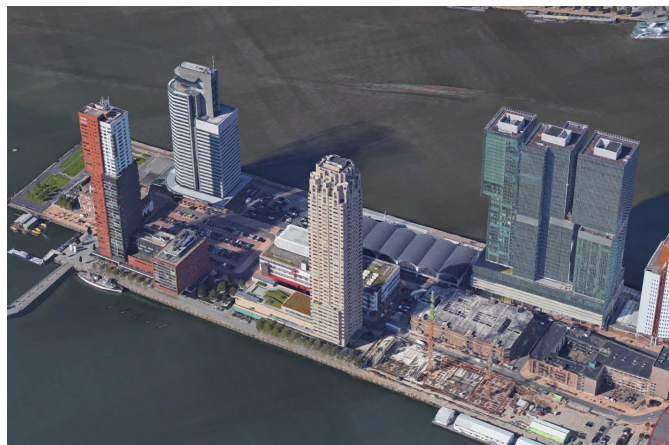


7.5 MARKET DEMANDS

Through this case studies analysis, a research of the current social and economical demands for a high-rise residential tower in Rotterdam city center is intended, together with an analysis of the most prominent architectural considerations for this type of building, regarding floor layout and programme.

6 recent high-rise buildings in the city centre of Rotterdam have been analysed, as a representative sample for current demands of tall housing buildings.

New Orleans

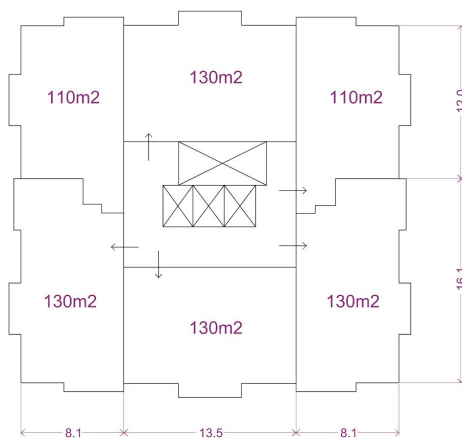


Building in the context. (Source: Poggenpohl and Google Maps)

Architect	Álvaro Siza/ADP architects	Technical advisor	Techniplan adviseurs
Roof garden	Quadrex landscape architects	Completed	2007 - 2010
Client	Vesteda	Height	160.5 m
Developer	TRS Ontwikkeling	GFA	60.000m ²
Engineers	D3BN Deerns DHV	Residential	234 housing units
		Leisure services	4500m ²
Main contractor	Besix	Retail	450 m ²

With a height of 160.5 meters, New Orleans building is the tallest residential tower in the Netherlands. The design of the building was carried out from the famous architect Alvaro Siza, in conjunction the dutch architects ADP. The main characteristic feature of the building is its external cladding with a yellowish natural stone. The concept of the building, designed by a portuguese architect tried to give a southern character to the tower, creating a warmth, and elegant feeling. The stability system of the tower is formed in an H-shape with concrete shear walls, and the vertical loading consist of 14 concrete mega-columns.

Situated in Wilhelminapier, an increasingly important area of Rotterdam. In recent years, the area has become one of the high-rise spots of the city, with a growing number of important projects, already developed or being developed, namely, Luxor Theatre, Hotel New York, and other Mega-projects, such as De Rotterdam, from OMA, and Montevideo residential building. The area is also the arrival and departure point for big cruise ships.

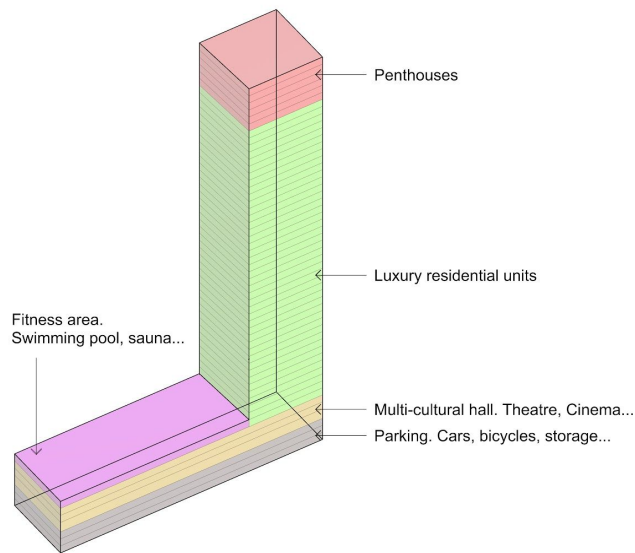


Typical floor plan

The base of the residential tower is approximately a perfect square of 29x29 meters, with a total gross floor area of 877m². The internal distribution is carried by a central concrete core, integrated with the vertical circulation and horizontal distribution of at least 6 apartments per storey. All the apartments in the building can be considered spacious and luxurious. There are up to 18 different types of apartments, with sizes ranging from 60 -211m², with bigger dimensions at the top.

In general, this can be summarised in three main types, Studios with 2 bedrooms, Apartments with 3 bedrooms, and Penthouses, with extra space. Being the tallest residential tower in The Netherlands, all the apartments have spectacular views over the city of Rotterdam, to the southeast and southwest the views are towards the harbour, and to the northeast and northwest towards the city centre. Because of the further reach of views there is an increasing value and luxury the higher the position of the apartment. At the top of building from 39 to 45 storeys, "The Crown", there are luxurious penthouses with sizes ranging from 130 - 211m²

The housing units have generous private loggias that allow fresh air and sunlight in the apartment, and at the same time shield from high winds. According to the developer, for dutch culture it is important the immediate connection with outside air.



At the foot of the building it is situated the LantarenVenster a multi-cultural hall, with five cinemas, one theatre and a roof terrace. The entrance to the residences is separated, in order to avoid noise and privacy problems. The development agency advertised the multi-cultural hall as an extra quality that is added to the residential use of the tower. There are also 3 accommodation rooms for external guests, that can be reserved by the residents of the New Orleans in case of a visit. The service costs for these rooms is included in the community service.

In the three storeys underground of the building, there is available parking spaces for residents, with both rental and sale possibilities. It is also a direct connection between the residential tower and the car park, for the comfort of the residents. On first floor, there is separated bicycle park, and on third floor, private storage facilities for each apartment, directly accessible from the central core. Finally, a 1000m² roof terrace, on the fourth floor, provides collective outdoor space for residents of the tower. The amenities are sun deck, swimming pool, sauna and fitness area, with exclusive use of residents of the building.

B-Tower

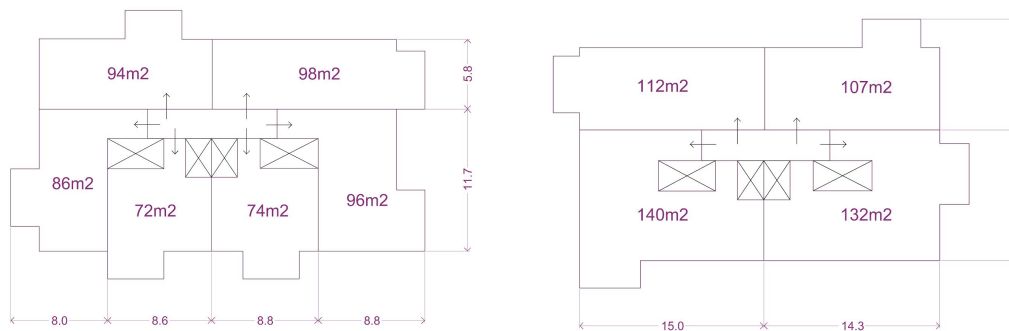


Image of the building in the surroundings. (Source: e-architect)

Architect	Wiel Arets Architects	Completed	2007 - 2010
Client	MAB Bouwfonds	Height	70 meters - 19 storeys
Developer	TRS Ontwikkeling	GFA	15000m ²
Engineer	Zonneveld Ingenieurs Simonies Raadgevende Ingenieurs	Residential	12500m ² 78 Residential units
		Retail	4000m ²
		Parking	64 car spaces

The project was erected in the plot of the previously demolished shopping mall Beurstraverse. One of the main requirements of the design, was a to limit the volume occupancy above the previous demolished building to 30%. This constraint resulted in a tower, instead of a linear block.

The sculpture of the building is composed out of three volumes of similar height, that are stacked on top of each other.



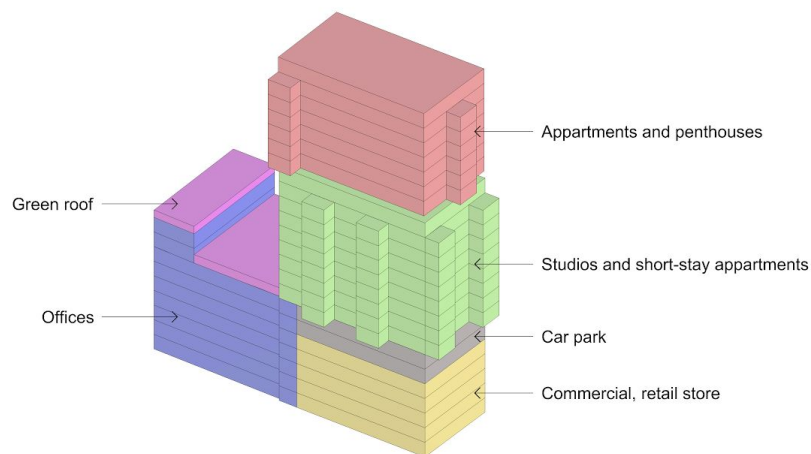
Left: Typical studio layout. Right: Typical apartments layout

The residential units are located around a central corridor where up to six apartments can be accessed in each storey. Two main volumes of gross floor area 840 m² and dimensions 17.5 and 31.9m shift with respect to the central core, creating two primary apartment layouts, studios or apartments.

The middle volume of the tower contains 54 studios with one bedroom, with sizes ranging from 55-80m², and up to 6 different types of units..

The upper volume contains 24 apartments and penthouses with sizes ranging from 80-120m². The penthouses are located in the top storey.

A series of cantilevered private balconies complete the housing units, with a size of 4.8x2.1 meters.



The building can be subdivided in three main volumes. The first one is a 4 storey plinth that hosts retail and office spaces. On top of this mass there are two more volumes slightly shifted from each other that host the residential units. On the ground floor there are two independent entrances from a pedestrian street, one is dedicated to the private access of the apartments with lobby and services, and another for public access to the commercial retail store. The façade of the commercial store is clad with transparent glazing in order to create an inviting effect.

On the 5th and 6th storeys, there are 64 parking spaces for the residents of the tower, with a private car elevator.

Finally, a green roof is placed on top of the lowest volume, in order to create a more pleasant view from above.

Montevideo

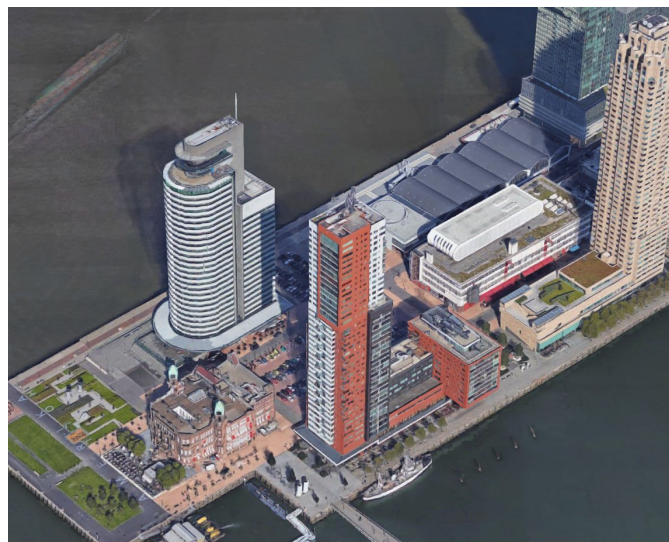


Image of the building in the surroundings. (Source: Mecanoo architects)

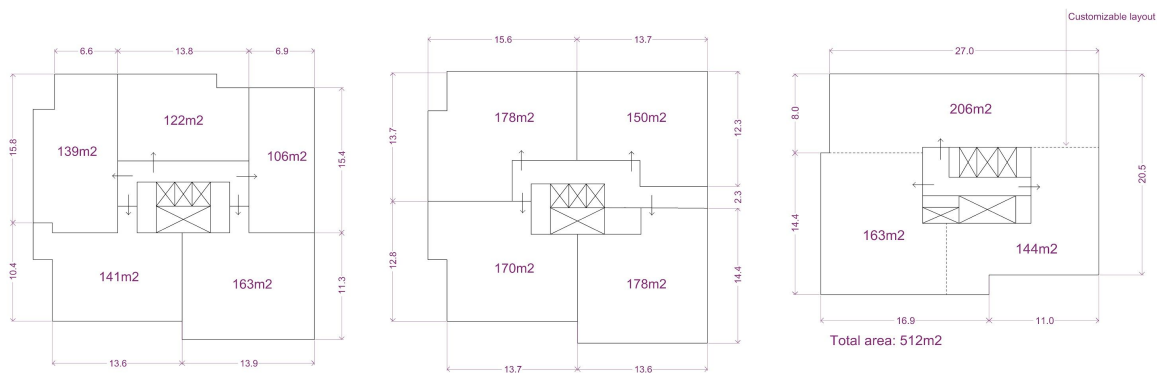
Architect	Mecanoo	Residential	36867m2- 192 units
Developer	ING Real Estate	Pool	905m2
Engineer	ABT Delft	Other services	Guest suites

			Gym, Sauna
Main contractor	Besix	Office	6129m ²
Completed	2003-2005	Retail	1608m ²
Height	140m - 43 storeys	Parking	8413m ²
GFA	57530 m ²	Parking spaces	270

The building is situated in Kop Van Zuid island or Wilhelminapier. The structure is hybrid echoing the typical dutch concrete construction and the american style steel structures. The Montevideo name resonates with the exotic names of previous warehouses in the dock, such as New Orleans, Santos, Baltimore, Havana.

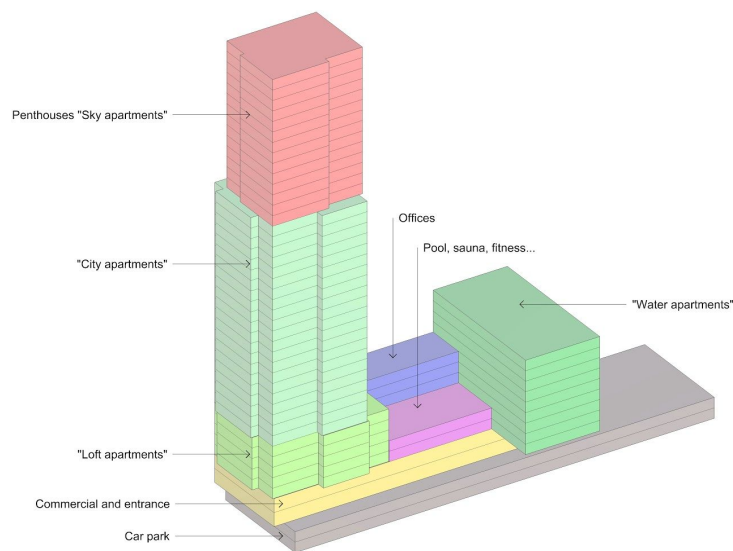
Most of the apartments can be considered luxurious, as they are spacious, luminous and have access to private balconies and collective indoor fitness area. Even though, there are multiple typologies and floor layouts, they can be grouped in three main categories, studios, apartments and penthouses or super-luxury apartments. These groups are distributed along the height of the residential tower, the higher the location of the residential unit, the more luxurious it is (bigger size, more façade and better views).

From 2th to 5th floor, in the bottom part of the tower, there are 20 residential units, with usually 5 apartments per storey. The sizes range from 96 to 163m². "Loft apartments"



Different floor layouts of the building. Left: bottom, loft apartments. Middle: middle, city apartments. Right: Top, sky apartments

From 6th to 27th floor, in the middle part of the tower, there are 94 residential units, with 4 apartments per storey and access from a central core. The sizes range from 93 to 184m². "City apartments". From 27th to 41th floor, in the middle part of the tower, there are 32 residential units, with customisable layouts and ceiling height of 3.2 meters. The sizes range from 144 to 262m². "Sky apartments"



The building can be divided in three main volumes, standing on top of one common plinth and an underground car park. The two outer volumes, 41 and 10 storeys, contain luxury apartments, the middle volume, is used for offices and indoor leisure activities, swimming pool, fitness club, sauna... for the private use of the residents of Montevideo. The main tower accommodates three main types of apartments, Loft, City and Sky with increasing sizes according to the height location in the building. In the top 13 storeys, the most luxurious housing units and penthouses are located. The shorter volume, cantilevers over the public space, and contains the most varied selection of housing units. The three main volumes are situated on top of a 2 storeys plinth that serves for private entrance and commercial use.

The Red Apple



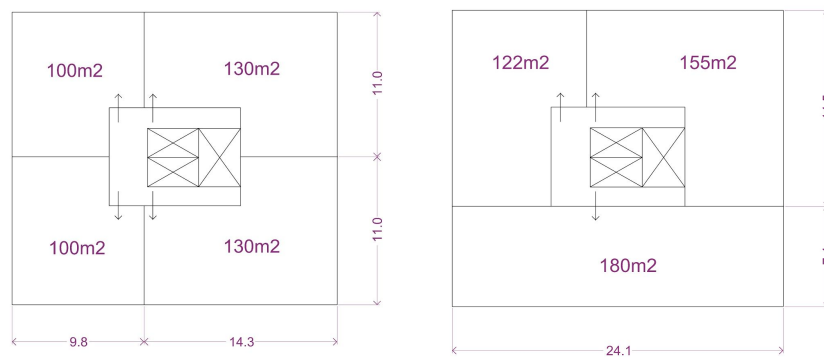
Image of the building in the surroundings. (Source: KCAP architects)

Architect	KCAP Architects & Planners Jan des Bouvrie	GFA	35000m2
Developer	Winnervest Investment Pte Ltd	Residential	29360m2 231 apartments
Engineer	TBI groep	Retails and restaurants	1500m2

Main contractor	Aannemersbedrijf v+h Boele & Van Eesteren	Office	3800m ²
Completed	2002-2009	Parking	340 places
Height	124m - 38 storeys		

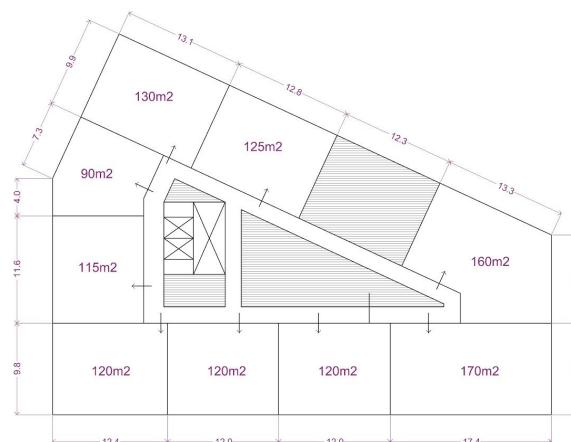
The Red Apple is a mixed-use development which rises 124 meters at the head of Rotterdam's Wijnhaven Islands. The development consists of two major volumes— a mixed use tower and a partly cantilevering apartment block building. The volumes are connected by a public plinth which anchors the volumes within the existing surroundings. All apartments of the two volumes are diagonally oriented and offer a maximum of transparency via floor-to-ceiling glass and provide extraordinary views.

The 38-storeys tower located in the west of the plot, has a central distribution with 4 apartments per storey. The sizes range from 100 to 180 m², with internal flexible compartmentation, open space, 2-3 bedrooms. Every apartment faces at least two different sides of the façade, so that they can receive maximum daylighting and provide better views. The top storeys, a different layout with 3 apartments per storey is used, for bigger residential units and penthouses.

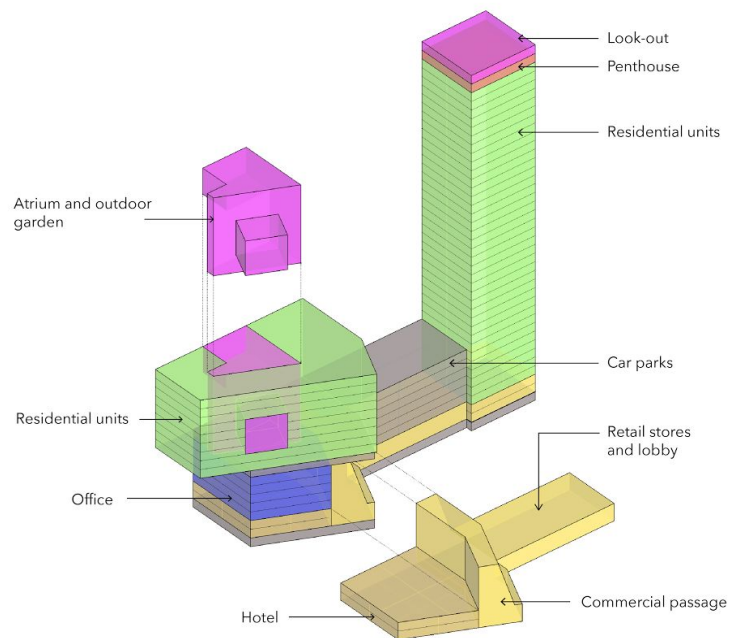


Typical floor layouts for the tower in the west

In the 9 storey block located towards the east, there is an internal atrium, that acts as corridor-deck distribution for 8 apartments around it. The sizes of the units range from 90 to 170m². The internal void also protrudes the block laterally in order to create pleasant internal space, with more daylight. Towards the north, a 5 storeys void connects the exterior with the internal atrium, and towards the south, there is a 3-storeys cut-out at the top creating a rooftop outdoor space for the residents of the building.



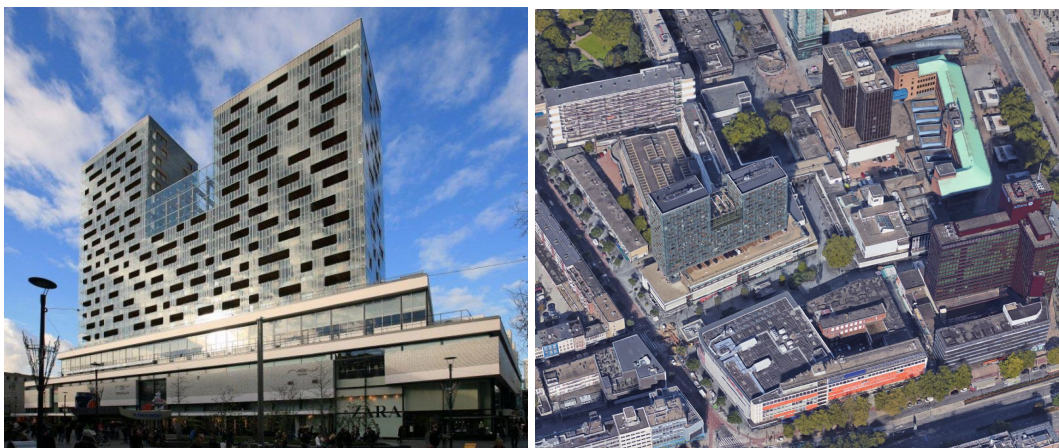
Typical floor layout for the residential block in the east



The building complex can be separated in three main volumes, tower, residential block and plinth. The lowest volume has a highly hybrid use, with retail stores, commercial passage, underground parking and a hotel. This multi-programme volume serves to connect the two residential block with the surroundings creating a connection with the public space and articulating a vibrant atmosphere.

On top of this plinth, there are 5 storeys of office space, and car parks that can be accessed with 4 car elevators. Added to that, there is an extra storey of car park on the deck of the low volume. The residential units are separated into one 9 storey block with an interior atrium and outdoor space at the top, and a 38 storeys tower with penthouses and lookout rooftop, that was later used for the technical installations.

Karel Doorman



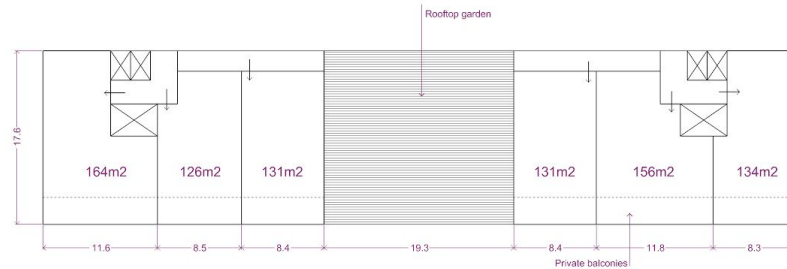
General image of the building (Source: Reflexxion architecture)

Architect	Ibelings van Tilburg Architecten	Completed	2006-2012
Client	DW Nieuwbouw	Height	70 meters

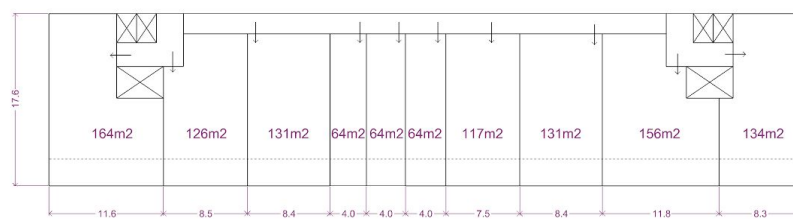
Engineer	Royal Haskoning DHV	GFA	10500m ²
Main contractor	Van Wijnen Dordrecht	Residential	114 units
Consultants	Marcel Vermeer Jessica Scholtes	Retail	11500m ²
Technical advisors	Peutz associates Wichers & Dreef	Parking	4000m ²

In the first place, the client commissioned the architects for the design of a conventional building in the existing plot. Needless to say, this option implied the demolition of the existing building, that was considered an important symbol of modernism architecture in the city of Rotterdam. It was for that reason, that Ibelings van Tilburg in collaboration with Royal Haskoning proposed to preserve the existing building, creating more programme on top of it, and making an ultra-light construction.

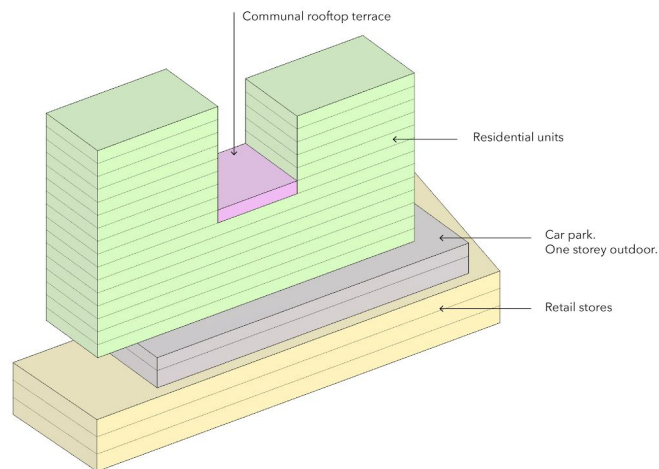
All the apartments can be accessed through an external deck corridor, or directly from one of the two cores. The position of the vertical circulation cores is symmetrical and they are located towards the sides. The deck corridor is connected to both cores due to fire regulation requirements (Residents must have at least two fire escape exits). There are 6 different categories of units according to the internal distribution and size, and all of them have a private terrace. The width of the units varies from 7.9 to 11.9m and depth 12.8 to 15 meters. The size of the apartments ranges from 44.5 to 164 m², with increasingly bigger apartments towards the top.



Typical floor layout, 10th to 15th storey



Typical floor layout, 4th to 9th storey



Schematic programme of the building (Own elaboration)

Retail stores are located in the basement, ground floor and first floor of the existing building. At the same time, each of the floors has a mezzanine level, providing up to 11.500m² of commercial area. On the second storey and roof deck there is a car park with 156 places that can be accessed by car lift from Crispijnstraat street. The residential units are split in two towers, 16 and 13 storeys, and one middle volume of 7 floors. In between, the two towers, on the rooftop of the middle volume, there is a communal roof garden for the use of the residents at 40 meters high.

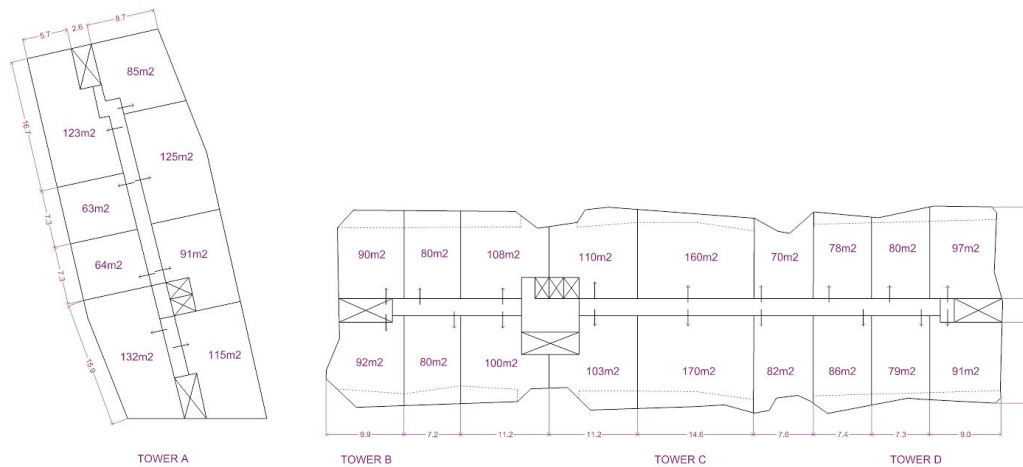
Calypso towers



General image of the building (Source: Architectuur)

Architect	Van der Laan Bouma Alsop architects	Completed	2009-2012
Client	Willows Estate	Height	70 meters - 22 storeys
Engineer	Mould engineering	GFA	72500m ²
Main contractor	Boele & van Eesteren	Residential	407 apartments
Consultants	Cauberg-Huygen	Commercial and offices	4800m ²
		Parking	500 places

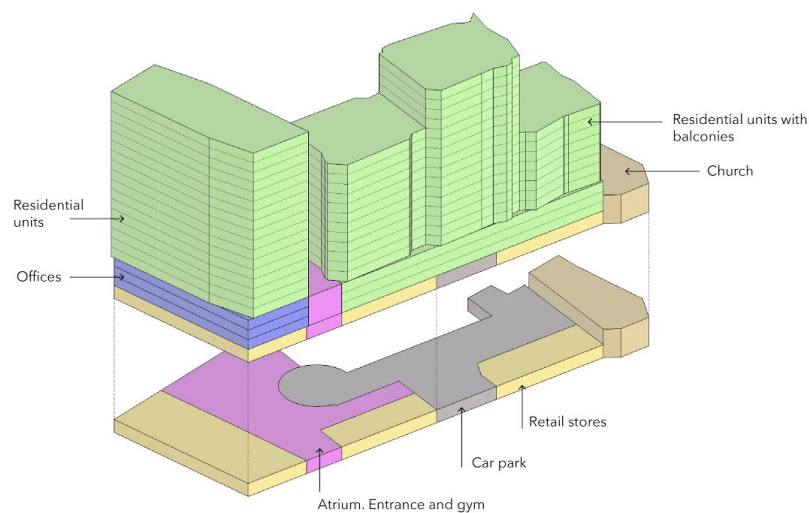
The project is composed out of four residential towers of different heights, united by a common plinth. The playful design of the façade, simulates faceted rocks that portray the idea of maximum housing differentiation.



Typical floor plan (Own elaboration)

The highest volume is tower A, with 22 storeys. The apartments are distributed around an inner corridor and a central core two central elevators. The fire stairs are located at the ends of the corridor. The other volume is composed out of three sub-towers, Tower B, 13 storeys, Tower C, 17 Storeys and Tower D, 9 storeys. The units are distributed parallel to an inner corridor and a central core with 3 elevators and 1 stairs. The emergency stairs are located at the ends of the corridor. Private balconies are irregularly shaped following the faceted shape of the building. In total there are 407 luxury apartments, with sizes ranging from 80 to 200m².

The buildings can be separated into three main volumes, north tower (with a more regular appearance), three united towers (more façade appearance and outdoor balconies) and a hybrid use plinth that unifies the development. The first four storeys of a plinth hosts 4800m² of retail stores and a supermarket in the first floor and offices in 2nd, 3rd and 4th storeys. Underground there is a car park that hosts 500 places. The main entrance is located in between the two main volumes, in a 4 storey atrium and a gymnasium. On one side of the plot, there is a new church, in replacement of the former St. Paul's church that was formerly located on the plot.



Schematic programme of the building (Own elaboration)

Conclusions

From the mentioned case studies of tall residential buildings in Rotterdam, some conclusions can be extracted regarding program distribution in this type of construction.

In opposition to a single storey house, tall buildings do not have direct connection to street level. Instead series of vertical and horizontal circulations lead the way to the apartments. Together with the mentioned exclusive views, this creates a feeling of separation. Whereas some residents, e.g. elderly, may experience that in positive terms (calmness and privacy especially in frenetic city centres), other users, e.g. young people, may consider it negative (isolation). Building plinth, lower sections of the building and in some case rooftops, are usually reserved for collective spaces, such as gym, outdoor terrace, atrium or swimming pool.

Living in a tall building in city centres, means exclusivity. All citizens of the city have access to the public space, and therefore street level views. However, only the few residents of a tall building can have the opportunity to contemplate the city from bird eye point of view. This creates a privilege that is further noticeable the more distinctive the view is compared to pedestrian level.

City centres are surrounded by public spaces and pedestrian routes that create the urban network of the city. New buildings should adapt to the existing context by enhancing the active street life in the city, with commercial and hybrid programmes in the plinth and lower parts of the building

Another characteristic feature for a tall building is the great number of housing units. From the brief historical view, it can be understood that, a continuous repetition in the same time of dwelling may lead to monotonous use of the building, with the same occupancy profiles, and in worst case can lead to deprivation of the building. On the other hand, increasing the variety of sizes and housing typologies in a tall building, offer more opportunities for customisable market and user demands (young, elderly, families, couple, divorced...) and creates a more dynamic atmosphere in the residential space.

PROGRAMME

Collective spaces

(Swimming pool, gym, atrium, outdoor terrace)

Commercial and multi-use plinth

Car-park above commercial plinth

FLOOR LAYOUT

Multiple types of housing units

studio, apartment, penthouse

(80-170m²)

Increase luxury towards the top

(Bigger apartments, window sizes, balconies)

PART 3

DESIGN STUDY - ARCHITECTURE

8. DESIGN STUDY (TTE) - ARCHITECTURE

- Existing building
- Timber concept
- Solar envelope
 - 21st December
 - 11 - 12.00 p.m. All-year
 - Combination Rotterdam policy
 - All-year multiple towers
- Programme

8.1 EXISTING BUILDING

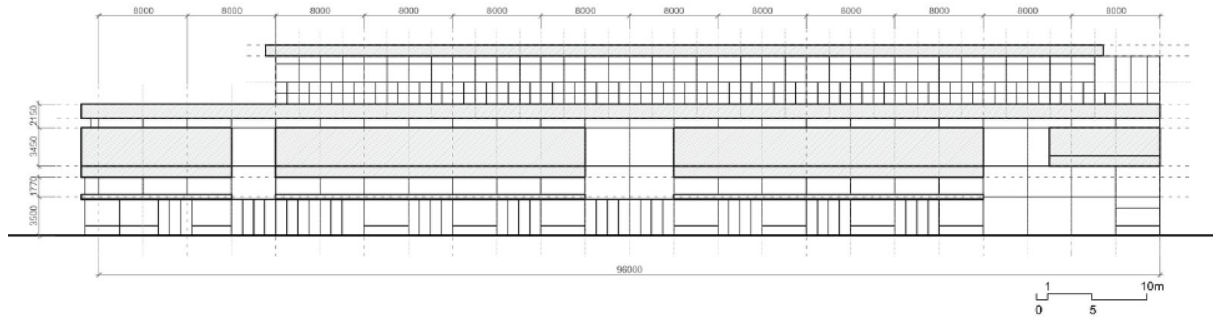


Historical photograph of Ter Meulen building in 1950s (left) and building condition in 2001 (right)

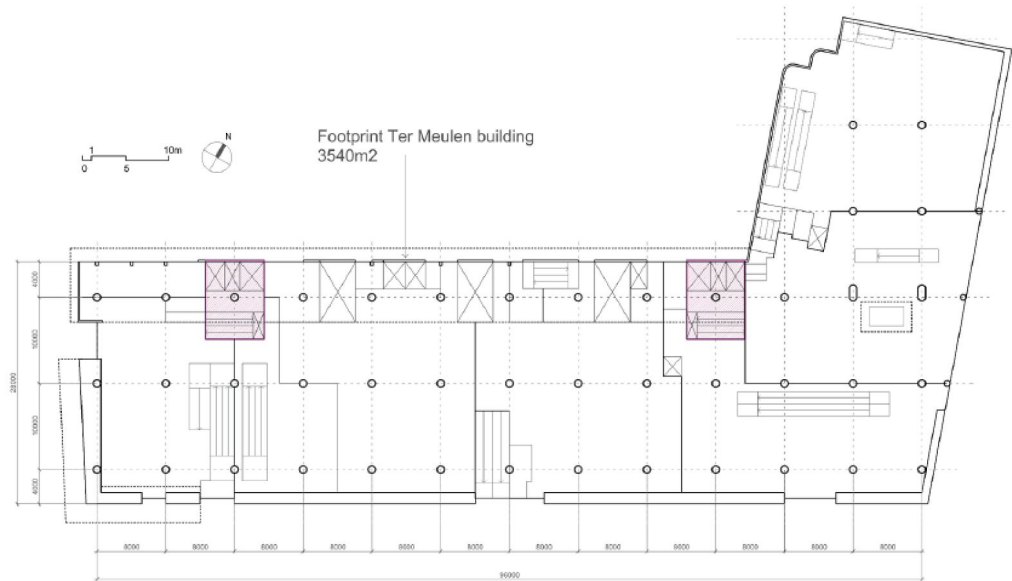
Location	Rotterdam	Construction	1948-1951
Architect	Van den Broek & Bakema	First extension	1970s
Client	Ter Meulen	Abandoned	1993

In 1951 Dutch architects Van den Broek & Bakema, designed Ter Meulen former department store. When it was opened, it had over four hundred employees, marking an important symbol of modernism architecture in Rotterdam. The building was originally designed with a flexible layout in order to host three retail stores, only separated by temporary lightweight walls. The façade was very transparent for the time, suggesting connection between inside and outside.

The basement and first floor were dedicated for commercial areas, the mezzanine level that could be accessed via escalator was used for restaurants, and in the second floor, offices and administration rooms were hosted. The building is next to Lijnbaan and Binnenweg, one of the oldest shopping areas in Europe. The second floor was intended to change use, and be transformed to commercial in a later stage, with one storey extension to be constructed. Therefore, the foundations and super-structure were calculated and dimensioned for that future adaptation. The layout of the building is open floor plan with concrete rigid frames of beams and columns taking lateral stability and vertical loading. In 1970s, two storeys, instead of the planned one floor, were added on top, of the original building, using a relatively light weight construction for the floors. In 1993, the company Ter Meulen went bankrupt and the building became was abandoned, leading to the architectural decadence in the following years.



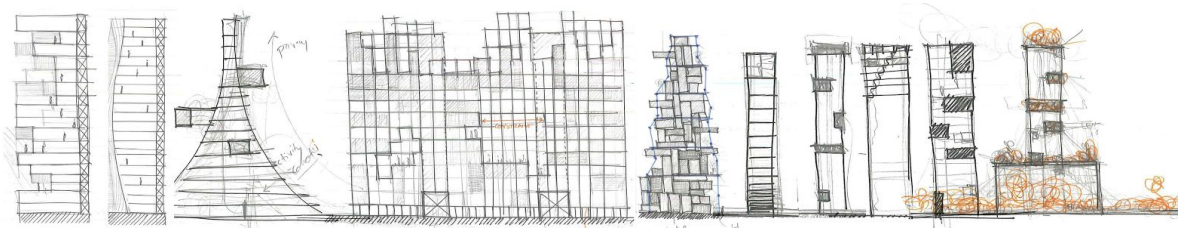
South elevation of Ter Meulen building in the original state (Own elaboration)



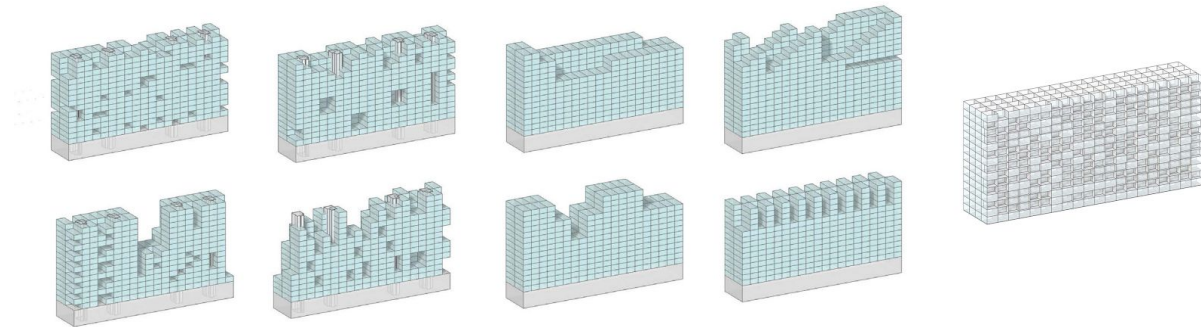
Basement and ground floor Ter Meulen building in original state (Own elaboration)

8.2 BUILDING MASS STUDIES

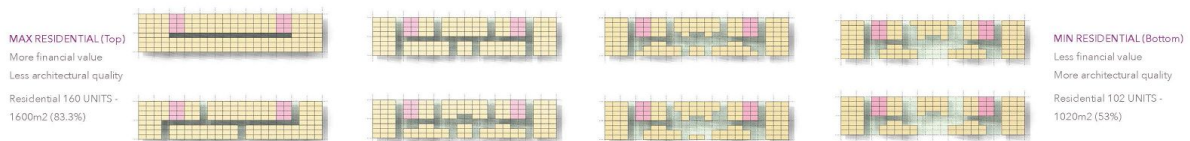
Silhouettes



Tri-dimensional arrangements of units



Interior distribution voids



HOUSING TYPOLOGIES

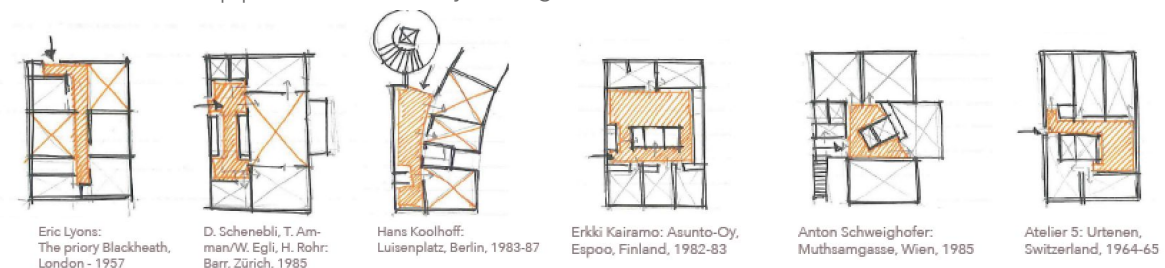
The floor plan idea expresses itself in the housing unit's internal organisation, whose possibilities and limitations derive from the opening and closing of spaces, their connections and groupings, the connection or isolation of functions, from the paths and the views.

The corridor type

The apartment is organised according to an axis along which the rooms are lined up on one or both sides. Especially important here is the endpoint of the axis, which in the best case, is the living room. If the corridor is too narrow and receives no natural light, the resulting impression will be an office hall. This can be counter-acted with links to living or dining room.

Inserted box

The apartment is visually interpreted as a large, open space with an inserted cube. These apartment appear much larger and more open than their actual dimensions would suggest. The inserted box, living room, kitchen or bathroom, divides up paths and allows day and night circulations.

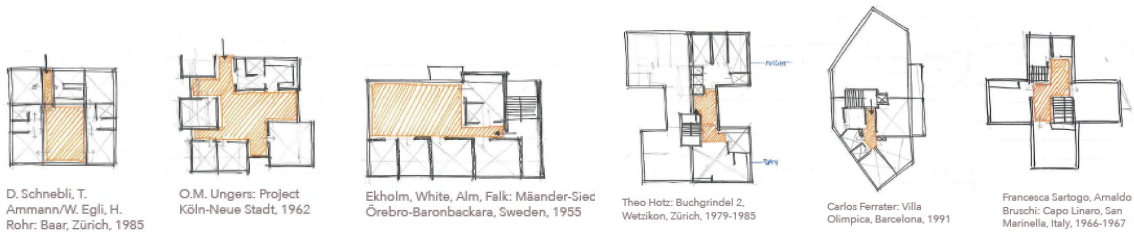


Living room as centerpoint

The floor plan develops around the living room; it is at once the centre and distributor, and almost all paths less through it. Area is added to the living room because of the associated reduction in hallway space, and even the individual rooms can be reduced for its benefit.

Clustering floor plan

The floor plan clearly separates the different functional areas inside an apartment: the living area with living room, kitchen, dining area; and the sleeping area with individual bedrooms and bath. Hobby rooms or workrooms can form a third area. Living and sleeping areas can also be separated by changing directions of the hallway.



The organic floor plan

This floor plan is based on a study of the paths of residents during different activities inside the apartment. The paths should be short, the pure hallways area minimal, and the spaces can flow into each other. One type is the flowing floor plan, where rooms flow into each other without the need of corridor.

Circuit floor plan

The constantly changing family situation (size, composition) and the increasing differentiation of housing needs are contradicted by the need for rationalization and standardization in the mass construction of housing. The favoured solution should be the conversion within the existing floor plan rather than a move to another apartment. The approaches range from modifiable external walls, modifiable internal walls within fixed outer walls. all the way to changing the form and size of the rooms by means of movable wall sections.

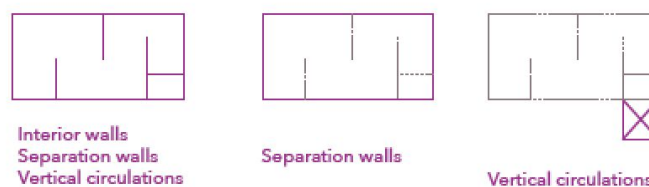


Flexibility of architectural elements in relation with structure

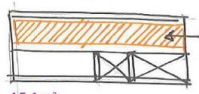
As analysed in the previously studied examples, architectural elements can be part of the load bearing system of the structure or can be substructure for architectural purposes. From these, three categories of flexibility can be extracted, namely interior walls, separation walls and vertical circulations contribute to the super structure, separation walls, and vertical circulation cores. Practice shows that most residents later tend to shy away from the expense of repositioning a wall and prefer participatory planning instead. Buildings and housing types are still experienced as something static in which one settles and to which one adapts. Today flexibility is restricted to apartment size. (Leupen 1999)

Housing differentiation

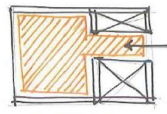
Demographic, social and cultural changes led to increasing dissatisfaction with the standard family dwelling. Youth housing, ageing population and a growing number of broken families called for smaller and more flexible dwellings. Differentiation is also an antidote against social insecurity. Nowadays, housing associations and developers no longer think in terms of 2, 3, or 4 rooms dwellings but try instead to cater all life-styles.



Studio 15-30m²



15.1m²
Murado, Elvira and Krahe
My space student housing
Trondheim, Norway, 2009 - 2011



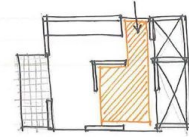
24.6m²
Atelier Zündel and Cristea
Residence
Paris, France, 2007 - 2010



25.7m²
3box
Social housing in the park
Paris, France, 2009

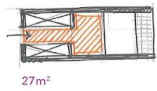


37.5m²
Esty Ilgaev & Michael Peled
Transformation of ha'aliya market
Tel Aviv, Israel, 2008

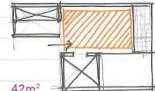


41m²
Alleswirdgut Architektur + feld72
Herzberg public housing
Vienna Austria 2007 - 2010

1 bedroom 30-70m²



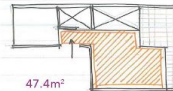
27m²



42m²

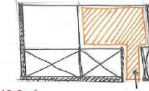


36.1m²



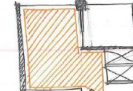
47.4m²

Atelier Data + MOOV
Dallas, United State, 2009 - 2012



49.2m²

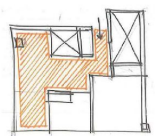
Raldon + Berengué
Coats Factory, Barcelona, 2008



67.7 m²

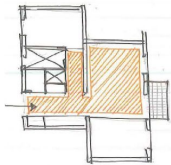
OMA, Metacity
Rotterdam, The Netherlands, 2009

2 bedrooms 70-115m²



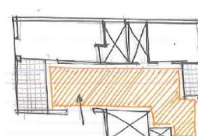
69.5 m²

REX, Mixed Use Low2No
Helsinki, Finland, 2009



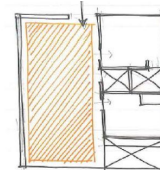
89.8 m²

AllesWirdGut Architektur + feld72
Herzberg public housing
Vienna, Austria 2007 - 2010



114.3 m²

BIG + Topotek1
Warehouse hybrid
Hamburg, Germany, 2009 - 2013



144 m²

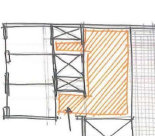
A-lab
Multifunctional stackable modules
Sandvika, Norway, 2008

3 bedrooms 90-150m² (left)/ 4+ bedrooms 90-170m² (right)



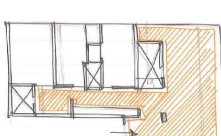
93.8 m²

REX, Mixed Use Low2No
Helsinki, Finland, 2009



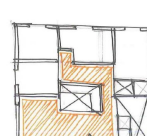
98.2 m²

Atelier Data + MOOV
Dallas, United State, 2009 - 2012



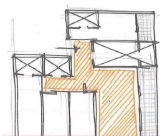
264.1 m²

La Dallman Architects
Macarthur square hybrid
Milwaukee, United States, 2007



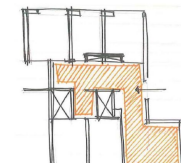
65.8 m²

3box, Social housing in the park
Paris, France, 2009



105.8 m²

Borja Peña
Toledo housing block
Toledo, Spain, 2009-2011



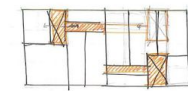
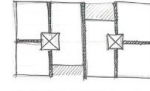
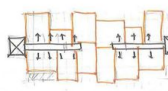
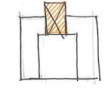
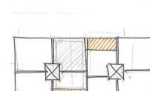
174 m²

Müller Sigrüst Architekten
Residential and commercial buildings
Kallbreite, Zurich, Switzerland, 2009 - 2014

Circulation arrangements

An efficient distribution in a tall building, also implies access to vertical circulation cores. In a simplistic way, we can distinguish two typologies, linear distribution along each storey, or arrangement of the units around central cores. In addition, many variants can be added, such as interior/exterior corridor, or core, size, etc.

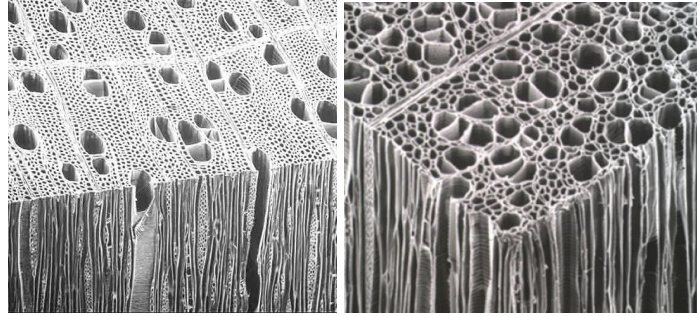
Needless to say, those parameters are highly affected other multiple considerations, such as shape of the building, user demands, architectural ambitions, or structural demands.



8.3 TIMBER CONCEPT

Wood microstructure

Wood is a natural material composed out of microtubes or tracheids glued together. Their functions are both to conduct water and provide structural support for the tree. They are approximately 2-4mm long, and roughly 30 μm wide. In general, the pattern composition and sizes depend on natural factors such as wood specie, soil conditions, climate or season-growth patterns. This cellular micro-structure defines most of the characteristics of the material, appearance, mechanical properties, durability, etc.



Different cellular microstructures of wood. (Source: Classes.st.edu)

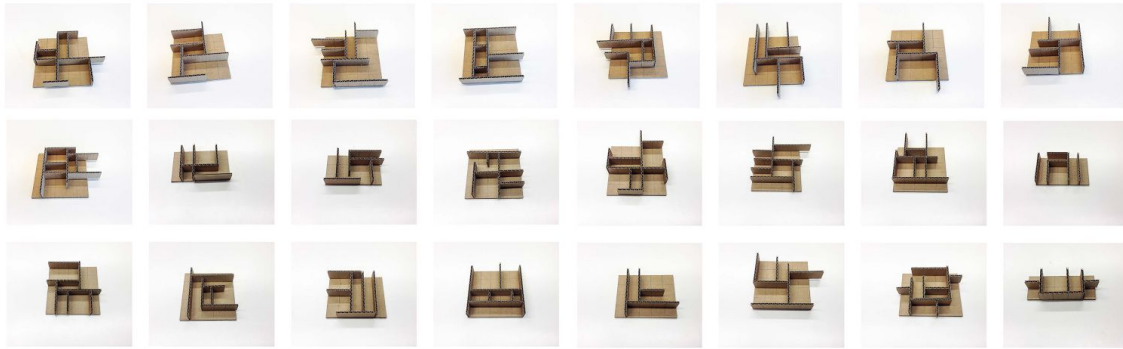
Engineered timber macro-structure

With the use engineered timber, a new dimension of the material can be achieved, probably with less natural sensations, but higher possibilities for prefabrication and consequently the creation of complex shapes and patterns. The beauty of natural cellular patterns has been the inspiration for many architects in the recent years. Examples of tridimensional lattices or gridshells can be found in architecture projects worldwide. This suggest the application of the cellular microstructure of wood in a bigger scale with the use of more advanced technologies, such as engineered timber, glulam, LSL, or CLT. Metro Parasol in Sevilla (Spain) by Jürgen Mayer is one example of this re-interpretation of wood microstructure in architectural projects.



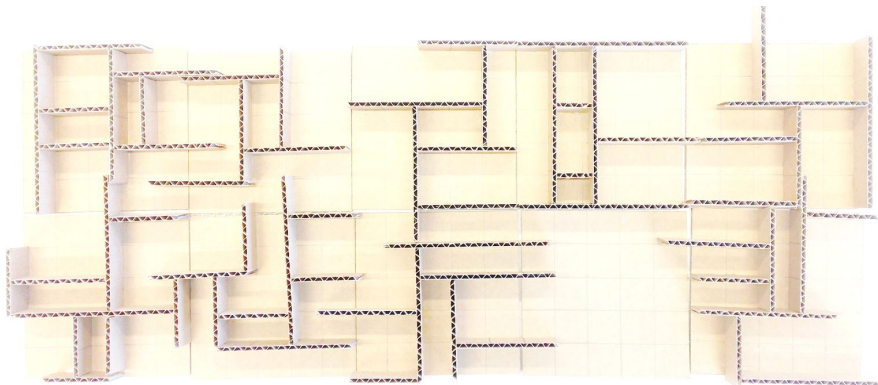
Metro Parasol, Sevilla (Source: Jürgen Mayer)

These profound differences in perception between timber and other traditional materials, should determine a different treatment regarding tall building structures. In a way, living inside a timber structure can be compared to living inside the art of nature.

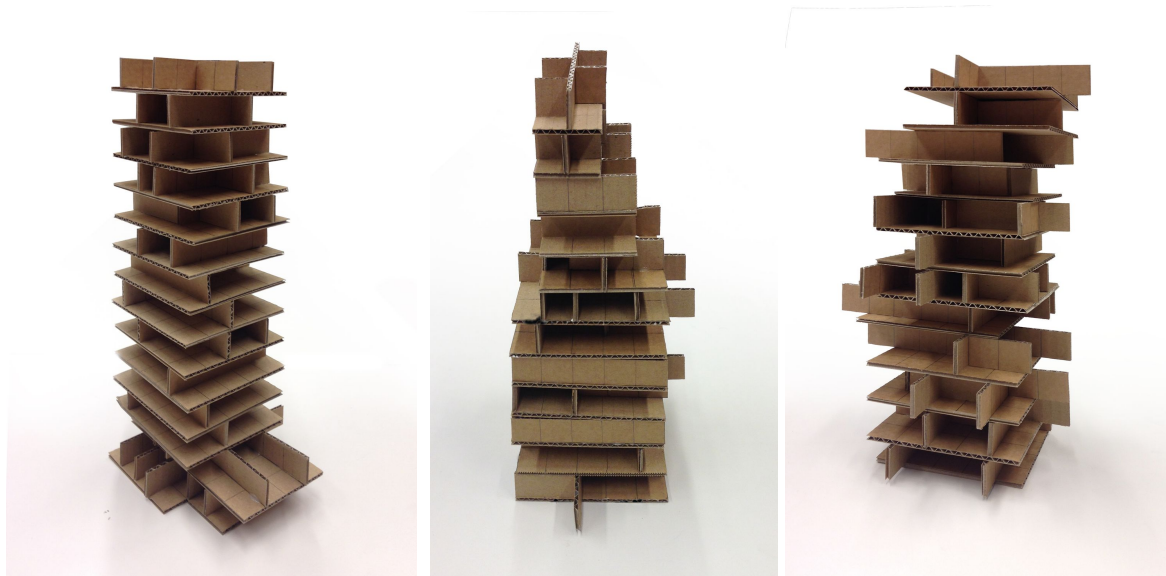


Model experimentation with different cellular patterns

With the intention of quickly perceiving spatial effects. and quickly create different patterns for cellular structure, a series of physical models was done.

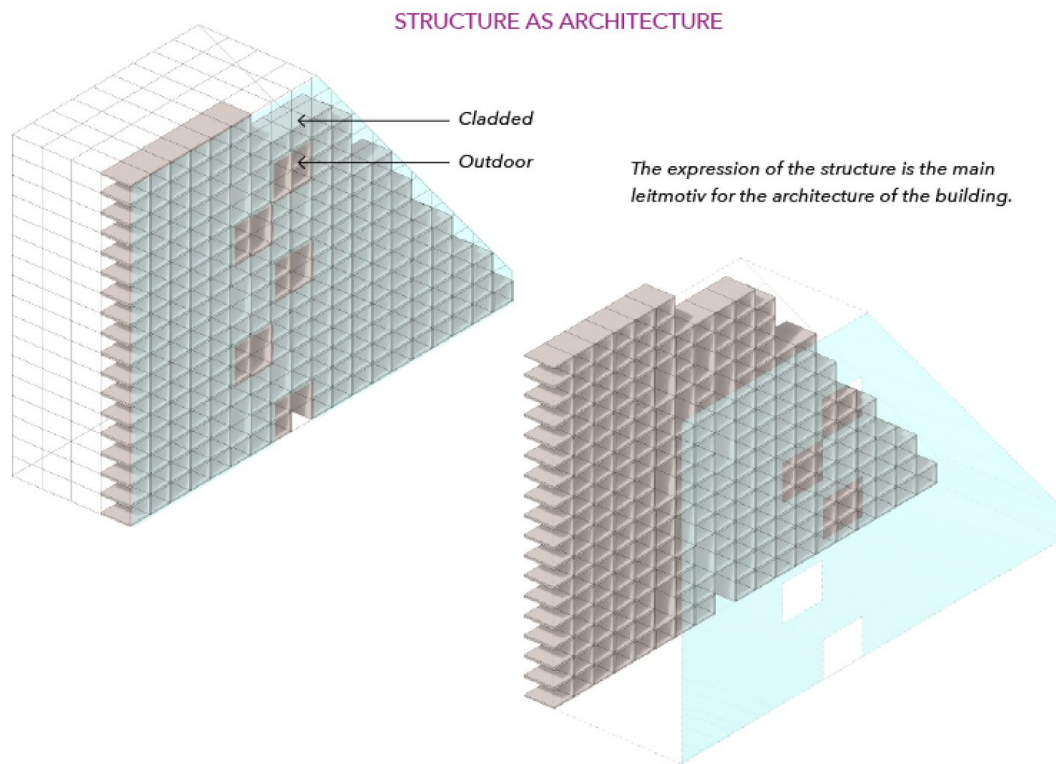


Lattice composition of CLT panels

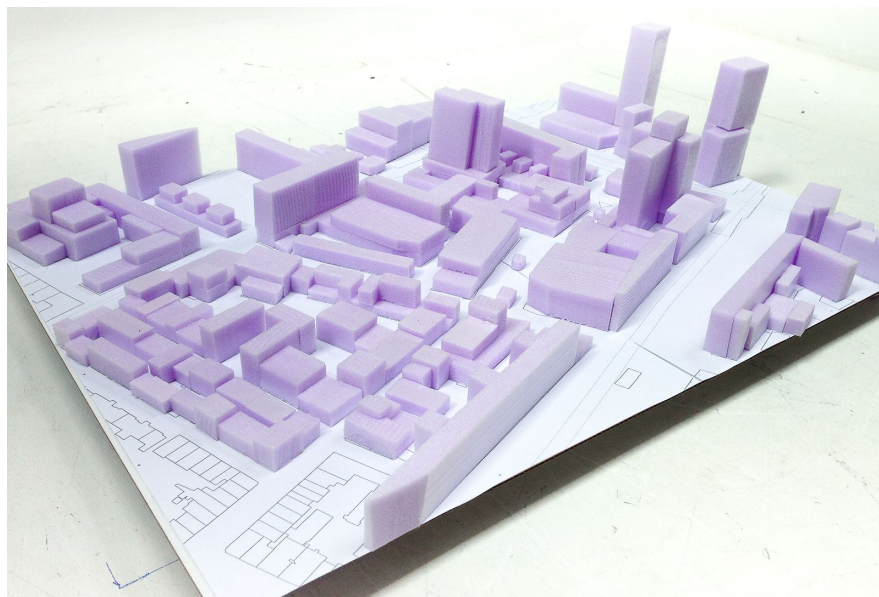


Different configurations for vertically stacking cellular modules

- The TTE, Tall Timber Extension should evidence the microstructure character of wood by making a structure with composition of cellular-lattice pattern. At the same time, the distinctive characteristic of the structure should be the main aspect of the architecture of the building, and be maximally exposed to the perception of the users, inside and outside.



The expression of the structure is the main leitmotiv for the architectural project

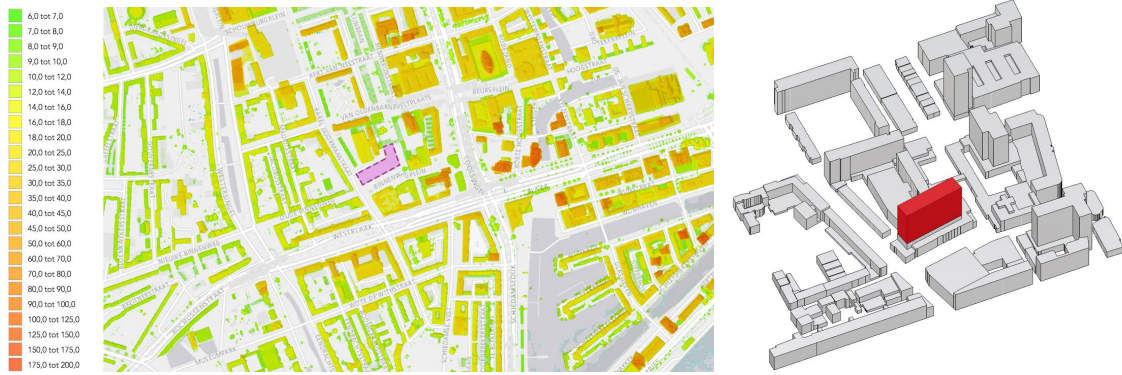


Model of the context (Own elaboration)

8.3 SOLAR ENVELOPE

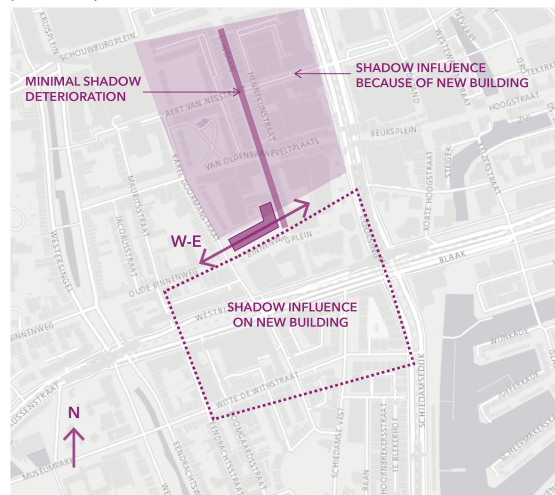
Solar rights of surrounding buildings

The orientation of the plot is East-West, which is ideal for incoming sunlighting incidence. In the image, we can see that there are not high-rise buildings located in the immediate south of the plot. This means that there is minimal obstruction of direct sunlight towards the future new construction.



Left, heights map and location (Source: Actueel Hoogtebestand Nederland). Right, optimal design volume regarding only incoming sunlight (Source: Own elaboration)

However, the construction of a tall building within an existing context may have also shadow influence in the surrounding buildings and public space, especially in highly densified zones. According to Rotterdam policy regulations, Lijnbaan street is considered of spatial quality for the city, which means that minimal shadow deterioration is allowed in the public space.



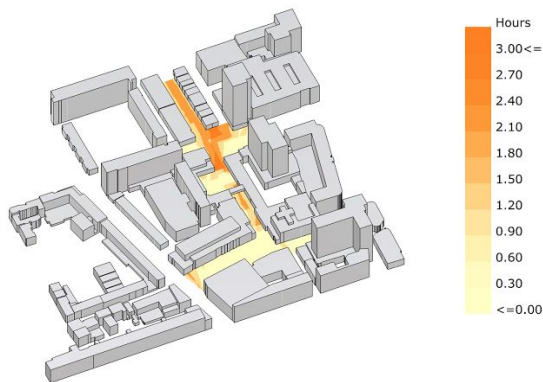
Scheme with areas of direct sunlight influences for new building and existing buildings, left, and (Own elaboration)

- The optimal design volume only regarding incoming sunlight is a tall disk.
- Minimal shadow deterioration of Lijnbaan street

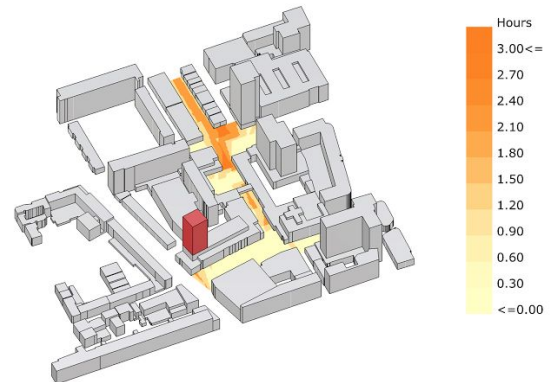
21st December

In order to test the shadow impact of a new tall building in the designated location, a preliminary shadow analysis was performed in Ladybug + Grasshopper software. The model was tested for the most restrictive situation, 21st December winter, a schematic mass volume of 20x20 meters base and 70 meters height placed in different positions within the existing plot. Only Lijnbaan and Binnenwegplein streets were tested, because of the special quality according to municipal policy and vibrant urban activity.

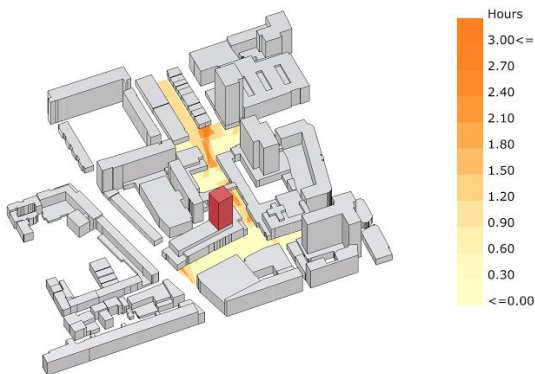
SUNLIGHT HOURS ANALYSIS
FOR PEDESTRIAN PUBLIC SPACE
Date: 21st December
Context: Lijnbaan & Binnenwegplein
Tested area: 15168 m2.



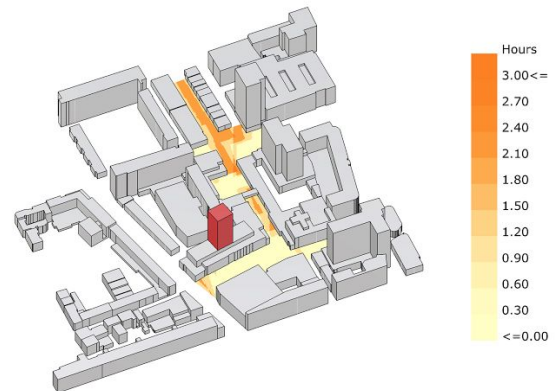
CONTEXT >1h sunlight : 34%
>2h sunlight: 16%



SCENARIO 2 >1h sunlight : 34%
>2h sunlight: 16%
- Most optimal position -



SCENARIO 1 >1h sunlight : 31%
>2h sunlight: 5.7%
- Less optimal position -



SCENARIO 3 >1h sunlight : 32%
>2h sunlight: 14%

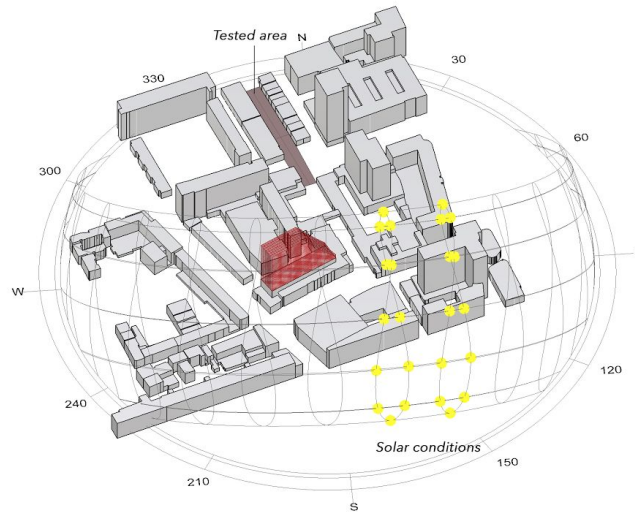
From the analysis, we can conclude that Scenario 2 and 3, are the most optimal design volumes regarding shadow impact in Lijnbaan street. It can also be concluded that an increasing height of the building towards the west side of the plot, may be also a suitable design option.

It is also appreciated that the degradation of shadow, (considering December 21st, the lowest position of sun) mostly occurs in the north part of the tested surface in Lijnbaan street.

11-12.00 p.m. All-year

Maximum solar envelope for no shadow degradation Lijnbaan street

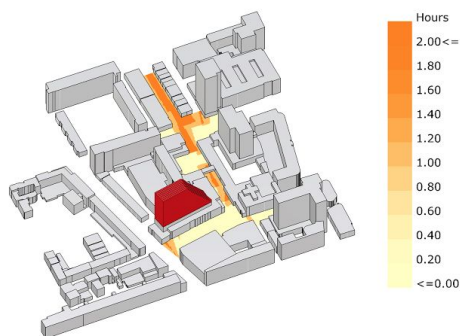
11.00 & 12.00 p.m. All-year



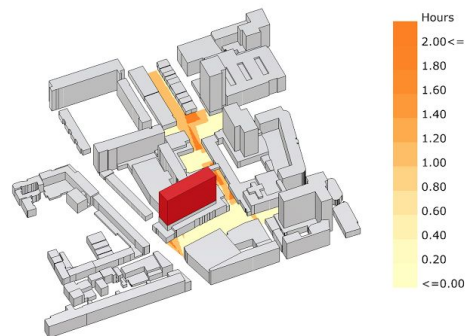
Maximum volume within the existing plot, for ensuring 2 hours sunlight in north part of Lijnbaan street. (Own elaboration)

A new model was created in order to obtain the maximum design volume without degradation of the public space sunlight. For the simulation, only the north part of the Lijnbaan street was tested (as this is the only part that still retains 2 hours sunlight per day). The model was calculated for 11.00 and 12.00 pm, the time when sun higher and has warmer temperature, impulsing the use of urban space.

SUNLIGHT HOURS ANALYSIS
FOR PUBLIC SPACE
Date: 21st December



SCENARIO 5 >1h sunlight : 31.3%
>2h sunlight: 12.6%
- Max volume for 2 hours sunlight -



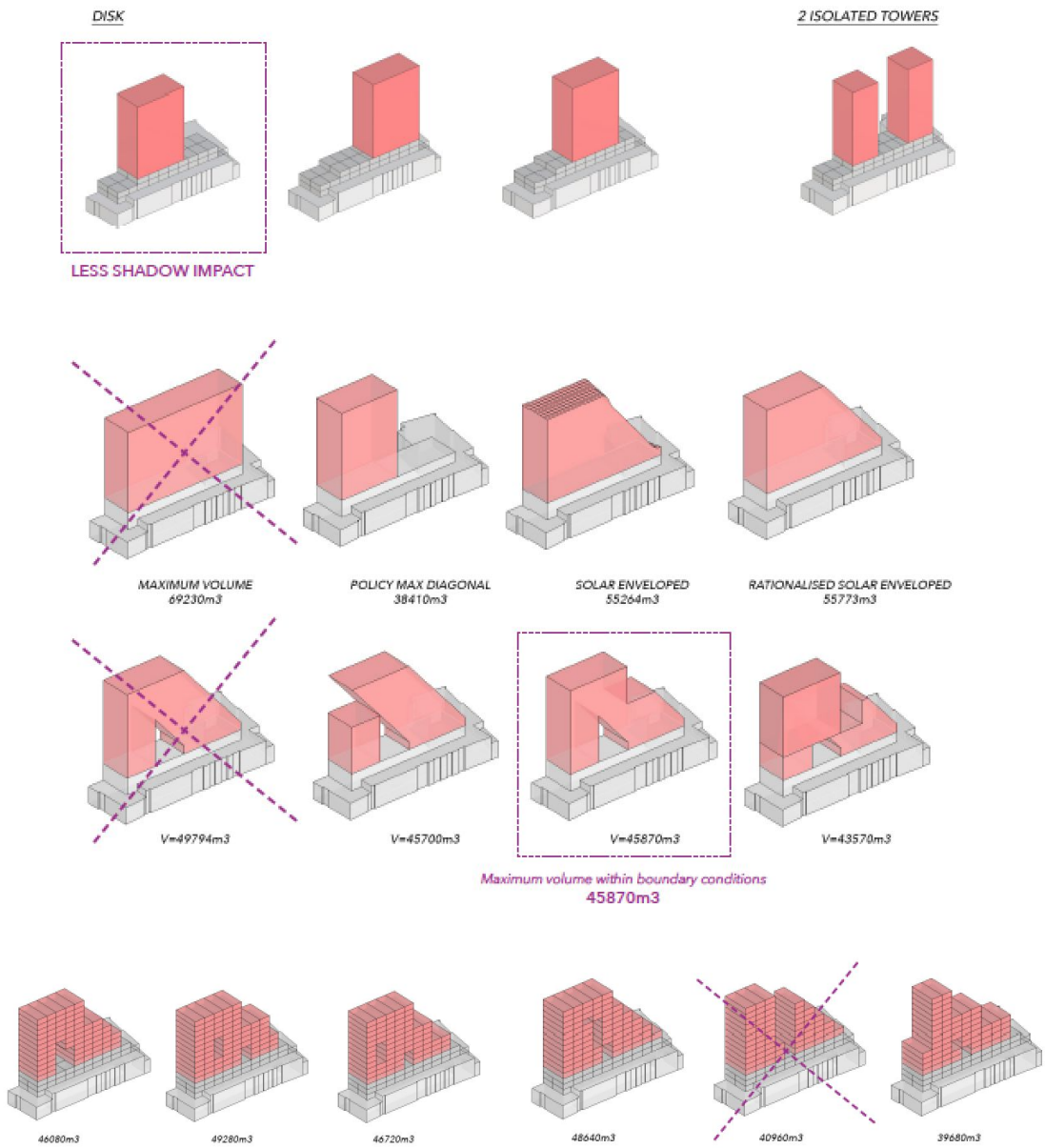
SCENARIO 2 >1h sunlight : 29.7%
>2h sunlight: 4.9 %

Comparison maximum solar envelope and maximum volume (Own elaboration)

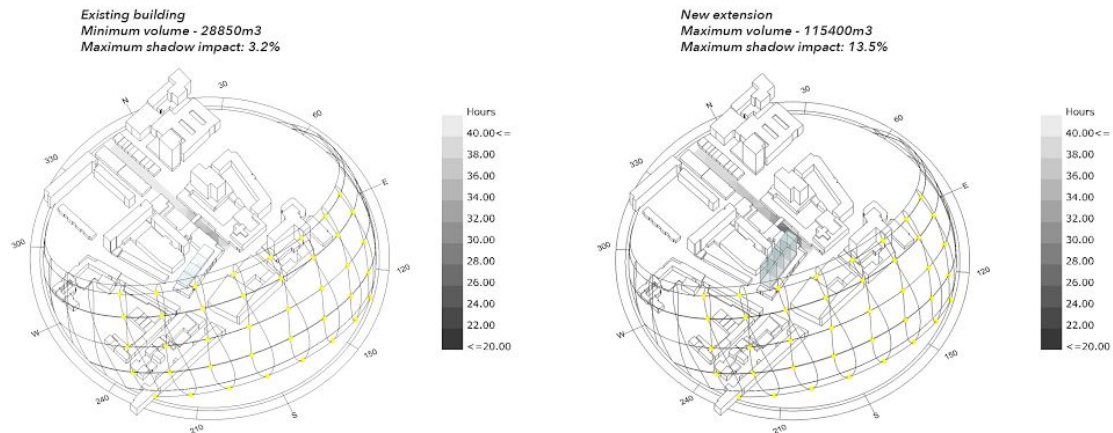
Combination with Rotterdam policy

The below images summarise different mass studies explored, giving the following constraints:

- 50% maximum volume above plinth (Rotterdam high-rise policy)
- Maximum 42 meters depth diagonal (Rotterdam high-rise policy)
- Minimum shadow impact on surroundings (solar analysis)
- Adaptation to structural grid of existing building



All year - Multiple towers



Comparison of maximum extremes, volume and shadow impact (Own work)

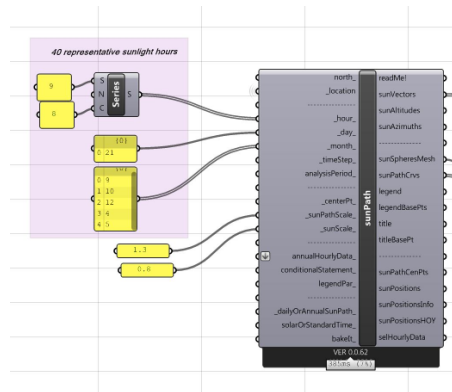
In the above image, we can see the main volume constraints maximum and minimum constraints, regarding building height limitations, minimum height of the existing building 20 meters, and maximum height allowed, 80 meters.

The computational workflow was done in Ladybug, Grasshopper, Galapagos/Octopus softwares. The main objective is to maximise the total average number of sunlight hours received by tested surface without new construction divided by number number of sunlight hours received by tested surface with new construction.

In order to simplify the design process and reduce computer calculation time, it is assumed that the present context does not have any shadow influence on Lijnbaan street.

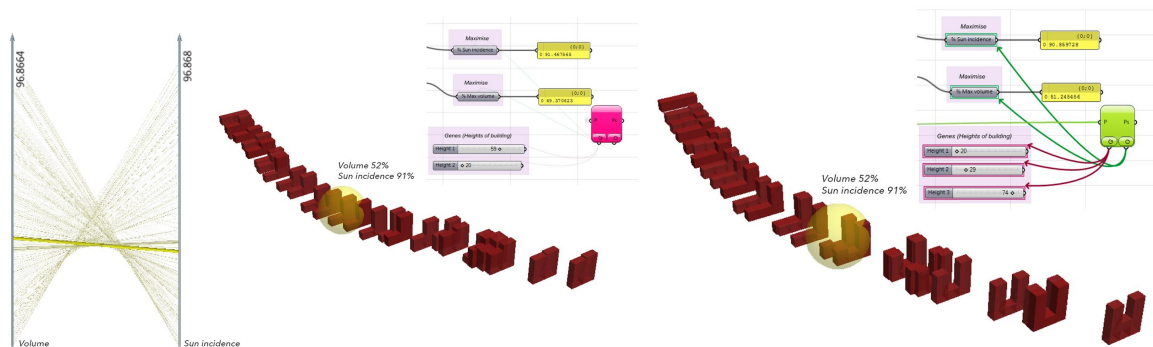
Representative sun positions

For a representative calculation of sunlight hours throughout the year, and reduce computer calculation times, 40 representative sun positions have been used in the sunlight analysis. September, October, December, April and May, on the 21st day of each month, and during 8 hours per day, (09:00-16:00)



40 representative sunlight hours input in Ladybug (Own work)

Needless to say, there is an apparent design conflict between two parameters, the building mass and the inflicted shadow, or sunlight hours on Lijnbaan street. In order to better comprehend that conflict, several multi-objective optimisation sequences were performed with the software Octopus, for creation of different sub-volumes with height ranges between 30-80 meters, searching for the optimal solution, regarding both maximum sunlight and volume.



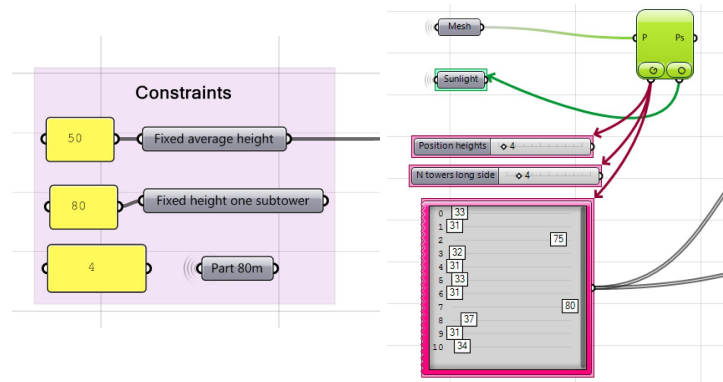
Pareto front solutions for 2-3 sub-volumes on building site (Own work)

Optimisation criteria

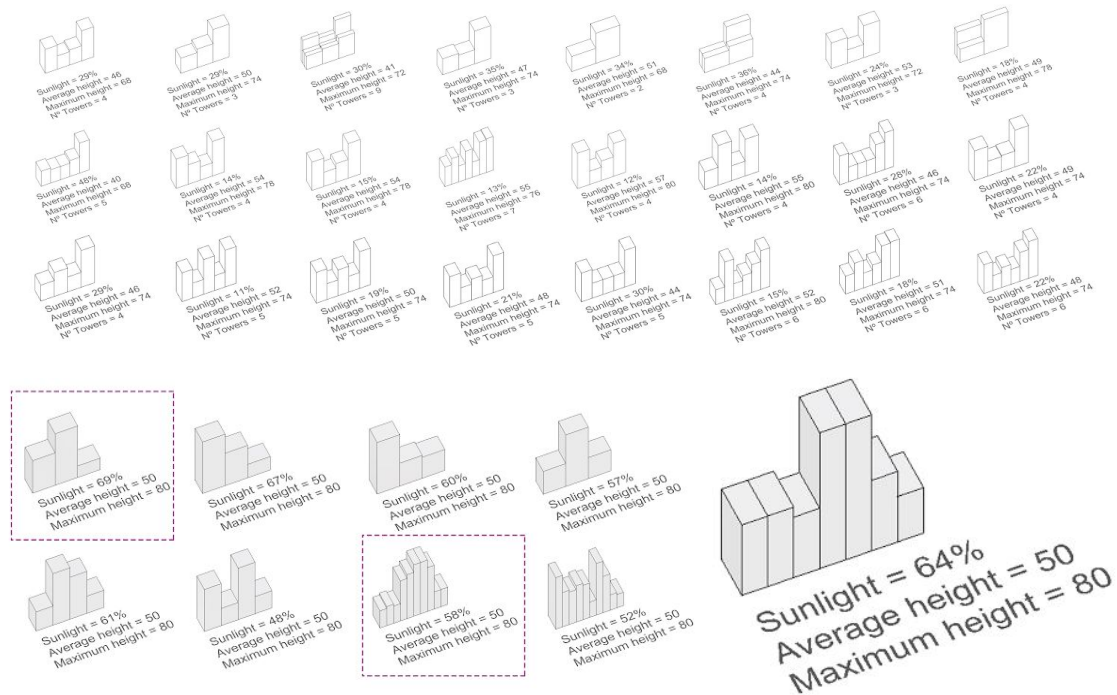
50 meters average height (Addition of sub-towers heights divided by number of sub-towers)

80 fixed maximum height of at least 1/4 of the sub-towers. 1/4 sub-towers fixed maximum height: 80m

Maximise sunlight hours on Lijnbaan street



Definition of parametric constraints and optimisation criteria (Own work)

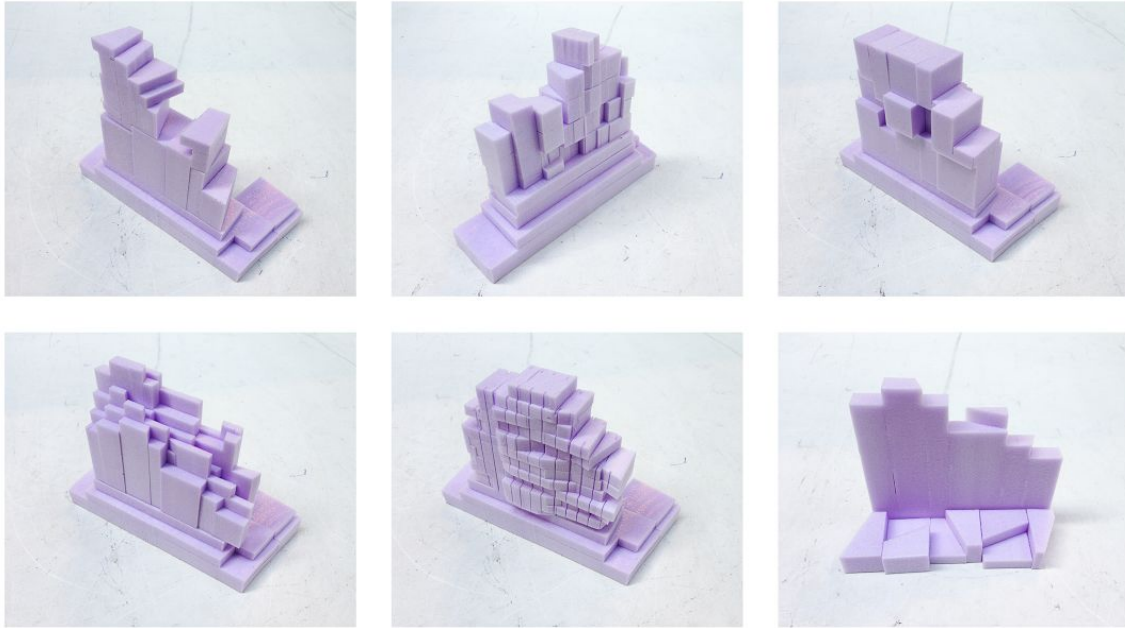


Selected "optimal" shapes for maximum average height and sunlight on Lijnbaan street (Own work)

SCULPTURE

"Landmark"

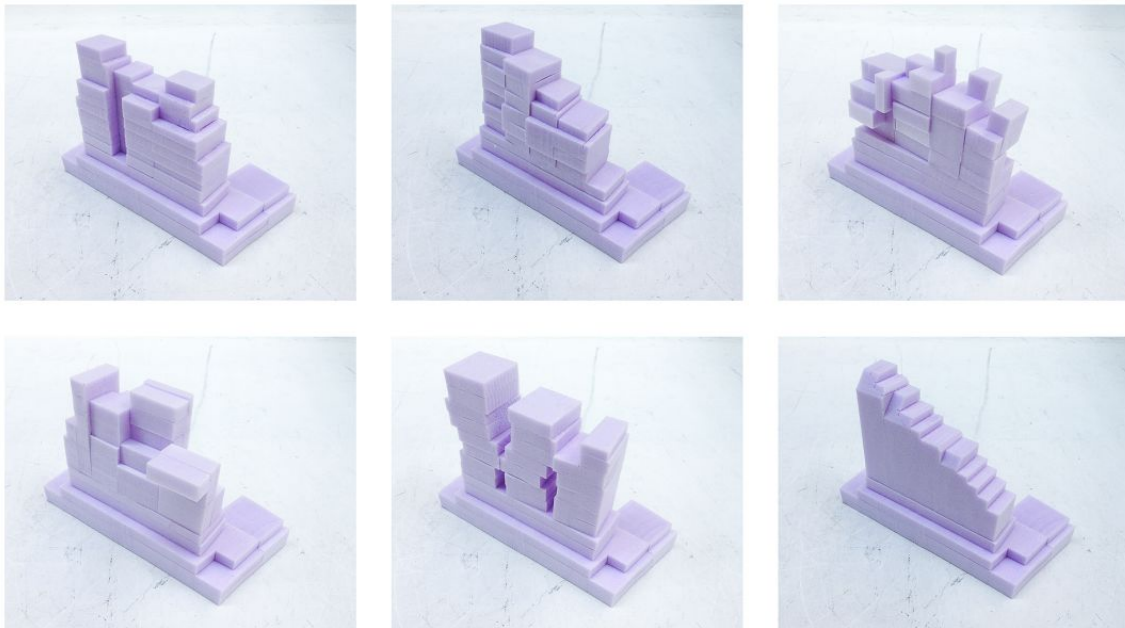
Due to the multiple high-rise buildings in Rotterdam city center, new high-rise buildings aim at achieving a landmark character that differentiates from the others. It is not in the scope of this study to discuss possible benefits or drawback that may come from this position, as well as other economic or architectural characteristics derived from that.



Formal study for landmark designs in the selected context (Own elaboration)

"Stacked boxes"

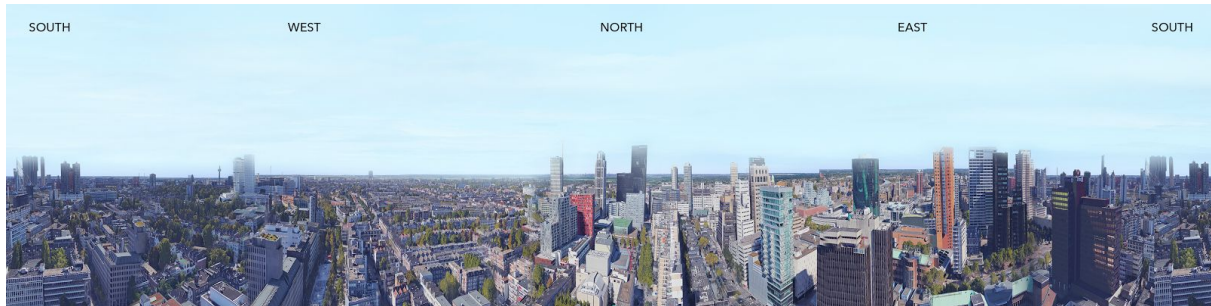
Another formal design strategy, may be the repetition of several boxes, containing the residential units or other building programmes in different sizes. This approach is more rational in terms of structural and economical grounds. At the same time, it helps creates functional floor plans and optimal distributions.



Stacked boxes design options (Own elaboration)

"Rooftop landmark"

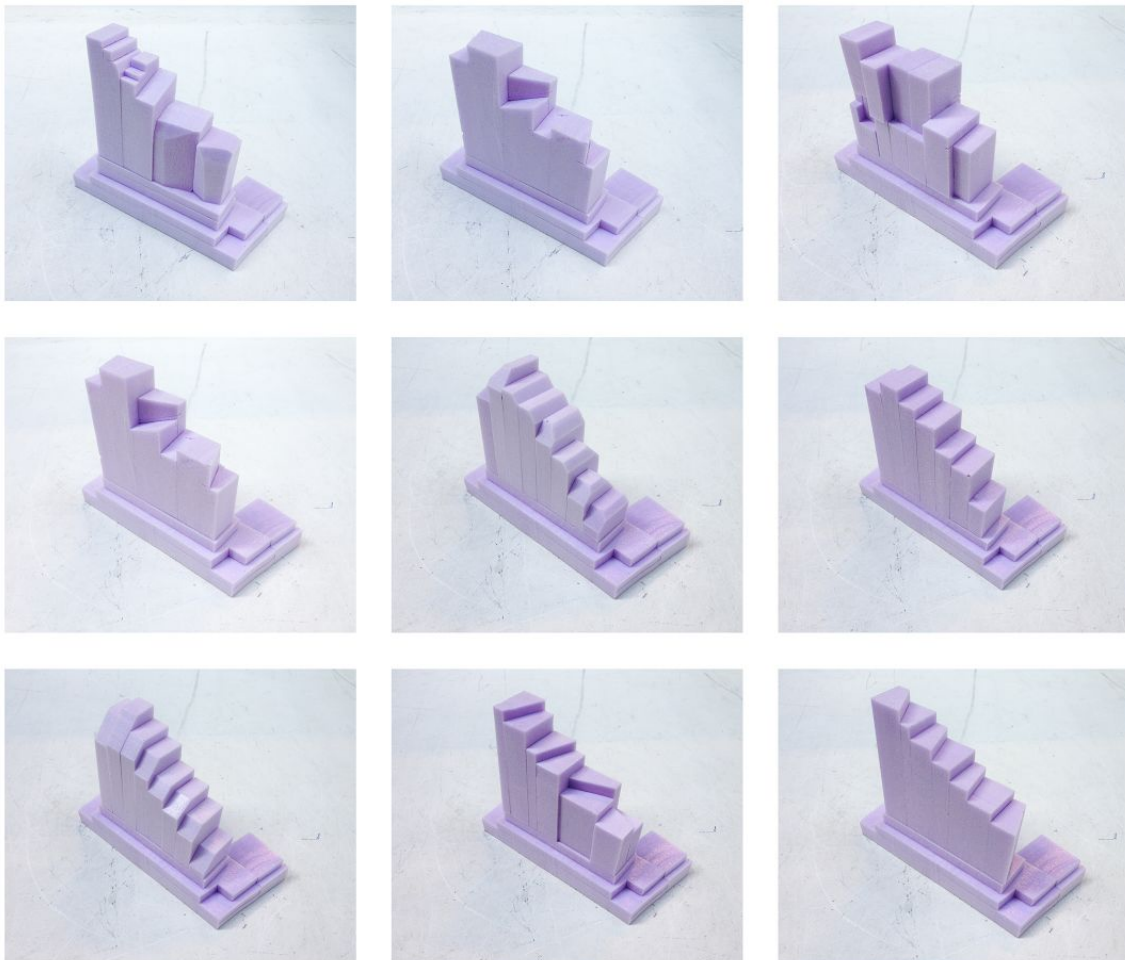
Living in a tall building is usually associated with the privilege of having unconstrained views over vast distances. This is often one of the most characteristic features that it is used for marketing a new high-rise building. Below, it can be seen a 360° panorama view in the specific context.



Composition 360° panorama views in intended location (Own elaboration)

- Towards south-west, some of the most distinctive landmarks are Mass River, Erasmus Bridge, De Rotterdam, Erasmus MC and Euromast.
- Towards north-east view, there is a wide variation of tall towers with residential or offices uses. Another distinctive feature in the urban level is the pedestrian artery Lijnbaan street, that defines a clear artery for the urban landscape.

In order to exploit the unique feature of the high views over the city, a series of rooftop studies were made, with the intention of providing rooftop terraces for the residents of the building.



Series of formal studies for rooftops of the tall building (Own elaboration)

PROGRAMME

From the mentioned case studies of tall residential buildings in Rotterdam, some conclusions can be extracted regarding program distribution in this type of construction.

In opposition to a single storey house, tall buildings do not have direct connection to street level. Instead series of vertical and horizontal circulations lead the way to the apartments. Together with the mentioned exclusive views, this creates a feeling of separation. Whereas some residents, e.g. elderly, may experience that in positive terms (calmness and privacy especially in frenetic city centres), other users, e.g. young people, may consider it negative (isolation). Building plinth, lower sections of the building and in some case rooftops, are usually reserved for collective spaces, such as gym, outdoor terrace, atrium or swimming pool.

- The TTE, tall timber extension should provide multiple collective spaces with rich variety heights, different orientations, indoor/outdoor, and abundant greenery.

Living in a tall building in city centres, means exclusivity. All citizens of the city have access to the public space, and therefore street level views. However, only the few residents of a tall building can have the opportunity to contemplate the city from bird eye point of view. This creates a privilege that is further noticeable the more distinctive the view is compared to pedestrian level.

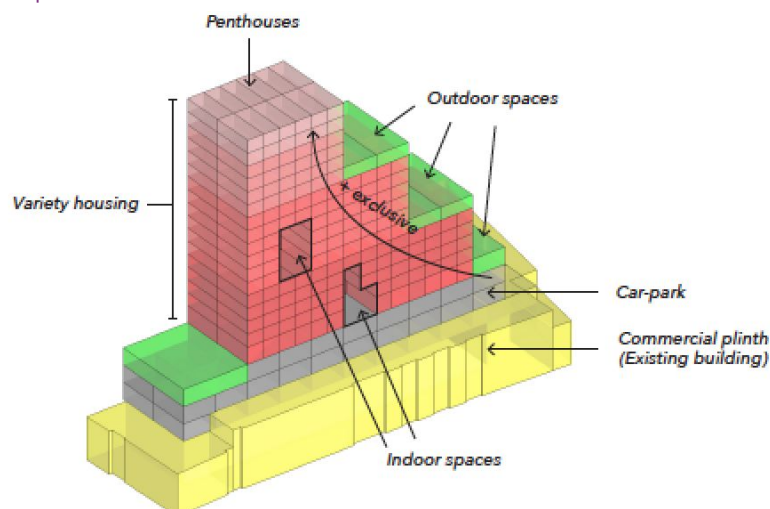
- The TTE, tall timber extension should have a gradation between ordinary to exclusivity, that is associated with the view. This translates to bigger apartments, window sizes and balconies towards the top

City centres are surrounded by public spaces and pedestrian routes that create the urban network of the city. New buildings should adapt to the existing context by enhancing the active street life in the city, with commercial and hybrid programmes in the plinth and lower parts of the building

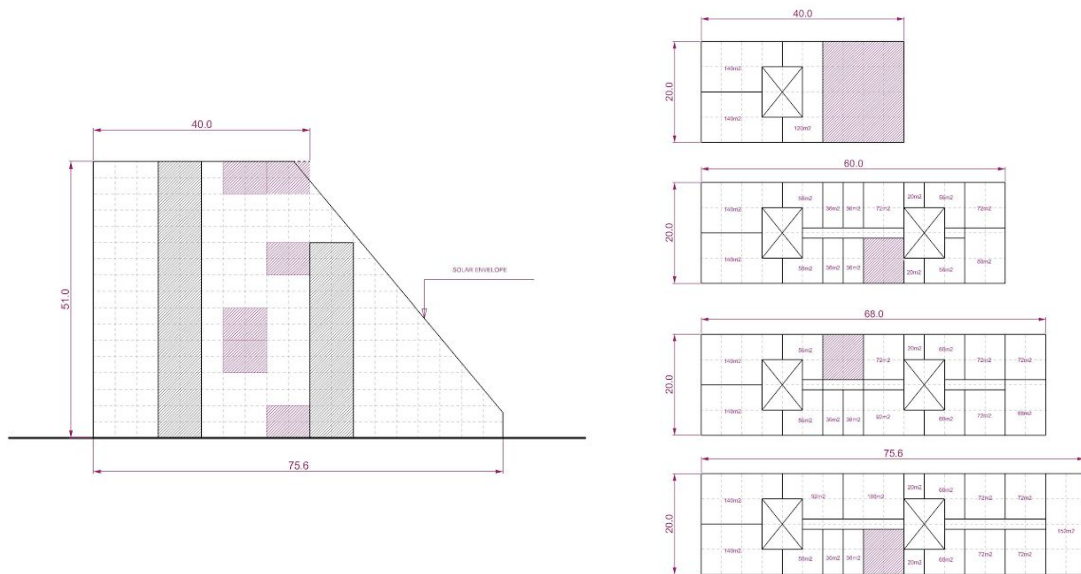
- The TTE, tall timber extension, should create urban attractive programme on the ground level, such as retail stores, cultural programme, or multi-use plinth. In addition, all utilitarian functions such as car-park or technical installations should be located above street level.

Another characteristic feature for a tall building is the great number of housing units. From the brief historical view, it can be understood that, a continuous repetition in the same time of dwelling may lead to monotonous use of the building, with the same occupancy profiles, and in worst case can lead to deprivation of the building. On the other hand, increasing the variety of sizes and housing typologies in a tall building, offer more opportunities for customisable market and user demands (young, elderly, families, couple, divorced...) and creates a more dynamic atmosphere in the residential space.

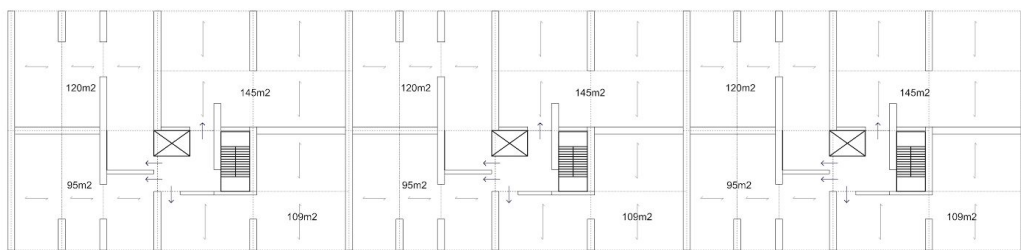
- The TTE, tall timber extension should include a high variety of housing units, (range of studios, apartments and penthouses), with different orientations and configurations, in order to be attractive to a wider social spectrum.



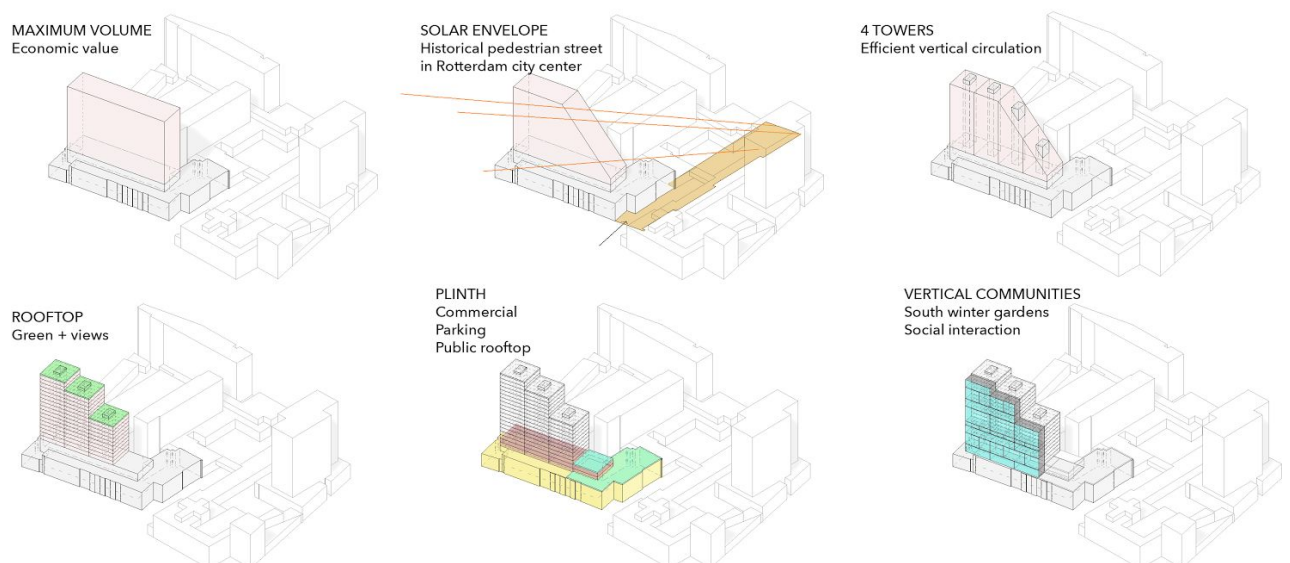
Programme demands for tall residential building in Rotterdam city centre



Variation of floor layouts along the height of the building with two main vertical circulation cores



Example floor plan with three vertical cores and four housing units per core



Main architectural design features of the TTE (Own elaboration)

Vertical social community

Often in tall buildings, the sense of community amongst the occupants is challenged by the fact that you hardly ever meet your neighbours, except coming and going via the lobby and at the lifts. In contrast, this design proposes a collective community, where social interaction is enabled and encouraged in numerous ways without compromising the need for privacy. The proposal contains a large selection of apartments reaching out to a diverse group of inhabitants, from small types suitable for student co-housing to larger family and live-work types, all grouped into vertical mini-communities.

Each group of similar apartments opens towards balcony spaces, creating "vertical mini-communities." Through balconies, glass winter gardens and roof terraces. The building incorporates an "inside-out perspective, where the social qualities of the building are a dominant driver for the design.

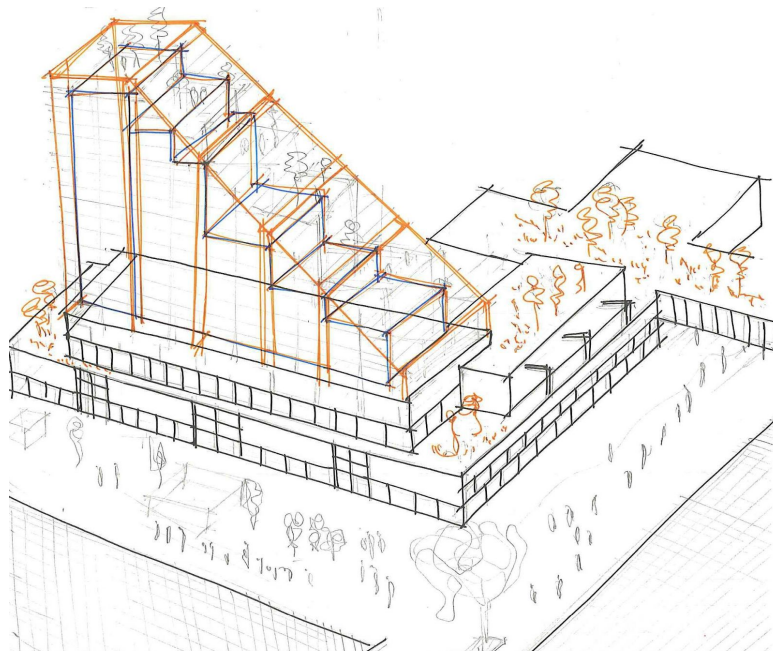
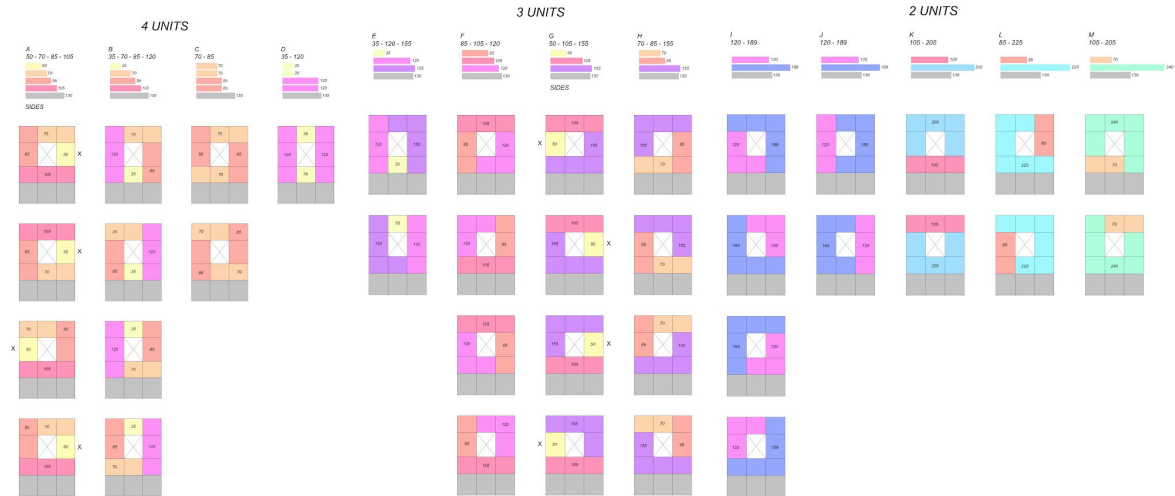


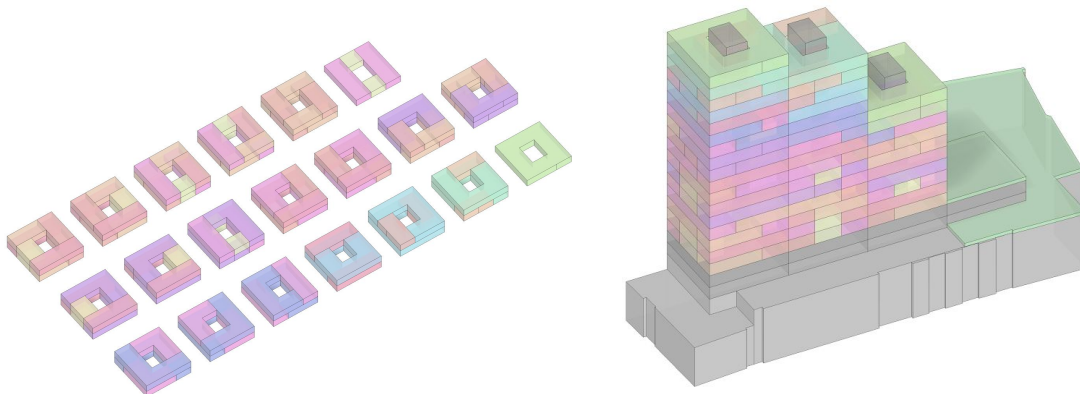
Illustration of design study in the context (Own elaboration)

CLUSTER OF APARTMENTS

Clustering of different housing units around a vertical circulation core. In every cluster, all units have to be different sizes. Every unit has to face North or South for allowing enough daylighting inside the apartment. This rule is considering the connection side connection with other clusters of units.



Combination of different combination of units per core (Own elaboration)



Pairing of the housing units with increase luxury and size towards the top (Own elaboration)

Location shear walls in housing clusters around central core

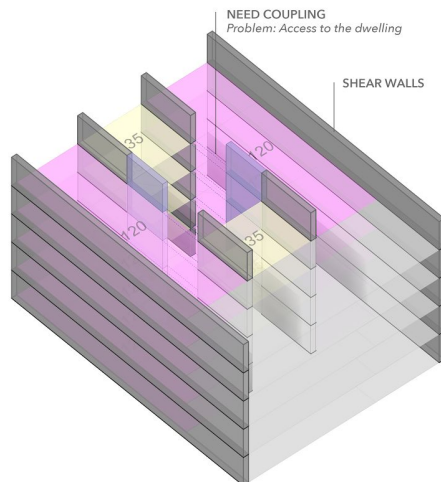
The use of CLT walls implies a high repetition of barriers and constraints in between the housing units, that clearly defines the main distribution of the dwellings. As a general rule, structural walls can be easily placed in between two housing units without any restriction to floor plan or flexibility. However, at the junction between vertical circulation core and the units, the placement of structure is more complicated as appropriate access to the dwelling must be allowed for a functional dwelling.

In these sense, only 50% of the lateral side of the core has been occupied by structural shear wall. At the same time, they are located in complementary directions, extending the original housing-division walls.

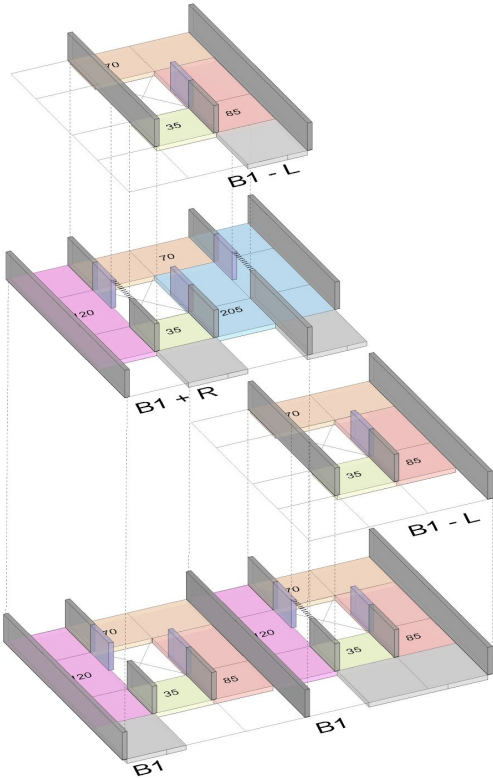


Integration of structural CLT walls within different housing units around central core (Own elaboration)

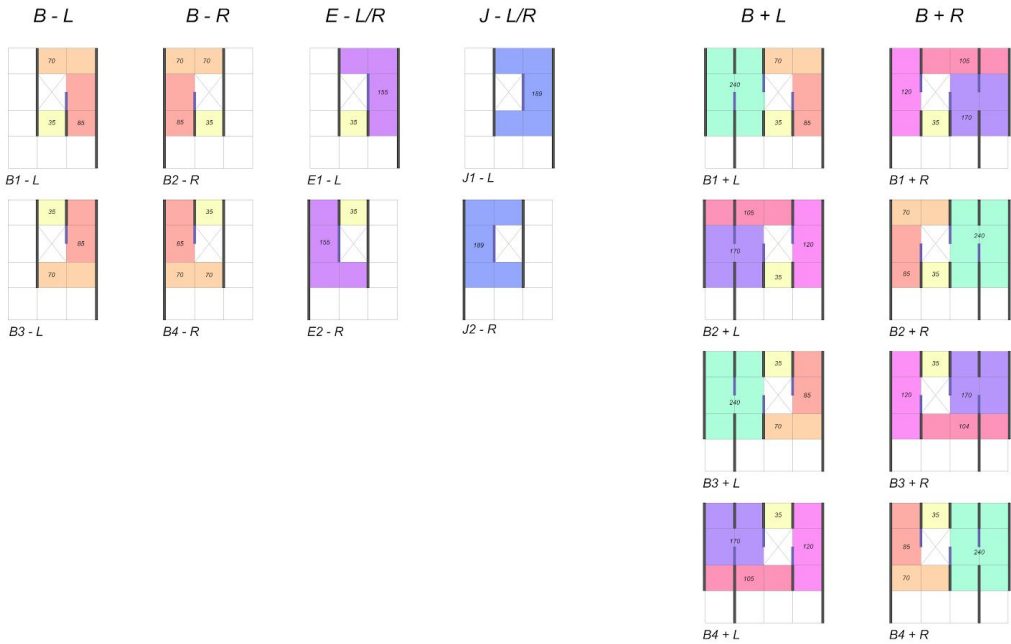
- Often shear walls have to be discontinued in order to allow access to the different housing units.
- If the shear walls work fully independently, as portrayed in the above image, they are 90% less effective. As a result, structural coupling of one storey (at least) is required, and thus, different clustering combinations have to be researched.



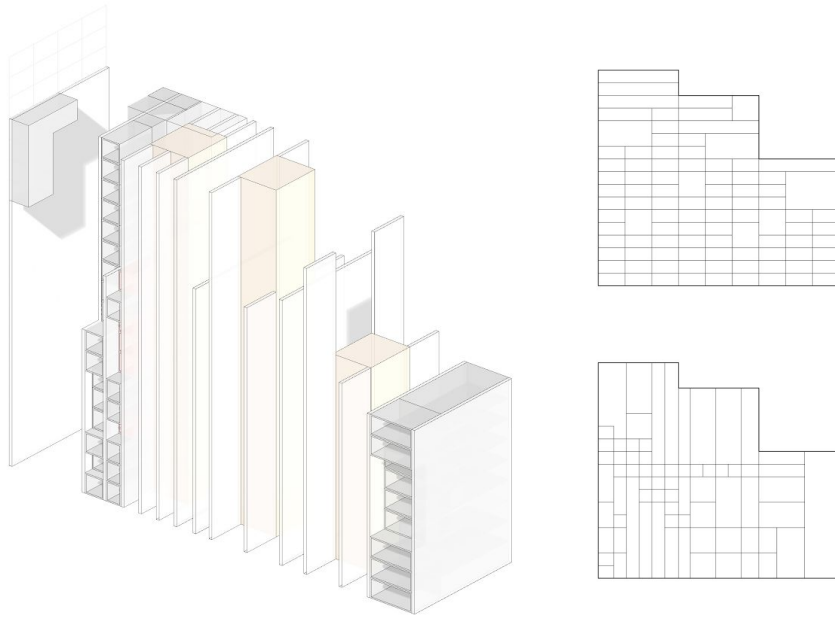
Simple vertical clustering D1, around central core. We can observe that two independent shear walls need coupling on above stories (Own elaboration)



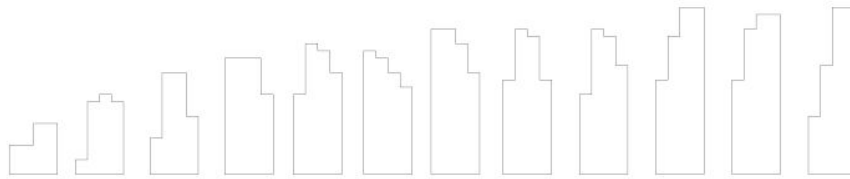
Exploded assembly of clusters coupling shear walls (Own elaboration)



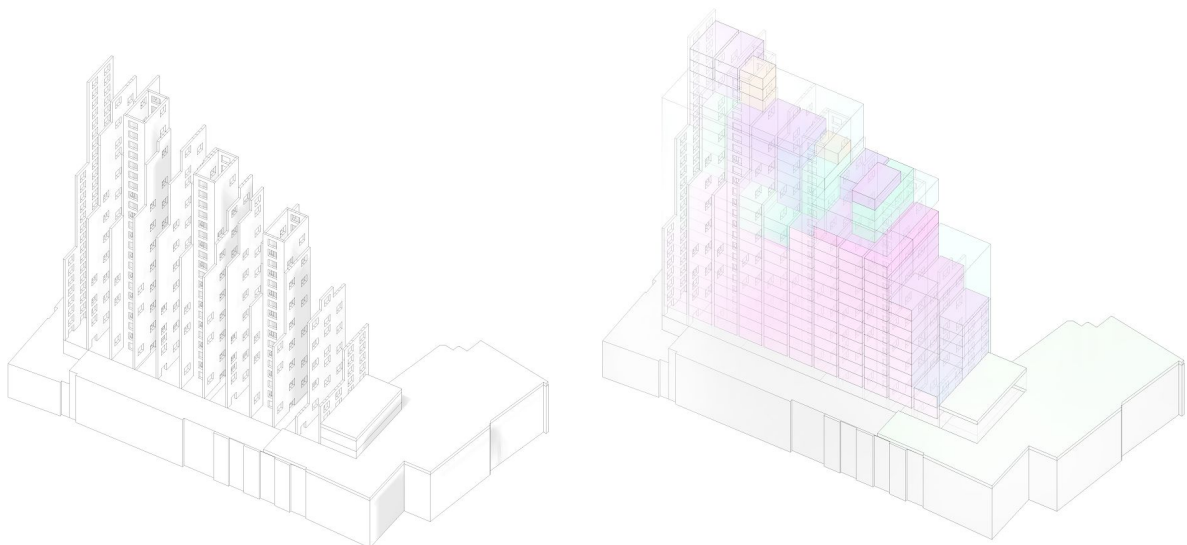
Different configurations for coupling stories (Own elaboration)



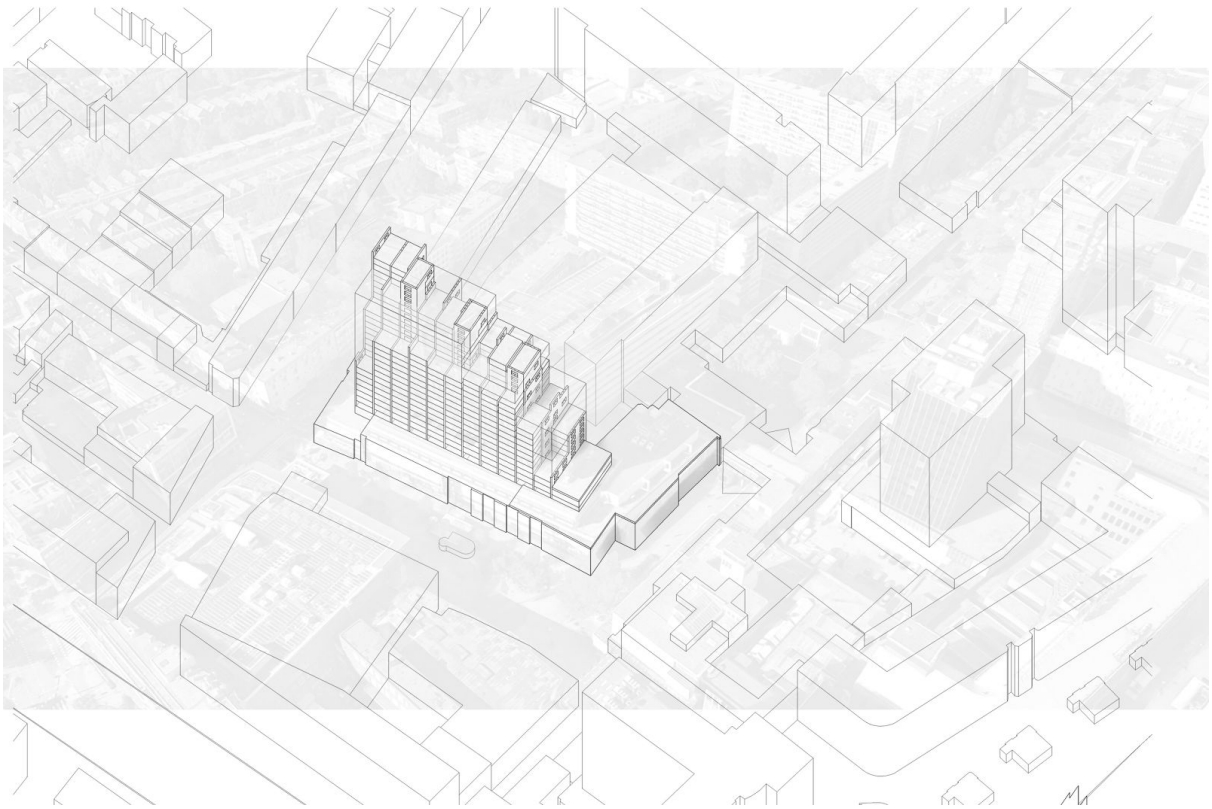
Experimentation integration shear walls, and housing units (Own elaboration)



Different profiles for structural shear walls (Own elaboration)



Scheme of perforated shear walls + integration with housing units (Own elaboration)



Collage/drawing building in the site (Own elaboration)

PART 4

DESIGN STUDY - STRUCTURE

9. TIMBER STRUCTURAL DESIGN

- Timber design
 - Modification factors
 - Long-term deflection
- Cross-Laminated Timber
 - Layer interaction
 - Rolling shear
 - Vibrations
- Concrete-Timber composite

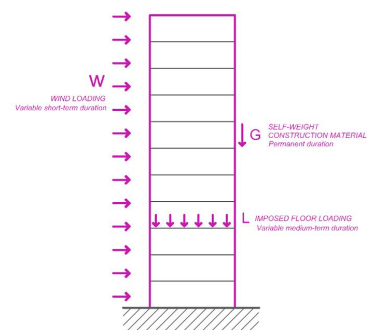
TIMBER

Timber is a live material. Its properties are anisotropic, they change with changes in environmental conditions and load duration has also an important effect upon strength and deformation. A series of “modification” factors are applied to the timber material, in order to adapt the mechanical properties to the surrounding environment and loading conditions.

Modification factors

The duration of the load significantly influences the strength and deformations of timber structures. With increasing load duration, the strength of timber decreases. The structural design must consider each load separately and assign a load duration class, that subsequently modifies the characteristic strength properties of the material.

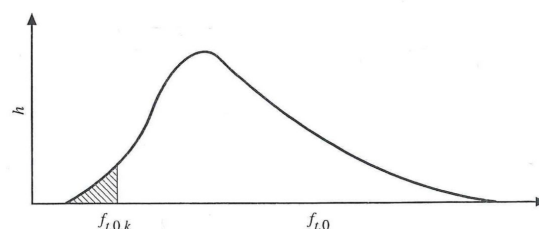
Load-duration class	Order of accumulated duration of characteristic load	Examples of loading
Permanent	more than 10 years	self-weight
Long-term	6 months – 10 years	storage
Medium-term	1 week – 6 months	imposed floor load, snow
Short-term	less than one week	snow, wind
Instantaneous		wind, accidental load



Load duration classes (Eurocode 5), and scheme with main loading scenarios in tall building (Own work)

In the specific scenario of a tall building structure, three major loading conditions are present:

- Wind loading (W), with a variable short-term duration
- Imposed or live floor loading (L), variable medium-term duration related to the use of the building
- Gravity loading (G), permanent self-weight of the construction materials.



Distribution of tensile strength of ungraded structural timber, where h is the frequency (STEP 1)

The strength properties of ungraded timber may vary up to 10 times between the weakest and strongest pieces. Structural design codes specify that only the highest 5% percentile of the tested population can be considered for Ultimate State verifications. Mean values can be used for Serviceability Limit States verifications.

The strength properties of timber are decreased by two factors, material safety factor (1.25 for CLT and glue laminated panels) and k_{mod} , a strength modification factor that account for load duration and environmental conditions. The design value of a timber material property is defined by the following expression:

$$X_d = \frac{k_{mod} \times X_k}{\gamma_M}$$

Where X_k is the characteristic value of strength property and γ_M is the partial safety factor for a material property. It has been proposed (Brander, 2016) that the partial safety factor of CLT can be the same as GLT. Note here, that material safety factor can be taken as 1.00 for Serviceability calculations

Fundamental combinations:		Material						
Solid timber	1,3	Standard	Service class	Load-duration class				
Glued laminated timber	1,25			Permanent action	Long term action	Medium term action	Short term action	Instantaneous action
LVL, plywood, OSB,	1,2	EN 14081-1	1	0,60	0,70	0,80	0,90	1,10
Particleboards	1,3		2	0,60	0,70	0,80	0,90	1,10
Fibreboards, hard	1,3		3	0,50	0,55	0,65	0,70	0,90
Fibreboards, medium	1,3	EN 14080	1	0,60	0,70	0,80	0,90	1,10
Fibreboards, MDF	1,3		2	0,60	0,70	0,80	0,90	1,10
Fibreboards, soft	1,3		3	0,50	0,55	0,65	0,70	0,90
Connections	1,3	EN 14374, EN 14279	1	0,60	0,70	0,80	0,90	1,10
Punched metal plate fasteners	1,25		2	0,60	0,70	0,80	0,90	1,10
Accidental combinations	1,0		3	0,50	0,55	0,65	0,70	0,90
		EN 636 Part 1, Part 2, Part 3 Part 2, Part 3 Part 3	1	0,60	0,70	0,80	0,90	1,10
			2	0,60	0,70	0,80	0,90	1,10
			3	0,50	0,55	0,65	0,70	0,90

Partial safety factors (left) and strength modification factors k_{mod} for timber products (Eurocode 5)

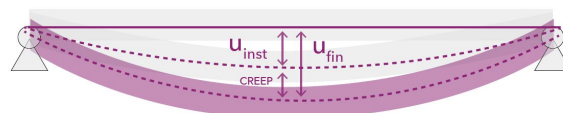
Eurocode 5, specifies three Service classes for timber structures are according to the relative humidity of the surrounding air:

- SC1, when relative humidity exceeds 65% only a few weeks per year
- SC2, when relative humidity exceeds 85% only a few weeks per year
- SC3, Climatic conditions leading to higher moisture contents.

According to the weather station in The Hague/Rotterdam airport, average relative humidity levels exceeds 85% humidity only a few weeks per year. Consequently, the timber structure will be used in Service Class 2.

Note at this point, that additional durability precautions (wood specie, rain protection, chemical treatments) will have to put in place if the structure is fully exposed to external weather. According to Unterweiser and Schickhofer, 2013 CLT structures should be restricted only to service classes 1 and 2.

Long-term deflections "Creep"



Timber is subjected to creep deformation, which means that under the same load acting on a structural member for a long period of time, the modulus of elasticity E , decreases. As a consequence additional deflection are experienced in over time. In addition, if moisture content varies, creep deformations may exceeds several hundred percent of the initial deformations. The creep influence can be estimated with material factor k_{def} , which values depend on service classes, duration of the load and moisture content.

Material	Standard	Service class		
		1	2	3
Solid timber	EN 14081-1	0,60	0,80	2,00
Glued Laminated timber	EN 14080	0,60	0,80	2,00
LVL	EN 14374, EN 14279	0,60	0,80	2,00
Plywood	EN 636			
	Part 1	0,80	–	–
	Part 2	0,80	1,00	–
	Part 3	0,80	1,00	2,50

Creep factor k_{def} , for GLT equivalent to CLT (Eurocode 5)

It can be assumed, that permanent loading always have an impact on long-term deformations. On the contrary, variable loading, e.g. live loading can have different implications on deflections depending on the duration. Design codes account for this effect, with the use of a quasi-permanent load reduction coefficient in the combination of variable loading. A limiting factor of $L/300$ is commonly used for final deflections (instantaneous + creep). The following expression can be used to calculate total deflections accounting for instantaneous and long-term effects:

$$u_{fin} = u_{fin;G} + u_{fin;Q;1} + \sum u_{fin;Q;i}$$

$$u_{fin;G} = u_{inst;G} (1 + k_{def})$$

$$u_{fin;Q;1} = u_{inst;Q;1} (1 + \psi_{2,1} \cdot k_{def})$$

$$u_{fin;Q;i} = u_{inst;Q;i} (\psi_0 + \psi_{2,i} \cdot k_{def})$$

u_{fin} = final deflection
 u_{inst} = instantaneous deflection
 u_G = deflection under permanent load
 $u_{Q,1}$ = deflection under leading variable load
 $u_{Q,i}$ = deflection under other variable load
 ψ_0 = load combination factor (see also [general][loads])
 $\psi_{2,i}$ = factor for quasi permanent value of variable load i
 k_{def} = creep factor

	ψ_0	ψ_1	ψ_2
1. Live load in buildings			
Category A: residential buildings	0.7	0.5	0.3
Category B: offices	0.7	0.5	0.3
Category C: buildings for meeting-places	0.7	0.7	0.6
Category D: trades	0.7	0.7	0.6
Category E: warehouses	1.0	0.9	0.8

Expression for long-term deflection calculations and reduction coefficient for live loading in buildings (EC5)

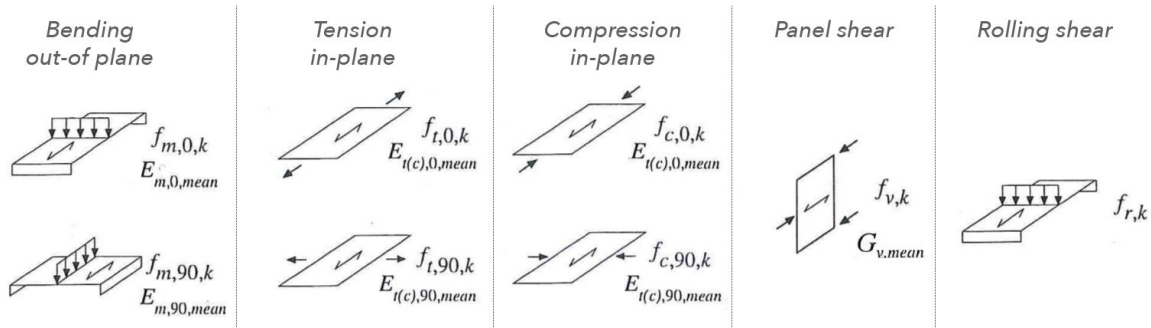
Since shear stiffness is comparatively low in timber, shear induced deflections need to be considered. Since the ratio E/G for timber and glulam is approximately 15, shear deformations can be estimated as an additional 15% to the bending deformations for span/depth ratios of 10, and 5% for span/lengths ratios of 20. It must also be noted here, that for deflections calculations of timber structures, mean values of Young's Modulus can be used.

Moreover, in timber structures connections are usually not as rigid as in concrete or steel. Therefore special attention should be taken to the flexibility of joints and fasteners. For example, in timber trusses joints flexibility can increase deflections up to 50%. In finite element analysis we can consider joint flexibility in such a way that the cross section areas, or all the elements are replaced by a fictitious decreased cross-section area.

CROSS-LAMINATED TIMBER

Wood is a natural material that has very different properties in different directions. Parallel to the grain (direction of the trunk tree) the strength of the material is particularly high, whereas perpendicular to the grain the strength properties are low. The tension strength of wood parallel to the grain can be up to 40 times greater than the tension strength perpendicular to the grain, depending on wood species, moisture content and other factors.

Individual layer orientation and corresponding material parameters have to be taken into account when calculating stresses and stiffness values in CLT panels. Due to the inherent complexity of timber material because of the anisotropy, and the CLT composition of timber sub-elements, it is of utmost importance to consider carefully the loading scheme and the appropriate distribution of stresses accordingly. The following image, summarise some of the mentioned loading scenarios, with the respective symbology for characteristic strength and modulus of elasticity.



Different loading scenarios and respective characteristic strength and elasticity moduli symbology (STEP 1)

Layer interaction

Different methods have been adopted for the determination of mechanical properties of CLT panels. For floor elements, experimental evaluation involves determination of flexural properties by testing full-size panels with specific span to depth ratio. Needless to say, every time layout, type of material or manufacturing parameters are modified, more tested is needed to evaluate accurate bending properties.

At the time of writing, there was not a well established design methodology for CLT element as mechanical properties vary according to manufacturer and timber species. The common strength grades for laminated are in the range between C16-C28. The panels are dried to a moisture content of 12%. The density of the panels at this moisture content is typically in the range of 470 kg/m³. Typical strength properties for C30 timber can be seen in the table below:

Strength properties				Stiffness properties			
Property	Fibers direction	Symbol	Value (N/mm ²)	Property	Fibers direction	Symbol	Value (N/mm ²)
Bending	Parallel	$f_{m,k}$	30	Modulus of elasticity	Parallel mean	$E_{0, mean}$	12000
Tensile	Parallel	$f_{t,0,k}$	18		Parallel 5%	$E_{0,05}$	8000
	Perpendicular	$f_{t,90,k}$	0,4		Perpendicular E0/30	$E_{90, mean}$	400
Compression	Parallel	$f_{c,0,k}$	23	Shear modulus	Parallel E0/16	G_{mean}	750
	Perpendicular	$f_{c,90,k}$	2,7		Perpendicular E0/160	$G_{r,mean}$	75
Shear	Parallel (In section plane)	$f_{v,k}$	4,0	Density	Characteristic	D_k	380
	Rolling shear	$f_{r,k}$	0,4		Average	D_{mean}	460

Typical strength and stiffness values for C30

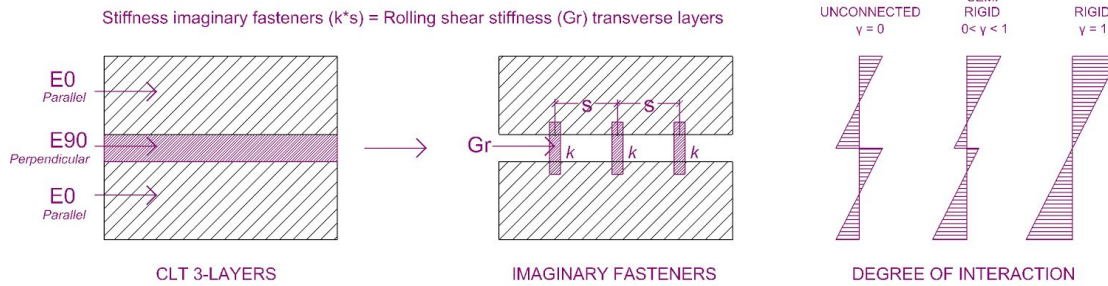
Composite Theory (Blass 2005)

This calculation method takes into account layers loaded parallel to the grain as cross layers loaded perpendicular to grain. The strength and stiffness properties of the panel are determined by using the gross cross section multiplied by a composition factor that considers the different laminations of the CLT panel. The composition factor, k , takes into account joint slip relative to a fictitious homogeneous cross section of equal thickness with the grain of all layers parallel to the direction of the stress. This theory neglect shear deformation in bending members, and therefore can only be used for high span to depth elements, $L/D > 30$ when loading is perpendicular to the plane and parallel to the grain of the outer layers.

	k_i	
	$k_1 = 1 - \left[1 - \frac{E_{90}}{E_0} \right] \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \dots \pm a_i^3}{a_m^3}$	
	$k_2 = \frac{E_{90}}{E_0} + \left[1 - \frac{E_{90}}{E_0} \right] \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + \dots \pm a_i^3}{a_m^3}$	
	$k_3 = 1 - \left[1 - \frac{E_{90}}{E_0} \right] \cdot \frac{a_{m-2} - a_{m-4} + \dots \pm a_i}{a_m}$	
	$k_4 = \frac{E_{90}}{E_0} + \left[1 - \frac{E_{90}}{E_0} \right] \cdot \frac{a_{m-2} - a_{m-4} + \dots \pm a_i}{a_m}$	

Loading	To the grain of outer skins	Effective strength value	Effective stiffness value
Perpendicular to the plane loading			
Bending	Parallel	$f_{m,0,ef} = f_{m,0} \cdot k_1$	$E_{m,0,ef} = E_0 \cdot k_1$
	Perpendicular	$f_{m,90,ef} = f_{m,0} \cdot k_2 \cdot a_m / a_{m,2}$	$E_{m,90,ef} = E_0 \cdot k_2$
In-plane loading			
Bending	Parallel	$f_{m,0,ef} = f_{m,0} \cdot k_3$	$E_{m,0,ef} = E_0 \cdot k_3$
	Perpendicular	$f_{m,90,ef} = f_{m,0} \cdot k_4$	$E_{m,90,ef} = E_0 \cdot k_4$
Tension	Parallel	$f_{t,0,ef} = f_{t,0} \cdot k_3$	$E_{t,0,ef} = E_0 \cdot k_3$
	Perpendicular	$f_{t,90,ef} = f_{t,0} \cdot k_4$	$E_{t,90,ef} = E_0 \cdot k_4$
Compression	Parallel	$f_{c,0,ef} = f_{c,0} \cdot k_3$	$E_{c,0,ef} = E_0 \cdot k_3$
	Perpendicular	$f_{c,90,ef} = f_{c,0} \cdot k_4$	$E_{c,90,ef} = E_0 \cdot k_4$

Composition factors (Blass 2005)

Mechanically Jointed Beams or "gamma method" (Eurocode 5 Annex B)

Scheme degree of interaction between layers with gamma method (Own elaboration)

It is the most common approach used in Europe, based on Annex B of Eurocode 5 (EN 1995:2004).

The behaviour is similar to beams connected together with mechanical fasteners with stiffness " k ", uniformly spaced at certain distances " s ". It can be used for bending strength and stiffness calculations and shear strength. Only layers acting in the direction of loading are considered. Shear deformation of longitudinal layers is neglected for length/depth ratio ≥ 30 . This method takes into account the rolling shear stiffness " G_r ". Longitudinal layers are taken as beam elements connected with "imaginary" fasteners with stiffness equal to the rolling shear stiffness of the cross layers. This method can be used for composite cross-section up to three layers (with two flexible layers). The degree of interaction between layers, γ , can range from 0 (with no interaction) to 1 (rigid connection). Common values are situated between 0.85-0.99.

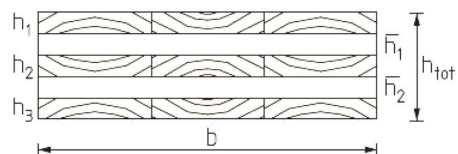
$$I_{ef} = \sum_{i=1}^3 (I_i + \gamma_i \cdot A_i \cdot a_i^2) \text{ with } A_i = b_i \cdot h_i; \quad I_i = \frac{b_i \cdot h_i^3}{12}$$

$$\gamma_1 = \frac{1}{1 + \frac{\pi^2 \cdot E_0 \cdot A_1 \cdot h_1}{G_r \cdot b \cdot l^2}} \quad \gamma_2 = 1 \quad \gamma_3 = \frac{1}{1 + \frac{\pi^2 \cdot E_0 \cdot A_3 \cdot h_2}{G_r \cdot b \cdot l^2}}$$

$$a_1 = (h_1/2 + \bar{h}_1 + h_2/2) - a_2$$

$$a_2 = \frac{\gamma_1 \cdot A_1 \cdot \left(\frac{h_1}{2} + \bar{h}_1 + \frac{h_2}{2} \right) - \gamma_3 \cdot A_3 \cdot \left(\frac{h_2}{2} + \bar{h}_2 + \frac{h_3}{2} \right)}{\sum_{i=1}^3 (\gamma_i \cdot A_i)}$$

$$a_3 = (h_2/2 + \bar{h}_2 + h_3/2) + a_2$$

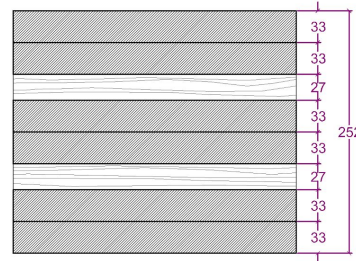


Expressions for Theory of Mechanically Jointed Beams (Van Egmond 2011)

Comparison

In order to understand the above mentioned methodologies, a CLT benchmark section of 8 layers was calculated with both methods. The mechanically jointed or “gamma method” has a limitation for the calculation of 5 layers, for that reason it has been assumed that two longitudinal layers of 33mm, behave in a fully composite way, and can be equal to a single layer of 66mm.

Cross section	CLT 8 layers
Built up	33-33-27-33-33-27-33-33
Timber class	C30
E0 (N/mm ²)	12000
E90 (N/mm ²)	400
Gr (N/mm ²)	75
Second moment of area (I)	1.19e+9mm ⁴
Bending stiffness (EI) Fully rigid	1.43e+13Nmm ²
Bending stiffness (EI) No layer interaction	8.64e+11Nmm ²

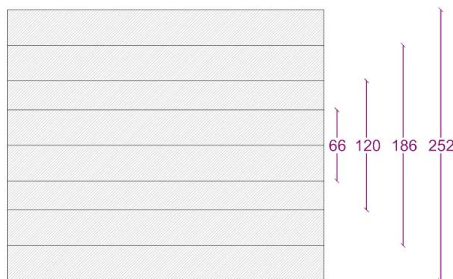


Benchmark section properties for comparison composite action methods (Own elaboration)

In the table below, we can see that the composite theory is more conservative than gamma method, resulting in 79% composite action. It can also be estimated that gamma method for the section above, considers almost complete composite action.

COMPOSITE THEORY

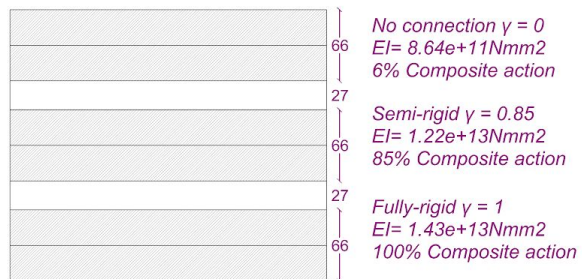
(Blass 2005)



$K1 = 0.73$
Bending stiffness= 1.14e+13Nmm²
79% Composite action

GAMMA METHOD

(Eurocode 5)



$\gamma = 0.96$
Bending stiffness= 1.40e+13Nmm²
97% Composite action

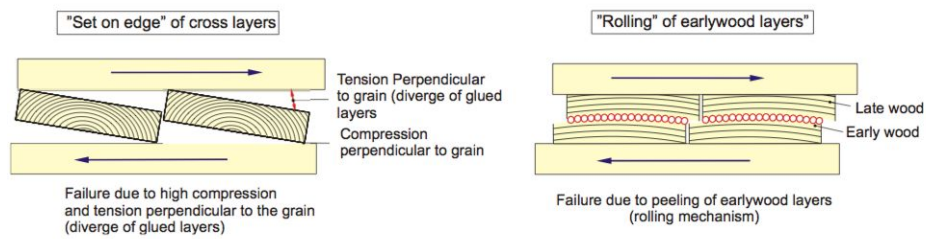
Comparison composite action for two calculation methods (Own elaboration)

A reduction of 20% for effective bending stiffness (out-of plane loading) can be estimated for composite action, based on results of Blass theory in 2005 for 5 or more layers CLT cross sections. It can be concluded that this methodology is more conservative than gamma method used and Eurocode 5, and thus, will be used in the structural design.

Rolling shear









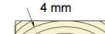
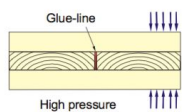
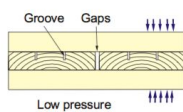
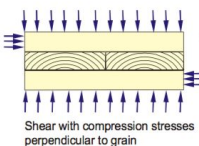
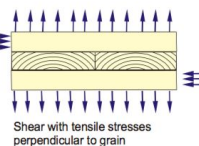
Timber high anisotropy results in low strength and stiffness properties perpendicular to the grain. Therefore, transverse layers of the cross-section will have to carry loads perpendicular to grain, as they are stressed with radial and tangential planes, a phenomenon called “rolling shear”. The rolling shear stiffness for the transverse layers has been found to be $Gr_{mean}/G_{mean}=0.10$.

This complex load carrying behaviour is in practical verifications generally reduced to a simple shear verification of a beam. The used design models are based on homogeneous single layers which represented by their thickness, stiffness and strength properties. Bending tests with CLT elements have proven that the common failure mechanism is initiated within the cross layers, in a combination of two modes, rotation of the cross layers and rolling of the earlywood zones.



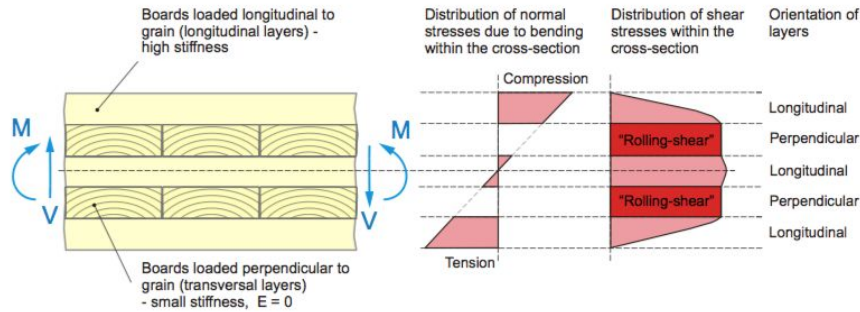
Failure mechanism within the cross layers of a CLT element, macroscopic (CIE5124 Handbook)

Numerical analysis show that the rolling shear modulus is not a material parameter, and can be considered distributed shear stiffness characteristic dependent on the structural wood parameters. Brandner 2016 suggests a minimum lamella width of 4 times higher than layer thickness for increasing rolling shear resistance. In the following table, the main influencing parameters on the rolling shear properties are listed..

Influencing parameters on the "rolling shear" properties	positive \longrightarrow negative
Board dimensions of the cross layer	1  8 1  4 1  2
Position of the boards in the log (sawing pattern)	 Half-radial sawing  Radial sawing  Tangential sawing
Annual ring width and density (Ratio earlywood : latewood)	 1 mm  2 mm  4 mm
Production Pressure Type of adhesive	 High pressure (> 0.6 N/mm ²)  Low pressure (e.g. 0.1 N/mm ²)
Type of loading	 Shear with compression stresses perpendicular to grain  Shear with tensile stresses perpendicular to grain

For a rough verification of layers stresses in plates a rigid connection between layers can be assumed. For verification in the serviceability limit state, the flexibility of the cross-layers has to be considered. Several authors (Schickhofer et al. 2010, Bogensperger et al. 2010, Blab and Flaig, 2012 and Flaig, 2013) suggest the verification of normal stresses in-plane only using net cross section area, i.e. the layers in the direction of stresses, and neglected the contribution of the transverse layers, E_{90} is considered to be zero. For slender members the possibility of lateral buckling has to be considered.

At the same time, due to unavoidable cracks and gaps in between layers of timber boards, it is considered that there is no transfer of normal stresses in the transverse layers (tension and compression perpendicular to grain).



Stress distributions of a CLT-element with unglued narrow faces of the boards (CIE5124 Handbook)

- Deformation and stress values have to be computed as composite with rigid layer interfaces.
- Rolling-shear stresses have to be considered.
- For calculation of cross-sectional values in the direction of the main axis, E_{90} can be neglected. +
- In addition to the bending effects, the influence of shear deformations has to be considered.
- To avoid undesired dynamical effects, deformations due to permanent forces (without creep-effects) have to be limited to 5mm
- For modelling of shear walls, simplified truss and frame models can be applied in conjunction with more detailed FEA analysis

Vibrations

Eurocode 5 specifies that timber floors should have a fundamental frequency of at least 9Hz. Otherwise special investigations should be carried out. Timber floor vibrations can be controlled with a proper combination of stiffness and mass. For a simply supported floor the fundamental frequency can be calculated with:

$$f_1 = \frac{\pi}{2 \cdot L^2} \sqrt{\frac{(EI)_l}{m}}$$

Where L is the floor span, $(EI)_l$ the equivalent plate bending stiffness of the floor about an axis perpendicular to the span direction, and m is the mass of the floor to consider calculated as $m = m_g + 30 \text{ kg/m}^2$ where m_g is the self-weight of the finished floor element and 30 kg/m^2 accounts for the permanent part of the service load.

The objective of the design is to provide a floor system with fundamental frequency different that in the annoyance range for the human body (7-12Hz)

The design code also recommends that floors that have higher fundamental frequency than 9Hz, should also verify deflection caused by static concentrated loading $P=1\text{kN}$ should not exceed 0.5mm, calculated as:

$$\delta_{1\text{kN}} = \min \left\{ \frac{PL^2}{42 \cdot k_\delta \cdot (EI)_l} ; \frac{PL^3}{42 \cdot s \cdot (EI)_l} \right\} \quad k_\delta = \min \left\{ \frac{B}{L} ; \sqrt[4]{\frac{(EI)_l}{(EI)_b}} \right\}$$

Where k is the coefficient accounting for the distribution of the concentrated load over the floor width depending on the bending stiffness of the floor with respect to an axis parallel to the span direction $(EI)_b$, B is the floor width, and s the spacing of the floor beams.

The contribution of the additional floor finishes layers (i.e. floating floor) can be considered in addition for the transverse bending stiffness $(EI)_b$ and can allow to distribute even more the concentrated load over the floor width. The unit impulse velocity response v , should verify:

$$\nu \leq b^{(f_1 \zeta - 1)} \quad \nu = \frac{4(0.4 + 0.6n_{40})}{m \cdot B \cdot L + 200} \quad n_{40} = \left\{ \left(\left(\frac{40}{f_1} \right)^2 - 1 \right) \left(\frac{B}{L} \right)^4 \frac{(EI)_t}{(EI)_b} \right\}^{0.25}$$

where $b=100$ and ζ is the modal damping ratio of the floor

Simplified method

Hu 2011b and 2012b developed a simplified method for a broad range of CLT floor systems with or without topping, with or without gypsum board ceiling, based on above mentioned principles.

$$l \leq \frac{1}{9.15} \frac{(EI_{eff}^{1m})^{0.293}}{(\rho A)^{0.123}}$$

Where EI , is the apparent stiffness in the span direction for 1 meter wide panel, ρ is the density of CLT, and A , area of cross section. The method was verified through laboratory study in Graz University of Technology.

Span	Thickness CLT
3 meters	100mm
6 meters	240mm

Vibration controlled spans for CLT panels

CONCRETE-TIMBER COMPOSITE

Timber-Concrete Composite (TCC) is a technology which focuses on optimising performance and material requirements by engineering a structural connection between timber and concrete components. Structural efficiency is gained by creating composite action between the two materials. This hybridization enables designers reduce cross section, increase spans, and achieve a sustainable architecture.

The structural behaviour of timber-concrete composite elements is usually governed by the shear connection between timber and concrete. As a consequence, conventional principles of structural analysis such as method of transformed section cannot be applied. Eurocode "gamma method" provides an approximation for these behaviour.

$$(EI)_{eff} = E_1 I_1 + \gamma E_1 A_1 a_1^2 + E_2 I_2 + E_2 A_2 a_2^2$$

With shear coefficient gamma and distances a_i given by:

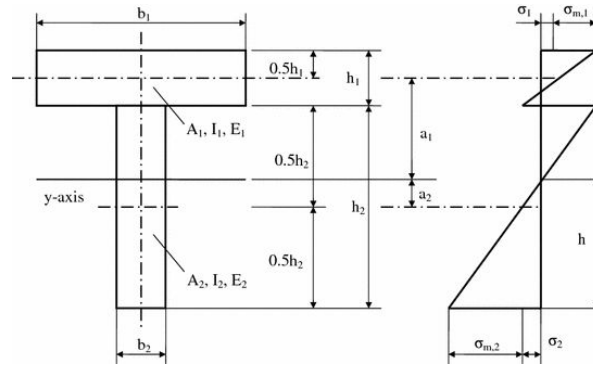
$$\gamma = \frac{1}{1 + \frac{\pi^2 E_1 A_1 s}{k L^2}} \quad a_1 = \frac{h_c + h_t}{2} - a_2 \quad a_2 = \frac{\gamma E_1 A_1 (h_1 + h_2)}{2 \gamma E_1 A_1 + E_2 A_2}$$

$i=1$ for concrete slab, $i=2$ for timber beam, s is the connector spacing, L the beam length, k the slip modulus of the connector and γ is the coefficient of interaction between the layers. The effective stiffness of the system can be used to calculate deflection, stress distribution and shear load in the fastener with the following expressions:

$$\delta = \frac{5qL^4}{384(EI)_{eff}} \quad \sigma_i = \frac{\gamma E_i a_i}{(EI)_{eff}} M \quad \sigma_{m,i} = \frac{0.5 E_i h_i}{(EI)_{eff}} M \quad F = \frac{\gamma E_i A_i a_i s}{(EI)_{eff}} V$$

where δ is the mid-span deflection of a simply supported beam, q is the uniformly distributed load, σ_i , $\sigma_{m,i}$ are the stress at the centroid and the flexural component of the stress in the concrete ($i = 1$) and timber ($i = 2$), F

is the shear load in the fastener, M is the bending moment and V is the shear force in the cross-section of interest.



Cross section (left) and stress distribution (right) of a composite beam with flexible shear connection (EC5)

As the shear connection is usually characterised by non-linear load-slip relationship, two slip moduli are considered k_{ser} for SLS and k_u for ULS design. The slip modulus k_{ser} , corresponds to the secant value at 40% of the load-carrying capacity of the connection $k_{0.4}$, whereas k_u , is equivalent to the secant value at 60% ($k_{0.6}$).

The verification of the composite beam in the long-term is more problematic. A simplified approach suggested by Ceccotti is based on the use of Effective Modulus Method, where the creep of concrete, timber and connection are accounted by reducing the elastic moduli and slip modulus according to the following expressions:

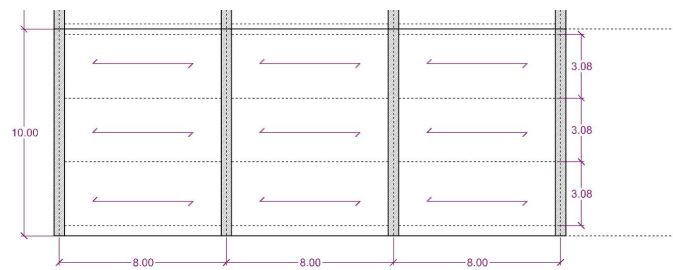
$$E_{1,eff} = \frac{E_1}{(1 + \varphi_1)} \quad E_{2,eff} = \frac{E_2}{(1 + \varphi_2)} \quad k_{eff} = \frac{k}{(1 + \varphi_c)}$$

where φ_1 , φ_2 and φ_c signify the creep coefficient of concrete, timber and connection respectively, and must be used in the aforementioned equations for long-term deflections.

9.1 FLOOR SYSTEM

- 190mm CLT
- 250mm CLT
- Lignatur
- Hollow core
- Concrete + CLT
- Glulam beams
- Alternative floor systems

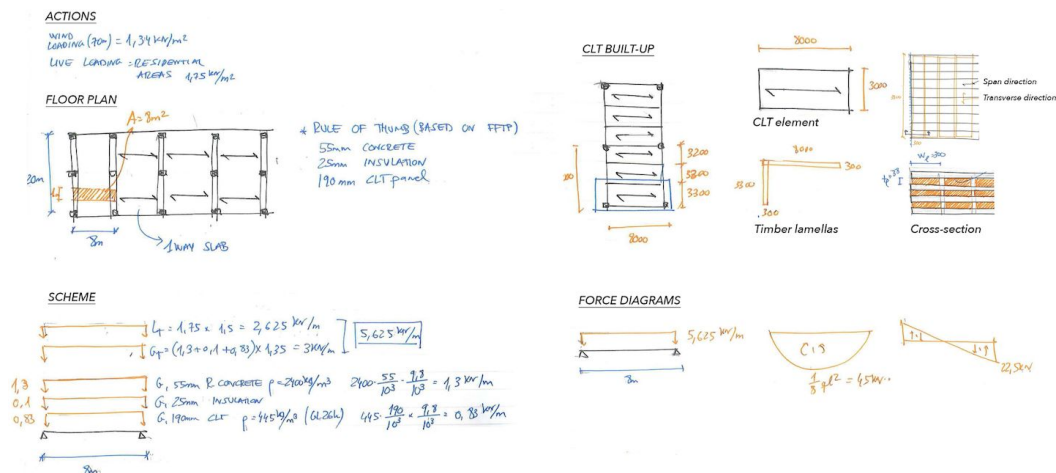
190mm CLT



Scheme of floor system for 8 meters span (Own elaboration)

For the first order of magnitude, the floor system proposed by Michael Green, in FFTP study is assumed. The system is composed out of 55mm concrete topping (non-structural), 25mm rigid insulation and 190mm CLT panel. This composition results in a distributed dead loading of 2.23KN/m².

- Live loading of 1.75KN/m² for residential use, according to Eurocode 1
- CLT element spans in one direction with 8 meters distances between supports.
- Floor behaves as one way slab, with simple supports.

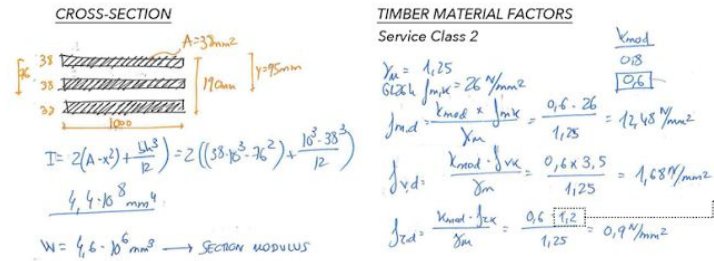


Actions and floor composition schemes (Own elaboration)

The CLT element is composed out of lamellas of equal thickness of 38mm, with transverse directions in each successive layer. The depth of the structure is 190mm, not considering topping, insulation or false ceiling.

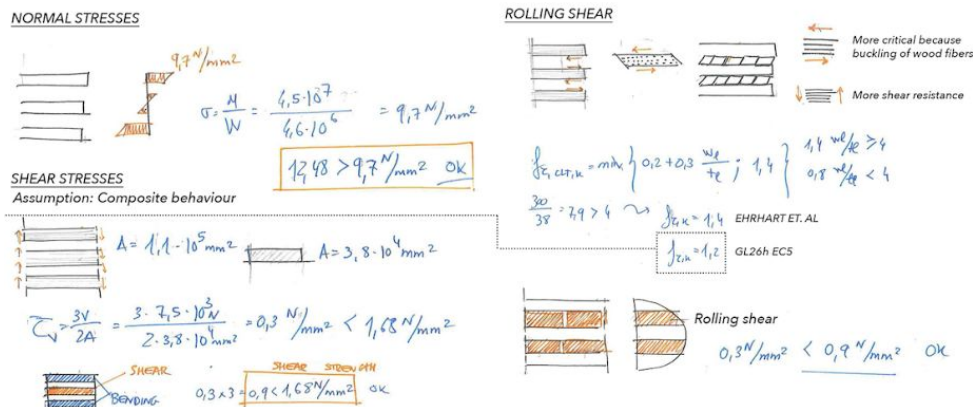
The following assumptions are taken:

- K Modification factor of 0.6 (Service Class 2).
- Partial material factor is taken as 1.25 (As in glue-laminated timber).
- Based on GL26h material, according to Eurocode 5.
- Design values 12.48, 1.68 and 0.9N/mm² for bending, shear and rolling shear strength.



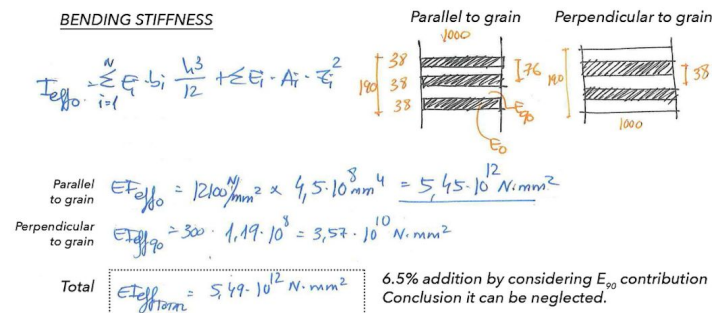
Cross section and timber modification factors (Own elaboration)

The section is assumed to behave in a fully composite way, which rigid connections between the laminations. This results in transfer of shear between the layers, with Rolling shear or planar shear in perpendicular direction to the fibres (as trying to rotate sandwiched lamellas), and section plane shear in parallel direction (as trying to cut the fibres). Note at this point, that the most critical stress and the common cause of failure is the rolling shear stresses, because the compressive forces that lead to internal buckling on the wood fibres.



Stresses calculations (Own elaboration)

For the calculation of normal stresses, it is assumed that transverse layers of timber do not contribute to the bending resistance. This can be argued because of the low Young Modulus of timber in the direction perpendicular to fibres (denoted as E_{90}). In the below image, equivalent bending stiffness has been calculated, with only an increase of 6.5% in bending resistance.

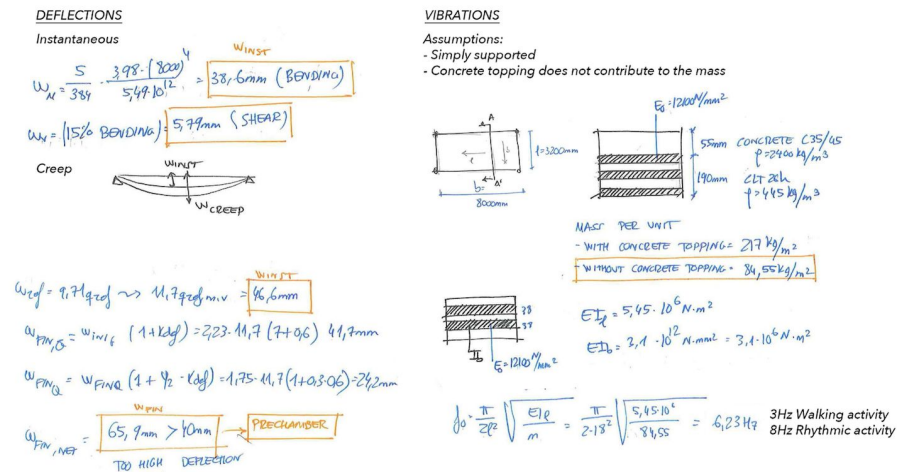


Bending stiffness considering contribution of transverse layers (Own elaboration)

The following assumptions have been taken:

- Characteristic loading combinations are assumed for instantaneous deflections
- Quasi-frequent loading combinations for long-term deflections due to creep
- Shear deformation accounts to 15-20% addition to bending deformations.
- 5% percentile materials values are considered for ULS calculations
- Mean material values are assumed for SLS calculations

- Concrete topping does not contribute to vibrating mass



Deflections and vibrations calculations (Own elaboration)

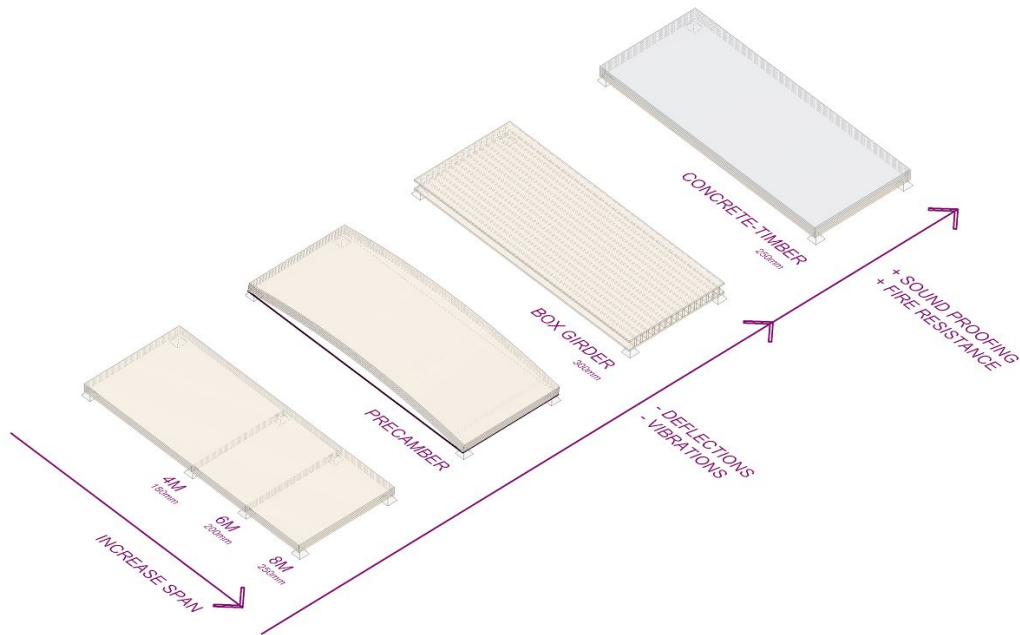
This results in 65.9mm total deflections, which is above the serviceability limits. A fundamental vibration frequency of 6.23Hz, slightly above walking frequency and below rhythmic activities such as dancing. Because of the residential use of the building, it is considered accepted to experiment some vibrations due to non-frequent rhythmic activities.

The proposed floor system satisfies all the structural criterias, except the deflections. In order to prevent that issue, the following measures can be applied:

- Reducing floor span, affecting architectural flexibility and layout
- Increasing the floor depth, increasing the amount of material and reducing storey height
- Use stiffer timber material. This measure is limited by manufacturers, availability and cost of the material.



Strategies for reducing deflections in floor system (Own elaboration)

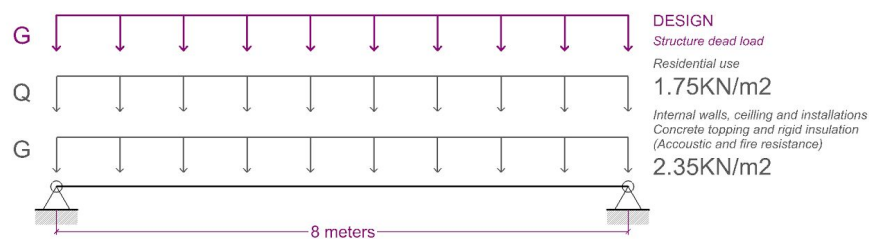


Timber floor systems for long spans (Own elaboration)

250mm CLT

The above measures can be complemented with the additional floor systems that make a more efficient use of the cross-section and use two materials with composite action. In order to compare multiple floor alternatives under the same criteria, the following constraints have been applied:

- 250mm floor depth (Manufacturer tables)
- 8m floor span (Structural grid of the existing building design study)
- CLT with base material C24
- 1.75kN/m² Live loading for residential use (Eurocode 1)



Loading scheme (Own elaboration)

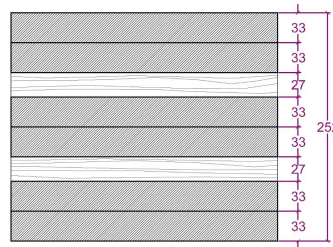
From manufacturers tables (Finforest), it can be extracted that a cross section of 252mm with 8 layers built-up can achieve up to 8.60 meters span for residential use loading.

Thickness (mm)	Layers	Build-up (mm)								Span (m)
61	3	17	27	17						2,05
81	3	27	27	27						2,80
105	5	27	17	17	17	27				3,55
135	5	27	27	27	27	27				4,40
174	6	33	27	27	27	27	33			5,65
201	7	27	33	27	27	27	33	27		6,90
219	7	33	33	27	33	27	33	33		7,55
240	8	27	33	27	33	33	27	33	27	8,15
252	8	33	33	27	33	33	27	33	33	8,60
264	8	33	33	33	33	33	33	33	33	8,90
285	9	33	33	27	33	33	33	27	33	9,45
297	9	33	33	33	33	33	33	33	33	9,75

Most representative CLT panel built-up and feasible spans for residential use, 1.75KN/m2 (Finnforest).

For the CLT benchmark it is assumed a creep factor, K_{def} 0.8 will be used for long-term or creep deflections. In addition, a frequent combination factor of 0.5, will be applied to variable loading for long-term deflections according to the Eurocode 1. An assumption of 80% composite action between layers is estimated from the theory method (Blass 2005).

Cross section	CLT 8 layers
Built up	33-33-27-33-33-27-33-33
Service Class	SC2
K_{def}	0.8
ψ_2	0.3
Timber class	C30
E_0 (N/mm ²)	12000
E_{90} (N/mm ²)	400
G_r (N/mm ²)	75



Benchmark 250mm CLT floor system (Own elaboration)

Second moment of area (I)	1.19e+9mm ⁴
Bending stiffness (EI) Fully rigid	1.43e+13Nmm ²
Bending stiffness (EI) No layer interaction	8.64e+11Nmm ²
Composite action	80%
Effective bending stiffness (E_{leff})	1.14e+13Nmm²

Structure dead load (KN/m ²)		1.1
Total permanent loading (KN/m ²)		2.45
Effective Bending stiffness (Nmm ²)	EI _(eff)	1.14E+13
Ref instantaneous deflection (mm)	W _{inst} (q _{ref})	4.68
Instantaneous deflection permanent loading (mm)	W _{inst,G}	10.99
Instantaneous deflection variable loading (mm)	W _{inst,Q}	8.19
Long-term deflection permanent loading (mm)	W _{fin,G}	19.79
Long-term deflection variable loading (mm)	W _{fin,Q}	10.15
Total long-term deflection (mm)	W_{fin,total}	29.94
Deflection limit (mm)	L/400	20.00

Calculation of deflections CLT 250mm (Own elaboration)

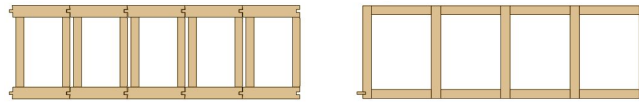
From the above calculations, a total long-term of 29.94mm is concluded. This deformation is above deflection limit recommended in Eurocode 5 (L/400). For that reason, different floor options are compared in the following chapters.

Precamber

From the calculations above, we can extract that the most restrictive design criteria for timber floor systems is usually derived from deflections. One way to reduce the adverse deformation due to long-term deflections or creep is with the use of camber. For floor systems, a value of camber equal to the dead load deflection is recommended. **A value of 10.99mm precamber would reduce the total long-term deflections to 19.94<20mm (deflection limit).**

LIGNATUR

Lignatur is a swiss company that have created a new timber floor system that is capable of integrating multiple function (installations, acoustics, structural performance) within reduced floor depth dimensions. The construction depth is comparable to a conventional concrete floor. The manufacturer provides a range of 90-480mm floor depths, and 16 meters lengths. One type of is made out with prefabricated "box elements" assembled together on site creating the complete floor. Other type is a conventional glued system made off site. Infill solid timber panels can be inserted within the timber frame for structural reinforcement, i.e at the bearings.



Lignatur floor systems, Box elements (left) surface subtype (right) (Lignatur, 2017)

Manufacturer tables indicate that a floor depth between 280-360mm would be required in order to meet structural criteria for 8 meters span within a deflection limit of $L/350$. In the image below, we can see that there is a distinction between structural floor and additional floor construction (q_A in the table) for fire and acoustics considerations.

			I (m)	4.5	5	5.5	6	6.5	7	7.5	8	8.5	9	10
I		$q_n = 200 \text{ kg/m}^2$ $q_A = 0 \text{ kg/m}^2$ $g = 47 \text{ kg/m}^2$	h (mm)	120	140	160	180	200	220	280	280	320	-	-
			w (mm)	8	8	9	9	10	11	7	9	9	-	-
1 2 4 III		$q_n = 200 \text{ kg/m}^2$ $q_A = 36 \text{ kg/m}^2$ $g = 89 \text{ kg/m}^2$	h (mm)	140	180	200	220	280	280	320	360	360	440	-
			w (mm)	9	8	9	10	7	10	10	8	10	9	-
1 3 5 6 II		$q_n = 200 \text{ kg/m}^2$ $q_A = 161 \text{ kg/m}^2$ $g = 39 \text{ kg/m}^2$	h (mm)	160	180	200	240	280	320	320	360	360	360	440
			w (mm)	8	10	11	10	9	9	12	10	13	17	18
1 3 5 IV		$q_n = 200 \text{ kg/m}^2$ $q_A = 116 \text{ kg/m}^2$ $g = 139 \text{ kg/m}^2$	h (mm)	180	200	220	280	280	280	280	320	360	360	440
			w (mm)	8	9	11	8	12	16	21	19	15	19	21
1 3 5 8 V		$q_n = 200 \text{ kg/m}^2$ $q_A = 244 \text{ kg/m}^2$ $g = 64 \text{ kg/m}^2$	h (mm)	180	200	200	220	240	280	320	320	360	360	480
			w (mm)	9	10	15	17	18	17	16	21	18	22	20
1 3 5 7 VI		$q_n = 300 \text{ kg/m}^2$ $q_A = 212 \text{ kg/m}^2$ $g = 68 \text{ kg/m}^2$	h (mm)	200	200	220	240	280	320	320	360	360	440	480
			w (mm)	9	13	15	17	16	16	21	18	23	20	24

deflection $w(q_k) = l/350$

- $q_d = (1 + k_{def}) \cdot (g_k + \psi_1 \cdot q_k)$
- creep coefficient $k_{def} = 0.6$
- permanent load $g_k = g + q_A$
- dead load incl. fill g
(data in tables for $h = 200 \text{ mm}$)
- load of floor construction q_A
- imposed load $q_k = q_{k1}$
- housing $q_{k1} = 2 \text{ kN/m}^2$, $\psi_1 = 0.5$
- education $q_{k1} = 3 \text{ kN/m}^2$, $\psi_1 = 0.7$

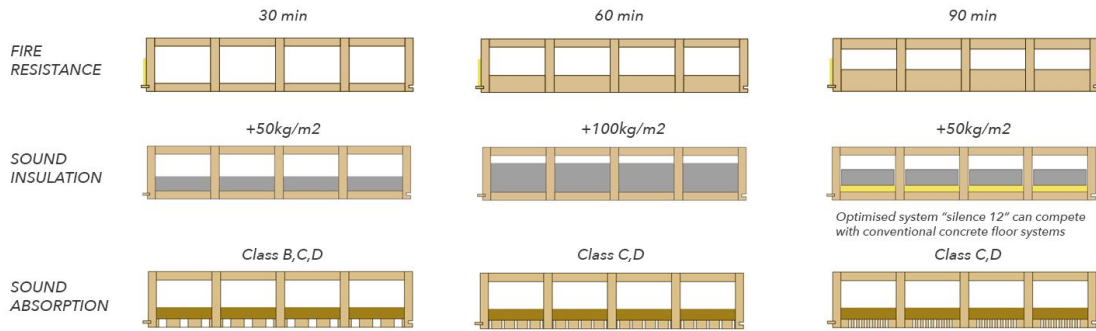
1. natural frequency f_1 , $(g_k + 0.3 \cdot q_k) \geq 8.0 \text{ Hz}$ or
 $8.0 \text{ Hz} > f_1$, $(g_k + 0.3 \cdot q_k) \geq 5.1 \text{ Hz}$
vibration acceleration a , $(g_k + 0.3 \cdot q_k) \leq 0.2 \text{ m/s}^2$
deflection w ($F = 1 \text{ kN}$) $\leq 1.15 \text{ mm}$

- 1 parquet flooring
- 2 Fermacell screed element 25mm
- 3 cement screed 50mm
- 4 wood fibre footfall sound insulation 20mm
- 5 mineral fibre footfall sound insulation 40mm
- 6 Fermacell honeycomb fill 30mm
- 7 bonded chippings 60mm
- 8 bonded chippings 80mm

- I LIGNATUR box element (LKE)
- II LIGNATUR surface element (LFE)
- III LFE with fill 50 kg/m^2
- IV LFE with fill 100 kg/m^2
- V LFE silence12
- VI LFE silence12 with acoustics

Different floor systems and structural performance (Lignatur, 2017)

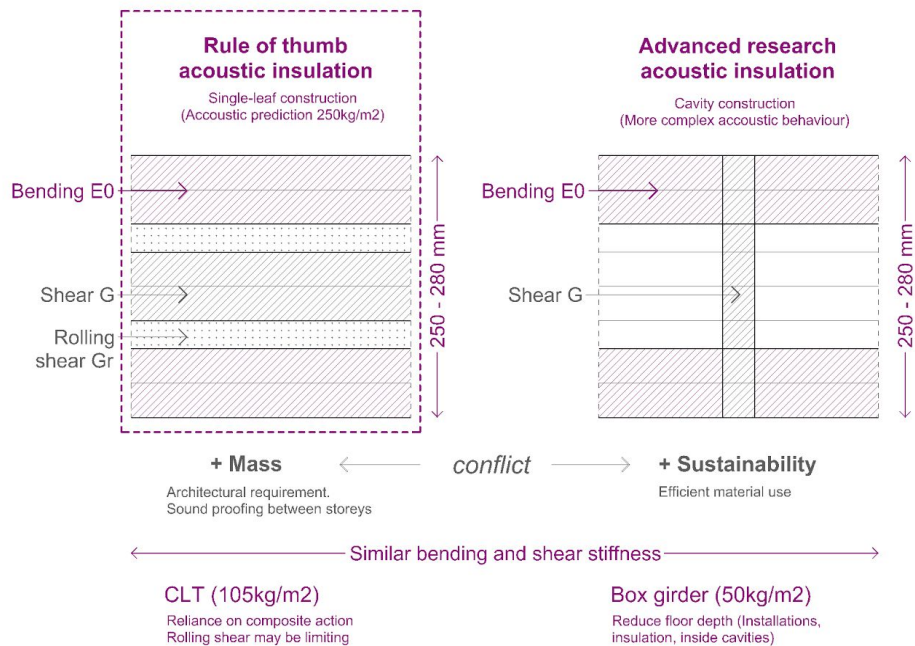
The main advantage of Lignatur structural system is that it can be complemented with floor coverings or infills in order to improve acoustic insulation and fire performance. For example, by adding extra concrete infills, sound insulation (depending on the mass of the system) is dramatically improved. According to Lignatur, optimised floor system Silence 12 can provide sound proofing similar to conventional concrete floors. Higher fire resistance is achieved by thickening the horizontal bottom layer of the system, and better room acoustics by perforating bottom layer with different patterns. In addition, installations can also be integrated in the system leading to a dramatic reduction in floor depth. **By using these configuration the underside of the floor system can be left exposed, showing the use of timber construction.**



Different possibilities of customisation of Lignatur floor system (Lignatur, 2017)

Comparison CLT - Lignatur

Similar structural performance - Floor span (L) 8 meters



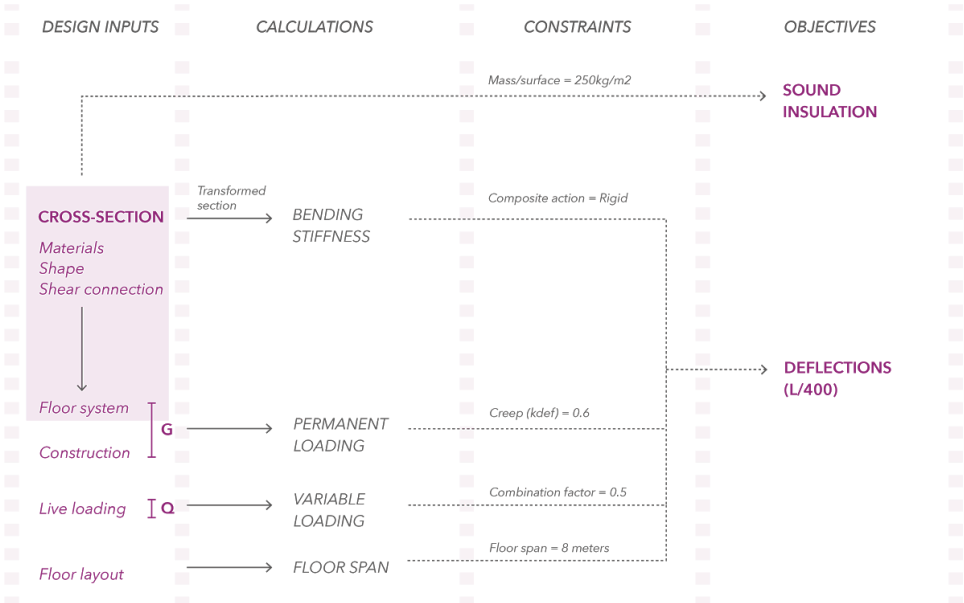
Comparison scheme between most common timber floor systems (Own elaboration)

CLT and box girder have very low dead loads. In order to compare the systems, very small difference in deflections will occur because of the additional loading of CLT panel, so it can be assumed that dead load is neglected. Both floor systems have similar bending and shear stiffness, that may result in a cross section of about 250-280mm (depending on other construction loads, such as finishes, and additional weight). Both entail complexities in the structural calculations regarding composite action of multiple layers and modification factors for timber material.

Acoustic requirements

The main advantage of box girder floor system is that floor depth can be slightly reduced by integrating installations, and acoustic insulation within the structural floor. On the other hand, because it is a lightweight cavity construction, the acoustic behaviour is complex to predict, and will require further research.

The most important advantage of CLT panel is that it is a single-leaf construction. This is very important for sound insulation between different storeys. For the purpose, of this study, in order to meet acceptable sound insulation criteria, a minimum surface/mass of 250kg/m² for a single-leaf floor system will be required.



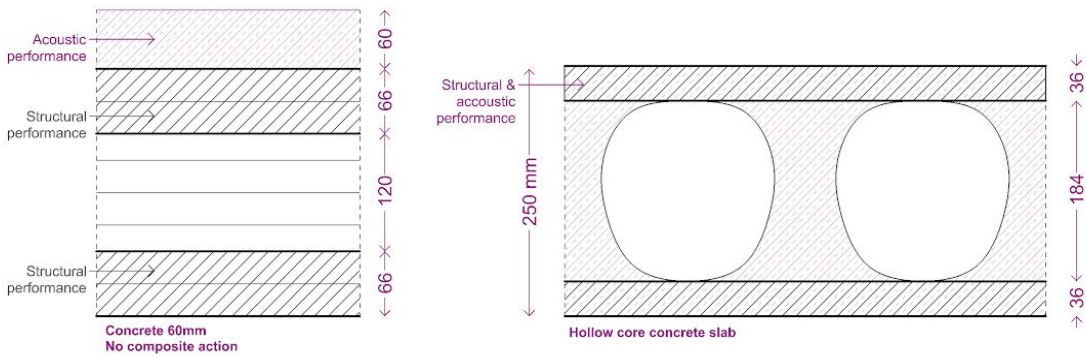
Interrelation sound insulation and structural performance in timber floor system (Own elaboration)

HOLLOW CORE

- Sound insulation - 250 kg/m²
- No shear deformations. (For a conservative estimation 20% additional instantaneous deflections can be taken)
- Fully rigid composite action between layers
- CLT made out of C24. E₀=11000 N/mm², Density= 420 kg/m³
- Concrete C35. E(uncracked) = 34000 N/mm², Density= 2400 kg/m³
- Live loading (Q) 1.75KN/m²
- Timber creep factor (kdef) = 0.6
- Concrete cracking factor = 0.6
- Combination factor variable loading = 0.5

Finishing (floor covering, ceiling...)	0.35KN/m ²
Lightweight separating walls	0.5 KN/m ²

Loading for floor construction



Comparison floor systems with acoustic considerations (Own elaboration)

Floor system	CLT 250mm	CLT+ Concrete topping	Hollow core
Floor depth	250 mm	310 mm	250 mm
Second moment of area	1.17e+9mm ⁴	1.17e+9mm ⁴	8.3+8mm ⁴

Bending stiffness (rigid)	1.29e+13Nmm ²	1.29e+13Nmm ²	2.82e+13Nmm ²
Structure dead load	1KN/m ²	1 + 1.41KN/m ²	3.7KN/m ²
Permanent (G) deflections	7.65mm	13.5mm	8.6mm
Variable (Q) deflections	7.24mm	7.24mm	3.3mm
Creep/cracking (G) deflections	4.59mm	8.1mm	5.16mm
Creep/cracking (Q) deflections	2.18mm	2.18mm	1.98mm
Total instantaneous deflections	14.89mm	20.74mm	11.9mm
Total long-term deflections	21.66mm	31.02mm	19.04mm
Acoustic requirements	NO	YES	YES
Structural requirements	YES	NO	YES

From the above table, it can be concluded that CLT and hollow core concrete slabs have similar structural performance regarding deformations, 21.66 and 19.04mm respectively.

However, because of the low density of timber compared to concrete, CLT floors require additional mass to meet acoustic requirements (250kg/m²) This can be accomplished by adding extra 60mm concrete screed on top. However, the additional floor construction loading, also entails a significant 43% extra deflections, compared to the same CLT floor system without concrete topping.

CLT + CONCRETE COMPOSITE

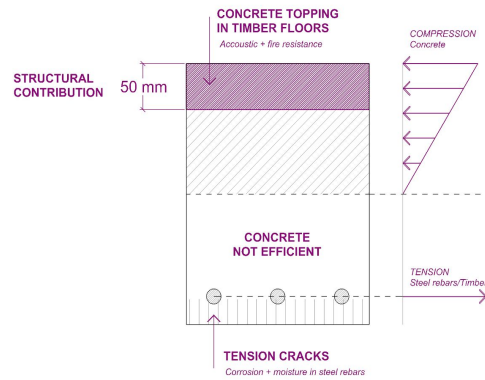


Timber composite panel (Structure craft)

Timber-concrete composite floor systems can be an interesting alternative for multi-storey buildings. The advantages of these systems is increase in load-carrying capacity, higher stiffness which leads to reduction in deflection and less susceptibility to vibrations, an improvement in acoustic performance and thermal properties and higher fire resistance. The concrete topping mainly resists compression, while timber beams or CLT panels resists tension, and an intermediate connection system transmits shear forces between the two components. In reinforced concrete slabs the lower part is ineffective since it is assumed not to contribute structurally, and often corrosion of steel rebars and moisture penetration may occur through the cracks. By replacing this part with solid wood deck, the self weight of the structure can be significantly reduced. The use of lightweight concrete with a density of 1.6Kn/m³ instead of normal concrete 2.3KN/m³ can reduce permanent loads up to 15%.

From the timber high-rise case studies, it can be extracted that sound transmission between the residential units, and fire resistance are common considerations in timber floor systems, and often are accompanied with research studies in conjunction with authorities. It is not the scope of this report to further analyse these considerations. As an orientative indication, an additional 25mm rigid insulation and 55mm concrete topping

has to be considered on top of the timber structural floor. In the case of Concrete-Timber composite floors, concrete topping can also contribute to the structural system, reducing the amount of concrete needed.



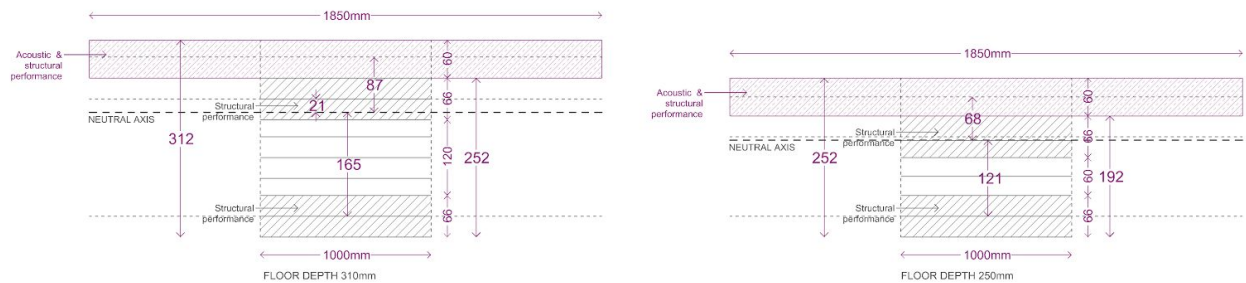
Scheme of concrete behaviour in floor system and potential for timber combination (Own elaboration)

For the purpose of this study, a connection between concrete and timber with ideally rigid behaviour will be assumed.

Ratio Young Modulus Timber (C24), 11000 N/mm² and concrete (C35, cracked), 20000 N/mm² = 1.85

Rigid connection between concrete and timber

Fully-rigid composite action CLT panel



Transformed sections 250+60 (left), 190+60 (right) (Own elaboration)

250 mm CLT + 60 mm concrete (Composite action)

$$\begin{aligned}
 & \left(\frac{60^3 \cdot 1000}{12} + 60 \cdot 1000 \cdot 37^2 \right) \times 20000 = 9,4 \cdot 10^{12} \text{ N}\cdot\text{mm}^2 \\
 & \left(\frac{66^3 \cdot 1000}{12} + 66 \cdot 1000 \cdot (21)^2 \right) \times 11000 = 5,8 \cdot 10^{11} \text{ N}\cdot\text{mm}^2 \\
 & \left(\frac{66^3 \cdot 1000}{12} + 66 \cdot 1000 \cdot (165)^2 \right) \times 11000 = 2 \cdot 10^{13} \text{ N}\cdot\text{mm}^2 \\
 & \underline{2,99 \cdot 10^{13} \text{ N}\cdot\text{mm}^2} \\
 & W = 1,78 \text{ g}\cdot\text{cm}^3 \quad G(230\text{mm}) = 3,26 \text{ kN/m}^2 \\
 & W_{\text{TIMBER}} = \frac{1,78 \cdot (6+0) + 1,78 \cdot 0,6 \cdot (6+1,5 \cdot 0)}{5,01 + 4,135} = 13,3 \text{ mm}
 \end{aligned}$$

190 mm CLT + 60 mm concrete (Composite action)

$$\begin{aligned}
 & \left(\frac{60^3 \cdot 1000}{12} + 60 \cdot 1000 \cdot (63)^2 \right) \times 20000 = 5,9 \cdot 10^{12} \text{ N}\cdot\text{mm}^2 \\
 & \left(\frac{66^3 \cdot 1000}{12} \right) \times 11000 = 2,6 \cdot 10^{11} \text{ N}\cdot\text{mm}^2 \\
 & \left(\frac{66^3 \cdot 1000}{12} + 66 \cdot 1000 \cdot (165)^2 \right) \times 11000 = 2 \cdot 10^{13} \text{ N}\cdot\text{mm}^2 \\
 & \underline{2,6 \cdot 10^{13} \text{ N}\cdot\text{mm}^2} \\
 & W = 2,66 \text{ g}\cdot\text{cm}^3 \quad G(193\text{mm}) = 3,06 \text{ kN/m}^2 \\
 & W_{\text{TIMBER}} = \frac{2,66 \cdot (4,31) + 2,66 \cdot 0,6 \cdot (4,31)}{12,8 + 2,6} = 20,4 \text{ mm}
 \end{aligned}$$

Effective bending stiffness and final deflections for 190 and 250mm CLT panels + 60 concrete (Own elaboration)

Concrete slab	60mm	60mm
CLT panel	190mm	250mm
Bending stiffness (rigid)	2.6e+13Nmm ²	2.99e+13Nmm ²
Structure dead load	0.8 + 1.41kN/m ²	1 + 1.41kN/m ²
Instantaneous deflections	12.8mm	8.56mm
Creep deflections	7.6mm	4.4mm

Total long-term deflections	20.4mm	13.3mm
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From this analysis, it can be concluded that composite action between timber and concrete can provide important benefits in both structural and acoustic aspects, leading to reduced depths for floor systems.

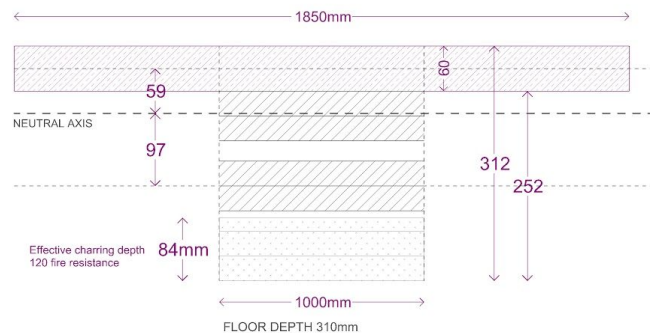
Fire check

120min fire resistance. No sprinklers/encapsulation (most conservative assumption)

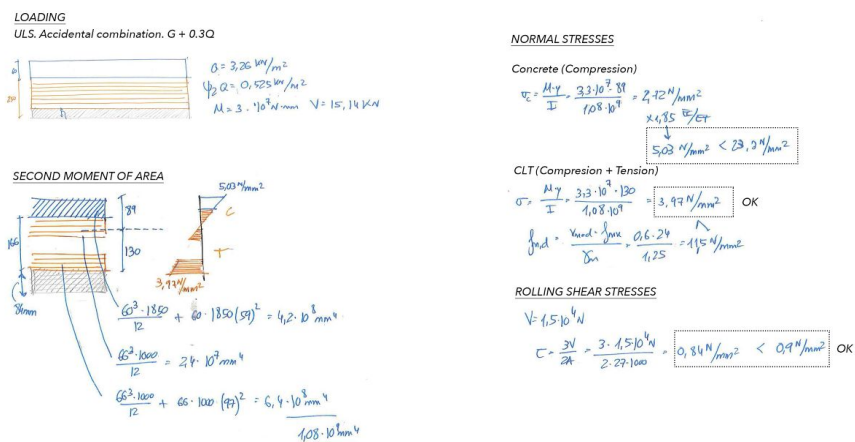
ULS. Accidental loading combination. G + 0.3Q

Charring rate = 0.7mm/min

Effective charring depth = 84mm (If sprinklers are used a reduction of 60min can be achieved, thus 42mm)



Transformed section considering effective charring depth (Own elaboration)



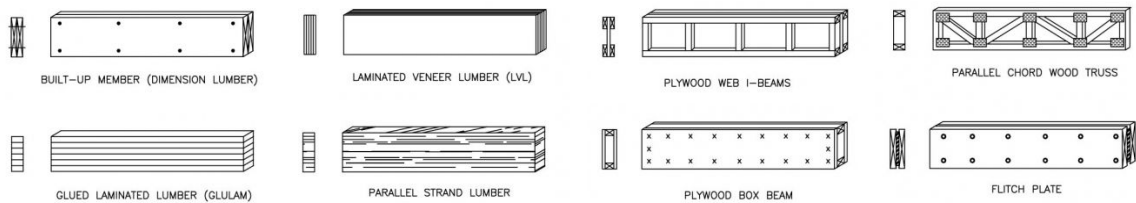
ULS checks for 250mm CLT + 60mm concrete topping (Own elaboration)

From the calculations above, we can extract that rolling shear stresses become the most predominant, regarding structural safety of the floor system in case of fire.

GLULAM BEAMS

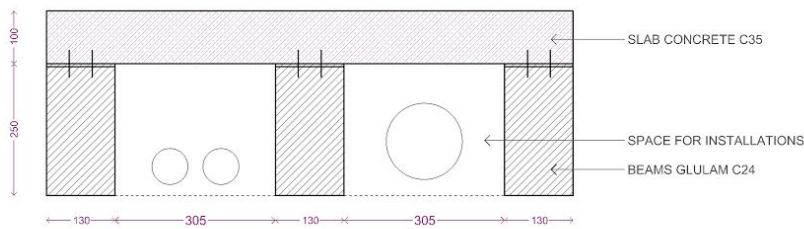
250mm glulam beams + 100mm concrete slab (60% installations space)

As it has been analysed beforehand, combining concrete and timber can create a more efficient system. Another alternative for timber floor systems is the use of conventional timber beams, that can liberate space for installations, and consequently reducing overall floor depth. This can be done in multiple ways, each with advantages and disadvantages regarding manufacturing, costs, and local production technologies. On top of that, the selection of one type of engineered timber beam or another may render very different structural performances, as buildups, materials and composition can be very different.

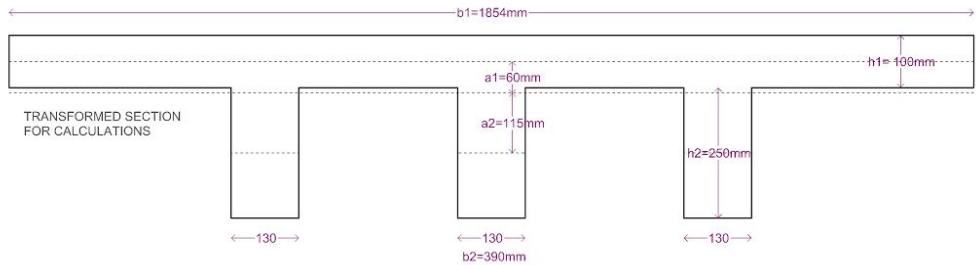


Different types of conventional timber beams (Internachi)

It is not in the scope of this study to compare, all different options. A preliminary indication will be given, regarding the combination of glued laminated beams, with 100 mm concrete slab, for acoustic insulation with an approximate mass/surface of 250 kg/m². Further research must be performed in order to evaluate shrinkage/swelling of the timber elements, composite action between layers, and shear connection between concrete slab and timber beams.



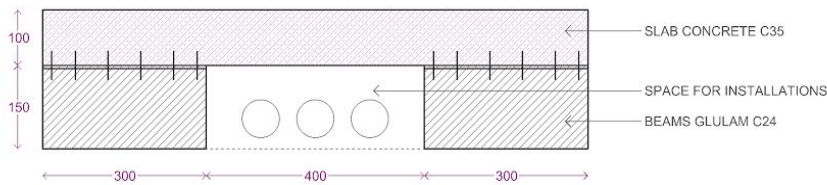
Timber beams and concrete slab section (Own elaboration)



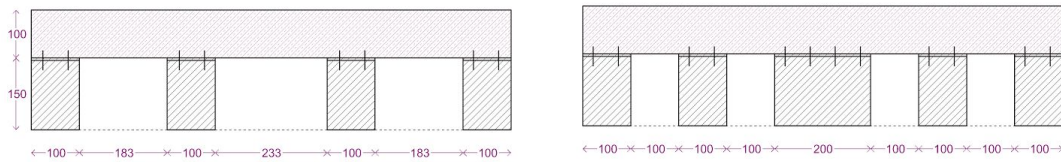
Transformed area for calculations (Own elaboration)

Composite behaviour	No	Yes
Floor depth	350mm	350mm
Bending stiffness (rigid)	$5.66\text{e}+12\text{Nmm}^2$	$2.88\text{e}+13\text{Nmm}^2$
Permanent loading	3.65KN/m ²	3.65KN/m ²
Total long-term deflections	76.3mm (X)	15mm

150mm glulam beams + 100mm concrete slab



Option 1, 300x150 mm glue-laminated timber beams + 100mm concrete slab (Own elaboration)



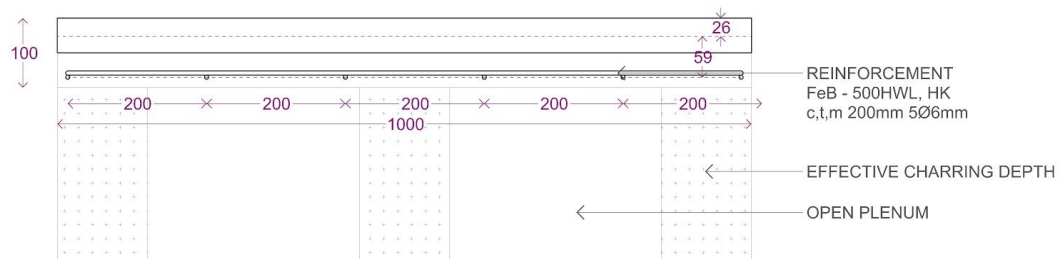
Option 2 (left) and 3 (right), combination of 100x150mm glulam beams with concrete slab (Own elaboration)

Option	1	2	3
Installation space	40%	60% (3 plenums)	40% (4 plenums)
Bending stiffness (rigid)	$1.8 \times 10^{13} \text{ Nmm}^2$	$1.07 \times 10^{13} \text{ Nmm}^2$	$1.39 \times 10^{13} \text{ Nmm}^2$
Permanent loading	3.65 kN/m^2	3.65 kN/m^2	3.65 kN/m^2
Total long-term deflections	24mm	40.4mm	29mm

In conclusion, if the structural floor depth is to be limited to 250mm, additional timber beams should be added, consequently reducing the installation space within the structural floor. For the analysed, 8 meters span, 250mm glulam beams + 100 mm concrete slab may be preferred, allowing appropriate structural and acoustic performance, and enough space for placement of installations.

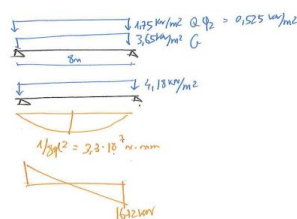
Fire check

Open plenum (most conservative assumption)



Transformed section for reinforced concrete slab (Own elaboration)

LOADING
ULS. Accidental combination, $G + 0.3Q$



CROSS-SECTION
Reinforcement rebars (Shear + tension)
Center to center 200mm. 5x6mm diameter. $A = 141 \text{ mm}^2$

CS
 $\sigma_m = 4.3 \text{ N/mm}^2$
 $\sigma_{ck} = 3.5 \text{ N/mm}^2$
 $\sigma_{ed} = 23.3 \text{ N/mm}^2$ (1.5)
 $E = 32000 \text{ N/mm}^2$
Rebars
FeB - 500HWL, HK
 $\sigma_{yk} = 435 \text{ N/mm}^2$
 $\sigma_{yd} = 378 \text{ N/mm}^2$ (1.15)
 $E = 210000 \text{ N/mm}^2$

NORMAL STRESSES

$E = 30000 \text{ N/mm}^2$
 $\frac{E_s}{E_c} > 7$
 $A = 141 \text{ mm}^2$
 $E = 210000 \text{ N/mm}^2$

$M = 3.3 \cdot 10^3 \text{ Nmm}$

Concrete (Compression)

$\sigma = \frac{M \cdot y}{I} = \frac{3.3 \cdot 10^3 \cdot 26}{104 \cdot 10^3} = 82.5 \text{ N/mm}^2 \rightarrow 107.25 \text{ N/mm}^2 \rightarrow \sigma = 0.04 \text{ N/mm}^2 < 23.3 \text{ N/mm}^2$ OK

Steel rebars (Tension)

$\sigma = \frac{M \cdot y}{I} = \frac{3.3 \cdot 10^3 \cdot 74}{104 \cdot 10^3} = 23.3 \text{ N/mm}^2 \rightarrow 27.9 \text{ N/mm}^2 \rightarrow \sigma = 61 \text{ N/mm}^2 < 378 \text{ N/mm}^2$ OK

SHEAR STRESSES

$\tau = \frac{V \cdot y}{I \cdot A} = \frac{3 \cdot 10^3 \cdot 10^3}{2 \cdot 141 \text{ mm}^2} = 130 \text{ N/mm}^2 < 278 \text{ N/mm}^2$ OK

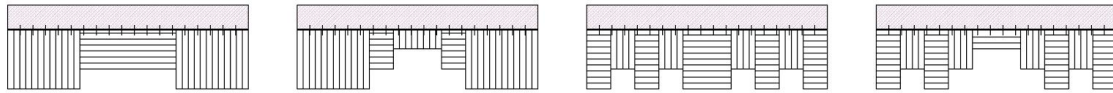
Ultimate limit states checks in case of 120min fire (Own elaboration)

In case of fire, considering the worst scenario, the timber glulam beams will be completely consumed for the fire before 120 minutes. This means that the 100mm concrete slab will have to take all the stresses preventing

the collapse of the floor. As a consequence, additional reinforcement bars of 6mm diameter, spaced 200mm from centre to centre is needed in order to take shear and tension forces.

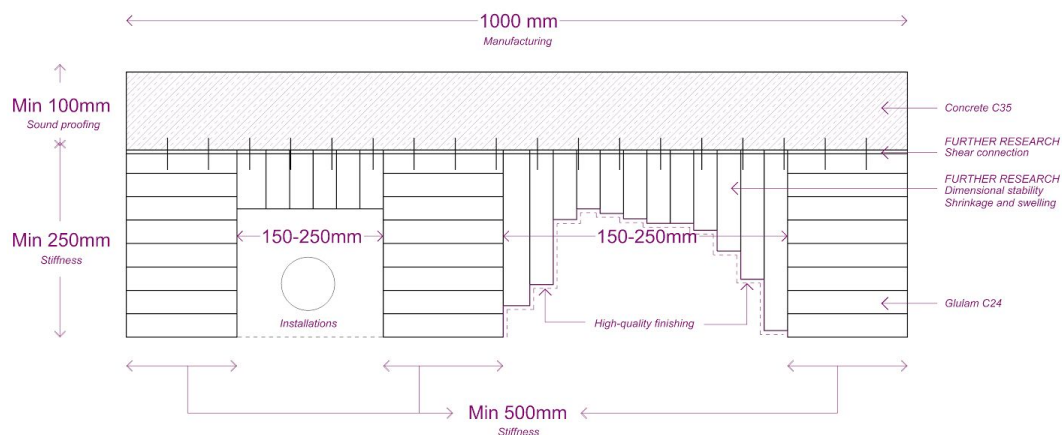
ALTERNATIVE FLOOR SYSTEMS

One of the most important advantages of engineered timber is the capacity to high-quality finishing, that can be left exposed enhancing the interior design. In comparison with traditional concrete floors, this is can be highly desired feature of timber construction. At the same time, it can be complemented with automated manufacturing techniques allowing for multiple design options to be created within reasonable manufacturing costs. The image below shows possible configurations of glulam timber beams with concrete slabs, creating a more sculptural underneath of the floor slab.



Different floor systems buildups exposing timber underneath (Own elaboration)

The image below, summarises different considerations for timber+concrete floor systems. A minimum of 100mm concrete slab should be provided for appropriate sound proofing between different storeys. For acceptable, structural performance, and considering deflections as the most restrictive criteria, 2-4 glue-laminated beams of at least 50% of the floor surface, and 250mm depth. Plenum spaces between glulam beams, can be dedicated to installations or be left exposed emphasizing timber material qualities. Other considerations are dimensional stability, which may lead to problematic construction details, and manufacturing costs.



Shrinkage and swelling

Timber can expand and decrease its volume depending on the surrounding moisture. When there high humidity, internal microcells inside wood get filled with water and consequently swell. Moreover due to the anisotropy of the material, these movements are higher in the transverse direction of the fibres, in the order of 20:1, compared to the longitudinal direction. These changes in dimensions need to be considered in the buildups of the floor systems and in the construction details.

Conclusions

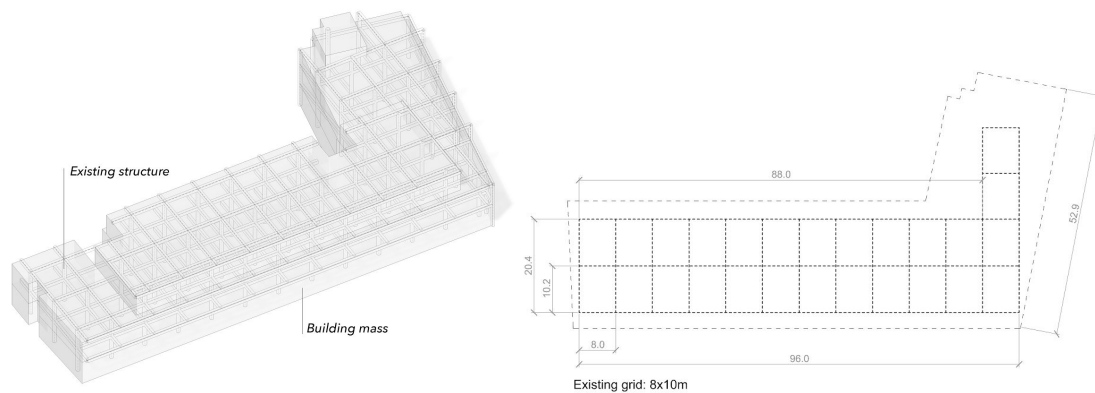
Three options timber floor systems:

- Lignatur (Approx 250-280mm)
- CLT + Concrete topping 60mm (Approx 250-280)
- Glulam beams + Concrete slab 100mm (Approx 300-350mm)

Glulam beams + concrete slab 100mm, meets all requirements, sound insulation, fire resistance, and structural performance, and allow placement of installations inside the structural depth. Moreover, it can provide alternative solutions for ceilings showing the potential of timber for interior design.

GRAVITY SYSTEM

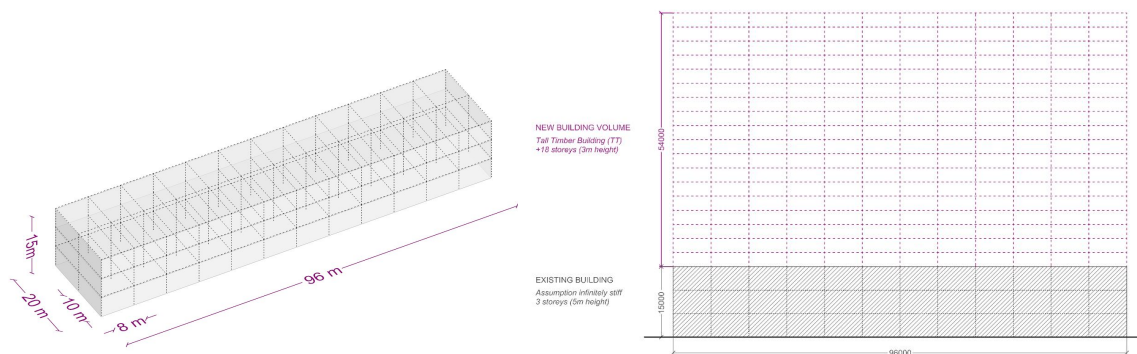
- Homogenisation section
- Buckling
- Wall - No openings
- Lintel design
- Wall - Openings



Ter Meulen existing building mass and structure (Own elaboration)

With the purpose of clarifying the implications of a Tall Timber Extension, first a simplification of the existing building volume is assumed. Ter Meulen building contains 3 storeys of approximately 5 meters each of them, resulting in 15 meters in total. A 70m TT, Tall building, would result in 55m, TE, Tall Extension (discounting 15m height of existing building).

In this chapter, for ease of the calculations and understanding of the implications of Tall Timber structure, the existing structure will be assumed as infinitely stiff, and will act as a higher foundation for the TTE.



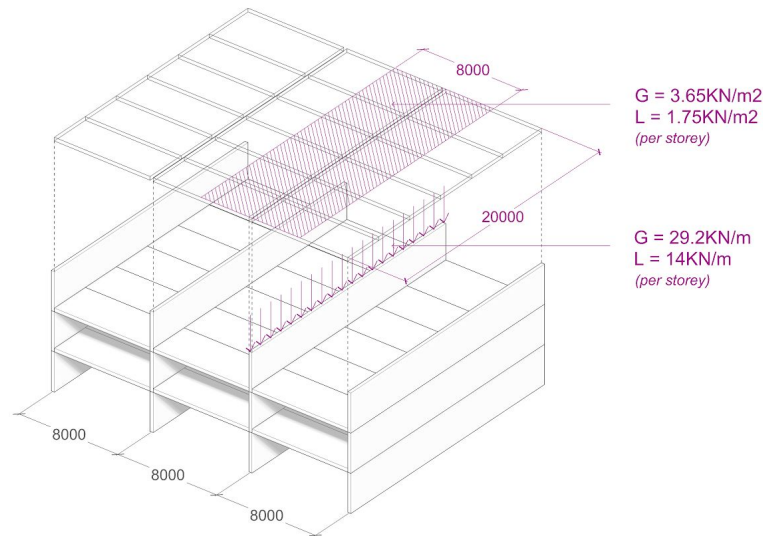
Preliminary simplification for design study (Own elaboration)

Loading scheme

Floor system (8 meters span) - Concrete-timber composite

Total permanent loading (G) = 3.65KN/m²

Live loading (L) = 1.75KN/m²



Loading distribution over wall (Own elaboration)

In any kind of buildings there are two different types of actions, Permanent (G) due to the dead weight of the materials, and Variable (Q). The last component cannot be predicted with total accuracy, and it is only an estimation of the imposed load that the structure will have to support due to use, changes, etc.

At the same time, the loading is multiplied by partial safety factors, γ 1.35 for Permanent loading and 1.50 for variable. Lastly, there is an extra variable load factor, ψ that accounts for variable loading combinations, assuming that not 100% of the loading will happen simultaneously, for instance, it is very unlikely that an earthquake, high winds and snow will occur together.

In tall buildings, multiple storeys are piled up together and each of them is multiplied by an unknown variable load (Q), and partial safety factor γ . According to Eurocode, for residential uses, load combination factor ψ , can be taken as 0.5 for all the floors except for the roof.

γ Partial safety factor for loading	1.35/1.2 Permanent loading (G)
	1.5 Variable loading (Q)
ψ Variable loads combination factor	1.0 One level
	0.5 (All floor levels, residential)

Loading combination factors (EC1)

Ultimate Limit States (ULS) can be calculated with the following expression:

$$\gamma_g \cdot G_k + \gamma_q \cdot Q_1 + \sum \gamma_{q,i} \cdot \psi_0 \cdot Q_i$$

$$1,35 \cdot G_k + 1,5 \cdot Q_1 + \sum 1,5 \cdot 0.5 \cdot Q_i$$

However, this assumption is a little more complicated because timber strength decreases with the duration of the load and the surrounding moisture, as has been stated previously. Kmod factor for permanent loading in Service Condition 1 or 2 is 0.6, and for imposed/live loading 0.9. Usually, this factor is applied in material strength formula, making it difficult to compare different actions with different Kmod factors.

Kmod	Design compressive stress
1 (Theoretical)	16.8N/mm2 (*)
0.9 (Short-term)	15.12N/mm2
0.6 (Permanent)	10.08N/mm2

Design compressive stresses for CLT panel with C24 timber, applying material safety factor of 1.25

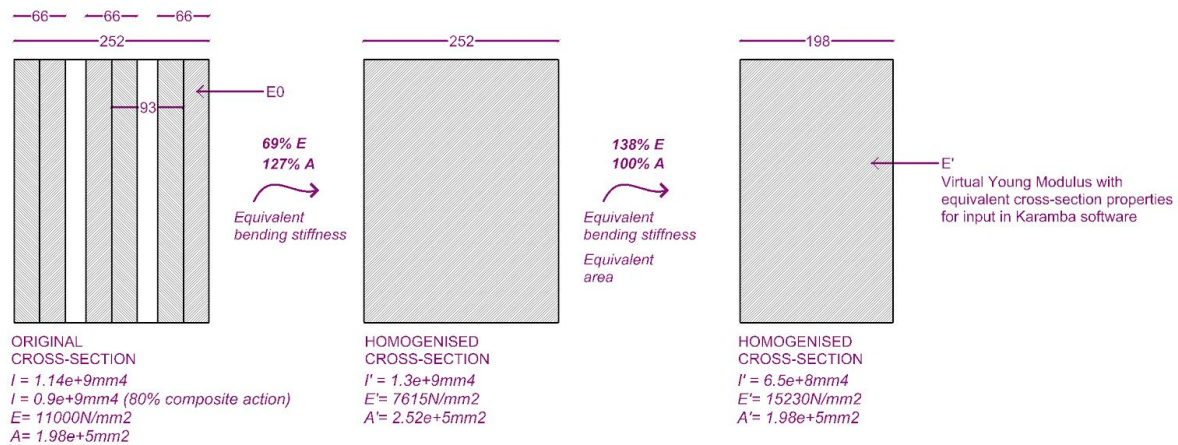
(*) Nevertheless it can also be applied in the loading combination expression, assuming a theoretical kmod factor of 1, and taking into account the duration of each type of action, in the following manner:

$$1,35 \cdot \frac{1}{0.6} \cdot G_k + 1,5 \cdot \frac{1}{0.9} \cdot Q_1 + \Sigma 1,5 \cdot 0.5 \cdot \frac{1}{0.9} \cdot Q_i$$

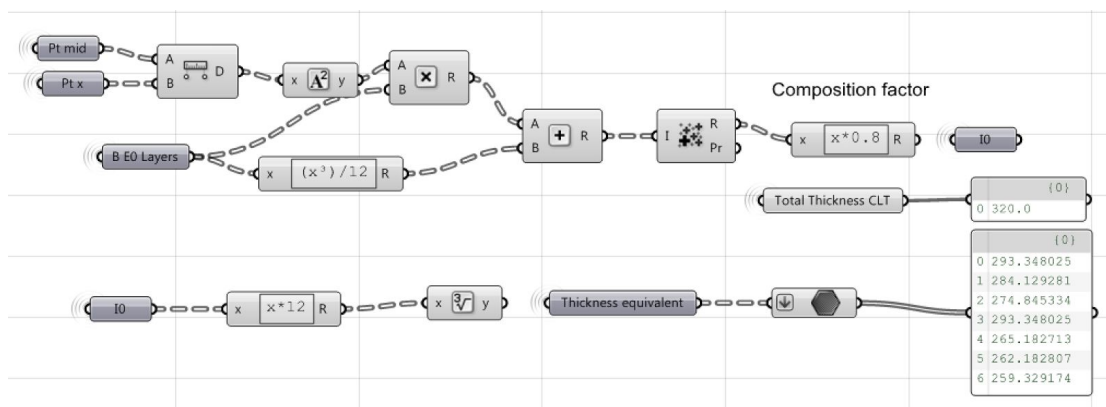
Homogenisation of cross-section

CLT panels are composition of layers. This means, that in the cross-section only the layers with E0 are assumed to take the loading parallel to the grain. However, most softwares usually require the input of a homogeneous cross-section for the structural calculations. This can be done with the Theory of Homogenisation of Section, in which a fictional cross-section and material is created, with equal bending stiffness to the original cross section.

$$E \cdot I = E' \cdot I'$$



Homogenised cross-sections for input in Karamba software (Own elaboration)

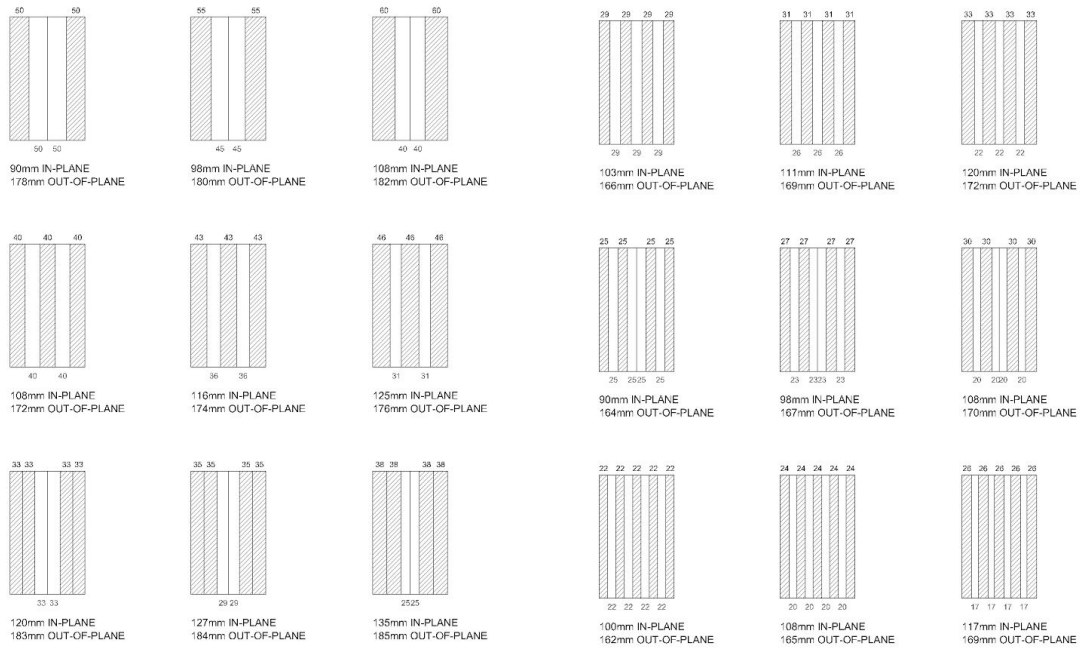


Parametric calculation of sandwich and homogenised cross-section (Own elaboration)

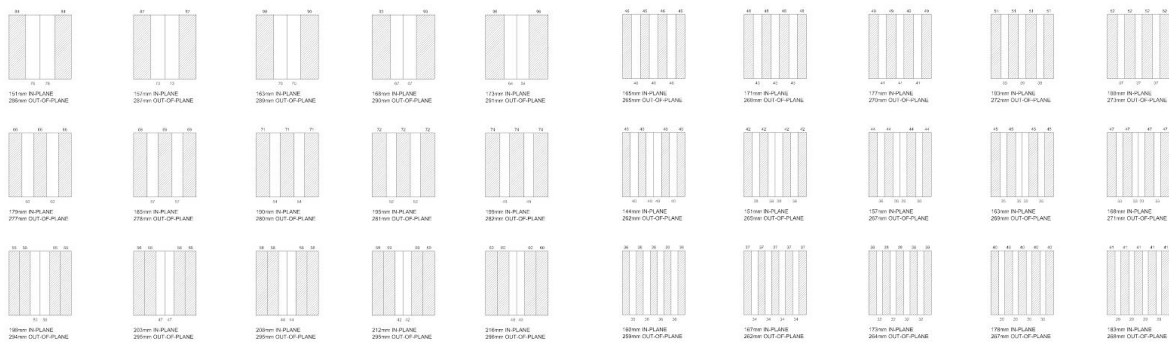
- Assumption of 90% composite action of In-plane loading depending on the effective area of lamellas parallel to grain

- Assumption of 80% composite action for Out-of-plane loading depending on the effective second moment of area of lamellas parallel to grain

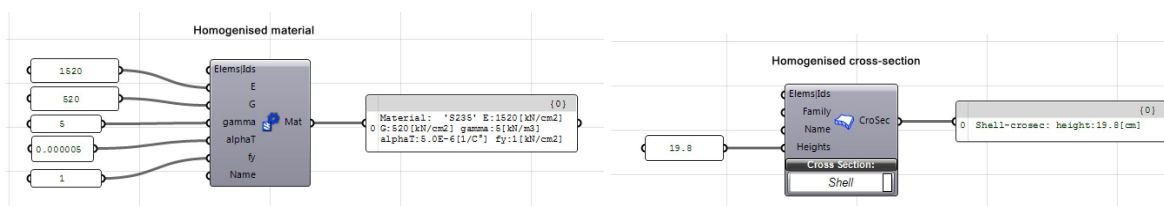
Thickness 200mm - C24



Thickness 320mm - C24



Equivalent homogenised CLT cross-section for structural calculations

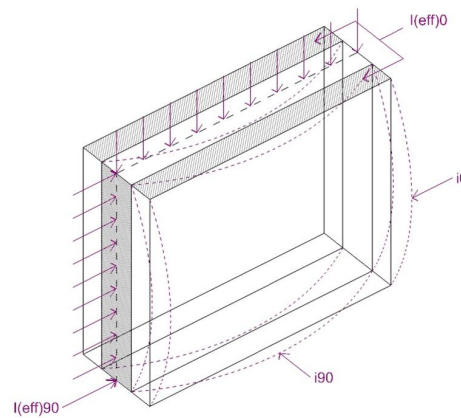


Homogenised cross-section and material for Karamba

Buckling

Buckling can be calculated according to Euler's critical load, where F = critical force, EI , bending stiffness, L , length of unsupported column, K , effective length factor.

$$F = \frac{\pi^2 EI}{(KL)^2}$$



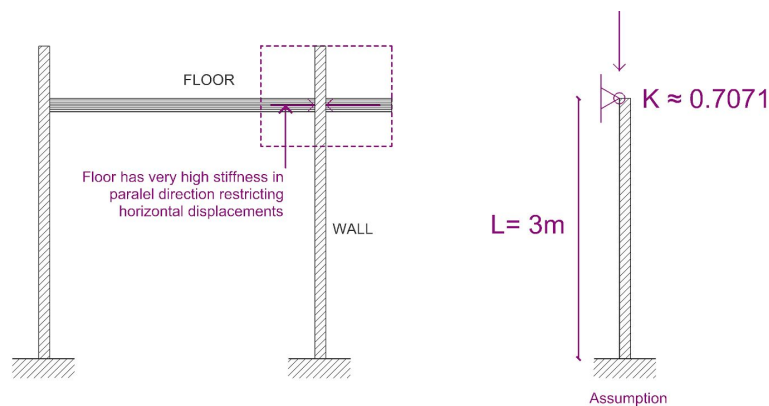
Scheme of buckling modes in CLT panel (Own elaboration)

Wall panels can also be designed using the Composite Theory (Blass 2005) to calculate the effective section properties for combined axial and bending stresses. Predefined tables provided by the manufacturers contain cross section values and modification factors for several types of panels see tables provided by Finnforest (Appendix 2).

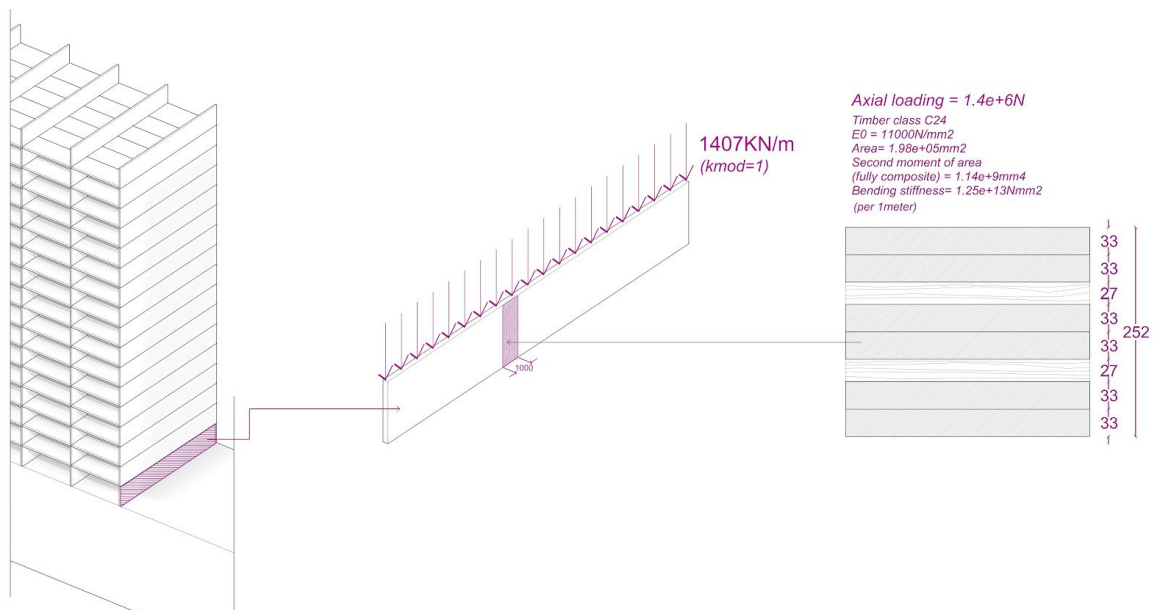
In order to account for composite effect, 80% decrease in bending stiffness of the original cross-section has been considered.

$$\sigma_{c,0,d} = \frac{N_d}{A_{net}} \quad i = \sqrt{\frac{I_{ef}}{A_{net}}}; \quad \frac{\sigma_{c,0,d}}{k_c * f_{c,0,d}} \leq 1$$

Where A_{net} is the cross section of the layers running parallel to the height of the panel, i radius of gyration, I_{eff} effective moment of area based on full cooperation between layers, and h is the height of the panel. The buckling behaviour of the wall can be considered over one floor-height, with one end pinned due to the high stiffness of the floor.



Bottom wall - No openings

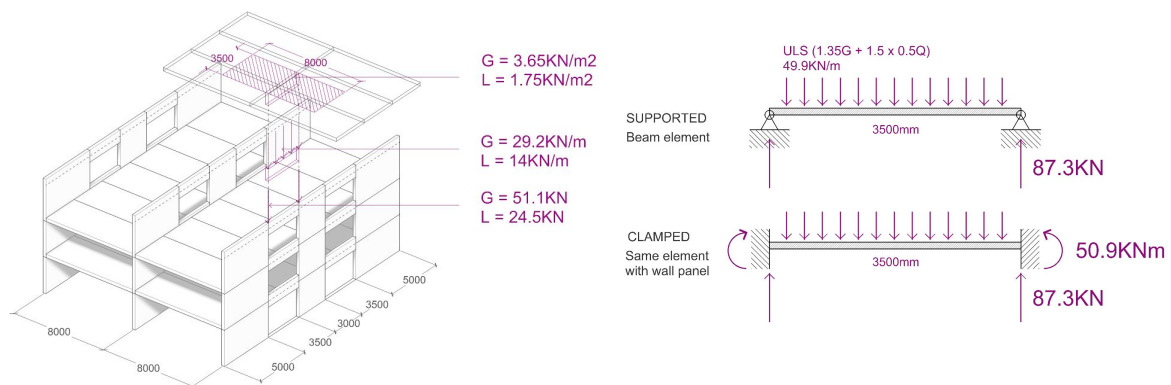


Hand calculations

Compression stresses	Buckling critical loading
7.07 N/mm ²	3.43e+6 N (rigid)
	3.19e+6 N (Composite)
0.42 OK	9.79/8.16 OK

Summary of hand calculations

Lintel design



Lintel design loading scheme (Own elaboration)

As it is necessary to create openings, a beam-lintel element must be used. Basically, there are two main types of connection with the wall element, and they are related to manufacturing of the different pieces:

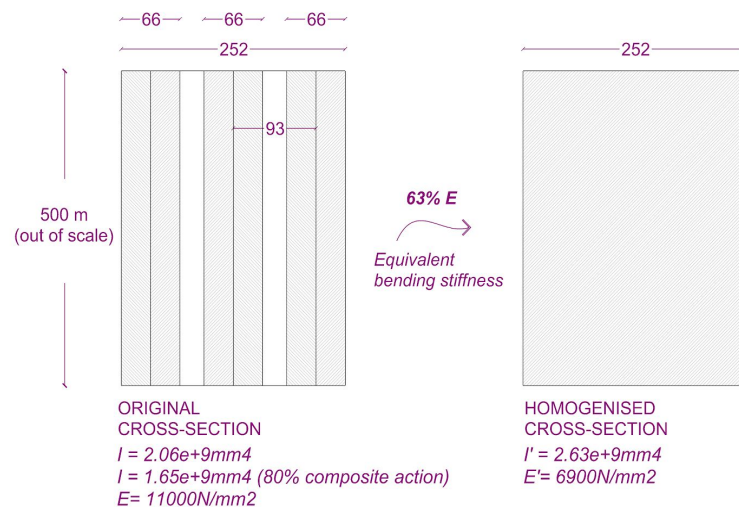
- If the lintel forms one piece with the CLT wall, it can be assumed as a clamped connection

- If the lintel is not the same piece with CLT wall (e.g. connected by steel elements), it is very difficult to achieve a completely rigid connection. In this case, an intermediate behaviour between simply supported and clamped is assumed as "semi-rigid connection"

Depth of lintel beam	Bending stiffness (reducing 80% composite action)	Final deflections (including creep) SIMPLE SUPPORTED	Final deflections (including creep) CLAMPED
1000mm	1.452e+14Nmm ²	0.87mm OK	0.174mm OK
500mm	1.815e+13Nmm ²	6.492mm OK	1.30mm OK
400mm	9.3+13Nmm ²	13.6mm	2.72mm OK

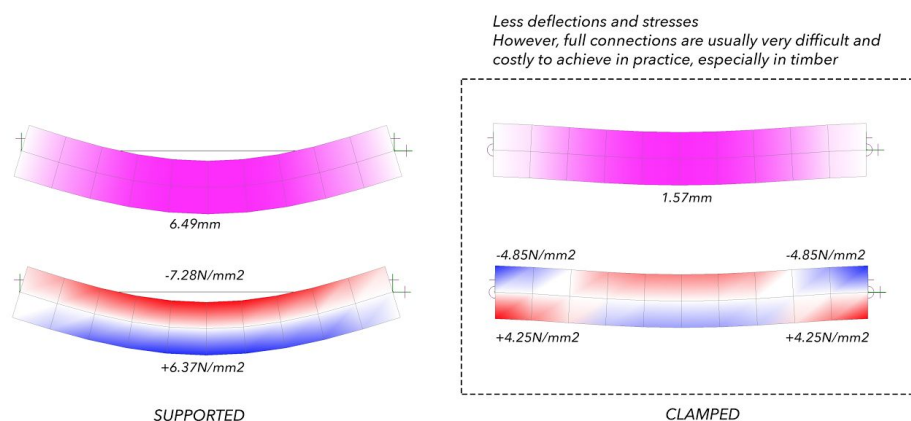
Summary of hand calculations. Deflection limit $L/400 = 8.75\text{mm}$

From the table above, it can be stated that there is a clear distinction in the lintel deflections depending on the type of connection used.



Homogenised section for lintel beam (Own elaboration)

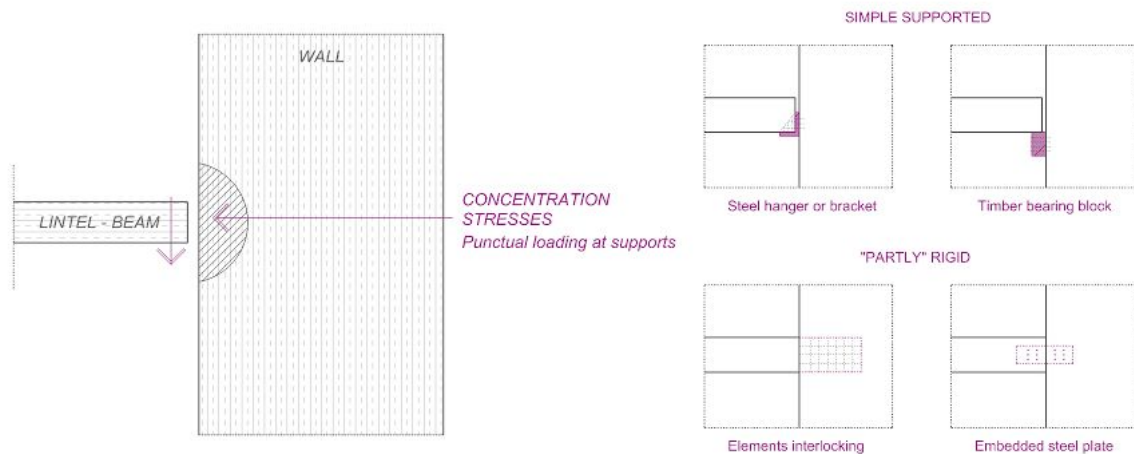
In order to account for long-term deflections, an additional 82% decrease of the Young Modulus has been assumed. (Deducted from hand calculations)



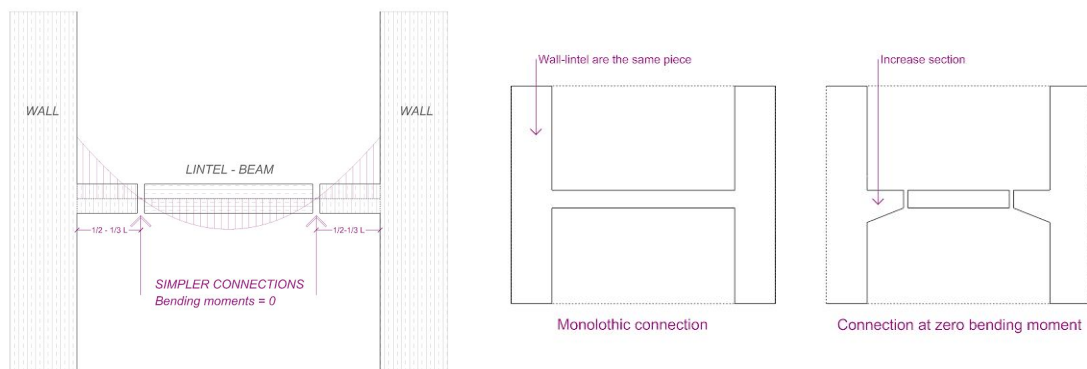
Computer calculations for simple and clamped connections of lintel-beam

In practice, fully rigid connections are very complex and costly to create. Therefore, an intermediate approximation "partly rigid" should be taken, stiffness = 0.5

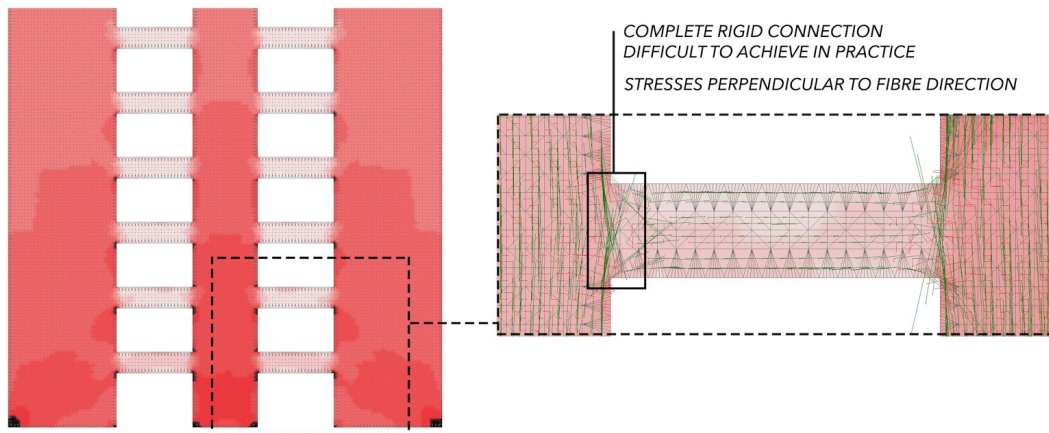
In timber structures the serviceability and durability of the structural elements, depends mainly on the connection type. At the same time other considerations, such as aesthetics, cost-efficiency and fabrication process play a significant role. Needless, to say a "theoretical" clamped connection can be highly beneficial from a structural point of view (stresses and deflections), however, it is usually very complex and costly to make in practice. One solution developed by SOM in the Timber Research Project is to use concrete at the edges of timber panels and beams (link and spandrel beams) creating a rigid connection. Other connection systems involved glue-in rods or intermediate steel elements. It must be noted here, that there is a great myriad of connection details, each with disadvantages and advantages. In the image below, 4 conventional connections types are schematised



Scheme of four possible connection types between wall-lintel beam (Own elaboration)



Monolithic dintel (left) and connection at $L/2-3$ for the span for simpler connection

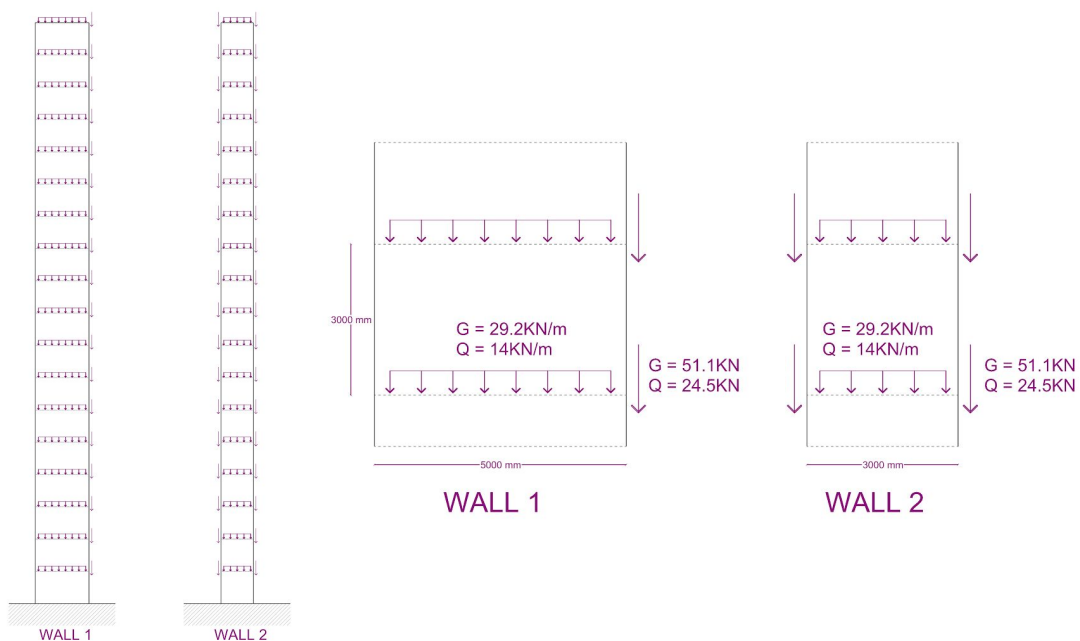


Stresses perpendicular to fibres in "clamped" connection in lintel

The introduction of wall openings leads to:

- Increase of normal stresses and buckling critical wall in central wall (This may not be a problem when wind loading is applied)
- Perpendicular to fibre stresses in lintel-beam

Wall design - Openings



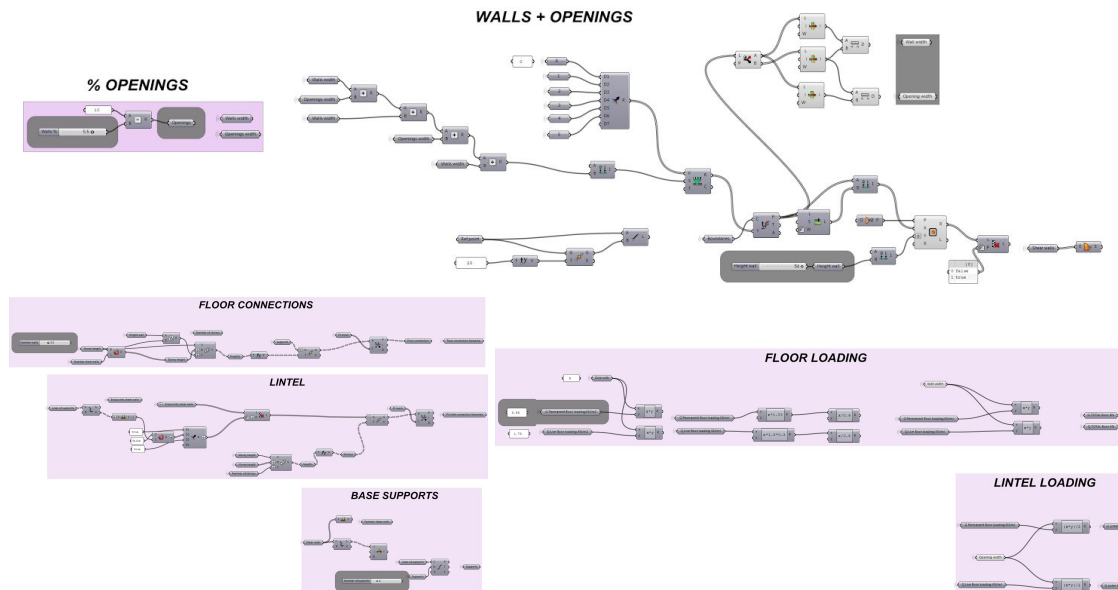
In order to test the implication of openings in the stresses of the walls, a parametric study was created. In this way multiple options regarding combination of loading, Gravity + Wind loadings, % of openings, height and cross-section dimensions, can be analysed.

Height = 70m

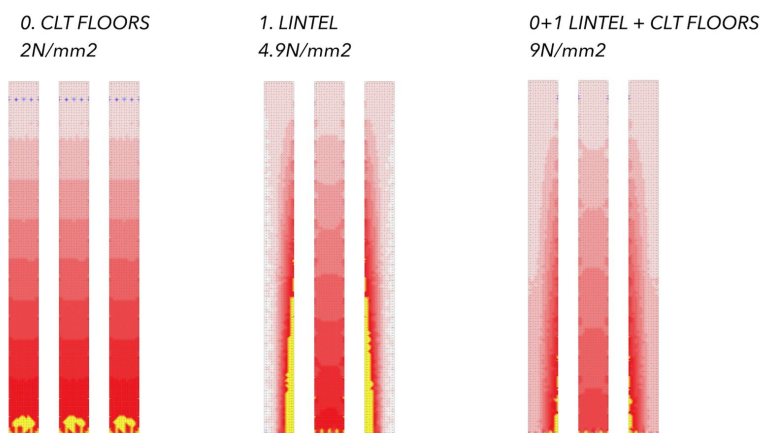
Cross-section = 198mm

Openings = 40%

Walls = 4.6 meters / Openings = 3 meters



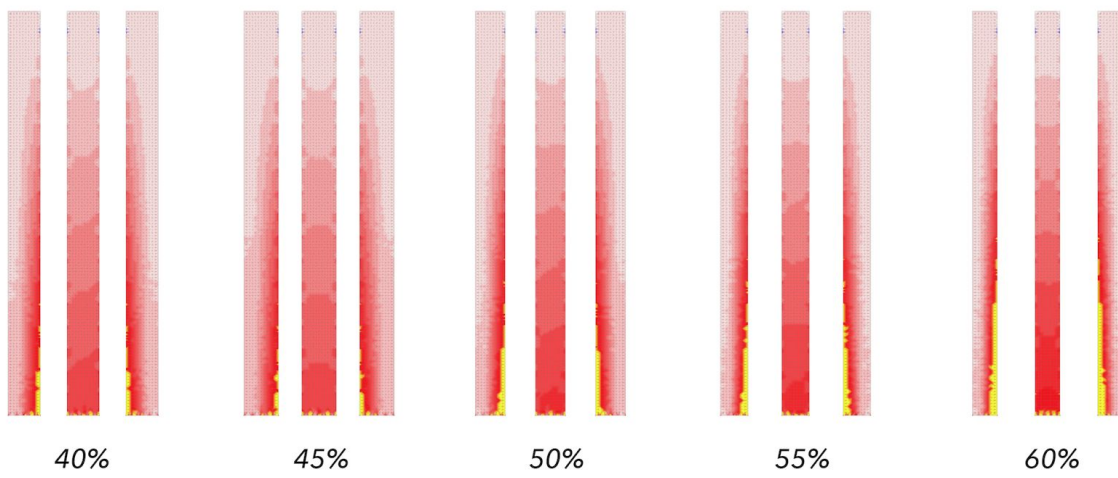
Creation of parameterized geometry and loading values for input in Grasshopper and Karamba
(Own elaboration)



From the above structural diagrams, it can be seen that the loading induced by the lintel, creates asymmetric distribution of normal stresses in the walls. This may not be an issue if lateral loading is considered.

Openings	Wall dimension	Opening span	Compressive stresses	Buckling critical load
40%	4.6m	3m	9N/mm ²	x14.2
45%	4.3m	3.5m	10N/mm ²	x10.7
50%	4.0m	4.0m	11.1N/m ²	x7.7
55%	3.7m	4.5m	12.7N/mm ²	x5.4
60%	3.3m	5m	13.1N/mm ²	x3.7

Parametric study for openings impacts in critical structural factors



Study of different proportions of openings and wall for gravity loading (Own elaboration)

12. LATERAL RESISTING SYSTEM

Wind loading

The building is located in Zone II, in an urban area

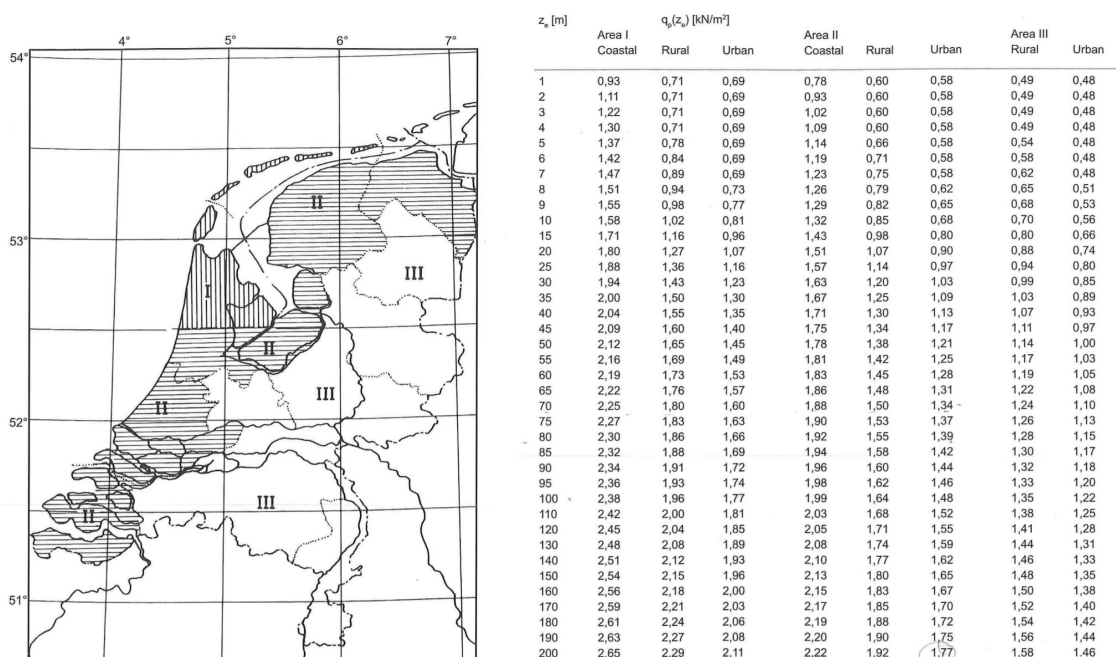
$$F_i = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref}$$

F_i = Wind force on a structure or structural component [kN]
 $c_s c_d$ = Structural factor¹ [-]
 c_f = Force coefficient for structure or structural component [-]
 $q_p(z_e)$ = Peak velocity pressure at reference height² z_e [kN/m²]
 A_{ref} = Reference area on structure or structural component [m²]

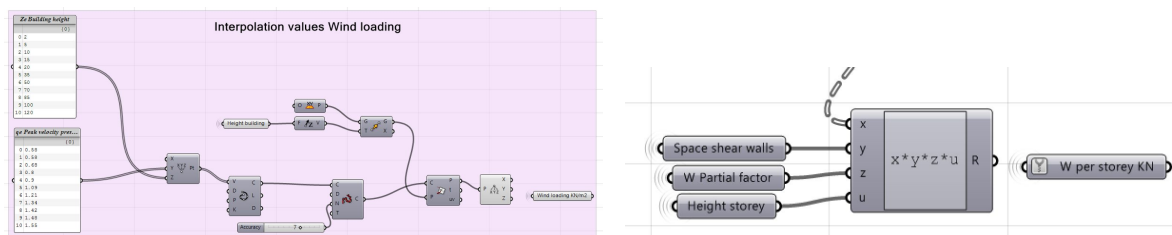
1 The structural factor $c_s c_d$ can be taken as 1 for regular, low-rise buildings, for more information see NEN-EN 1991-1-4

2 For high buildings ($h > b$) the velocity pressure may be assumed to decline towards ground level, for details see NEN-EN 1991-1-4

Simplified wind loading formula (NEN-EN 1991-1-4)



Wind regions in The Netherlands (left) and peak velocity pressures (right) (NEN-EN 1991-1-4)

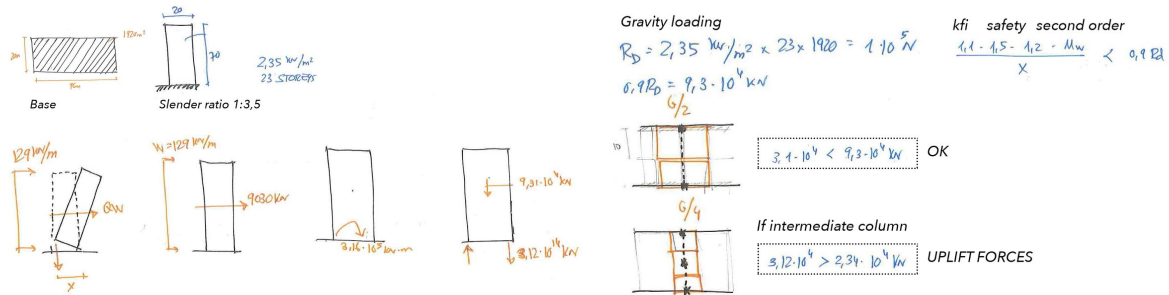


Interpolation of wind loading for parametric input in Grasshopper (Own elaboration)

Distribution of wind loading per storey. Half of the storey, assuming no creep influence

The building is situated in the city of Rotterdam wind region II, according to national codes, in an urban area. As a consequence a peak wind pressure of 1.34 kN/m² has been estimated. Wind loading have been assumed as a linearly distributed load applied uniformly over the total façade area of the building 20 x 96 meters width, and 70 meters height. Lateral loading is equally distributed in all the shear walls.

Lateral stability



Rough calculations for lateral stability, considering vertical support in the middle and at the extremes of the building (Own elaboration)

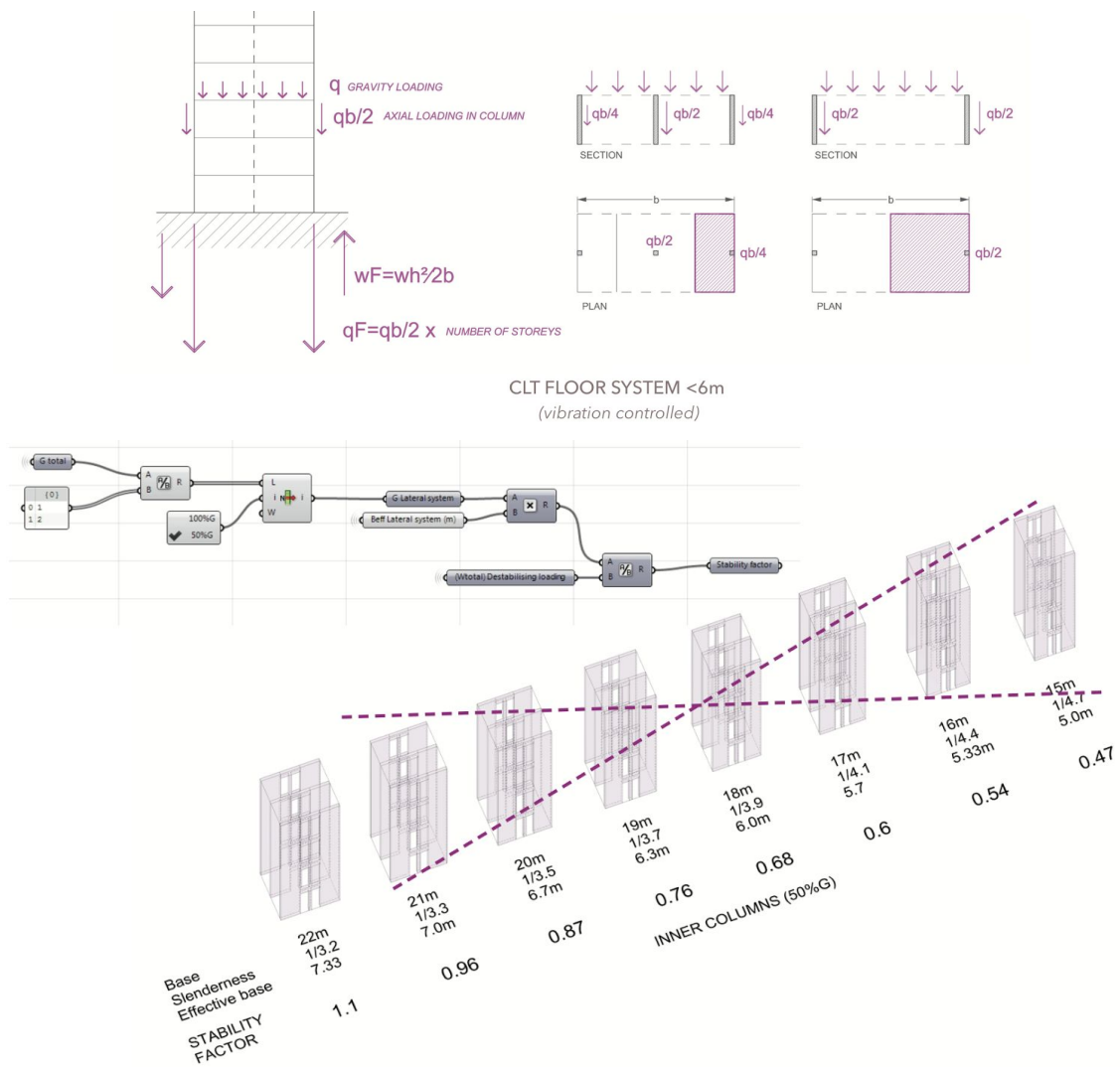
No inner columns



In order to avoid uplifting forces at the base of the Tall Timber building, the following recommendations are to be considered:

- No inner columns taking away gravity loading from perimeter columns
- Base depth > 16 meters
- Slender ratio > 4.4

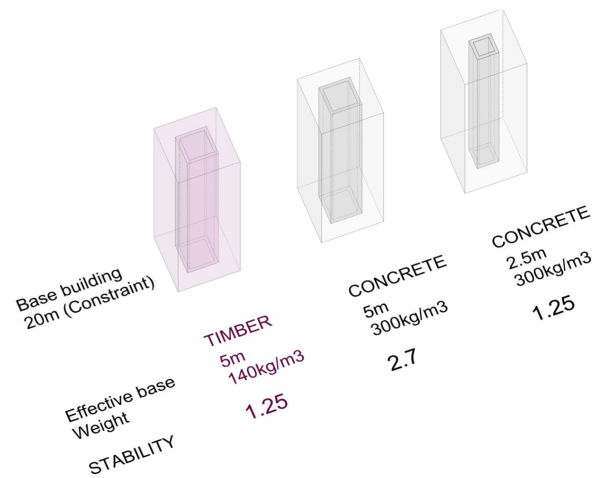
One inner columns (50% gravity loading)



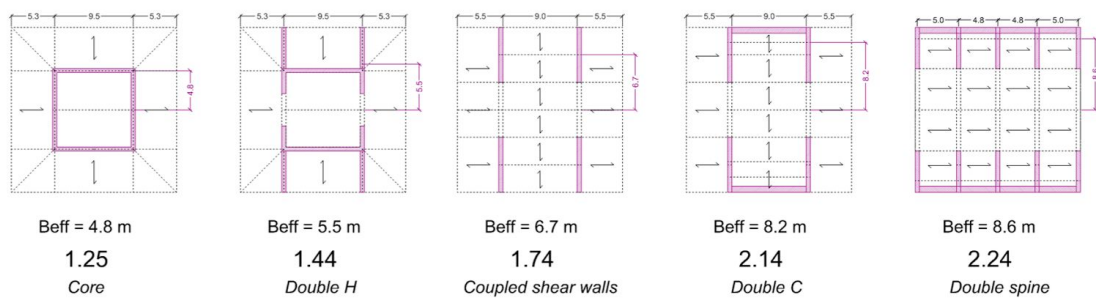
In order to avoid uplifting forces at the base of the Tall Timber building, the following recommendations are to be considered:

- One inner column takes away 50% gravity loading from perimeter columns
- Base depth > 22 meters
- Slender ratio > 3.2

Comparison timber - concrete

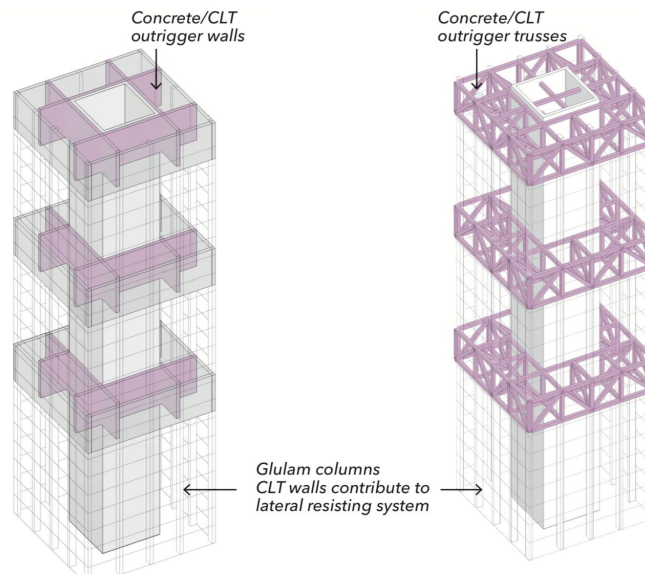


Wing shear walls



- Up to 90% more efficient engaging perimeter with wing shear walls

Outriggers activating perimeter columns



Lateral stiffness

- Wind will be considered as short-term loading, consequently deformations due to creep will not occur.
- Mean material values for shear and elasticity moduli will be considered for SLS

Shear walls act as cantilever beams that resist lateral loading mostly through bending stiffness. With high length to depth ratios, shear deformation have small impact on the total deflection, and can thus, can be neglected.

- For the effective bending stiffness only lamellas with fibres axially loaded (parallel to acting stresses, i.e. boards oriented vertically) will be considered.
- Dynamics and second order effects for deflections have been estimated according to expression

$$w_{H;m} = \frac{qH^4}{6EI_{base}}$$

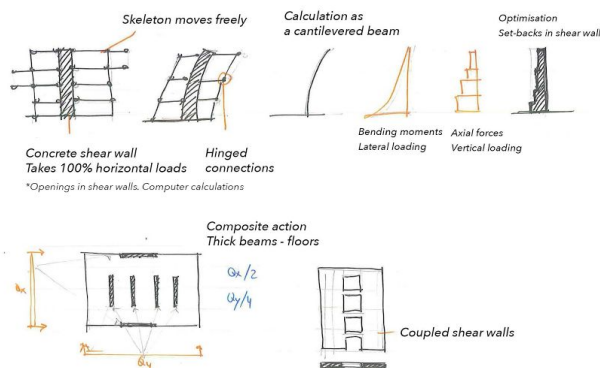
- Maximum lateral deflection at the top 82mm (H/850), accounting for rotational stiffness in foundations

Serviceability limit states (SLS) can be calculated with the following expression:

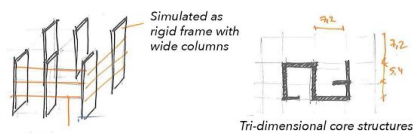
$$G_k + \gamma \cdot Q_1 + \Sigma \cdot \psi_0 \cdot Q_i$$

Traditional structural systems for tall buildings (concrete and steel)

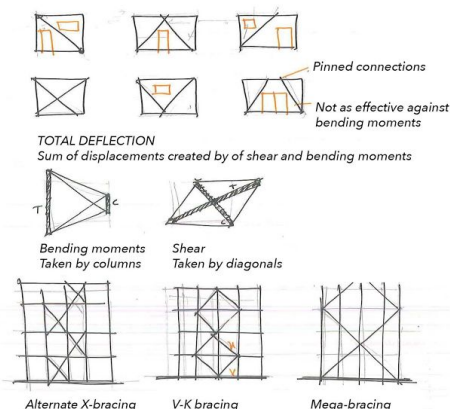
SHEAR WALLS - Up to 35 storeys



COUPLED SHEAR WALLS - Up to 50 storeys



TRUSSED FRAMES - Up to 50-60 storeys

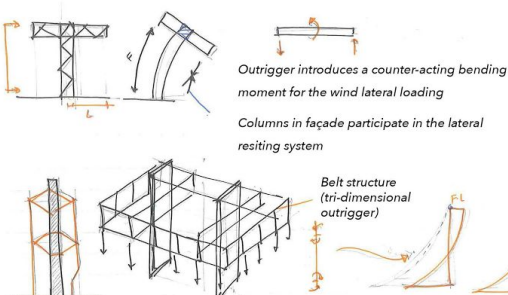


INFILLED FRAMES

Behaviour similar to trussed frames
Calculation method, as diagonal in compression

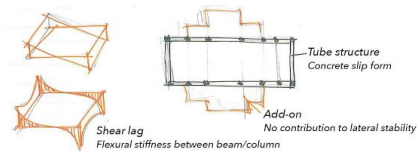


OUTRIGGERS



Example. De generale Bank (The Netherlands)
Set back in façade

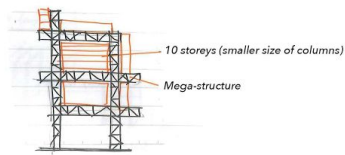
TUBE STRUCTURES - Up to 35 storeys



(Minimise shear lag by using equally columns in façade)



MEGA-STRUCTURES

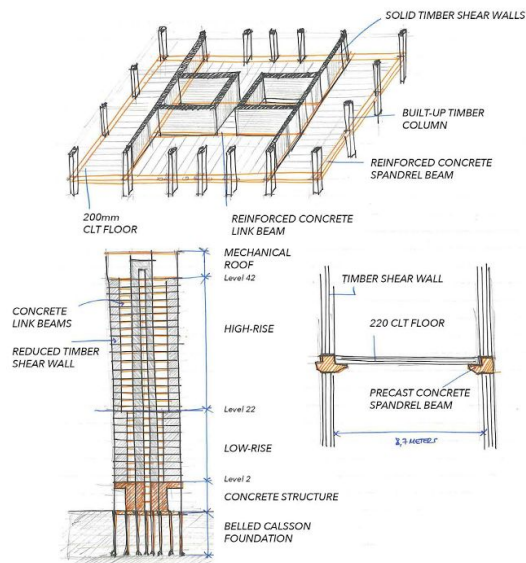


COMMON SIMPLIFICATIONS FOR CALCULATIONS

- Structural elements behave elastically, and principle of superposition can be applied
- Floors have infinite flexural stiffness in their own plane
- Out of plane bending stiffness of floors and shear walls can be neglected.
- Contribution from stiffnesses of non-structural elements can be neglected
- Foundations have infinite rigidity
- Torsional and warping stiffness of beams, columns, and planar walls are neglected
- Shear deformation in slender elements (length/width > 5) neglected
- Structural connections are either fully resistant or hinged
- The effect of cracks in reinforced concrete can be included by reducing second moment of area: 50% for beams and 80% for columns

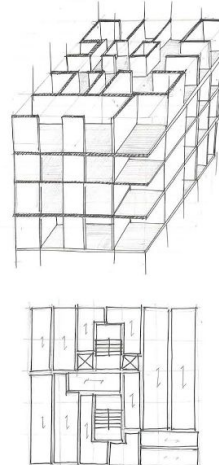
Structural systems for tall timber structures

CONCRETE-TIMBER (SOM 2013)

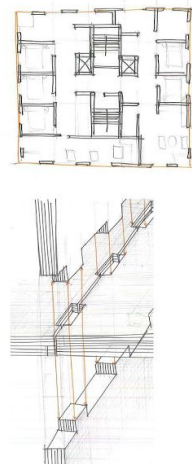


CELLULAR CLT

HONEYCOMB STRUCTURE
- Plate bracing of shear walls in three directions



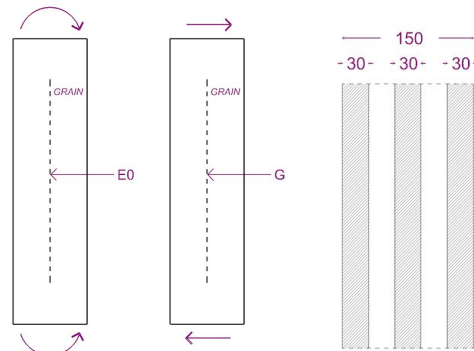
ERECTION
- 2 CLT panel types
- Mobile crane
- 6 truckloads every 3 days



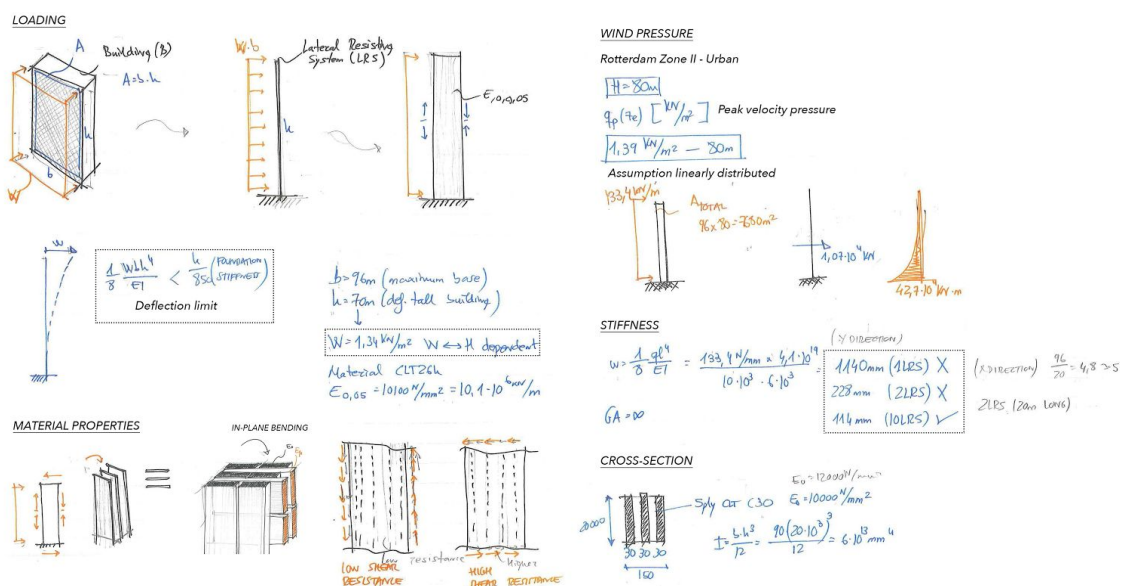
In conclusion, three structural typologies can be utilised with timber structures:

- Coupled shear walls with CLT panels (They can also be coupled with concrete)
- Glulam trussed frames, with steel details

Shear walls CLT 150mm



Scheme of material values for cross section(right) (Own elaboration)

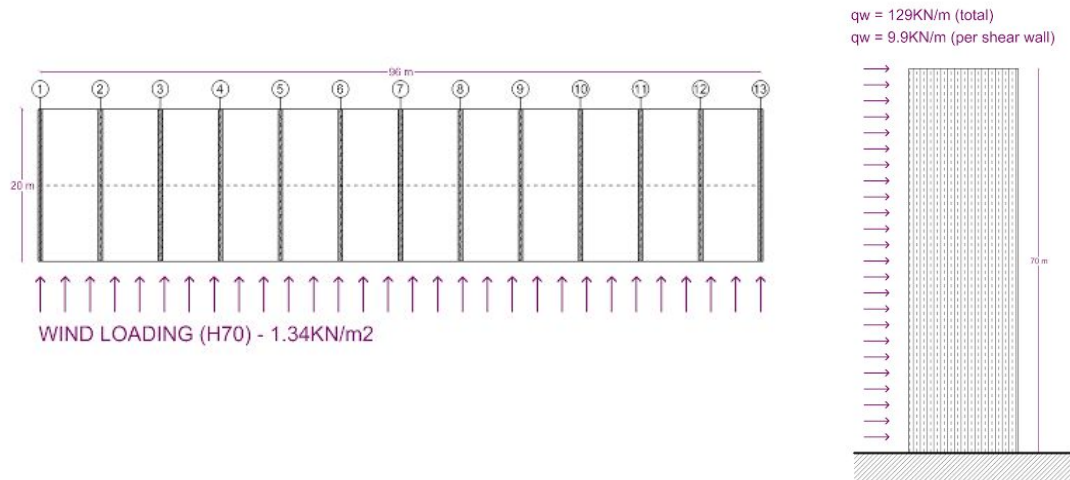


Calculation of wind pressure, cross section CLT and number of shear walls needed (Own elaboration)

Behaviour CLT panel	Effective bending stiffness Nmm ²	Hand calculations (mm)	Computer calculations
Fully-rigid	1.45e+18	44.43 mm	44.1mm
Blass, 2005	1.29e+18	49.92 mm	49.94 mm

Further research. Storey index have effect, non-uniform wall up to the height.

The above calculations, are preliminary indications of the minimum number of shear walls that are necessary in the building. Nevertheless, this is only a quick assumption, as 20 meters long shear walls are not suitable for the architectural layout. For this reason, a further study about possibilities of making openings in the CLT shear walls panels will be considered. From the floor system calculations, we can conclude that reasonable spans are in the order of 8 meters within the existing grid. The below image, show a possible structural layout, integrating gravity and lateral resisting systems in 8 meters repetition, resulting in 13 shear walls, with 9.9KN/m lateral loading.



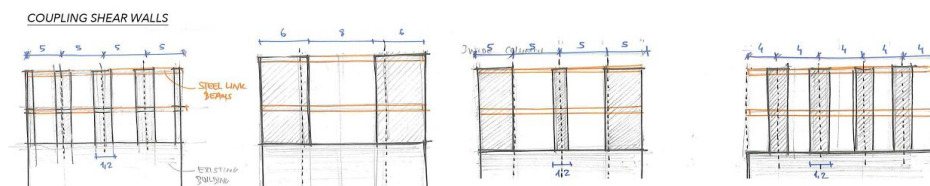
Schemes for lateral loading (Own elaboration)

- At least 13 shear walls (150mm CLT) running the full depth of the building are needed.

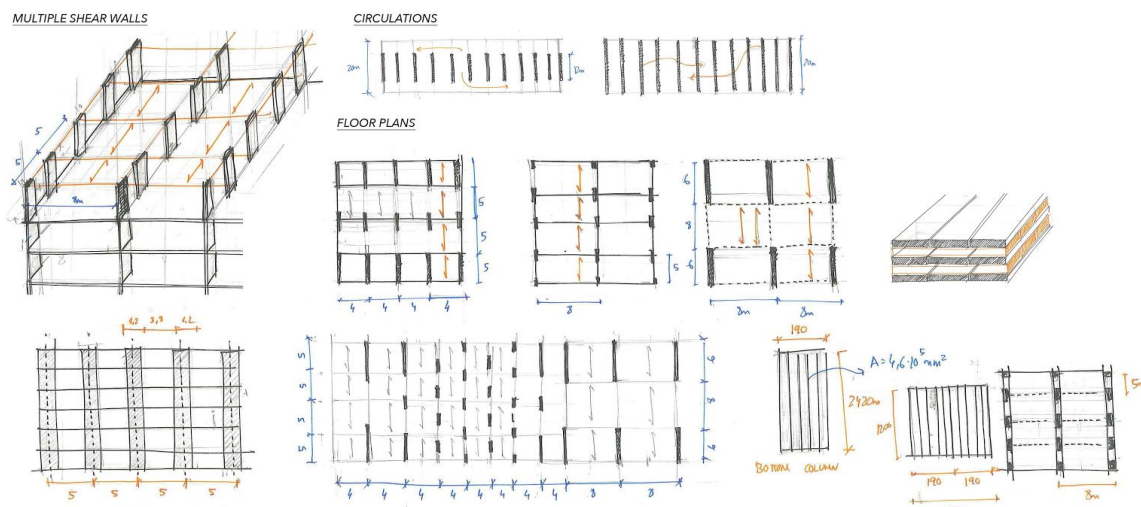
This has a great impact on the architectural layout. The following strategies can be taken:

- Option 1. Shear walls with openings for circulations and more flexibility
- Option 2. Cores
- Option 3. Use stiffer timber or other material (concrete, steel)

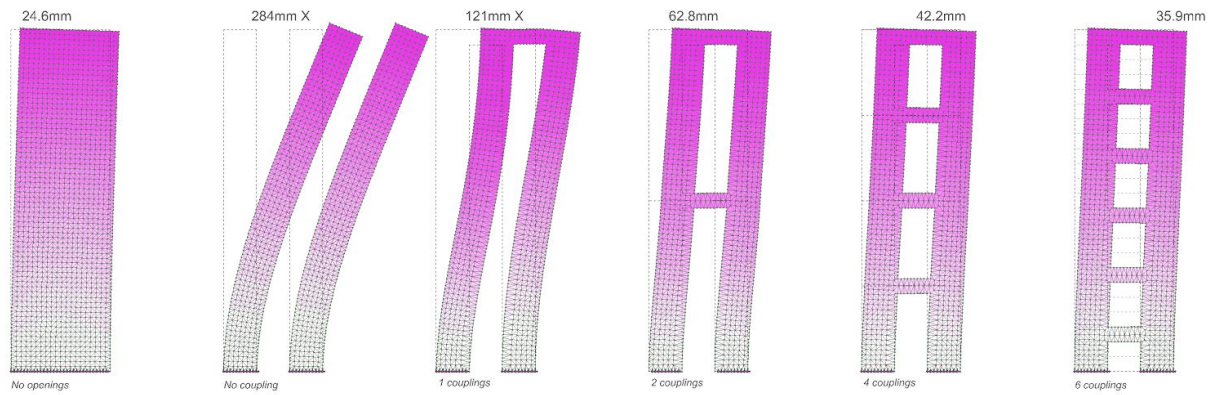
Option 1. Openings + coupling



Different distribution of shear walls and openings (Own elaboration)

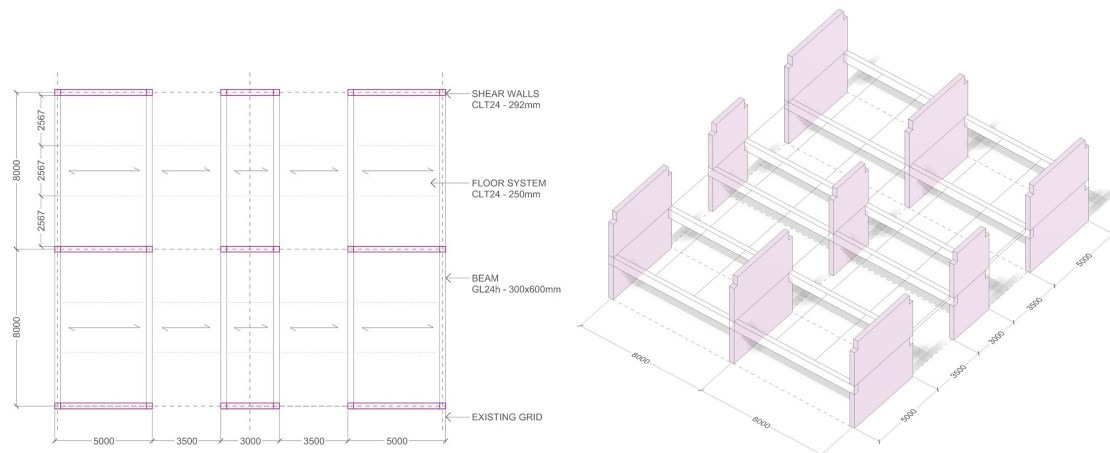


Floor plans, and architectural layout integration with CLT shear walls (Own elaboration)



Different coupling options for CLT 150mm shear wall (Own elaboration)

26 shear walls - 5 meters + 13 shear walls - 3 meters



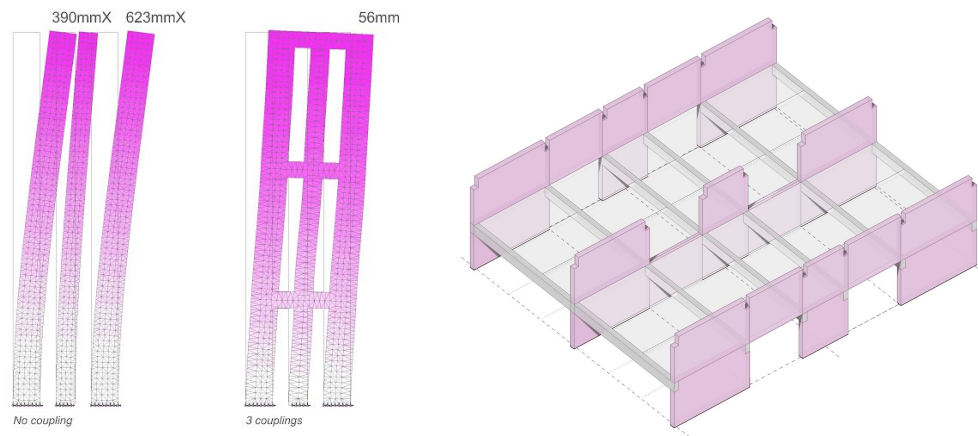
Floor plan and 3D scheme with uncoupled shear walls (Own elaboration)

- Wind load distribution of each separate shear wall is proportional to the bending stiffness

Shear wall	Second moment of area Nmm ²	Wind loading	Deflections (mm)
5 meters	2.06e+12	45%	623mm X
3.5 meters	4.5e+11	10%	390 X

Need at least three couplings (storey height) along height of the building.

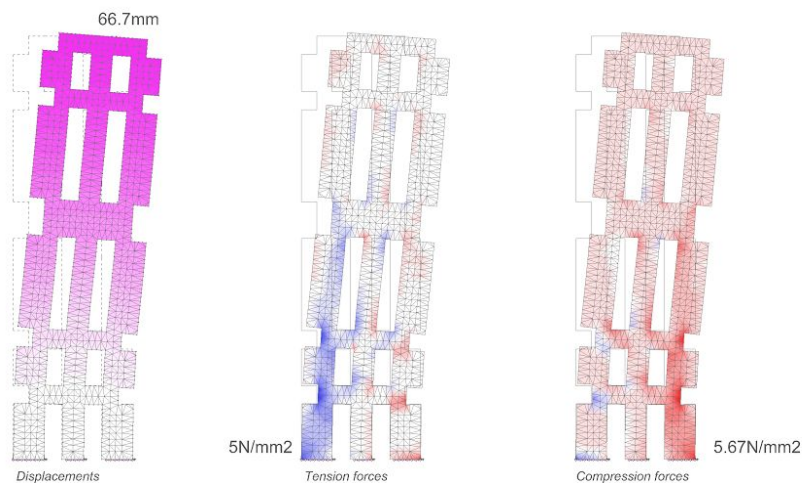
Alternate couplings in different storeys



Storey height couplings (Own elaboration)

The main issue with using big couplings is a significant interruption of the architectural layout. One solution to the problem is to work closely with the architect and distribute structural coupling in an alternate way in the floor layout. In this way, structural performance, horizontal circulation and residential circulations can be satisfied.

Multiple openings

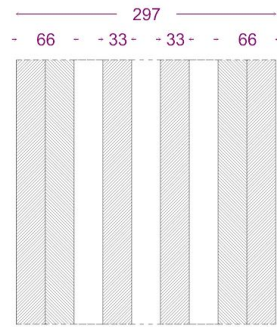


Another solution is to create multiple openings in the shear walls. In this way, a minimum of two penetrations for each shear wall are possible. Nevertheless, further analysis must be performed regarding this solution, especially as sharp cuts may create stresses perpendicular to fibres in timber that need to be controlled.

Option 2. Cores

Wind loading: 1.34kN/m². Height: 70m. Stiffness limit $H/850$. $E_0 = 11000\text{N/mm}^2$

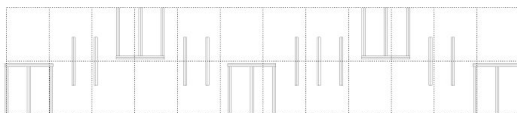
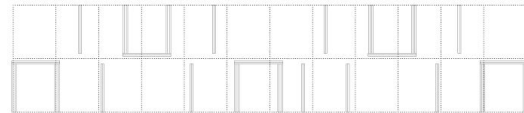
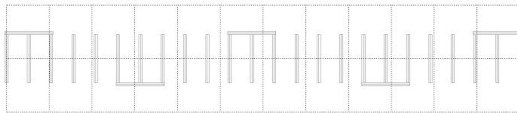
The cross section of the shear walls is a CLT panel of 297mm total thickness, according to Finnforest manufacturer (see appendix) composed out of 9 layers of C24 timber class.



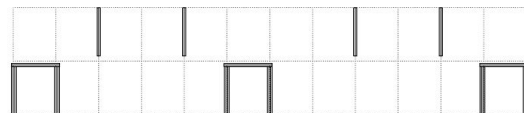
Shear wall length	Second moment of area Nmm ²	Stiffness	Minimum number shear walls
6.7 meters	5e+12	1	113
9 meters	1.2e+13	2.4	48
12 meters	2.88e+13	5.76	20
16 meters	6.8e+13	13.6	9
20 meters	1.3e+14	26	5
9 meters concrete	1.2e+13	4.8	24

Increasing the cross-section of the shear walls from 150mm to 297mm , or double 297mm, results in a great reduction of the number of shear walls and consequently more architectural flexibility.

24 SHEAR WALLS
2x CLT 297mm
L= 9meters



CONCRETE



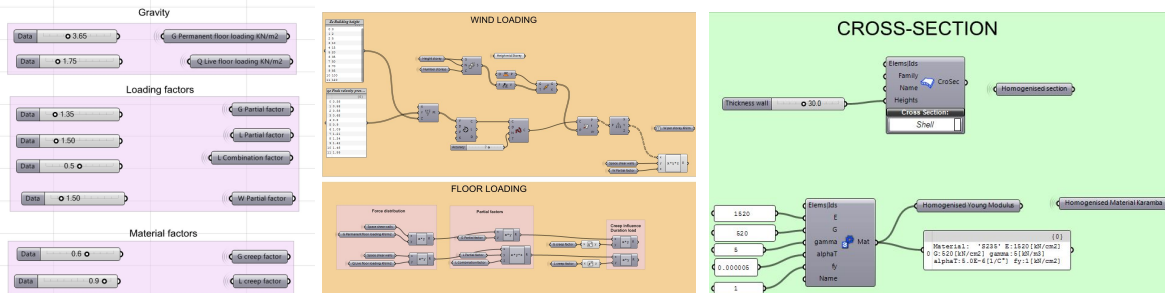
Different positions of shear walls or cores for lateral loading (Own elaboration)

In the above schematic drawing, we can appreciate more flexibility if concrete material is used in the lateral resisting system

Second order calculations

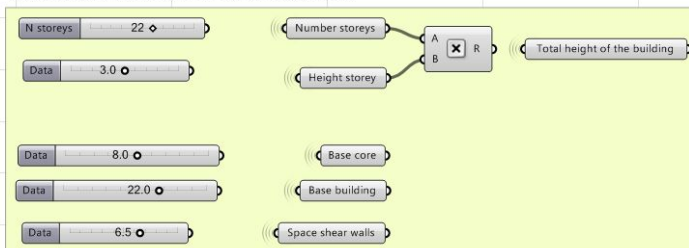
Parametric model

Constraints and parameters

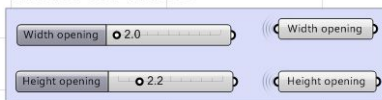


Definition of loading factors, interpolation of wind loading and homogenised cross-section for Grasshopper and Karamba (Own elaboration)

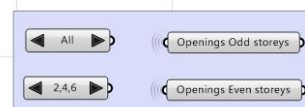
OVERALL BUILDING DIMENSIONS



WIDTH OPENINGS



NUMBER OPENINGS PER STOREY



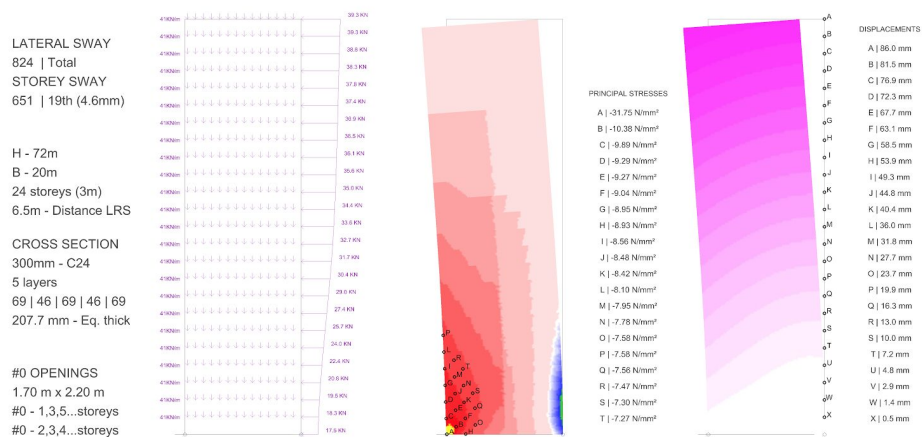
STRUCTURAL CALCULATIONS



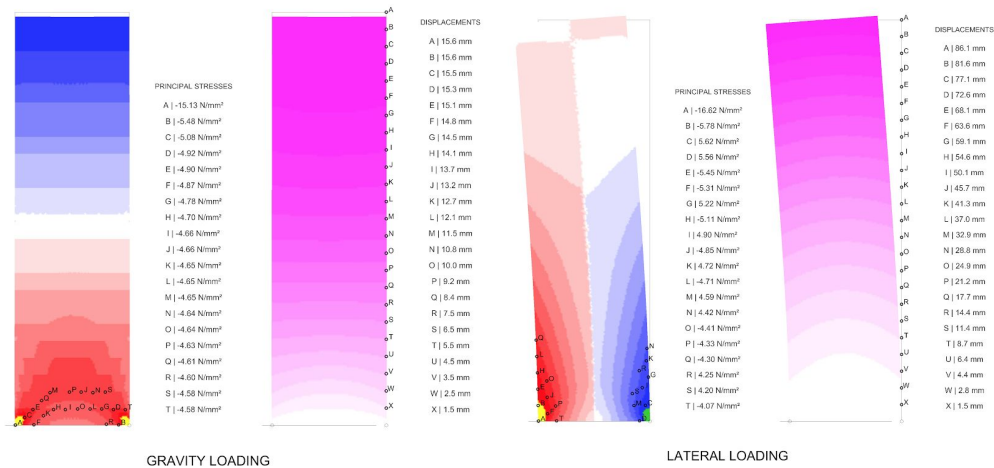
Defined parameters for second order calculations of shear walls (Own elaboration)

Influence of height of the building

In order to compare the effect of lateral and gravity loading in the shear walls, a CLT cross-section with 5 layers, 69/46/69/46/69 and 300mm nominal thickness have been assumed.



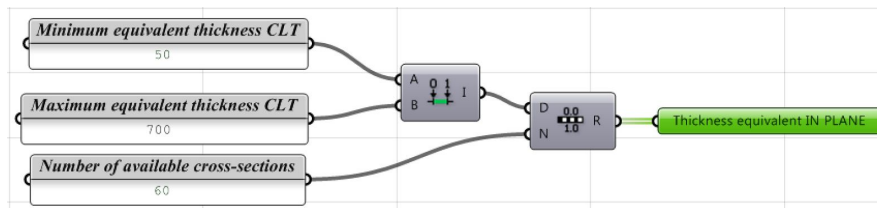
SECOND ORDER



Total height	Number of storeys	Lateral sway	Storey sway index	Max storey sway
72m	24 storeys	824	651	4.6mm (19th storey)
66m	22 storeys	1080	857	3.5mm (17th storey)
60m	20 storeys	1447	1153	2.6mm (15th storey)
54m	18 storeys	1990	1589	1.9mm (13th storey)
45m	15 storeys	3425	2711	1.1mm (9th storey)

Comparison structural values for different building heights, assuming a nominal CLT cross-section of 300mm

Optimization of cross-section



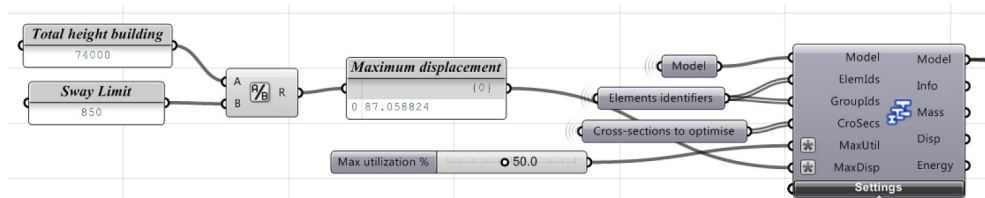
Definition for cross-section optimization (Own elaboration)

0 : 'CLT' height:5[cm]	20 : 'CLT' height:26.67[cm]	34 : 'CLT' height:41.83[cm]
1 : 'CLT' height:6.08[cm]	21 : 'CLT' height:27.75[cm]	35 : 'CLT' height:42.92[cm]
2 : 'CLT' height:7.17[cm]	22 : 'CLT' height:28.83[cm]	36 : 'CLT' height:44[cm]
3 : 'CLT' height:8.25[cm]	23 : 'CLT' height:29.92[cm]	37 : 'CLT' height:45.08[cm]
4 : 'CLT' height:9.33[cm]	24 : 'CLT' height:31[cm]	38 : 'CLT' height:46.17[cm]
5 : 'CLT' height:10.42[cm]	25 : 'CLT' height:32.08[cm]	39 : 'CLT' height:47.25[cm]
6 : 'CLT' height:11.5[cm]	26 : 'CLT' height:33.17[cm]	40 : 'CLT' height:48.33[cm]
7 : 'CLT' height:12.58[cm]	27 : 'CLT' height:34.25[cm]	41 : 'CLT' height:49.42[cm]
8 : 'CLT' height:13.67[cm]	28 : 'CLT' height:35.33[cm]	42 : 'CLT' height:50.5[cm]
9 : 'CLT' height:14.75[cm]	29 : 'CLT' height:36.42[cm]	43 : 'CLT' height:51.58[cm]
10 : 'CLT' height:15.83[cm]	30 : 'CLT' height:37.5[cm]	44 : 'CLT' height:52.67[cm]
11 : 'CLT' height:16.92[cm]	31 : 'CLT' height:38.58[cm]	45 : 'CLT' height:53.75[cm]
12 : 'CLT' height:18[cm]	32 : 'CLT' height:39.67[cm]	46 : 'CLT' height:54.83[cm]
13 : 'CLT' height:19.08[cm]	33 : 'CLT' height:40.75[cm]	47 : 'CLT' height:55.92[cm]
14 : 'CLT' height:20.17[cm]	34 : 'CLT' height:41.83[cm]	48 : 'CLT' height:57[cm]
15 : 'CLT' height:21.25[cm]	35 : 'CLT' height:42.92[cm]	49 : 'CLT' height:58.08[cm]

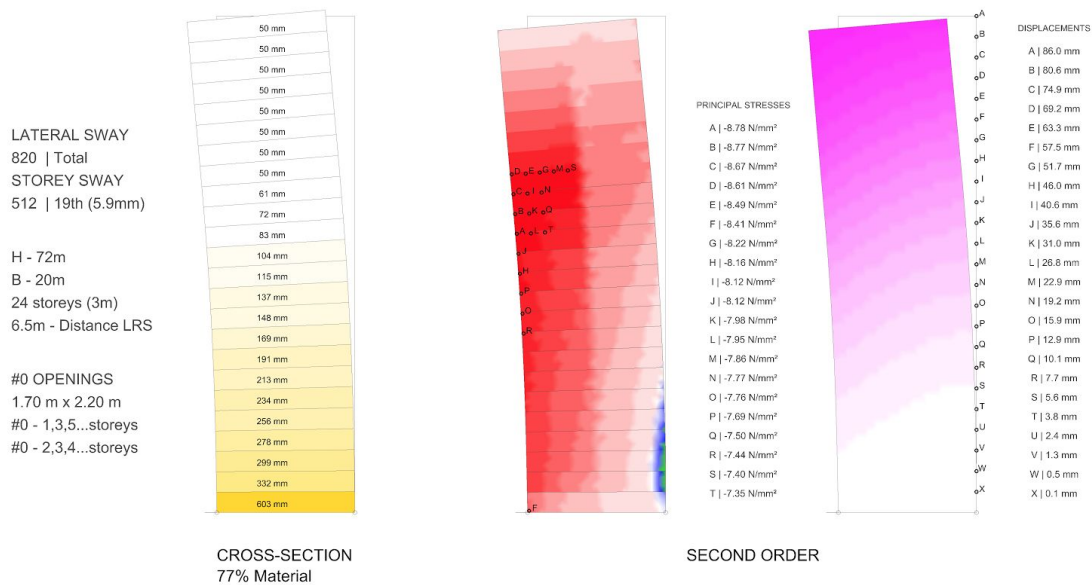
Example list of available cross-sections for optimization (Own elaboration)

- Optimisation criteria: Sway Limit $H/850$ accounting for foundation stiffness

In the image below, we can also see that 50% utilization limit is also imposed on the criteria, however, displacements at the top are usually the most restrictive criteria.



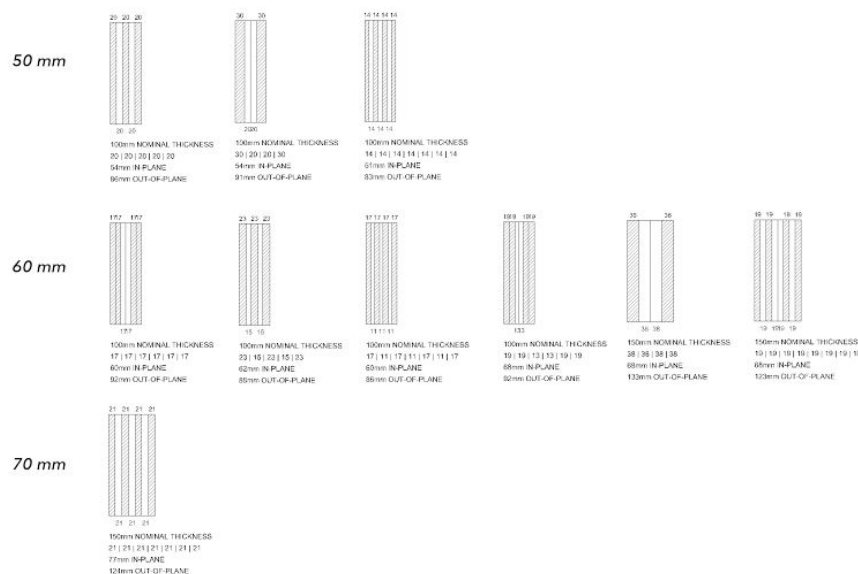
Optimisation criteria with Karamba components (Own elaboration)

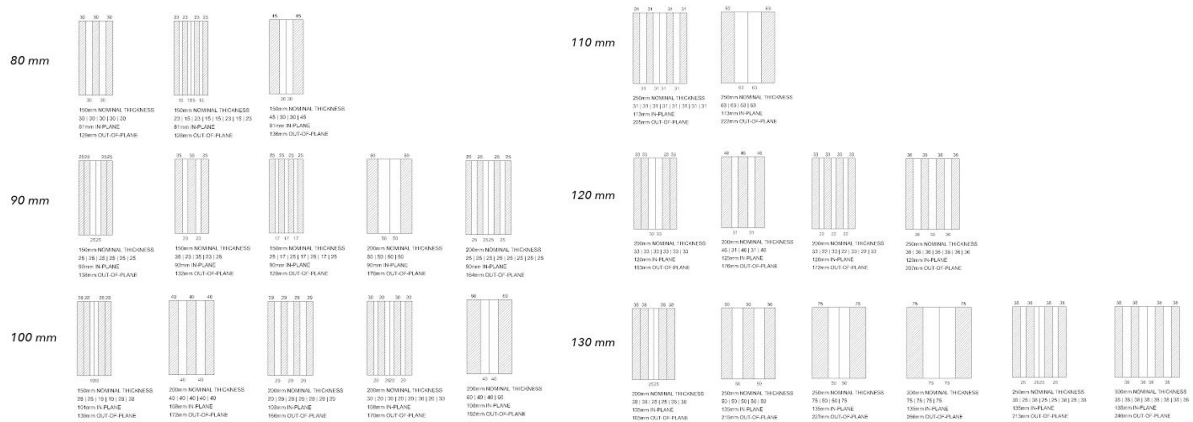


- Optimization of cross-sections along the height of the building for maximum efficiency, can lead to 23% material reduction.

The structural computer calculations are performed with an equivalent thickness of cross-sections. In order to transform this to nominal thickness the below table can be used.

- CLT CROSS-SECTIONS ORGANISED ACCORDING TO EQUIVALENT IN-PLANE THICKNESS -





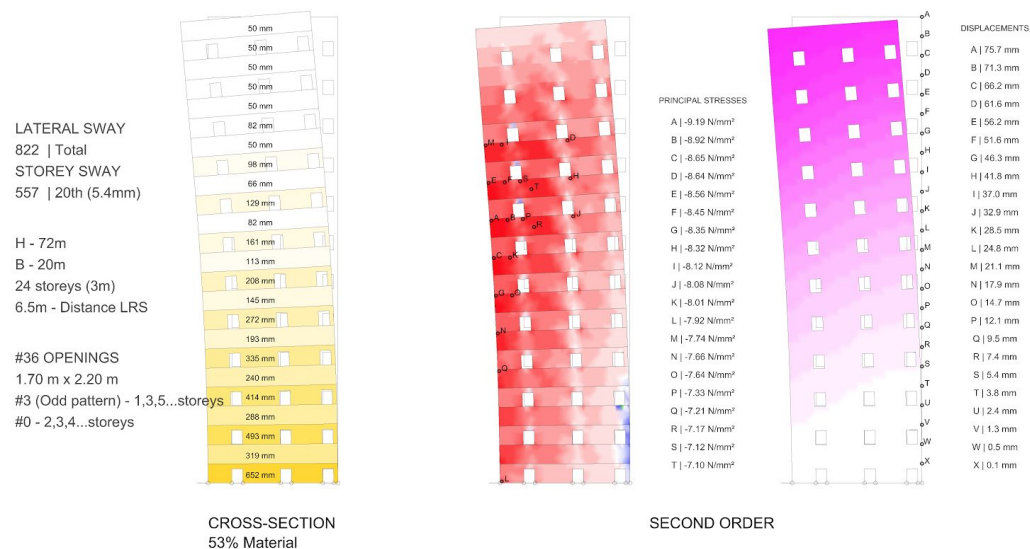
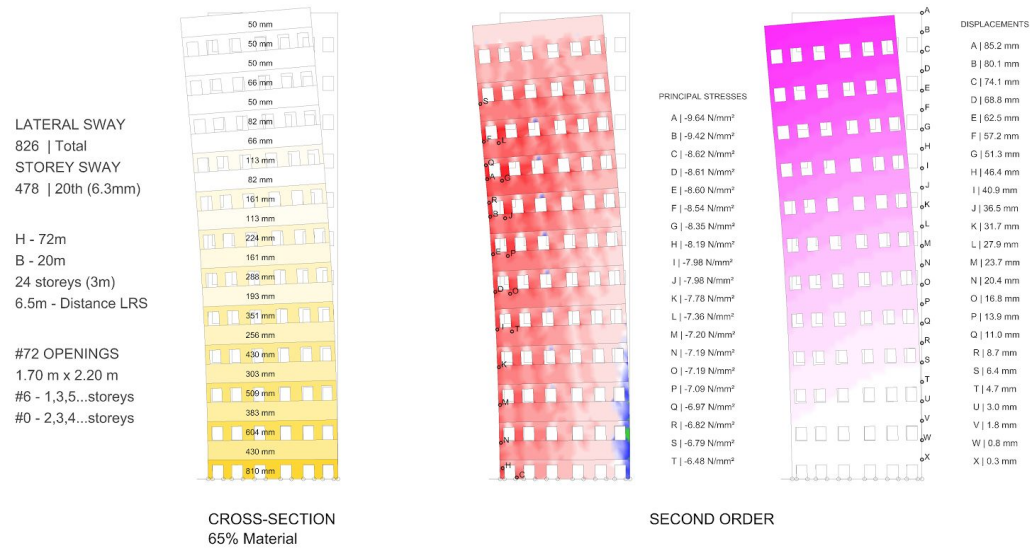
Study openings

From the literature and case studies, we can deduce that any building Tall Timber building will require important presence of the structure in the floor plans. This is due to the lower stiffness of the material compared with concrete or steel. At the same time, as timber is a natural material it may have imperfections that have to be considered in the structural design, and consequently the stiffness capacity is decreased even further. This is also corroborated with the structural analysis, hand and computer calculations, resulting in multiple shear walls occupying the total base of the building.

- Needless to say, any building serves an architectural programme, and consequently floor plans must be optimised for maximum flexibility and possibilities of circulations. One common method for achieving this aim, is the introduction of multiple openings within the structural walls for serving horizontal circulations of the building.

The following calculations study the impact of a series of openings in the shear walls for integration of structural and architectural performance.

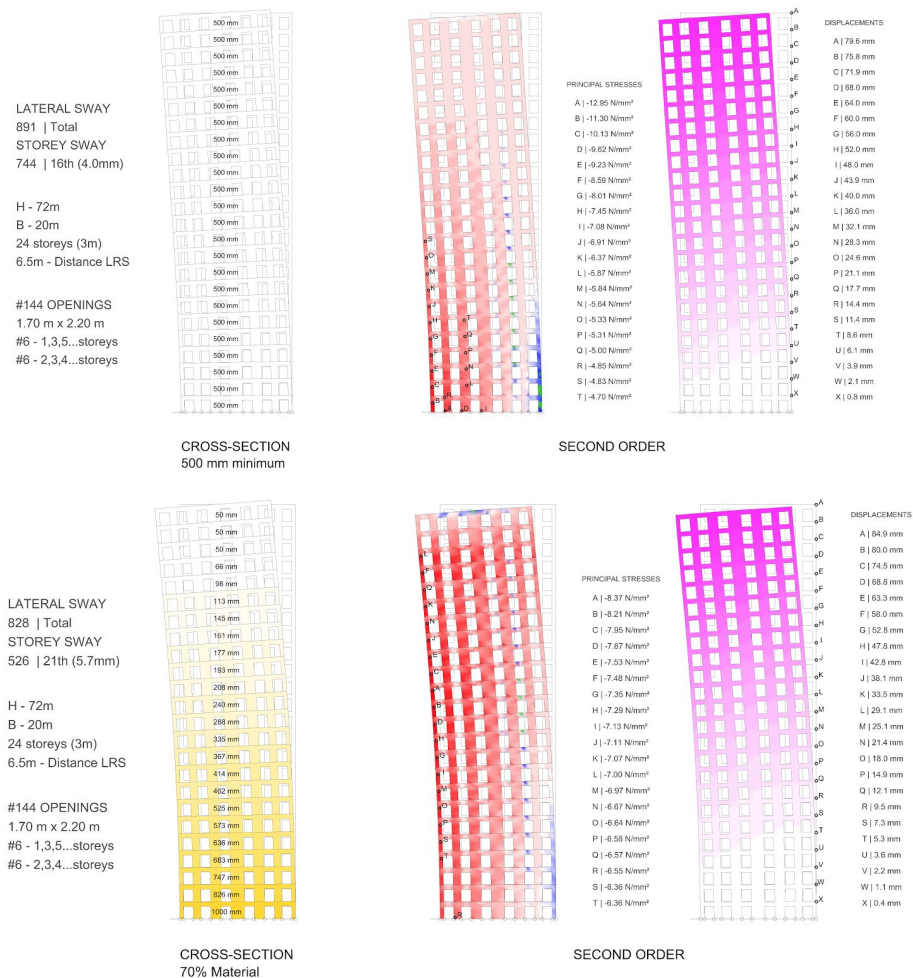
In odd storeys (1,3,5th). 36,72 openings



- In the calculations above, we can see that by optimising the cross-section an equal utilization of material can be achieved along the height of the building, thus resulting in material savings up to 50-60%.

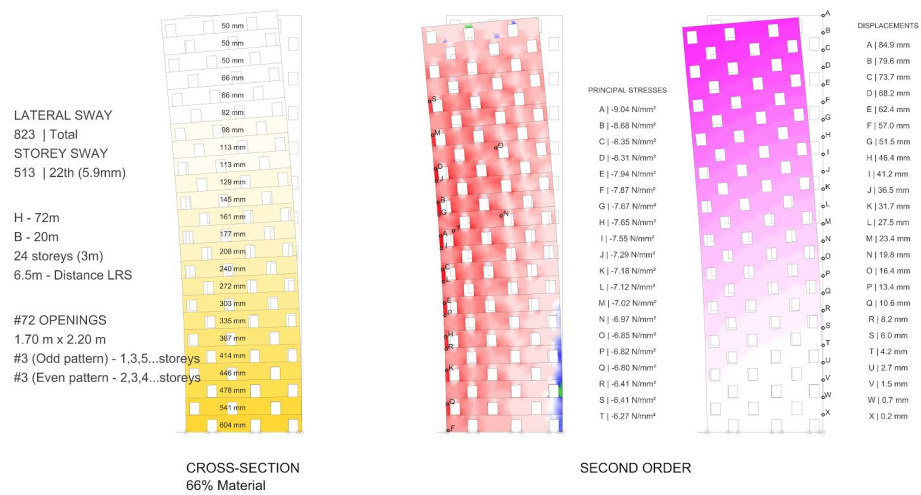
- The introduction of openings, results in dramatic increase of cross-sections at the bottom storeys (400-800mm equivalent thickness of CLT24). In this scenario, using stronger material, CLT28, CLT32, or concrete can reduce the amount of material consequently increasing the net floor area.

In every storey - 144 openings

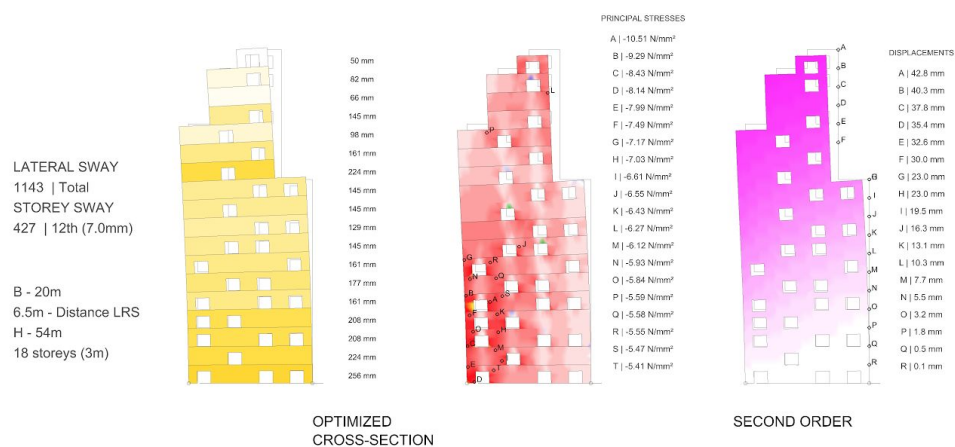
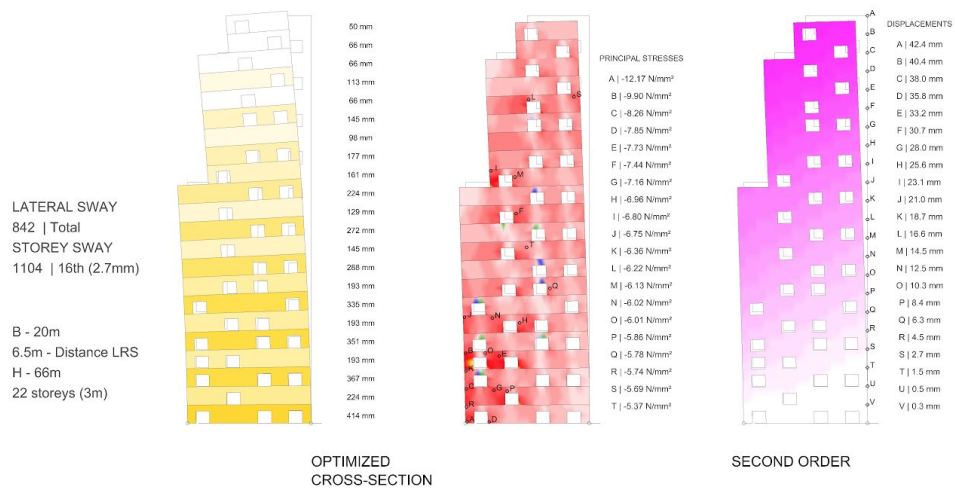


- The influence of openings in the shear walls, can increase the equivalent section from 200 to 350mm , openings only in odd storeys, and to 500mm, openings in every storey, doubling the amount of material required in order to meet deflection limits at the top of the building.

In alternate storeys



Alternative shear wall profiles



PART 5

REFLECTION

14. REFLECTION

(TTE) TALL TIMBER EXTENSIONS

Conventional research vs conventional design methods

TTE, or Tall Timber Extension is a new research topic, and there are not many available sources for comparing the data. The research methodology tries to analyze several case studies for each of the subtopics, Tall residential buildings, Tall Timber structures, and Extensions on top of an existing buildings, in order to provide a complete canvas for the research. All of the three categories share something in common, the overall approach towards a sustainable building. However, integrated global approach towards a building is not a conventional research topic, probably because of the great number of parameters involved, or the great dependency on site/project specific constraints.

On the other hand, in design disciplines, it is highly common to focus on the overall building first, and reduce the perspective towards details later. Needless, to say, this difference in conventional research and conventional design methods was not foreseen at the beginning of the research project, and created some difficulties. On the other hand, literature review of several case studies was a very useful process to understand technical limitations and extracting structural guidelines for Tall Timber buildings and Structural Extensions.

Sustainable construction typology

Within the master track of Building Technology, the Sustainable Graduation Studio aims at developing new techniques that can make a sustainable contribution to the way we construct our buildings. Tall Timber Extensions, addresses the mentioned questions in three combined ways. Tall Buildings (alternative architectural solution for crowded cities) Timber Buildings (alternative sustainable structural material) Extensions (alternative construction method in order to avoid demolition and preserve existing buildings). At the same time, these three parts share sustainability in building structures as the main common line.

Difficulty for fully-integrated building approach

In my opinion, overall building studies are a great necessity nowadays. Traditionally, cathedrals were designed and built under the guidance of the only one person, that was able to control design disciplines, construction techniques, logistics, etc. That person was known as "the great master", take Antonio Gaudí as an example. Today, with a growing specialisation in construction, this integrated vision has been lost, and building are subdivided in an endless list of disciplines, façade, structure, climate, computational, architecture, project management, glass specialist, timber specialist, infographies, BIM, sustainable specialist, etc.

However, at the interfaces between disciplines there are usually conflicts that usually result in less architectural quality of the building. This issue is constantly ignored in conventional research methods, that constraint building complexity into a few parameters to be researched, and do not consider the riches of an integrated approach. In every building a myriad of important issues are interconnected, and it is not a simple task to separate them and ignore the implications on other requirements. For example, client demands, site-specific constraints, architect ambitions, technical limitations use different research methodologies, but in practice they should be smoothly combined into a high-quality architectural object.

Designing is an intuitive (re)search for the unknown

It is a common saying, that researching is looking for something you do not know. Designing is an intuitive and personal process, that is demonstrated through consecutive design iterations. It may have a less strict methodology, but this does not mean that it is not research, or that is is illogical.

In the graduation studio, the suggested methodology was based on academic research, which draws first a clear route and heads towards the planned objective. Personally, I found it very difficult to create a good design with this methodology, as every decision has to be planned in advance and "proved" with logical arguments. Coming back to Gaudí, his research method was experimenting with shapes and concepts, by creating physical models and drawings and iteratively testing architectural expression and structural efficiency.

Designing is experimenting, it is working hard, and develop your own individual methodologies, and allowing flexibility in the process. It is often the case, that best designs come when you do not know the outcome. In conclusion, it is an intuitive exploration of your design intentions through different formats. This process, even though, sometimes may not follow a step-by-step approach, it helps in creating innovative research, as it is able to connect different topics that can make the difference.

For example, Gaudí connections between nature-like shapes and form-finding can be very difficult to achieve with academic research methodologies. In conclusion, academic research can provide solid grounds and rational results that can be suitable for structural design. However, more intuitive design methods can provide an interesting alternative for architecture concepts.

15. FURTHER RESEARCH

Acoustic properties

Floor systems in tall buildings are not only subjected to structural requirements, they should also meet enough sound insulation between floors. Nevertheless, acoustic behaviour of a given construction system is not a straightforward answer. A comparison between single-leaf constructions such as CLT panels, with cavity constructions such as Lignatur floor system, with concrete hollow core. Concluding in an optimisation of structural performance meeting sound insulations requirements for multi-storey buildings.

Composite action

Preliminary methods for analysing composite action between different interlayers of CLT panels, and concrete, providing simple rule of thumbs and criterias for the use of engineers and architect in the design a custom made CLT or CLT-Concrete construction.

Timber beams types for floor systems

It is not in the scope of this study to compare, different options for timber beams built up, Laminated Veneer, Glue laminated Timber, Parallel Strand Timber, etc.... A preliminary indication will be given, regarding the combination of glued laminated beams, with 100 mm concrete slab, for acoustic insulation with an approximate mass/surface of 250 kg/m². Further research must be performed in order to evaluate shrinkage/swelling of the timber elements, composite action between layers, and shear connection between concrete slab and timber beams.

Shrinkage and swelling

Timber can expand and decrease its volume depending on the surrounding moisture. When there high humidity, internal microcells inside wood get filled with water and consequently swell. Moreover due to the anisotropy of the material, these movements are higher in the transverse direction of the fibres, in the order of 20:1, compared to the longitudinal direction. These changes in dimensions need to be considered in the buildups of the floor systems and in the construction details.

Behaviour in fire

Behaviour of composite cross-sections in case of fire

Large void spaces, concealed spaces on mass timber structures.

Structural adhesives in CLT panels leading to delamination of CLT panel layers

Creep and differential shortening between concrete and timber

PART 6
APPENDIX

WIND-INDUCED ACCELERATIONS

In order to understand the dynamic effects and vibrations induced in a tall timber structure the case study “The Treet” was used as a design example. The structural system has certain resemblances to similar buildings made with steel frames. Even though, material strength/ density is 50% higher for timber than for steel, because of the choice of cross sections, connections, fire design and other considerations, the overall structure results in a similar stiffness and mass to a similar building with steel frames. (Malo, Abrahamsen & Bjertnes, 2016)

The design loads were determined with Eurocodes standards (CEN 1990 2002, CEN 1991 2002), and national annexes. For “The Treet” building the wind loading was the dominating load case, with a maximum wind speed of 44.8m/s calculated with a corresponding wind pressure of 1.26kN/m².

The wind load was applied as a transient static load on all four sides of the building with additional checks for diagonal directions (45°, 135°). Because of the regular geometry of the construction wind tunnels test were not necessary. The U-shape may have some local wind effect on the façade, but minimal influence on the global structure. Because of the low ground acceleration compared to other countries, and Norwegian regulations, seismic loading is not necessary to consider when wind prevails.

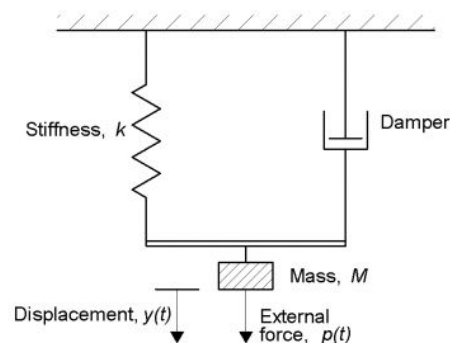
Lateral loading	Gravity Permanent loading	Gravity Live loading
Wind: 1.26kN/m ² Seismic not considered	Glulam and CLT: 4.5kN/m ³ Concrete decks: 25kN/m ³	Apartments: 2.0kN/m ² Common areas, corridors, stairs : 3.0kN/m ² Balconies 4.0kN/m ² Gym: 5.0kN/m ²

For the Treet, as the height above the parking garage is 45m, the resulting fundamental frequency is around 1Hz. The glulam structural frame is designed to work as a truss with a total equivalent damping ratio of 0.019. The connections are without eccentricities and the members are mainly subjected to axial straining. It is therefore likely that the material damping in the glulam members will be about 0.005.

In the structure, there are few connections in the vertical columns of the glulam frames, and therefore their effect is believed to be negligible. It is more likely that the flexibility of the joints connecting beams and diagonals in the external truss may influence the horizontal deformation and dynamic performance of the building. Several scenarios to model connections were considered in order to investigate how axial and rotational stiffness of the joints affect the structural performance of the model.

Basic notions about lateral vibration of structures

When a building is subject to fluctuating wind it might start to vibrate, leading to the discomfort of the occupants. The simplest vibrating system can be explained from a single mass attached to a spring. If the mass, M is displaced from the equilibrium, and then released, it starts to bounce iteratively, following a displacement (y) against time (t).



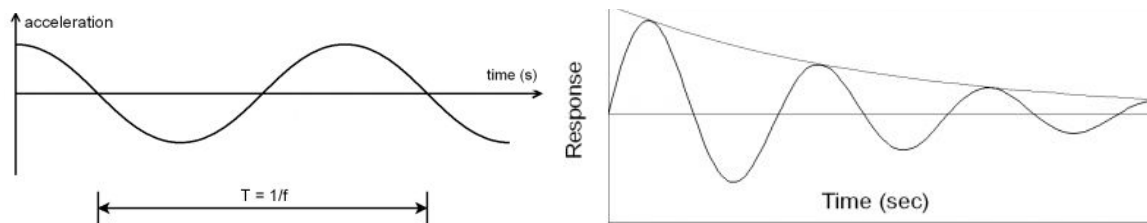
Schematic model of a simple vibrating system (Source: Steelconstruction.com)

The motion of the vibrating system can be defined with three parameters:

- Frequency (rate of vibration)
- Amplitude (accelerations)
- Damping (energy dissipation)

The frequency of the vibrating system, is the rate at which the system vibrates, and it is normally quoted in Hertz (cycles per second). It is proportional to the square root of stiffness k , divided by the mass, M .

Damping refers to the loss of mechanical energy in the system. In the absence of damping the motion would continue indefinitely. In practice all vibrating systems encounter a degree of damping. In the case of tall buildings, the energy dissipations (damping) sources may include friction of the connections, fit-out or internal partitions. As the energy is taken out of the system through damping, the amplitude (accelerations) reduces until the motion eventually ceases. The amount of damping is therefore critical to determine the duration of the accelerations.



Response of vibrating system without damping (left) with damping (right) (Source: Steelconstruction.com)

Therefore the acceleration of the building depend on:

- Natural frequency of the building. (Rate of vibration)
- Vibrating mass
- Damping within the building. (Energy dissipation)

Natural frequency (Rate of vibration)

The fundamental frequencies and vibrational modes of a tall building are dependent on the stiffness and mass, as well as the distribution. There is little experience on those serviceability aspects for tall timber buildings.

However, large glulam trusses have been widely used in modern timber bridges, providing good structural behaviour. A rule of thumb of steel tall buildings is that the lowest fundamental frequency can be estimated by $46/H$ (with the height of the building in meters). (CEN 1991-1-4 2002).

Damping (Energy dissipation)

It is common that buildings do not provide mechanism for dissipation of energy. This means that a small change of the damping properties might have large influence on the accelerations of the building.

According to the literature the energy dissipation for lightly damped linear systems can be estimated, by:

$$\xi = \delta / 2\pi = \eta / 2$$

where ξ is equivalent viscous damping, δ is the logarithmic decrement and η is the so-called loss coefficient. The total equivalent viscous damping may be approximated by:

$$\xi = \xi_{struct} + \xi_{mat}$$

The material damping in wooden members ξ_{mat} is caused by the internal friction in the material, while ξ_{struct} is caused by energy dissipation in connections, e.g., between the steel dowels and timber elements.

For glulam members made out of Norway spruce, the material damping, ξ_{mat} can be estimated as:

- 0.005 for member with low shear stresses (truss, columns, etc...)
- 0.011 for members with high shear stresses (high beams, short spans beams, shear walls, etc...)

In the connections, the structural damping, ξ_{struct} is caused by friction between mating surfaces and micro-crushing, cracking or compression of timber at local level. Reynolds et al. 2012 estimated an equivalent viscous damping ratio of 0.019 for a single dowel embedded in wood. Although not directly comparable to slotted-in steel-plates and dowels, it may serve as an indication.

Building usually also contain other secondary structural elements, such as internal walls and façade that may provide some contribution to the dissipation of energy. Eurocode normative recommends values of structural damping between 0.015-0.019 for structural design of timber bridges with mechanical steel connections.

It is also worth mentioning, that up to date, the knowledge about damping properties of large timber building is insufficient. (Chapman et al. 2012). It is to be noted that these damping ratios are solely estimations, or best engineering for the time being based on other testes structures. Only full scale measurements will reveal the real damping ratios. For glulam framed structural system a total equivalent damping ratio of 0.019 can be used.

Glulam members	Dowelled connections	Glulam structural frame
0.005-0.010	0.010 - 0.020	0.015 - 0.025

Material damping ratios (Eurocode 5)

Global dynamic behaviour

Eurocode normative (CEN 1991-1-4 2002) establish the guidelines on how to calculate the peak accelerations in a building due to dynamic wind loading. ISO 10137 2007 recommends design criteria for wind-induced vibrations in order to evaluate the serviceability of a building, while Boggs, 1995 provides guidance for human response to vibrations. The general calculation of the standard deviation of the wind-induced accelerations in the horizontal direction,

$$\sigma_{ax}(Y, Z) = c_f \cdot \rho \cdot I_v(Z_s) \cdot V_m^2(Z_s) \cdot R \cdot \frac{K_y K_z \Phi(Y, Z)}{\mu_{ref} \Phi_{max}}$$

c_f is the force coefficient

ρ is the air density

$I_v(z_s)$ is the turbulence intensity at height z_s above ground

$v_m(z_s)$ is the characteristic mean wind velocity at height z_s

z_s is the reference height

R is the square root of the resonant response

K_y, K_z are constants given in Eurocode

μ_{ref} is the reference mass per unit area

$\phi(y, z)$ is the mode shape

ϕ_{max} is the mode shape value at the point with maximum amplitude

The characteristic peak acceleration for a point (Y, z) is defined as:

$$k_p = \sqrt{2 \ln(v \cdot T)} + \frac{0.6}{\sqrt{2 \ln(v \cdot T)}}$$

v = frequency of the evaluated mode shape, $T = 600s$

The equivalent mass per square meter can be defined as

$$\mu_e = \frac{\int_0^h \int_0^b \mu(y, z) \cdot \phi_1^2(y, z) dy dz}{\int_0^h \int_0^b \phi_1^2(y, z) dy dz}$$

$\mu(y,z)$ is the mass per unit area of the structure
 $\Phi_1(y,z)$ is the mode shape.

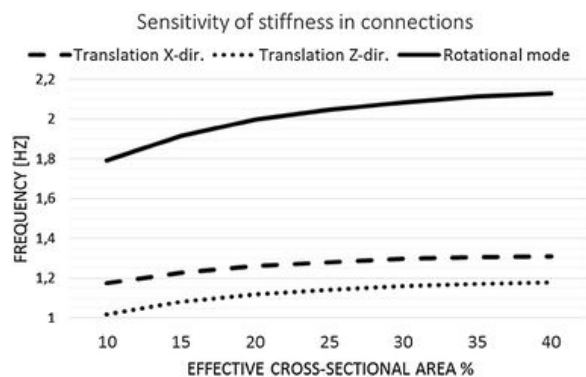
Structural analysis

In order to evaluate the axial stiffness of the connections, a parameterized model was created for quickly studying changes in the joint behaviour. Every structural element connected to two joints had a special element and material type attached at both ends. The length of these “special short elements” was equal to the height of the structural member (400-500mm)

The effect of the axial stiffness of connections was studied by reducing cross-sectional area of the special end elements. Since the axial stiffness is linear with the cross-sectional area, the ratio between the reduced sectional area to the original member area is directly comparable to the relative axial stiffness, k_{rel}

$$k_{rel} = A_{red}/A_{member}$$

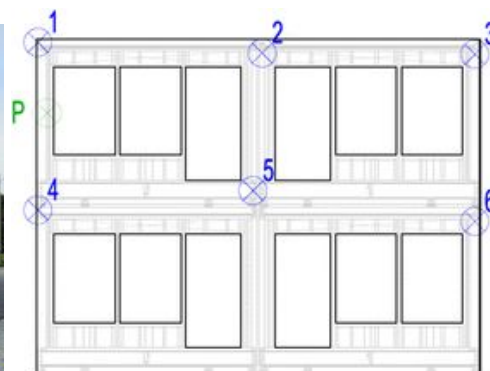
Ranges between 10-40% of connection axial stiffness were investigated, by reducing the sectional area of the end elements in the same proportion. The effect of connection axial stiffness in the building natural frequency is shown in the following figure.



Effect of reduced stiffness of connections in global stiffness of the structure.
 (Malo, Abrahamsen & Bjertnes, 2016)

Thus, reduced axial stiffness in the connections has small effect on the global stiffness. Values $k_{rel} > 0.25-0.2$ may have significant influence in natural frequencies and maximum accelerations.

In order to measure the natural frequency of vertically stacked modules, the responsible engineering consultant Sweco Norway AS, together with the University of Technology, NTNU performed full-scale non-destructive dynamic testing of the modules in cooperation with the manufacturer.

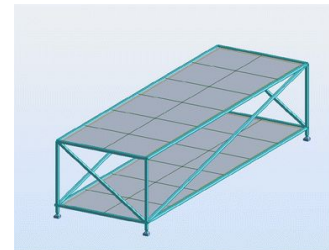


Left, 2x2 modules test setup. Right, location of accelerometers on short front side.
(Malo, Abrahamsen & Bjertnes, 2016)

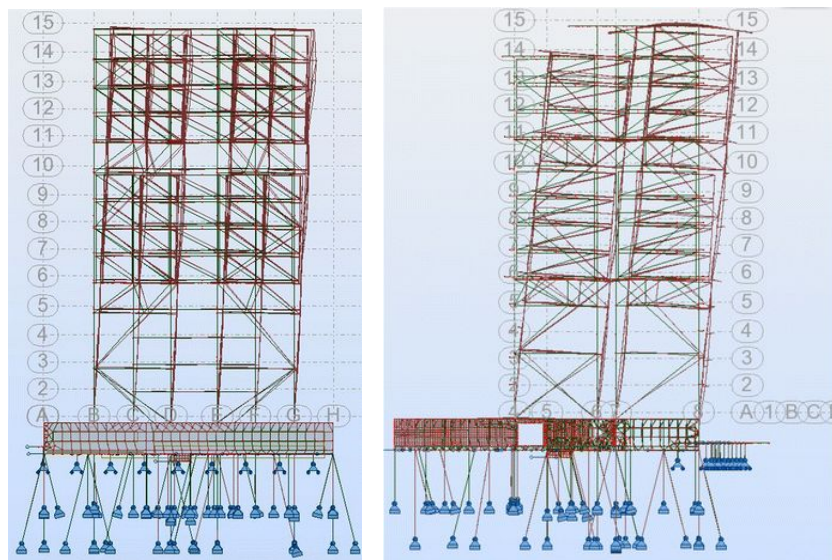
The following conclusions were extracted:

- The dynamic properties of stacked modules are dependent on the number of stack modules in vertical direction, but not in horizontal direction. With the exception of torsional mode
- The modules behave much softer in transverse direction, and therefore have lower fundamental frequency in that direction
- The stacks of modules appear to have higher fundamental frequencies (2.6-6.3Hz) than the overall glulam frame (1Hz)

A simplified truss-frame variant was used in the final design verification. The walls in the modules were represented by vertical beam-elements and braces. For the floor and ceiling stiffer shell-elements were used. The mass distribution of the modules was incorporated into the shell-elements. All beams and braces are pinned at the joints. The horizontal stiffness was modelled with the braces, and was tuned to harmonise with the test results.



The experimental test of the modules showed that a stack of four modules has a higher natural frequencies than the global response of the building. Therefore it was decided to avoid other connections other than the base. The stacked modules behave like a rigid body, following the vibration of the slabs.



Left, vibrational mode 1. East-West: Frequency: 0.75 Hz. Right, vibrational mode 2. North-South: Frequency 0.89Hz (Malo, Abrahamsen & Bjertnes, 2016)

Only the two lowest vibrational modes, 1 and 2, are of interest for the wind-induced vibrations

The external cladding and glazing of the building is attached to the truss frame. Hence, the wind load will not affect the modules directly.

The basis wind velocity in Bergen is 26m/s for a mean return period of 50 years. The characteristic wind velocity on site is calculated for a return period of 1 year, according to CEN 1991-1-4 (2002), and hence estimated to 19.1m/s. The wind-induced peak accelerations for vibration modes 1 and 2 were determined to 0.048 and 0.051m/s², respectively at roof level for one-year return period. Boggs in 1995, established the acceleration limit for nausea at 0.098m/s², and perception limit as 0.049m/s² for approximately 50% of the population. The perception limit for 2% of the population is 0.02m/s². Based on this information some of the residents in the top floors might in rare cases feel vibrations, but it is unlikely that they will feel uncomfortable.

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