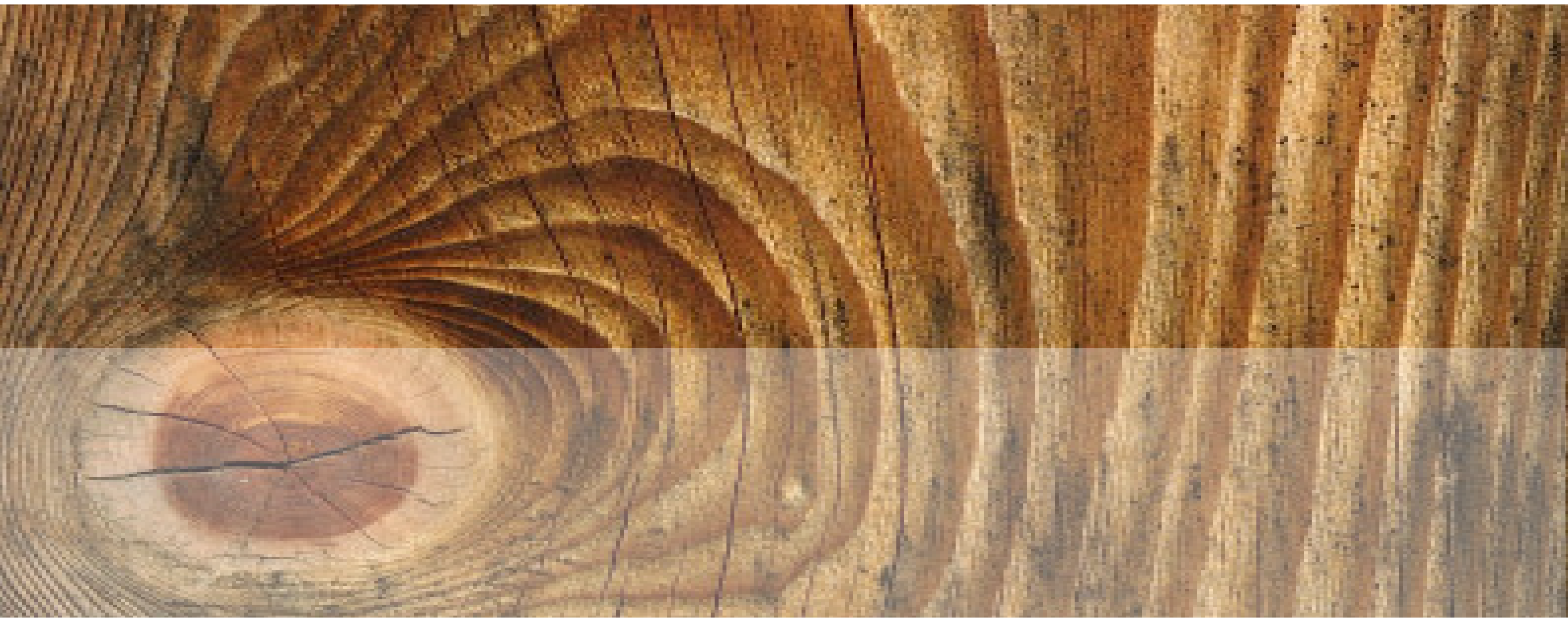
Master’s Thesis
Medium rise timber buildings in the Netherlands



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ABSTRACT

Building with timber is gaining in popularity throughout the world. Increasing environmental awareness and interesting new products from the timber industry provide a basis for new construction designs.

Due to the environmental advantages of timber construction, a worldwide interest in medium rise timber buildings is expected. A lot of research is conducted and pilot buildings are made in order to gain more insight in the performances and structural behaviour of medium rise timber buildings.

Meanwhile in the Netherlands, timber is rarely used as primary material in medium rise buildings. With only a few erected medium rise timber buildings, the production of these buildings in the Netherlands is still very small compared to the total medium rise production.

Because of this, timber as a construction method is an uncommon choice for designers and contractors to make. This results in a little and slow growing experience with timber. Therefore this research examines the technical feasibility of timber as a load bearing structure in medium rise buildings based on Dutch design requirements. This is achieved by designing a 12 storey medium rise building using timber as the primary load bearing structure.

The design of the structure is based on the massive timber construction, consisting of cross laminated timber (CLT) panels. CLT panels consist of several layers stacked on one another at right angles and glued together in a press over their entire surface area to obtain both wall and floor panels. The material is extremely strong and stiff considering its low density and is also quite easy to process and assemble with ordinary tools making the use of CLT in medium rise buildings very suitable.

Knowledge is also gained by studying three existing medium rise buildings comprising a CLT load bearing structure. The studies show the utmost importance of a good preparation and understanding of the material for acquiring a high performance building.

The design proofs the technical feasibility of timber as a load bearing structure when the following governing design aspects are taken into consideration:

- Control of horizontal drift: Control of horizontal drift may become governing in buildings where few shear walls are incorporated in the design.

- Bearing resistance: In medium rise buildings, large compression stresses perpendicular to the grain occur floor panels which are difficult to carry.

- Fire safety: Fire safety is the third mayor design feature considering medium rise timber buildings. A double layer of gypsum fibre board is needed for reducing the fire propagation and for ensuring a fire resistance of 120 minutes.

CHAPTER 1, INTRODUCTION

1.1 Background

Building with timber is gaining in popularity throughout the world. Increasing environmental awareness and interesting new products from the timber industry provide a basis for new construction designs. If we take a look at the characteristics of timber we find that it offers many advantages. Timber graded into strength class C24 (characteristic bending strength = 24 N/mm^2) is comparable to the strength of commonly used concrete while only having 1/5 of its weight. This difference in weight allows a lighter foundation and requires less lift capacity during construction. From an aesthetical point of view, timber does not rust and can be easily produced in various shapes. Besides, timber buildings are able to decrease on-site construction time significantly compared to in-situ cast concrete buildings. Other advantages are found in the energy consumption. Compared to steel and concrete, timber has excellent insulation characteristics, has ideal values of embodied energy and is easy to recycle.

The embodied energy is defined as the sum of all the energy required to make a material. According to the study ‘‘Environmental Properties of Timber’’ [13], the embodied energy for timber buildings will be lower than buildings constructed of concrete, brick and steel. The reason for this is found in the raw material for timber which is wood from trees. Sunlight provides the energy that drives the biological process that produces wood. Since this is a natural energy source without negative environmental impacts, the embodied energy quotient is ignored. Once harvested, relative little energy is required to transform the wooden logs into timber compared to other building materials. For example, the fossil fuel energy required to manufacture rough sawn timber is 1.5 MJ/ kg, while the manufacture of steel requires 35 MJ/ kg and aluminium requires 435 MJ/ kg.

According to the study, ‘‘Building materials energy and the environment’’ [20], CO^2 balances will also be favourable for timber construction. During its growth phase, trees bind carbon dioxide to it and retain it for many years until the tree eventually dies or burns. The same is true for timber products, which stores carbon dioxide until it is burned. The precise values of the energy and CO^2 balances of building materials depend on many factors, but it can be concluded that the use of timber construction will in general result in lower energy and CO^2 balances than when concrete or steel is used.

Due to the environmental advantages of timber construction, a worldwide interest in medium rise timber buildings is expected. A lot of research is conducted and pilot buildings are made in order to gain more insight in the performances and structural behaviour of these buildings. The world tallest timber building is located in London, England. This nine storey high building is from the first floor up completely made of timber. It will however not take long to be surpassed as the tallest timber building because there are plans to build a 14 storey high timber building in Milan.

Meanwhile in the Netherlands, timber is rarely used as primary structural material in medium rise buildings. With only a few erected buildings like Malmö hus in Almere, the production of multi storey timber buildings in the Netherlands is still very small compared to the total production of medium rise buildings.

Most Dutch contractors and designers relay on their knowledge and experience with concrete. This results in structures with a high value of predictability. This stands in contrast with timber structures where the knowledge of producing medium rise timber buildings is lacking. Timber in the Netherlands has also the disadvantage of suffering of several preconceptions about the material characteristics. Poor fire safety and poor sound insulating properties are preconceptions that are based on bad experiences. Malmö Hus can be taken as an example of a bad experience due to mistakes in the design. The contractor encountered several problems during construction with the sound insulation of the structure. The sound insulation value resulted much less than the predicted sound insulation od the structure. Late adjustments in design needed to be performed in order to overcome this sound insulating problems. More on Malmö Hus can be found in chapter 4.1.

1.2 Structural timber systems enabling medium rise buildings

Important for the design of a medium rise timber building is the used structural system. Many studies on medium rise buildings have been made in the past years and several systems are developed for increasing the maximum height of the timber buildings. The four most promising structural systems which enables tall timber buildings are reviewed.

1) Light weight timber frame construction

Light weight timber frame construction is the most common form of timber construction. The timber that is used consists of small sections of sawn timber that is used as wall studs and floor joists. Structural walls and floors gain their strength and stiffness from rigid panels like plywood that are sheeted to wall studs and floor joists to form part of the wall or floor section. Internal walls and floors are often also lined with gypsum plasterboard in order to improve the fire safety and acoustic properties.

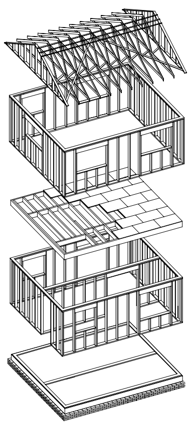

Figure 1.1 Light weight timber frame
Source: Timber frame construction
4th edition

Figure 1.2 St. James Building
Six storey light weight timber frame building
Source: Trada

One of the largest structural challenges of light weight medium rise timber frame buildings is to maintain the overall stability of the building. The weight of timber frame construction buildings is relatively light and does not help much in preventing push over by wind loads. This results in high tension forces in wall panels which are hard to transfer.

The building height however, is according to Grantham in multi storey timber frame buildings [19], limited to about six stories due to the limited perpendicular to the grain strength of top and bottom rails in walls and effects of shrinkage.

2) Heavy timber frame construction

Heavy timber frame construction is a construction method which consists of large beam and columns which are connected by for example steel dowels. The timber used for the beams and columns can be of glulam or sawn timber. Heavy timber frame structures are stabilized by either diagonal bracings or by portal frames with moment resisting connections. Diagonal bracings limits the free use of spaces and should therefore be placed in closed wall sections. The portal frame connections requires moment transmission connections of high strength and stiffness. A connection that suits this type of structure very well is the tube connection [21]. The tube connection is an high capacity connection developed at Delft University of Technology which can be used to transmit large bending moments.

While the Light weight timber frame construction is limited to six storeys, the Heavy timber frame construction has the potential to be used for taller buildings [31]. However, the problem that may exist with this system is that the cross sections of the beams and columns are quite thick and in order to ensure fire safety they must be protected by plaster board making the total cross sections even thicker. This is not problematic in office buildings, but can be unwanted in residential buildings.

Figure 1.3 House de Wiers
Five storey heavy timber frame building
Source: JDdV Architecten

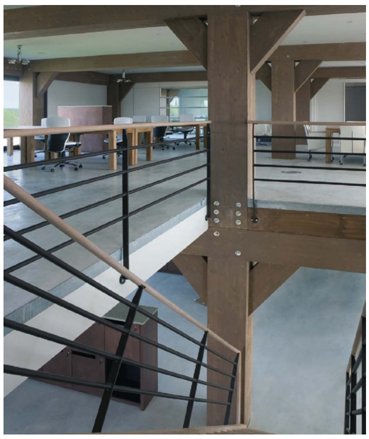

Figure 1.4 House de Wiers
Interior view
Source: JDdV Architecten

3) Post-tensioned timber frame construction.

A relatively new system of timber construction is the use of post-tensioning timber elements. Buchanan A., Fragiacomo M., Pampanin S., and Smith T., [7] studied this system and conducted research on the design and construction of post-tensioned timber buildings. This system was originally developed for pre-cast concrete construction but can also be used in medium rise buildings made from laminated veneer lumber (LVL) or glulam timber. In this way, wind loads are resisted by pre-stressed timber frames or walls which can be used separately or in combination. A case study of a six storey timber office building is analyzed and a virtual design is made which allows investigation of different methods of structural analysis and the development of many connection details for rapid construction. The overall building costs of the case study resulted to be comparable to steel and concrete designs. Based on testing and analysis to date, the feasibility of the post-tensioned solution is evident.

4) Massive timber construction

Massive timber construction is a construction method which makes use of large solid timber wall and floor panels. Different systems of massive timber construction are available differentiating in gluing or mechanically fastening timber boards into massive timber panels. Cross laminated timber (CLT) is the main form of massive timber construction and have become increasingly popular in the last ten years. CLT panels consist of several layers stacked on one another at right angles and glued together in a press over their entire surface area to obtain both wall and floor panels. The panel dimensions are such that entire walls can be produced in one element. CLT panels are extremely strong and stiff considering their low density and are also quite easy to process and assemble with ordinary tools.

The current highest timber residential building (nine stories) is constructed with CLT panels and proved to be a great success in terms of speed of construction and costs. The potential of CLT in medium rise buildings is very high considering the maximum height which can be constructed with the panels, which is about 12 – 15 storeys, depending on the layout of the building.

Figure 1.5 NMIT Arts and Media Building
Three storey post-tensioned timber frame building
Source: Irving Smith Jack Architects

Figure 1.6 NMIT Arts and Media Building during construction
Source: NZ Wood

It can be concluded that CLT panels are very promising for the use in medium rise buildings. The material has proven to be a viable form of construction and showed that it can reach to heights never seen before with timber. Because of these features the use of CLT panels for the load bearing structure of medium rise buildings is used in this study.

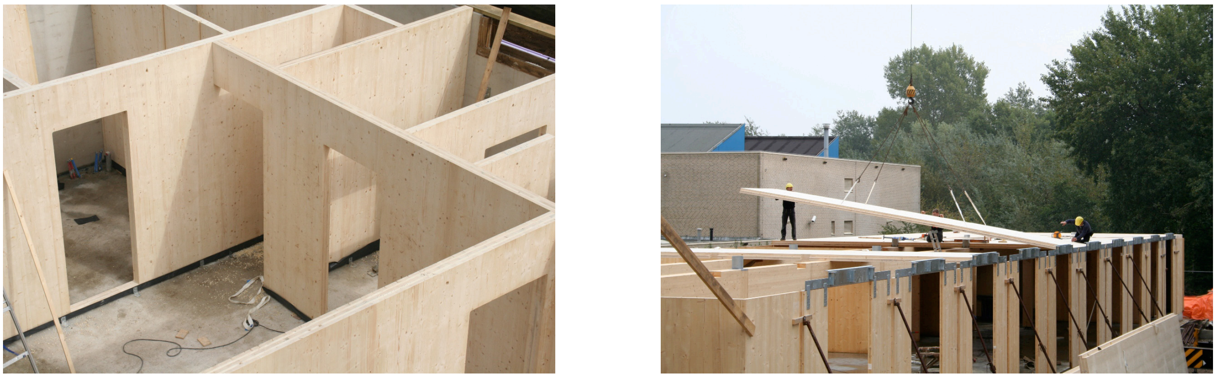

Figure 1.7 Left and Right,
Massive timber construction using CLT
source: KLH

1.3 Objective of the research

In this study the technical feasibility of timber as a load bearing structure in Dutch medium rise buildings is examined. The following research question will be the main focus of this study:

Is it possible to use timber as a load bearing structure in Dutch 'medium rise' buildings?

The term medium rise buildings is in this study understood as buildings of more or less than ten stories in height.

The primary objectives of this research are the following:

1. To investigate the properties and limitations of timber as structural material in medium rise buildings
2. To investigate the design of medium rise timber buildings and their characteristics.
3. To demonstrate the technical feasibility of a medium rise timber building based on Dutch requirements.

1.4 Approach to the work

The objectives of this study will be achieved by reviewing the governing Dutch building requirements concerning medium rise buildings. After this, the properties of cross laminated timber is thoroughly analyzed and examined.

Next, three medium rise timber buildings are studied. In this section, several aspects of the buildings are discussed like the structure and the building physical performances. The three buildings that are studied are: Malmö Hus in Almere, the Netherlands; Limnologen in Växjö, Sweden and Stadthaus in London, England.

Finally an existing 12 storey concrete building (Inntel Hotel) is chosen as a reference in order to make a design with a timber load bearing structure based on the Dutch design requirements. Special attention is given to the stability and fire safety of the structure since this will be the governing design aspects. Based on this design, conclusions can be made and the research question can be answered.

The following scheme represents the progression of the research.

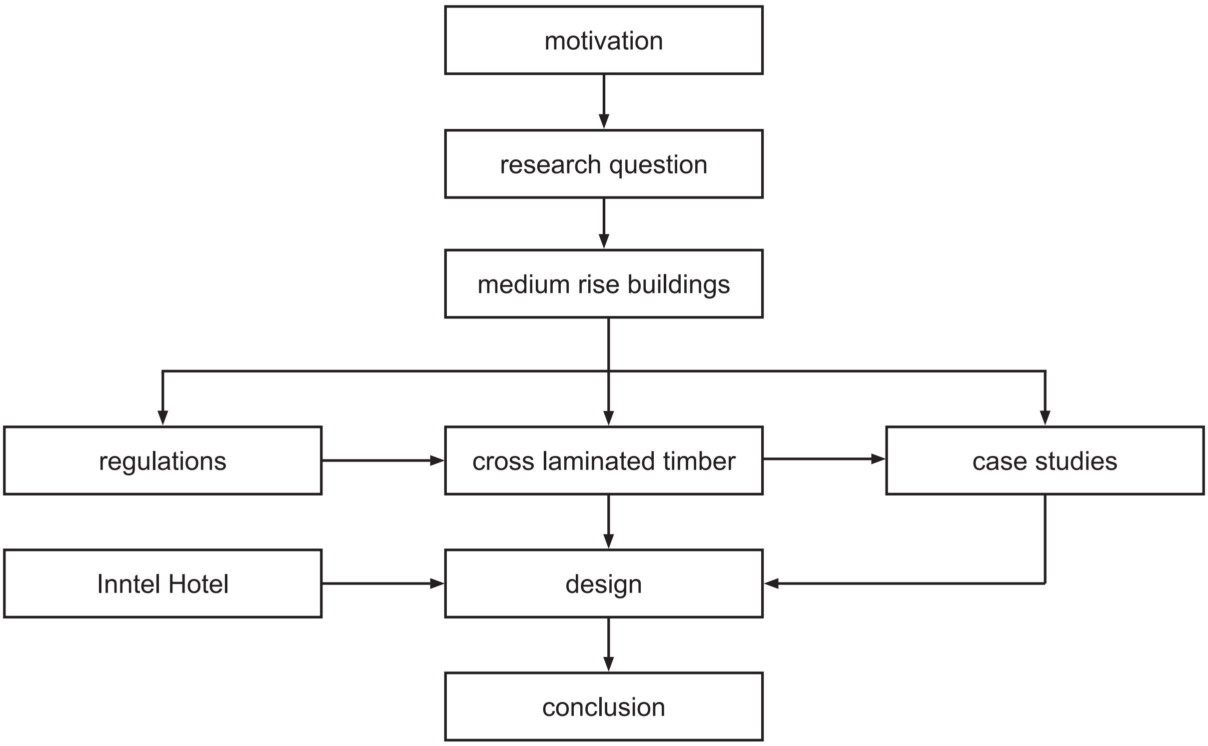

Figure 1.8 Schematic model of research

CHAPTER 2, DUTCH BUILDING REGULATIONS

There are many different demands and regulations that have to be taken under consideration when building medium rise buildings. Therfore, this chapter reviews the regulations wich are material dependent and which are of most interest for designing medium rise buildings.

The Dutch national building regulations is represented by 'Bouwbesluit 2003' (building decree in English) which is a performance based code. This document is supported by Dutch standards NEN which express provision methods for the regulations. It is however possible to select other technical solutions and methods if these comply with the requirements of the mandatory provisions based on the principle of equivalence.

The main regulations put on Dutch buildings can be seen in figure 2.1.

The requirements presented in this chapter are three of the most crucial to deal with in medium rise timber buildings.

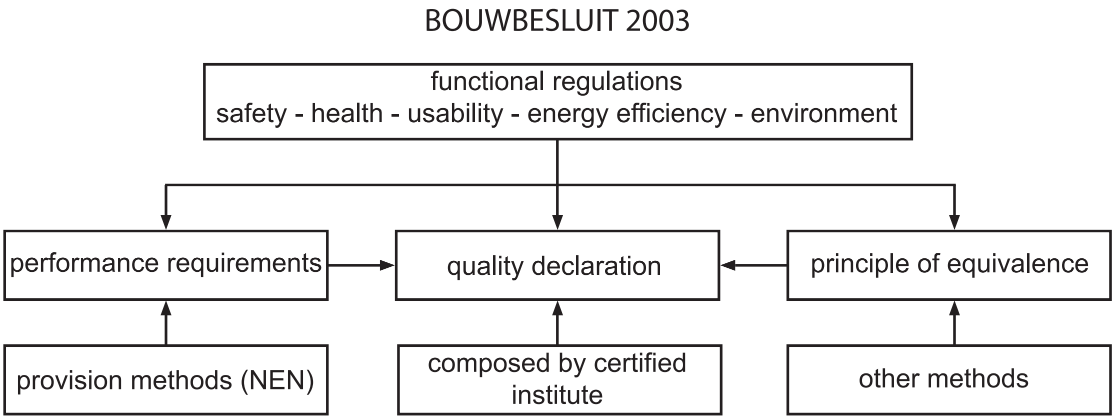

Figure 2.1 Relation scheme from bouwbesluit 2003

2.1 Structural safety requirements

The Dutch building decree refers for structural requirements to NEN 6702, TGB Loads and Deformations. The decree states that for certain loads, predefined ultimate limit states in NEN 6702 may not be exceeded.

There is also made a distinction in material, therefore the structural requirements fore timber structures are set in NEN 6760, Timber Structures.

At the time of writing the thesis, the use of European structural requirements (Eurocode series) are also allowed. With the introduction of the renewed 'bouwbesluit' in year 2012 the Dutch structural NEN standards will expire and the use of the Eurocodes will become obligatory.

2.2 Fire safety requirements

With the introduction of bouwbesluit 2003, performance requirements applies for the Netherlands regarding safety. These requirements are independent of the materials that are used for the structure of the building, but are exclusively dependent on the function and height of the building.

In case of fire, a reasonable time must be available for escape and investigation without the danger of structural collapse. Therefore the decree sets requirements for the fire resistance of the main load bearing structure in relation with its collapse and it sets requirements for the fire resistance in relation to the separating function of building elements.

A structural element is part of the main load bearing structure of a building when the building collapses or partial collapses when that structural element is removed, known as progressive collapse. Stabilizing elements can be excluded from the main load bearing structure when failure of the stabilizing element does not induce progressive collapse. This is only possible when several stabilizing elements are present and when they are situated in different fire compartments.

2.2.1 Fire resistance in relation with the separating function

The spread of a fire to other fire compartments (other residential areas or other storeys) must be avoided. Therefore the building decree sets requirements to the resistance against internal and external fire propagation, named wbdbo. These wbdbo requirements apply for al fire separating building components and are in relation with its separating function.

The following criteria must fulfilled:

- flames or hot gasses are not allowed to pass the separating component;
- the temperature on the unheated side is not allowed to surpass a given temperature (less than an average increment of 140°C) (applies for walls and floors);
- the radiation on the unheated side is not allowed to surpass a given radiation level (less than 1 kW/m2 on a one meter distance);
- no structural collapse is allowed during a required time. Load bearing walls have to be designed with fire load cases (according to NEN 6702 or Eurocode 0).

The structural elements that are not regarded as main load bearing structure but does have a separating function have always a minimum fire resistance of 30 minutes.

The fire resistance of these elements are increased to 60 minutes in two cases:

- If the elements have a fire separating function in a building with three or more storeys (higest storey floor on 5 metre or higher);
- If they have to protect the safety stair case against fire propagation

2.2.2 Fire resistance in relation with the main load bearing structure

The decree sets different requirements to the main load bearing structure of residential buildings and non-residential buildings.

Residential buildings

In order to give occupants the opportunity to leave the residential building and to enable the fire brigade to investigate the building, the decree sets requirements to the resistance of building structures. This involves the fire resistance in relation with collapse of:

- Building structures around smoke free escape routes
- Parts of the main load bearing structure

Smoke free escape routes in large residential areas (floor space of more than 500 m^2) and residential buildings must remain usable for 30 minutes.

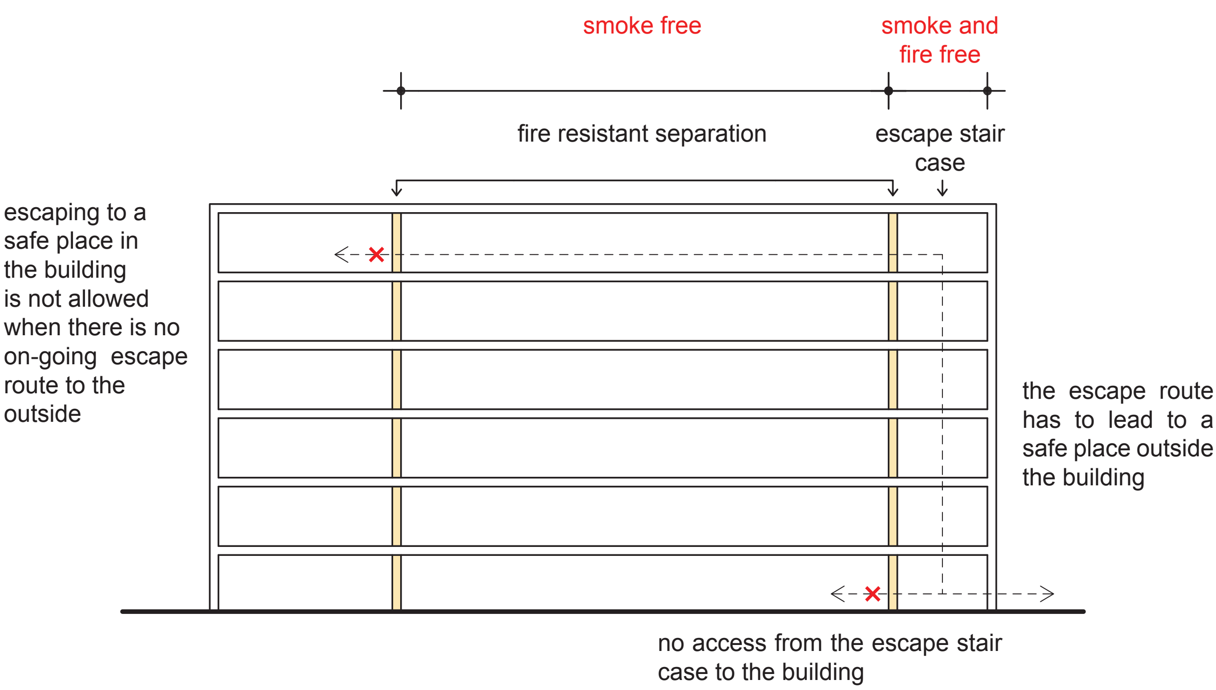

Figure 2.2 requirements for the escape route of residential buildings
Source: Verdiepingbouw met staal;
Redrawn by the author

The requirements for the main load bearing structure are related to the level of the highest residential area, see figure 2.3. The requirement is 30 minutes lower if it can be demonstrated that the fire load density is lower than 500 MJ/m^2 floor space and if the residential areas are not situated in residential buildings where the highest floor is situated at a height of maximum seven metres above the ground.

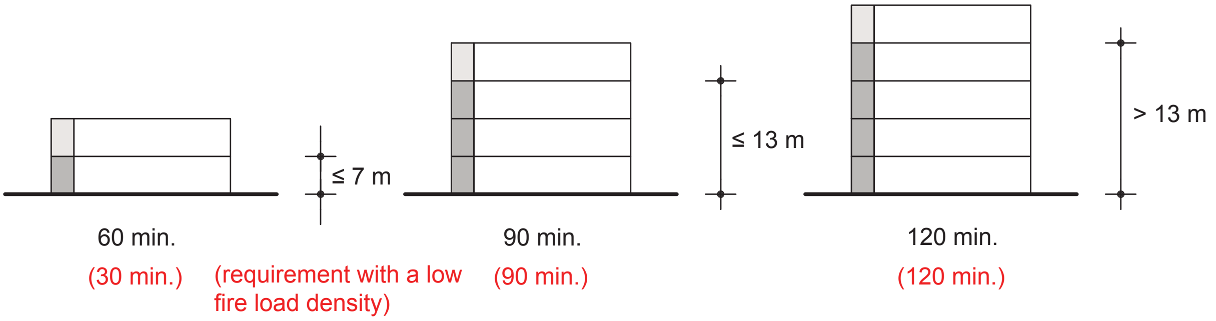

Figure 2.3 Requirements for the main load bearing structure of residential buildings
Source: Verdiepingbouw met staal;
Redrawn by the author

Non-residential buildings

The requirements of the decree for the main load bearing structure of non-residential buildings is set with the same reasoning as for residential buildings. The difference is that there is a distinction in building function with non-residential buildings. Sometimes stricter requirements are set if the main load bearing structure of a non-residential building contains areas with day and night residence. So there is a distinction between ”sleeping buildings” and ”non-sleeping buildings”.

The requirements of sleeping buildings (like hotels, hospitals and prisons) are higher than the requirements for non-sleeping buildings (like offices, company buildings, schools and stores). The higher the building, the higher the requirements, see figure 2.4. The requirement is 30 minutes lower if it can be demonstrated that the fire load density is lower than 500 MJ/m^2 floor space. Incombustible materials like steel and concrete almost always meet this criterion while CLT structures do not.

Reduction of the requirements

According to the decree, the fire resistance of the main load bearing structure can be reduced in the following situations:

- With a low fire load density
- With the use of a sprinkler system

As said before, the requirements can be reduced if it can be demonstrated that the fire load density is lower than 500 MJ/m^2. The value 500 MJ/m^2 is comparable with 25 kg of spruce, so this reduction is not applicable for CLT structures since 25 kg of spruce represents 60 mm of CLT per square metre which is very little.

A sprinkler system extinguish a fire in an early stage. The expansion of the fire and damage is therefore limited. In the Netherlands, the effect of a sprinkler system is expressed in a reduction of the fire resistance of the main load bearing structure. The reduction can be 30 or 60 minutes. The reduction which can be used is dependent of the local authority, it is therefore recommended to consult the local authority and fire department in an early stage of the design phase.

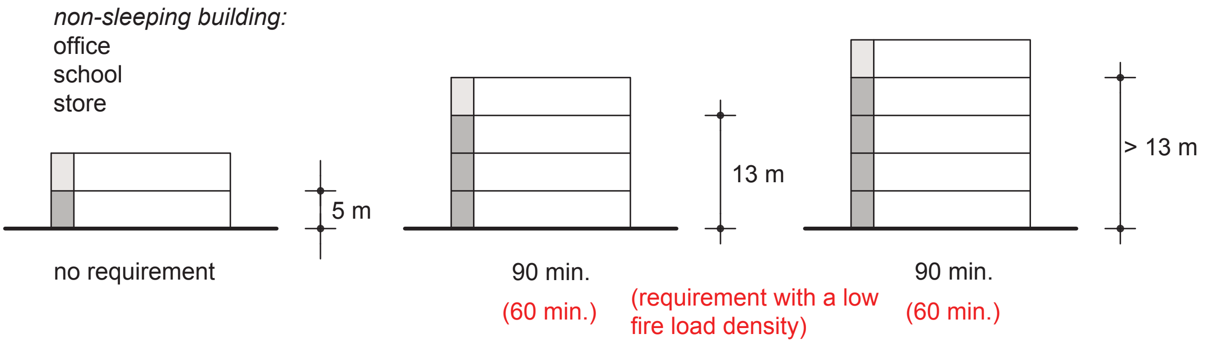

Figure 2.4 Requirements for the main load bearing structure of non-residential buildings
Source: Verdiepingbouw met staal
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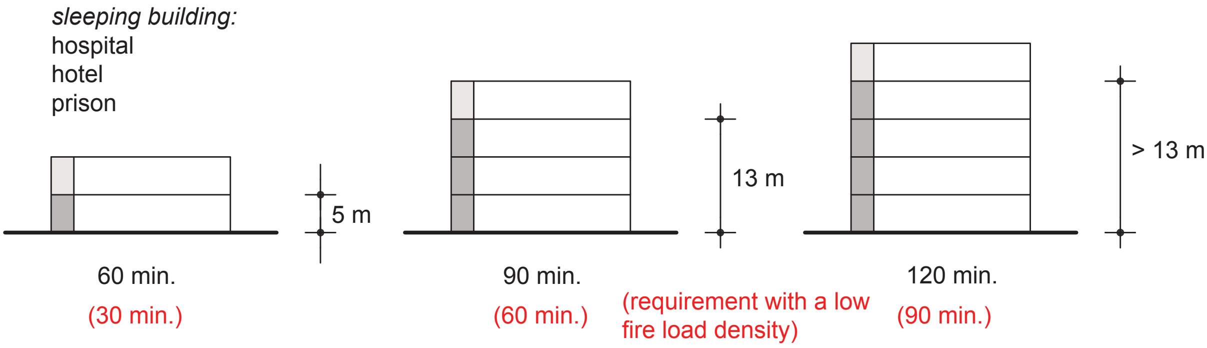

Figure 2.5 Requirements for the main load bearing structure of non-residential buildings
Source: Verdiepingbouw met staal
Redrawn by the author

2.2.3 Reaction to fire

Besides resisting fire, the building decree sets requirements on the reaction to fire of building materials. This regards the combustibility and smoke development of the used materials in fire and smoke compartments.

Building materials must be tested and evaluated by European provisions so that the building material can be assigned to a fire class. Three properties are tested to assign a specific fire class, these are: fire propagation, smoke development and flame droplets.

The European fire class system is divided over seven classes A1, A2, B, C, D, E and F, see table 1. A1 is the highest class and therefore the safest. Materials which are incombustible and have no contribution to the fire development, like steel, are placed in this class. Materials that ignite quickly and develop intense heat are placed in the lowest class which is class F.

Table 1 European fire class

Euro class	Propagation of fire	Contribution to fire	Practice
A1	flashover not possible	no contribution	not combustible
A2	flashover not possible	very little contribution	practical not combustible
B	flashover not possible	limited contribution	hardly combustible
C	flashover after 10 min.	average contribution	combustible
D	flashover between 2 - 10 min.	high contribution	well combustible
E	flashover in less than 2 min.	very high contribution	very combustible
F	not tested	not determined	extremely combustible

The fire class regarding floors and floor coverings are the same as the normal fire class in table 1 but are denoted by a subscript fl, so the classification of a floor in class B is denoted by B_{fl}.

Materials that are classified in class A1 does not require tests for smoke development and flame droplets. These properties are only tested for class A2 till D.

Table 2 European fire class

Euro class	Smoke production	Flame droplets
A1	no test needed	no test needed
A2	s1 - slight smoke production	do - no droplets
B		
C	s2 - average smoke production	d1 - droplets burn less than 10 sec.
D	s3 - large smoke production	d2 - droplets burn more than 10 sec.
E		
F		

Table 3 represents the regulations regarding reaction to fire of structures. The table is a general guideline for several building functions. For the regulations of a specific building function, one has to refer to the building decree.

Table 3 regulations considering reaction to fire

Structure adjacent to:	Propagation expressed in European fire class
Inside	
Fire and smoke free escape route	B, C (non enclosed escape routs like a corridor)
Smoke free escape route	B, C (non enclosed escape routs like a corridor)
Other	D
Outside	
Fire and smoke free escape route	B, C (non enclosed escape routs like a corridor)
Smoke free escape route	B, C (non enclosed escape routs like a corridor)
Ground floor facade up to 2,5 m in height	B
Other	D
Floor, stairs	
Fire and smoke free escape route	C_{fl}
Smoke free escape route	C_{fl}
Other	D_{fl}

2.3 Acoustic requirements

Buildings must be designed and constructed in such a way that the generation and spread of annoying noise is limited. The sound insulation in a structure is affected by several factors, like:

- the structure
- the composition of separating walls
- the composition of separating floors
- floor coverings
- the design and type of façades and windows
- the dimensions of rooms
- possible construction flows

Therefore the building decree sets requirements on:

- Protection against external noise (section 3.1)
  The façade protects the residential function against traffic noise, aircraft noise and other external noises. The performance, the characteristic noise reduction, is set by NEN 5077
- Protection against installation noise (section 3.2)
  Several installations, like a toilet, a water heater or elevator, produce unwanted noise in buildings. The performance, the characteristic noise reduction of installations, is set by NEN 5077
- Noise control between residential area with the same function (section 3.3) To protect buildings against airborne and impact sound, requirements are set by NEN 5077 for the characteristic sound-insulation index
- Limitation of reverberation (section 3.4) Limitations on reverberation are set to prevent annoying sounds in buildings. Requirements are imposed to the total sound absorption in enclosed spaces by NEN-EN 12354-6
- Noise control between residential area with a different function (section 3.5) This section states the requirements for airborne and impact sound insulation between residential areas set by NEN 5077

Sound insulation facade

The sound insulation of a facade depends on the sound insulating properties of the facade and the dimensions of the receiving room behind the facade.

The building decree states a minimal noise protection ($G_{A;k}$) of 20 dB(A). The maximum sound pressure level in an area according to the decree is no more than 35 dB(A). The minimum noise protection of 20 dB(A) combined with the maximum sound pressure level of 35 dB(A) gives the maximum sound load 55 dB(A). When the sound load increases, for example due to traffic noise, the sound insulation has to increase in proportion.

Installation noise

The minimum requirements for installation noise in residential areas ($L_{i;A;k}$) is 30 dB(A). This requirement could have consequences for a structure with nearby installations. When functional uses are above each other, service shafts demands extra attention, especially if this is the case of a standpipe. Connections between ducts and structure must be avoided in all cases.

Sound insulation between rooms

The requirements for sound insulation in residential areas are set by minimum values for airborne sound insulation (I_{lu}) and impact sound insulation (I_{co}). The decree states space independent values for airborne sound insulation resulting in the characteristic sound insulation-index ($I_{lu;k}$) for airborne sound (tabel 4). The advantage of space independent values is the possibility to compare different structures with different materials and properties.

Table 4 sound insulation requirements between rooms

Location	Airborne $I_{lu;k}$ (dB)	Impact I_{co} (dB)
Between two residential functions: from a confined space (for example a traffic area) to a residential area of an adjacent residential function.	≥ 0	≥ 5
In the same residential function	≥ -20	≥ -20
Residential function situated in a residential building: from a confined space (for example a traffic area) to a residential area of an adjacent residential function.	≥ 0	≥ 5

2.4 Conclusion

After reviewing the most important requirements for Dutch medium rise buildings, the following conclusions can be made which are of most importance for the design of the medium rise building in chapter 5.

- Since the actual Dutch structural requirements will soon become to expire, the European structural requirements (Eurocode series) with the Dutch national annex will be used.
- Regarding fire safety, a medium rise building must be designed to acquire a fire resistance of 120 minutes. This requirement can be decreased when sprinklers are included in the architectural design of the building.
- The reaction to fire of CLT panels must be investigated since this has consequences for the application of the panels. If the panels are classified in a low fire class it means that the panels must be covered with a protective layer. The possibility of designing areas with uncovered timber panels is then discounted.
- Regarding acoustics, the performance of CLT and CLT structures must be investigated in order to fulfil the requirements. The airborne and impact sound insulation values of table 4 are of most importance and will be governing for the acoustic performance of the building.

CHAPTER 3, CROSS LAMINATED TIMBER PANELS

Cross laminated timber panels are massive timber panels which consist of several softwood boards stacked on one another at right angles and glued together. Because of the cross wise gluing under high pressure, CLT panels have fibres running in two directions which makes the panels very stiff and stable.

CLT offer opportunities to use timber in situations where designers would normally use traditional materials such as steel, concrete and masonry. As with any structural material it is essential that designers understand how to achieve its potential while respecting its limitations. Therefore this chapter reviews scientific research and the application in practice of the panels. Besides structural aspects, this chapter also covers non structural aspects of design including acoustic performance and fire safety.

Figure 3.1 Cross laminated timber panel
Source: Finnforest

3.1 Scientific research

3.1.1 Resisting of lateral loads

One of the most important design issues considering medium rise timber buildings is resisting of lateral loads. Many studies have been made in the past years covering this subject. For instance, Frangi F., and Smith I., [17] presented an overview of general design issues for tall timber buildings. In the research the term tall timber building applies to buildings that are at least 10 stories tall, with the practical upper limit being about 20 stories. In tall slender buildings, the important internal actions are horizontal shearing forces and axial forces due to gravity and bending. Controlling the horizontal deformation can be problematic. Particularly in order to control the amplitude of the horizontal deformation, it can be anticipated that it will often be impossible even for timber buildings in the region of eight or ten stories high to employ all timber solutions if the building geometry is slender. For very tall slender structures a composite system can be a solution. The composite system can consist of timber substructures arranged around a structural steel or reinforced concrete core that adds stiffness for global control of horizontal deformation.

Another study on resisting lateral loads is conducted by Chapman J., [8]. Three timber based systems are studied for resisting lateral loads that will ensure relatively open floor spaces. The lateral load resisting systems are termed: framed, circular core and shear walls. Since only relatively low stresses occur in the proposed three systems, they can be built with timber that is mainly below ‘structural grade’ which is more economical. The lateral load resisting systems are considered from the viewpoints of structure, architecture and economics. The framed system is a heavy timber frame comprising timber beams and columns. The circular core is made of large CLT panels that are assembled together to form a vertical circular tube. The shear wall system comprises large CLT panels in each external wall. The circular core resulted to be very efficient and could be suitable for buildings higher than six stories. Regarding architecture, all three of the proposed lateral load resisting systems seems to have limitations. The form of the tube core cannot be modified, except to vary the diameter and replacement of the circular shape with an ellipse. The shear wall system relies on long walls on each external wall which impede light entering the building. The most flexible arrangement would be a frame system on each external wall. It would leave the floor areas free except for internal columns. Also, windows can be placed within the frame construction allowing light to enter the building.

The shear wall system could be improved by making fenestrations so that light can enter the building. The problem however is that strength and stiffness of the shear walls is reduced by the fenestrations. The behaviour of these fenestrated CLT shear walls is studied by Dujic B., Klobcar S., and Zarnic R., [10]. The main goal of their study was to provide information on how to estimate the racking strength and stiffness of CLT walls with openings and to recognize how the shape and location of the openings influence the shear capacity and stiffness of the walls. To evaluate the shear strength and stiffness reduction for different fenestrations, a numerical model was utilized and verified with experimental tests on full size CLT walls. To reduce the number of tests a numerical parametric study was performed for 36 opening configurations with three different lengths.

The study concluded that non-fenestrated CLT walls have a relatively high stiffness and load bearing capacity. Therefore, the critical elements that govern the CLT shear wall design are anchors connecting the panels to the building foundation. The fenestrated wall panels with large openings have a lower shear stiffness but its load bearing capacity is not significantly reduced. The parametric study showed that the openings with an area up to 30% of the wall surface do not reduce the load bearing capacity significantly, while the stiffness is reduced for about 50%. The reason for this is because failure is mostly concentrated in the anchoring area and in the corners around openings with smashing and tearing of the timber.

3.1.2 Fire safety

Another important design aspect besides resisting lateral loads is fire safety. Fontana M., Frangi A.,and Knobloch M., [15] studied several fire design concepts for tall timber buildings. Fire safety is an important design aspect because there is a fundamental difference between tall buildings and low rise buildings with regard to evacuation and fire resistance criteria as people in tall buildings can often not be evacuated by the fire brigade using external equipment. So Fontana et al conclude that tall buildings should be designed in a way that the occupants can survive a full burn-out of the fire compartment while remaining in another part of the building. A total burn out without loss of structural stability and some main compartmentation must be guaranteed by the building structure. For the use of timber this often leads to the protection of the timber by non combustible material or to mixed construction. A feasibility study shows that mixed tall timber-concrete buildings in combination with technical and organizational measures can be built to be as safe as typical traditional non combustible tall buildings.

The fire behaviour of individual CLT panels is studied by Frangi A., Fontana M., Knobloch M., and Bochicchio G., [16]. The fire behaviour is experimentally and numerically studied. Particular attention is given to the comparison of the fire behaviour of CLT panels with homogeneous timber panels. The results of a FE-thermal analysis have shown that the fire behaviour of CLT panels depends on the behaviour of the single layers. If the charred layers fall off, an increased charring rate needs to be taken into account for the next layer. The same effect was observed for initially protected timber members after the fire protection has fallen off. Thus, the fire behaviour of CLT panels can be strongly influenced by the thickness and the number of layers. Fire tests conducted with horizontal furnaces showed that the charred layers of 3-layered CLT panels fell off earlier than in the case of homogeneous timber panels. Thus, the measured charring rate of 3- layered CLT panels was higher than for homogeneous timber panels. Fire tests on CLT panels performed with a vertical furnace did not show falling of charred layers and the measured charring rates well agreed with the one dimensional charring for homogeneous solid timber panels. Thus, vertical structural members (walls) may show a better fire behaviour in comparison to horizontal members (slabs) due to a less pronounced falling of the charred layers. Further, the behaviour of the bonding adhesive at high temperature can influence the falling of the charred layers and thus play an important role in the evaluation of the fire behaviour of CLT panels. The calculation of the charring depth of cross-laminated solid timber panels should take into account the influence of the falling of charred layers as well as the influence of longitudinal and transverse joints, for example by assuming a notional charring rate βn that is higher than the one-dimensional charring rate $\beta 0$.

3.1.3 CLT design

A general design method for CLT panels is presented by Blass J., and Fellmoser P., [2]. The reason for this is that the design rules for CLT panels are presently given in technical approvals, where also strength and stiffness values based on tests are included. The stress distribution and the deformation behaviour of CLT panels were analyzed using the composite theory. The shear influence in solid wood panels was determined using the shear analogy method. The rolling shear modulus significantly influences the load and deformation behaviour of CLT panels. For span to depth ratios of at least 30, the influence of shear may be disregarded for loading perpendicular to the plane. In this case, the composite theory is taken as a basis for the design of solid wood panels. The calculation considers both, layers loaded parallel and perpendicular to the grain. Strength and stiffness of CLT panels may be determined using basic strength and stiffness values of the single layers, taking into account the homogenization caused by the parallel loading within a layer. This is a conservative method and disregards the lamination effect leading to a considerable improvement of the panels compared with boards representing the base material. The researchers tested numerous panels which showed that the strength and stiffness values of CLT panels composed of C24 may be calculated based on properties of strength class GL28h. Finally a strength class system for CLT panels is given in order to simplify the design of the panels. In this system, the characteristic strength, stiffness and density values of CLT panels are given depending on type of stress and direction of stress with regard to the grain direction of the outer skins.

Another design method is presented by Mestek P., Kreuzinger H., and Winter S.,[22]. Their research is based on the shear analogy and explains the influence of shear deformation of the cross layers and the resulting consequences to the load bearing behaviour of the panels. The main focus lies in the stresses caused by concentrated loads. Normal and shear stress distribution in the area of a concentrated load is calculated and evaluated for uniaxial spanned systems according to the shear analogy and by an FEM-calculation with shell elements. The analysis showed that in the immediate area of a concentrated load the longitudinal stress does not stay plane. This causes stress peaks in the single layers, which can be taken into account by the method of the shear analogy. A simplified method shows how to consider the influence of shear deformation on the longitudinal strain of a simply supported beam stressed by a concentrated load. The static systems consisted of simply supported beams loaded in the middle of the span. Their cross sections were always symmetric with a constant thickness of the single layers. The varying parameters were the ratio of span to thickness and the number of layers of the panel. The analysis focused on the comparison of the longitudinal stresses according to the shear analogy and composite theory. The analysis demonstrated that the increase of the longitudinal stresses due to the influence of the shear deformation only depends on the ratio of span to thickness and, for symmetric cross sections, the number of layers has an irrelevant influence.

An important design consideration in medium rise timber buildings is the low compression strength perpendicular to the grain of the timber. Serrano E., and Enquist B., [24] tested the compression strength of square CLT specimens consisting of three layers. One test set up consisted of a uniform compression over the complete surface of the specimen, similar to what is defined in the current European test standard for glued laminated timber. In addition, several other test set-ups involving loading of only parts of the specimens square surface by line loads was investigated. The use of line loads aimed at investigating the effect of load distribution within the test specimen, and also to investigate possible boundary effects when the line load is applied close to the specimens edge. In practical design, the line load used in the tests would correspond to the load transfer from a wall structure clamping a CLT-slab. The results showed that the compression strength, defined according to Eurocode 5 as the load at which a 1% permanent compressive strain is obtained, is indeed dependent on the relative size of the load application area, its orientation relative to the surface grain direction and its distance to the edge of the specimen. Other conclusions are that test methods, test evaluation methods as well as design criteria for compression perpendicular to grain in CLT should be revisited.

3.1.4 Joints

Blass J., and Uibel T., (2006) [3] developed a proposal for calculating the load carrying capacity of joints with dowel type fasteners in CLT panels which are positioned perpendicular to the plane of the panels. The researchers examined the parameters of CLT panels for calculating the load carrying capacity of dowel type fasteners. For the characteristic density of CLT panels made of spruce a value of 400 kg/m³ is proposed. On the basis of a statistical analysis of 617 embedment tests they were able to determine the embedment strength of CLT panels for dowel type fasteners. The presented functions depend on type and diameter of fasteners, density and particularly on the angle between load and grain direction of the outer layers. The load carrying capacity of a steel to CLT panel connection is derived. Tests were also made in order to verify the calculation model. When the load in the tests reached the load carrying capacity, the connections showed almost ideal plastic load displacement behaviour. Even plug shear or splitting of the outer layers do not initiate brittle failure. These facts showed the reinforcement effect in crosswise laminated structures.

In continuation of the research project, Blass et al [4] examined the load carrying capacity of edge joints with dowels and screws in CLT. For calculating the capacity, the parameters of the embedment strength and withdrawal capacity were examined. The test program includes tests with two different load directions and five possible positions of fasteners with different diameters in relation to the thickness of the layers and in relation to the grain direction. On the basis of statistical analysis of a multitude of test results it was possible to develop functions for predicted values of these parameters. The validity of the presented equations is limited to CLT with a characteristic density of 400 kg/m³ made of spruce. The tests verified the calculation of the load carrying capacity. The long term behaviour of laterally loaded edge joints is also examined. During the long term test, the displacements are measured periodically and the climate is recorded. After three years the specimens will be unloaded and the remaining load carrying capacity will be determined in short term tests.

Follesa M., Brunetti M., Cornacchini R., and Grasso G., [14] investigated mechanical joints between CLT panels working in the same plane (i.e. vertical joints between wall panels and horizontal joints between floor panels). Tests were made in terms of joint strength, stiffness, ease and speed of execution and total costs including the costs of fasteners and labour. Three different joints were tested, which were (figure 3.2): The tenon joint with double groove on the panel edges and a central multi-layered timber panel, the half lapped joint and the leaf joint with a groove on the same side of each panel and a connecting multi-layered timber panel. Tests on two very different types of fasteners, screws and nails, were made to point out the differences in terms of strength, stiffness, simplicity and velocity of construction and costs. The comparison between test results and calculated values according to Eurocode 5 shows that, due to the cross laminated effect, test results are more than 1.5 times higher than the calculated ones. The in-place tests shows that the nailed joints are very cheap in comparison with screwed joints. The cheapest is the nailed half lapped joint, although the difference between the three types of nailed connections are very small.

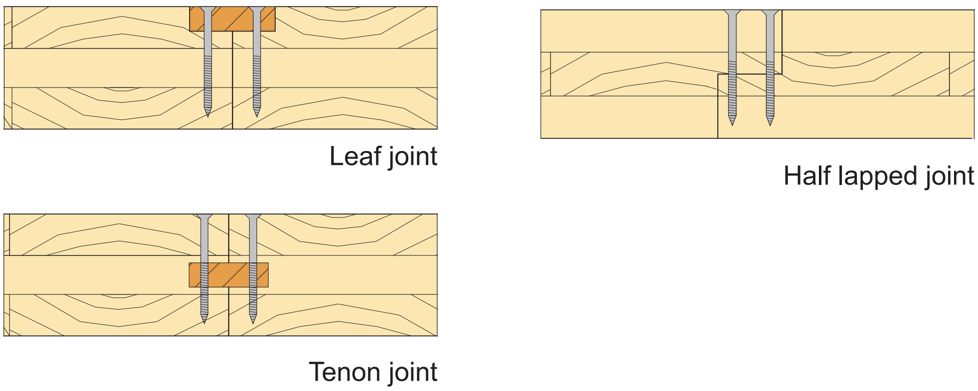

Figure 3.2 Tested joints
Source: Drawn by the author

3.1.5 Costs

The success of medium rise timber buildings requires competitive cost and cost efficient production of buildings. However, cost-efficient production of these buildings is still in a learning phase. The timber itself may not significantly influence the production cost, but there is potential for cost reduction in the activities associated with a change from concrete to timber. There is not much direct profit in structural costs of medium rise timber buildings but experts however, see high potential in reducing overall construction cost in careful layout design of load bearing walls, prefabrication and construction methods. Few cost studies have yet been conducted, but existing data [18] indicates that medium rise timber buildings costs 8 to12% more than reinforced concrete buildings. However, with increased construction, the production cost of medium rise timber buildings could be reduced due to the influence of “learning”.

3.2 Structural design

3.2.1 Structural properties of a CLT structure

The principal load bearing structure of a CLT building is the same as any other massive building structure like precast or in situ concrete which consists of large wall and floor sections used to cover large areas. This is a great advantage because floor and wall panels can be designed having a load bearing and separating function. A CLT structure is best comparable with a precast concrete structure, but has one great advantage which is its reduced dead weight. The densities of CLT and concrete are respectively 500 kg/m^3 and 2400 kg/m^3. This means that less dead weight lays down on the foundation and no heavy tower crane is needed for the placing of the CLT panels (a mobile crane is sufficient) which can safe construction costs considerably.

A CLT structure is lighter than an equivalent concrete structure, but is still heavier than a timber frame structure. This is useful because tall buildings needs a high dead weight in order to reduce overturning forces that are generated by wind loads. Thus the increased dead weight of the CLT structure results in less mechanical holding down requirements compared to the timber frame structure.

Governing for the design of medium rise timber buildings will be horizontal loads caused by wind loads. The principle of the load transfer is demonstrated in figure 3.4. Horizontal wind loads acting on the façade are transferred to the floors. These horizontal loads are then transferred to the stabilizing walls which results in tension, compression and shear forces. The lower sides of the wall elements needs to be anchored to the underlying structure so tension and shear forces can be transferred to the underlying floor and eventually to the foundation. The tension forces can become quite large when little dead weight is present. It is therefore useful to design the structure in such a way that as much as possible dead weight is carried by the stabilizing walls.

Figure 3.3 Massive timber structure
Source: Buro Tichelaar

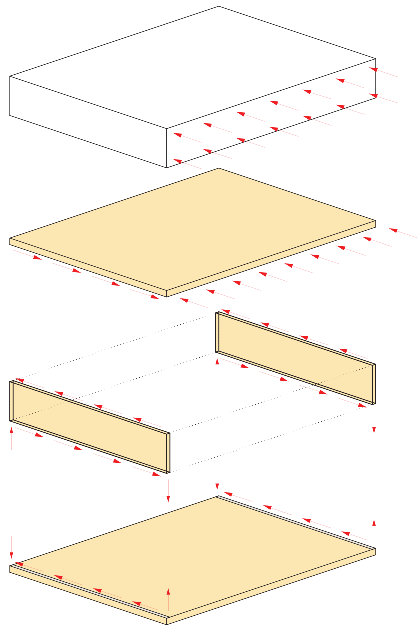

Figure 3.4 Diaphragm action between walls and floors
Source: Drawn by the author

The use of CLT panels has the following structural advantages:

- Few defects due to the inherent robustness of the panels during transport and construction.
- High axial capacity due to the large bearing area.
- High shear strength to resist horizontal loads.
- Shallow floor structures.
- Dead weight reduces the need for mechanical holding down to resist overturning forces.
- Structural fixings are easy to provide.
- The structure contributes to the fire resistance.
- Enhanced airtightness.
- In some situations, second fix items and cladding can be fixed directly to the panels using light weight power tools.

3.2.2 Structural principles

Just like timber frame construction, CLT structures can be build using the platform or balloon frame method. Each method has its advantages and disadvantages and the choice for the most suitable method depends on several considerations.

The platform frame method

The platform frame method is the most commonly used method for construction. With this method each floor forms a working platform for the next storey. So the wall panels are placed first and the floor panels are placed above. Above this floor, the wall panels of the next storey are placed and so on. The advantage of this method is that joints between wall and floor elements are being squeezed together and do not open up. A disadvantage is the occurrence of compression stresses perpendicular to the grain in floor elements clamped between the walls. The maximum allowable compression stress perpendicular to the grain of timber is quite low. So when building height is increased, the stresses perpendicular to the grain of the lower storey floors increases also. Eventually no more storeys can be added due to crushing of the lower floor elements. In order to built higher, it is necessary to decrease the stresses perpendicular to the grain of the floor elements which can be done by increasing the bearing area on the floor panel.

The balloon frame method

With the balloon fame method the load bearing wall elements are continuous and run from the soleplate to the eave line with intermediate floor structures supported inside the wall on a ledger. Because the floor panels are supported inside the wall panels, no stresses perpendicular to the grain occur, which means that the floor panels are not governing for the maximum weight the wall panels like the platform method. The main disadvantage of this method is the lack of a working platform during construction for work on upper floors. Whereas construction workers can readily reach the top of the walls being erected with platform construction, balloon construction requires scaffolding to reach the top of the walls, which are often two or three stories above the working platform.

Hybrid structures

There are several ways to combine a CLT structure with other building methods like timber frame construction to produce a more efficient structural form. External non-load bearing walls comprising highly insulated panels can be used to acquire a more energy efficient building. With this kind of structures, care has to be taken with differential movement between load bearing and infill panels.

CLT floor panels can be used to form a timber-concrete composite floor, where the CLT panel is used as a permanent formwork. When timber and concrete are connected with shear plates and screws, a combined floor structure is acquired with better performances than both materials on their own. However, the disadvantage of this hybrid structure is the waiting time for the concrete to harden and the exposure to moisture of the timber when pouring the concrete.

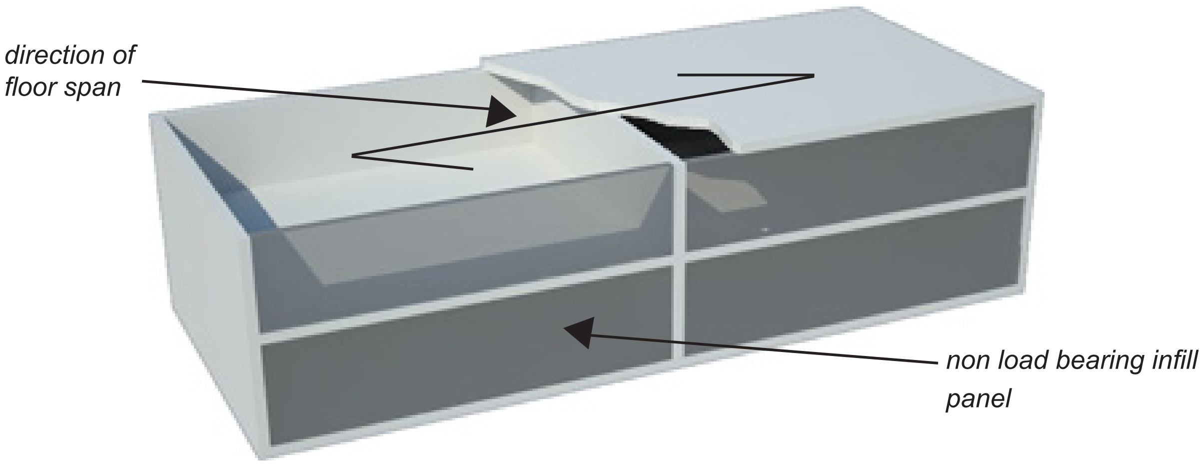

Figure 3.5 CLT hybrid structure
Source: Trada WIS 2/3-61;
Redrawn by the author

3.2.3 Detailing

There are many possibilities for the connection of CLT panels. All connection types are easy to provide and require the use of lightweight power tools.

Six key connections are presented which are used to join cross laminated timber wall and floor panels.

Wall details

Wall panel – foundation (Detail 1)
Wall panel – wall panel (Detail 2)
Wall panel – wall panel, T-section (Detail 3)

Floor details

Floor panel – floor panel (Detail 4)
Wall panel – floor panel, platform method (Detail 5)
Wall panel – floor panel, balloon method (Detail 6)

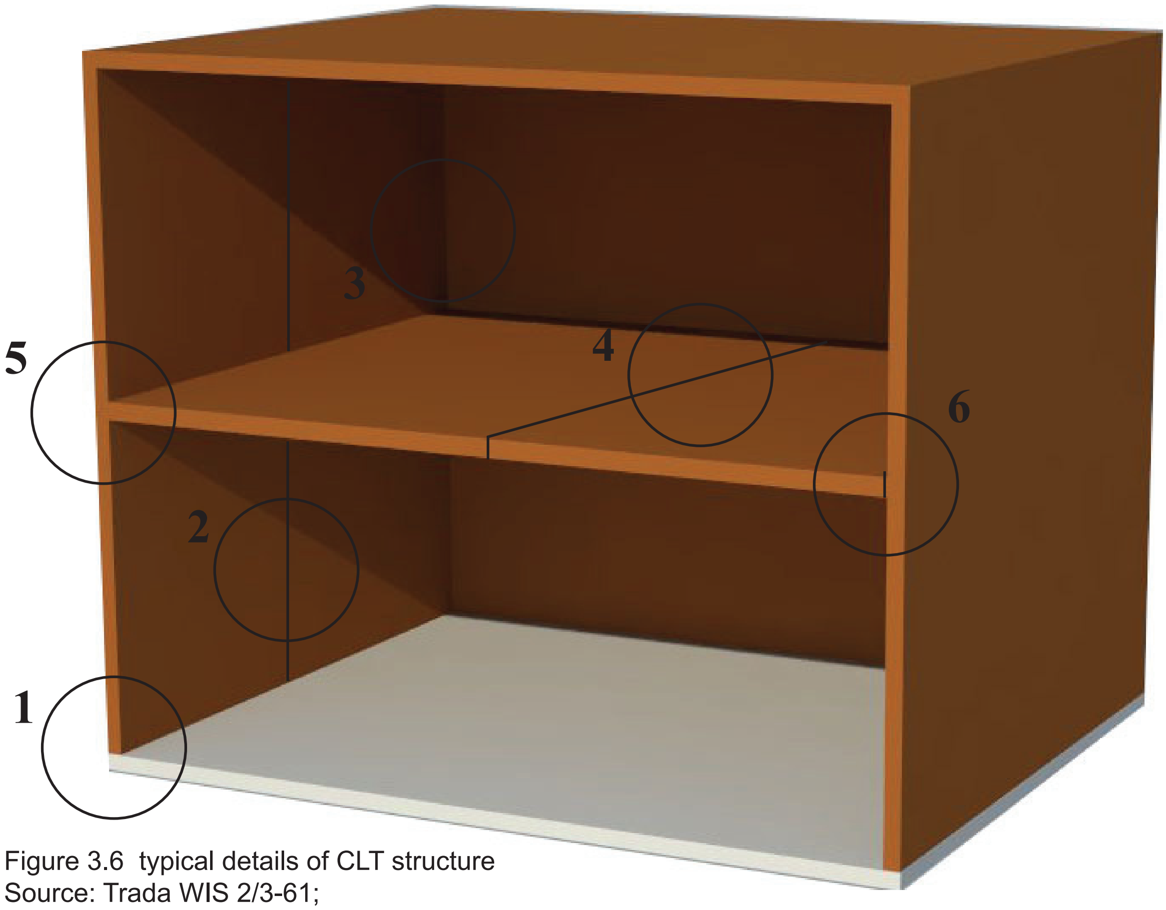

Figure 3.6 typical details of CLT structure
Source: Trada WIS 2/3-61;
Redrawn by the author

Wall panel – foundation (Detail 1)

The wall panels on the ground floor usually sit on a level soleplate with DPC foil fixed to the concrete floor slab. A soleplate will ensure that a level surface is provided before the CLT panels arrive on site. The soleplate can be positioned, fixed and grouted to enable erection and progress unhindered by the need to level the wall panels. It is important to have enough margin between soleplate and slab to provide the necessary grout.

In medium rise design, the soleplate will become governing due to the limited compression stress perpendicular to the grain. The soleplate can therefore be leaved out if the concrete floor slab is flatten out. To provide a full bearing area, wall panels are placed on a grout bedding and are fixed with a steel angle. Note that the anchors must be designed to resist the applied horizontal loads.

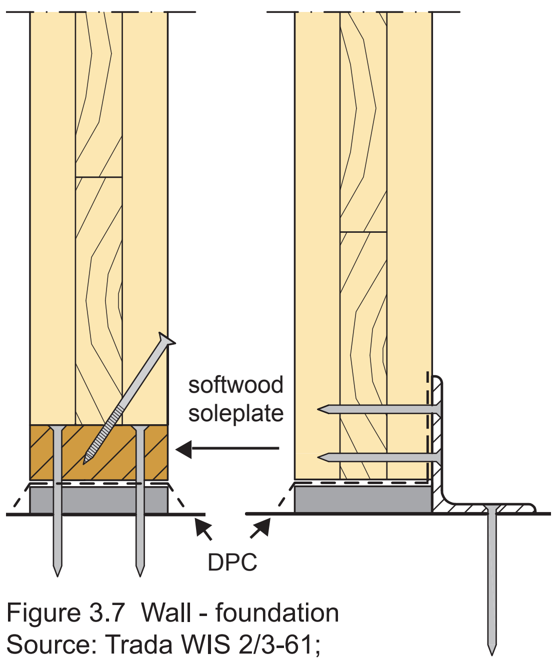

Figure 3.7 Wall - foundation
Source: Trada WIS 2/3-61;
Redrawn by the author

Wall panel – wall panel (Detail 2)

The vertical connection between wall panels may use an engineered timber jointing piece and offers the potential to embed service runs. Other connection methods are like those shown in detail 4.

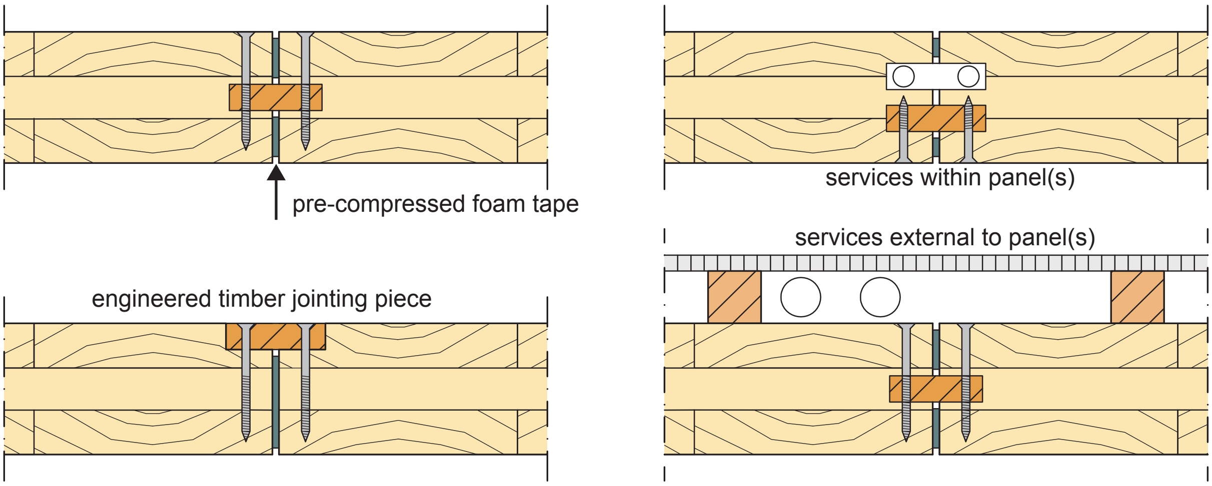

Figure 3.8 Wall - wall, straight
Source: Trada WIS 2/3-61;
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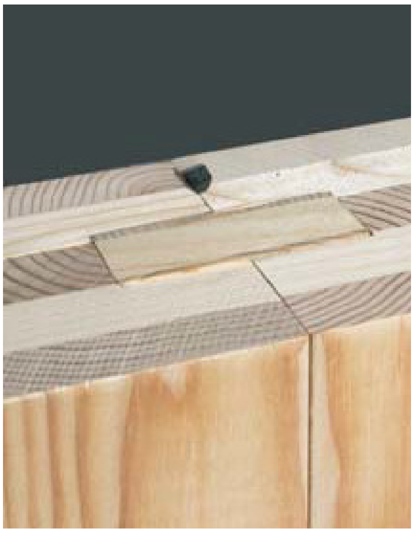


Figure 3.9 Wall connection with jointing piece,
Source: Binderholz

Figure 3.10 Services within panel
Source: Binderholz

Wall panel – wall panel, T-section (Detail 3)

Screws are drilled from the outer to the inner panel. The use of pre-compressed foam tape ensures the air tightness of the connection.

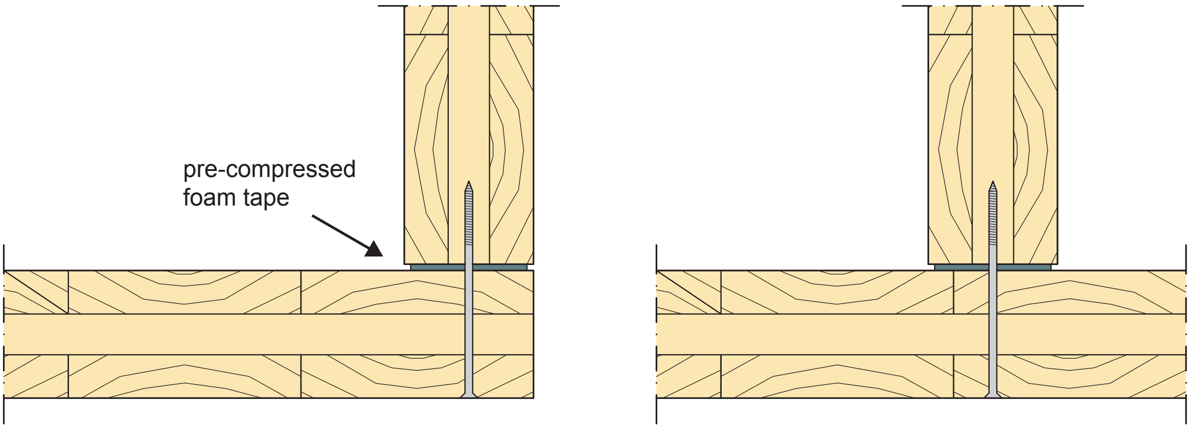


Figure 3.11 Wall - wall
Source: Trada WIS 2/3-61;
Redrawn by the author

Floor panel – floor panel (Detail 4)

This connection is usually a half-lapped joint milled in the factory. The elements are connected with screws or an LVL jointing piece.

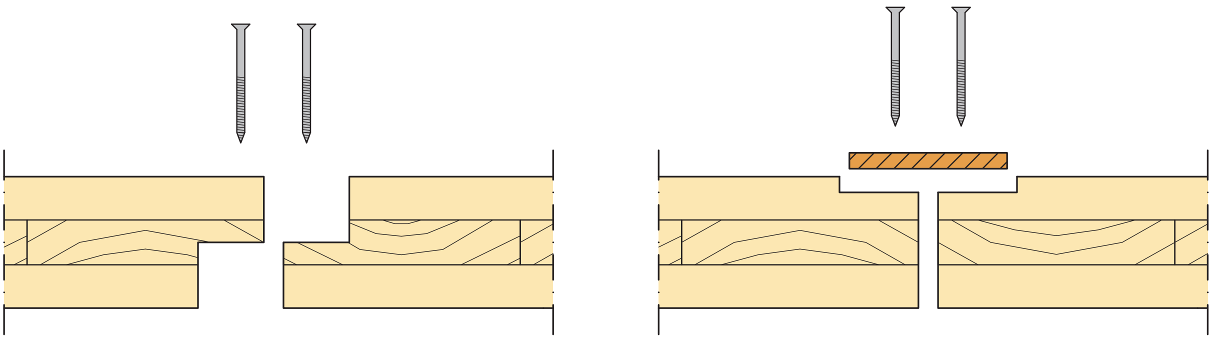


Figure 3.12 Floor - floor
Source: Trada WIS 2/3-61;
Redrawn by the author

Figure 3.13 Half-lapped joint
Source: Trada WIS 2/3-61

Figure 3.14 Securing of floor panels
Source: Binderholz

Wall panel – floor panel, platform frame method (Detail 5)

This connection can be made using a steel angle or by drilling a screw over an angle in the wall and floor panel.

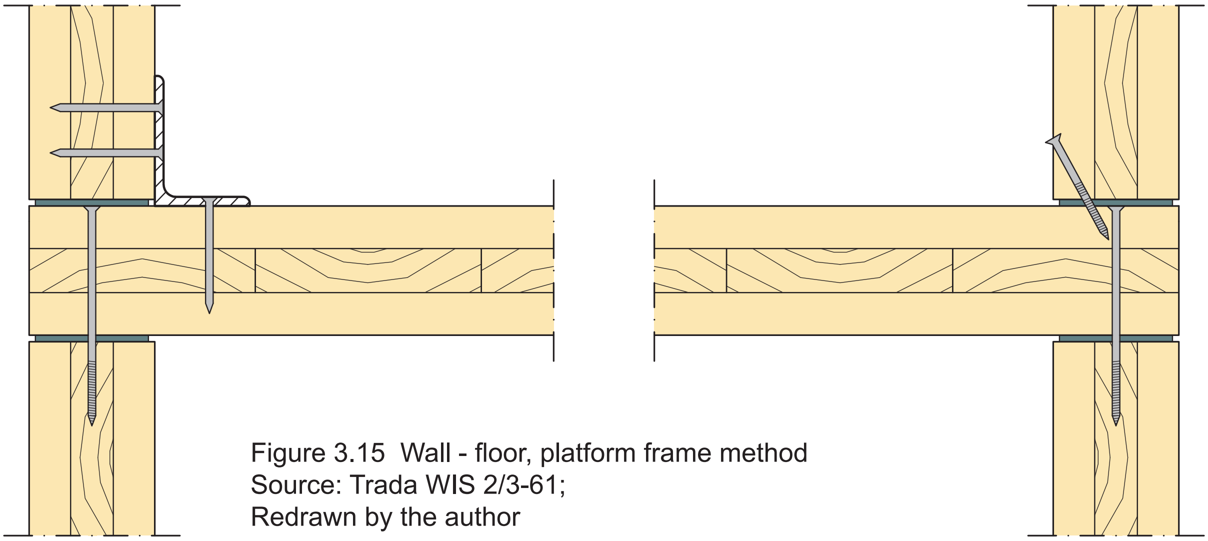


Figure 3.15 Wall - floor, platform frame method
Source: Trada WIS 2/3-61;
Redrawn by the author

Figure 3.16 Platform frame method
Source: Trada

Figure 3.17 Balloon frame method
Source: Informationsdienst Holz

Wall panel – floor panel, balloon frame method (Detail 6)

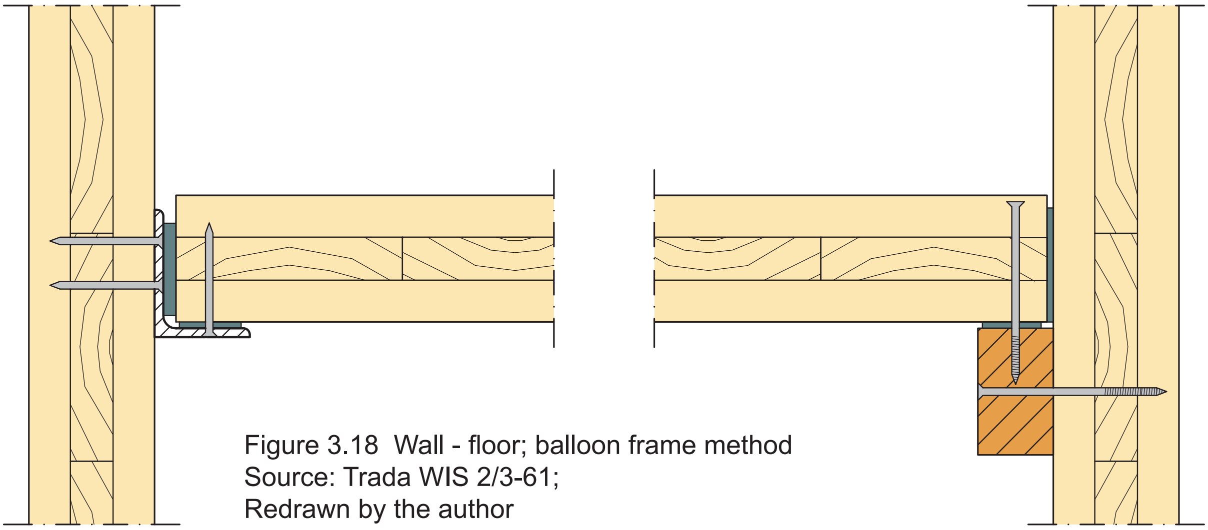


Figure 3.18 Wall - floor; balloon frame method
Source: Trada WIS 2/3-61;
Redrawn by the author

3.2.4 Architectural design

Floor structures with spans of 8 metres can be achieved with the use of CLT panels. Because of this, it is possible to design buildings and dwellings with a common Dutch arrangement. The use of CLT panels gives designers a large degree of flexibility due to its high performances and simplicity. It is useful to place stabilizing elements in the façade or near staircases and sanitary sections. In this way the designer has even more freedom in his arrangement.

The system is very flexible because expansions or rearrangements are easy to realize when the design takes them into account. If for instance only the external and separating walls are designed as load bearing elements, the arrangement of the interior space can be completely free. The interior can now be adjusted to different occupier needs.

There are nearly no limitations to the design of a CLT structure and its finishes. Depending on the building physical performance, there are various lining materials available, which can be treated in any imaginable way (paint, wallpaper, plaster, etc), or kept untreated. There are likewise all materials and systems available for the facade (masonry, wood, etc)

3.3 Production and properties

3.3.1 Production

Cross laminated timber is produced mainly on the basis of boards from the edge of the trunk. The boards are excluded from flaws such as knots in order to produce high quality panels. The next step is to elongate the boards with a finger joint. Finger jointing is a method of connecting timber members to make a continuous member. Finger joints are made of a set of complementary rectangular cuts in two or more pieces of timber which are locked and glued together as shown in figure 3.19.

After making the finger jointed board members, they are edge glued to each other to Make single layered panels. The single layered panels are connected to each other at right angles using adhesives on the sides of the panels.

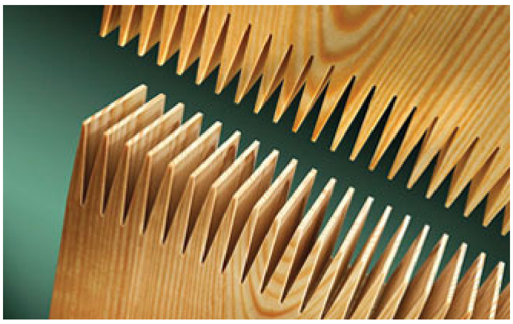


Figure 3.19 Precision finger joint
Source: www.kaiserbeaum.com

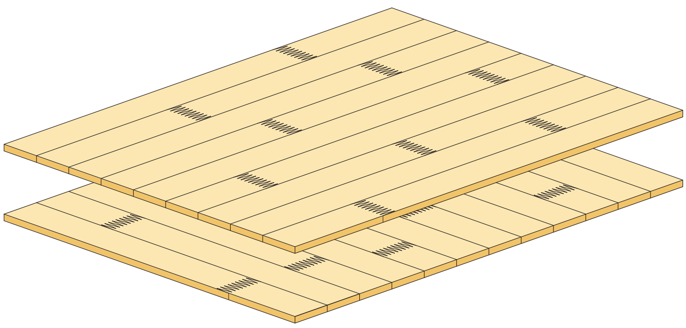


Figure 3.20 Edge gluing of boards
Source: Drawn by the author

Different kind of glues are being used for the production of the panels. Polyurethane (PUR) adhesives are normally used (formaldehyde and solvent free) although the less health friendly MUF and PRF adhesives may be used as well. The type of adhesive that is used is prescribed in the documentation of the manufacturer.

The completed CLT panels consists of three, five, seven or more layers. Their sizes vary by manufacturer, but typical widths are 0,6 1,2 and 2,95 metre while length can be up to 24 metre (transportation regulations may impose size limitations). The thickness of the panels can be up to 0,5 metre.

Their are different types of panel classes available which in most cases depends on the manufacturer. There is often a distinction between industrial and residential application and panels that are visible and non visible. The wood of the non visible quality is not sorted by its appearance. Therefore it is possible that every panel in this class has another color. The panels with visible quality have a uniform color and different wood species can be used like larch or douglas. The grain direction of the outer layers of the panels are normally oriented parallel to the applied loads. This is done in order to maximize the resistance of the panels. So the exterior layers for wall elements run parallel to the vertical loads and likewise for floor and roof systems the exterior layers run parallel to the span direction.

A nice aspect of CLT panels is that manufactures can incorporate cut-outs for windows, doors, ducts and chases with CNC routers in the factory. Electrical installations can also be integrated inside the panels. The recommendations on this matter is to stay ten centimetres from the edge of the panel and the grooves must be made along the grain direction of the outer layer.

All this extra labour does makes the panels more expensive. The price of the panels is based on the thickness and gross material, so timber that is cut out is also included in the price of the panel. Late modifications to openings or additional service runs is not well appreciated and must be avoided. Modifying panels on site can also be costly and time consuming and may effect the structural integrity of the panel. So it is essential that all openings are correctly designed so that they are correctly incorporated into the panels.

residential, visible | industrial, visible | not visible

Figure 3.21 Visible qualities
Source: Finnforest, Leno technical brochure

There are at least three suppliers of CLT panels in the Netherlands:

Table 5 suppliers and products

Supplier	Origin	Product
Stora Enso	Austria	CLT
	www.clt.info	
Enicon	Austria	KLH solid timber panels
	www.enicon.nl	
Finnforest	Germany	Lenotec
	www.finnforest.nl	

During transport and assembly, it is necessary to protect the panels from weathering by a tarp. Rain has no direct effect on the panels as they will dry quickly, but can leave stains on the panels. The stains are of less importance on normal quality panels but are very unwanted on visible quality panels because they discolour the panels.

3.3.2 Mechanical properties

Different methods have been adopted for the determination of the basic mechanical properties of CLT panels. Some of these methods are analytical in nature, while others are experimental. For floor elements, experimental evaluation involves determination of flexural properties by testing full-size panels or sections of panels with a specific span to depth ratio. The problem with the experimental approach is that every time the layout, type of material, or any of the manufacturing parameters change, and thus more testing is needed to evaluate the bending properties of all the different products.

At the time of writing this thesis, only a draft [28] is available on how to specify CLT panels. Since there is no European standard, the mechanical properties vary according to the manufacturer and timber species. Spruce is the most used timber species, but larch and pine panels are also available. The common strength grades for the laminates are in the range of C16 to C24 while some manufacturers also offer glulam grades GL24h to GL28h.

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/m^3 for spruce and 590 kg/m^3 for larch. The moisture content of the panels at delivery is typically 10 to 14 %. A proper moisture content is necessary because it has a significant effect on the performance of the timber. A stable moiture content prevents this effects and prevents dimensional variations and surface cracking of the panels.

Some typical strength properties of CLT panels composed of C24 timber can be seen in table 6.

Table 6 strength properties

Strength and stiffness properties CLT in N/mm² Timber - C24		
Bending strength		
- parallel to the grain direction	$f_{m,k}$	24
Tensile strength		
- parallel to the grain direction	$f_{t,0,k}$	14
- normal to the grain direction	$f_{t,90,k}$	0,4
Compression strength		
- parallel to the grain direction	$f_{c,0,k}$	21
- normal to the grain direction	$f_{c,90,k}$	2,5
Shear strength		
- parallel to the grain direction (in plane of slab)	$f_{v,k}$	2,5
- rolling shear	$f_{r,k}$	1,0
Modulus of elasticity		
- parallel to the grain direction	$E_{0,mean}$	11000
- normal to the grain direction	$E_{90,mean}$	340
Shear modulus		
- parallel to the grain direction	G_{mean}	690
- normal to the grain direction	$G_{r,mean}$	60
Charachteristic density	ρ_k	350
Average density	ρ_{mean}	420

3.4 CLT design

3.4.1 General

There are two different types of stresses which can occur in CLT panels due to loading perpendicular to the plane (floors) and loading in the plane of the panel (walls). Additionally, the strength and stiffness of CLT panels depends on the direction of the loading, parallel or perpendicular to the grain direction of the outer layers.

The primary direction of the load bearing capacity generally corresponds to the outer layer orientation so that structural elements are usually oriented in their stronger direction with the external layers running parallel to the line of action of the load. So for walls the outer laminates run vertically and for floors the outer layers run parallel to the span direction.

Influence of shear deformation

The stress distribution and the deformation behaviour of the panels loaded perpendicular to the plane both depend on the shear deformation due to the flexible bond between the individual layers. The rolling shear modulus for the transverse layers has been found to be:

$$\frac{G_{r,mean}}{G_{mean}} = 0{,}10$$

Bearing considerations

When the panels are cross grain loaded such as platform floors built into walls or soleplates positioned beneath wall panels (for example at ground floor level), the allowable compression perpendicular to the grain may govern the axial load capacity of the wall. This is because the characteristic compression stresses perpendicular to the grain for softwood is low compared with the compressive strength of the wall panels parallel to the line of action of the force. There are a number of possible solutions to overcome this, for example:

- reduce compression stresses by increasing the bearing area by using a thicker wall panel
- Reinforce the bearing area for example by using screws within the cross grain materials, see figure 3.22
- make use of balloon framing where floors are supported inside the walls

Where high point loads occur, for example where beams are supported on the wall panels, steel bearing plates may be required to reduce bearing stresses to acceptable limits.

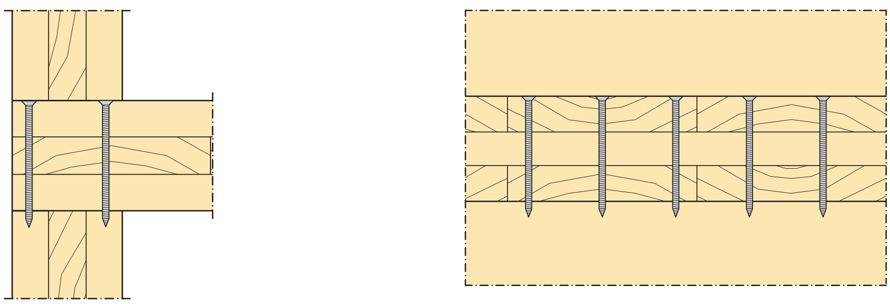


Figure 3.22 reinforcement of bearing area
Source: Trada GD10;
Redrawn by the author

3.4.2 Design method

The Eurocode does not specifically discuss the use of cross laminated timber. However, the calculation can be carry out as plate or slab, this can be achieved with different theoretical approaches.

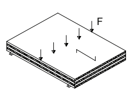
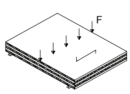
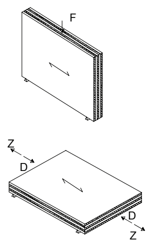
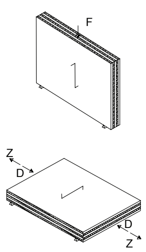
Design by the Composite Theory (after Blass)

This calculation method takes into account layers loaded parallel to the grain as cross layers loaded perpendicular to the grain. The strength and stiffness of the panel is determined using the gross section properties multiplied by a composition factor k that takes account of the different laminate build-ups of the panel.

The composition factor k is the ratio between the strength or stiffness of the CLT panel, taking into account joint slip, relative to a fictitious homogenous cross section of equal thickness with the grain direction of all layers parallel to the direction of the stress.

The composite theory does not take account of shear deformation in bending members and should therefore only be used for high span to depth ratios where the effects of shear deformation is small. L/D > 30 when loading is perpendicular to the plane and parallel to the grain of the outer layers and L/D > 20 when loading is perpendicular to the plane and perpendicular to the grain of the outer layers.

Table 7 composition factor k, taken from reference [2]

	k_i
	$k_1 = 1 - \left[1 - \frac{E_{90}}{E_0}\right] \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + ... \pm a_1^3}{a_m^3}$
	$k_2 = \frac{E_{90}}{E_0} + \left[1 - \frac{E_{90}}{E_0}\right] \cdot \frac{a_{m-2}^3 - a_{m-4}^3 + ... \pm a_1^3}{a_m^3}$
	$k_3 = 1 - \left[1 - \frac{E_{90}}{E_0}\right] \cdot \frac{a_{m-2} - a_{m-4} + ... \pm a_1}{a_m}$
	$k_4 = \frac{E_{90}}{E_0} + \left[1 - \frac{E_{90}}{E_0}\right] \cdot \frac{a_{m-2} - a_{m-4} + ... \pm a_1}{a_m}$

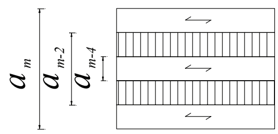


Figure 3.23 Build-up of panel

Table 8 effective values of strength and stiffness of CLT panels, taken from reference [2]

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	$f_{c,90,ef} = f_{c,0} \cdot k_4$	$E_{c,90,ef} = E_0 \cdot k_4$

When the panels are designed by the composite method, the strength and stiffness properties of the panels can be determined using the strength and stiffness values of GL28h according to reference [2]. This is possible due to the lamination effect which leads to considerable improvements. However, the GL28h values are still conservative when compared with test results.

Design by Theory of Mechanically Jointed Beams (Eurocode 5 Annex B)

The method given in Eurocode 5 for the calculation of effective bending stiffness takes into account shear deformation. Because of the shear deformation in cross layers, this calculation method also applies to small span to depth ratios.

Only the effective bending stiffness of the layers oriented parallel to the line of action of the loading are considered. The stiffness of the cross layers are disregarded. The shear deformation of the cross layers is taken into account using a reduction factor γ which reduces the effective moment of inertia of the longitudinal layers separated from the neutral axis by the cross layers, where:

$I_{ef} = \sum_{i=1}^{3}(I_i + \gamma_i * A_i * a_i^2)$ with $A_i = b_i * h_i$; $I_i = \frac{b_i * h_i^3}{12}$

$$\gamma_1 = \frac{1}{1+\frac{\pi^2 * E_0 * A_1 * \overline{h_1}}{G_r * b * l^2}} \qquad \gamma_2 = 1 \qquad \gamma_3 = \frac{1}{1+\frac{\pi^2 * E_0 * A_3 * \overline{h_2}}{G_r * b * l^2}}$$

$$a_1 = \left(h_1/2 + \overline{h_1} + h_2/2\right) - a_2$$

$$a_2 = \frac{\gamma_1 * A_1 * \left(\frac{h_1}{2} + \overline{h_1} + \frac{h_2}{2}\right) - \gamma_3 * A_3 * \left(\frac{h_2}{2} + \overline{h_2} + \frac{h_3}{2}\right)}{\sum_{i=1}^{3}(\gamma_i * A_i)}$$

$$a_3 = \left(h_2/2 + \overline{h_2} + h_3/2\right) + a_2$$

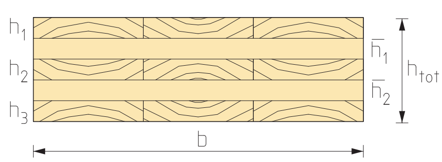


Figure 3.24 Build-up of panel

The bending stress is determined at the boundary of the boards. The bending stress in the middle of the boards may remain unconsidered.

$$\sigma_{m,i,d} = \pm \frac{M_d}{I_{ef}} (\gamma_i * a_i + h_i/2) \leq f_{m,d}$$

Shear design

The verification of the shear performance is done by determination of the shear stress in the decisive plane, see figure 3.25. Two conditions must be satisfied where one condition verifies the maximum shear stress $\tau_{v,d}$ in the centreline of the panel. The second condition verifies the rolling shear in the layers that are placed perpendicular to the span direction.

The failure conditions are calculated with the following two equations:

$$\frac{\tau_{v,d}}{f_{v,d}} \leq 1; \qquad \frac{\tau_{r,d}}{f_{r,d}} \leq 1$$

The shear tress can be calculated with the shear stress formula:

$$\tau = \frac{V_d * S\left(E_{(i)}, z_{(i)}\right)}{E * I_{ef} * b}$$ according to reference [25]

V_d = the design shear force;
S = the first moment of area depending on the Young's modulus E and the centre of gravity z of the layers;
I_{ef} = the effective moment of inertia;
b = the width of the panel.

The calculation of the shear stress in this way is a bit time consuming. The calculation can therefore be simplified by verifying the maximum shear stress with the maximum allowable rolling shear stress:

$$\tau_{v,d} = \frac{1{,}5 * V_d}{b * h} \leq f_{r,d}$$

This method is conservative but will not give problems since shear will in most cases not be governing in CLT design. See calculation example 1 in section 3.4.4 for the difference in shear calculation.

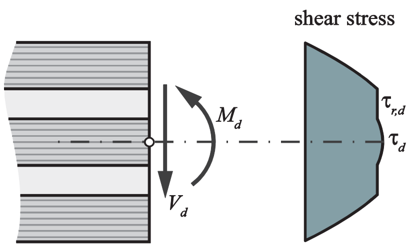


Figure 3.25 Shear stress in panel
Source: CLT designer [25]

Continuous beams and point loads

The effective moments of inertia I_{ef}, calculated using these methods are approximations and are calculated for mainly uniform distributed loads and simple spans.

In the case of high point loads and very short spans, a more precise calculation method is required, such as a Finite Element Method.

The calculation of the I_{ef} using the Eurocode method also depends on the span length. Therefore, for continuous beams the structural engineer should assess whether the approximation of I_{ef} can be applied.

Vibration

The design requirements in Eurocode 5 relate solely to residential floors having a fundamental frequency greater than 8 Hz. Floors with a fundamental frequency less than 8 Hz require special investigation, and are not covered in the code. Eurocode 5 requires that the fundamental frequency of the residential floor is greater than 8 Hz and for a rectangular floor with overall dimensions l x b, simply supported along four sides and with timber beams spanning in the l direction, the approximate value of the fundamental frequency f_1 can be calculated with equation:

$$f_1 = \frac{\pi}{2l^2}\sqrt{\frac{(EI)_L}{m}}$$

m = the mass per unit area in kg/m^2.
L = the floor span in metre.
$(EI)_L$ = the equivalent plate bending stiffness of the floor about an axis perpendicular to the beam direction in Nm^2/m.

The mass of the floor should be taken as the permanent actions only, without including partition loads or any variable actions.

The calculation of the floor deflection for the static unit point load deflection criteria of Eurocode 5 is not readily applicable to solid slabs because the Eurocode does not cover floors constructed with solid slabs. For preliminary design with one-way spanning floors, the deflection due to a 1 kN point load applied to a metre wide strip of floor could be considered.

The unit impulse velocity requirement of Eurocode 5 should also be checked against the limits given in the Dutch national annex.

Wall panel and column design

Wall panels may be designed using the Composite Theory to calculate the effective section properties for the effects of combined axial and bending stresses due to vertical (in plane) and horizontal (perpendicular to plane) loading in accordance with 6.2.3 and 6.2.4 of Eurocode 5.

Several predefined tables can be used from suppliers to calculate CLT wall panels. These tables (table 9 and 11) contain cross section values and modification factors for several panels. Caution has to be taken when using these kind of tables since the stiffness is calculated assuming full cooperation between the layers of the panel. This is not quite true since rolling shear in the perpendicular layers prevents full cooperation between the layers.

Table 10 is re-calculated using composition factors. The radius of gyration using composition factors are lower which will result in a lower modification factor k_c. The difference in calculation is shown in example 3 in section 3.4.4.

Calculation with full cooperation between layers:

$$\sigma_{c,0,d} = \frac{N_d}{A_{net}} \qquad i = \sqrt{\frac{I_{ef}}{A_{net}}}; \qquad \lambda = h/i$$

A_{net} = the net cross section of the layers running parallel with the height with the panel;
i = the radius of gyration;
I_{ef} = the effective moment of inertia of the panel, based on full cooperation between the layers of the panel;
h = the height of the panel.

Verification of failure condition:

$$\frac{\sigma_{c,0,d}}{k_c * f_{c,0,d}} \leq 1$$

Calculation with composite action:

$$\sigma_{c,0,d} = \frac{N_d}{k_3 * A_{gross}} \qquad i = \sqrt{\frac{I_{full}}{A_{gross}}}; \qquad \lambda = h/i$$

A_{gross} = the gross cross section of the panel;
I_{full} = the full moment of inertia of the panel;
k_3 = the composition factor.

Verification of failure condition:

$$\frac{\sigma_{c,0,d}}{k_c * k_3 * f_{c,0,d}} \leq 1$$

Table 9 cross section values

Panel	Net cross section A_{net}	Radius of gyration i
Type	cm^2	cm
51	340	1.77
61	340	2.25
71	540	2.33
81	540	2.81
85	540	2.82
93	660	3.15
95	610	2.98
99	660	3.43
105	710	3.48
115	810	3.68
125	710	4.33
135	810	4.48
147	930	4.89
153	990	4.99
165	990	5.47
174	1200	5.38
186	1320	5.62
189	1350	6.21
201	1470	6.57
207	1530	6.70
219	1650	7.06
231	1650	7.59
240	1860	7.45
252	1980	7.83
264	1980	8.30
273	1860	9.02
285	1980	9.29
297	1980	9.76

Source: Finnforest

Table 10 cross section values with composition factors

Composit factor	Eff. cross section A_{ef}	Radius of gyration i
k_3	cm^2	cm
0,68	347	1,47
0,57	348	1,76
0,77	547	2,05
0,68	551	2,34
0,61	519	2,45
0,72	670	2,68
0,65	618	2,74
0,68	673	2,86
0,69	725	3,03
0,71	817	3,32
0,58	725	3,61
0,61	824	3,90
0,64	941	4,24
0,66	1010	4,42
0,61	1007	4,76
0,70	1218	5,02
0,72	1339	5,37
0,72	1361	5,46
0,74	1487	5,80
0,75	1553	5,98
0,76	1664	6,32
0,72	1663	6,67
0,78	1872	6,93
0,79	1991	7,27
0,76	2006	7,62
0,69	1635	7,88
0,70	1995	8,23
0,68	2020	8,57

Source: Calculated by the author

Table 11 modification factors

Modification factors		
λ	ω	k_c
0	1.00	1.00
10	1.00	1.00
20	1.00	1.00
30	1.00	0.98
40	1.03	0.96
50	1.11	0.92
60	1.25	0.85
70	1.45	0.74
80	1.75	0.61
90	2.22	0.50
100	2.74	0.42
110	3.32	0.35
120	3.95	0.30
130	4.63	0.25
140	5.37	0.22
150	6.17	0.19
160	7.02	0.17
170	7.92	0.15
180	8.88	0.14
190	9.89	0.12
200	10.96	0.11

Source: Finnforest

Shear stresses in bending members may be calculated with the total thickness of the panel if gaps between or grooves within single laminations are not wider than 6 mm and the sum of the thicknesses of the cross layers is not smaller than 1/3 of the total thickness of the panel.

Shear stresses in shear members may be calculated with the total thickness if the laminations within the single layers are edge bonded. If these conditions are not fulfilled, the shear stresses can be calculated using the sum of the longitudinal layers only.

Where concentrated loads occur on wall panels, such as beneath a steel beam bearing, the wall has to be checked for local compression stress using a net area based on the effective section properties of the longitudinal laminates only. The width of the section resisting the local compression force should be taken as the width of the beam bearing.

Window and door lintel

Where an opening occurs in a panel, the resulting lintel properties can be calculated based on the dimensions of the laminates running parallel to the lintel (the transverse laminates). When the width of the supporting pier is at least as wide as the height of the lintel section, the lintel may be assumed to be fixed at its supports.

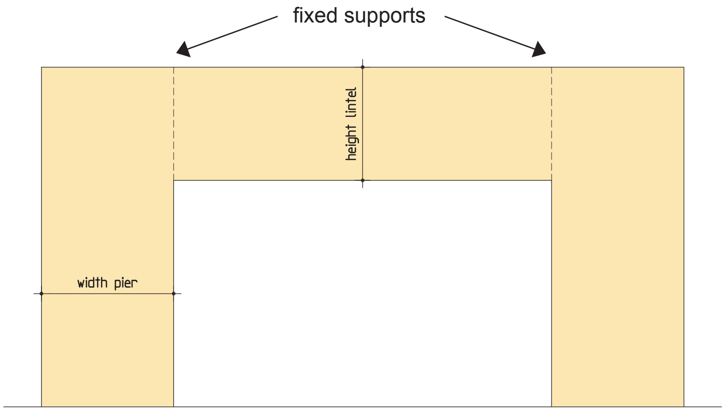


Figure 3.26 Wall panel with opening

Joints

- Joints in the plane side of the panel

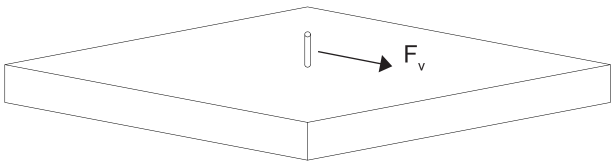


Figure 3.27 Dowel in plane side

For panels with single layer thickness t_{lay} > 9 mm, the embedding strength of solid timber should be used depending on the characteristic density of the laminations of the panels and on the angle between force and grain direction of the outer layer.

For fully threaded self tapping screws the characteristic embedding strength can be calculated with:

$f_{h,k} = 0{,}019 \rho_{lay,k}{}^{1{,}24} d^{-0{,}3}$ according to the draft, see reference [28].

d is the outer diameter of the fully threaded self tapping screw in mm and $\rho_{lay,k}$ is the characteristic density of the outer layer of the panel in kg/m^3. The embedding strength is independent of the angle α. This corresponds to the research results of Blass and Bejtka [1] for self tapping screws.

The characteristic embedding strength for dowels and bolts in the plane side of the panel can be calculated independently of the layer thickness and lay-up with:

$$f_{h,k} = \frac{32(1-0{,}015d)}{1{,}1sin^2\alpha+cos^2\alpha}$$ according to the draft, see reference [28].

Where d is the nominal diameter of the fastener in mm and α is the angle between load and grain direction of the outer layer.

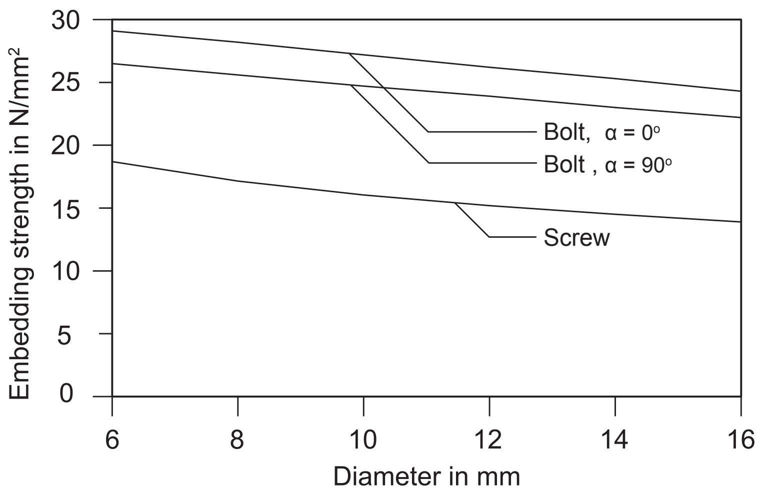
The embedding strength for dowels is derived with a minimum 5th percentile density of 400 kg/m^3. This density is determined by Blass [3] by analysing 2299 test specimens out of a range of products from four different manufacturers.

The graph shows that the embedding strength for bolts and dowels is much favourable than for screws. The angle between load and face grain direction makes the embedding strength of bolted connections decrease but is still significant higher than the embedding strength for screws. This makes bolted and doweled connections much stronger than screwed connections. However, screwed connections will in most cases be preferred over bolted connections due to the simplicity of the connection. Bolted and doweled connections will in most cases be used when large forces must be transferred.

The effective number of fasteners can be taken as:

$n_{ef} = n$ according to the draft, see reference [28].

The slip modulus k_{ser} should be calculated according to table 7.1 of Eurocode 5.



Graph 1 Embedding strength for bolted and screwed connections with various diameters

- Joints in the narrow side of the panel

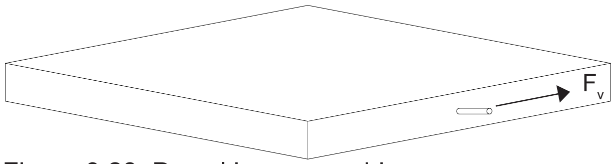


Figure 3.28 Dowel in narrow side

The characteristic embedding strength for nails and self tapping screws without predrilled holes in the narrow side of cross laminated timber can be calculated with:

$f_{h,k} = 20d^{-0,5}$ according to the draft, see reference [28].

Where d is the nominal diameter of the fully threaded self tapping screw or nail in mm.

The characteristic embedding strength for dowels and bolts in the narrow side of cross laminated timber can be calculated with:

$f_{h,k} = 9(1 - 0,017d)$ according to the draft, see reference [28].

Where d is the nominal diameter of the bolt or dowel.

The effective number of fasteners can be taken as for solid timber.

Other calculation related design rules like minimum spacings, edge and end distances can be found in the draft [28].

3.4.3 Design tables

The following design tables can be used as starting point for the calculation of floor and wall panels. The tables give an indication of the panel height that is needed for a given span and load. The chosen panel can then be verified.

Floor span

Table 12 is a floor span table which indicates the floor thickness based on a given span. The floor thickness is based on dead and live load from the Dutch code NEN 6702.

- the bolded layers in the build-up of the panels run parallel to the span direction
- fire safety and acoustics is not taken into consideration

Floor load assumptions:

Dead load:

Self weight is dependent of panel thickness

Roof:

- roof finish, 0,20 kN/m$^{2;}$

Floor:

- floor covering, 0,35 kN/m^2;
- light seperating walls, 0,50 kN/m^2;

Live loads:

Roof	1,00 kN/m^2
Dwelling	1,75 kN/m^2
Office	2,50 kN/m^2
School	4,00 kN/m^2
Store	5,00 kN/m^2

The floor panel for dwellings with a thickness of 147 mm and 4,85 metre span is verified in calculation examples 1 and 2 in section 3.4.4

Table 12 floor span table

Thick-ness	Layers	Build-up	Roof m	Dwelling m	Office m	School m	Store m
51	3	**17**-17- **17**	2,65	1,55	1,55	1,50	1,40
61	3	**17**-27- **17**	3,10	2,05	2,05	1,75	1,65
71	3	**27**-17- **27**	3,65	2,20	2,20	2,10	1,95
81	3	**27**-27- **27**	4,05	2,80	2,80	2,40	2,20
85	5	**17**-17- **17**-17- **17**	3,95	2,75	2,75	2,35	2,15
93	3	**33**-27- **33**	4,60	3,25	3,25	2,75	2,55
95	5	**17**-17- **27**-17- **17**	4,30	3,05	3,05	2,60	2,40
99	3	**33**-33- **33**	4,85	3,45	3,45	2,90	2,70
105	5	**27**-17- **17**-17- **27**	4,95	3,55	3,55	3,00	2,80
115	5	**27**-17- **27**-17- **27**	5,30	3,85	3,85	3,25	3,00
125	5	**27**-27- **17**-27- **27**	5,60	4,10	4,10	3,45	3,20
135	5	**27**-27- **27**-27- **27**	5,90	4,40	4,35	3,70	3,45
147	5	**33**-27- **27**-27- **33**	6,50	4,85	4,85	4,10	3,80
153	5	**33**-27- **33**-27- **33**	6,65	5,05	5,05	4,25	3,95
165	5	**33**-33- **33**-33- **33**	7,00	5,35	5,30	4,50	4,20
174	6	**33**-27- **27**-**27**-27- **33**	7,30	5,65	5,60	4,75	4,40
186	6	**33**-27- **33**-**33**-27- **33**	7,65	6,00	5,90	5,05	4,70
189	7	**27**-**27**-27- **27**-27- **27**-**27**	8,25	6,50	6,30	5,40	5,00
201	7	**27**-**33**-27- **27**-27- **33**-**27**	8,70	6,90	6,70	5,75	5,35
207	7	**27**-**33**-27- **33**-27- **33**-**27**	8,85	7,10	6,85	5,90	5,50
219	7	**33**-**33**-27- **33**-27- **33**-**33**	9,30	7,55	7,25	6,25	5,80
231	7	**33**-**33**-33- **33**-33- **33**-**33**	9,65	7,90	7,55	6,55	6,10
240	8	**27**-**33**-27- **33**-**33**-27- **33**-**27**	9,85	8,15	7,75	6,70	6,25
252	8	**33**-**33**-27- **33**-**33**-27- **33**-**33**	10,20	8,60	8,15	7,05	6,60
264	8	**33**-**33**-33- **33**-**33**-33- **33**-**33**	10,45	8,90	8,40	7,35	6,85
273	9	**33**-**33**-27- **27**-33- **27**-27- **33**-**33**	10,65	9,15	8,65	7,60	7,05
285	9	**33**-**33**-27- **33**-33- **33**-27- **33**-**33**	10,90	9,45	9,00	7,90	7,35
297	9	**33**-**33**-33- **33**-33- **33**-33- **33**-**33**	11,05	9,75	9,25	8,15	7,60

Source: Finnforest

Wall design

Table 13 is a design table for wall panels which indicates the wall thickness with a given load and length. The share of dead and live load cannot be estimated like for the floor panels, therefore the load on the wall panels is taken as the sum of dead and live load and is unfactored. The assumptions are:

- input loads are unfactored loads.
- fire safety and acoustics is not taken into consideration.
- Timber quality C24.

The wall panel with a thickness of 71 mm and 2,5 metre height is verified in calculation example 3 in section 3.4.4

Table 13 wall design table

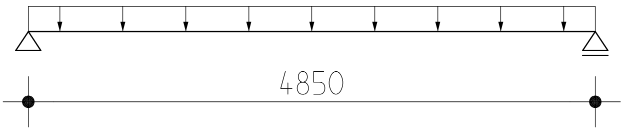
Load (kN/m)	Height in m		
D.L. + L.L	2,5	3	3,5
75	61	71	81
100	61	71	93
125	71	81	93
150	71	93	99
175	81	99	99

3.4.4 Calculation examples

Three calculation examples are made in order to demonstrate the use of the discussed design methods.

Example 1: Floor panel design by the composite theory

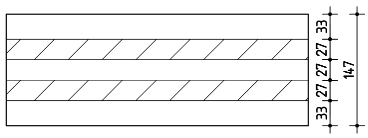
Floor panel length is 4850 mm, the thickness can be estimated with the span tables.
Thickness floor panel: 147 mm



$\frac{4850}{147} = 33 > 30$ so the compostite method may be used

Loads

Dead load:



Self weight = 0,62 kN/m²
Floor covering + seperating wall = 0,85 kN/m²

Total dead load: G = 1,47 kN/m²
Live load: Q = 1,75 kN/m²
Design load q_d = 4,39 kN/m

M_{max} = 12,91 kNm/m¹; V_{max} = 10,65 kN/m¹

CLT properties

CLT	147	C24*
Bending perp. to plane, parallel to grain $f_{m,k}$ =	28,0	N/mm²
Shear perp. to plane, parallel to grain $f_{v,k}$ =	2,5	N/mm²
MOE mean perp. to plane, parallel to grain $E_{0,mean}$ =	12000	N/mm²
MOE mean perp. to plane, perp. to grain $E_{90,mean}$	340	N/mm²
A_{gross} =	147000	mm²

* C24 with strength and stiffness values of GL28h according to Blass [2].

K_{mod} =	0,8
Y_m =	1,25
k_{def} =	0,6
Ψ_2 =	0,3

$$k_1 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) * \frac{81^3 - 27^3}{147^3} = 0{,}84$$

$$(EI)_{ef} = 12000 * \frac{1000 * 147^3}{12} * 0{,}84 = 2{,}67 * 10^{12}\ Nmm^2/m$$

Flexure check:

Verification of failure condition:

$$\sigma_{m,d} \leq f_{m,d} \qquad (6.11 - EC5_{1.1})$$

$$\sigma_{m,d} = \frac{12{,}91*10^6}{2{,}67*10^{12}} * 12000 * \frac{147}{2} = 4{,}26\ N/mm^2$$

$$f_{m,k,ef} = 28 * 0{,}84 = 23{,}52\ N/mm^2$$

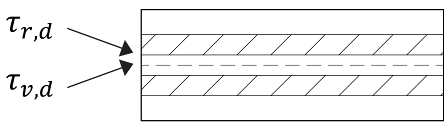
$$f_{m,d} = \frac{k_{mod}*f_{m,k}}{\gamma_m} = \frac{0{,}8*23{,}52}{1{,}25} = 15{,}05\ N/mm^2 \qquad 4{,}26 < 15{,}05 \ \text{ok}$$

Shear check:

Two situations must be verified for shear, the maximum shear stress in the centreline of the panel and the rolling shear stress in the layers that are placed perpendicular to the span direction.

Verification of failure condition:

$$\tau_d = \frac{V_d * S\left(E_{(i)} * z_{(i)}\right)}{E * I_{ef} * b} \leq \begin{matrix} f_{v,d} \\ f_{r,d} \end{matrix}$$



$$S_1 = 12000 * 33 * 1000 * 43{,}5 + 340 * 27 * 1000 * 27/2 = 1{,}73 * 10^{10}\ Nmm$$

$$S_2 = 12000 * 1000(33 * 57 + 27/2 * 27/4) + 340 * 27 * 1000 * 27 = 2{,}39 * 10^{10}\ Nmm$$

$$\tau_{r,d} = \frac{10{,}65 * 10^3 * 1{,}73 * 10^{10}}{2{,}67 * 10^{12} * 1000} = 0{,}07\ N/mm^2$$

$$\tau_{v,d} = \frac{10{,}65 * 10^3 * 2{,}39 * 10^{10}}{2{,}67 * 10^{12} * 1000} = 0{,}10\ N/mm^2$$

$$f_{r,d} = \frac{k_{mod}*f_{r,k}}{\gamma_m} = \frac{0{,}8 * 1{,}0}{1{,}25} = 0{,}64\ N/mm^2 \qquad 0{,}07 < 0{,}64 \ \text{ok}$$

Unity check: $\frac{0{,}07}{0{,}64} = 0{,}11$

$$f_{v,d} = \frac{k_{mod}*f_{v,k}}{\gamma_m} = \frac{0{,}8 * 2{,}5}{1{,}25} = 1{,}6\ N/mm^2 \qquad 0{,}10 < 1{,}6 \ \text{ok}$$

Unity check: $\frac{0{,}10}{1{,}6} = 0{,}06$

With the simplified method the verification becomes:

$\tau_{v,d} \leq f_{r,d}$

$\tau_{v,d} = \frac{1{,}5*10{,}65*10^3}{147*1000} = 0{,}11\ N/mm^2$ $\quad 0{,}11 < 0{,}64$ ok

Unity check: $\frac{0{,}11}{0{,}64} = 0{,}17$

When the unity checks of both methods are compared, it can be seen that the simplified method is more conservative. However, the stresses are quite low so shear will in most situations not be governing. The simplified method is therefore a better option to use.

Deflection check:

Verification of failure condition:

$u_{fin,tot} \leq 0{,}004 * l$ and $u_{bij} \leq 0{,}003 * l$ (10.2 and 10.3 - NEN 6702)

$u = \frac{5*q*l^4}{384*EI}$

$u_{inst,G} = 4{,}00\ mm$

$u_{inst,Q} = 4{,}72\ mm$

$u_{fin,G} = u_{inst,G} * (1 + k_{def})$ $\quad u_{fin,G} = 4{,}00 * (1 + 0{,}6) = 6{,}40\ mm$

$u_{fin,Q} = u_{inst,Q} * (1 + \psi_2 * k_{def})$ $\quad u_{fin,Q} = 4{,}72 * (1 + 0{,}3 * 0{,}6) = 5{,}57\ mm$

$u_{fin,tot} = 6{,}40 + 5{,}57 = 11{,}97\ mm$ $\quad \leq 0{,}004 * 4850 = 19{,}40\ mm$ ok

$u_{bij} = 11{,}97 - 4{,}00 = 7{,}97\ mm$ $\quad \leq 0{,}003 * 4850 = 14{,}55\ mm$ ok

Deflection is the governing design factor in this case.

Example 2: Floor panel design by the Theory of Mechanically Jointed Beams

The same floor span and loads as the previous example are considered.

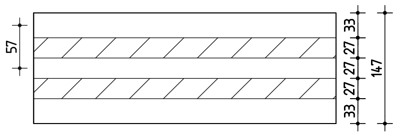
CLT properties:

CLT	147	C24
Bending perp. to plane, parallel to grain $f_{m,k}$ =	24,0	N/mm^2
Shear perp. to plane, parallel to grain $f_{v,k}$ =	2,5	N/mm^2
Rolling shear $f_{r,k}$ =	1,0	N/mm^2
MOE mean perp. to plane, parallel to grain $E_{0,mean}$ =	11000	N/mm^2
A_{gross} =	147000	mm^2

K_{mod} =	0,8
Y_m =	1,25
k_{def} =	0,6
Ψ_2 =	0,3

$$\gamma_1 = \gamma_3 = \frac{1}{1+\frac{\pi^2 * 11000 * 1000 * 33 * 27}{60 * 1000 * 4500^2}} = 0,93 \qquad \gamma_2 = 1$$

$$a_2 = 0$$



$$a_1 = a_3 = (33/2\ +\ 27 + 27/2) - 0 = 57\ mm$$

$$I_{ef} = 2071 * 10^5\ mm^4$$

Flexure check:

$$\sigma_{m,d} = \frac{12,91 * 10^6}{2071 * 10^5}(0,93 * 57 + 33/2) = 4,33\ N/mm^2$$

$$f_{m,d} = \frac{k_{mod} * f_{m,k}}{\gamma_m} = \frac{0,8 * 24}{1,25} = 15,24\ N/mm^2 \qquad 4,33 < 15,24 \ \text{ok}$$

Shear check:

Verification with the simplified method.

$$\tau_{v,d} \leq f_{r,d}$$

$$\tau_{v,d} = \frac{1,5 * 10,65 * 10^3}{147 * 1000} = 0,11\ N/mm^2 \qquad 0,11 < 0,64 \ \text{ok}$$

Deflection check:

$$u = \frac{5*q*l^4}{384*EI}$$

$$u_{inst,G} = 4{,}65\ mm$$

$$u_{inst,Q} = 5{,}53\ mm$$

$$u_{fin,G} = u_{inst,G} * (1 + k_{def}) \qquad u_{fin,G} = 4{,}65 * (1 + 0{,}6) = 7{,}44\ mm$$

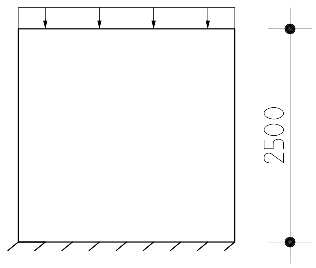
$$u_{fin,Q} = u_{inst,Q} * (1 + \psi_2 * k_{def}) \qquad u_{fin,Q} = 5{,}53 * (1 + 0{,}3 * 0{,}6) = 6{,}53\ mm$$

$$u_{fin,tot} = 7{,}44 + 6{,}53 = 13{,}97\ mm \qquad \leq 0{,}004 * 4850 = 19{,}4\ mm \qquad \text{ok}$$

$$u_{bij} = 13{,}97 - 4{,}65 = 9{,}32\ mm \qquad \leq 0{,}003 * 4850 = 14{,}55\ mm \qquad \text{ok}$$

Deflection is also the governing design factor in this case.

Example 3: Wall panel design



Wall panel height is 2500 mm, the thickness can be estimated with the span tables.

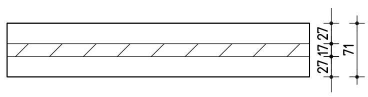
Thickness wall panel: 71 mm

CLT properties:

CLT	71	C24
Compression parallel to grain $f_{c,k}$ =	21,0	N/mm^2
A_{gross} =	71000	mm^2
A_{net} =	54000	mm^2
K_{mod} =	0,8	
Y_m =	1,25	

Calculation without composition factor:

Verification of failure condition:



$$\sigma_{c,d} \leq k_c * f_{c,d}$$

$$N_d = 150\ kN/m; \qquad I = 2\left(\frac{1}{12} * 1000 * 27^3 + 27 * 1000 * 22^2\right) = 2{,}94 * 10^7\ mm^4$$

$$N_d = 150\ kN/m; \qquad I = 2\left(\frac{1}{12} * 1000 * 27^3 + 27 * 1000 * 22^2\right) = 2{,}94 * 10^7\ mm^4$$

$$i = \sqrt{\frac{I}{A_{net}}} = \sqrt{\frac{2{,}94 * 10^7}{54000}} = 23{,}33; \quad \lambda = \frac{h}{i} = \frac{2500}{23{,}33} = 107 \ \Rightarrow k_c = 0{,}36$$

$$\sigma_{c,0,d} = \frac{150 * 10^3}{54000} = 2{,}78\ N/mm^2; \quad f_{c,0,d} = \frac{k_{mod} * f_{c,0,k}}{\gamma_m} = \frac{0{,}8 * 21}{1{,}25} = 13{,}44\ N/mm^2$$

$0{,}36 * 13{,}44 = 4{,}84\ N/mm^2$ $\qquad 2{,}78 < 4{,}84$ ok

Calculation with composition factor:

Strength and stiffness values of GL28 can be used according to Blass, reference [2].

$$k_3 = 1 - \left(1 - \frac{E_{90}}{E_0}\right) * \frac{17}{71} = 0{,}77$$

$$I_{full} = \frac{1}{12} * 1000 * 71^3 = 2{,}98 * 10^7\ mm^4$$

$$i = \sqrt{\frac{I_{full}}{A_{gross}}} = \sqrt{\frac{2{,}98 * 10^7}{71000}} = 20{,}5; \quad \lambda = \frac{h}{i} = \frac{2500}{20{,}5} = 122 \ \Rightarrow k_c = 0{,}29$$

$$\sigma_{c,0,d} = \frac{150 * 10^3}{0{,}77 * 71000} = 2{,}74\ N/mm^2;$$

$$f_{c,0,d,ef} = \frac{k_{mod} * k_3 * f_{c,0,k}}{\gamma_m} = \frac{0{,}8 * 0{,}77 * 26{,}5}{1{,}25} = 13{,}06\ N/mm^2$$

$0{,}29 * 13{,}06 = 3{,}79\ N/mm^2$ $\qquad 2{,}74 < 3{,}79$ ok

From calculation example 3 it can be seen that lower strength values are obtained when the composite method is used. The difference is not very large and the composite method is stated in the research of Blass [2] to be on the safe side. Calculation of wall panels is therefore conducted with composition factors.

3.5 Fire safety

Considering the behaviour of wood based materials and solid timber when subjected to fire, wood based materials will burn and are therefore rated as combustible. Solid timber is not readily ignited and there are few recorded cases where timber will have been the first material to be ignited. Solid timber will require surface temperatures well in excess of 400 °C if the material is to ignite in medium to short term without the pressure of a pilot flame. Even when a pilot flame is present the surface temperature will have to be in excess of 300 °C for significant time before ignition occurs.

Timber being combustible will spread fire across its surface, the phenomena of a number of ignitions each triggering an adjacent ignition. As timber is not readily ignitable the speed at which flames will spread across its surface is also reasonable for a combustible material. The rate at which timber releases heat is obviously very dependent upon the nature of the initial heating regime, the availability of oxygen and the density, shape and size and the place where the member is being located.

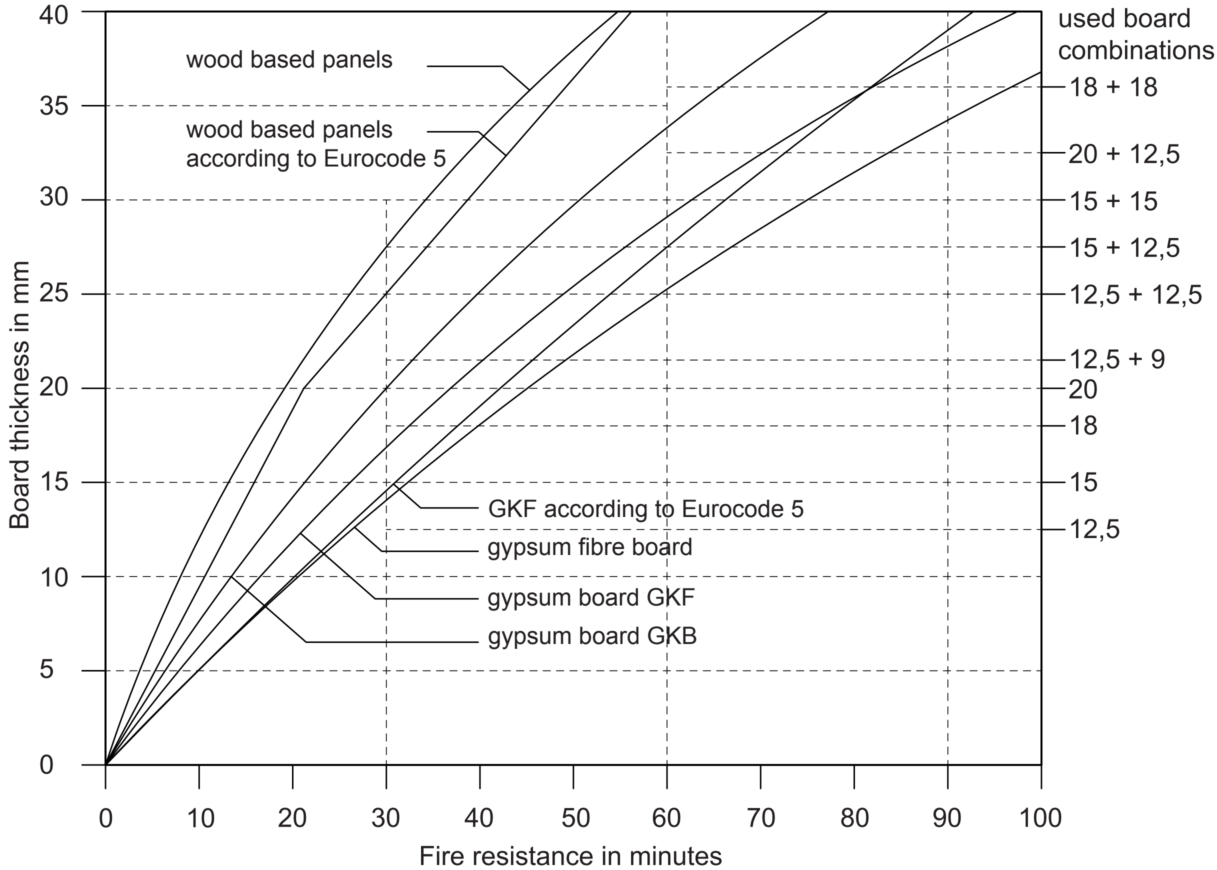
3.5.1 Resistance to fire

When timber or wood based materials are exposed to a fully developed fire they exhibit many desirable characteristics. Whilst the exposed surfaces will ignite when the heat flux becomes great enough and initially burn fairly vigorously it soon builds up a layer of insulating charcoal. As wood is a poor conductor of heat there is very little transmission of heat into the remaining un burnt material.

In the case of solid timber, the core section remains cool only a short distance behind the burning zone. As a consequence the temperature of the residual section is cool and the construction does not have to accommodate damaging thermal expansions. Also because the core remains cool, all of the cold state physical properties of the timber are retained and any loss of load bearing capacity is as a result of reduced cross-section, rather than a change in the physical properties.

Charring rate experiments conducted in Switzerland [16] found that the adhesive used in the manufacturing of CLT panels can have a significant impact on the charring rate. This was because the protective char layer that forms and insulates the unburned timber from fire, fell off in layers when some polyurethane adhesives were used. When CLT panels with more traditional adhesives were used, the charring rate was found to be the same as that assumed for solid timber and glulam members.

The fire resistance of CLT panels can be determined using the design method in Eurocode 5, with a charring rate comparable to softwood of 0,7 mm/min and with the use of fire resistant linings. Various materials can be used to achieve a required fire resistance, see graph 2.



Graph 2 Fire resistance of various materials
Source: Staalframebouw handboek

3.5.2 Reaction to fire

The reaction to fire of CLT panels is the same as the homogeneous timber panels where it is composed of.

CLT panels are classified in Euro class D with a smoke production class s2. Since no flame droplets occur when timber is on fire the flame droplet class is d0.
CLT floor panels are classified in D_{fl} and s1.

This classification means that timber and CLT panels cannot be used without additional measures in fire and smoke compartments and escape routes.

One solution is to treat the timber to make it more fire retardant. When done properly, the treated timber is classified in Euro class B. However, the problem with this method is that while the reaction to fire is greatly improved, the fire resistance of the timber is kept the same. Therefore, this method is not efficient when a fire safety of 120 minutes is required.

A more efficient method of improving the reaction to fire of timber is to cover it by gypsum fibre board which is classified in Euro class A2 and is very fire resistant. The gypsum fibre board must not be mistaken with ordinary gypsum board since this board has a rapid flame spread and is classified in Euro class D and can therefore not be used in fire and smoke compartments and escape routes.

3.5.3 Design method

The fire resistance of the panels can be designed using either reduced section properties (A), or in combination with fire resistant lining (B):

A. The reduced cross-section method:
Using the known charring rate, the section can be analyzed for a residual section following a fire in accordance with NEN 6069:2005 and Eurocode 5. Additional timber layers may be applied to the CLT section which are assumed to be sacrificial charring layers with no structural function during a fire.

An alternative method for calculating the fire resistance given in Eurocode 5 is the reduced properties method which uses reduced material properties for the 'hot' design, however the use of this method is currently precluded by the Dutch National Annex.

B. A fire resistant lining such as plasterboard can be applied in accordance with Eurocode 5

Calculation example 1:

The floor panel from calculation example 1 and 2 of chapter 4.3.4 is now checked for a fire resistance of 120 minutes with the reduced cross section method.

The design load for the fire design can be calculated with the accidental load combination $q_{d,fi} = G + \psi_2 * Q, \quad q_{d,fi} = (1{,}47 + 0{,}3 * 1{,}75) = 2{,}0\ kN/m$

The reduced cross section is calculated by reducing the initial cross section with the effective charring depth d_{ef}.

$$d_{ef} = d_{char,n} + k_0 * d_0 \qquad \text{(4.1 - EC5}_{1.2}\text{)}$$

$k_0 = 1;$ $d_0 = 7mm;$ $d_{char,n} = 120 * 0{,}7 = 84\ mm;$

$d_{ef} = 84 + 1 * 7 = 91\ mm;$ $d_{fi} = 147 - 91 = 56\ mm;$

Section modulus of the unburned section: $W = \frac{1}{6} * 1000 * 33^2 = 1{,}8 * 10^5\ N/mm^2$

$M_{max,fi} = 8{,}9\ kNm$
$V_{max,fi} = 7{,}9\ kN$

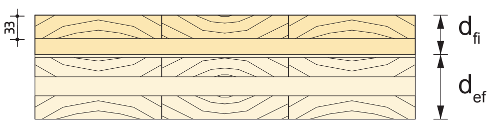


Figure 3.29 Reduced cross section

For a required fire safety over a time t, it must be verified that:

$$E_{d,fi} \leq R_{d,t,fi} \tag{2.7 - EC5$_{1.2}$}$$

Flexure check:

Verification of failure condition:

$$\sigma_{m,d,fi} < f_{m,d,fi}$$
$$\sigma_{m,d,fi} = \frac{8{,}9 * 10^6}{1{,}8 * 10^5} = 49{,}44\ N/mm^2$$

$$f_{m,d,fi} = k_{mod,fi} * \frac{f_{20}}{\gamma_{m,fi}} \tag{2.1 - EC5$_{1.2}$}$$

$$f_{20} = k_{fi} * f \tag{2.4 - EC5$_{1.2}$}$$

$k_{fi} = 1{,}15;$ $\qquad f = 24\ N/mm^2;$

$f_{20} = 1{,}15 * 24 = 27{,}60\ N/mm^2;$ $\qquad f_{m,d,fi} = 1 * \frac{27{,}60}{1} = 27{,}60\ N/mm^2$

$49{,}44 > 27{,}60$ *do not fulfill the requirement*

Shear check:

Verification of failure condition:

$$\tau_{v,d,fi} < f_{r,d,fi}$$

$$\tau_{d,fi} = \frac{1{,}5*7{,}9*10^3}{56000} = 0{,}21\ N/mm^2$$

Only the parallel layer is taken into account:

$f_{20} = 1{,}15 * 1{,}0 = 1{,}15\ N/mm^2;$ $\qquad f_{v,r,fi} = 1 * \frac{1{,}15}{1} = 1{,}15\ N/mm^2$

$0{,}21 < 1{,}15$ ok

Calculation example 2

In the previous example it was obvious that the cross section was too small to fulfill the failure conditions. The floor panel is therefore re-designed with the protection of 2 layers of gypsum fiber board.

Two values must be calculated in order to determine the charring depth of the panel: t_{ch} and t_a, where t_{ch} is the time of failure of the protected layer. After failure of the protected layer the charring rate of the timber is faster and will return normal after a time t_a.

$$t_{ch} = 2{,}8 * h_p - 14 \qquad \text{(3.11 - EC5}_{1.2}\text{)}$$

H_p is the thickness of the plasterboard. In this case with two layers of plasterboard, only 80 % of the first layer may be added to the thickness h_p.

$$t_{ch} = 2{,}8 * (15 + 0{,}8 * 15) - 14 = 61{,}6\ mm$$

$$t_a = min \begin{cases} 2t_f \\ \frac{25}{k_3 \beta_n} + t_f \end{cases} \qquad \text{(3.8 - EC5}_{1.2}\text{)}$$

$k_3 = 2;$ $\qquad \beta_n = 0{,}7$

$$t_a = \begin{cases} 2*61{,}6 = 132{,}2\ mm \\ \frac{25}{2*0{,}7} + 61{,}6 = 79{,}5\ mm \end{cases} \rightarrow \quad t_a = 79{,}5\ mm$$

The charring rate between $t_f \leq t \leq t_a$ must be multiplied with the factor $k_3 = 2$ $\beta = 2 * 0{,}7 = 1{,}4\ mm/min$, with this charring rate it is possible to calculate the charring depth at $t = t_a$. The maximum value for t_a is 25 mm; after $t = t_a$ the charring rate changes back to its normal value 0,7 mm/min.

The charring depth at $t = t_a = (79{,}5 - 61{,}6)1{,}4 = 25{,}06\ mm$

$d_{ef} = 53{,}4 + 1 * 7 = 60{,}4\ mm \qquad d_{fi} = 147 - 60{,}4 = 86{,}6\ mm$

Section modulus of the unburned section: $W = 7{,}8 * 10^5\ N/mm^2$

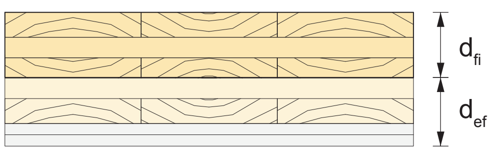


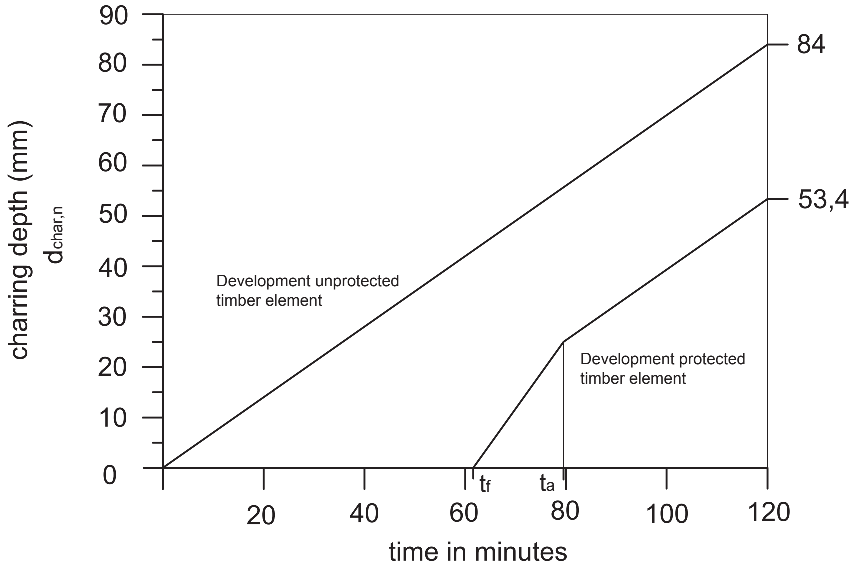
Figure 3.30 Reduced cross section

Flexure check:

$$\sigma_{m,d,fi} = \frac{8{,}9 * 10^6}{7{,}8 * 10^5} = 11{,}41\ N/mm^2$$

$11{,}41 < 27{,}60$ ok

The difference between the unprotected and protected CLT element can be seen in graph 3. Due to the increased charring rate after failure of the protected layer, the final charring depth of the timber results higher than one might expect. The reason for the increased charring rate after failure of the protected layer is that at that time, the fire temperature is already at a high level while no protective char layer exists to reduce the effect of the temperature.



Graph 3 Development of charring depth of protected and unprotected timber

Calculation of the fire resistance of wall panels is the same as for floor panels. The charring depth of the burned section is first calculated and next the verification of the wall panel with the reduced cross section.

3.6 Acoustics

To plan out an architectural design into different components is a complex process. A CLT structure must be designed in such a way that it does not conflict with its building physical principals. For example the connection of a stabilizing wall is not allowed to form an annoying sound leak. It is therefore important that the building physical performance of a CLT building is considered in an early stage.

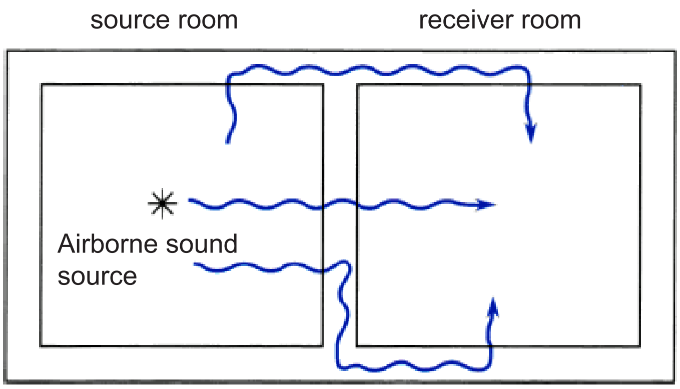
When considering sound transmission through walls, CLT panels are more similar to lightweight masonry construction (where the mass of the wall contributes to the acoustic performance) than timber frame wall panels (where the layers of plasterboard attached to the panels will provide much of the sound reduction between dwellings). The problem with this kind of light weight building structures is that there isn't an accurate calculation method for assessing the acoustic performance of the structure. New calculation methods are under development by TNO [6] but are not yet finished. Until then, manufacturers can be consulted for tests on various building configurations and details in order to gain insight in acoustic performances. Pre-completion testing during construction may also be needed to demonstrate compliance of the structure.

Attention has also to be paid with tested configurations from publications. These have due to several causes little meaning with the actual building practice. The most important one is the reduction of sound insulation of walls and floors due to flanking sound transmission. The flanking sound transmission can only be measured with complete joints or field study. Claims on basis of laboratory measurements have to be studied with caution. For conversion to practical values a reduction of 6 dB is accepted, although differences of 10 dB are possible.

3.6.1 Acoustic performance of CLT

The acoustic performance of CLT panels is rather poor. The fact remains that the panels posses to little mass to fulfil the acoustic requirements. Nevertheless, a much better sound insulation can be acquired with some few adjustments in design. The most wise thing to do is to consider the acoustic performance of a building in an early stage. Sound leaks must be detected and if possible interrupted with a flexible material. Especially the connection between wall and floor element needs to be interrupted with a flexible material to improve its acoustic performance. The flexible material will break the sound transmission and prevent open cracks between wall and floor elements due to shrinkage of the CLT panels.

CLT structures have also just like other timber structures problems with flanking sound transmission. Flanking sound transmission is sound that propagates through a structure and is transmitted in another room (Figure 3.31). Results of tests without taking any adjustments in sound transmission, have shown that 3% of the total sound transmission consist of direct sound transmission and 97% out of flanking sound transmission [26].



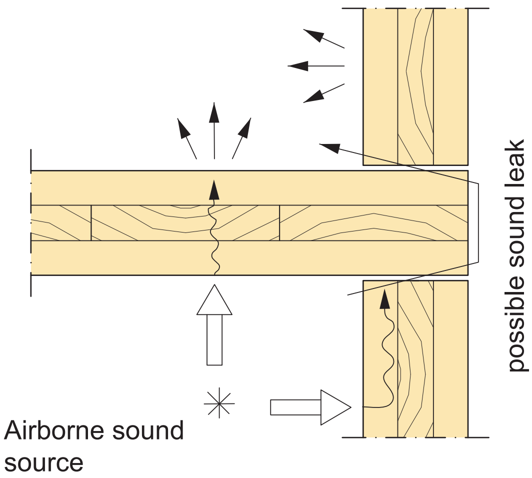


Figure 3.31 Flanking sound transmission
Source: Bouwfisica, van der Linden
Drawn by the author

Luckily there are various options for upgrading the acoustic performance of CLT elements and structures (tabel 15 and 16). Some of the options are briefly discussed.

Facing walls

Facing walls are used to provide a better sound insulation for CLT walls or to limit the transmission of impact sound. The walls can comprise boards on frames. The boards need to have a small bending stiffness to acquire a better sound insulation. These flexible boards are almost all used board materials with a thickness up to 20 mm, like plasterboard and gypsum fibreboard. The best results are achieved when the framing is kept free from the CLT panels, or when a flexible material is used to connect the frames with the CLT panels. Filling the cavity with mineral wool avoids the occurrence of annoying resonances.

Cavity walls

The use of cavity walls is a good option if high acoustic values are required ($D_{nT;A}$ > 55 dB). It is necessary that each leave is airtight and the cavity must be continuous to avoid flanking sound transmission, this means that floors are not allowed to be continuous. Again by filling the cavity with mineral wool the occurrence of annoying resonances can be avoided.

Floor coverings

There are several floor coverings available depending on the required sound insulation. The floor coverings are distinguished by wet and dry systems. The wet system consists of a cement or calcium sulphate bonded layer, while the dry system consists of a gypsum layer.

The wet system is used when a high insultion value is required. With this system there is also the possibility to integrate piping in the covering layer. If pipes are integrated with the covering layer, they should be kept free from the floor where they exit them to prevent sound leaks. The thickness of the covering layer depends on the used material. Calcium sulphate layers are stronger than cement layers, so they can be thinner. Calcium sulphate layers are usually 50 mm thick and cement layers 65 mm.

Floating floors

A floating Floor is a floor covering constructed over a flexible layer. The floor is kept free from the walls by means of a border strip to obtain the stated insulation level.

The flexible layer provides the acoustic separation between structural and covering floor. The acoustic performance of the flexible layer is governed by the material, thickness and stiffness of the enclosed air layer. Especially the flexibility of the flexible layer isgoverning the acoustic performance. The flexibility is expressed as the dynamic stiffness and is dependent on the choice and thickness of the flexible layer. Materials like special compressed mineral wool or other fibre materials can be used as flexible layer. When wet systems are used water proof barriers must be applied. This is important to prevent sound leaks due to penetration of cement water in the flexible layer.

The sound insulation of the floor can decrease over time, this is due to the ageing of the flexible material. The insulation reduction in practise however is no more than 3 to 4 dB if proper material is used and during construction no damage is done to the flexible material.

Suspended ceilings

A suspended ceiling is a secondary ceiling hung below the structural floor. The cavity is sometimes used for HVAC and very commonly used to conceal piping and wiring. The cavity is often filled with insulation material for a better sound insulation.

The following improvements can be expected for various suspended construction types:

- Porous boards (wood wool cement, mineral wool etc.) can provide improvements of 3 - 5 dB
- Ceilings with thick panels who are closely fitted in the suspension structure provide improvements of 3 - 8 dB
- Closed ceilings who are not, or flexible, connected to the floor and have sufficient mass ($m \geq 5$ a 10 kg/m^2) and reasonable cavity height ($\geq 0{,}2$ m) provide improvements of 10 - 15 dB

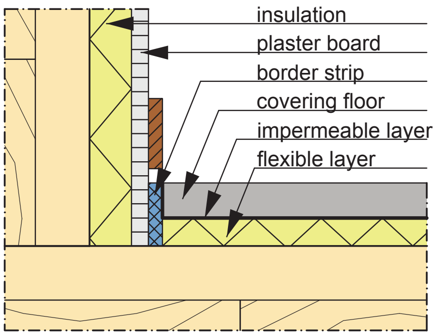
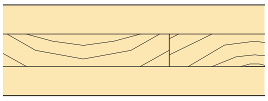
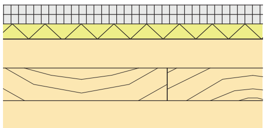
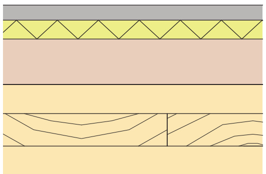
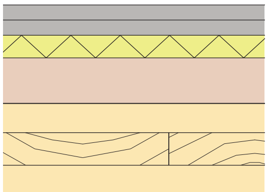
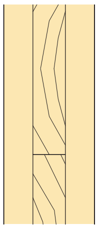
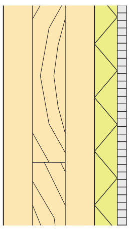
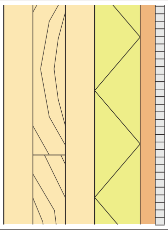
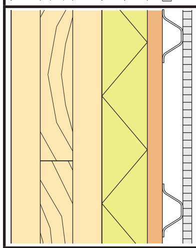


Figure 3.32 Floating floor
Drawn by the author

Tabel 15 Acoustic improvement of floor panel Source: Fermacell geluidgegevens voor Finnforest Leno

Detail	Description	Thickness [mm]	R_w / $I_{lu;lab}$ [dB]	$L_{n;w}$ / $I_{co;lab}$ [dB]
	basic CLT floor panel 135 mm	135	39 / -13	87 / -25
	Fermacell floor element (25 mm / 30 kg/m²); 20 mm mineral wool; floor panel 135 mm	180	52 / 0	61 / -3
	Fermacell powerpanel (20 mm / 49 kg/m²); 25 mm mineral wool; 60 mm fermacell honeycomb (90 kg/m²); floor panel 135 mm	240	64 / +12	50 / +8
	2 * Fermacell powerpanel (20 mm / 49 kg/m²); 30 mm mineral wool; 60 mm fermacell honeycomb (90 kg/m²); floor panel 135 mm	265	64 / +12	38 / +21

Tabel 16 Acoustic improvement of wall panel Source: Fermacell geluidgegevens voor Finnforest Leno

Detail	Description	Thickness [mm]	R_w [dB]	$I_{lu;lab}$ [dB]
	basic CLT wall panel 135 mm	135	39	-13
	wall panel 135 mm; 35 mm timber frame; 30 mm mineral wool; 12,5 mm Fermacell gypsum fiberboard	183	44	-8
	wall panel 135 mm; 80 mm timber frame; 60 mm mineral wool; 12,5 mm Fermacell gypsum fiberboard	228	47	-5
	wall panel 135 mm; 80 mm timber frame; 60 mm mineral wool; 27 mm horizontal spring frame; 12,5 mm Fermacell gypsum fiberboard	265	54	+2

3.7 Thermal performance and moisture protection

3.7.1 Thermal performance

The building decree sets minimum requirements for the thermal insulation. The basis of the requirements are the two codes: NEN 1068 Thermal insulation of buildings and NEN 5128 Energy performance of residential functions and residential buildings.

The minimum specific heat resistance R_C of facade, roof and floor for residential areas is R_C = 2,50 m² * K/W. The R_C values are often higher due to the energy performance coefficient EPC which indicates the energy efficiency of a building. The EPC value for dwellings is currently 0,8 and will be decreased in 2015 to 0,4 and again in 2020 to 0,0.

CLT has the same fundamental thermal properties as the timber from which it is made. In terms of heat capacity and thermal resistance timber is average among building materials. Values for CLT are improved simply by increasing its thickness.

Important properties are thermal conductivity λ (the rate of transferring heat) and the specific heat capacity c (the ability to retain heat). For example, CLT and lightweight concrete block materials have a similar thermal conductivity while CLT has a greater specific heat capacity. Therefore a 70 mm CLT panel has a thermal mass similar to a 100 mm lightweight concrete block.

The requirements in the building decree are easy to achieve with CLT panels because of its high heat capacity and low thermal conductivity. A CLT panel with a thickness of 100 mm and density of 500 kg/m³ has a thermal conductivity of λ = 0,13 W/mK. The specific heat resistance of the panel is in this case Rm = d/λ = 0,77 m²*K/W.

An external wall composed from 100 mm CLT, 12 mm gypsum board, cavity and masonry, needs only 60 mm of mineral wool to achieve the required R_C = 2,5 value. This stands quite in contrast with the same external wall composed with a 100 mm concrete wall, which has almost no contribution to the heat resistance of the external wall and needs a layer of 90 mm of insulation to fulfil the requirements. (λ = 2,0 W/mK, and Rm = d/λ = 0,05 m²*K/W)

See figure 3.33 for a comparison of the wall structure.

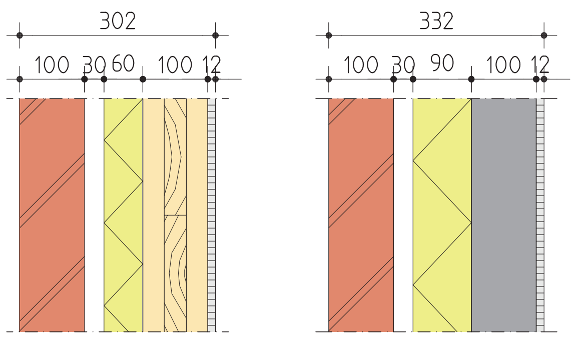


Figure 3.33 Comparison between a CLT and concrete facade with the same specific heat resistance
Source: Drawn by the author

Airtightness

The thermal performance will be comprised if the structure does not achieve adequate airtightness. Airtightness is the prevention of air flowing through the building structure. The flow of air from outside to inside, can lead to unpleasant drafts and it can influence the indoor climate and energy savings. When the structure does not include a vapour control layer, the system relies entirely on the detailing of joints to achieve airtightness. Joints that are screwed together may suffice but will depend on good workmanship. Airtightness of CLT structures is normally achieved with pre-compressed foam tape within the joint or breathable tape across the outside joint.

3.7.2 Moisture protection

CLT panels are highly resistant to short time moisture exposure. The layers of polyurethane glue act as vapour retarders and retain structural integrity upon incidental exposure to moisture.

Moisture vapour is also permitted to transport through the panels. This is permitted because of the vapour permeable nature of the timber which allows the transfer of molecular moisture without trapping it and creating conditions for mould growth. Because of this phenomenon, CLT buildings are vapour open and breathable structures where vapour control layers are not required.

Caution has to be taken when vapour barriers are used on the outside, for example with bituminous roofs. Vapour barriers on the inside are always required in these cases. The reason for this is quite simple. Because of the exterior vapour barrier, moisture is hindered to transfer to the outside. The moisture will condense in the structure and the panels will deteriorate as a consequence.

3.8 Longterm behavior

3.8.1 Durability

The durability of CLT panels will depend on the used timber for its manufacture and the level of exposure to the weather. In most cases, CLT is suitable for service classes 1 (heated internal) and 2 (unheated internal or covered external) in Eurocode 5.

Heartwood durability is: for spruce, not durable to slightly durable; for pine, slightly durable to moderately durable; and for larch, moderately durable. Sapwood portions are not durable against fungi or wood boring insects and since both sapwood and heartwood are present in CLT, the panels will be liable to decay if their moisture contents exceed 20% for an extended period of time. Therefore, it is important that these timbers are not exposed to continuous wetting by providing a drained and ventilated cavity behind the cladding. Structural timber must also be at least 150 mm above finished ground level to prevent splashing rain on the bottom side of the timber.

3.8.2 Longterm shortening

Longterm shortening is due to a combination of moisture content reduction, joint tightening and elastic load effects. The deformation of the panels is related to the direction of the grain. Because CLT panels have boards running in two directions, there will always be at least one layer in either direction which makes the panel better resistant against shrinkage than normal timber.

The maximum deformation in service (per % timber moisture change) is 0,02% within the plane of the panel and 0,24% perpendicular to the panel. (excluding the effects of elastic shortening and creep).

Whilst some movement within buildings is to be expected, it will greatly depend upon variations in the environmental conditions like moisture content and applied loads. These conditions and loads may change during the construction and lifetime of the building. It is important to gain a good estimation of the range of conditions and loads that parts of the building will experience and understand the propensity for movement of different materials and construction.

In a worked example of a 12 storey CLT building [30], the estimated long term frame shortening due to elastic shortening, creep and moisture content changes is approximately 37 mm. It has been shown by the platform timber frame industry that up to 40-50 mm frame shortening can be accommodated with correct detailing of vertical services, supported cladding and lining materials and so a limit of 15 storey's seems achievable for CLT structures using the platform method with correct detailing. A detailed explanation of how to deal with differential movement in timber buildings can be found in [9].

3.9 Conclusion

This chapter reviewed the properties of cross laminated panels as a load bearing element with an emphasis on the use in medium rise buildings in the Netherlands. The panels showed to have good characteristics and have high potential for being used in future medium rise projects when the following aspects are taken in mind.

The governing structural design factor will normally be the crushing of fibres in the lower floors clamped between walls due to loads perpendicular to the grain. A way to overcome this problem is by increasing the bearing area by using a thicker wall panel. Another alternative is the use of the balloon frame method where floors are supported inside the walls. The disadvantage of this method is the absence of a working platform which facilitates construction.

Another design aspect which is governing is the fire safety of the structure. The fire safety requirement for medium rise buildings in the Netherlands is 120 minutes. This will result in most cases in a double fire resistant lining and a section of the timber which is sacrificed as insulating charcoal layer.

The acoustic properties of CLT is also an aspect to take in mind because the panels lack mass to fulfil the acoustic requirements without additional measures. Fortunately, the acoustic performance of CLT can be improved considerably by disrupting flanking sound transmission and by using facing walls, floor coverings and suspended ceilings.

Another factor to take in mind is the long term behaviour of the CLT panels. Once a building is completed the timber panels start slowly to reduce their moisture content. Due to the reduction of the moisture content and elastic load effects, the building can shorten several centimetres. This shortening must be accommodated with the correct detailing of vertical services and lining materials. By using the correct detailing a height of 15 storeys is theoretical achievable for CLT using the platform method.

CHAPTER 4, CASE STUDIES

To gain a better understanding of the use of CLT panels in tall buildings, three case studies are made of existing medium rise timber buildings. The structure of the buildings is investigated and the building physical performances and requirements are reviewed.

The first building is Malmö Hus in Almere, the Netherlands. This five storey building is the highest residential timber building in the Netherlands. This building is interesting to study because despite its relative low height, the contractor encountered several structural problems during construction.

The second building is Limnologen in Växjö, Sweden. Limnologen is actually the name of four identical 8 storey high timber buildings. Limnologen is the first medium rise building in Sweden whose construction is based on timber and has been conducted and evaluated in close collaboration with leading experts in timber buildings.

The third building is Stadthaus in London, England. Stadthaus is currently the tallest timber residential building in the world and is therefore an excellent subject for investigation.

Figure 4.1 Malmö Hus
Source: www.architectenweb.nl

Figure4.2 Limnologen
Source:Midroc property development

Figure 4.3 Stadthaus
Source: Waugh Thistleton Architects

4.1 Case study 1, Malmö Hus

Figure 4.4 Malmö Hus
Source: Presentation Buro Tichelaar, symposium

Project information

Building type:	Medium rise apartment block, 5 storeys,
Completion date:	2008
Location:	Almere, The Netherlands
Architect:	Tigchelaar architecten
Structural engineer:	Bartels Ingenieurs voor voor Bouw en Infra
Timber engineer:	H.E. Lüning Adviesburo voor technische hout-constructies
Timber supplier:	Finnforest Holland
Timber elements:	CLT panels for internal walls, lift and stair cores; timber frame for external walls; Kerto ribbed floor panels
Total building time:	1,5 year
Cost:	± € 6 M

4.1.1 Introduction

Malmö Hus is a five storey residential block in Almere constructed entirely from timber from the first floor upwards which is unique in The Netherlands. CLT panels are used for internal walls and lift core while timber framed walls are used for the external walls. Kerto ribbed panels are used for the floor which have a free span of 7,5 metre.

During construction several problems were encountered which delayed the building time by half a year bringing the total building time to 1,5 year. Even with the delays, the building time was faster than an equivalent building made from reinforced concrete. This equivalent building was calculated to be completed in 2 years.

The idea for making a timber building in The Netherlands came from an excursion to Sweden of Tichelaar architects. After visiting the dwelling exposition ”the city of tomorrow” Tichelaar became fascinated of the Scandinavian timber building method and had the desire to design a similar building back home.

Malmö Hus houses 56 starter apartments from which four apartments are situated on the ground floor for disabled people. Each apartment has an individual storeroom situated at ground floor. Six small starter companies and eight ateliers are situated at the middle curve of the south side of the building. The lift and stairwell are situated on the east side and are entirely made of timber. Above the storerooms on the north side, a slender bridge connects the building which makes the building u-shaped.

4.1.2 Structure

The initial design of Malmö Hus was based on a complete timber structure. The local authority did not agreed with this design and required a stone like material for the ground floor to ensure the required fire safety. Because of this requirement, the first floor is made of reinforced concrete and the ground floor walls are made from sand lime bricks. The four upper storeys consists of timber.

CLT panels are used for internal walls and stabilizes the building (figure 4.5). To ensure a good acoustic separation between apartments, the party walls consists of two CLT panels with thickness of 115 mm and 60 mm cavity. The longitudinal stabilizing walls have a thickness of 81 mm and are continuous over two storeys. The Kerto floor panels are used because a massive timber floor resulted inefficient over a 7,5 metre span. The floor panels span from party wall to party wall and are connected with the stabilizing walls in order to transmit horizontal forces.

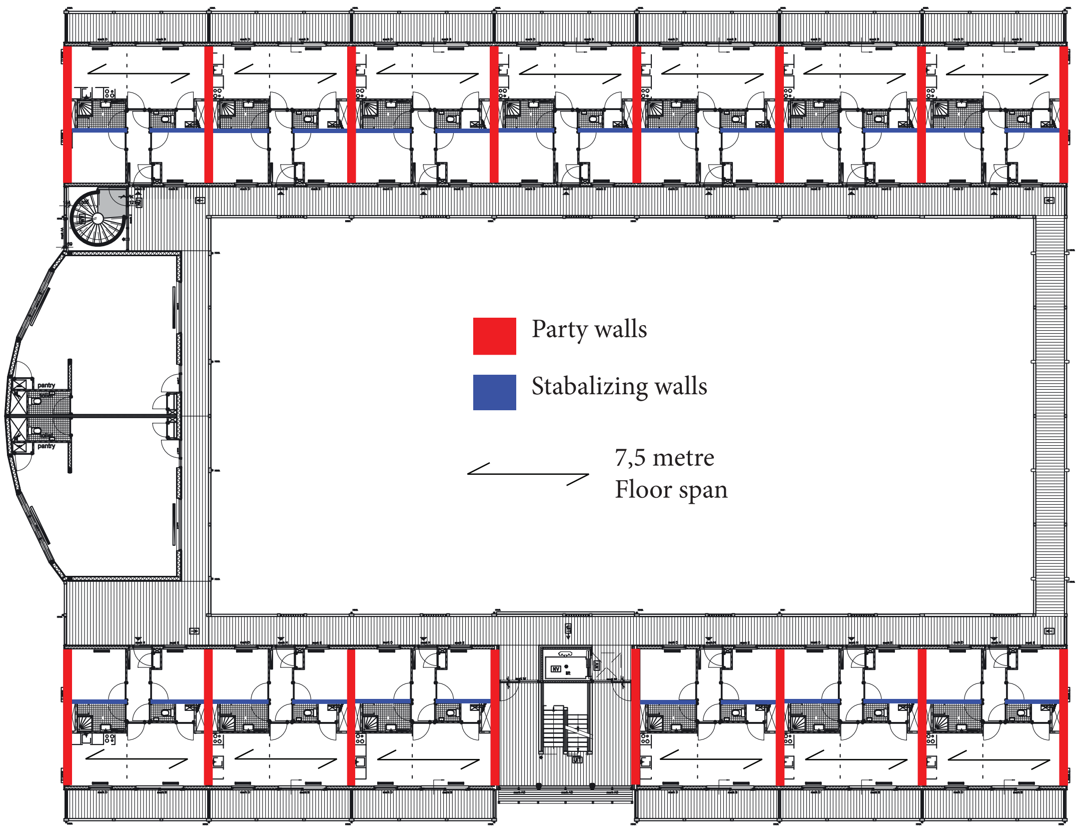


Figure 4.5 Floor plan
Source: Houtblad 2008 - 08
Edited by the author

The problems encountered during construction was the result of the rapid transition between design phase and construction. Construction already begun while the design was still in development. The choice for CLT panels was made in a late state and required special provisions. Details were re-drawn and the building sequence was changed. The specification phase, (bestekfase) was basically skipped and the local authority gave their additional fire safety requirements in a late state.

The most important problem however, was the fact that the stability of the building could not be acquired by the lift and stair cores. Stability is now acquired by 2 CLT panels (81 x 3.000 mm) over two storeys in every apartment. This is a good structural solution but influence the sound insulation in a negative way. Short construction time which is typical for timber construction was therefore delayed by a half year.

4.1.3 Performances

Acoustics

Several sound measurements were performed during construction which revealed the poor sound insulation of the structure. The impact sound insulation was particularly poor in the rooms where the stabilizing walls were situated (3 dB under the required 5 dB). Several adjustments to the floor and stabilizing wall were made in order to improve the acoustic insulation. An additional 25 mm layer of anhydrite is placed on the floating floor and an additional 12,5 mm gypsum board is placed to the ceiling.

Figure 4.6 Continuous stabilizing walls
Source: Houtwereld 2007 - 06

Figure 4.7 Joint sensitive to flanking sound
Source: Houtwereld 2007 - 06

Figure 4.8 Floors discontinuous walls
Source: Houtwereld 2007 - 06

Figure 4.9 Continuous stabilizing walls
Source: Houtwereld 2007 - 06

Two different adjustments were made regarding the stabilizing walls. In one wing of the building all linings were already placed on the stabilizing walls, so they added an additional 12,5 mm gypsum board. In the other wing of the building the lining placement was not advanced, so instead of the original 15 mm gypsum board a gypsum fibre board is placed on a frame with mineral wool (figure 4.10).

New measurements were performed after the adjustments. The measurements showed a great improvement of the characteristic sound insulation with a maximum of 16 dB for airborne and 6 dB for impact sound insulation. The airborne sound insulation fulfil the requirements very well, but the impact sound insulation is just one dB above the required 5 dB. It was therefore better to use thicker and heavier CLT panels for the stabilizing walls.

Fire safety

The building has a fire safety requirement of 90 minutes. This requirement is achieved by placing 15 mm gypsum boards. The idea was to leave some CLT panels in sight. If the CLT panels were to be in sight, they obviously had to be thicker in order to achieve a fire resistance of 90 minutes. The thicker panels meant considerably higher costs so the designers dropped the idea.

Long term effects

The initial plan was to use a stone like material for lift and stair cores, but in order to avoid differential movement between the timber and cores, the designers opted to use CLT for the cores as well. The final deformation per storey resulted in not more than 2 mm.

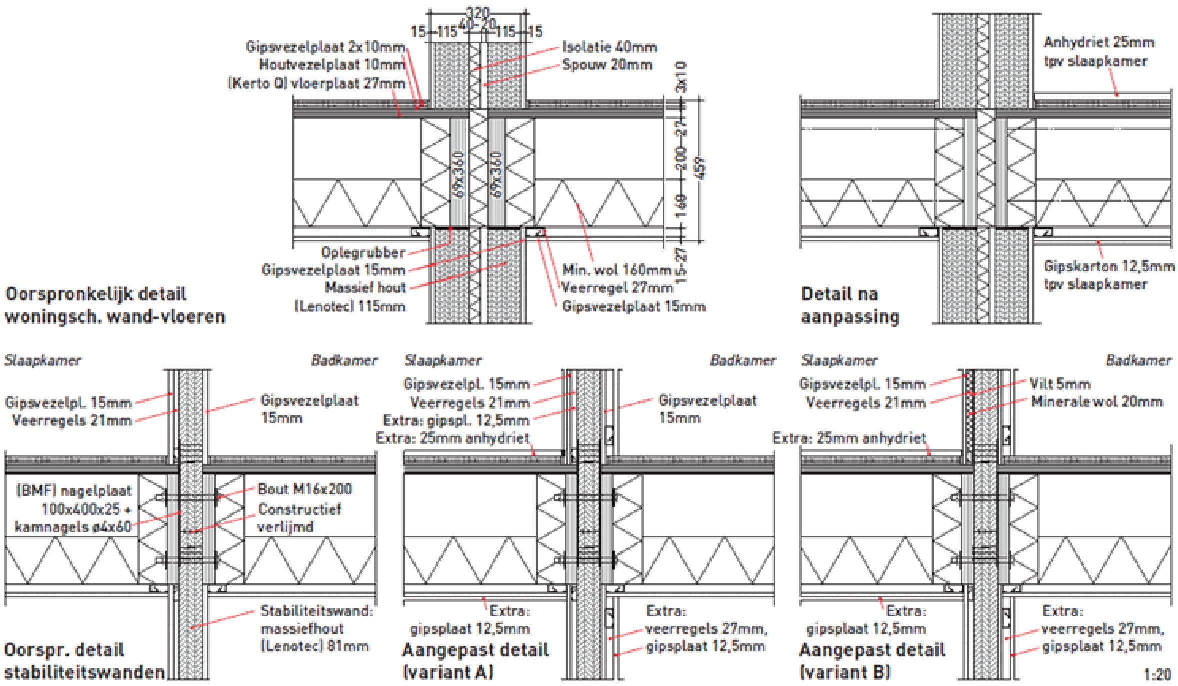


Figure 4.10 Acoustic improvements of the structure
Source: Bouwwereld, 2008 - 08

4.2 Case study 2, Limnologen

Figure 4.11 Limnologen
Source: Växjö University

Project information

Building type:	Medium rise apartment block, 8 storeys
Completion date:	2008
Location:	(4 buildings) Växjö, Sweden
Architect:	Ola Malm, ArkitektBolaget
Structural engineer:	Martinsons AB
Timber supplier:	Martinsons Byggsystem
Timber elements:	CLT panels for floor, roof, internal and external walls, lift and stair cores; timber frame for internal walls
Shell construction time:	4 days per floor
Total building time:	Built in two stages between 2006 and 2009; approximately 17 months per stage
Cost:	320 M SEK (€ 35 M); € 1800 – 2350 per m^2

4.2.1 Introduction

In mid 19th century large fires occurred in timber buildings in Sweden. Because of these fires the Swedish authorities decided to forbid construction of medium rise timber buildings. However in 1994, after a 120-year ban, the Swedish building legislation changed into a performance based format. This building code made it possible to use timber as the main load bearing material in medium rise buildings (> 2 storeys).

The four buildings are part of the project Valle Broar, which aims to support timber building technology and create the modern timber city in Växjö. Limnologen is the first medium rise building in Sweden whose construction is based on timber and has been conducted and evaluated in close collaboration with leading experts in timber buildings who are active at universities within Sweden and abroad.

The buildings have a combined floor area of 10700 m^2 spread over 134 apartments (33 or 34 per building). The development includes a parking deck and community facilities and is well situated at the shore of lake Trummen.

4.2.2 Structure

The main load bearing structure of the buildings are CLT elements which are used as walls and floors and traditional timber framed walls are used for apartment separation. All exterior walls and the apartment separating walls are load bearing elements. Glulam beams and columns are used to reduce deformations in critical locations. The ground floor is made of reinforced concrete. Concrete is used due to its high self-weight which facilitates the anchoring of the storeys above.

To prevent lift-up as a result of wind loading, 48 tension rods have been mounted in every building. These tension rods are anchored in the concrete of the first floor and extend all the way to the top floor. In this way forces are transferred between storeys and down to the foundation. During construction of the fifth floor, some of the tension rods were tensioned so that they could stabilize the structure. All the tension rods were tensioned when the whole structure was completed. The advantage of this method is that load transferring connectors between walls are not necessary. The disadvantage of this method is that the tension rods must be re-tightened after some time due to relaxation of the steel, creep deformations and shrinkage of the timber.

The floor system is developed by Martinsons and consists of a composite floor made of CLT panels and glulam beams. The CLT panel acts as the top plate and is glued and screwed to the glulam web elements. The glulam web elements consist of two glulam elements which are connected using adhesives. At delivery the floor elements includes parts of the installations (ventilation, water, electricity and sprinklers) and parts of the insulation and self supporting ceiling. The advantage of this floor system is the high acoustic performance and the integrated installations. However, the floor panels are very thick, approximately 500 mm.

Figure 4.12 Site plan
Source: Documentation of the Limnologen Project, by Eric Serrano

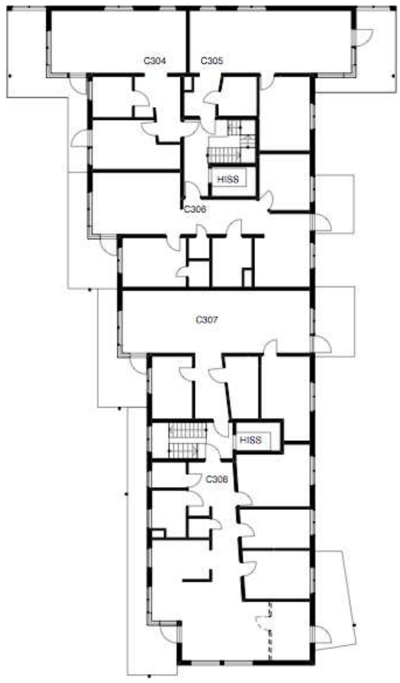


Figure 4.13 typical floor plan
Source: Documentation of the Limnologen Project, by Eric Serrano

In order to make the building process moisture proof, all the wall and floor elements were wrapped in plastic film for transportation. On site, the contractor used a pre-requisite weather protection system which was a large tent with an integrated overhead crane. The crane had a maximum capacity of 3,3 tonnes while the maximum weight of an element was 2 tonnes. The tent was made out of a synthetic sail supported by a slim and light weight steel structure which could move in longitudinal direction, see figure 4.14. Because of the tent, labour could continue during rain which would not have been possible without the tent because of the wetting of the timber panels.

Figure 4.14 Tent with overhead crane
Source: Växjö University

Figure 4.15 Placing of elements
Source: Växjö University

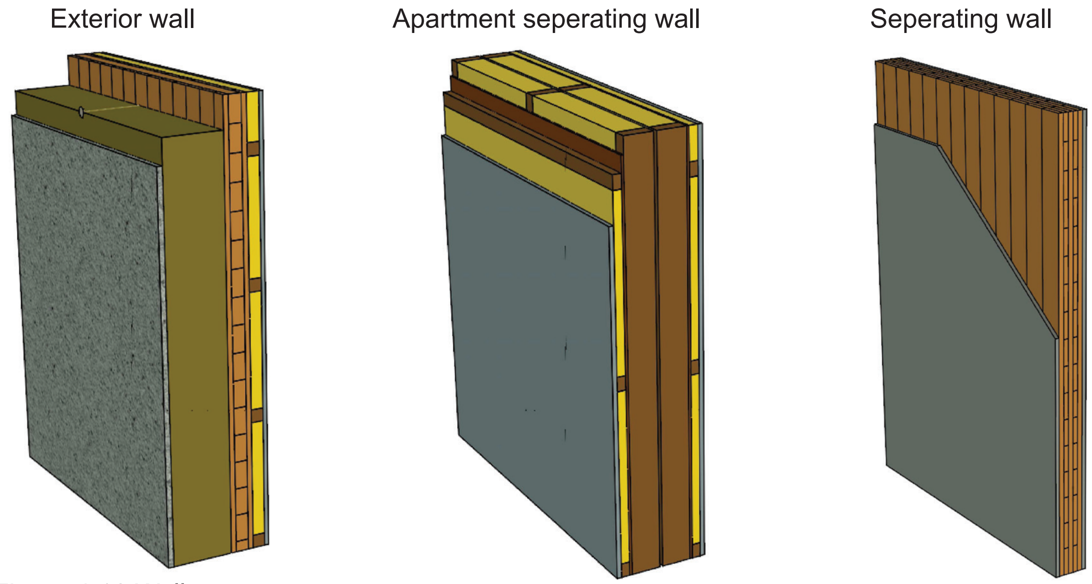


Figure 4.16 Wall systems
Source: Martinsons

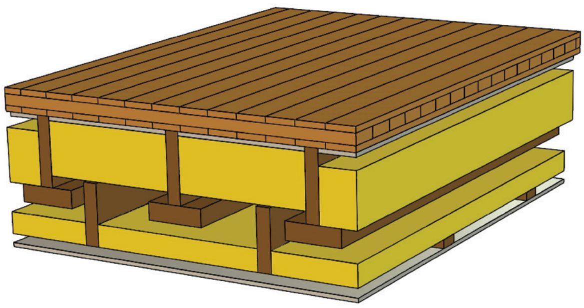


Figure 4.17 Floor system
Source: Martinsons

Figure 4.18 Stabilizing tie rod
Source: Linnaeus University

4.2.3 Performances

Acoustics

The property developer demanded in an early stage that at least sound insulation class B (''good acoustic environment'') should be achieved. The Swedish sound insulation class B requirements are 57 dB or higher for airborne sound R'_w + C50-3150 and 52 dB or less for impact sound level $L'_{n,w}$ + CI, 50-2500.

To fulfil these requirements the walls and floor slabs are discontinuous in order to reduce flanking sound transmission. A polyurethane sealant is used between walls and flange of the floor elements. The screw and washers used to connect the floor and wall panels are also fitted with polyurethane to reduce sound transmission.

Fire safety

The four buildings are equipped with residential sprinklers. This is not needed according to the Swedish legislation, but made designs possible that otherwise would not be possible. The South facade consists of timber, which is only allowed if the building is equipped with sprinklers. The vertical distance between the windows on the north-west facade has been minimised and the timber surface of the CLT panels of the balconies are visible from below. These designs were possible since it can be shown that the fire safety of the building is sufficient.

The Swedish requirements on fire safety are independent of the material used in the load bearing structure. The buildings have more than three storeys so they are classified in class BR1 (a fire safety class with the highest requirements). The buildings consists of fire compartments and are designed in class EI60 (integrity and insulation for 60 minutes).

The EI60 class can be applied if the fire load density is less than 200 MJ/m^2 or with a higher fire load density if the building is protected with automatic sprinklers.

Long term effects

Deformation measurement are performed at Limnologen building B (the first building erected). The measuring equipment was permanently mounted along a vertical line on the outside of the CLT panel. This equipment measures the vertical relative displacement storey by storey. The measurements started after all self weight were in place including gypsum boards, installations etc. The results obtained are in line with the expected long-term deformation. See figure 4.19, 4.20 and 4.21.

The graphs shows that the sharp increase in displacement during te first year is thought to be caused by the notable decrease in moisture content and other factors such as creep. The displacement after a year is reduced when moisture content increases (the building expands). The displacement is in relation with the moisture content and continiues to increase over time probably due to creep effects.

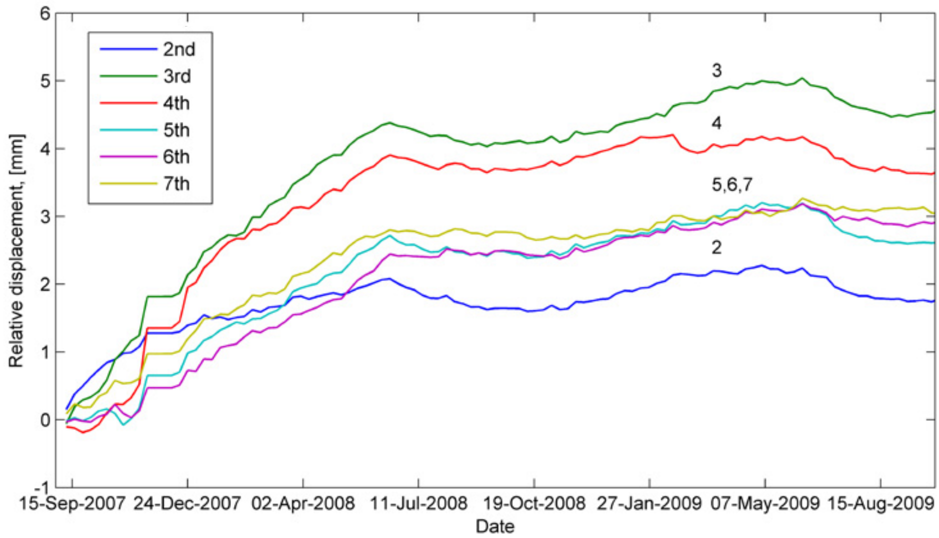


Figure 4.19 Deformations after 3 years
Source: Vertical relative displacements in a medium-rise CLT-building. [11]

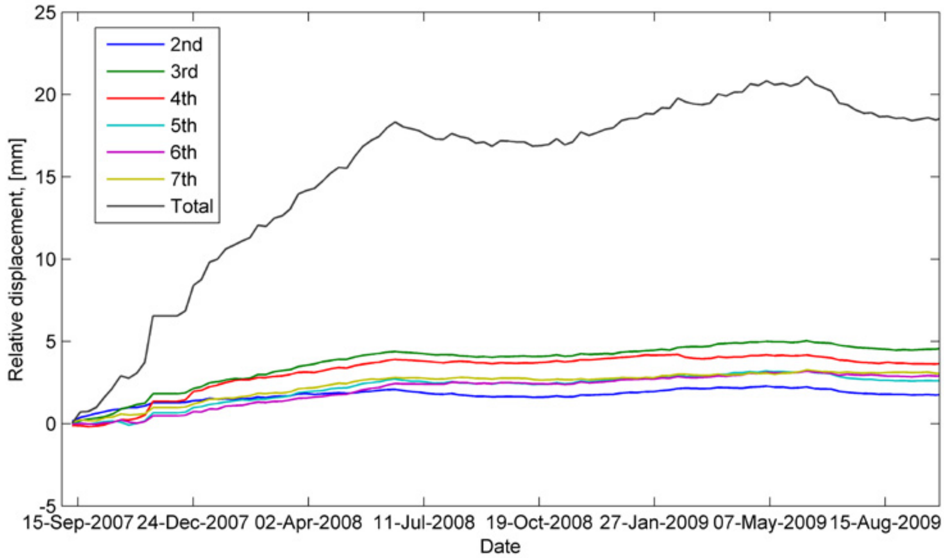


Figure 4.20 Deformations after 3 years
Source: Vertical relative displacements in a medium-rise CLT-building.

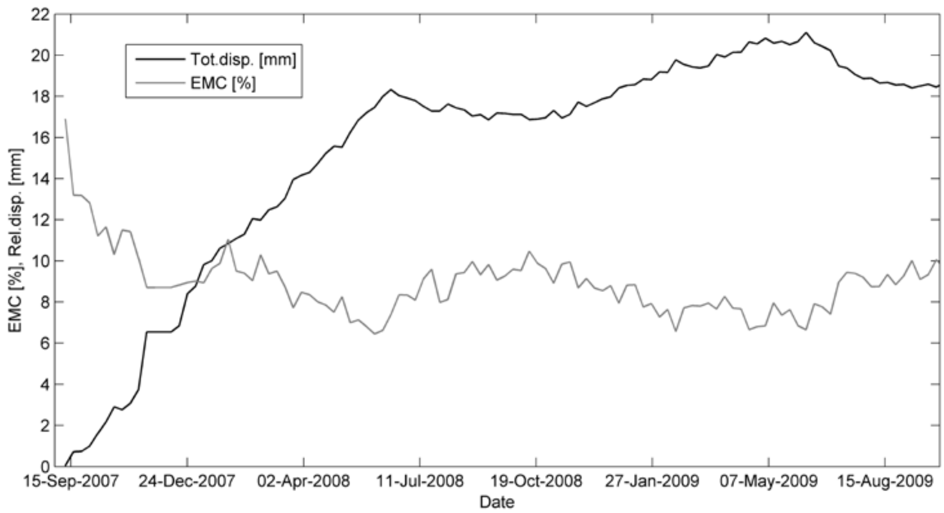


Figure 4.21 Deformations after 3 years
Source: Vertical relative displacements in a medium-rise CLT-building.

4.3 Stadthaus

Figure 4.22 Stadthaus
Source: Waugh Thistleton Architects

Project information

Building type:	Medium rise apartment block, 9 storeys,
Height:	30 metre
Completion date:	2008
Location:	London, England
Architect:	Waugh Thistleton Architects
Structural engineer:	Techniker
Timber supplier:	KLH UK
Timber elements:	CLT panels for floor, roof, internal and external walls, lift and stair cores
Shell construction time:	3 days per floor
Total building time:	49 weeks
Cost:	£ 3,8 M, € 4,4 M; € 1630 per m^2

4.3.1 Introduction

Stadhaus (German for townhouse) is a nine storey residential block in Hackney, East London, constructed entirely from CLT from the first floor upwards while the ground floor is made from reinforced concrete. It is designed by Waugh Thistleton Architects and was the tallest timber residential building in the world on its completion in 2008.

Stadhaus occupies a site of 17 x 17 metre and houses 29 apartments: 19 private sale units; 9 affordable tenancies and one shared ownership. The development includes a landscaped playground for children on the south side, which parents can overlook from half the apartments.

The interest in using CLT arose from an environmental position and a desire to make timber more readily accepted in the UK, especially for medium rise buildings which has only been feasible with inorganic materials like steel, concrete and masonry.

Stadhaus is divided into two sections that are independently owned, accessed and serviced (figure 4.23). The ground floor houses commercial floor space, two separate entrances, stores and plant provisions.

Above the ground floor sits three storeys of social housing units and five storey of private residential units. There are eleven one-bed, ten two-bed, five three-bed and three four-bedroom apartments. The fourth storey marks a change in floor layouts and external elevations.

The housing Association required a separate entrance for the private sale apartments so the architects mirrored the ground floor plan from one side of the building to the other, with an identical entrance on each side. This approach is consistent the whole way up, with two staircases and two lifts that are identical but with opposite orientations.

Site Plan

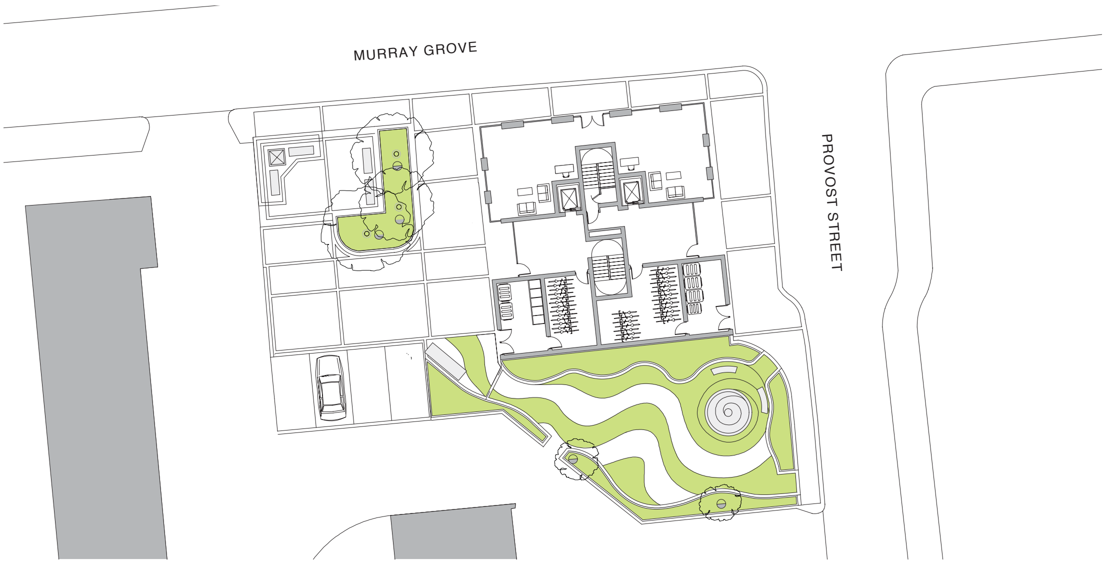
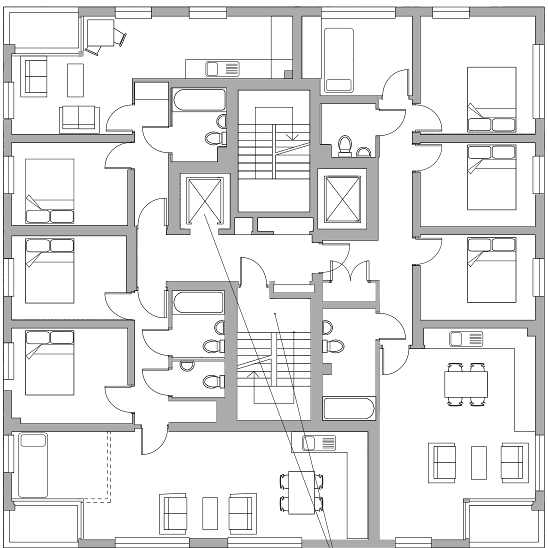


Figure 4.23 Site plan
Source: Waugh Thistleton Architects

Third floor plan (apartments for tenants)



Fifth floor plan (private owners)

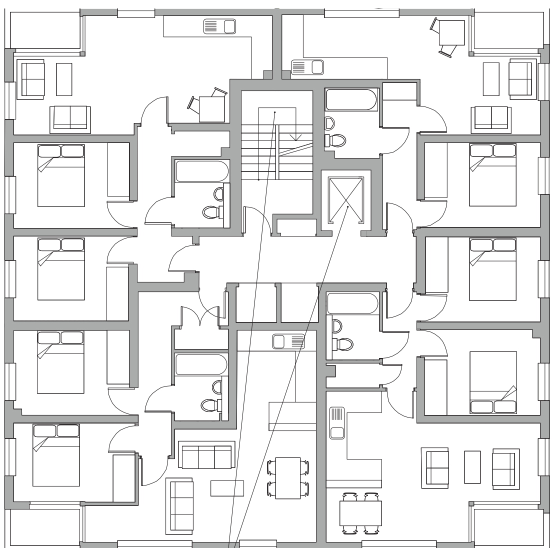


Figure 4.24 Floor plans
Source: Trada

lift and stairs for levels 1-3, terminate at 4th floor

lift and stairs for levels 4-8 run full height but don't have access to levels 1-3

A building of this form would normally be produced as a reinforced concrete frame, an energy intensive process producing upwards of 125 000 kg of carbon. In contrast Stadhaus stores 185 000 kg of carbon within the timber structure. To put these figures in perspective, the UK requirement (Merton rule) of ten per cent carbon reduction through on-site renewable energy would equate to the same saving only after 200 years of the building's constant use [27].

4.3.2 Structure

The building has a cellular structure with apartments in a honeycomb pattern around the central core (lift shafts and stairwells). All walls are load bearing and stabilizes the building except the central core which is kept free from the structure in order to improve the acoustic performance of the building.

The ground floor, first floor and ground floor walls consist of reinforced concrete. The structural engineers thought that reinforced concrete would accommodate better to the difference in layout between ground and first floor and that it would be easier to ensure good damp proofing with a concrete sub-structure.

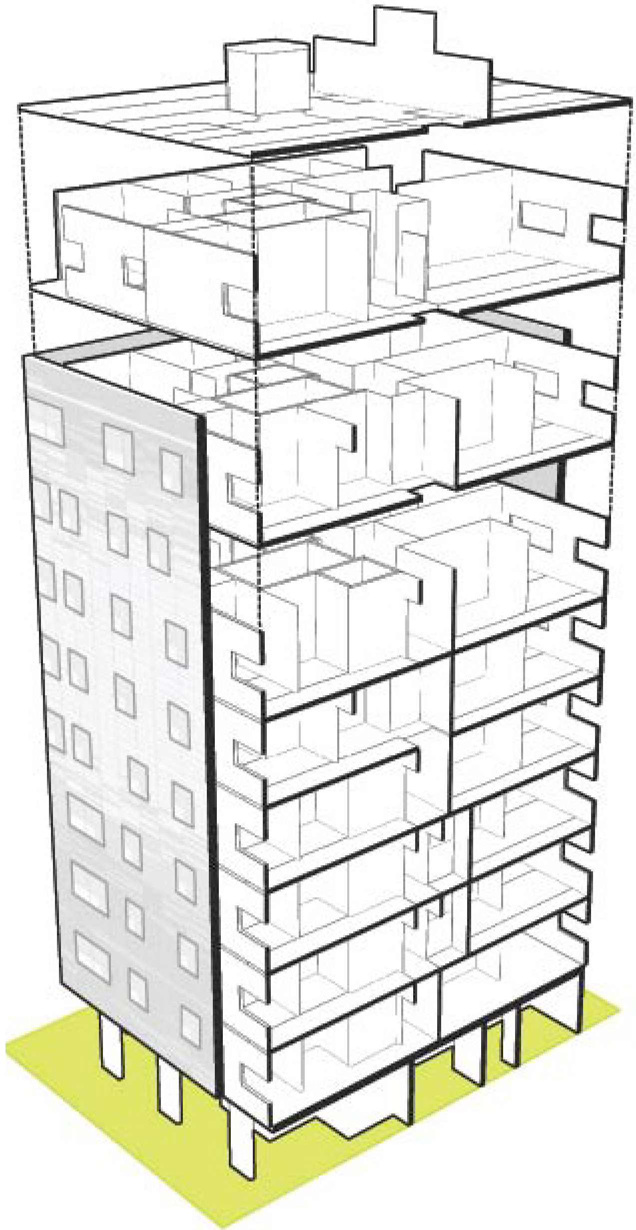


Figure 4.25 Isometric view of structure
Source: Trada

The structure of the first to ninth floor, consists entirely of CLT panels and is build using the platform frame method. All panels were prefabricated and included cut-outs for doors and windows. When the CLT panels arrived on site, they were immediately craned into position which reduced dramatically the time on site.

Progressive collapse is prevented by ensuring that primary floor panels are continuous over a minimum of two supporting walls (the floor panels are discontinuous between apartments and public spaces) see figure 4.26. These panels are than designed either to span twice the distance as normal service or to cantilever when supports are removed. In certain places in the building the internal organisation made these arrangements impossible to achieve, so alternative secondary load paths are designed. The primary load path is generally direct from floor down to the walls. When accidental load paths are mobilised these lead through external walls acting as deep beams spanning over damaged areas.

Compressive stresses are generally low throughout the structure. However the platform frame method with wall panels building up above one another, combined with the changing floor layout half-way the building, means that in certain places loads are sufficient to crush the side grains of the CLT floor panels. In such locations the floor panels are reinforced with arrays of screws to transfer the forces.

Although more expensive than an equivalent reinforced concrete frame, CLT brought significant overall savings. For example, an equivalent concrete building was estimated to take 72 weeks, whereas the CLT building required only 49 weeks. The erectors brought a large mobile crane, which eliminated the need for a tower crane that would normally be needed for a concrete structure. Scaffolding was needed to fix the cladding, but not to erect the CLT structure. The four-man Austrian crew was on site three days a week and accomplished the entire structure erection in 27 working days, over nine weeks.

4.3.3 Performances

Acoustics

The designers did a lot of research to fulfil the acoustic requirements. Eventually for party walls they used 2 layers of 9 millimetre thick plasterboard on each side. The floor panels consist of a compressed insulation layer, 55 mm screed, where under-floor heating is installed and a suspended ceiling.

The layers, moving independently of the floor gave acoustic separation that proved to exceed the UK requirements with an average of 55 dB protection between flats and 53 dB between floors. (The UK requirements for airborne sound insulation ($D_{nT,w}$ + C_{tr}) for walls, floors and stairs is 45 dB, and impact sound insulation (L'_{nTw}) for floors and stairs is 62 dB).

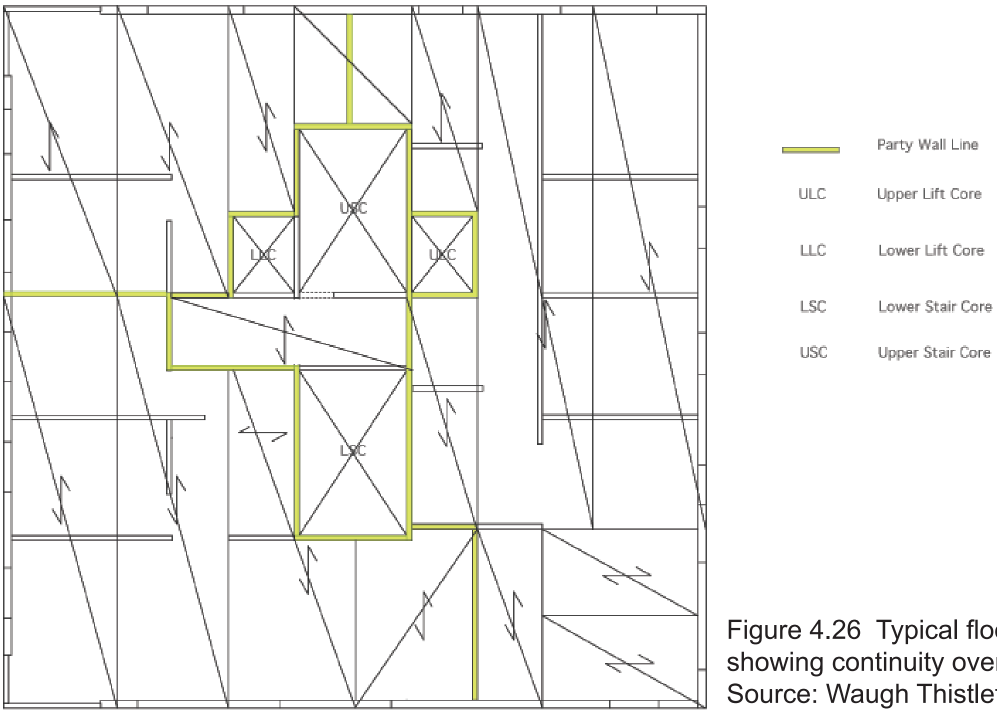


Figure 4.26 Typical floor panel layout showing continuity over internal walls
Source: Waugh Thistleton Architects

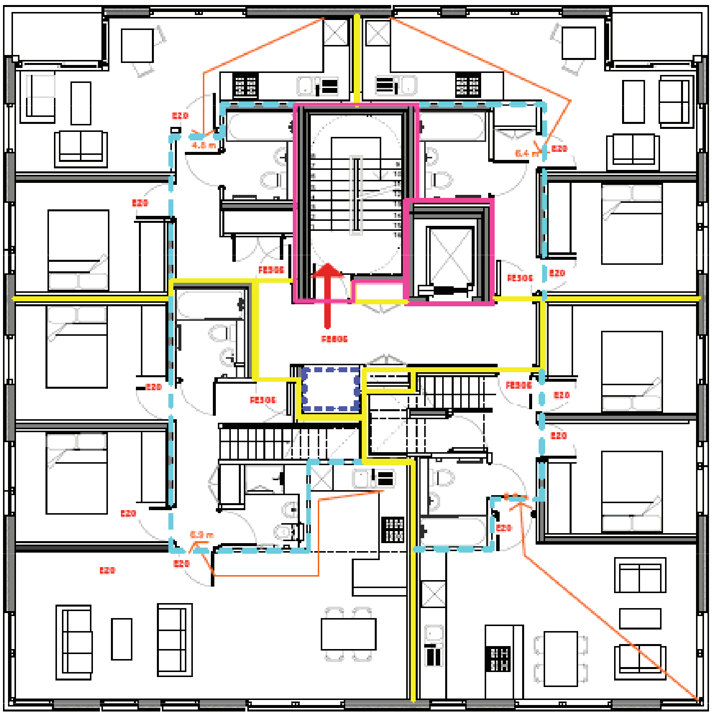
Fire safety

Fulfilling the UK requirements for fire safety was relatively straightforward. The CLT panels are designed to resist fire by calculating charring rates. The panels are designed for a resistance of 30 minutes and sized to retain their structural integrity with this loss of section. Combined with two layers of plasterboard the panels achieve a fire resistance of 90 minutes which fulfils the UK requirements.

Long term effects

Building tall timber buildings exaggerates the consequences of creep and moisture movement. In Stadthaus these effects are reduced by mobilising all the available walls to carry vertical loads in an even distribution. Compressive stresses are limited to 50 percent of their ultimate limit state to reduce long term deflections. The designers made a preliminary design model of long term effects which predicted a storey shortening of 3 mm and an eventual shortening of 25 mm of the building.

The cladding, internal finishes, vertical circulation components, handrails and trims are detailed to allow for shortening, expansions and contractions between floors. Also by avoiding concrete cores there was no need to resolve the differential movement between the concrete and timber.



- 30 mins (– – –)
  - internal corridors/ living space
- 60 mins (_____)
  - apartment / apartment
  - public corridors / apartment
- 120 mins(_____)
  - apartment / lift & stair cores

All bearing walls / floor systems - 90 mins

Figure 4.27 Fire safety design
Source: Presentation Canadian wood council: Reaching high(er) with CLT

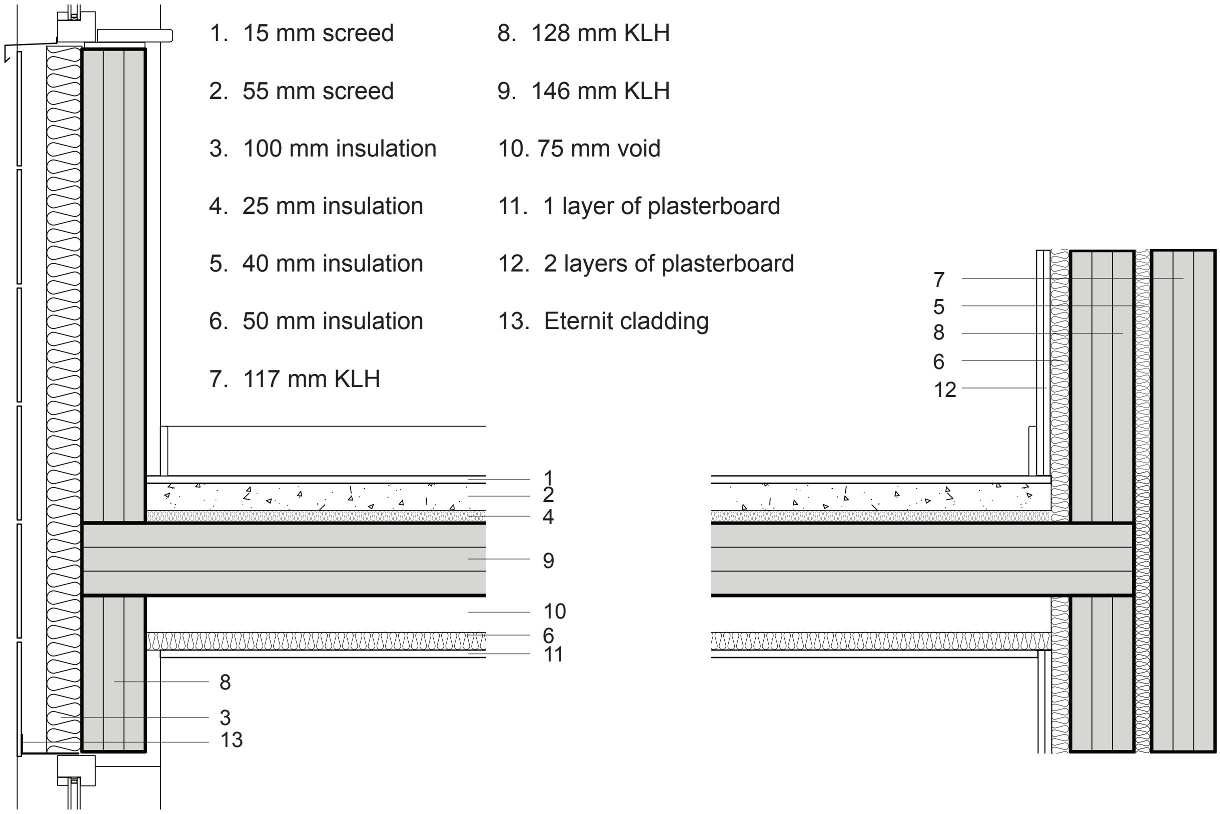


Figure 4.28 Left: Section at external wall; Right: section at lift shaft
Source: Trada

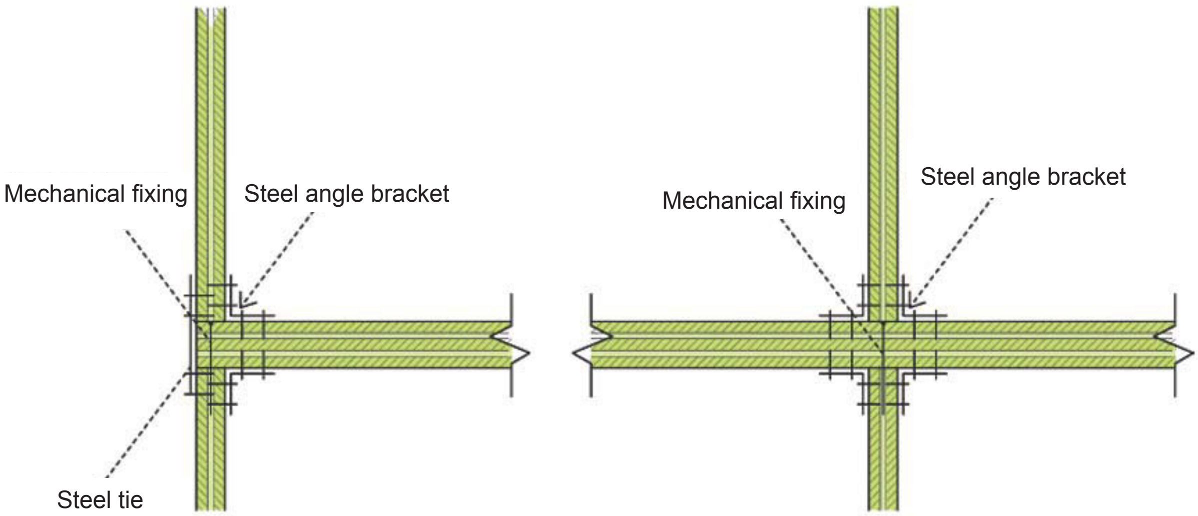


Figure 4.29 Typical external and internal wall to floor connection details
Source: Waugh Thistleton Architects

Figure 4.30 Wall arrangement
Source: Trada

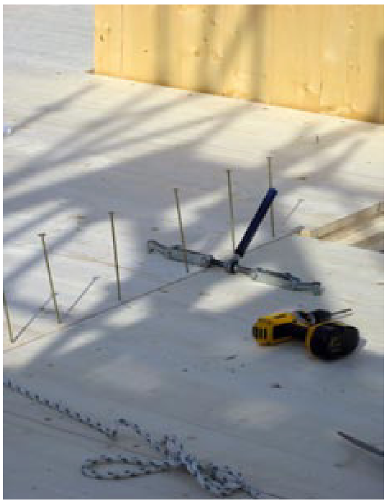


Figure 4.31 Self-drill woodscrews installed using light weigt tools
Source: Trada

Figure 4.32 Floor installation
Source: Trada

Figure 4.33 Floor installation
Source: Trada

Figure 4.34 Internal view
Source: Trada

Figure 4.35 Easy fixing of services to ceiling
Source: Trada

4.4 Conclusion

This chapter reviewed three medium rise timber buildings of three different countries. The characteristics of the buildings are: rapid construction time, lightweight structure and environmentally friendly. The characteristics and performances are summarised in table 17. The table shows that all three buildings are constructed using the platform approach. The presence of a working platform during construction and the interruption of flanking sound is decisive for the use of this construction method. The downside of this construction method becomes evident in the case of Stadthaus where locally some floor panels had to be reinforced by screws due to exceeding of the maximum compression perpendicular to the grain.

All three buildings are designed to have a fire resistance of 90 minutes or less. For Dutch medium rise design a fire resistance of 120 minutes is required. This means that the solutions in the case studies will not suffice for the Dutch situation unless the fire resistance is lowered by means of automatic sprinklers. Therefore, medium rise timber buildings in the Netherlands must probably be designed with additional fire protection in comparison with the presented solutions in the case studies.

Comparing the acoustic performance of the three buildings is difficult due to the different testing methods of the sound insulation in the three countries. The case studies show that the sound insulation of timber buildings will not result problematic when enough care is taken with the acoustic performances. When wall and floor panels are designed to have facing walls and floor coverings, the acoustic performance of the building will most likely fulfil the requirements.

The long term shortening of the buildings is despite their height well below the practical maximum of 50 mm stated in section 3.8.2. All three buildings are constructed without concrete core in order to evade problems with differential movement between timber and concrete. This solution is easy and very practical and will therefore be adopted in the design in chapter 5.

Tabel 17 Characteristics and performances

Building	Number of storeys	Structure	Fire safety	Acoustics	Long term shortening
Malmö Hus	5	platform method	90 minutes	$I_{lu;k}$ = 16 dB I_{co} = 6 dB	8 mm
Limnologen	8	platform method	60 minutes	-	21 mm
Stadthaus	9	platform method	90 minutes	$D_{nT,w}$ = 55 dB	25 mm

CHAPTER 5, DESIGN OF A MEDIUM RISE TIMBER STRUCTURE

5.1 Introduction

The requirements set on buildings, for instance structural or fire safety requirements, vary by country and are in most cases decisive for the use of a certain building material. It is therefore difficult to say that the same building in for instance Sweden, can also be build in The Netherlands. So the use of timber in medium rise buildings in The Netherlands is only possible if it can be demonstrated that the buildings satisfies the Dutch building requirements.

In order to demonstrate the compliance of a timber structure in a medium rise building in The Netherlands, an existing building is chosen who serves as a model for assessing the timber structure. The building that serves as a model for a timber structure is the Inntel hotel in Zaandam. The Inntel hotel is chosen because it is a 12 storey medium rise building with a box like arrangement which makes the use of timber easy to apply. Remarkable of the Inntel hotel is that it has the appearance of being build with timber but in fact is constructed with reinforced concrete. Therefore it would be interesting to see if the building could have been made with an actual timber structure instead of only having the appearance of having a timber structure.

The replacement of a reinforced concrete structure with a timber structure needs extra attention regarding the stability and fire safety. The relative small weight of a timber structure may lead to large tension forces in the foundation and the relative small bending stiffness may lead to a large horizontal drift. The load bearing structure requires a fire resistance of 120 minutes and may lead to thicker structures. The design of the timber structure will therefore mainly be focused on these aspects.

As mentioned in chapter 3, it is very difficult to estimate the acoustic performance of light weight structures. It requires a lot of knowledge and experience in building acoustics to give realistic approximations of the acoustic performance. Therefore the acoustic performance of the structure is not explicitly taken into account.

Figure 5.1 Inntel Hotel
Source: WAM architecten

5.2 Inntel Hotel

The Inntel hotel is a twelve storey high building which stores 160 rooms and includes a bar-restaurant, swimming pool, wellness centre, Finish sauna and Turkish steam bath. It has essentially a square plan and is in part constructed on a viaduct. The building represents a monumental stacking and interpretation of various green painted house types typical of the Zaan region, ranging from a stately notary's dwelling to workers cottages.

The varied fenestration, broad protruding sections and bay windows lend depth and an expressive relief to the façade, as can be seen in figure 5.2.

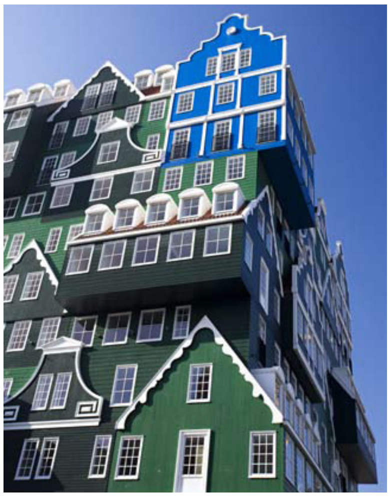


Figure 5.2 East facade
Source: WAM architecten

The dimensions of the building are based on a grid size of 3,6 metre. This gives an overall surface area of 28,8 x 21,6 square metre. The total building height is approximately 39 metres which is divided over 12 stories where the majority of the stories have a storey height of 3 metre and clear height of 2,6 metre. The apartment separating walls and floors consists of concrete and are quite thick. This is not due to structural requirements but is needed in order to fulfil the acoustic requirements. The load bearing walls have a thickness of 220 mm while the floors have a thickness of 180 and 230 mm. The concrete floors have a 40 mm covering floor which makes the total thickness of the floors 220 and 270 mm.

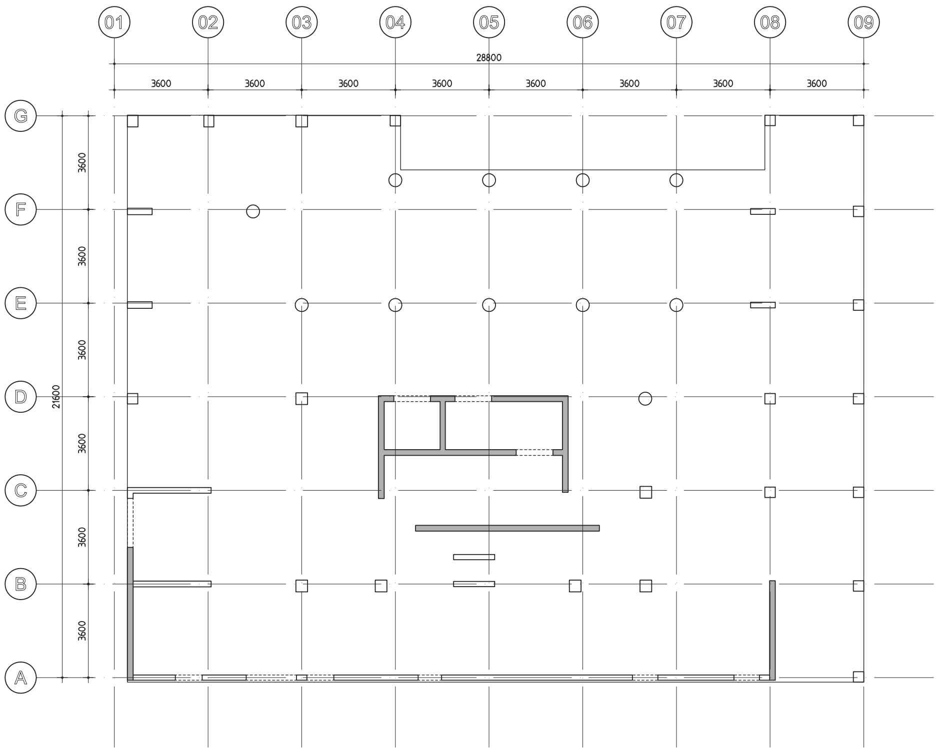


Figure 5.3 Ground floor plan
Source: Drawn by the author

Walls coloured in grey are stabilizing

The main load bearing structure of the building consists of cast and pre-cast concrete walls and concrete floors which consists of a pre-cast panel with a cast structural topping (in Dutch breedplaatvloer). The walls on the ground floor are in situ cast and the walls on the upper floors are pre-cast. Locally this is deviated with the presence of large recesses, in this case several storeys are cast is situ.

Stability is provided by the interior walls and floors and not by the central core or the façade. The walls are continuously stacked on one another from the second to the eleventh storey. The walls are not continuous over the ground floor (in the building plan called first storey) because the ground floor has a more open floor plan and consists therefore mainly of columns. The stability of the twelfth storey is provided by a steel structure, the stabilizing walls are here not present.

Because of the open floor plan on the ground floor, the stabilizing walls on the second storey are situated on concrete columns. Wind forces must be transferred to the foundation which means that the columns under the stacked walls transfer the wind moment. Columns are not suitable for transferring large shear forces so the shear force resulting from the wind load is transferred through the floor to the deviated positioned walls on the ground floor, see figure 5.7.

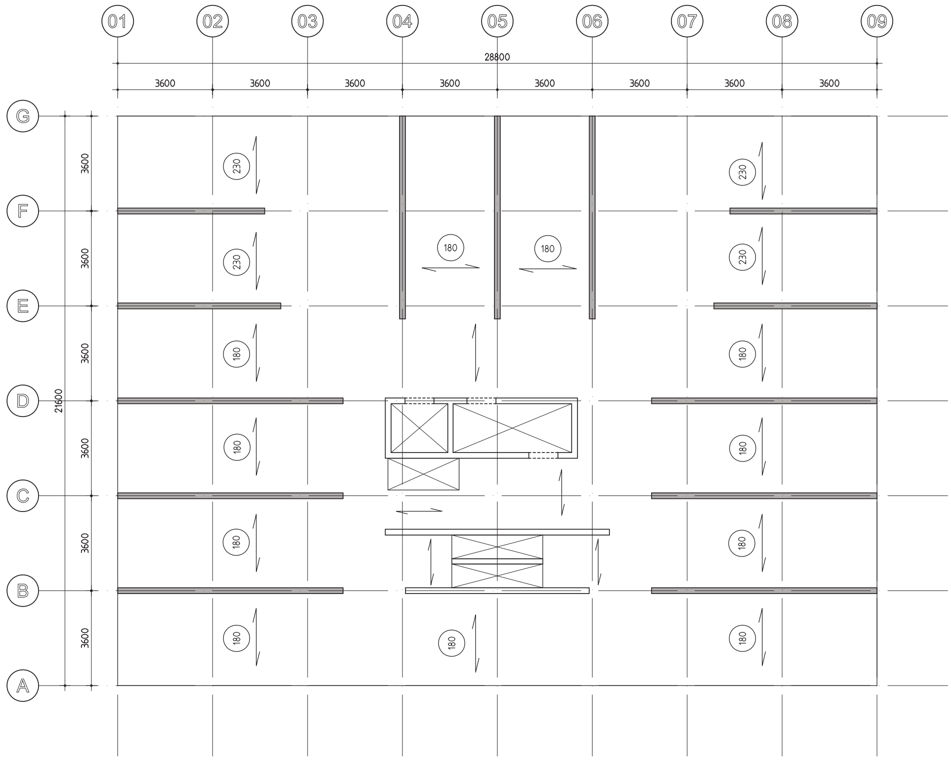


Figure 5.4 Typical floor plan
Source: Drawn by the author

Walls coloured in grey are stabilizing

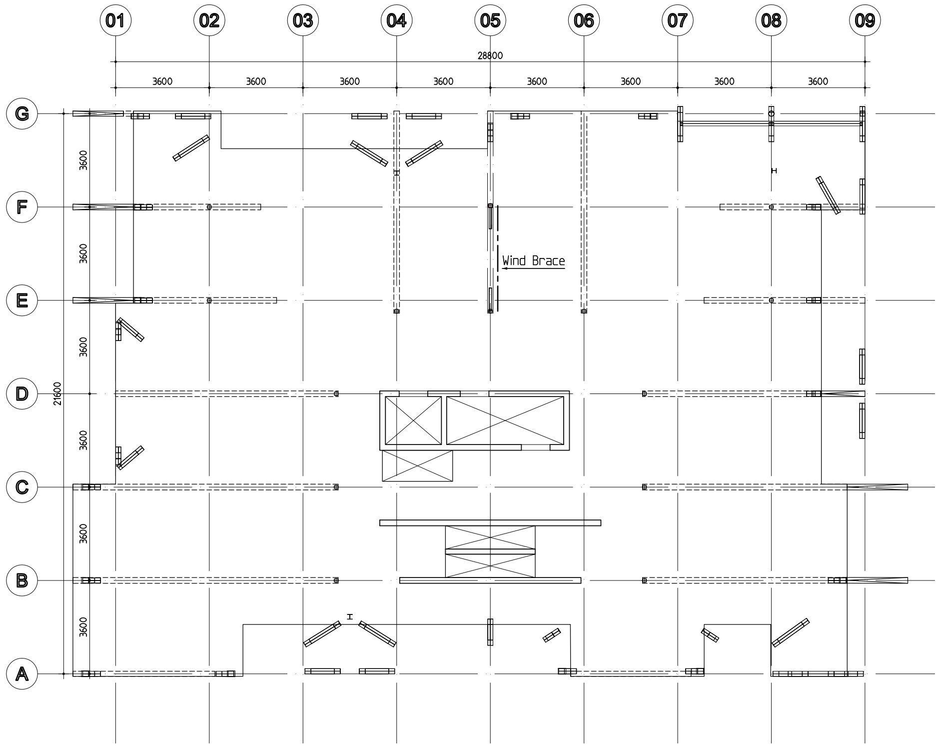


Figure 5.5 Twelfth storey
Source: Drawn by the author

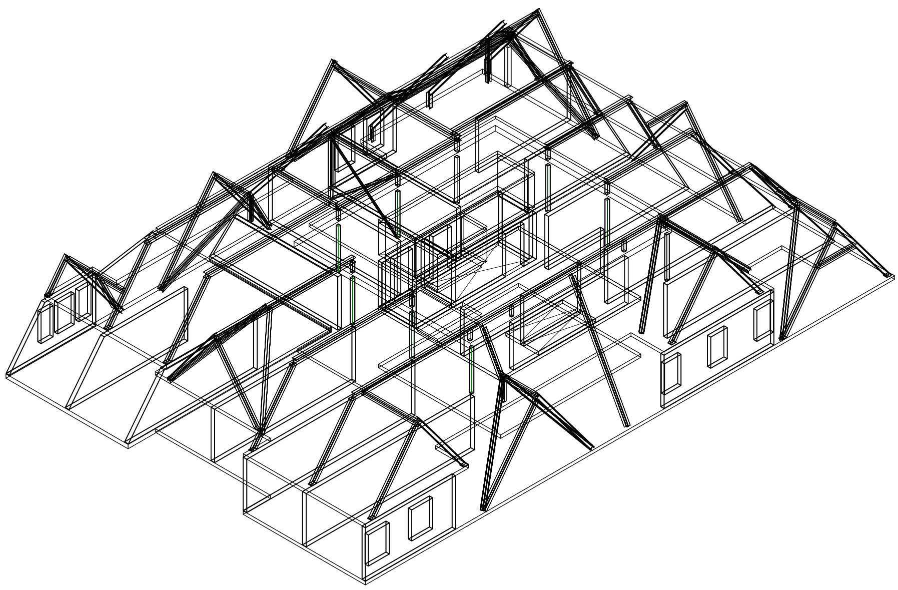


Figure 5.6 Roof structure
Source: DHV (structural engineer)

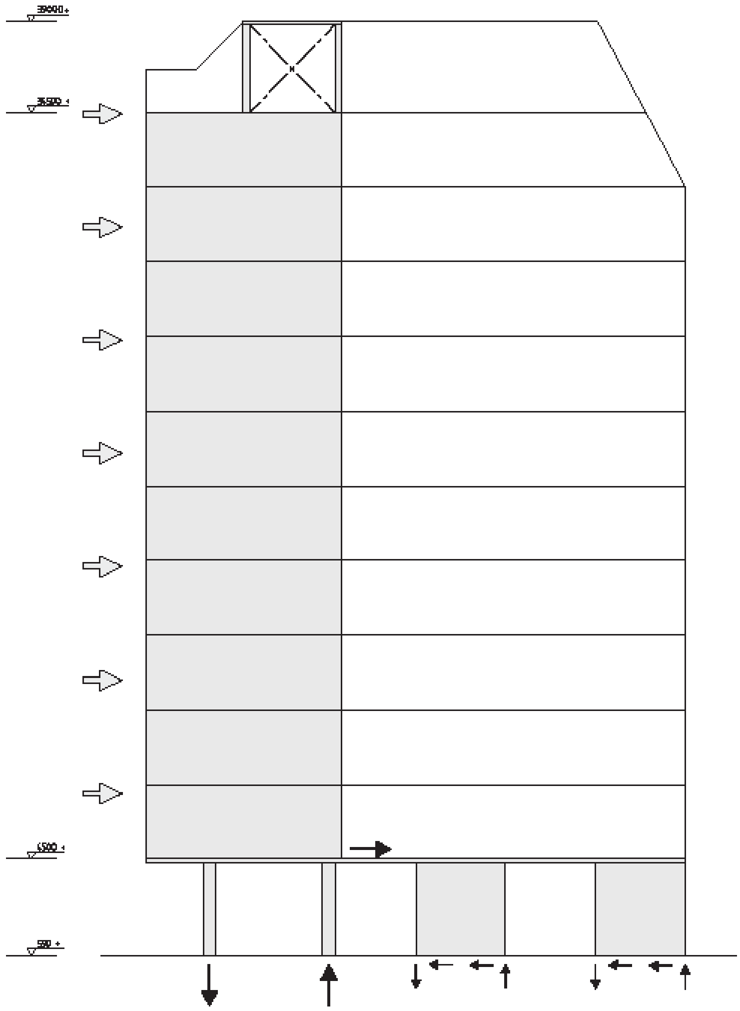


Figure 5.7 Schematic view of transferring wind loads to foundation

5.3 Structural design

The basis for the design of the timber structure are the following assumptions and preconditions:

- The loads from the structural engineer DHV are used (appendix B)
- The overall structure of the building is designed as a concrete substructure and timber superstructure
- The building is designed using the same principle load bearing structure
- Only the governing timber floor and wall panels are calculated
- Fire safety is ensured with passive fire protection and a fire resistance of 120 minutes
- The acoustic performance is not taken into account
- Verifications are made using the Eurocode and the Dutch national annex

5.3.1 Construction method

An important consideration when designing a timber structure is to choose a suitable construction method. As explained in the previous chapters, there are a number of construction methods each with its own advantages and disadvantages. The platform method is popular because it provides a working platform during construction. A disadvantage is the load perpendicular to the grain which occurs in floor panels between walls which might become governing with a building of this height. The opposite is true for the balloon frame method where stresses perpendicular to the grain doesn't occur in floor panels. The disadvantage however is the absence of a working platform and the sensitivity of flanking sound transmission.

There exists also the possibility of interrupting the flanking sound transmission of the balloon frame method by not designing the walls to be continuous over several stories. This method interrupts the flanking sound transmission while still has the advantage of the absence of stresses perpendicular to the grain in floor panels. The downside of this method however, is the numerous elements that are required for assembling the structure. This makes this method less efficient and thus more expensive when compared to the platform and balloon frame method.

If we take the previous arguments into consideration, we can conclude that the platform method is the most suitable construction method for the Inntel hotel based on the ease of construction, fast construction time and little sensitivity to flanking sound.

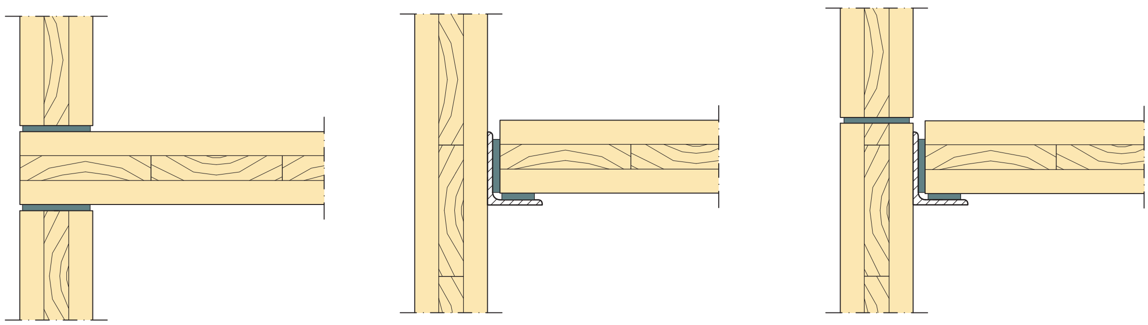


Figure 5.8 Possible methods of construction
Source: Drawn by the author

For transport reasons the maximum length of the floor panels is kept to a maximum of 15 metre. The restriction of 15 metre is still useful because large sections of the building can be covered by a single panel. This can be seen in the lower side of the building (figure 5.9) where the floor panels stretches from grid line A to E with a length of 14,4 metre.

The panels on the upper side of the building between gridline 03 and 07 span in the transverse direction perpendicular to the three interior wall panels. The distance between the two gridlines is 17,2 metre and is too large to be covered with a single floor panel. The distance is therefore covered with 3 floor panels spanning from gidline 03 to 04, 04 to 06 and 06 to 07.

The support of the floor panels on gridline 04, 06 and E of the panels spanning from line 04 to 06 and from line G to E are designed with overlap. This has been done for its better structural integrity and for its better fire safety when compared to other typical supports, see figure 5.10.

The support with overlap has a better fire safety because the critical cross section is only loaded by the support reaction of the floor, while the other two supports in figure 5.10 are also loaded by a normal force from the wall which can get very large when storeys increase.

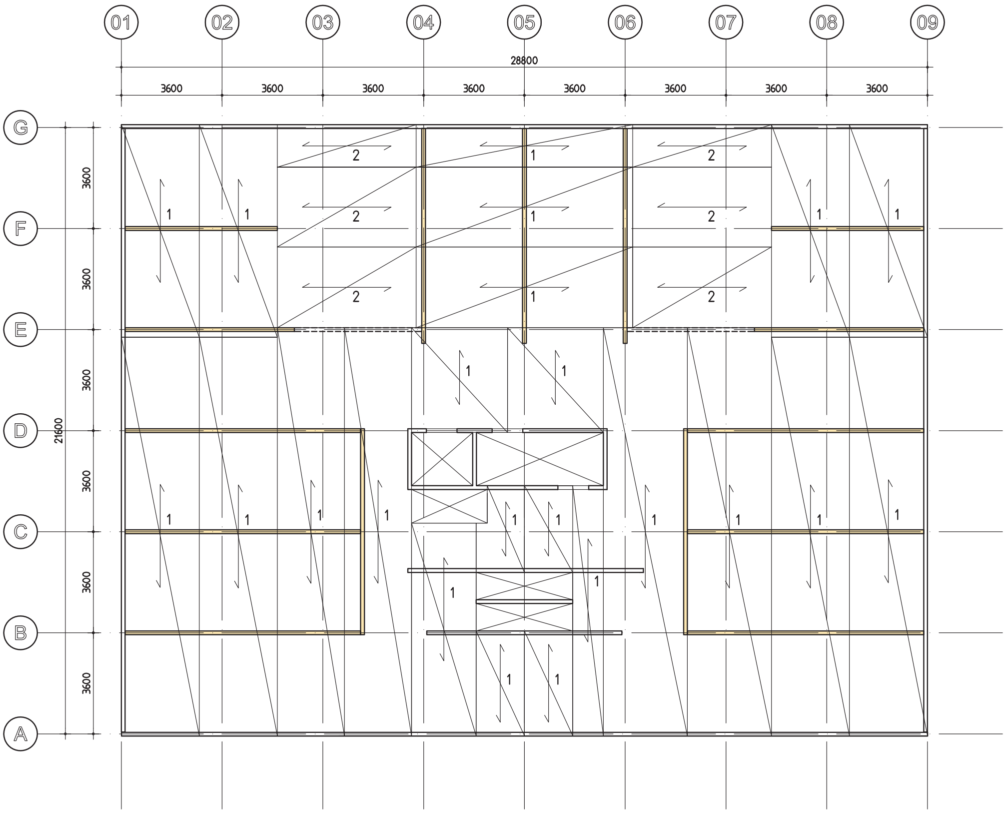


Figure 5.9 Typical floor plan
Source: Drawn by the author

1 = CLT 135
2 = CLT 162

All walls CLT 135
Coloured walls are stabilizing

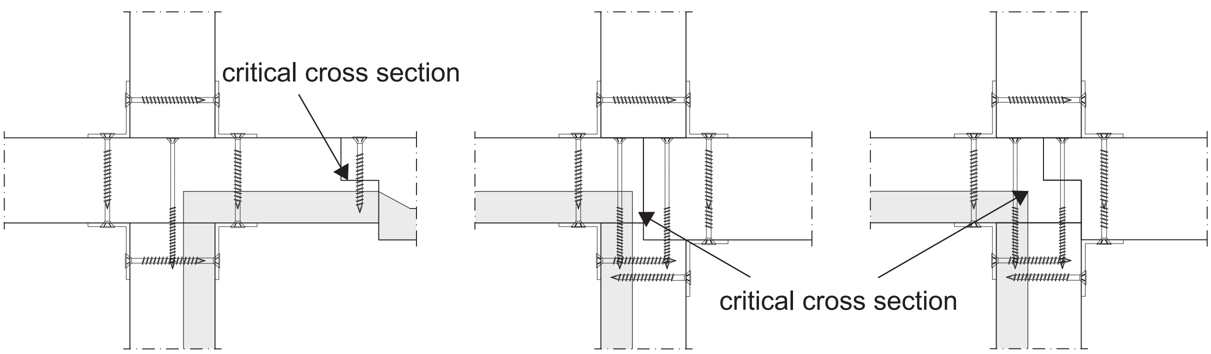


Figure 5.10 Different supports with burned section in grey after 120 minutes of fire

5.3.2 Loads

Estimations of the thickness of the load bearing elements have to be made in order to draft the loads.The thickness of the CLT panels is estimated with the help of the span tables from chapter three. Two different floor thicknesses are used in the building. One with a span of 3,6 metre which is estimated to be 135 mm and one with a floor span of 5 metre which is estimated to be 162 mm. The wall panels are estimated to be 135 mm. All timber elements are covered by two layers of gypsum fibre board with a thickness of 15 mm to ensure the required fire resistance.

Overview loads (kN/m²)

Flat roof	G_k	CLT 135						0.57
		roofing + insulation						0.25
		ceiling						0.50
								1.32
	Q_k			ψ_0	=	0.00	extreme	1.00
Sloped roof	G_k	CLT 135						0.57
		gypsum board t=	0.03	16				0.48
		tiles + insulation						0.50
								1.55
	Q_k			ψ_0	=	0.00	extreme	0.00
Service room	G_k	CLT 243						1.02
		floor covering						0.50
		ceiling						0.50
								2.02
	Q_k			ψ_0	=	0.50	extreme	6.00
Floor h= 135	G_k	CLT 135						0.57
		floor covering						0.50
		ceiling						0.50
		seperating walls						0.80
								2.37
	Q_k			ψ_0	=	0.40	extreme	1.75
Floor h= 162	G_k	CLT 162						0.68
		floor covering						0.50
		ceiling						0.50
		seperating walls						0.80
								2.48
	Q_k			ψ_0	=	0.40	extreme	1.75
Load bearing wall	G_k	CLT 135						0.57
		gypsum board t=	0.06	16				0.96
								1.53
Facade	G_k	CLT 135						0.57
		gypsum board t=	0.03	16				0.48
		finishing						0.50
								1.55

Wind load according to NEN 6702

Zaandam area 2, build environment → $p_w = 1{,}25\ kN/m$

$h = 39\ m$; w ⊥ $numerical\ axis = 28{,}8\ m$; $c_{dim} = 0{,}90$

Pressure plus suction → $0{,}8 + 0{,}4 = 1{,}2$

$p_{rep} = c_{dim} c_{pe} p_w = 0{,}90 \cdot 1{,}20 \cdot 1{,}25 = 1{,}35\ kN/m^2$

$h = 28{,}8\ m \rightarrow p_w = 1{,}10\ kN/m$

$p_{rep} = 0{,}90 \cdot 1{,}20 \cdot 1{,}10 = 1{,}19\ kN/m^2$

Suction on roof

$c_{pe} = 0{,}7$

$p_{rep} = 0{,}90 \cdot 0{,}7 \cdot 1{,}25 = 0{,}79\ kN/m^2$

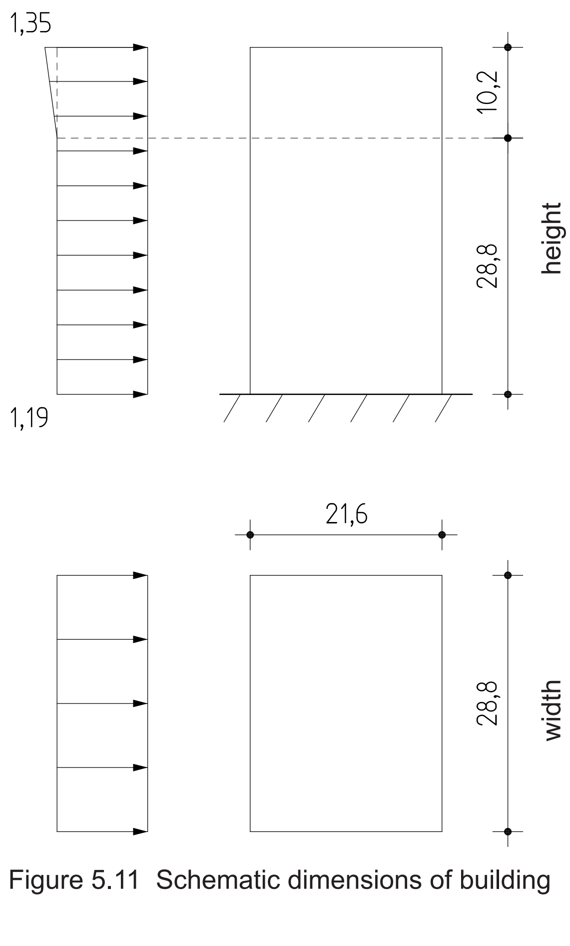


Figure 5.11 Schematic dimensions of building

5.3.3 Load bearing structure

This section covers the design of governing elements of the load bearing structure. In order to minimize the length of this chapter only the interesting calculations and results are presented. The complete calculations can be found in appendix A.

5.3.3.1 Roof

The steel structure on the 12th storey is replaced by a timber structure. The wind brace on gridline 06 in the original steel structure is left out in the timber design. The stability of the 12th storey is instead provided by CLT wall panels around the central core which makes the structure more robust.

The steel structure which mainly consists of steel beams HEA 260 and rectangular steel columns 150 * 150 * 12,5 mm are replaced by glulam beams and columns GL 24h 240 * 360 mm and GL 24h 200 * 200 mm respectively.

Figure 5.12 Roof plan
Source: Drawn by the author

1 = CLT 135
3 = glulam beam 240 * 360

All walls CLT 135
Coloured walls are stabilizing

Glulam beam

The governing beams for design are the two beams on gridline E. The deflection of the beam is governing while the verifications for ultimate limit state design are easily achieved.

Overview of governing verifications

Verification	Unity Check
Flexure	*0,77*
Shear	*0,38*
Compression ⊥ support	*0,67*
Deflection	*0,98*

Glulam column

The governing column for design is the one near intersection E – 05. The load on the column is small, N_d= 38,19 kN, and results in a small unity check:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} = \frac{0,95}{0,74 \cdot 15,36} = 0,08$$

Governing for design of the column, however, is the fire safety. The reduced cross section after 120 minutes of fire results in a low instability factor and reduces the strength of the column considerably. The unity check for stability during fire is:

$$\frac{\sigma_{c,0,fi}}{k_c \cdot f_{c,0,fi}} = \frac{2,76}{0,14 \cdot 27,60} = 0,71$$

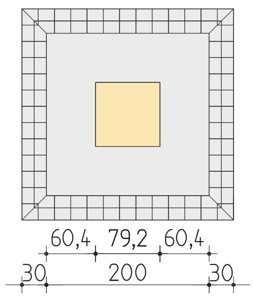


Figure 5.13 Column with reduced cross section

5.3.3.2 Floor panels

The two governing CLT floor panels in figure 5.14 are calculated.

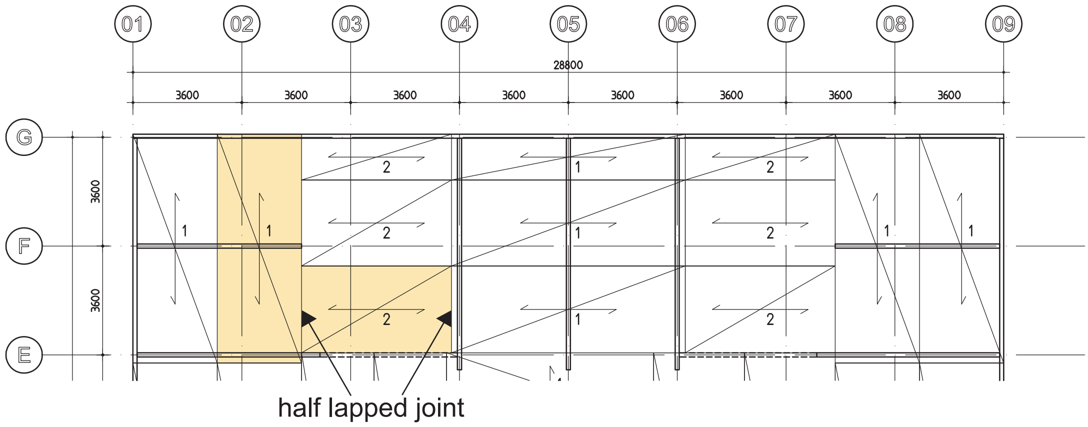


Figure 5.14 Calculated floor panels
Source: Drawn by the author

Both panels satisfy very well the ultimate limit state verifications. Governing for the two panels is the maximum allowable deflection and the shear force in the half lapped joint during fire. The half lapped joint is governing because the reduced cross section of the joint is dangerously small: h_{ef}= 67,5 - 60,4 = 7,1 mm.

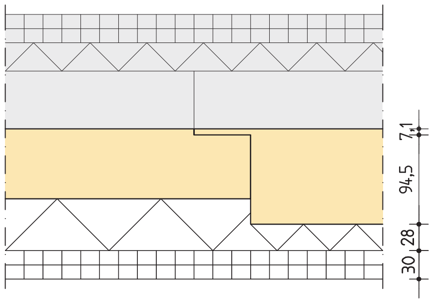
The fire safety of the half lapped joint can be improved by using locally a thicker plasterboard. When a thickness of 20 mm and 15 mm is used, the reduced cross section becomes: $h_{ef} = 67{,}5 - 50{,}6 = 16{,}9$ mm and satisfies the fire resistance verifications.
The shear check during fire is with the simplified method is:

$$\frac{\tau_{d,fi}}{k_v \cdot f_{r,d,fi}} = \frac{0{,}67}{0{,}67 \cdot 1{,}15} = 0{,}87 \;\leq 1$$

Overview of governing verifications

Verification	**U.C. CLT 162**	**U.C. CLT 135**
Flexure	*0,40*	*0,75*
Shear half lapped joint	*0,65*	*0,64*
Deflection	*0,77*	*0,93*

2 layers of 15 mm plasterboard



2 layers of 15 and 20 mm plasterboard

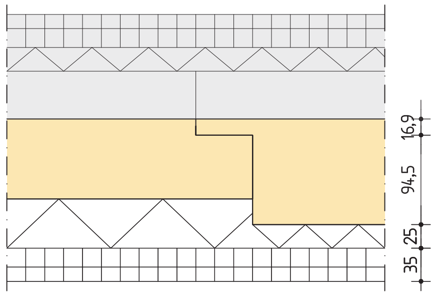


Figure 5.15 Half lapped joint with reduced cross section

5.3.3.3 Wall panels 2nd storey

Because of the open floor plan on the ground floor, the walls on the second storey are supported by columns. The vertical loads are transferred by compression arches to the supporting columns where large forces are to be expected. The load transfer can be aproximated and analyzed with a strud and tie model.

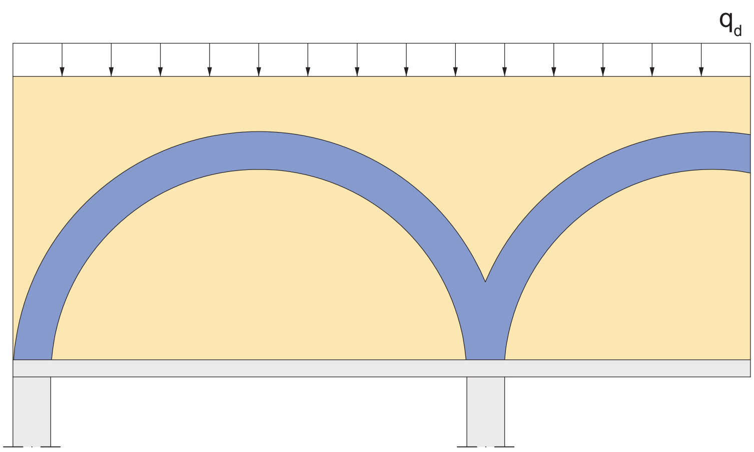


Figure 5.16 Compression arch in wall

When the forces in the strut and ties are known, the dimensions of the compression arches and tension ties can be calculated. When the compression arches are not disproportionally large, the structure suffice.

The long term is governing for design. The reaction forces are first calculated and next by equilibrium the strut and tie forces.

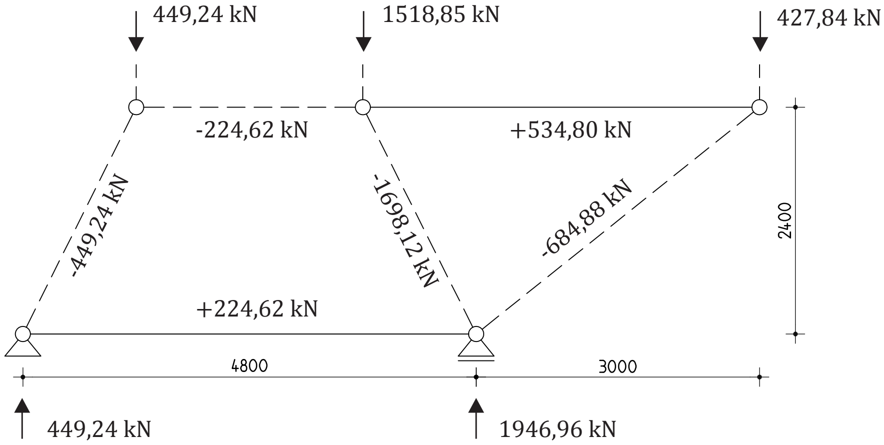


Figure 5.17 Strut and tie model

The height of the largest compression arch is:

$$f_{c,0,d} = \frac{k_{mod} \cdot f_{c,o,k}}{\gamma_m} = \frac{0,6 \cdot 21}{1,25} = 10,08\ N/mm^2 \rightarrow \frac{1698,12 \cdot 10^3}{81 \cdot 10,08} = 2080\ mm$$

Dimension of compression arch is 81 * 2080 mm

The height of the largest tension tie is:

$$f_{t,0,d} = \frac{k_{mod} \cdot f_{t,o,k}}{\gamma_m} = \frac{0,6 \cdot 14}{1,25} = 6,72\ N/mm^2 \rightarrow \frac{534,80 \cdot 10^3}{81 \cdot 6,72} = 983\ mm$$

Dimension of tension tie is 81 * 983 mm

Verification of shear force:

$$V_d = 1519,12\ kN;\ \ \tau_d = \frac{1,5\ V}{A} = \frac{1,5 \cdot 1519,12 \cdot 10^3}{135 \cdot 3000} = 5,63\ N/mm^2$$

$$f_{v,d} = \frac{k_{mod} \cdot f_{v,k}}{\gamma_m} = \frac{0,6 \cdot 2,5}{1,25} = 1,20\ N/mm^2$$

$$\frac{\tau_d}{f_{v,d}} = \frac{5,63}{1,20} = 4,69 > 1$$ not ok

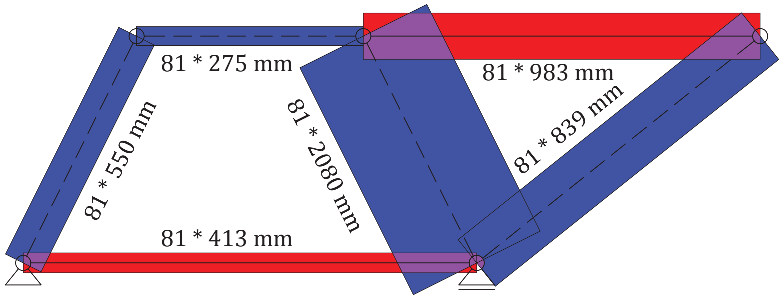


Figure 5.18 Dimensiones of strut and tie model

blue is in compression
red is in tension

As can be seen in figure 5.18, the size of the middle compression arch is too large and therfore not in relation with the dimensions of the panel and will not suffice. The shear stress is also too large to be resisted with a CLT panel. This has as a consequence that the structure of the first and second storey are designed in reinforced concrete and the structure of the third to twelfth storey in timber.

5.3.4 Stability

Wind load in the numeral direction of the grid will be governing since there are only three walls which provide the stability of the building. The three stabilizing walls when designed in timber does not seem to be in proportion with the quite larger floor plan. Preliminary calculations confirms this when considering the horizontal drift of the building.

The design calculations assume an even distributed load over each of the stabilizing walls. This is done because the floor which transfers the horizontal load has a high stiffness compared to the stabilizing walls which act as translation springs. The total horizontal drift on top of the building with three CLT 135 panels results in a horizontal drift of 102 mm while the requirement is 78 mm. The horizontal drift of the building is 30 percent more than the requirement and must obviously be decreased. The only way to effectively decrease the horizontal drift is by reducing the load subjected to the stabilizing walls. This is done by adding two stabilizing walls with the same stiffness near gridline 03 and 07, see figure 5.19, which reduces the horizontal drift to 65,7 mm. Fortunately the layout of the building makes these additions possible on every storey, by simply replacing separating walls with stabilizing walls.

In the previous section, the calculation of the second storey wall panels showed that the CLT panels cannot carry large shear and compression forces. The added CLT panels must therefore be carried by concrete walls on the second storey. The concrete walls have no acoustic requirement so they don’t have to be as thick as the other concrete walls. A wall thickness of 120 mm will most likely suffice.

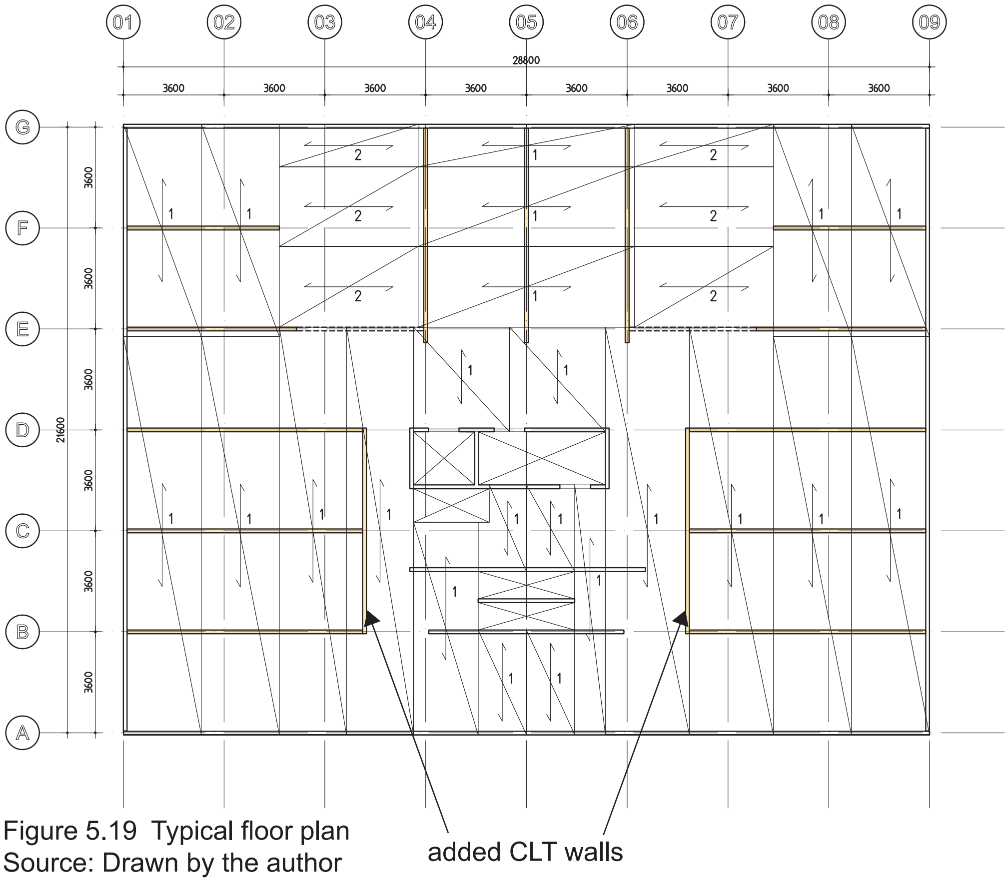


Figure 5.19 Typical floor plan
Source: Drawn by the author

5.3.4.1 Rotational stiffness

Preliminary calculations of the horizontal drift were made assuming that the shear walls are rigid. This approximation is valid to get a rough estimation of the horizontal drift but is not valid for an accurate design. The reason for this is the rotation that will take place between two stacked wall panels. The rotation is caused by the indent of the intermediate floor panel in the pressure zone and the deformation of the joints in the tension zone of the panel. The loads on the wall are favourable in this situation, because the more load is present on the wall, the higher the restoring moment which results in a higher rotational stiffness. Therefore, the Eurocode states that favourable dead load must be multiplied by a factor of 0,9 and favourable live load must not be considered.

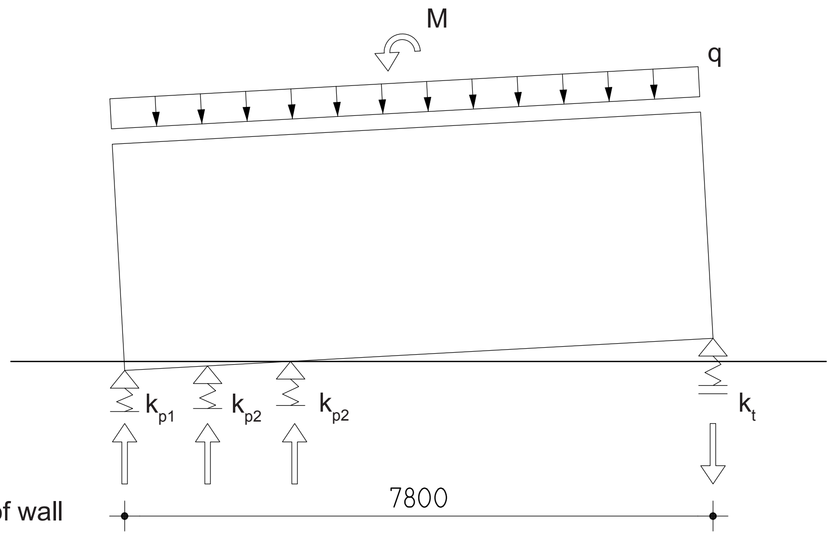


Figure 5.20 Rotation of wall

The spring and rotational stiffness is calculated for the panels of the third and fourth storey.

3rd storey

The bending moment acting on the wall is: $M_{rep} = 3409{,}91\ kNm$

The representative load on the 3rd storey wall is: $q_{rep} = 143{,}39\ kN/m$

The restoring moment in serviceability limit state design is:

$$M_{r,rep} = \frac{1}{2} \cdot 143{,}39 \cdot 7{,}8^2 = 4361{,}92\ kNm$$

$$M_{rep} < M_{r,rep}$$

The bending moment on the wall is smaller than the restoring moment, so no tension force and joint slip occurs. The 3rd storey floor is made of concrete so no indentation occurs, so it can be concluded that no rotation of the wall panels occurs.

4th storey

The rotational stiffness of the 4th storey is a bit more complicated to determine than the 3rd storey. The deformation of the pressure and tension joint must be calculated by determining the spring stiffness of the joints.

The spring stiffness on the pressure side is: $k_p = \frac{EA}{h}$

When nodes are used with a centre to centre distance of 520 mm, the spring stiffness becomes:

$$k_{p1} = \frac{340 \cdot 260 \cdot 135}{135} = 8{,}8 \cdot 10^4\ N/mm$$

$$k_{p2} = \frac{340 \cdot 520 \cdot 135}{135} = 17{,}7 \cdot 10^4\ N/mm$$

The spring stiffness of the tension joint consist of several series and parallel spring stiffnesses which must be added together to form the total spring stiffness of the joint. The spring system of the upper side of the joint is shown in figure 5.22. Since the joint is symmetrical, the same stiffness of the upper side of the joint is present in the lower side of the joint. So in order to calculate the stiffness of the whole joint, the equivalent stiffness of the upper side of the joint is halved because the upper and lower side of the joint is a parallel connection.

The first spring in the joint, k_1, is caused by the slip of the connecting elements. It is assumed that eight dowel like elements are needed for design.

The second spring in the joint, k_2, is caused by the deformation of the steel angle. It is assumed that a steel angle with a width of 200 mm is needed for design.

The third spring in the joint, k_3, is caused by the indentation of the steel angle in the timber floor panel.

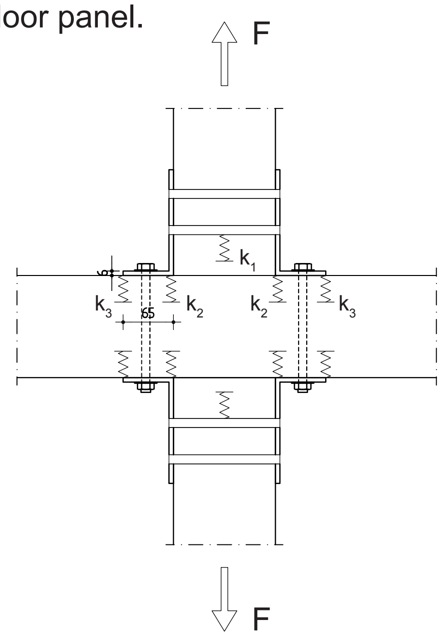


Figure 5.21 Springs in tension joint

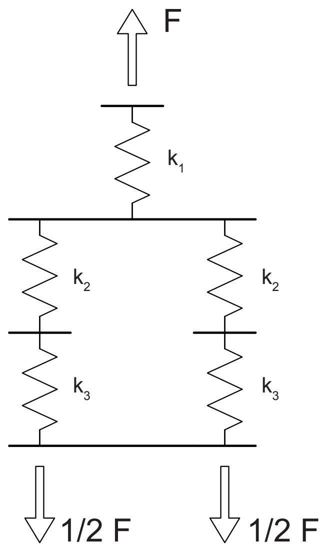


Figure 5.22 Spring system upper side of joint

$$k_{ser} = 2 \cdot \frac{{\rho_m}^{1,5} d}{23} = 2 \cdot \frac{350^{1,5} \cdot 12}{23} = 0,7 \cdot 10^4 \; N/mm$$

$$k_1 = 2\, n\, k_{ser} = 2 \cdot 8 \cdot 0,7 \cdot 10^4 = 10,9 \cdot 10^4 \; N/mm$$

$$k_2 = \frac{3EI}{l^3} = \frac{3 \cdot 2,1 \cdot 10^5 \cdot \frac{1}{12} \cdot 200 \cdot 6^3}{33^3} = 6,3 \cdot 10^4 \; N/mm$$

$$k_3 = \frac{E_{90} A}{h} = \frac{340 \cdot 65 \cdot 200}{135} = 3,3 \cdot 10^4 \; N/mm$$

$$k_{eq} = \frac{1}{\frac{2}{k_1} + \frac{2}{k_2} + \frac{1}{k_3}} = 1,0 \cdot 10^4 \; N/mm$$

The equivalent spring stiffness k_{eq} is much lower than each individual spring stiffness which is typical for a parallel system. The total stiffness of the connection k_t is:
$k_t = 1/2 \cdot 1{,}0 \cdot 10^4 = 0{,}5 \cdot 10^4\ N/mm$

Now that the stiffnesses of the supports are known it is possible to calculate the rotation of the wall panel and the reaction forces acting on the wall and floor panel.

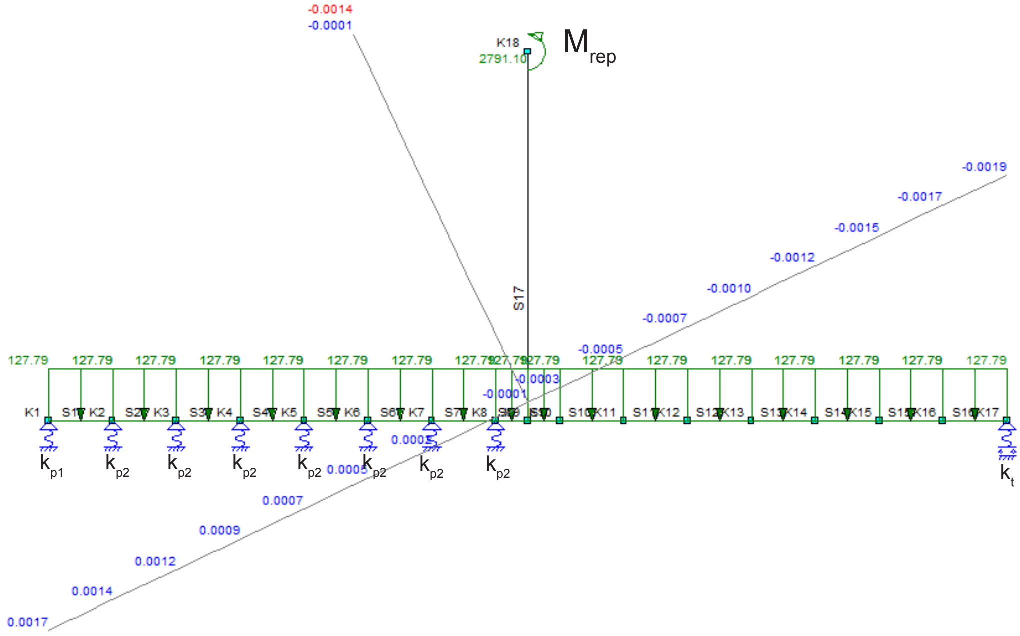


Figure 5.23 Rotation storey 4, nodes c.t.c. 520 mm, generated with MatrixFrame, values of deformation in metre

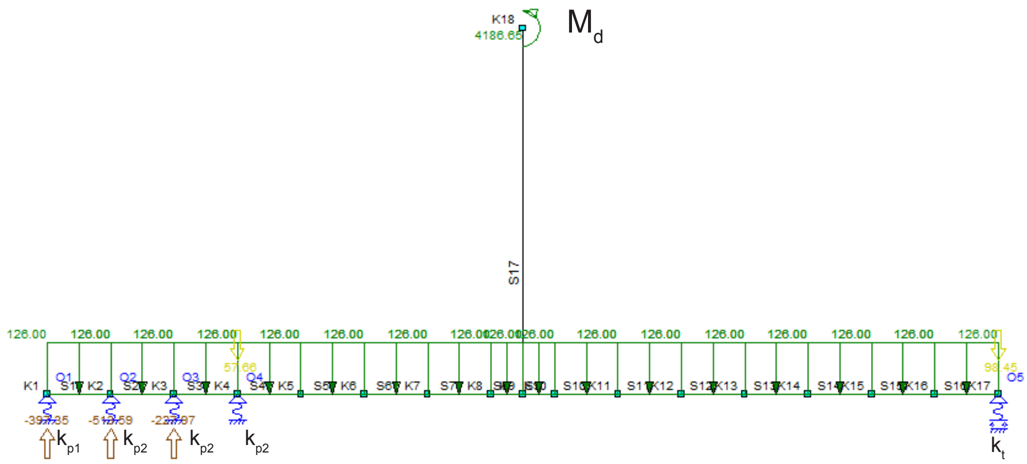


Figure 5.24 Support reactions storey 4, nodes c.t.c. 520 mm generated with MatrixFrame

The rotation of the wall in serviceability state design is: $\varphi = 0{,}462 \cdot 10^{-3}\ rad$

The rotational stiffness of the wall is:

$$C = \frac{M}{\varphi} = 0{,}60 \cdot 10^7\ kNm/rad$$

The reaction force in the outermost part of the panel is governing for design.
The reaction force is: $R_d = 397{,}35\ kN$

The compression stress perpendicular to the grain in the floor panel is:

$$\sigma_{c,90,d} = \frac{R_d}{A} = \frac{397{,}35 \cdot 10^3}{135 \cdot 260} = 11{,}32\ N/mm^2$$

The maximum permissible compression stress perpendicular to the grain of a CLT panel with quality C24 is $f_{c,90,d} = 1{,}80\ N/mm^2$ and is a lot lower than the occurring compression stress.

It can be concluded that this configuration of the wall does not satisfy the requirements. The stiffness of the tension joint is low and results in a low rotation stiffness and high compression force in the panels. The tension joint must be therefore be stiffened to overcome these problems.

The joint is stiffened by stiffening the steel angle with steel plates which prevent the angle from opening. A steel plate beneath the wall panel also prevents the steel angle from rotating. These measures eliminates the stiffnesses k_2 and k_3 from the equivalent spring stiffness of the upper side of the joint.

The stiffness of the joint is now: $k_t = 1/2 \cdot 10{,}9 \cdot 10^4 = 5{,}45 \cdot 10^4\ N/mm$

The steel plate beneath the wall panel does not only function for stiffening the tension joint, it also increases the bearing area in the compression side of the panel. This is necessary since stiffening of the tension joint alone is not enough to reduce the compression stress to reasonable values. The steel plate is one metre long and placed on either side of the wall panel since wind load can act in two directions. The increased bearing area is stiffer and must be taken into account.

$$k_{ps1} = \frac{340 \cdot 260 \cdot 360}{135} = 2{,}35\ \cdot\ 10^5\ N/mm^2$$

$$k_{ps2} = \frac{340 \cdot 520 \cdot 360}{135} = 4{,}70\ \cdot\ 10^5\ N/mm^2$$

For further decreasing the compression stress and increasing the rotational stiffness, the tension joint is made double with a centre to centre distance of 300 mm.

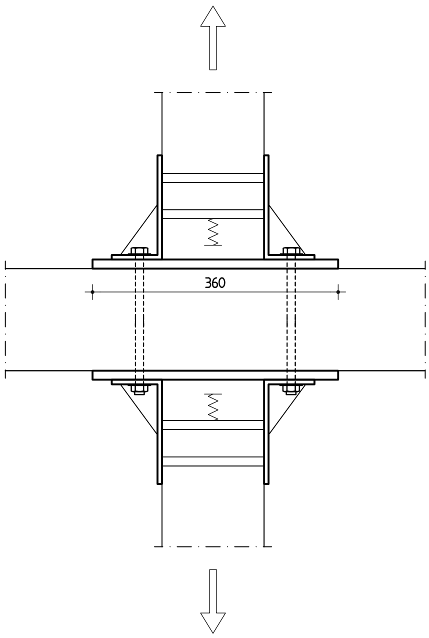


Figure 5.25 Stiffened tension joint

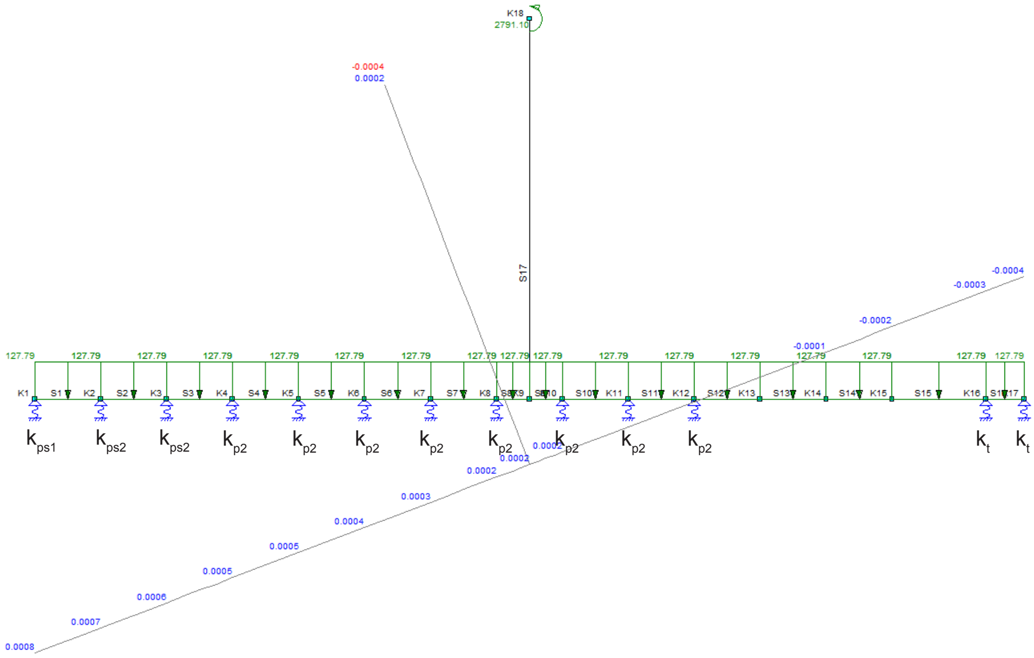


Figure 5.26 Rotation storey 4, nodes c.t.c. 520 mm
generated with MatrixFrame, dimension of deformation in metre

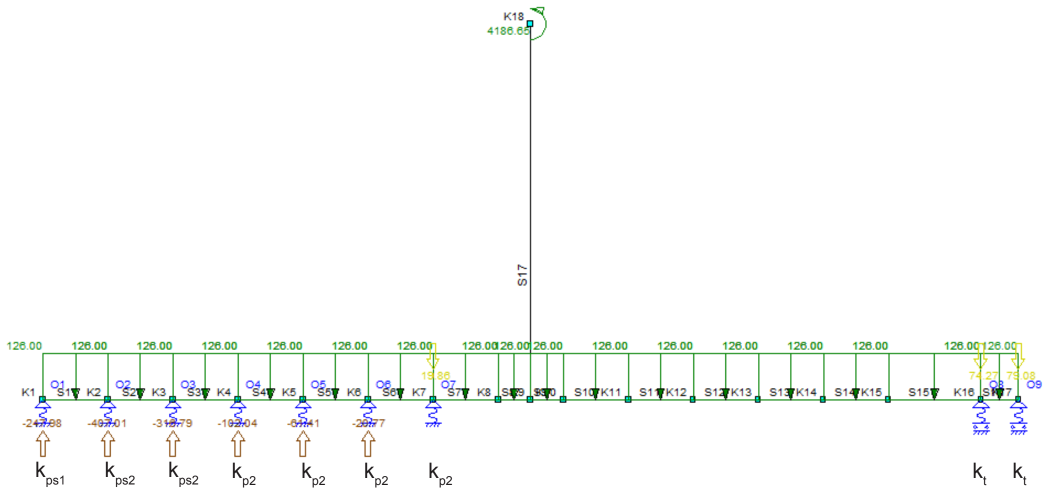


Figure 5.27 Support reactions storey 4, nodes c.t.c. 520 mm
generated with MatrixFrame

In this new situation the rotational stiffness has increased from $0{,}60 \cdot 10^7\ kNm/rad$ to $1{,}94 \cdot 10^7\ kNm/rad$. The support reaction in the outermost part has decreased from $397{,}35\ kN$ to $247{,}98\ kN$.

Due to the rotation of the walls, the horizontal drift on top of the building has increased from 65,7 mm to 77,5 mm but still satisfies the requirement of a maximum horizontal drift of 78 mm.

For simplicity, the drift is calculated using the rotational stiffness of the fourth storey for the fifth to twelfth storey. In reality the rotational stiffness increases with every storey since the tension and compression forces in the panel decreases. The drift is therefore slightly overestimated.

As an alternative to the presented solution, the wall panels can be continuously connected with the floor panels. The wall panel in figure 5.28 is designed with hold down fixings over the entire length of the panel with a centre to centre distance of 600 mm. The wall is calculated with the same spring stiffnesses as the variant with hold down fixings on the end of the wall panel.

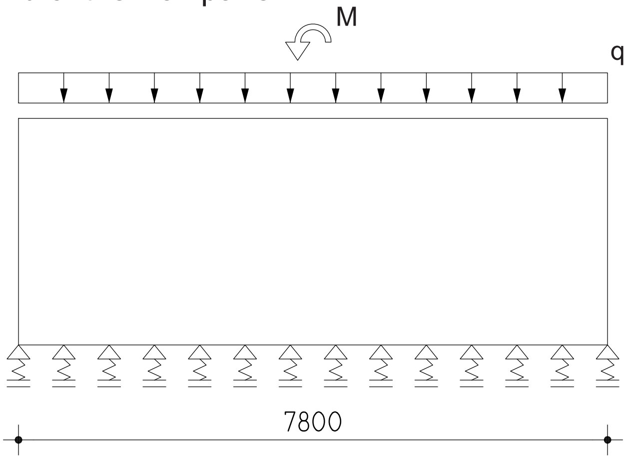


Figure 5.28 Alternative variant

By placing hold down fixings over the entire wall length the rotational stiffness is further increased from $1{,}94 \cdot 10^7\ kNm/rad$ to $2{,}22 \cdot 10^7\ kNm/rad$ and the support reaction decreased from $247{,}98\ kN$ to $208{,}02\ kN$.

The alternative variant has a higher rotational stiffness and a lower compression force. The downside of this variant is that a lot more stiff hold down fixings are required in order to gain a relative small increase of performance.

The assembly on site of the wall panels and hold down fixings will also be time consuming making the overall cost of the structure more expensive. This variant is therefore considered uneconomical for design.

5.3.4.2 CLT wall panel design

3rd storey, wall panel CLT 135

Governing for the design of the wall panels is compression in the outermost part of the panel caused by the dead load and the wind moment.

The compression stress in the outermost part of the panel is:

$$\sigma_{c,0,d} = 14{,}67\ N/mm^2$$

The maximum permissible compression stress is:

$$f_{c,0,d} = 15{,}12\ N/mm^2$$

Verification of failure condition:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} = \frac{14{,}67}{0{,}72 \cdot 15{,}12} = 1{,}35\ > 1$$ not ok

The shear walls in the numeral direction does not satisfy the design requirement. The thickness of the shear walls in the numeral direction are increased to 142 mm.

Wall panel CLT 142:

Verification of failure condition:

$$\frac{\sigma_{c,0,d}}{k_c \cdot f_{c,0,d}} = \frac{11{,}05}{0{,}74 \cdot 15{,}12} = 0{,}98 \ < 1 \qquad \text{ok}$$

The reason for the big difference in verification between the two panels with almost the same width is that the inner layers of the CLT 142 panel is more favourable for carrying loads. The shear and fire safety verifications results in low unity checks and are not governing for design.

Overview of verifications

Verification	**Unity Check**
Stability	*0,98*
Shear	*0,27*
Fire safety: stability	*0,51*

4rd storey, wall panel CLT 135

Again governing for design is compression caused by the wind moment.

Overview of verifications

Verification	**Unity Check**
Stability	*0,96*
Shear	*0,23*
Fire safety: stability	*0,70*

5.3.4.3 Bearing resistance of floor panels

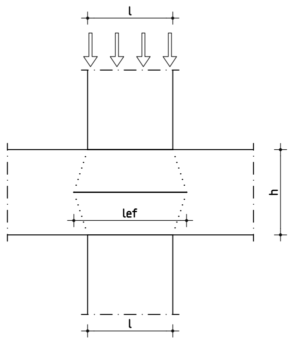
One of the most critical aspects of medium rise buildings using the platform method is the bearing resistance of floor panels between walls. The floor panels are loaded perpendicular to the grain direction and have a significant smaller load carrying capacity than the parallel to the grain loaded wall panels.

The verification of compression stresses perpendicular to the grain in Eurocode 5 is determined by $\sigma_{c,90,d} \leq k_{c,90} f_{c,90,d}$. The factor $k_{c,90}$ is set to be equal to 1 and in some cases can be increased to 4 depending on the configuration of the floor panel. The reason for this is that the compression load will be carried over by the fibres to the neighbouring unloaded sides. This effect will be higher for smaller widths.

In this design, the floor panels are continuous over the wall panels which results in the use of equation 6.6 of Eurocode 5 for determining $k_{c,90}$:

$$k_{c,90} = \left(2{,}38 - \frac{l}{250}\right)\left(\frac{l_{ef}}{l}\right)^{0{,}5}$$

$$l_{ef} = 0{,}5\left(l + l + \frac{2h}{3}\right)$$



The bearing area must be checked for two load combinations. The first one is the short term combination which takes account for the dead and live load and a compression force caused by the wind moment. The second one is the long term combination which takes only the dead and live load into account. As can be seen in table 18, the short term combination is governing.

The forces on the lower stories of the timber structure are quite high and result in high stresses perpendicular to the grain in the floor panels. The stress in the 4th storey floor panel is:

$$\sigma_{c,90,d} = \frac{N_d}{A} = \frac{1028 \cdot 10^3}{135 \cdot 1000} = 7{,}61\ N/mm^2.$$

The maximum permissible compression stress perpendicular to the grain is: $f_{c,90,d} = 1{,}80\ N/mm^2$ and is well below the occurring stress in the floor panel. The maximum increase of the compression stress by $k_{c,90}$ is $4 \cdot 1{,}80 = 7{,}20\ N/mm^2$ and is still not sufficient for carrying the compression force. This means that measures must be taken for reducing the compression stress in the floor panels.

Table 18 Compression forces in floor panel per storey

Storey	Short term in kN/m	Long term in kN/m
10	198	68
9	307	97
8	427	126
7	560	156
6	704	185
5	860	214
4	1028	243

The following options can be used for reducing the compression stress:

- Increasing the bearing area;
- Reinforcing the bearing area;
- Avoiding cross grain material;

Increasing the bearing area

The most simple way to reduce compression stresses is by increasing the bearing area of the floor panel. By increasing the wall thickness, the compression force is distributed over a larger bearing area which results in smaller compression stresses perpendicular to the grain. Table 19 shows the necessary wall thickness for carrying loads perpendicular to the grain.

The capacity of a 135 mm width bearing area can be factored with $k_{c,90} = 2,12$ which results in a relative large bearing capacity. As the width of the bearing area increases, $k_{c,90}$ decreases until a width of 360 mm where $k_{c,90}$ becomes less than 1. So the 135 mm bearing capacity must be doubled at the 4th storey, while the bearing area needs to be quadrupled and results in thick wall panels.

Increasing the bearing area by using thicker wall panels is in this case inefficient and therefore not appropriate.

A more efficient structure is acquired when the bearing area is divided over two sections. Two bearings will have a higher load carrying capacity than a single bearing with an equivalent thickness, provided that there is enough space between the bearings for load distribution, see figure 5.29. The capacity of the double 135 mm bearing area is 1030 kN/m, while the capacity of the 340 mm bearing area is just 664 kN/m.

Table 19 Wall thickness per storey and corresponding bearing capacity

Storey	Short term in kN/m	Wall thickness	$k_{c,90}$	Capacity in kN/m
12 - 8	427	135	2,12	515
7	560	160	1,97	567
6	704	400	1	720
5	860	480	1	864
4	1028	580	1	1055

Bearing capacity is 1030 kN/m

Bearing capacity is 664 kN/m

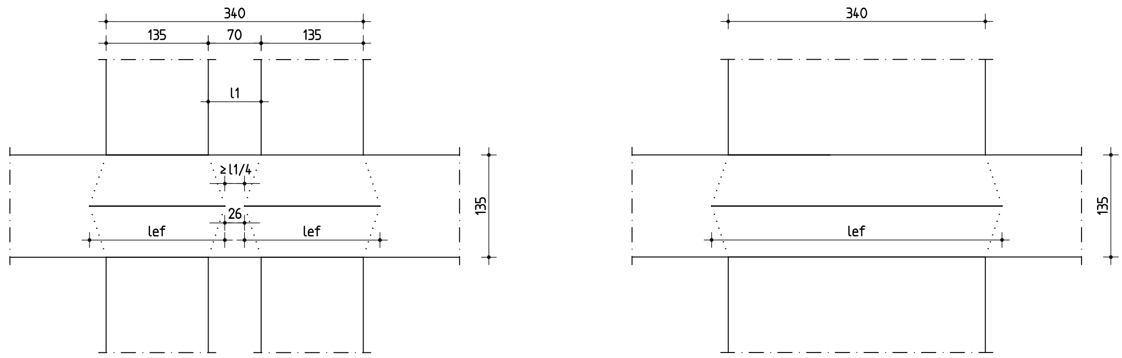


Figure 5.29 Double bearing versus single bearing

The use of two CLT panels in a wall will half the loads in the wall panels and bearing area. This means that the thickness of the wall panels can be decreased making the total wall thickness smaller. The thickness of the gypsum plasterboards on the wall panels can also be decreased since more timber is used which will increase the available charring depth. The cavity between the panels can be filled with an insulation material which makes the acoustic performance of the structure much better than a single CLT wall. The downside of using walls with double CLT panels is that a lot more timber is used in the structure, which makes the building more expensive.

The large compression stresses occur only in the outermost parts of the wall panels. This means that it could be useful if only the outermost parts of the bearing area is increased. This can be achieved by making a steel plate beneath the wall panel which distributes the load over a larger area.

The steel plate of the stiffened tension joint of section 5.3.4.1 can be used for this purpose. The steel plate has a width of 360 mm so it is known that the steel plate is too small for acquiring the required capacity. However, the bearing capacity can be increased by reinforcing the bearing area beneath the steel plate with screws instead of using a large steel plate with a width of 580 mm.

When the bearing capacity is reinforced with screws, the capacity depends on two failure conditions. The first one is the unreinforced bearing capacity plus the push through and buckling resistance of the screws and the second failure condition depends on the spread of the compression force in the panel.

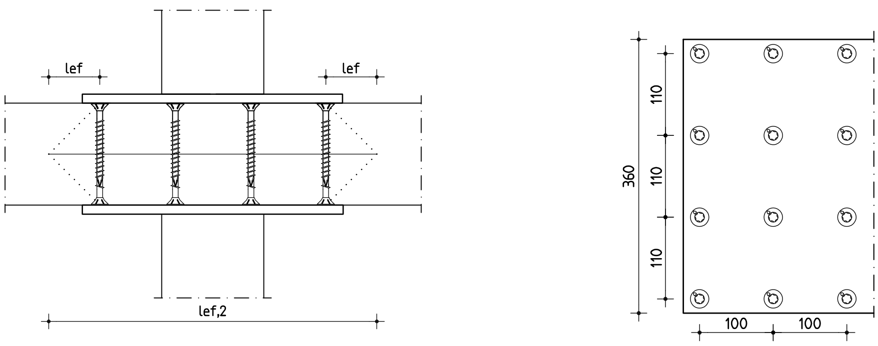


Figure 5.30 Reinforced bearing

The capacity of the bearing area reinforced with screws $\emptyset 12$ with a centre to centre distance of 100 mm can be calculated with the following equation:

$$F_{90,Rd} = min\begin{cases} k_{c,90} \cdot B \cdot l \cdot f_{c,90,d} + min(F_{ax,Rd}; F_{ki,Rd}) = 648 + 340 = 988\ kN \\ B \cdot l_{ef,2} \cdot f_{c,90,d} = 839\ kN \end{cases}$$

The compression force is still too high for the floor panel. Governing is the compression spread in the floor panel so reinforcing the panel even more with more screws makes no difference. What does make a difference is increasing the maximum permissible compression strength by using a higher quality CLT panel.

The design compression stress perpendicular to the grain of GL28h panels is: $f_{c,90,d} = 2{,}16\ N/mm^2$.

The bearing capacity is now:

$$F_{90,Rd} = 1007\ kN$$

Verification of failure condition:

$$\frac{F_{90,d}}{F_{90,Rd}} = \frac{1028}{1007} = 1{,}02 \approx 1 \quad \text{ok}$$

The maximum allowable compression stress is just exceeded but regarded as acceptable.

Reinforcing the bearing area

An alternative way of increasing the bearing capacity is by placing dowels in the floor panel which transfer the vertical loads directly to the underlying wall panel. In this way compression perpendicular to the grain is avoided in the floor panel.

The dowels can be made from timber and must be placed beneath the stiff parallel board layers of the wall panel. The thickness of the board layers is 27 mm which is also the maximum diameter of the dowels.

The total amount of dowels can be calculated with the surface area of the dowel and the maximum allowable stress of the timber.

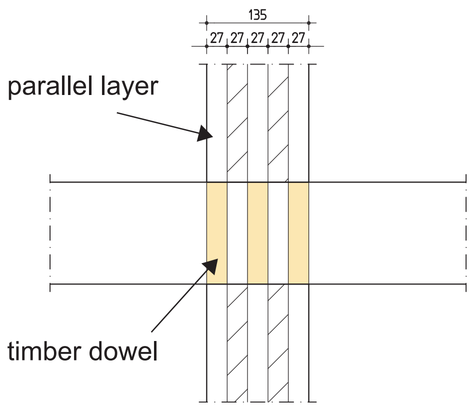


Figure 5.31 Reinforced bearing with timber dowels

Timber dowels with strength class C24 $\emptyset 27\ mm$; $A = \pi r^2 = 573\ mm^2$

Maximum allowable stress: $f_{c,0,d} = 15{,}12\ N/mm^2$

3 dowels in a row, $F_{dowel} = 15{,}12 \cdot 573 = 8{,}66\ kN$

Total dowels needed: $\frac{1028}{8{,}66} = 119$ dowels.

When the dowels are placed in rows of three, the centre to centre distance for 120 dowels will be: $\frac{1000}{40} = 25\ mm.$

The amount of dowels and small centre to centre distance clearly show the weakness of the timber dowels. The use of steel rods is therefore more advantageous in this situation.

The maximum allowable force in one steel rod $\emptyset 14$ with steel quality 8.8 is $F_{ki,Rd} = 53{,}20\ kN.$

The total dowels needed over 1 metre is: $\frac{1028}{53{,}20} = 20$

When 10 dowels are used in rows of two, a centre to centre distance of 100 mm is needed.

The force in the steel rods is high and will result in a large stress in the wall panels. The rods must therefore be covered with a steel plate which distributes the compression force over the wall panels.

It is clear that large compression forces can be transferred with the use of steel rods. The downside however is that the difference in stiffness between the reinforced and unreinforced area of the floor panel can become very large due to shrinkage of the floor panel. When the difference in stiffness is large, most of the compression force will be transferred to the reinforced area which eventually will result in overstressing of the steel rods and wall panel.

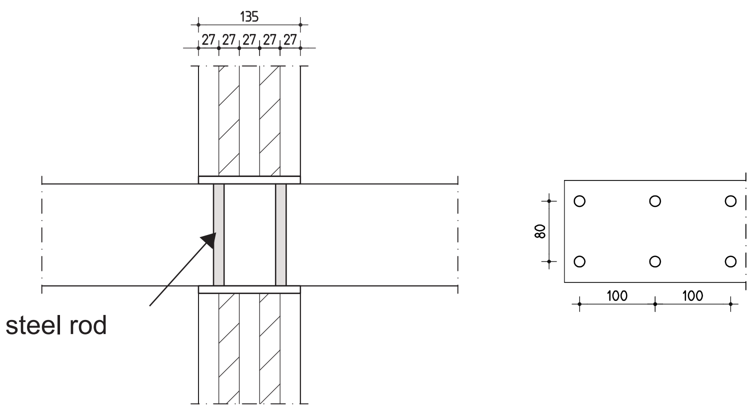


Figure 5.32 Reinforced bearing with steel rods

This means that the wall panel is not anymore fully supported by the floor panel but is supported by the reinforced bearing area, see figure 5.33. This will lead to overstressing of the reinforced area and overstressing of the wall panel, like the 2nd storey walls in section 5.3.3.3. Reinforcing the bearing area of the floor panel on the ends of the wall panel with this stiffness difference is therefore not advised. However, this method could be used when the full length of the floor panel is reinforced. In this way, differences in stiffness will not occur.

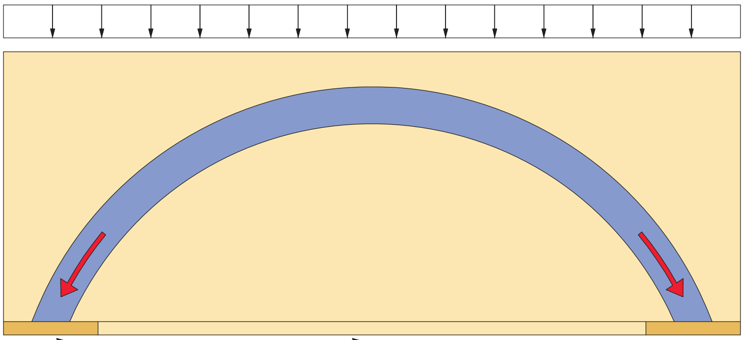


Figure 5.33 Load transfer to reinforced bearing due to large stiffness difference in bearings

Avoiding cross grain material

Another method of dealing with high compression forces perpendicular to the grain is by avoiding the occurrence of these forces by using the balloon frame method. By making several storey walls continuous and hanging the floor panels between the walls, compression perpendicular to the grain is avoided.

When the wall panels are made continuous over several stories, the 7,8 metre wide walls must be made in three sections of 2,6 metre because the maximum width of CLT panels is only 3 metre.

Because the 7,8 metre wide wall is divided in three sections, the stiffness of the wall is reduced. The stiffness difference between 3 separate panels and a 7,8 metre long panel is: $\frac{7{,}8^3}{3 \cdot 2{,}6^3} = 9$. This means that the 3 panels must connected so that they can work together for increasing the stiffness that is necessary for controlling the drift of the building. This connection can be made with a double leaf joint, see figure 5.35.

The joint can be made with screws Ø10 with a vertical centre to centre distance of 150 mm. However, the problem with this connection is that the cooperation factor γ between the panels is very low, γ = 0,18. The combined stiffness of the three panels with a cooperation factor of 0,18 is still low, so the stiffness of the joint must be increased. Using a smaller centre to centre distance for the screws has no significant effect and will only increase the cost of the joint. The stiffness of the joint can be increased by gluing the connecting boards to the CLT panels. According to J. Vessby in reference [29], the glued leaf connection has a very high stiffness comparable to a panel with no connection.

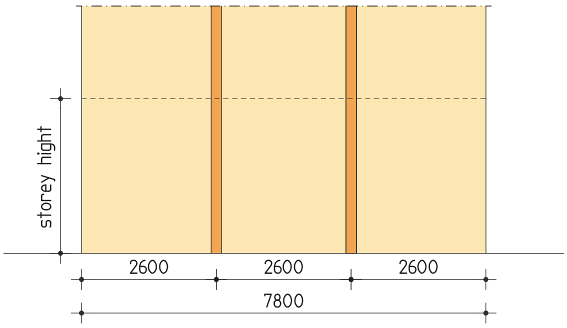


Figure 5.34 Vertical continuous panels

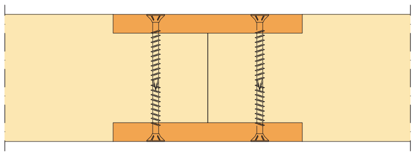


Figure 5.35 Double leaf joint

The problem with this construction method in this case, is that the connected panels have a large dimension. This means that the connection between the panels cannot be prefabricated because the wall panel would then be impossible to transport. The connection must therefore be made on the building site making this construction method more labour intensive and when we realize that this construction method lacks a working platform, we can be conclude that this method is very unpractical and expensive.

Construction with the balloon frame method can be simplified if a working platform is available. This can be achieved if features of the balloon and platform frame method are combined. By making the outermost part of the wall with the balloon method and the interior part with the platform method, high compression stresses can be carried and a working platform is available for simplifying construction. Two variants can be designed, see the two figures in figure 5.36.

The left figure has in the outer section continuous wall panels and in the inner section continuous floor panels. The inner section function as platform and facilitates the making of the vertical connection between wall panels. The right figure has in the inner section continuous floor panels and no continuous wall panels. Compression perpendicular to the grain in the outer parts is avoided by placing the wall panels on top of each other and hanging the floor panels between the walls.

Both options in figure 5.36 are suitable for avoiding high compression stresses perpendicular to the grain. However, both options have a major design flaw which is the difference in stiffness between the inner and outer section of the wall. Both options have a stiffer outer section which in time will be carrying most of the loads which as result will overstress the wall panels.

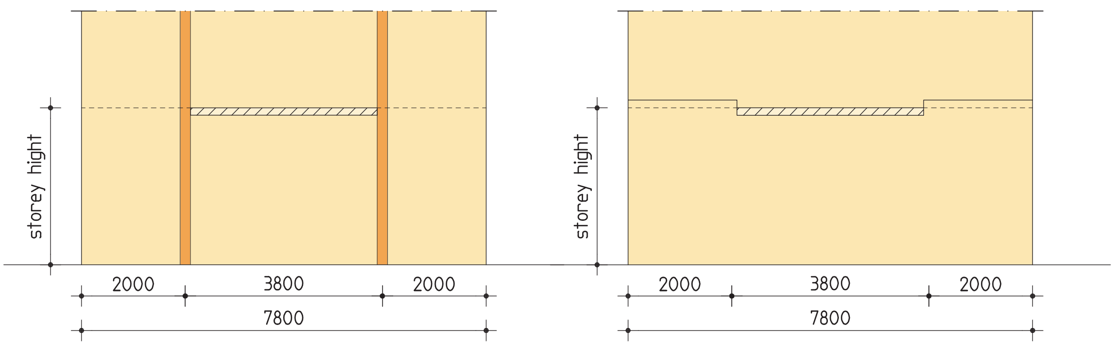


Figure 5.36 Wall sections with combined balloon and platform method

Further developing of both options leads to stacking of wall panels on one another and hanging the floor panels between wall panels, see figure 5.37. The vertical joints are now replaced by horizontal joints and can be prefabricated. The floor panels function as a working platform, while the difference with the platform method is that the floor panels are not placed on top but hanging between the wall panels.

From al the presented solutions, the combined balloon – platform frame construction is the most practical and economical method for dealing with high compression stresses perpendicular to the grain. Therefore this method is used for the construction of the 3rd to the 7th storey.

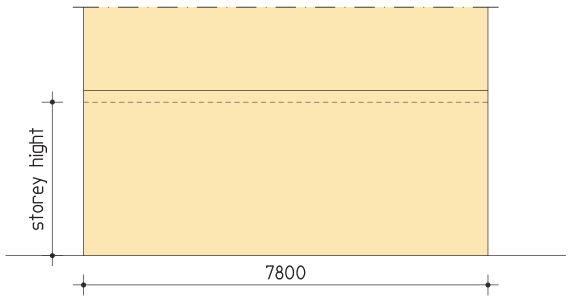
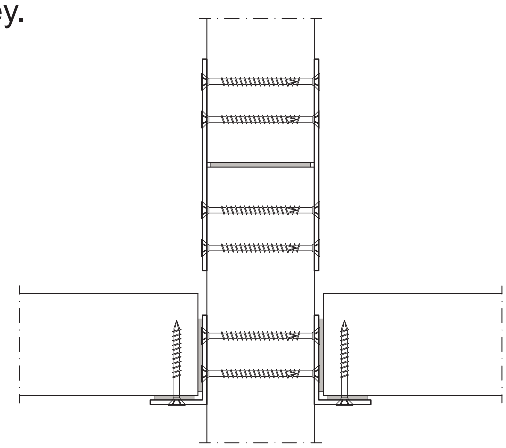


Figure 5.37 Wall section with combined balloon and platform method



Connection detail

5.3.4.4 Connection design

Hold down fixings:

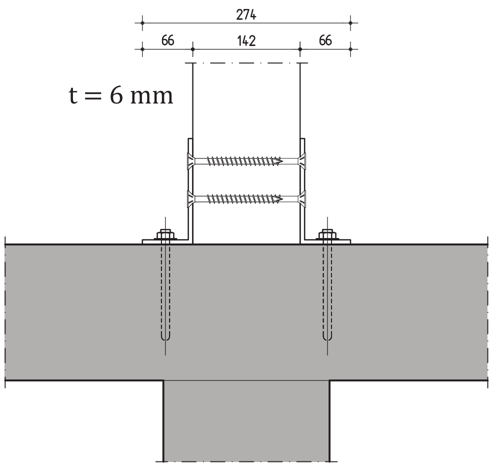
Several connections can be designed for holding down the panels when tension forces occurs. Two different variant are calculated for the third and fourth storey. The first variant is designed using screws and steel angles, while the second variant uses dowels with a steel plate in the centre. The hold down fixings of the platform method are first shown followed by the fixings of the combined balloon – platform method.

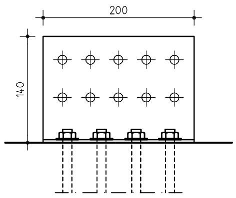
3rd storey,

Tension force: $103{,}47\ \mathrm{kN}$

Variant 1

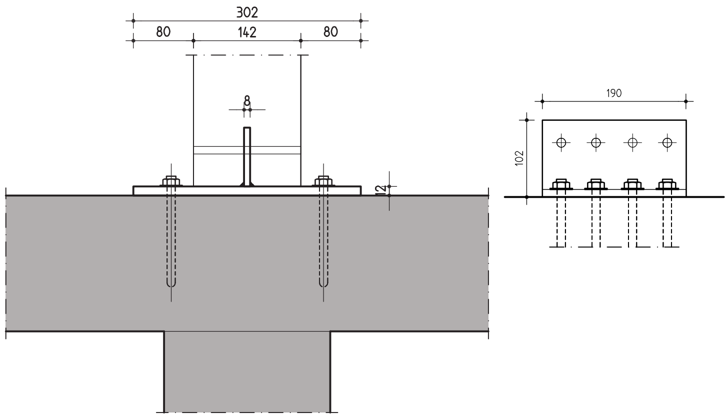
- 20 screws $\emptyset 12$
- 8 anchors $\mathrm{M}12$





Variant 2

- 4 dowels Ø12
- 8 anchors M12

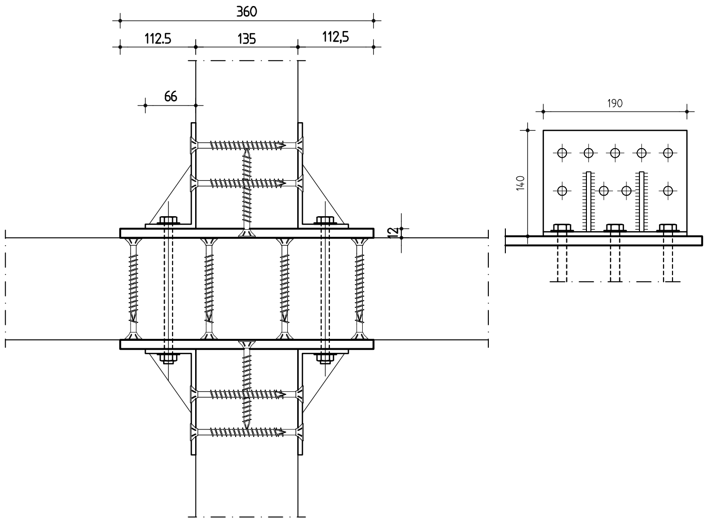


4th storey,

Tension force: 79,08 kN

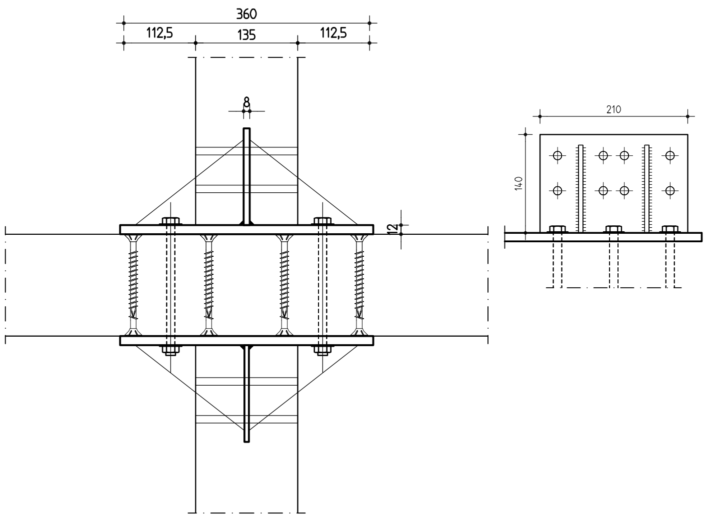
Variant 1

- 18 screws Ø12
- 6 bolts M12
- stiffened steel angle



Variant 2

- 8 dowels Ø12
- 6 bolts M12
- stiffened steel plate



Calculations show that the dowel variant is much stronger than the screwed variant. This can be seen in the amount of screws that are required for transferring the tension forces.

The hold down fixings on the 4th storey are stiffened with steel plates in order to acquire the required stiffness of the fixing. Remarkable is that the doweled variant on the 4th storey is well over dimensioned. Only three dowels are required for ultimate limit state design, but eight are needed to achieve the required spring stiffness of the joint.

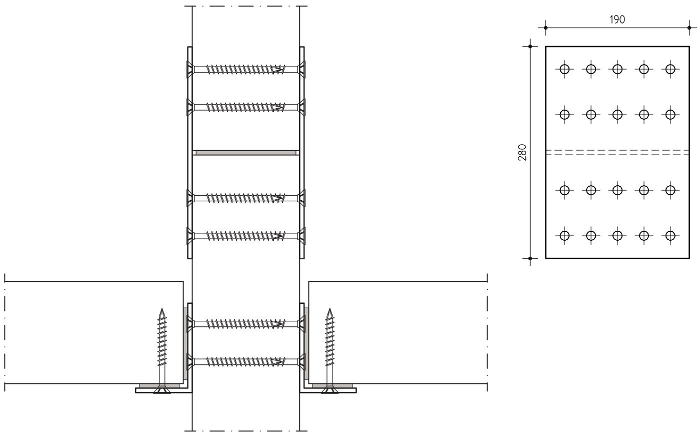
A third variant using bolts can be designed for holding down the panels. The problem, however, is the low stiffness of the connection due to hole tolerances of the bolts. The use of this variant is therefore not suitable for this design.

The hold down fixings in the combined balloon – platform frame method can be designed in a similar way. The difference is that stiffening of the joint is not necessary since the connecting steel plates are infinitely stiff compared to the screws and dowels. The stiffness of the joint is therefore only dependent on the amount of used screws which is similar to the stiffened joints of the platform method.

4th storey,

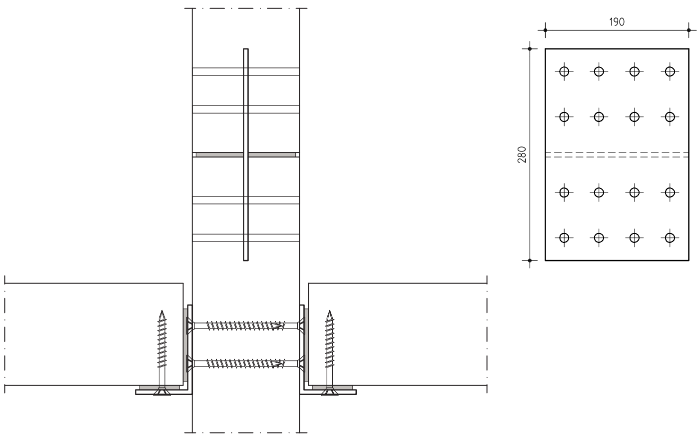
Variant 1

- 18 screws Ø12



Variant 2

- 8 dowels Ø12



No extra measures are required for the fire safety of the fixings. The wind load is factored by 0,3 in the fire design and results in a wind moment that is smaller than the restoring moment caused by the dead load on the panels. This means that no tension force occurs in the panels and therefore no hold down fixings are needed in the fire design.

It can be concluded that the need for high stiffness of the fourth storey makes the fixings of the platform method uneconomical. This stands in contrast with the fixings of the combined balloon – platform method which don't need stiffening, are simple, and inexpensive.

The bearing and stiffness problems are more easily solved by using the combined balloon – platform method. In this way compression high compression stresses perpendicular to the grain in the floor panels is avoided as well as the need for stiff hold down fixings, making the structure more economical. The structure of the Inntel hotel is therefore constructed from the 3rd to 7th storey with the combined method and from the 8th to 12th with the platform method.

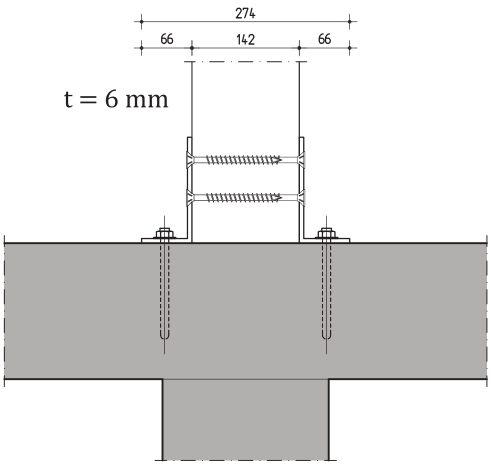
Construction with the combined method is on purpose not continued to the roof. This is done because construction with the combined method requires every element to be placed separate. With the platform method, the floor panels are made continuous which is more efficient and speeds up construction time.

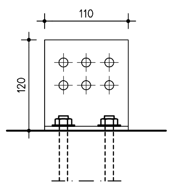
Sliding fixings:

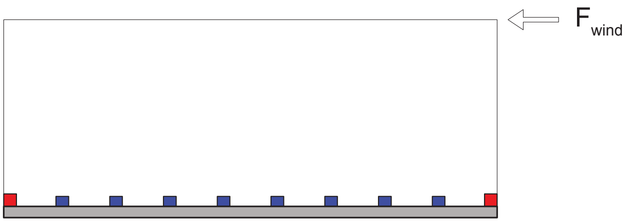
The horizontal wind load is too large to be resisted by the dead load and friction between wall and floor panels. Mechanical sliding fixings are required for resisting the horizontal load. Hold down fixings are not taken into account to contribute with the sliding fixings.The sliding fixings are spread over the length of the wall for an even distribution of the shear force from wall to floor.

3rd storey,

Shear force: $356{,}10\ \mathrm{kN}$
8 sliding fixings
c.t.c. $850\ \mathrm{mm}$



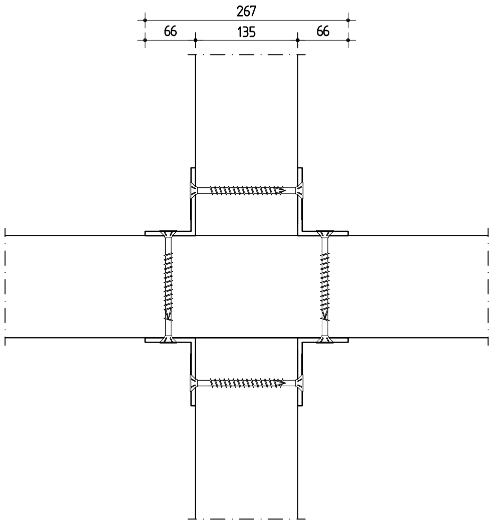


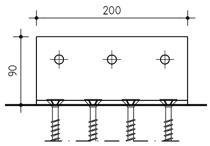


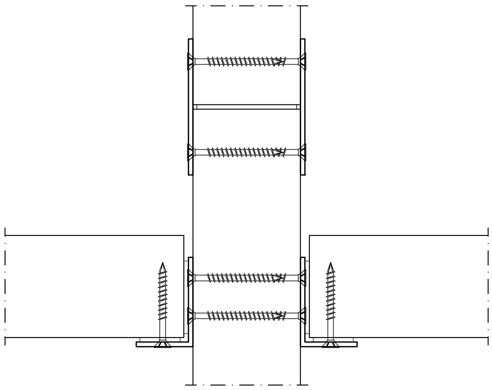
red = hold down fixing
blue = sliding fixing

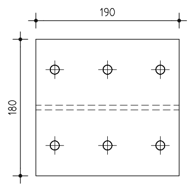
4th storey,

Shear force: $207{,}56\ \mathrm{kN}$
8 sliding fixings
c.t.c. $800\ \mathrm{mm}$

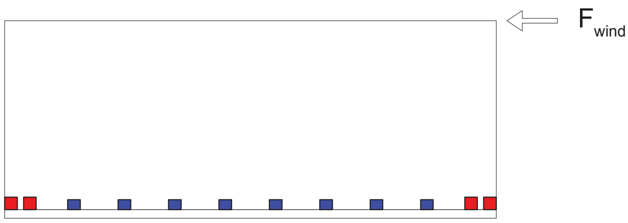








Alternative for combined method



red = hold down fixing
blue = sliding fixing

5.3.4.5 Floor diaphragm

Horizontal loads are transferred to shear walls by floor panels through diaphragm action. The floor plane is in this case loaded in plane and can be considered as a shear beam.

The floor plane must provide equilibrium with the horizontal wind loads in order to transfer the forces to the shear wall and eventually to the foundation. The bending moment makes equilibrium with a tension and compression force. The height of the lever arm between the tension and compression force is conservatively taken as 7,8 metre. This approach is conservative because the lever arm will in reality be much larger than now is assumed.

A steel tie is placed at the perimeter of the floor plane in order to carry the tension force. This steel tie is fixed to the floor panels with screws Ø10 with a centre to centre distance of 600 mm.

The maximum shear force occurs at the shear wall and is included in the design of the sliding fixings in section 5.3.4.4. The next governing shear force is located in the longitudinal joint near the shear wall and is carried by screws in the half lapped joint

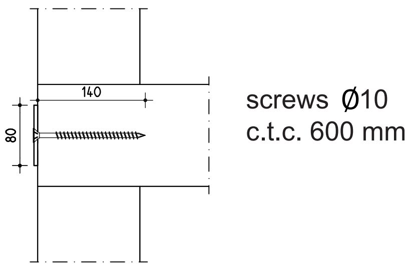


Figure 5.38 Steel tie

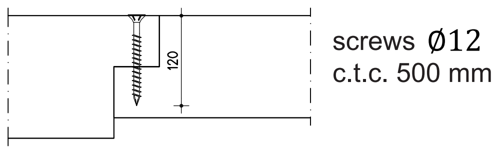


Figure 5.39 Longitudinal joint

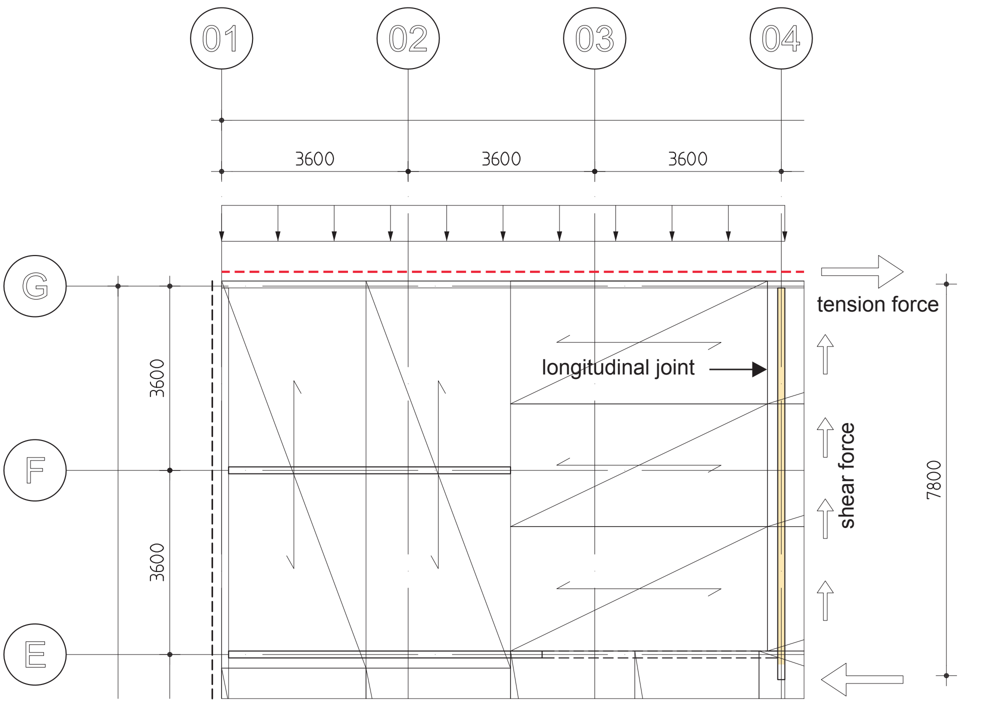


Figure 5.40 Compression arch and tension tie in section of floor plan

5.3.5 Façade design

Governing for the design of the façade is the section on gridline G on the 5th storey. Because of the arrangement of the floor plan, the wall panel in this section carries the highest load. The wall panel is carried by a beam and columns and are also calculated.

5.3.5.1 CLT wall panel design

The end support of floor panels is usually designed as a hinged support which is free to rotate on its supporting wall. In reality, this rotation is often to some degree prevented and a certain fixed end moment results. The wall panel must be verified with this additional fixed end moment.

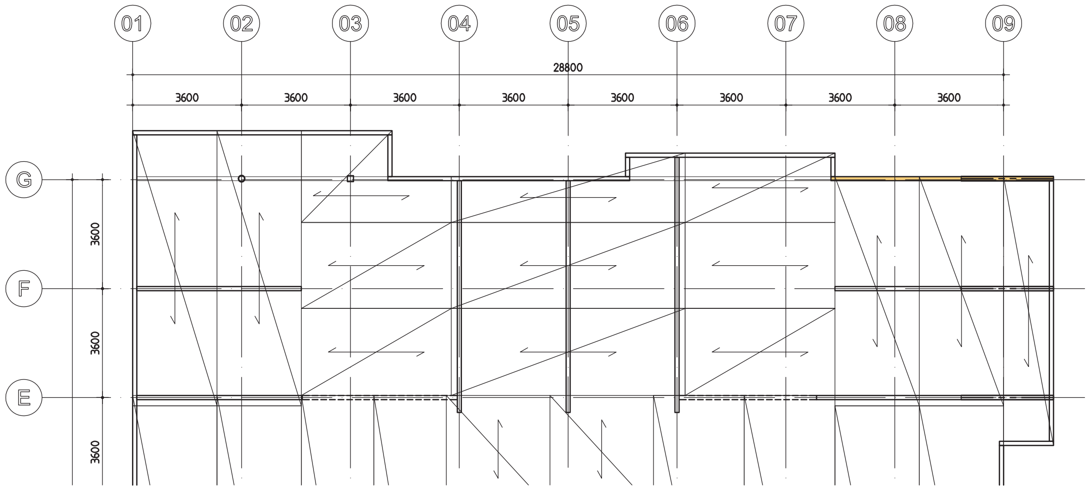


Figure 5.41 Part of 6th floor plan

coloured section is calculated

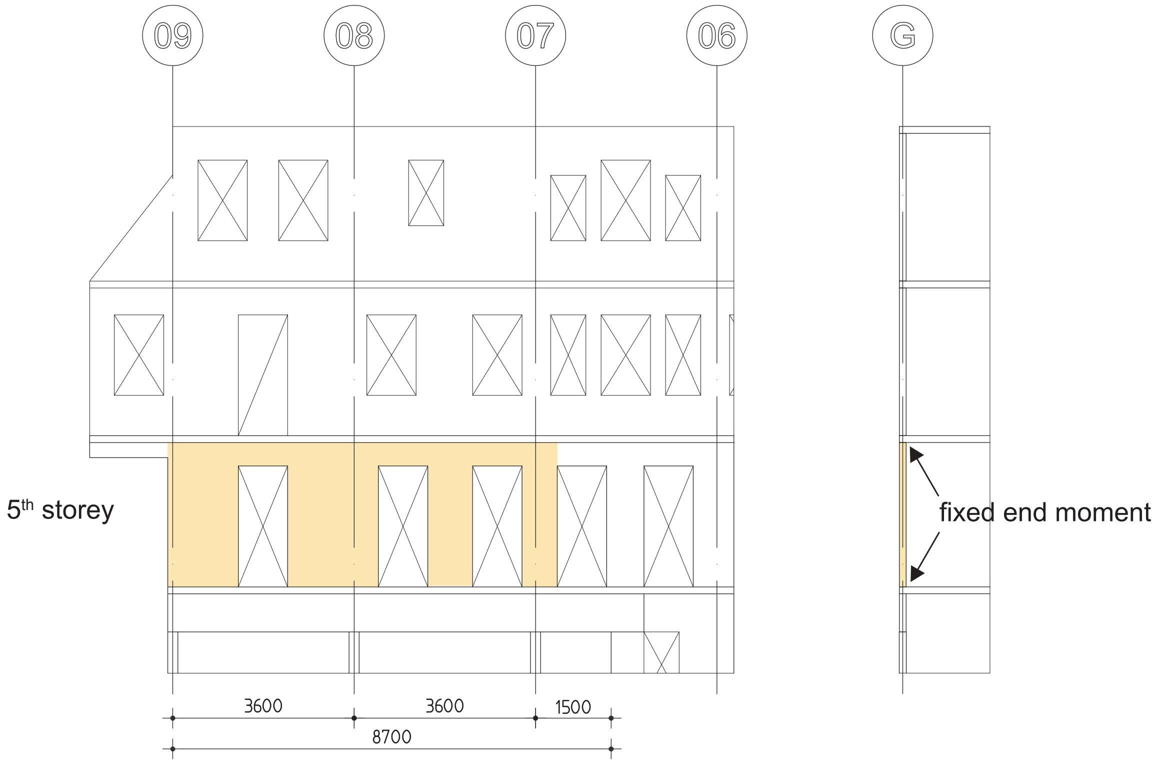


Figure 5.42 Part of facade

The fixed end moment is calculated by assuming 2 springs as support of the floor panel. The stiffness of the support determines the degree of end fixing of the floor panel.

The stiffness of the supports is calculated based on the stiffness of the floor panel. The stiffness of the wall panel is not taken into account since the stiffness of the wall panel is much higher than the stiffness of the floor panel.

From the overview of verifications it can be seen that the fixed end moment has a considerable effect on wall and floor panels. The average stress in wall panels is quite low so the increment in stress is still far beneath the maximum permissible stress of the wall. Different is the case for the floor panel where the fixed end causes large reaction forces resulting in a high compression stress in the panel.

If necessary, the stress can be decreased by placing a high pressure supporting rubber pad (figure 5.44). The rubber enables rotation of the floor panel and decreases the stiffness of the fixed end. In this case, the fixed end moment decreased from 5,26 kNm to 2,11 kNm which is a reduction of almost 60%.

Overview of verifications

Verification	**U.C. with fixed end**	**U.C. without fixed end**
Stability	*0,39*	*0,25*
Bearing resistance	*0,83*	*0,49*
Fire safety: stability	*0,64*	*0,56*

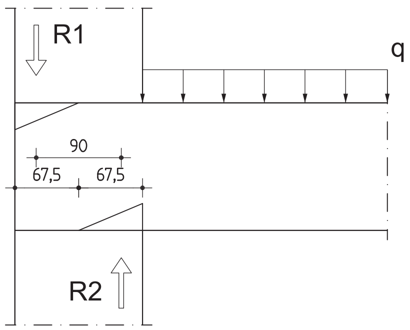


Figure 5.43 Fixed end

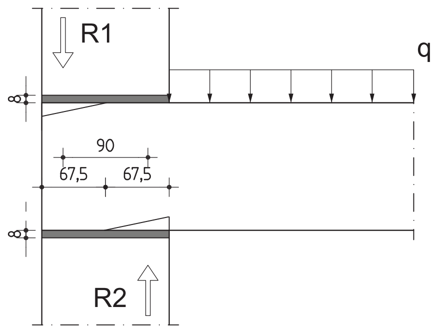


Figure 5.44 Fixed end with rubber pad

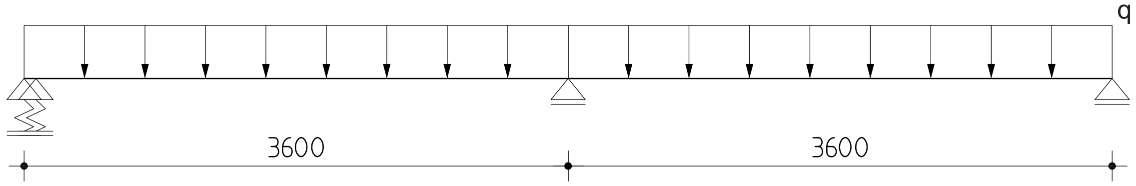


Figure 5.45 Mechanical scheme with fixed end support

4th storey beam and column

The beam and columns on the 4th floor that carry the façade are designed in steel. The shear force in the beam is 330,92 kN and is too large for a timber beam. Therefore a steel HEB 260 is designed.

The columns can in fact be designed in timber. The middle column is calculated with a rectangular cross section 250 * 250 mm with timber in strength class GL24h. The cross section verifies the requirement in normal design but fails in the fire design. The column verifies the failure condition if a higher strength class column or a larger cross section is used. Nevertheless, there is chosen to use the steel circular column of the original design for its aesthetical look. The circular steel column filled with concrete is more slender and nicer shaped than the rectangular timber column which has a total thickness of 310 mm when covered with plasterboard.

The detail of the connection can be seen in figure 5.46.

Overview of verifications

Element	**Verification**	**Unity Check**
Steel beam HEB 260	*Flexure*	*0,69*
	Shear	*0,65*
	Deflection	*0,24*
*Column 250*250 GL24h*	*Stability*	*0,61*
	Fire safety: stability	*1,13*

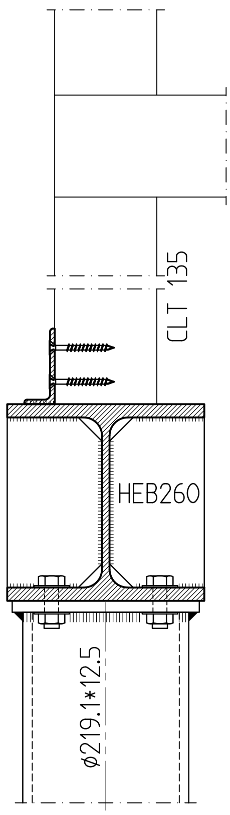


Figure 5.46 Facade connection

5.3.5.2 Protruding sections

The protruding sections in the façade are either situated parallel or perpendicular to the floor span direction. The protruding sections that are situated parallel to the span direction are carried by cantilevered floor panels.

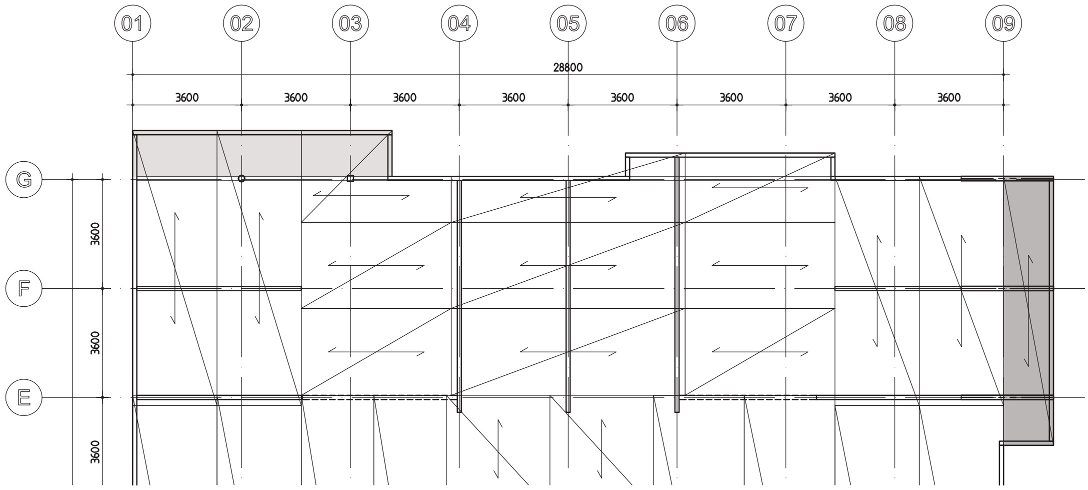


Figure 5.47 Part of 6th floor plan

section parallel to floor span
section perpendicular to floor span

Protruding sections parallel to floor span

The governing design check for the floor panel is the deflection of the cantilever. The deflection of a CLT 135 panel is 24,24 mm while the requirement is 12,8 mm. The deflection of the panel is too large so the bending stiffness of the floor must be increased.

When a CLT 165 panel is used, the increase in height is quite small, 30 mm, while the increase in stiffness is a little more than 80%. Because of the higher stiffness of the panel, the deflection is reduced to 12,7 mm and fulfills the requirement.

The already designed CLT 162 panel has almost the same thickness but differs a lot in stiffness and cannot be used. So for practical and aesthetical reasons, the CLT 162 panels are replaced by CLT 165 panels.

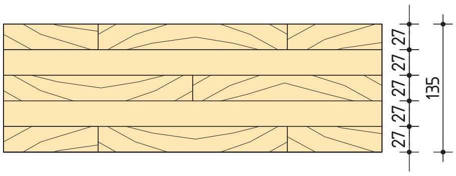


Figure 5.48 CLT 135
Bending stiffness EI = 1760 kNm^2

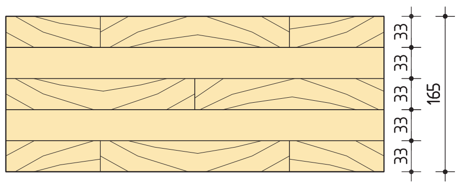


Figure 5.49 CLT 165
Bending stiffness EI = 3190 kNm^2

Protruding sections perpendicular to floor span

In the situation where the protruding sections are situated perpendicular to the floor span, cantilevering of floor panels is not possible. The cantilever is instead designed with beams that are anchored to the wall panels with glued in bolts.

Making the cantilever with timber beams is not possible because a high stiffness is required. The beams are therefore made of steel sections HEB 160 which are anchored on two sides with three glued in bolts, see figure 5.51.

Three load combinations are checked to verify the tension force in the bolts: The construction phase, the use phase and fire safety. The construction phase is governing for the design of the bolts since there is little load on the fixed end which provides for the restoring moment.

The force in the two end bolts is $F_{a,x,Ed} = 6{,}93\ \mathrm{kN}$ and result small compared to the ultimate capacity of the bolts $R_{a,x,d} = 31{,}59\ \mathrm{kN}$. The connection could be optimized but care has to be taken since the bolts behave brittle when failure arise.

Hanging the floor panels to the wall panels above is not advised because of the increased risk of collapse during fire. The reduced cross section of the wall panel provides a small area for screws to be anchored.

Overview of verifications

Element	**Verification**	**Unity Check**
Steel beam HEB 160	*Flexure*	*0,85*
	Shear	*0,27*
	Deflection	*0,64*
Glued in bolt	*Axial*	*0,22*

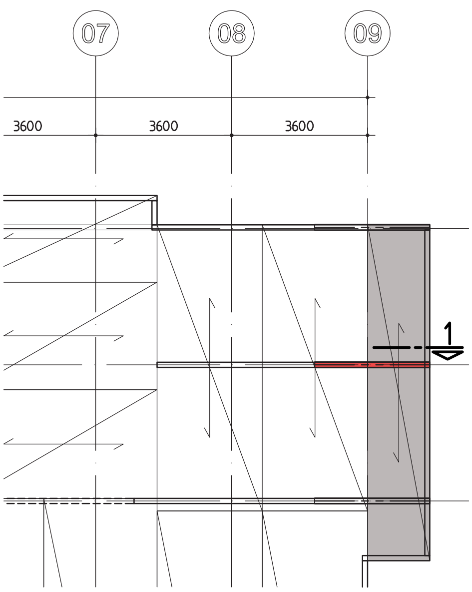


Figure 5.50 Part of sixth floor plan

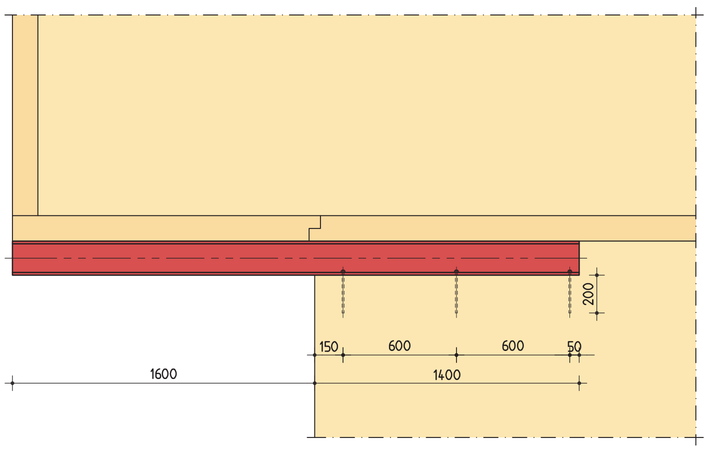


Figure 5.51 Cross section 1

5.3.6 Building weight and deformation estimate

Building weight

The weight of the building from the original concrete and timber design is from the third storey up calculated. This is done in order to compare the difference in building weight of the two building methods. The weight calculation resulted in 65,4 MN for the concrete design and 24,7 MN for the timber design. The timber design is form the third story up 62 % lighter in weight than the concrete design. This amount is significant lower and results in less weight on the foundation.

Vertical deformation estimate

The total deformation of the timber structure is estimated to be 31 mm. Like the Limnologen case study, most of the deformation is caused by the change of moisture content of the building. The change in moisture content causes a deformation of 23 mm, while only 8 mm is caused by long term load effects.

The small share of the deformation due to load effects is favourable because the differential movement between load bearing non load bearing CLT panels is very small and can be neglected. This can be seen in the 4th storey where the largest long term shortening occurs. The shortening of the storey due to load effects is 1,44 mm and will have no significant effect on the slope of the floor.

The total deformation of the building and each storey is not severe and can be accommodated with correct detailing of vertical services, supported cladding and lining materials.

Special attention needs the lift shafts in the building. It is important that the lift shafts are kept free from the CLT structure to allow for differential movement. Lift shafts can be made from concrete, masonry or steel. In each of these cases, the structure carrying the lift will not shorten like the timber. A cavity is therefore necessary between CLT and shaft that can act as a movement joint.

Over time, a difference in height will develop across the threshold between the lift shaft and the floors. This difference must be recalibrated by the lift engineer during maintenance of the lift.

5.4 Summary and comparison

The most important design features are summarised and a comparison is made between the timber and concrete design.

Summary

The timber structure is made from the third storey up of CLT panels using the platform frame construction. The first and second storey are made of concrete because large shear and compression forces occur in the transition from walls to columns which the panels are not able to carry.

All the wall panels have a thickness of 135 mm, except for the stabilizing wall panels on the third storey which have a thickness of 142 mm. The panels on the third storey are slightly thicker because the compression force caused by the dead weight and the wind moment resulted to be too high for the panels with a thickness of 135 mm.

Two main floor panels are used in the building. The panels having a clear span of 3,6 metre have a thickness of 135 mm while the panels with a clear span of 5 metre have a thickness of 165 mm.

The horizontal drift of the structure resulted to be too large so two stabilizing CLT wall panels are added to the design. The addition of the two wall panels alone was not enough. The hold down fixings which prevents the structure from uplift had to be stiffened in order to reduce the extra rotation of wall panels due to slip of the fixings. This stiffening resulted in uneconomical fixings so the wall panels from the 3^{th} to 7^{th} storey are made with the combined balloob - platform method.

The protruding sections in the facade are carried by steel beams Heb 160 fixed between wall panels or by cantilevering the floor panels. The floor panels with cantilever have a thickness of 165 mm in order to achieve the maximum allowable deflection of 12,8 mm.

For fire safety reasons all the timber elements are covered by two layers of 15 mm gypsum fibre board. This is locally deviated by two layers of 15 and 20 mm gypsum fibre boards where floor panels are carried by half lapped joints.

The hold down and sliding fixings are made of steel angles and are vulnerable for high temperatures after failing of the protective gypsum boards. Fortunately the fixings are not necessary in the fire design so no extra fire safety measures are necessary for ensuring the fire safety of the fixings.

Comparison

Roof

The original design of the roof structure of the Inntel hotel consists of a steel frame covered with gypsum boards to acquire a fire safety of 120 minutes. This is not very different for the timber design where a timber frame consisting of beams, columns and CLT panels carries the load. The most significant difference in design is the replacement of the steel wind brace. One single wind brace is not considered robust enough for the timber design, so two CLT wall panels replaces the brace.

Stability

The three stabilizing walls in the numeral direction of the floor plan resulted in a too large horizontal drift of the structure. For reducing the drift, 2 extra stabilizing walls are placed in the design. The addition of the two panels is also the most significant difference between the two designs, see figure 5.53 and 5.54.

Wall and floor thicknesses

The difference in wall and floor thicknesses of the two designs is insignificant. The concrete design needs heavy mass for fulfilling the acoustic requirements which results in large thicknesses of walls and floors. The same is true for the timber design where additional layers of gypsum board are needed for fulfilling the acoustic and fire safety requirements. The difference in sections can be seen in figure 5.52.

Building weight

Even though massive timber panels and a considerable amount of gypsum boards are used in the timber design, the difference in building weight is still very large and clearly in favour of the timber design. The timber design results in a weight reduction of 62% compared to the concrete design which means that a considerable saving can be made in foundation costs.

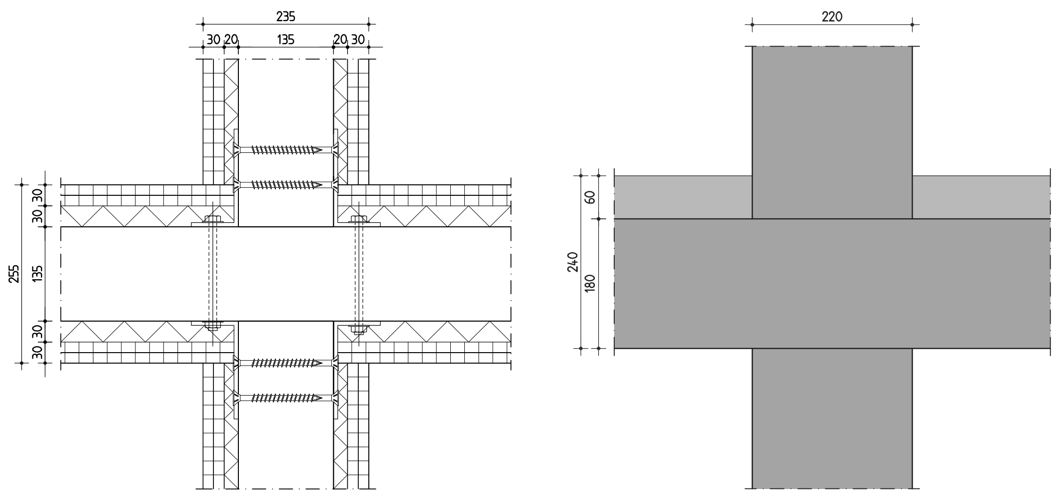


Figure 5.52 Difference in thicknesses

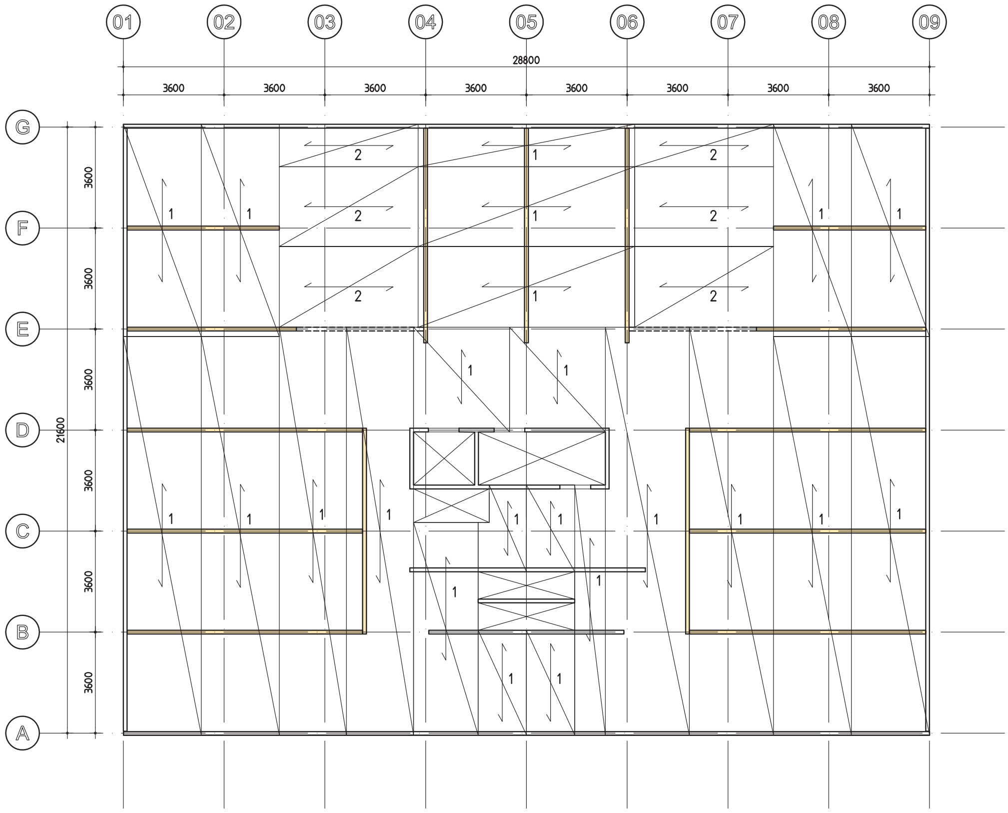


Figure 5.53 Typical timber floor plan
Source: Drawn by the author

1 = CLT 135
2 = CLT 165

All walls CLT 135
Coloured walls are stabilizing

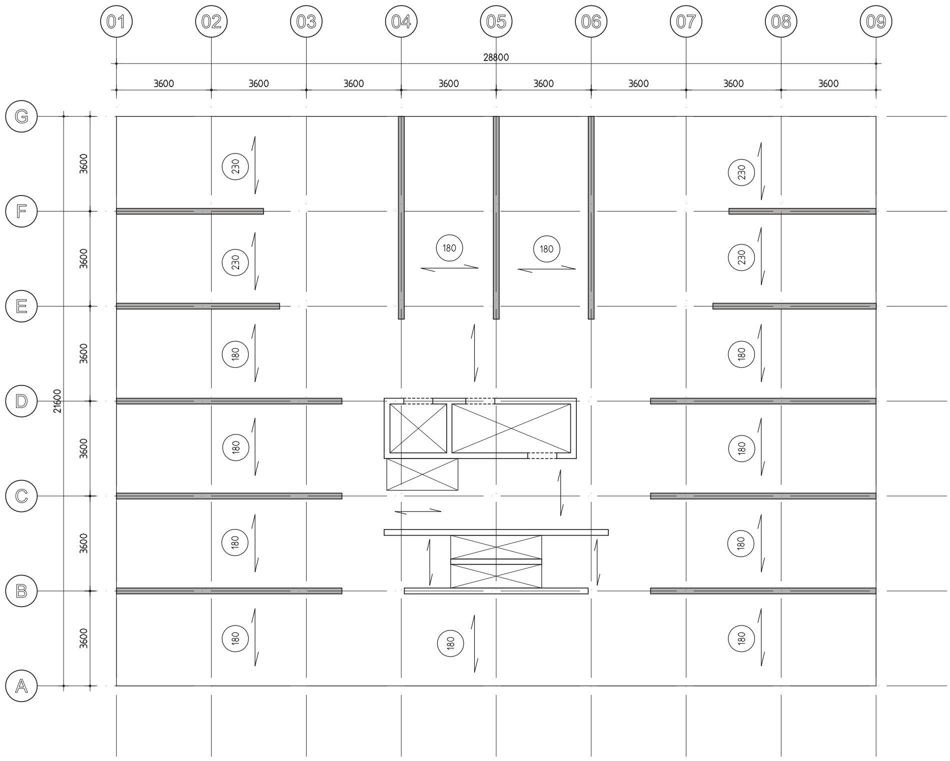


Figure 5.54 Typical concrete floor plan
Source: Drawn by the author

Walls coloured in grey are stabilizing

CHAPTER 6, CONCLUSION AND RECOMMENDATIONS

The main goal of this research study was to examine the technical feasibility of a timber load bearing structure in Dutch medium rise buildings. The most promising structural timber system is chosen and further analyzed. Three timber buildings were analyzed which provided practical knowledge on design and construction of medium rise timber buildings. Finally, a timber load bearing structure is designed for an existing 12 storey building based on the Dutch requirements.

The initially formulated research question was “Is it possible to use timber as a load bearing structure in Dutch ’medium rise’ buildings”. This question can be answered with a yes. The CLT properties, case study and design have proven that a medium rise building in the Netherlands can be constructed with a timber main load bearing structure. Each building is of course different and have different functions. A timber load bearing structure will therefore perform in some cases very well and in other cases less.

Buildings which are most suitable for a timber load bearing structure are buildings which have little deviations in the structure and where enough space is reserved for shear walls, like apartment buildings. Buildings where large open spaces are required, like office buildings, are less suitable for timber construction. The use of timber construction is possible when the following design aspects are taken into consideration.

1 *Control of horizontal drift*

Control of horizontal drift may become governing when few shear walls are incorporated in the design. This will in general not lead to problems with apartment buildings since in most cases many shear walls are present, resulting in stiff structures. Different is the case for buildings which require open floor spaces like for example office buildings. Adding shear walls or using the facade as stabilizing structure will help controlling the maximum drift.

2 *Bearing resistance*

The bearing resistance in buildings using the platform frame construction is another important design issue. Medium rise buildings have to carry high compression forces in the lower stories. These compression forces result from the self weight of the building and horizontal wind loads. The compression forces will result in high compression stresses perpendicular to the grain in the timber which is difficult to carry. Buildings with many shear walls, like apartment buildings, will have less problems with this phenomenon, since the compression forces due to the wind load will be smaller. Changing from construction method, like the balloon frame method, will give good results when high compression forces are to be expected in the building.

3 *Deviations in structure*

Buildings with heavily loaded walls which are supported by columns with wide spacings are not suitable for timber construction. The reason for this is that large shear and compression forces occur in these wall panels which the timber is not able to carry. In these situations a transitional concrete structure must be designed for carrying these forces.

4 *Fire safety*

Timber is a flammable material which means that the timber will ignite during fire and thus must be protected. Dutch medium rise buildings must be designed to have a 120 minutes fire resistance which in most cases will lead to a fire protection consisting of two layers of gypsum fibre board. The fire resistance can be decreased when a different fire safety concept is used, for example an active fire protection with sprinklers.

5 *Shortening of the structure*

A timber structure will shorten over time due to load effects and the reduction of moisture content. The shortening of the structure is especially important to analyse in medium rise buildings since this will have a greater effect than in low rise timber buildings. The shortening may affect other building elements if details are not properly designed to deal with the resulting differential movement.

Recommendations

A medium rise timber building can be improved when the following subjects are studied.

A Transfer as much dead load as possible to the stabilizing structure. This prevents large tension and compression forces in the floor panels. A lower compression force facilitates the verification of the bearing resistance of the floor panels and a lower tension force reduces the need of expensive stiffening of the hold down fixings. If large compression forces does occur, avoid cross grain material by using the balloon frame method.

B If possible, utilize the facade as stabilizing structure. When the facade is not heavily fenestrated, the stiffness of the building will be improved considerably.

C The fire resistance requirement of 120 minutes can be reduced by using active fire safety concepts like sprinklers. The reduction of the fire resistance may lead to considerable savings of over building costs by the reduction of the thickness of fire resistant linings.

D More research has to be done in order to guarantee a well performing acoustic structure.

E More research has to be done considering the building costs of the timber structure. The success of medium rise buildings in The Netherlands requires competitive cost and cost efficient production of buildings which can only be obtained from research and learning.

F The use of timber as the only load bearing material should not be pursued. The use of concrete and/ or steel is in many cases inevitable and will result in a more efficient structure.

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