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Comparison of Different Construction Methods for Multi-Storey Timber Buildings

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Abstract

In this master thesis, different timber materials and construction methods are examined concerning the application in multi-storey timber buildings.

Although wood as a construction material has many advantages, it is today only used very little for larger constructions, concrete and steel dominate the building industry. Planners and engineers are often lacking necessary expertise to utilise timber in a modern, effective way.

In order to promote the development of modern, efficient multi-storey timber buildings, the goal of this master thesis is to contribute to a better understanding of the wood's characteristic properties and the behaviour of timber in larger engineering structures.

For this purpose, first a literature research has been conducted to gather technical knowledge about modern timber methods. It can be concluded that, although wood as a natural material has some unfavourable properties, with an appropriate design, those problems can be overcome. Modern engineered wood products improve the basic material's properties noticeable and are important especially for multi-storey timber buildings.

Secondly, a design has been carried out for three different structural variants of the same existing multi-storey timber building. By means of this design, general conclusions could be made concerning the suitability of those different construction methods. Frame constructions are very effective for providing stability for the overall structure. Prefabricated methods like panel constructions have economic advantages, should, however, only be used in combination with other methods for larger timber structures. Mass timber methods are very well suited for multi-storey timber buildings and will supposedly play an important role in the future.

The main outcome of this master thesis is that there are no large hindrances for the use of wood in multi-storey buildings today and that timber structures still have much unused potential.

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Foreword

During my studies of civil engineering at the Leibniz Universität Hannover (LUH) in Germany and the Universitetet i Agder (UiA) in Norway, there had always been one building material that fascinated me the most, not only because of its exciting mechanical properties, but also because of its long history and tradition, its architectural value and last but not least because of its beneficial environmental properties: wood. The courses about wood as a construction material that I attended were interesting, but I wanted to work more with this material. Moreover, my experiences both at the university and working at an engineering office at the same time showed that concrete and steel still dominate the larger engineering structures today. That was why I decided to write my master thesis about multi-storey timber buildings.

The second decision I made then was to write the master thesis in Norway, which is known for its rich wooden building culture. I had already been at the UiA in Grimstad during by bachelor's studies, this time I returned to finish my master' studies.

Together with my supervisor at the UiA, Katalin Vertes, I developed the task and the research questions for the master thesis. The work included an extensive literature research to get up to date concerning the modern timber engineering, as well as the redesign of an existing multi-storey timber building using three different construction methods. The choice of the building was quite natural – after doing some research, I decided to work with the highest wooden building of today, Treet in Bergen, the second largest city in Norway ("Treet" is the Norwegian word for "the tree"). Especially the design of the three variants of this building was challenging, but it helped me to make some important experiences that one does not learn in any lecture, e. g. concerning the development of a structural concept.

I could not have completed this work without the help from different people. Most of all, I want to thank my supervisor Katalin Vertes. She had been the first person to teach me about wood when I had been at the UiA for the first time in 2014 and 2015. Now she was the person to guide me while working at this master thesis. In spite of having to travel a lot, she was always reachable and assisted me with helpful advise or just reassurance when I had doubts.

Another thank you is also meant for my friends and my family both in Norway and in Germany, for their support and for helping me to clear my head when I needed it.

Finally, I thank you, the reader, for being open minded about this work, hoping to be able to contribute to your knowledge about timber or at least to give you an interesting reading experience.

Lucas Bienert

Grimstad, 30.11.2018

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Abbreviations and Symbols

Abbreviations

3D	three-dimensional
CAD	computer-aided design
CLT	cross-laminated timber
CNC	computerised numerical control
CO	load combination (followed by the corresponding number)
DIN	"Deutsches Institut für Normung" (German Institute for Standardization)
EC	Eurocode
EN	European Standard
EQU	equilibrium limit state
FE	finite element
FEM	finite element method
FSC	Forest Stewardship Council
GEO	geotechnical limit state
GHG	greenhouse gases
glulam	glued laminated timber
LC	load case (followed by the corresponding number)
LCA	life cycle assessment
LVL	laminated veneer lumber
MBO	"Musterbauordnung" (German building regulation)
M-HFHHolzR	"Muster-Richtlinie über brandschutztechnische Anforderungen an hochfeuerhemmende Bauteile in Holzbauweise" (German regulation for fire safety requirements on timber elements)
OSB	oriented strand board
PEFC	Programme for the Endorsement of Forest Certification
PSL	parallel strand lumber
SLS	serviceability limit state
STR	structural limit state
TMT	thermally modified timber
ULS	ultimate limit state
VOC	volatile organic compounds

Latin Symbols

a	years
c	compression
CO_2	carbon dioxide
E	Young's modulus
f	strength
f_c	compression strength
f_t	tensile strength
G	permanent loads
h	hours
k_{mod}	modification factor for the strength of wood
Q	variable loads
SL	stress level
t	tension
T	time, load duration
u	ultimate
y	yield point

Greek Symbols

γ	specific weight
γ_G	partial safety factor for permanent loads
γ_M	material safety factor
γ_Q	partial safety factor for variable loads
ε	strain
λ	heat conductivity
σ	stress
σ_p	reduced preliminary design strength
ψ	combination factor

1. Introduction

Although wood is one of the oldest materials that have been used for buildings, today it has mostly been replaced by concrete and steel. This master thesis deals with different construction methods that can be used for multi-storey timber buildings [1] to analyse the wood's potential in this field, also in comparison to concrete and steel.

One of the main motivations to use wood is that it is renewable and environmentally superior to those other materials. Globally, the construction industry is responsible for 40 % of the total depletion of natural resources, 40 % of the consumption of the total primary energy, 15 % of the usage of fresh water, 25 % of all waste and 40 to 50 % of all greenhouse gas emissions [24, 39]. Using wood instead of concrete and steel has therefore great potential to promote a more sustainable society.

It is today no technical challenge to build small two- or three-storey houses from wood. But the global population increases, and a considerable part is moving to the large cities, where living space gets scarce. Therefore, the majority of new buildings will be erected in the cities and will probably be multi-storey constructions. For timber to really make a change, the challenges regarding multi-storey timber buildings must be addressed.

The goal of this master thesis is to contribute to this development by discussing the properties of modern wood products and structures and by analysing the suitability of timber methods for multi-storey buildings. As a method for the analysis, a design will be performed of three different alternative timber structures for an existing multi-storey timber building.

First of all, the society perspective shall be described in chapter 2, also looking at the different situations in Norway and Germany. This will start with a summary of the historical development of wooden buildings and construction methods. After that, the situation of today is presented. In chapter 3, the theoretical background shall be established. Here it is important to take a look at the rules and standards that must be followed in the design. Based on that, the basis for the following design can be created. After defining the central research questions for the following work in chapter 4, the different timber structures, materials and connections that are available today or are being developed will be examined in chapter 5.

Finally, the above mentioned method, the design analysis is presented in chapter 6. Today's world's highest timber building, Treet [2] in Bergen (Norway) will be redesigned to evaluate three different construction methods. Those three variants will be a modern frame construction featuring a large grid, a panel construction relying heavily on prefabrication, and a mass timber construction using cross-laminated timber (CLT). A preliminary design is conducted for all three constructions. This includes shaping the structural idea and determining dimensions of cross-section and connections. This part of the design will be given the most attention, because this is where the most engineering problems must be solved and the general structure is shaped. Based on the

[1] In the context of this master thesis, a multi-storey building is considered to be a building with at least three, but also more than ten storeys. The term timber building refers to a building with the load-bearing structure made from timber or engineered wood products. While the word wood is a more general term that describes the basic material extracted from trees, timber is any kind of wood product used for building purposes.

[2] The construction of Treet was finished in 2015 and it is by today (2018) with 14 storeys and a height of about 51 m still the highest timber building in the world [10]. However, there is another timber building under construction in Norway that will be even higher, Mjøstårnet in Brumundal. After the projected completion of the project in 2019, it will have 18 storeys with a total height of 81 m [22].

preliminary design, the final design according to the Eurocode standards is performed, including a finite element (FE) analyses. The idea is to design three different constructions for the same building and to compare them afterwards considering different aspects such as the efficiency of the load-bearing structure or environmental aspects, cf. chapter 7.

After a discussion of the results in chapter 8, the final conclusions of this master thesis are presented in chapter 9, where the earlier stated research questions will be answered. Based on those conclusions, recommendations concerning the future of multi-storey timber buildings can be given in chapter 10.

2. Society Perspective

2.1. Historical Background

In early history, timber was the most important building material for residential buildings. One of the main reasons for that is that it was readily available in large quantities, although this differed from region to region. Accordingly, some countries have a rich history of building with timber today, while others rely more on other materials. Moreover, wood is relatively easy to shape, in contrast to e. g. stone.

The world's oldest still standing timber building is the Hōryū temple in Japan, which was built in the 7. century [7]. The oldest timber building in Norway is the Borgund church from the 12. century [26, 1]. These examples prove that timber buildings are very durable and can persist for a practically unlimited amount of time if planned and maintained correctly. Important elements of a good design concerning the durability are that water and moisture are not hindered from leaving the building and that damaged components can be replaced easily [26, 2].

In Norway, wood has been the most important material for all kinds of tools, including ships and buildings, since the time of the Vikings. That is because of Norway's wide-spread forests. Wooden houses are an important part of the Norwegian culture, only in a few countries like the USA and Canada is the share of timber building in all existing buildings comparably high. Today, 75 to 80 % of all newly built residential buildings in Norway are timber buildings [18, 7].

The first wooden houses in Norway were probably palisade buildings made from logs driven into the earth vertically, but this kind of structure did not prevail because of the decay of the wood in the ground. Further development lead to stave buildings like the well-known Norwegian stave churches. A different method that was used from early in the history is the log construction, which was widely used until the 19. century. Over the time, more methods were developed, leading to more efficient building constructions that needed less material and were more durable. An example is the traditional frame construction which was introduced around 1700 not only in Norway, but also in other European countries. In the beginning, the space in-between the frames was filled with e. g. brickwork to seal the house against the weather. Around 1900, the buildings got sheathing both on the inside and the outside and the spaces in the framework were left free, which saved much material, i. e. unnecessary weight and costs. In the middle of the 19. century, those cavities were then filled with insulation, which improved the indoor climate. Today, the development of new methods is still ongoing. Prefabrication becomes more and more important and new technologies as e. g. compound materials are used [18, 7-10].

In the early history, Germany was also covered with rich forests. In contrast to Norway, however, deforestation and industrialisation developed faster and stronger, possible reasons for that are the easier topography and Germany's central position in Europe. During the industrialisation, wood was more and more replaced by steel and later also reinforced concrete, because the wood working processes were too slow and therefore more expensive [20, 6]. This was mainly due to labour-intensive connections such as dovetail connections. Furthermore, the timber industry was more focused on traditions, also because of its long history [25, 5], and could therefore not compete with the modern steel and concrete industry.

During this time, the way people regarded the different building materials changed. Only brick houses were regarded as sufficiently durable, wooden houses were especially short-lived and the fire risk was higher [25, 6]. Many of these views of timber survived until today, although the timber technology changed fundamentally.

In the beginning of the 20. century, during the first world war, the timber industry developed strongly again. Many constructions were needed for the war, many houses were destroyed, and steel was needed otherwise [25, 6]. Fast panel constructions were preferred [25, 9]. After the first and second world war, again mostly brickwork and concrete structures were used, which were faster and cheaper to build than wooden structures.

Today, about 10 % of the existing constructions feature wood as main building material in Germany, in the Nordic countries the share amounts to 80 to 85 % [24, 408], which is probably mainly due to the different historic development. The technologies that are used today, however, do not differ as much between the countries because of easy exchange of technical knowledge.

2.2. Evolution of Timber Technologies

While the previous chapter focused on the general development in Germany and Norway, this chapter deals with the development of the timber technologies themselves. It is important to know how the timber technologies changed over time to have a better understanding of today's technologies. Furthermore, it can help to better understand the doubts that exist amongst both the clients and the architects, planners and engineers.

The oldest wood product is, of course, timber made from solid wood, which was used almost exclusively until the 20. century. Nonetheless, there were some early inventions that combined individual wood members to form bigger components that could span larger distances. When those early engineered wood products were developed, the individual members were connected using mechanical fasteners such as nails, bolts or dowels. One of the earliest examples is an arch component made from two to three curved planks that stand upright and are linked together by cross-members, developed by a French architect as early as 1561. It allowed for longer spans, with at the same time lower horizontal shear forces at the supports, compared to the traditional couple roof. In 1830, another French engineer developed a beam made from stacked horizontal planks hold together by bolds, which can be seen as the predecessor of the modern glued laminated timber (glulam) [20, 10].

The mechanical fasteners used for the engineered wood products were widely replaced by synthetic glues in the early twentieth century [26, 7], those allowed a superior stress distribution and better load-slip-behaviour. The founder of modern glued wood products such as glulam was Otto Hetzer (1846–1911), who obtained his patent on glued timber constructions in 1906 [26, 67]. Some of the earlier inventions that used synthetic glues, developed in the first half of the 20. century, include plywood, a panel made from thin layers of veneer, and particle board, a panel made from fine wood chips and sawmill shavings [26, 84]. Later, glulam became commercially available. Some of the oldest buildings still in use that are made featuring glulam as the main bearing element are the railway stations in Malmø (built in 1922) and Stockholm (1925), both located in Sweden [26, 67]. More recently developed products include oriented strand board as well as the solid wood-like parallel strand lumber and laminated strand lumber, all made from strands of wood [26, 84]. Those developments made it possible for wood to be used in large and complicated engineering structures.

Over the time, the planning was more and more moved from the construction site to the office. More complex and efficient technologies required more pre-planning.

In the 21. century, the development was characterised by new technologies like CAD (computer-aided design), CNC (computerised numerical control) and the so-called industry 2.0. This allowed for the wood products to be manufactured very efficiently, even costumer specific products could be produced economically. This again helped the planners to design more efficient buildings with specially tailored wood products. Additionally, making use of the manifold possibilities to shape timber and engineered wood products opened up great new possibilities for architects and engineers.

Today's research focuses amongst other things on improvements of the wood's properties, since herein still lie the biggest problems and challenges. This concerns e. g. the decay of wood and its comparably low stiffness. More about today's research and developments will be described in chapter 5.4.

2.3. Today's Global Situation

With the development of new technologies that make wood competitive again, the demand for wood products rises. There is, however, a gap between demand and (sustainable) supply of wood, one of the most important issues of today is therefore the use of wood from sensitive ecosystems, e. g. rain forests, in a non-sustainable way. The world's total forest area decreased in the time between 1990 and 2000 by 8,3 million ha/a and from 2000 to 2010 by 5,2 million ha/a, most losses were observed in the tropical regions in South America and Africa [24, 314].

Another issue is the conflict between the industrial use of forests to produce wood products and the value of forests for recreation and for a healthy ecosystem. Forests are not only very important for the health of the local ecosystems, but also for the global environment and climate.

To dissolve those conflicts, measures must be taken at different points. On the production side, sustainable methods must be established worldwide, reforestation and forest plantation are important parts of this measure. The development of new technologies for harvesting and manufacturing leads to a higher efficiency and thus a reduced wood demand. Finally, new products that use smaller diameter and lower quality wood can slow down the rising demand for wood [26, 81-82], some examples for this approach are addressed in chapter 5.1.4 about engineered wood products.

To promote sustainable forestry, certification is needed. On a global scale, there are two different systems for certification of forests, the Forest Stewardship Council (FSC) and the Programme for the Endorsement of Forest Certification (PEFC). The latter is today's biggest certification system [27, 18-19]. Additionally, there are also national certification systems like the Norwegian "Levende Skog", this will be taken up again in chapter 3.1.2.

In spite of all the issues described above, wood will definitely play an important role in the future. With appropriate management, those problems can be overcome, because wood is still sufficiently available. In Europe, the wood growth exceeds the felling each year, and consequently, the forest area increased by 0,7 million ha/a during the period from 2000 to 2010 [24, 314]. In Germany, one third of the current annual yield of wood would suffice to use wood for all newly built constructions [23, 43] and in Norway, the new growth of wood per year is more than twice as high as the usage [27, 25].

2.4. Summary of the Society Perspective

The purpose of this chapter was to give an introduction into building with timber from a society point of view. It started with a summary of the historical development, which showed that wood has been used for building purposes for centuries, longer than e. g. concrete and steel, but the technologies have changed significantly. Large-scale structures have become possible. Moreover, former problems concerning the combustibility and the decay of wood have largely been overcome. Old wooden buildings like the Norwegian stave churches prove that wooden buildings can be very durable, provided good planning and maintenance. Still, there are wide-spread reservations against timber amongst both the clients and the planners, and the timber industry is only now starting to develop.

Today's global situation was described, pointing at the environmental issues connected to the production of wood products concerning unsustainable forestry.

Before dealing in detail with the different timber technologies, some of which have already been mentioned in this chapter, it is necessary to first take a look at the theoretical background in the next chapter. This includes the international and national rules and standards that form the basis for any timber design. Of central importance for all European countries are the European standards including the Eurocodes (EC). After that, a description of selected national rules and standards in Norway and Germany will be given.

The design basis, which is based on those rules, will complete the theoretical background for the subsequent examinations of the timber technologies and the design analysis.

3. Theory

3.1. Rules and Standards

3.1.1. European Standards

Many different standards play a role in the design of timber buildings. The design is conducted according to the Eurocode 5 (EC5), i. e. EN 1995-1-1. Additional standards regulate the different materials, give requirements for manufacturers and define material properties. Furthermore, there are some products that are not or not yet standardised, where the characteristic properties are specified by the manufacturer or a testing laboratory commissioned by the manufacturer. Table 3.1 gives an overview over the European standards that are important for the design of timber constructions.

Table 3.1: Overview over European standards for the design of timber constructions

product	main standard	product standard	properties standard	design standard
solid wood	EN 14081-1	EN 14081-1	EN 338	EN 1995-1-1
glulam	EN 14080	EN 14080	EN 14080	
laminated veneer lumber	EN 13986	EN 14374 + EN 14279	(manufacturer)	
plywood		EN 636	EN 12369-2	
oriented strand board		EN 300	EN 12369-1	
particleboard		EN 312		
hard fibreboard		EN 622-2		
medium hard fibreboard		EN 622-3		
medium dense fibreboard		EN 622-5		
solid wood panels [3]		EN 13353	EN 12369-3	-
cross-laminated timber (CLT)	EN 16351	EN 16351	(manufacturer)	-

As can be seen from this table, for some products like laminated veneer lumber (LVL) and cross-laminated timber (CLT), no properties standards have been worked out yet, so that characteristic properties must be taken from technical approvals provided by the manufacturer. Moreover, the EC5 does not mention either solid wood panels or CLT. The design rules of the EC5 must therefore be adapted for those materials, that concerns mainly the choice of adequate safety factors. This will play a role later on in this master thesis, when it comes to the design the CLT construction of Treet. It can be revealed at this point that, based on research results, it is recommended to use the same design factors for CLT as for glulam [19, 934].

In addition to those timber specific standards, Eurocode 0 (EC0) describes the general design concept, and in the different parts of Eurocode 1 (EC1), all the types of loads are defined. It is not

[3] Solid wood panels are panels made from one or several plies of wood planks; they can have the same composition as CLT panels, but in contrast to CLT the planks have not been sorted into strength classes and the glue has not been tested.

within the scope of this master thesis to go into detail concerning those basic standards. At some points, though, the standards allow e. g. the use of different formulae. Moreover, every country can specify their own set of safety factors and combination factors and can adapt the standards within specific boundaries in the corresponding national annexes. To avoid misunderstandings and to have a solid foundation for the EC design (cf. chapter 6.4), design basis is established in chapter 3.2.

To give some examples of the differences that arise from the different national annexes, a short comparison of the Norwegian and the German national adjustments shall be presented.

Quite naturally, there are big differences concerning the determination of the environmental loads like wind and snow loads. This is mostly due to the fact that the environmental loads highly depend on the geographical situation in the different countries. The Norwegian national annexes to EC1 part 1-3 (snow loads) and EC1 part 1-4 (wind loads) feature detailed tables with factors for the calculation of the loads at different places. In the German national annex to EC1 part 1-4, an accidental load case [4] for especially high wind loads in the flat northern regions is added.

Concerning the timber specific standard, EC5, the German national annex features extensive additions, e. g. a simplified approach for the design of timber connections using $\gamma_M = 1{,}1$ as partial safety factor instead of $\gamma_M = 1{,}3$. Furthermore, one can find additions for different details that are not covered by the main standard, e. g. for strengthening of transverse connections using fully threaded screws and for traditional woodworking joints.

3.1.2. National Rules and Standards in Norway and Germany

Besides the described standards which define the technical requirements, additional national rules define further requirements, concerning e. g. the layout of the building, fire safety measures and the execution of the building project.

In Germany, that concerns the "Musterbauordnung" [9] (MBO, German building regulation) together with the timber specific "Muster-Richtlinie über brandschutztechnische Anforderungen an hochfeuerhemmende Bauteile in Holzbauweise" [8] (M-HFHHolzR, German regulation for fire safety requirements on timber elements). The fire safety requirements for wood products are quite strict, making it necessary to make a deviation request if timber is to be used visibly in buildings with more than three storeys [5]. This leads to higher costs for timber constructions.

In Norway, it is only allowed to use timber in buildings higher than three storeys at all since 1998. In a research document commissioned by the Norwegian government in 2013, it is found that, as a long-term effect of that, the timber industry is still not fully developed and that expertise is missing [13, 4]. This may sound strange considering the rich history of building with timber which was described previously. The problem, however, is that wood is almost exclusively used for smaller residential buildings, but not for larger structures. The strategy of the Norwegian government aims at using more wood for public buildings, thereby supporting the timber industry [13]. This measure could definitely also be transferred to Germany, where the situation of the timber industry is approximately the same, if not worse. But a comparable strategy has not yet been worked out.

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[4] The design according to the EC standards is based on the analysis of different load cases with specific combinations of the different loads. Accidental load cases are reserved for exceptional loads like fire or earth quakes.

[5] Actually, the requirements are linked to the height of the ground floor of the highest inhabited storey above the ground. The limit is set to 7 m, which corresponds approximately to a building with three storeys.

A different aspect, that has already been mentioned earlier in the description of today's global situation, is forest certification. Norway has its own forest certification system "Levende Skog", which fulfils the requirements for the PEFC certification. It was founded in 1998 and aims at the promotion of a more sustainable use of the forests, balancing the three central aspects industrial and economical interests, environmental issues and social interests (cf. chapter 2.3). In the "Levende Skog" system, the forest industry, labour unions, recreation organisations and environment organisations work together. Today, all Norwegian forests are certified after this standard [27, 19].

3.2. Design Basis

3.2.1. Parameters

The preliminary design was performed using simplified loads (see next chapter) and reduced material properties to account for the necessary safety margins, e. g. $\sigma_p = 5\ \mathrm{N/mm^2}$ for the strength of wood parallel to the grain (both bending, compression and tension). This allows for a relatively quick preliminary design with results that generally are on the safe side. The preliminary material properties are summarised in the preface of the preliminary design documentation, which is attached to this document (see appendix A).

For the EC design, the material properties were taken from the relevant European standards, which were summarised in Table 3.1. Since the designed building is located in Norway, the Norwegian national annexes of the EC were used.

According to the Norwegian rules, every structure must be assigned a reliability class. Residential buildings generally belong in reliability class 2. This means that the building has to be assigned to the planning control class PKK2 and the execution control class UKK2, which require a detailed quality control [4, NA.A1.3.1(901)-(904)].

Because of differences between the recommendations in the main EC standards and the Norwegian national annexes, Table 3.2 to Table 3.4 summarise the safety and combination factors that have been used in this master thesis, which correspond to the Norwegian national annexes.

Table 3.2: Selected material safety factors [3, NA.2.4.1]

material	material safety factor γ_M
solid wood	1,25
glulam, CLT	1,15
plywood	1,15
connections	1,3

Table 3.3: Partial safety factors according to EC and the Norwegian national annex [4, NA.A1.3.1]

load	favourable/ un-favourable	partial safety factor	
EQU: basic load combination for permanent and transient loads			
G_j: permanent loads	uf	γ_{Gj}	1,20
	f	γ_{Gj}	0,90
Q_1: leading variable load ($i = 1$)	uf	$\gamma_{Q,1}$	1,50
	f	$\gamma_{Q,1}$	0
Q_i: further variable loads	uf	$\gamma_{Q,i}$	1,50
	f	$\gamma_{Q,i}$	0
STR, GEO: basic load combination for permanent and transient loads			
G_j: permanent loads	uf	γ_{Gj}	1,20
	f	γ_{Gj}	1,00
Q_1: leading variable load ($i = 1$)	uf	$\gamma_{Q,1}$	1,50
	f	$\gamma_{Q,1}$	0
Q_i: further variable loads	uf	$\gamma_{Q,i}$	1,50
	f	$\gamma_{Q,i}$	0
EQU = equilibrium limit state STR = structural limit state GEO = geotechnical limit state			

Table 3.4: Selected combination factors according to EC and the Norwegian national annex [4, NA.A1.2.2]

load	combination factor		
	ψ_0	ψ_1	ψ_2
live loads			
A: residential buildings	0,7	0,5	0,3
H: roofs	0	0	0
snow loads	0,7	0,5	0,2
wind loads	0,6	0,2	0

It must be noted that, since CLT is not considered in the EC, assumptions had to be made for the selection of appropriate safety factors and other parameters. In a research project from 2018, it is recommended to use the same safety factors as for glulam (cf. Table 3.2) [19, 934]. Accordingly, other factors like the modification factor k_{mod} to account for effects of moisture and load-duration, were also taken over from glulam.

For the calculation of the deformations, EC5 allows two different methods, a general method and a simplified method for structures that consist of components that all have the same creeping behaviour. The three design variants that are analysed in this master thesis fulfil this requirement (cf. chapter 6.2.1), therefore the simplified approach is applied for the EC design (see [3, 2.2.3]).

All other parameters are described in the documentation of the EC design, cf. appendix A.

3.2.2. Load Calculation

For the preliminary design, simplified loads were used to get a quick idea of the general magnitude of the resulting forces. The self-weight of the floors, for example, was assumed to be 2 kN/m^2, the life load and the snow load on the roof were both set to 2 kN/m^2, too. Only the wind loads were calculated in more detail, because they have a strong influence on the system of a multi-storey building and highly depend on the location of the specific project.

For the EC deign, a detailed load calculation was performed according to EC1, using the Norwegian national annexes. The documentation of the load calculation can be found attached to this document, cf. appendix A. Different load cases (LC) had to be distinguished, those are summarised in Table 3.5.

Table 3.5: Load cases

LC	description	load duration
1	self-weight	permanent
2	snow	short-term
3	wind from the front	instantaneous
4	wind from the side	instantaneous
5	live load (category A)	medium-term

The individual load cases were combined in different load combinations (CO). Because the load bearing behaviour of wood is time depended (see chapter 5.1.1), different load cases with different load durations had to be considered, cf. Table 3.6.

Table 3.6: Load combinations

CO	self-weight	leading variable load	accompanying variable load	load duration
1	self-weight	-	-	permanent
2	self-weight	live load	-	medium-term
3	self-weight	snow	live load	short-term
4	self-weight	live load	snow	short-term
5	self-weight	wind from the front	snow, live load	instantaneous
6	self-weight	wind from the side	snow, live load	instantaneous
7	self-weight	live load	snow, wind from the front	instantaneous
8	self-weight	live load	snow, wind from the side	instantaneous
9	self-weight	snow	wind from the front, live load	instantaneous
10	self-weight	snow	wind from the side, live load	instantaneous
11	self-weight	wind from the front	-	instantaneous
12	self-weight	wind from the side	-	instantaneous

Combinations CO1 to CO10 are meant for the calculation of the maximum compressive forces. Since for compression, in contrast to tension, buckling must be considered, these combinations will be decisive for the determination of the required cross-sections.

CO11 and CO12 are meant for the tension design, which is e. g. important for the design of the anchorage [6] in the panel and the CLT construction. The corresponding lower partial safety factor for the self-weight was used ($\gamma_G = 1{,}0$ instead of $\gamma_G = 1{,}2$).

3.2.3. Software

The preliminary design was mostly performed without the use of design software. Stab2d, a simple program for the analysis of two-dimensional frameworks, was used to determine internal forces and deformations. Microsoft EXCEL © was used for the stability analysis of the panel and the CLT construction. The stability analysis was conducted to determine the shear forces and bending moments in the individual walls due to the global wind loads.

The EC design required more accurate calculation methods. The FEM software RFEM © by Dlubal Software GmbH was used to model and analyse the overall structure of all three variants. With RFEM, both one-dimensional members, two-dimensional surfaces and three-dimensional volumes can be modelled. One of its major advantages is the extensive library of materials and the incorporation of many standards including the EC and its national annexes. RFEM can automatically combine loads according to load combinations. The details of the FEM analysis using RFEM are explained in chapter 6.5.1.

Additionally, an analysis of a selected connection detail will be carried out to compare with the calculations according to EC. The reasons for this will be explained in chapter 5.2. For this analysis, the FEM software ANSYS was used. ANSYS, in contrast to RFEM, is a program that focuses more on in-depth scientific calculations of details instead of on the analysis of whole engineering structures. It also includes an orthotropic material model which is required for wood.

3.3. Summary of the Theory

In the previous chapter, the theoretical basis for the examinations and the design analysis that will follow in the next chapters has been worked out. A brief overview of the rules and standards that deal with timber has been given. Some rules are still influenced by the former problems of wood products like the German M-HFHHolzR which does not allow visible wood members in buildings with more than 3 storeys due to fire safety reasons. In Norway, there were comparable rules until relatively recently, which can explain the weak position of the timber industry today. With the new strategy to support the timber industry in Norway, however, the situation begins to change.

[6] Anchorage refers to the connections between walls above each other, which are responsible for transferring tensile forces. Tensile forces occur at the bottom of the building where the global bending moment due to the wind loads is split into compression and tension forces.

4. Research Questions

The central research question for the following examinations is:

1. Which materials and construction methods are best used for an efficient design of multi-storey timber buildings from a structural point of view?

Some secondary questions have already been dealt with in the previous work:

2. How did the timber technology develop, and which new technologies are developed today?
3. What are the main differences between Germany and Norway concerning the way to construct timber buildings?
4. How do national rules influence the development of the timber building industry especially in Germany and Norway?

The answers to those questions will be summarised together with the answers to the other questions in the conclusions in chapter 9.

Further questions that will be discussed in the following include:

5. What are the main advantages and drawbacks of timber in comparison to steel and concrete, also considering environmental aspects?
6. Which role do connection play in the design of timber buildings?
7. How can finite element software be used to effectively design timber constructions and connections?
8. How well do the regulations in the standards reflect the real bearing behaviour of timber constructions and connections?

Concerning the design analysis, this master thesis focuses on the structural aspects of the timber materials, while the concrete and steel members will not be taken into account. Fire protection and economical aspects will only be discussed briefly in the text.

In order to answer those questions, the next chapter will deal with the properties of timber materials and structures in more detail. Some of the aspects mentioned in the previous chapters will be picked up and discussed, like the environmental issues and the new developments of today.

The main method to find answers to those question, however, is the design of the multi-storey timber building Treet, which will be presented in the following chapter.

5. Case/Materials

5.1. Timber Materials

5.1.1. General Properties of Solid Wood

Above all else, wood is a natural material. This has some great advantages but also includes some challenges. This chapter deals with both, describing important aspects to keep in mind when planning a timber building, and especially one as challenging as a multi-storey building. Firstly, some general properties of wood are presented, followed by an examination of its mechanical properties, which are important for the later design.

One of the characteristic advantages of wood is its aesthetic surface, it does not need to be covered and is often used for architectural purposes. This is true for solid wood as well as glulam and most of the engineered wood products. Moreover, with today's technology it can be formed into almost any possible shape (a good example for that is mass timber like CLT which will be described later). Besides that, wood stores CO_2, one of the main greenhouse gases (GHG). 1 m^3 of timber contains about 0,92 tonnes CO_2 [27, 26], thereby actively contributing to slowing down the global warming. In addition to that, wood is a renewable material because it always regrows.

A characteristic drawback of wood is that its properties vary a lot, depending on the used wood species, but also between samples of the same species because of its general inhomogeneity and defects like knotholes. This makes designing a reliable structure more difficult, larger safety factors must be used. On the other hand, different species can be used for different purposes, which makes different strength classes and economic adjustments to the design possible. For structural elements, fast growing softwoods like spruce and fir are preferred. In Norway, the Norwegian spruce and pine are mainly used. Hardwood species like oak or beech are used for special, highly loaded components such as wood dowels [21, 32].

To come back to the general disadvantages of wood, it is important to mention the decay. This is most of the time a consequence of moisture inside the wood. Measures to enhance the wood's properties concerning decay therefore focus on sealing the wood against the infiltration with water. Today, many natural substances and techniques are available, there is no need for using poisonous, environmentally damaging chemical wood preservatives. One option for the coating of the wood are biological substances from natural sources, examples are chitosan, produced from chitin, or pine oil [27, 43]. However, coating is in general very expensive, small defects can destroy the positive effects and regular maintenance is necessary [26, 238]. A completely different possibility is the biological, chemical or physical modification of the wood. One requirement for any such method is that no poison is used and no poisonous substrates are emitted. An example of chemical wood modification is the furfurylation. Furfuryl alcohol is used, under heat it reacts chemically with the wood molecules, thus rendering the wood surface more resistant against decay. Furfuryl alcohol is derived from furfural, which is produced from biomass waste, making the furfurylation a renewable method. The process is still quite costly though, so that it is not yet economic in most cases [24, 319-320]. An example of physical modification is the manufacture of thermally modified timber (TMT) by means of a special heat treatment. All those modification methods, however, have the drawback that they still have a negative influence on the wood's properties including the loadbearing behaviour and the tendency to crack [20, 26].

Another challenge for timber structures is the combustibility of the wood. But here again, with good planning, any problems concerning the fire safety can be resolved. When exposed to fire, wood creates a layer of coal on its surface, which protects the remaining part of the cross-section. The rate that the thickness of the coal layer increases with is practically constant over time, which makes it possible to determine the remaining cross-section after a certain amount time under the influence of fire quite accurately. EC5 part 1-2 for the fire design of timber structures is based on this principle. From those explanations it also follows that massive timber elements like CLT have advantages compared to lighter framework structures when it comes to fire safety, because of the smaller surface in relation to the volume.

All those aspects are important to keep in mind when designing buildings using wood products. But of course, for the structure itself, the mechanical properties are most important.

First of all, wood is anisotropic, more precisely orthotropic, that means that its mechanical properties are different in three perpendicular directions, parallel to the grain, radial and tangential, cf. Figure 5.1. This is due the wood's internal structure, consisting of long grains that run parallel to each other along the trunk of a tree. Additionally, the cross-section of a tree trunk is built up out of rings, each year a new growth ring is added to the outside, which all together leads to the three different directions.

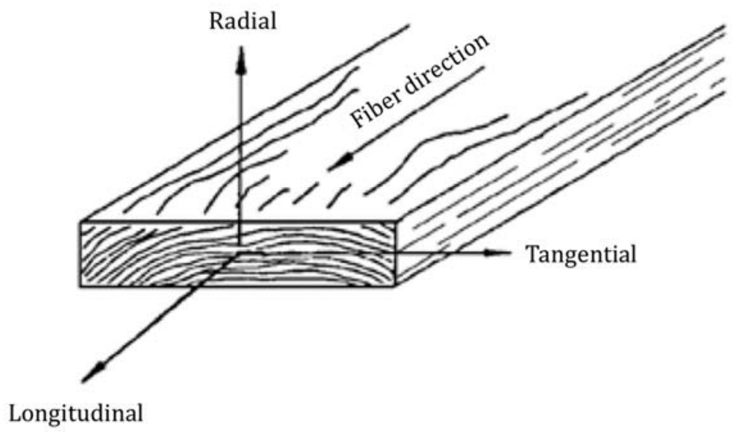


Figure 5.1: Three principal axes of wood with respect to grain direction and growth rings [16, 5-2]

In design practice, however, no difference is made between the radial and the tangential direction, only parallel and perpendicular to the grain are distinguished. Figure 5.2 shows the stress-strain-diagram of solid wood parallel to the grain. The tension strength f_t, the compression strength f_c and the strains ε at the corresponding points are pointed out.

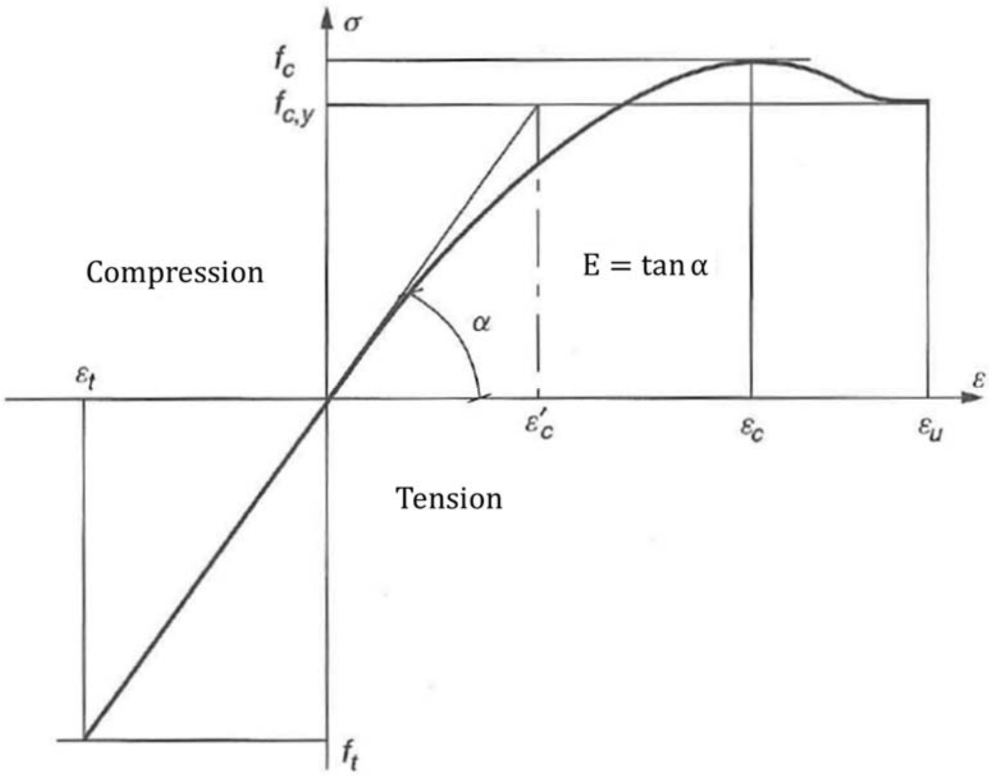


Figure 5.2: Stress-strain-diagram for solid wood [26, 203]

The strength parallel to the grain is highest, since this is the main direction a tree must carry loads in in the nature. Perpendicular to the grain, wood has a limited bearing capacity for compression, but this can lead to large settlements. This is especially interesting for multi-storey buildings, where rather small settlements can add up over all the storeys to a considerable deformation which can endanger the stability of the overall structure. Compression perpendicular to the grain should therefore generally be avoided in multi-storey buildings. The tension strength perpendicular to the grain is even smaller, about 30 to 50 times smaller than parallel to the grain [26, 19-20]. Aside from that, it is interesting to note that a higher strength class of wood does not lead to a considerably higher tensile strength perpendicular to the grain. Knots in the wood, however, which lead to a grading in a lesser strength class, seem to have a positive effect on the tensile strength perpendicular to the grain [26, 112-113].

In compliance with those findings, failure of timber structures in practice is in most cases a result of tension perpendicular to the grain. This occurs e. g. at connections with metal fasteners, at holes and notches and at the apex of gable roof beams. Changes in moisture content can also lead to eigenstresses and cracks which increase the danger of fracture perpendicular to the grain [26, 153]. The behaviour of wood under stresses perpendicular to the grain is not yet fully understood. Empirical methods are commonly used, like minimum distances of metal fasteners, or stresses perpendicular to the grain are avoided completely, e. g. by applying additional strengthening like fully threaded screws in the apex of gable roof beams [26, 19-20]. The failure itself is characterised by a rather brittle behaviour [20, 15].

Another important factor that influences the wood's properties is time. Wood loses its strength and stiffness under long-term loads. Unlike most other materials, this behaviour cannot be neglected, for solid wood the strength after ten years of loading can be reduced by up to 40 % [26, 21]. The long-term behaviour of wood is still a part of the research. Test results show a logarithmic relationship between the loading duration and the strength for bending, cf. Figure 5.3.

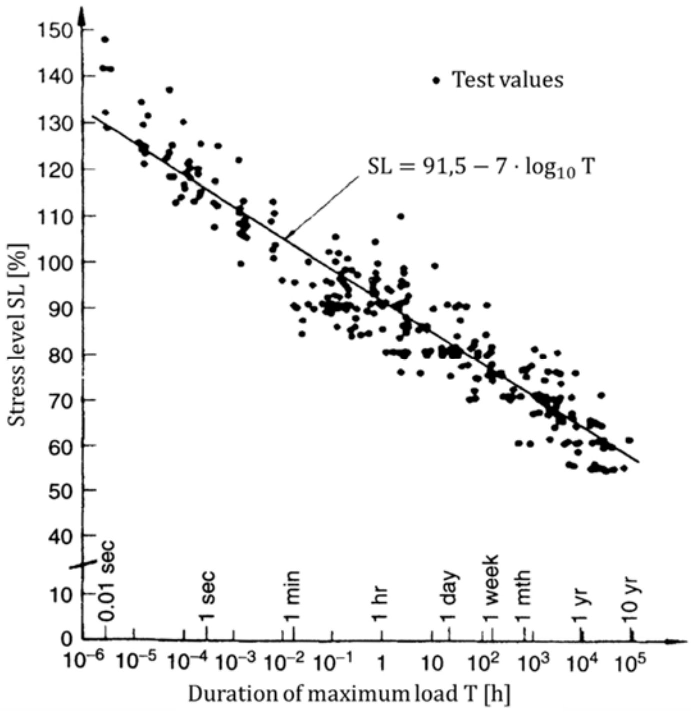


Figure 5.3: Decrease of the bending strength of timber depending on the load duration [26, 135]

For the tests that the above diagram is based on, specimens were loaded with a constant bending load at different stress levels and the time until failure was measured. The longer the load duration, the lower the acceptable stress level. For other failure modes than pure bending, only very little studies exist. These suggest, however, that for any kind of loading parallel to the grain, the behaviour follows approximately the curve from above, while loading perpendicular to the grain leads to a stronger decrease in strength [26, 141]. In the EC5, the influence of the load duration is considered via the modification factor for the wood's strength k_{mod}.

Moisture in wood has already been mentioned in connection with the decay. It also shortens the time to failure for members subjected to long-term loads [26, 141]. Moreover, with increasing moisture content, both the strength and the Young's modulus of timber decrease. The moisture content in the wood will adapt to the humidity of the surrounding air. That means that different climatic conditions lead to different moisture content in the wood and thus have a direct effect on the bearing behaviour. This effect is included in the EC5 vie the same modification factor as for the load duration, k_{mod}.

Besides the static moisture content in the wood, the changes of this content also have an impact on some the wood's properties. As the air humidity generally changes over the time of a year (depending on the region), a timber member that is not in a protected indoor climate, will go through many cycles of moisture changes, cf. Figure 5.4.

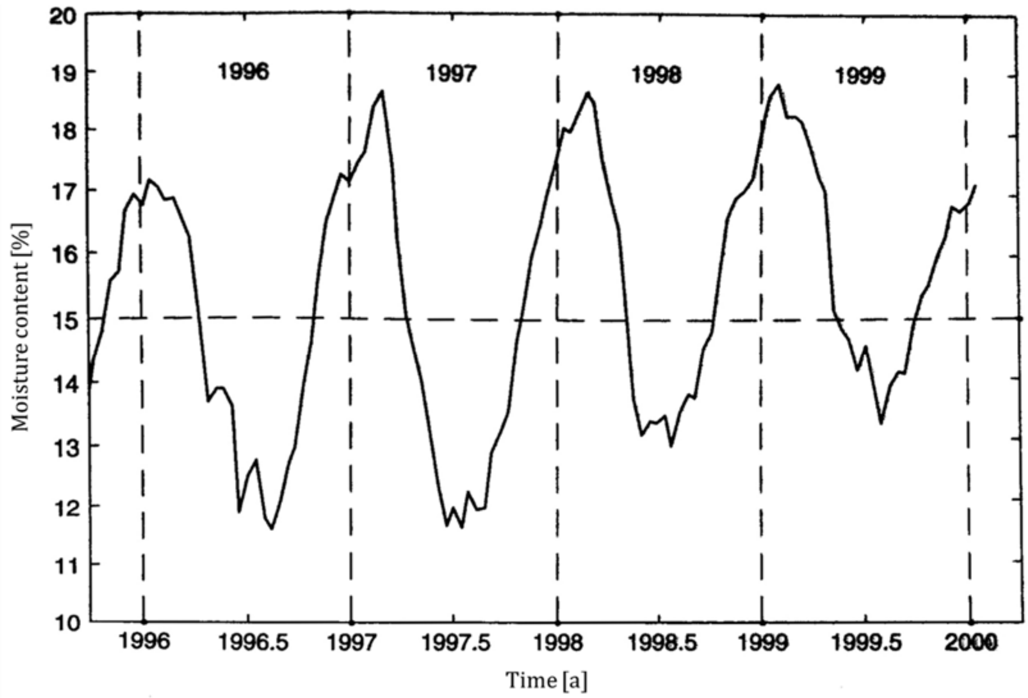


Figure 5.4: Mean moisture content in wood (glulam 90×100×600) versus time in a barn in Southern Sweden (Åsa) [26, 155]

Those changes of moisture content will amongst other things lead to increased creep deformations [26, 141].

Avoiding high moisture contents and moisture changes is important to ensure that the wood's properties do not change for the worse. Furthermore, decay effects must be excluded to guarantee durability and a long lifetime of a timber structure. To achieve this, some measures have already been mentioned in the beginning of this chapter. A general design rule is to keep wood members in dry surroundings or at least to allow water coming in contact with the wood to run off and evaporate without hindrances. It is also recommended to keep distances between wood members and components made from different materials to prevent moisture from gathering in the groove that can damage the wood. A very effective measure to protect wooden components from water is applying a layer on the component (e. g. wooden cladding) that can easily be replaced.

5.1.2. Wood in Comparison to Other Materials

Now that a general basis is created concerning the characteristic properties of wood, this basis shall be extended by comparing wood to other materials, namely reinforced concrete and steel, which are its most important competitors.

One aspect that has already been mentioned is the wood's variability and inhomogeneity, which leads to higher uncertainties concerning its mechanical (and other) properties. Both steel and concrete have much lower or practically no such variability and inhomogeneity, although concrete, because it is often produced directly on site, also has some uncertainties. This is directly reflected in the material safety factors that are defined in the different Eurocodes for those three materials: $\gamma_M = 1{,}3$ for solid wood [7], $\gamma_M = 1{,}5$ for concrete and $\gamma_M = 1{,}0$ for steel.

[7] Some engineered wood products, because they have a lower inhomogeneity, have lower safety factors, e. g. $\gamma_M = 1{,}2$ for laminated veneer lumber.

Furthermore, the orthotropy of wood has been explained. While steel is purely isotropic, i. e. its mechanical properties are the same in any direction, reinforced concrete features a different kind of anisotropy. Although both its basic materials, concrete and steel, are isotropic, the final product has a distinctive anisotropy depending on the layout of the reinforcement.

Another important aspect is the failure behaviour. Whereas wood has a rather brittle failure behaviour, concrete and especially steel can develop large plastic deformations before failure. On the other hand, wood has good stress distribution properties, which are advantageous for combined loads like compression and bending.

To give an idea of the overall performance and capacity of the three materials, some interesting properties are summarised in Table 5.1.

Table 5.1: Mechanical properties of steel, concrete and wood in comparison [1]

property	steel S355	concrete C30/37	glulam GL24h
yield strength f	355 N/mm^2	30 N/mm^2	24 N/mm^2
specific weight γ	78,5 kN/m^3	25 kN/m^3	3,7 kN/m^3
heat conductivity λ	50 $W/(m \cdot K)$	2,3 $W/(m \cdot K)$	0,12 $W/(m \cdot K)$
Young's modulus E	210000 N/mm^2	33000 N/mm^2	11600 N/mm^2

As one could already guess from the previous descriptions, steel is superior to wood in most mechanical properties. This can be partly compensated, however, by the wood's low self-weight, which makes more light-weight, efficient structures possible. The small weight combined with good elasticity is also advantageous for loads from earth-quakes. What still is a challenge especially in structures with large spans, is the wood's low Young's modulus, which leads to large deformations (even if the material would not fail). In such cases, special structures are needed like arched beams or lattice girders. Vibrations and an inferior sound protection are further problems connected to the wood's low Young's modulus and weight. Steel beams, on the other hand, can be used for larger spans without special measures, despite the steel's large weight.

One interesting property besides the mechanical behaviour is the insulation capability. The heat conductivity of wood is about 20 times smaller than that of concrete and more than 400 times smaller than that of steel. Moreover, wood regulates the climate inside a building by adapting its moisture content. Consequently, wood can fulfil multiple functions in a building, not only load-bearing but also building physical ones (and, additionally, architectural ones, as described before).

Concerning the decay, steel and concrete have advantages. But, as explained earlier, if designed properly, decay can be prevented, and steel and concrete require special measures as well to avoid corrosion, which can be seen as the "decay" of steel. In reinforced concrete, the concrete cover is normally dimensioned to protect the steel bars from corrosion for 50 years. Steel requires expensive coating or galvanising when exposed to the weather.

When it comes to logistics and the erection of a building, the wood's low self-weight is again advantageous. This also makes a high degree of prefabrication possible, allowing for very economic structures. In addition to that, wood products can relatively easily be adjusted on the construction site. Here, steel has some disadvantages, while (non-prefabricated) concrete allows for the highest degree of adaption.

Looking at ecological aspects, wood reveals its greatest advantages. Because of the low weight, less energy is needed for transport and erection. Wood is infinitely available and does not damage the environment if sustainable forestry is applied. In contrast to that, while the raw materials for steel and concrete are abundant as well, their exploitation is often connected to environmental damages. When manufacturing timber, the waste wood can be used to power the factory. No additional energy is needed when using modern, efficient power production with heat recovery. And when it comes to the production of the final building product, timber uses in general less energy than concrete or steel [27, 26].

Recycling at the end of the lifetime of a building can effectively reduce the demand for the raw materials and is today possible for all three materials. The recycling of steel is most effective, while concrete can only be crushed and used as aggregate for new concrete again. It is, however, also possible to use by-products from steel and power production like slag and fly ash for new concrete [24, 412]. Wood can be reused for engineered wood products or, as a last possibility, burned and used as an energy source.

Since ecological aspects play a more and more important role in our society and are one of the central factors when planners decide to use wood for a certain project, those aspects will be addressed in more detail in the next chapter, focusing again more on the wood itself. As a means to analyse the ecological aspects, life cycle assessment (LCA) will be used.

5.1.3. Aspects from Life Cycle Assessment

Before starting to analyse aspects from LCA, a quick explanation of LCA methods shall be given. “LCA is a methodology used to analyse complex processes which focus on dealing with the input and output flows of materials, energy and pollutants to and from the environment from a life cycle perspective” [24, 49]. Its objectives are, on the one side, to quantify the environmental properties of a product or process, and on the other side, to assess potentials for improving the product or process [24, 49]. Without knowing much more about LCA, it can be guessed that to quantify all the environmental influences of a product in one number or even in several numbers is practically impossible. The results of an LCA analysis are never exactly correct because of the high complexity of the processes and because it is not possible to predict the future accurately. This must be kept in mind when interpreting LCA results. Hence, LCA is especially suited to compare different alternatives by means of relative results, not so much to obtain absolute results for one product or process. Looking at the relative results, LCA gives correct results almost every time [24, 417-418].

A strong tool for the interpretation of the results is an evaluation matrix, which balances environmental, economic and social factors e. g. for the selection of a material for a specific project [24, 44]. (A similar matrix will be used at the end of this master thesis in chapter 7.2 to compare the three different construction methods that will be designed and analysed, extending the tool by technical aspects.)

Important aspects in an LCA study for building products are

- resource efficiency,
- energy efficiency,
- GHG emissions,
- pollution prevention and
- waste management after the end of the lifetime [24, 421].

The resource efficiency includes the exploitation of the raw materials as well as reuse and recycling.

The energy efficiency is a big factor which includes all the required energy from depleting the raw materials, to manufacturing, transporting and maintaining the product, also called embodied energy. Influencing factors are amongst others the lifetime of a product and the amount of maintenance that is required. To ensure a long lifetime, measures must be taken to preserve the wood, as has already been explained. The embodied energy represents normally around 10 to 60 % of the total energy used during the whole lifetime of a building, which includes the energy for heating and using the building. As buildings become more energy-efficient concerning heating and the use of electricity, the embodied energy in the materials becomes proportionally more important [24, 421].

GHG emissions is another central aspect for the whole lifetime of a building. Wood as a construction material can be carbon neutral or even carbon negative with good recycling [24, 418], because carbon is stored in the wood. However, taking into consideration aspects from a full LCA, studies showed that the total amount of avoided carbon is actually 2,1 times higher than the carbon stored in the wood itself. This is due to less carbon emissions in the industrial processes compared to other materials and usage of by-products from the manufacturing of wood products as biofuel to replace fossil fuels [24, 326]. Concerning the forestry, some studies found that intensive forestry has less CO_2 emissions than traditional forestry. This is because the productivity is much higher, therefore more CO_2 can be stored. Nonetheless, it must be kept in mind that CO_2 is not the only factor for LCA and an intensive forestry can produce conflicts with other aspects, e. g. the biodiversity [27, 27].

Figure 5.5 shows the carbon content and the embodied energy of some selected building materials. It must be noted that the carbon stored inside the wood is not considered in the presented values. Moreover, the values are given related to 1 kg of the corresponding material, but different amounts are needed for different materials in the same structure. The ratio of the total required weight of material between a wooden structure and a comparable concrete structure can be up to 1:5 [24, 413].

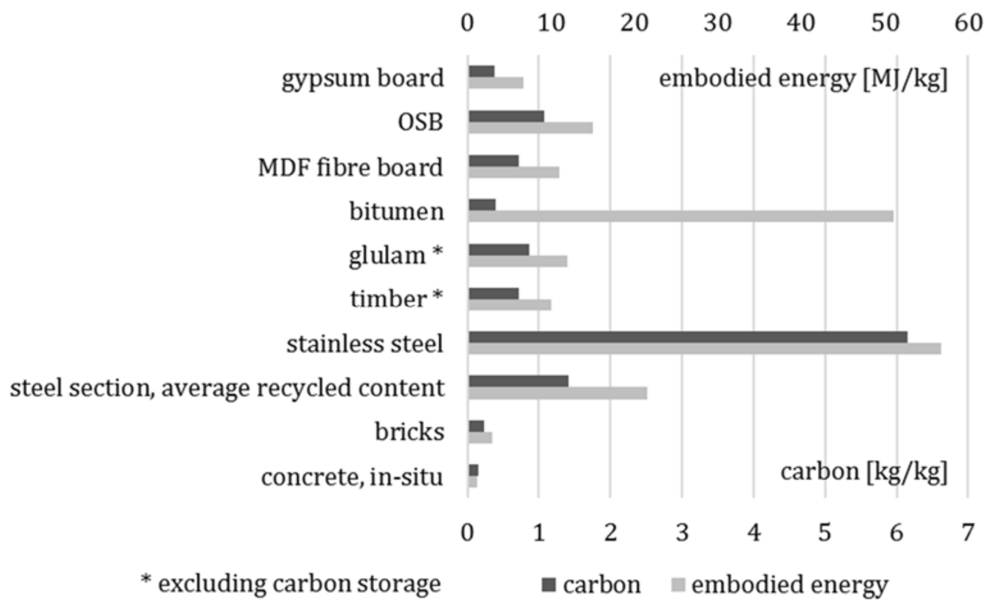


Figure 5.5: Embodied energy and carbon content of different building materials based on [24, 415]

Pollution prevention concerns the release of polluting gases during both the manufacturing process and the use of a product [24, 45-46].

With the waste management, potentially including recycling, the cycle closes again.

Buildings are very complex structures since materials and members generally fulfil several functions. Because of that and because different materials require different quantities to fulfil a specific function, the materials cannot be directly compared. Moreover, all the components inside a building influence each other in some way or another. The best way to perform an LCA analysis is therefore to compare whole buildings that have the same functionality. Conclusions from various such studies show that wooden constructions have in general lower environmental impact than equivalent structures using non-wood materials [24, 321].

Besides the choice of material, structures that can easily be disassembled are advantageous from an LCA point of view, because it improves recycling capabilities. In wood buildings this can be achieved by preferably using bolts instead of adhesives. Monolithic concrete structures, on the other hand, have to be demolished for recycling [24, 418].

A controversial point concerning modern wood products is the roll of prefabrication. Prefabricated products have a lower environmental impact during the manufacturing, the construction and at the end of their life, but the transportation has a much higher impact [24, 438]. The distance between the building site and the factory is critical [24, 452]. The distances are generally much larger for specially manufactured products than for individual standard components. This is highly dependent on the specific project.

To sum up, an example of a full LCA study conducted in 2000 by Börjesson and Gustavsson shall be given. It compared the net CO_2 emissions of two similar four-storey apartment buildings, one in Sweden featuring a wood frame structure, the other in Helsinki made from a concrete frame structure. The results showed that the wood construction released 30 to 133 kg/m^2 less CO_2 into the atmosphere. Later additional research confirmed the general results and found that concrete structures need 16 to 17 % more total energy than wooden structures. Those results are, however, only true for northern Europe, where wood is available locally [24, 414-416].

Looking into the future, wood still has unused potentials. One potential is to increase recycling and to use more wastes to replace fossil fuels for energy or heat production. E. g. in Norway, only about 5 % of the wood from buildings is recycled, 70 % is used for energy production, 9 % is send to landfills, 2 % is composted and 17 % is used for unknown intentions [27, 44]. To achieve improvements, different industries like the forestry, energy, building and the waste industry must work together [27, 26]. Wastes, hitherto little used species and recycled material can well be used for new engineered wood products, which is the topic of the next chapter.

5.1.4. Engineered Wood Products

Engineered wood products are today already an important part of the supporting structure in most buildings. Their advantages are obvious: Whereas solid wood is only available in the form of linear members and the cross-section area is limited by the size of the tree trunk, engineered wood products allow arbitrary cross-sections and also two-dimensional members like plates. Moreover, the properties of the wood can be improved purposefully.

Engineered wood products are in general made from boards, veneers, strands and flakes of wood that are joined together with the use of adhesives. The final products can be of variable form, e. g. boards or timber-like beams. The final products are much more homogeneous than the initial solid wood, knots and other defects can be eliminated or at least evenly arranged over the whole component [26, 82-83].

The manufacturing of engineered wood products rises the efficiency of the timber production because it also allows the use of parts from the initial logs that cannot be used for solid wood beams, like the branches and the waste from the production of solid wood. This allows for a more efficient use of the wood, compared to the manufacturing of traditional solid-wood members, which has a relatively high degree of waste because rectangular cross-sections are cut from a round log.

There are many different engineered wood products available today, and more are being developed.

One of the most widely used engineered wood products is glued laminated timber (glulam). Glulam is made of several planks that are glued together on top of each other, forming a cross-section that can be as high as 2 m [26, 7]. With glulam it is possible to overcome one of the solid wood's limiting factors, which is the size. The biggest possible dimension for solid wood is about 300 mm, thus the maximum possible span of a timber beam in practice is about 5 to 7 m [26, 7]. Glulam allows practically arbitrary cross-sections for spans of up to 100 m [26, 8]. Using finger joints, beams of theoretically any length can be produced. Due to transportation limitations, however, the maximum length in practice is limited to 16 to 20 m [26, 68]. It can be produced in both straight and curved forms, so that e. g. whole arches can be produced as one solid wood component.

As with other engineered wood products, glulam has enhanced material properties in comparison to solid wood, the strength and stiffness are increased, while the variability is smaller [26, 69], this is depicted in Figure 5.6.

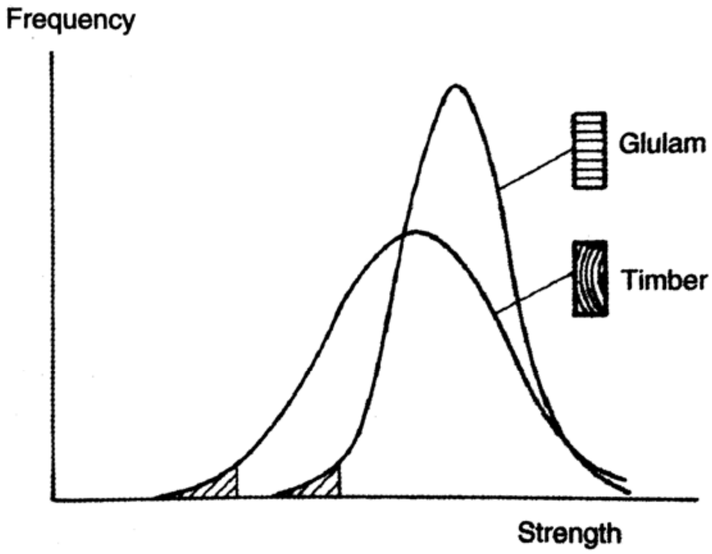


Figure 5.6: Comparison of the frequency distribution of the strength of glulam and solid wood [26, 19]

Since the height of the cross-section is normally much higher than its thickness, stability failure modes such as lateral torsional buckling are of higher importance for glulam than for solid wood.

A different engineered wood product is parallel strand lumber (PSL), which was developed with the idea to utilise forest wastes such as branches. It is made from long, thin strands of wood with a length of up to 2,4 m and can be manufactured with cross-sections of up to 280×480 mm, which lies in the same range as the possible cross-sections with solid wood. It is therefore mainly used as a substitute for solid wood, with the benefits that it makes the forestry more efficient and has a higher bending strength as well as less shrinkage and splitting compared to solid wood [26, 88-91].

Laminated veneer lumber (LVL) is made from layers of wood veneer that are glued together, all with the orientation of the grain along the long axis of the member. Cross-sections of up to 90×1200 mm are possible, the length can be 25 m. It can be used for beams in small and big constructions and is also commonly used for flanges of I-joists [26, 93-95].

I-joists are bending members with a more efficient structural shape, because the material is concentrated in the outer regions of the cross-section, the flanges. Those are commonly made from solid wood or LVL. The web is typically made from oriented strand board (OSB) or plywood panels, i. e. wooden panels with a high shear capacity [26, 97]. One drawback of I-joists is their vulnerability to fire because of the thin web.

For plywood, layers of wood veneer are glued together, but unlike for LVL, those layers are rotated by 90° to each other, forming a board that has good bearing capacity in both directions and a more isotropic behaviour. Oriented strand board (OSB) is made from small strands of underutilised wood species. Both products can be used as sheathing for walls and floors, where they provide stiffening of the component and act as cover at the same time. OSB, however, has the highest emissions of volatile organic compounds (VOC) of all engineered wood products [27, 35] and should therefore be used with care in residential buildings.

A more recently developed engineered wood product is cross-laminated timber (CLT), which is a mass timber product. It consists of layers of parallel boards, 10 to 35 mm thick, which are then glued together at 90° to each other to form both massive plates and massive members with large cross-sections. Where the breadth of glulam cross-sections is limited by the breadth of the individual boards, members made of cross-laminated boards can have arbitrary dimensions in both directions. The cross-lamination leads to a more homogenous behaviour compared to solid wood. It also reduces the shrinkage and swelling to an insignificant amount [20, 52-55]. Furthermore, CLT panels are in general airtight (depending on the specific product) [21, 115]. They are therefore mainly used for floors and walls, where they can fulfil the function of an airtight membrane in addition to the loadbearing and the stiffening. CLT is today readily available in practically all possible dimensions.

Besides the mechanical properties, there are also noticeable price differences. Although economic aspects are not within the scope of this master thesis, Table 5.2 gives an idea of the price differences, which will of course play a role in the planning of a timber building.

Table 5.2: Prices of some selected engineered wood products in comparison to solid wood [20, 51]

material	price
solid wood	300–400 €/m^3
glulam	1000 €/m^3
parallel strand lumber	1500 €/m^3

A central drawback of all the engineered wood products is that they use synthetical adhesives. Those are typically urea- or phenol-formaldehyde which are made from non-renewable mineral oil. Moreover, they emit formaldehyde during their lifetime (in different amounts, depending on the product). New developments aim at reducing the use of adhesives or using natural alternatives such as lignin-based adhesives [24, 317].

In the future, when environmental restrictions may lead to less available large-size solid wood, engineered wood products will become more important [26, 83]. Today, solid wood is already meeting its limits when it comes to structures like multi-storey timber buildings. Products like glulam and CLT open up a completely new way of timber design. It will be seen in chapter 6.2 that none of the analysed structures uses solid wood as a central structural material because larger cross-sections are required than what is possible with solid wood. It has also been explained how deformations due to compression perpendicular to the grain or moisture changes can be decisive for multi-storey buildings. In this regard, the described engineered wood products are superior to solid wood, too.

5.2. Timber Connections

The missing link before moving on to discussing entire timber structures are the connections. Connections are of major importance for timber constructions and especially for multi-storey buildings because of the high loads. Evaluation of damaged timber buildings after extreme wind showed that connections are one of the major weak points [28, 81].

There are, in general, two possibilities to form connections:

- glued connections or
- connections with dowel-type fasteners (nails, bolts, screws, dowels, commonly made of steel) [26, 303].

While glued connections are mainly applied on prefabricated components, e. g. for finger joints in glulam elements, dowel-type fasteners are well suited for any on-site assembly. Since the properties of glued connections are most of the time already included in the description of the properties of the corresponding engineered wood product, the following discussions will focus more on connections using fasteners. Nonetheless, it shall be noted that new techniques are being developed that also allow the economic manufacture of glued joints on-site. Table 5.3 summarises the main advantages and disadvantages of glued connections.

Table 5.3: Advantages and disadvantages of glued connections [26, 334]

advantages	disadvantages/problems
• higher rigidity • can be highly automated • makes special connections possible, e. g. finger joints • the cross-section is only minimally damaged, or not at all	• more sensitive to unskilled manufacture • lower inherent redundancy • in general, no on-site manufacture possible • often very brittle behaviour • emission of VOC • complex mechanical behaviour, stress peaks

Focusing on connections with dowel-type fasteners from now on, first some general aspects will be explained before looking at the different types of connections in more detail.

Results of experiments show correlations between several parameters and the capacity or failure behaviour of the connection.

One important parameter is the slenderness, i. e. the ration between the side width of the wood member and the dowel diameter, which leads to different failure modes and thus very different loadbearing capacities, see Figure 5.7. A low slenderness generally means that there are no plastic deformations in the dowel, only in the wood. A medium slenderness leads to one plastic hinge in the dowel, while for a high slenderness secondary plastic hinges develop [17, 68-69]. Larger wood thicknesses generally lead to a higher maximum load, a higher stiffness and larger deformations.

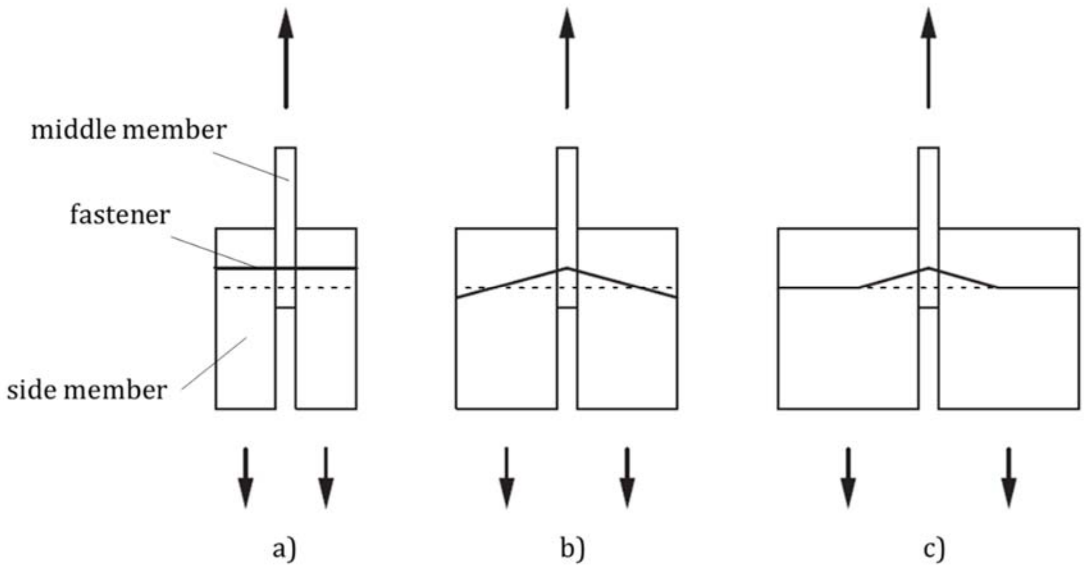
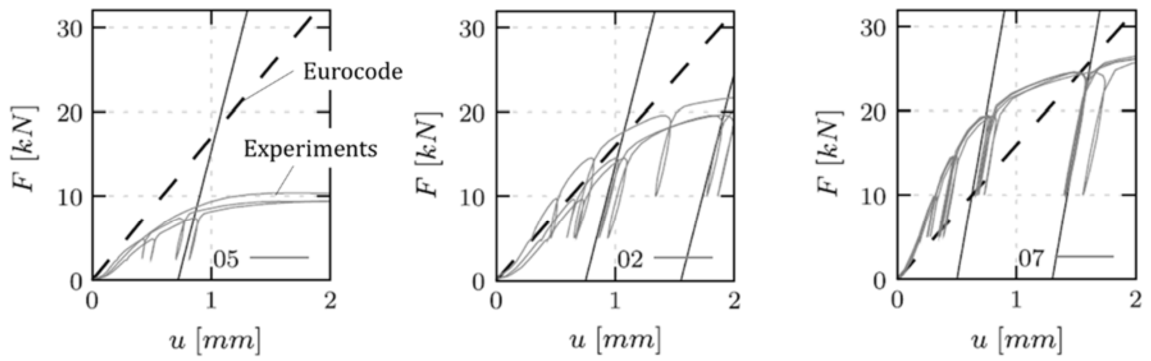


Figure 5.7: General failure modes of a timber connection with a) low slenderness, b) medium slenderness and c) high slenderness [17, 68]

Moreover, there is a linear correlation between the density of the wood and the maximum load and stiffness of the connection. A connection with high density wood is characterised by a higher capacity, less deformations in the direction of the grain and a more brittle failure behaviour with splitting perpendicular to the grain (as was explained in chapter 5.1.1, the tensile strength perpendicular to the grain does not change much even for strong wood and thus becomes decisive in an otherwise high strength connection). The spacing between the fasteners in a multi-fastener connection and their end distances also have a strong influence on the capacity of the wood since too small distances provoke the risk of splitting [17, 70-74].

Johansen developed a yield theory that can be used to determine the failure mode and to calculate the resistance of a wood connection, although it is not suited to determine the load-slip behaviour. According to this theory, there are three main aspects that effect the strength of a connection: the bending capacity of the dowel, the embedding capacity of the wood or engineered wood product and the withdrawal resistance of the dowel. The embedding strength depends mainly on the density of the wood [26, 316-322]

Today's design rules for connections in e. g. the EC5 standard are mainly based on empirical investigations and the Johansen-theory. Comparing the findings of the experiments with calculated values according to EC5 reveals a good accordance for the stiffness and conservative values for the strength for medium slenderness connections. For small slenderness, EC5 overestimates the stiffness and the strength, while for high slenderness, EC5 underestimates the stiffness but overestimates the strength again [17, 76-79], cf. Figure 5.8 and Figure 5.9.

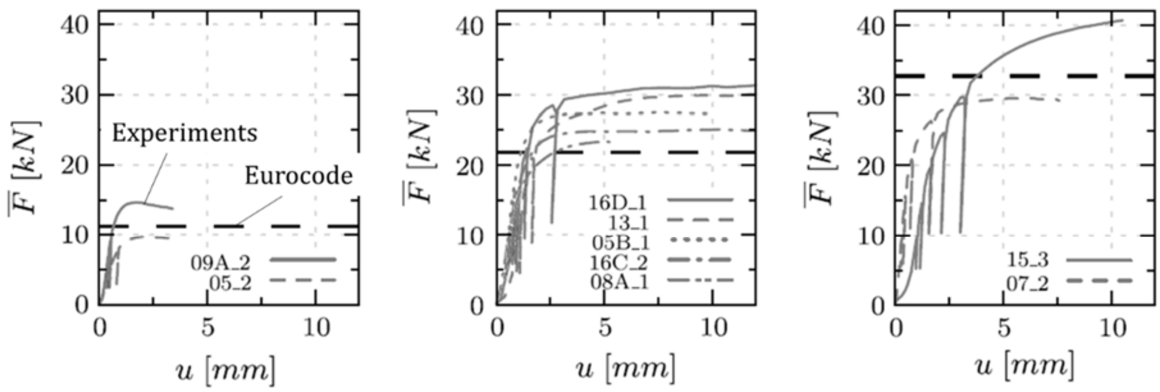


a) Specimens with small slenderness

b) Specimens with medium slenderness

c) Specimens with high slenderness

Figure 5.8: Comparison of experimentally determined stiffnesses of selected tests with corresponding design values from EC5 for specimens with different slenderness [17, 77]



a) Specimens with small slenderness

b) Specimens with medium slenderness

c) Specimens with high slenderness

Figure 5.9: Comparison of strength of selected tests with design values from EC5 [17, 77]

Because of those discrepancies, the applicability of the EC5 rules for connections is limited and an optimised design is difficult [17, 67]. Since connections are a central part of any timber structure, especially of multi-storey buildings, one section in chapter 6 will be dedicated to analysing a selected connection in one of the investigated structures using FEM to gain a better understanding of its behaviour.

The rest of this chapter will focus on the characteristics of the different connection methods using dowel-type fasteners.

The first type of connection that will be discussed are traditional timber joints, some examples are shown in Figure 5.10.

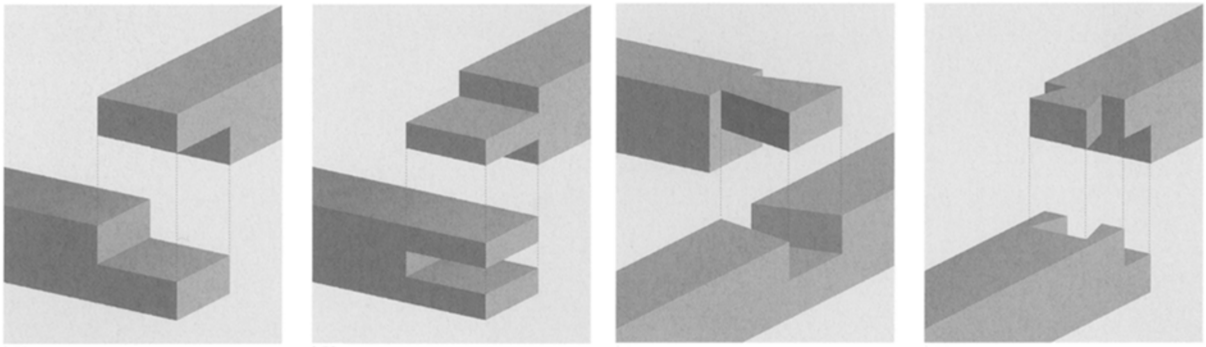


Figure 5.10: Examples of traditional timber joints [21, 58]

Their main characteristic is that they do not use much steel, fasteners are only required to secure the individual components against unwanted movements. Those connections therefore have a good fire resistance. On the other hand, they can have a complicated geometry, especially if tension forces must be transferred. With today's modern CNC wood-working machines, however, it is possible again to manufacture such joints economically [26, 304-305].

Right after the traditional timber joints, nails have been used for a long while in timber buildings. Today they are relatively cheap, often no pre-drilling is needed, and many different connections are possible [26, 306-307], cf. Figure 5.11. One disadvantage is that they have a relatively low loadbearing capacity, which limits their applicability especially in multi-storey timber buildings.

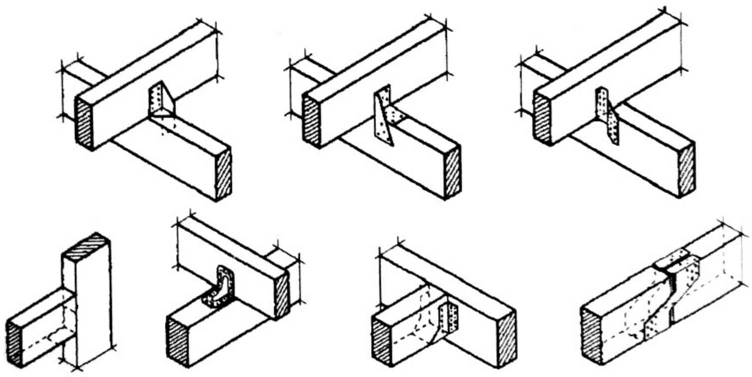


Figure 5.11: Examples of nailed joints [26, 306]

This is where bolts and dowels come in, they can be used for high-strength connections in big and complex structures. Both fasteners are normally made from metal, with much larger diameters than nails (a common bolt is e. g. an M20-bolt with 20 mm diameter). Bolts are threaded and have a counter nut and washers, while dowels have a smooth surface and are completely hidden inside the wood member, cf. Figure 5.12.

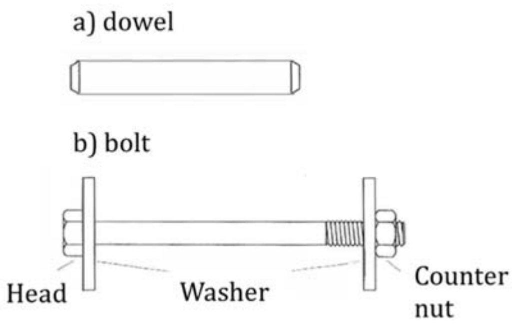


Figure 5.12: Steel dowels and bolts [1, 9.44]

The large diameters requires the wood to be pre-drilled which makes such connections more labour-intensive [26, 307]. On the other hand, when prefabrication is used, the holes can already be produced in the factory. The choice of diameter effects the loadbearing capacity and behaviour, larger diameters lead to higher acceptable loads but also a brittle behaviour due to splitting (cf. explanations concerning the slenderness of a connection above). Examples of typical applications for dowels or bolts are illustrated in Figure 5.13.

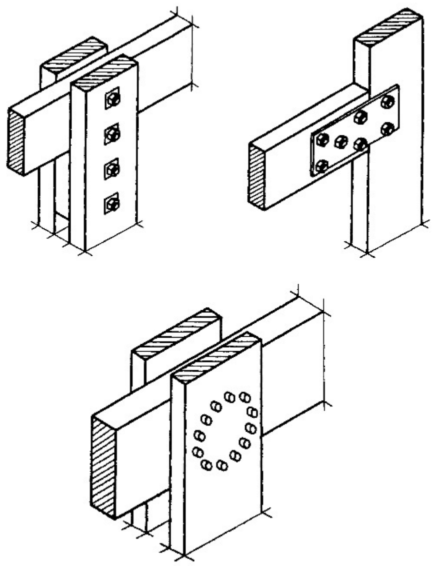


Figure 5.13: Examples of joints using bolts or dowels [26, 308]

A rather new development are screw joints, with the screws being subjected to tensile loads. This is a big difference to all other methods described so far, which rely on the shear capacity of the fasteners. Modern hand-held tools to easily drive screws into the wood have led to screws more and more replacing nails. At the same time, they can be disassembled more easily [26, 307-308]. Moreover, modern fully threaded, self-tapping screws do not require pre-drilling [26, 328]. The self-tapping screws make new, effective connections possible, some examples are shown in Figure 5.14.

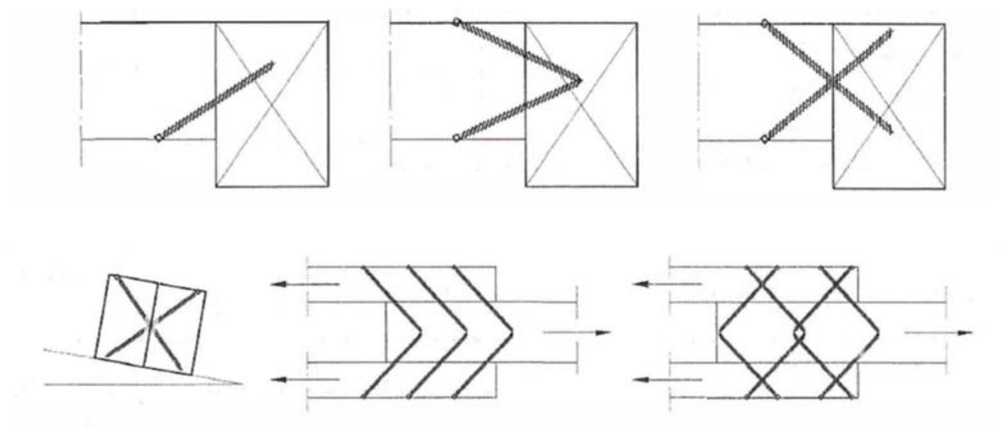


Figure 5.14: Examples of applications for fully threaded screws [1, 9.55]

A different possible application of fully threaded screws is the reinforcement of wooden members perpendicular to the grain. This ensures a ductile behaviour of the connection and increases the loadbearing capacity, since a failure perpendicular to the grain, i. e. splitting, is prevented [26, 311-313] [26, 317]. The positive effects of this reinforcement are not covered by the Johansen-theory, though, and are therefore not considered in the EC5 yet [17, 79].

Another drawback of the Johansen-theory and the regulations in the EC5 is that the theory has been developed only for single fasteners. The design of connections with several fasteners is based on the properties of the connection with one fastener, considering an effective number of fasteners, which respects amongst other things the spacing between the fasteners. For multi-fastener connections, reinforcement perpendicular to the grain is especially interesting. With reinforcement, group effects can be excluded, so that the load-carrying capacity of the connection can be assumed to be the sum of the capacities of all fasteners [26, 322-324]. Proof for the vastly different loadbearing behaviour is depicted in Figure 5.15. Not only is the acceptable load of the connection higher, but the ductility is increased strongly, too, which can be seen from the large deformations that occur before the connection fails.

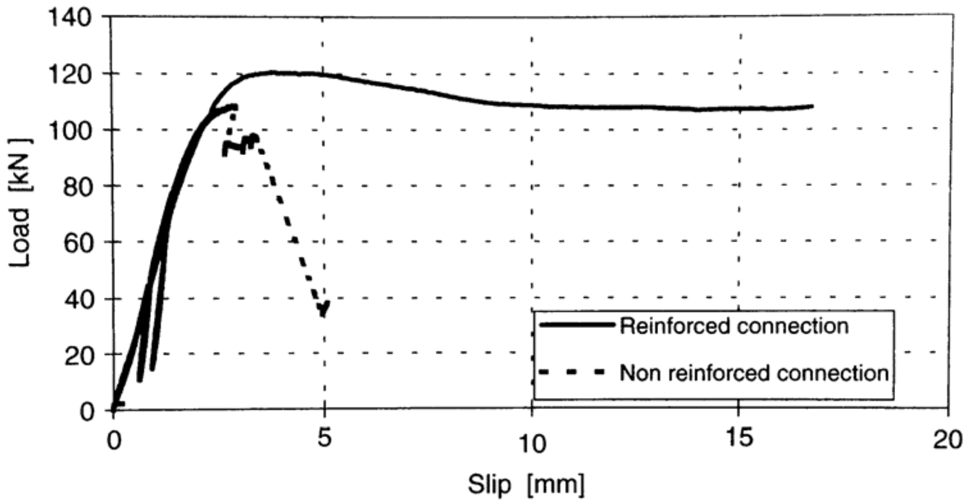


Figure 5.15: Loadbearing behaviour of a reinforced connection in comparison to a non-reinforced connection [26, 324]

When using connections with several fasteners, it is important to allow for moisture induced movements. Connections where the wood is rigidly fixed at more than one connector should be avoided because this can lead to cracks when shrinkage occurs due to changes in the moisture content [26, 160].

With the description of the behaviour of multi-fastener connections, the chapter about connections is completed. While the descriptions and discussions so far dealt with wood in general, or specific products and individual components and connections, the next step is to look at entire timber structures that consist of those components. In the next chapter, the differences between the different structural methods and their variants are described, giving an idea of which methods are best used in which cases. Based on those findings, the design analysis in chapter 6 will be planned and conducted.

5.3. Timber Structures

5.3.1. Timber Framing

In general, two main construction methods are distinguished. Timber framing structures, featuring one-dimensional elements in the main loadbearing structure, will be examined in this chapter. Mass timber structures, which use mass timber materials like CLT, will be the topic of the following chapter.

Structures that use a timber framing method are generally characterised by an open load bearing structure, a clear force distribution and a clear distinction between the load bearing function, the stiffening function and the room separation. The loads are concentrated in the individual members, which thus can be adapted accordingly, concerning e. g. the cross-section. Connections are more challenging than for mass timber structures because of the high concentrated loads and become therefore often decisive for the dimensioning of a component.

The following variants will be discussed in this chapter:

- traditional frame construction,
- panel construction,
- balloon construction and
- modern frame construction.

The traditional frame construction spread in Europe around the year 1700 (cf. chapter 2.1) and is today often associated with medieval timber houses. The framework consists of horizontal and vertical beams as well as diagonals for the stability and stiffening, see Figure 5.16. The framework is often visible from the outside with the cavities filled with e. g. brickwork. This method is mainly used for small one- or two-storey buildings, the erection of larger buildings becomes very costly [21, 55].

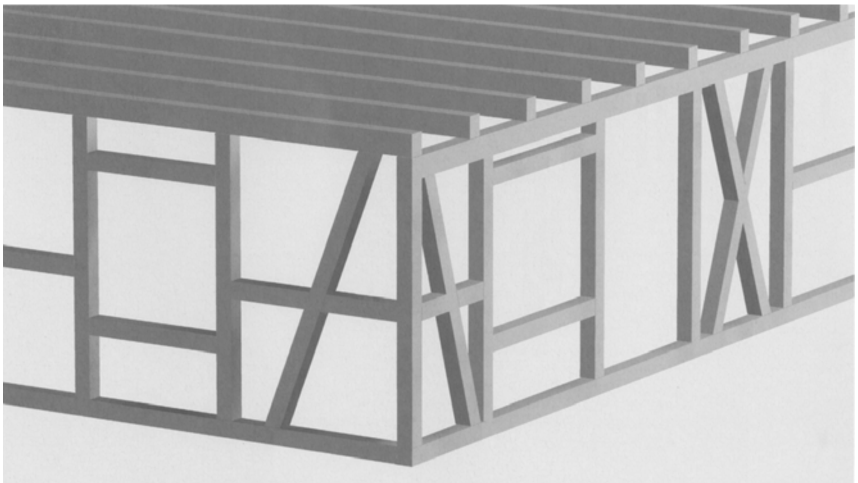


Figure 5.16: Traditional frame construction [21, 56]

The panel construction is an enhancement of this method and is illustrated in Figure 5.17 (left).

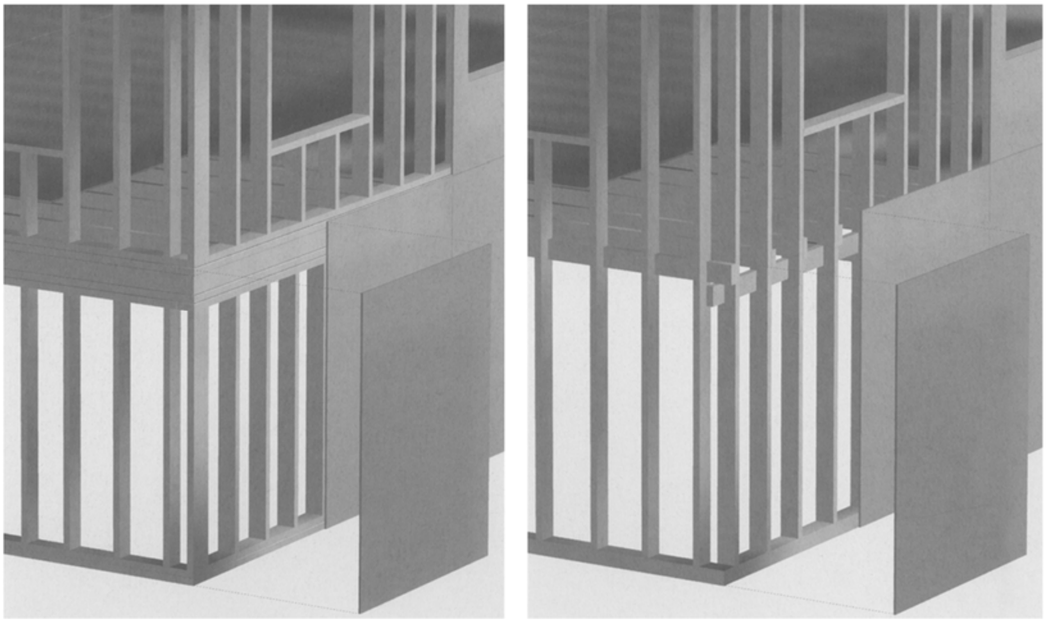


Figure 5.17: Panel construction (left) and balloon construction (right) [21, 60-61]

It replaces the diagonals with wooden panels made of plywood or OSB as sheathing. This allows insulation to be placed in the same layer as the load bearing structure, in-between the closely spaced vertical members, the studs. The big advantage of this kind of structure is the beneficial way the individual elements work together, because the sheathing prevents the studs from buckling in the direction of their weak axis. This makes very slender construction possible, the panel construction is therefore probably the most structurally efficient system.

The wall studs are as high as one storey, which makes this method highly suitable for prefabrication. Walls and floors or even entire rooms or modules can be prefabricated. This also leads to a simple erection, no special equipment is required. From a structural point of view, prefabricated wall panels with nailed sheathing are highly redundant. A relatively low grade of timber can be used [26, 386].

The panel construction is commonly used for smaller buildings with one to three storeys. For multi-storey buildings, the walls need higher shear resistance and vertical load bearing capacity to withstand the high wind loads and vertical loads from the self-weight as well as the live load.

New developments to increase the shear capacity of walls include brace systems between the studs or additional sheathing [26, 404].

The balloon construction is similar to the panel construction, with the main difference that the wall studs go over the whole height of the building or at least over several storeys (cf. Figure 5.17 (right)). Vertically continuous members are always advantageous in multi-storey buildings because they lead to less deformations due to shrinkage and gravity loads. On the other hand, prefabrication is no longer possible in the same degree as for the panel construction [26, 238] [21, 60-61].

Finally, the modern frame construction is also based on the traditional frame construction, but takes the timber framing to the next level. An example of a modern frame construction is depicted in Figure 5.18.

Figure 5.18: Example of a modern frame construction: school in Wil, Switzerland [21, 86]

In contrast to the previously described methods, the modern frame construction features big spans, with the loadbearing components being widely spread on a large, regular grid. The load-bearing structure consists of a system of columns, main and secondary beams. The walls do not carry any loads, the horizontal loads are rather transferred by the vertical members via bending or via large-span diagonals. This opens up new architectural possibilities like big windows. It also offers maximum freedom when it comes to subdividing the interior into rooms, changes are possible without affecting the structure. Besides, the enclosing envelope formed by the walls can be arranged outside of the load carrying structure. This has three main advantages: Firstly, the load carrying structure and enclosing envelope are completely independent, which allows for easy connections. Secondly, the walls act as protection of the load carrying structure from the weather, and thirdly, the load carrying structure stays visible from the inside.

For the modern frame construction, larger cross-sections are required, therefore, glulam is the preferred material. An important part of the construction are the connections between the columns and the main beams. There are different possibilities, the decision will depend on project-specific structural requirements and architectural aspects. One can use continuous columns with two-part continuous beams attached to the sides of the columns, or with simply supported beams in-between the columns, attached to them with their front ends. Another possibility is to use two-part columns or forked columns with the beams in-between [21, 86-94]. To sum up, the modern frame construction is highly suited for multi-storey buildings.

5.3.2. Mass Timber

Mass timber constructions consist of two-dimensional components, i. e. walls that carry the vertical loads and the shear, and floors which account for the horizontal load distribution. In comparison to frame constructions, mass timber constructions have a more complex behaviour. This is because the components perform different functions at once, like the load bearing, the stiffening and the room separation. The loads are more distributed and therefore generally lower. Aside from the purely structural properties, mass timber is also characterised by a higher weight which leads to a better sound protection. The fire safety is increased, too, since two-dimensional elements are only affected by the fire on one side, instead of on up to all four sides for one-dimensional elements.

The different construction methods using mass timber that will be examined in this chapter are

- log constructions,
- stacked plank constructions and
- CLT-constructions.

The log construction has already been mentioned in chapter 2.1 concerning the historical development in Norway. It is one of the oldest methods and is still in use, although in modified form. An overview over different variants of the log construction are shown in Figure 5.19, with the oldest ones on the left and more recent developments to the right. Any of those log constructions consists of logs that are horizontally stacked on top of each other. The third one from the right shows a prefabricated sandwich element. The next two are thermally insulated log walls which are assembled on site.

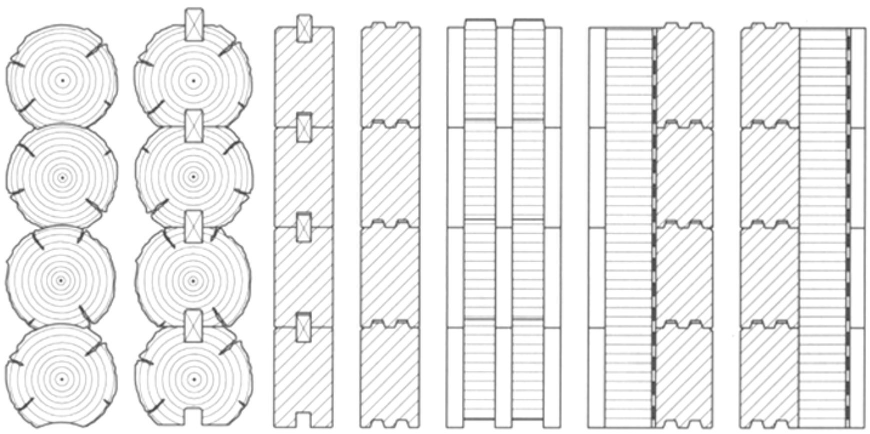


Figure 5.19: Different log construction variants [21, 53]

The biggest problem with the log construction is that large compressive stresses occur perpendicular to the grain. As explained earlier, this leads to high settling, about 25 mm per storey, which is why log construction are not suited for multi-storey buildings [21, 50-53].

A different mass timber variant is the stacked plank construction, where planks with thicknesses between 20 and 50 mm are stacked vertically next to each other to form walls or horizontally next to each other for floors, cf. Figure 5.20. The planks are connected with each other on the flat sides with either adhesives or with metal fasteners such as nails or dowels. Wooden hardwood dowels can also be used. While the planks themselves provide the compression or bending strength, the connection (either the glue or the fasteners) must be designed for the shear. Both prefabrication or fabrication on site are possible. The thickness of the wall or floor equals the breadth of the planks and is commonly between 80 and 240 mm [21, 122].

Figure 5.20: Corner detail of a stacked plank construction [21, 122]

In a similar fashion, glulam beams can be used to form massive walls and floors. The individual beams are then joined using a tongue and groove connection [21, 176].

Finally, today's most important massive timber method is the CLT construction. The properties of CLT and its advantages compared to solid wood have already been discussed in chapter 5.1.4. In contrast to the other massive timber methods, the CLT elements take on all necessary structural functions in one single element. As floor elements they also allow biaxial load carrying, which makes more effective structures possible. CLT elements are exclusively prefabricated and often adapted to the specific project concerning openings for windows or technical installations. The assembly on site is therefore very simple, only a minimum of connections is required. An example of a CLT-construction is shown in Figure 5.21.

Figure 5.21: Example of a CLT-construction: evangelic church in Regensburg, Germany [5]

5.4. New Developments

In the previous chapters about the timber design basics, both the basic timber materials and products, the connections to join those products and finally whole timber structures have been discussed. To complete this chapter, a small outlook into the future of the timber design shall be given by examining today's research and new developments.

In accordance with the structure of the previous chapters, first some new wooden materials or products shall be described, followed by new connection technologies and newly developed structural methods.

One new material is moulded wood, which is made from solid wood that is compressed by about 30 %, then cut into poles, which can finally be glued together again to form many different shapes. With this technique, e. g. circular hollow tube cross-sections become possible. The goal is to manufacture highly efficient shapes in the style of steel cross-sections [20, 28].

Another new material can be manufactured by pressure treatment of solid wood and is called pressed wood. The basic solid wood is heated to 130 °C and compressed at 5 MPa. Thus, the cross-section is reduced by about 50 % by eliminating most of the pores. The tensile and compressive strengths parallel with the grain are doubled, the bending strength increases by a factor of 2,5 and the shear strength by a factor of 1,7. Some of the disadvantages that are still being worked on include a higher risk of splitting and less beneficial properties concerning glued joints [20, 26-28].

Connections are maybe the field of timber design where most innovations are developed today. Different manufacturers bring many new products onto the market, a selection shall be presented here. The ways that already established timber connections are designed and used also change.

Diagonal screws have already been mentioned and are a good example of how already established connectors are used in new ways. Since they are subjected to tension instead of shear, the load-bearing is much more effective. They can even be used to connect two members that do not touch each other, e. g. because there is a thin layer of insulation between them [20, 16].

The environmental problems with synthetic adhesives have also been discussed. The research focuses not only on using more natural substances but also on optimising the time it takes to manufacture such a connection. Today's standard glues need about 20 minutes open time (the time between the application of the adhesive and the joining of the two components), 15 minutes press time (when the cramping pressure is applied), and only a very short resting time afterwards. New polyurethane adhesives only need 5 minutes open time and 2 minutes press time, which allows for a more productive manufacturing and makes the manufacturing of glued joints on site possible [20, 17]. An example of a product that is already available is the on-site finger joint developed by HESS [6].

Still under development are glued connections that do not use artificial adhesives at all, but rather make use of the wood's own glue, the lignin. This method is called wood welding. Applying frictional heat, the lignin inside the wood liquifies at temperatures above 200 °C and can then be used to glue components together. It hardens in just a few seconds [20, 24-25].

Glued-in steel tubes and steel plates are connectors that can be used for new kinds of joints. The steel tubes are around 50 mm in diameter and 125 mm long and are glued into the wood parallel to the grain, which allows for an optimal load transfer adapted to the woods characteristic structure. Steel plates work like tongue and groove joints and can be assembled on site, secured with bolts [20, 21].

Another new development that also considers environmental aspects are dowels made from hardwood, trying to minimise the use of environmentally questionable steel. Hardwood dowels can be used as substitutes for steel dowels, e. g. to connect secondary beams to main beams or in the stacked plank construction, as mentioned earlier. Hardwood dowels have already been used successfully, but they are not yet included in the EC5. It is possible to reach the same load-carrying capacity as with steel dowels, although the failure is generally more brittle [20, 22].

When it comes to whole timber structures, new developments include composite structures that feature both wood and another material as main structural materials. Timber and concrete can be combined to form timber-concrete composite floors, where the wood is mainly responsible for the tensile stresses while the concrete takes on the compression stresses. In comparison to floors that are purely made from wood, the composite floors have much better sound protection and a higher stiffness. The higher weight can contribute to improve the eigenfrequency of the whole building. That is particularly interesting for multi-storey buildings, where wind-induced vibrations can lead to dangerous resonance effects. According to the basic equation for the eigenfrequency of a structure, $\mathrm{f} = \sqrt{\mathrm{K}/\mathrm{M}}$, where K is the stiffness and M the mass of the system, a higher mass can be used directly to decrease the eigenfrequency and get it out of the range of the excitation due to wind turbulences. In addition to those advantages, the concrete also provides a very good fire protection.

Concrete can be used for different kinds of timber floors, e. g. joist floors or stacked plank floors, hollow-box floors where the cavities are partially filled with concrete are possible, too [21, 180-181].

The disadvantages of such hybrid systems mostly come from the different properties and the different behaviour of the used materials, e. g. concerning temperature deformations, creeping and shrinkage. The structure must be designed in a way that allows for relative movements between components with different materials to avoid restraint stresses. In general, hybrid systems are also more expensive because different trades are required on the construction site.

5.5. Summary of the Case/Materials

The basic properties of wood as a constructional material have been explained, most of those properties are either directly or indirectly linked to the fact that wood is a natural material. In comparison to concrete and steel, wood has a lower strength, which can, however, be compensated by its low self-weight, which allows for efficient, light-weight structures. A challenge with timber is its low Young's modulus concerning deformations and vibrations. In terms of a design based on EC5, the serviceability limit state, which deals with those aspects, often becomes decisive. This will be picked up again in the next part.

Further characteristics of wood include that its strength depends on the load duration and moisture content inside the wood and that it can decay. Durability is therefore important to consider in the design from an early point onwards. The biggest advantages of timber buildings in comparison to those made from steel or concrete are environmental ones, this has been discussed in detail considering aspects from LCA.

Modern structures and new developments carry the timber technologies further and make them competitive again. Different solutions have been presented. Engineered wood products like glulam, plywood or CLT play a central role in today's timber constructions. Concerning the design of the overall structure, it can be concluded that modern timber framing structures, CLT mass timber constructions and panel constructions are best suited for multi-storey buildings. In general, and especially for buildings with many storeys, clear load paths are helpful and allow for a more efficient design.

Based on the findings and conclusions from this chapter, the next chapter will be dedicated to the design and analysis of three different variants of the timber building Treet.

6. Method

6.1. Procedure

While the purpose of the previous work was to create an objective, up-to-date summary of timber technologies, pointing out aspects that are especially interesting for multi-storey buildings, the further work will be more experimental. By means of the design, new insights into the described construction methods shall be derived. The goal is to make some small contributions to take the building with timber into the next stage.

Before beginning with the design, first the procedure that was followed during the design shall be presented.

The first decision that had to be made was which kind of structures should be analysed. In the summary of the case/materials (cf. chapter 5.5), three structures have already been pointed out. Consequently, the following three design variants were defined:

a) frame construction [8]
b) panel construction
c) CLT construction

Another decision, which is closely related to the first one, was to only analyse pure variants, i. e. structures that use only one single structural method. At the end of the design of the different variants, one of the main conclusions will be that all the construction methods have advantages and disadvantages and that the best solution will be a combination of the above that makes use of each method's characteristic advantages. However, how the final result will look like heavily depends on the specific project, but the intent is to find conclusions and give recommendations that are as general and apply for as many different projects as possible. Besides, analysing only the pure variants makes it much easier to carve out each method's own characteristics.

The subsequent design followed three main steps:

1. draft design
2. preliminary design
3. EC design

The draft design started once the design variants were defined, first drafts were made to get an idea of the overall structural system. Some important details were also already planned to check the general feasibility of the general system. The structural concepts for the three variants will be laid out in the next chapter 6.2.

The purpose of the preliminary design is to calculate first estimates of the dimensions of the components and choose which materials and connections shall be used. The documentation of preliminary design can be found attached to this document, cf. appendix A, and is discussed in chapter 6.3.

The EC design is required to prove the feasibility of the whole construction based on the EC5 standard. The documentation of the EC design is also attached (cf. appendix A). Some comments on this phase of the design are described in chapter 6.4.

For this master thesis, more emphasis is put on the general design, i. e. design phases 1 and 2, and less on the final detailed design. The goal is not to design a building in all details, but rather to

[8] In accordance with the terms used before, variant a) is a modern frame construction. For reasons of better readability, this variant will in the following chapters simply be referred to as frame construction.

check if the construction is feasible and to analyse the overall characteristics. For those purposes, a very detailed design is not necessary and could even distract from the more important aspects.

Since a multi-storey building is a complex structure, FEM was used for both the design and the general analysis of the structure. Moreover, as mentioned before, since connections are such an important part of any timber construction, a selected connection shall be examined also using FEM. The FEM calculations will be described in chapter 6.5.

6.2. Design Variants

6.2.1. Structural Concept

Treet, the multi-storey timber building that is the object of this analysis, has already been presented in the introduction (cf. chapter 1). Here, first the original building shall be described quickly to create the basis for the draft design of the three variants.

Treet has 14 storey and an underground car park, its ground dimensions are a/b = 20,65/22,34 m, the highest point lies 47,48 m above the ground (without the carpark). The building is exclusively used for residential purposes. Each storey consists of four to five apartments that are connected by a corridor. A big staircase and a lift in the middle of the building connect the storeys vertically. A secondary staircase is added at the side of the corridor. The architectural plans of the building can be found in the attachment (cf. appendix A).

Treet consists of rectangular modules which are placed next to and on top of each other. Those modules are prefabricated and are fully equipped including a kitchen and a bathroom. A framework made of glulam on the outside and in-between the modules is responsible for the overall stability. The fifth and the tenth storey are a so-called power storeys, featuring additional glulam bracings to achieve a high stiffness so that those storeys can act as a platform on which the next modules are placed. Prefabricated concrete slabs on top of the power storeys provide extra weight to control the wind-induced vibrations by adapting the eigenfrequency of the building. The stairs are made from CLT [14].

To be able to concentrate more on the general structure than project-specific details, some simplifications of the layout of the building were accepted (cf. architectural plans):

- The 15. storey is considered to be a full storey, only the area of the corridor, the lifts and the staircases protrudes one storey higher.
- The balconies are neglected.
- The doors and windows, also those in the outside walls, are considered to be of the same height as one storey, the parts of the walls above and below those openings are neglected. This is especially important for the analysis of the walls for the panel and the CLT construction (the length of each wall is calculated as the distance between two openings).
- The staircases and the lift are neglected and considered as floor area with the corresponding live load.
- A constant height for all storeys is assumed. It is calculated as the average height per storey of the original building.
- The room for the lift and stairways is left open from the side, so that the lift can easily be added at the end, after the main structure is finished.

With those simplifications, the structural concepts of the three variants were worked out. General important criteria for a good design, that should be considered from the first stage on, are (in no special order):

- intended usage
- structural safety
- architectural value
- economical aspects
- environmental aspects
- fire safety
- thermal protection
- noise protection
- possibility of prefabrication
- suitability for extensions
- laying of service installations
- required maintenance
- erection methods
- deconstruction

Concerning the choice of materials, the same basic materials were used for the frame and panel variants (glulam for the main structural components and plywood for the structural sheathing) to make a direct comparison more valid. Plywood is used instead of OSB because of the rather high VOC emissions of OSB (cf. chapter 5.1.4). The massive timber construction uses a different material (CLT), since the material is in this case one of the aspects that shall be compared.

6.2.2. Frame Construction

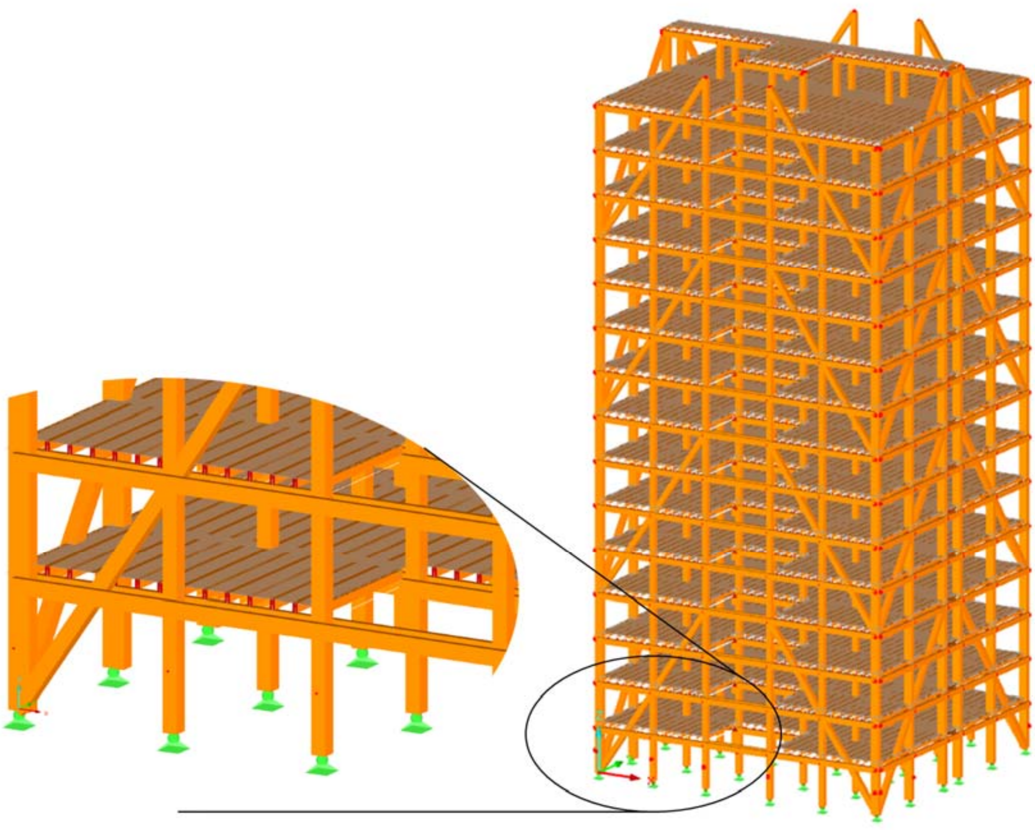


Figure 6.1: RFEM model of the frame construction

As depicted in Figure 6.1, the frame's load-bearing structure consists of columns that go continuously through all storeys, two-part beams that are attached to the columns on either side and diagonals in the outside walls in-between the columns. Thus, large frames are created on all four sides of the building that carry the global bending moment due to the wind loads.

In an earlier stage of the design, it was first planned to have the diagonals on the outside of the load-bearing structure. This would have had a higher architectural value because the diagonals would have been visible from the outside. However, during the design, it became obvious that the required connection between the outer diagonal and the column was not possible, mostly because of the high forces combined with the eccentricity of the diagonal. The first design was therefore changed to have the diagonals in the same layer as the columns, which is advantageous for the load transfer.

The columns are placed on a six by six grid with an average distance of about 4,3 m. The continuous columns with the compound beams on the sides make an optimal load-bearing behaviour possible, because no holes are necessary for the beams, which would weaken the cross-section. Another advantage of arranging the beams on the sides is that the stresses perpendicular to the grain are minimised. As explained before, this is of great importance for multi-storey timber buildings.

The beams run parallel to the front of the building, where the grid lines have similar distances. Continuous beams with equal spans are statically more economic than single-span beams or continuous beams with varying spans.

The floor joists are placed on the beams and go perpendicular to the front. Because of the varying distances in this direction, the joists are designed as single-span beams, which can be adapted to the corresponding spans. This also improves the noise protection because the floors can be divided between apartments.

Attached to the top of the floor joists, a structural sheathing made of plywood accounts for the shear stiffness of the floors and distributes the live loads and the self-weight of the floor over the joists. The floors act as horizontal beams that transfer the wind loads into the frames on the sides.

One important factor for the design was to have as simple connections as possible. For multi-storey buildings with a very large number of connections, costly connections can lead to economic problems.

The connection of the beams to the columns is realised by using steel bolts. The bolts are loaded in double shear which makes them more efficient. The holes for the bolts can be pre-drilled in the factory, allowing for a quick assembly on site. The connection of the diagonals to the columns was more complex due to the expected large forces. The connection is realised using internal steel plates inside the wood members and steel dowels, see Figure 6.2. The big advantage of the internal steel plates is that multiple plates can be used, creating two shear planes each, which increases the capacity per fastener. To avoid conflicts with other connections (especially in the corners, where two diagonals from perpendicular directions meet at the same column), some special measures had to be taken. On the one hand, the cross-section of the column must be large enough for the connections to be next to each other. On the other hand, completely hidden steel dowels are required, so that the beams can continue along the side of the connection. At the middle columns, the steel plates simply go through the column, directly connecting the two diagonals with each other.

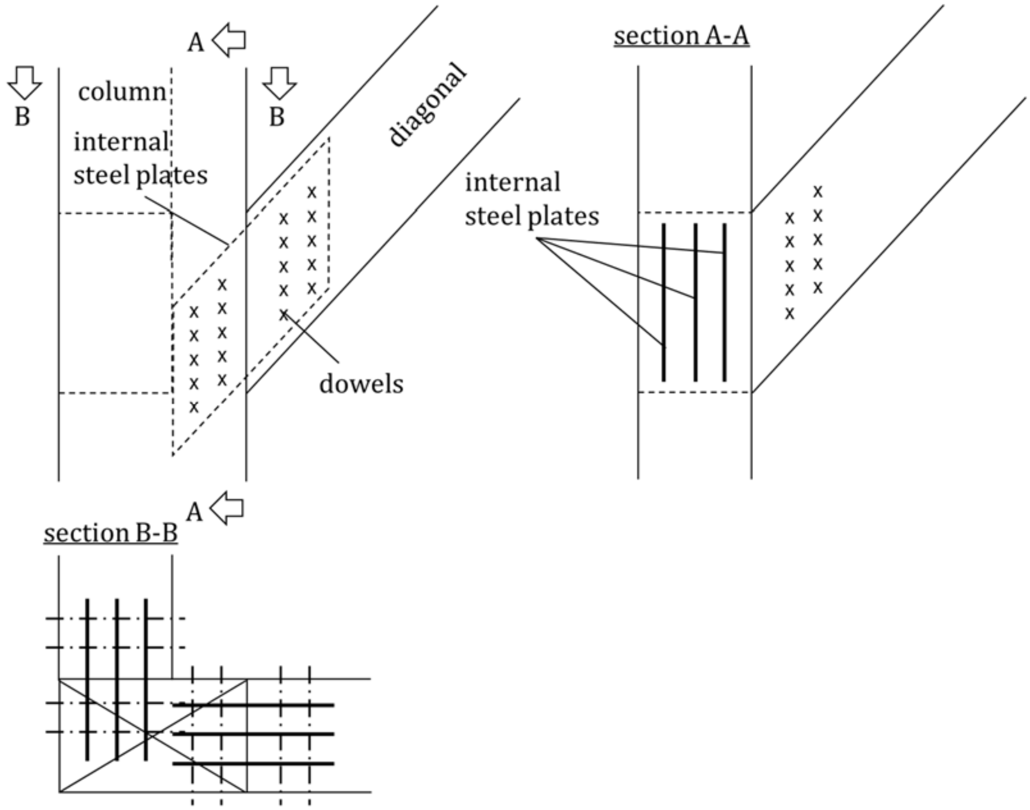


Figure 6.2: Sketch of the connection between the column and the diagonal (not to scale)

One challenge was that the force from the diagonal acts on the column under an angle to the grain. To prevent the wood from splitting, fully-threaded screws are driven into the column perpendicular to the grain as reinforcement.

6.2.3. Panel Construction

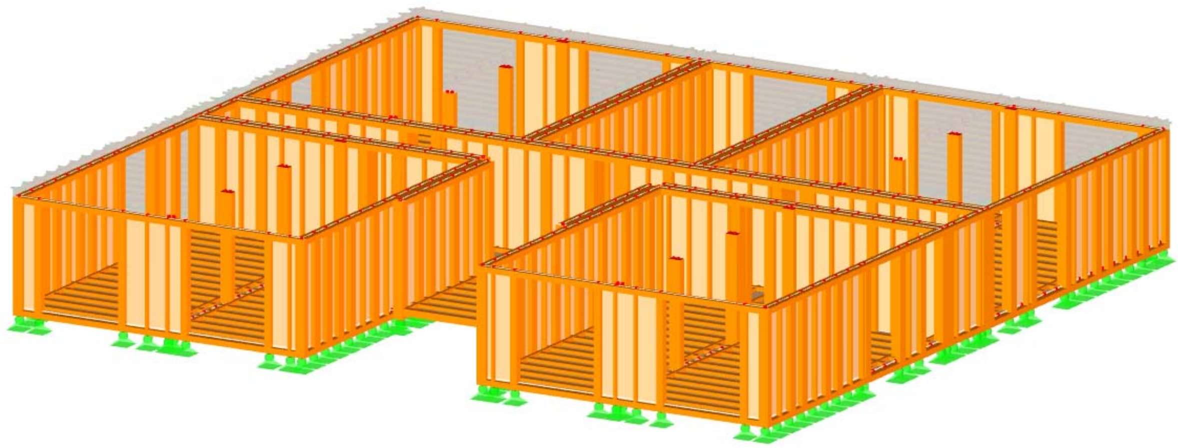


Figure 6.3: RFEM model of the second storey of the panel construction

The central concept of the panel construction is inspired by the original structure of Treet using modules. Each storey consists of 12 modules that are premanufactured, transported to the construction site and then connected. The modules consist of the walls and the ground floor, cf. Figure 6.3, and can thus be fully equipped including service installations, the bathroom and the kitchen. The upper side is closed when the next modules are placed on top.

The individual elements of the panel construction include the studs, the blocking in-between the studs, the structural sheathing nailed to the outside and inside of the walls, the floor beams, the floor joists, and the structural sheathing of the floors, cf. Figure 6.4. The studs carry the vertical loads, at the outer walls, they are also subjected to bending from the wind pressure. The floor joists carry the load from the floors and transfer them via the floor beams into the studs. The blocking stabilises the walls in the direction perpendicular to the studs. The main function of the sheathing is to carry the shear stresses in the walls. The floors act as diaphragms, transferring horizontal loads as shear forces into the walls. All members are single span beams due to the nature of the individual modules.

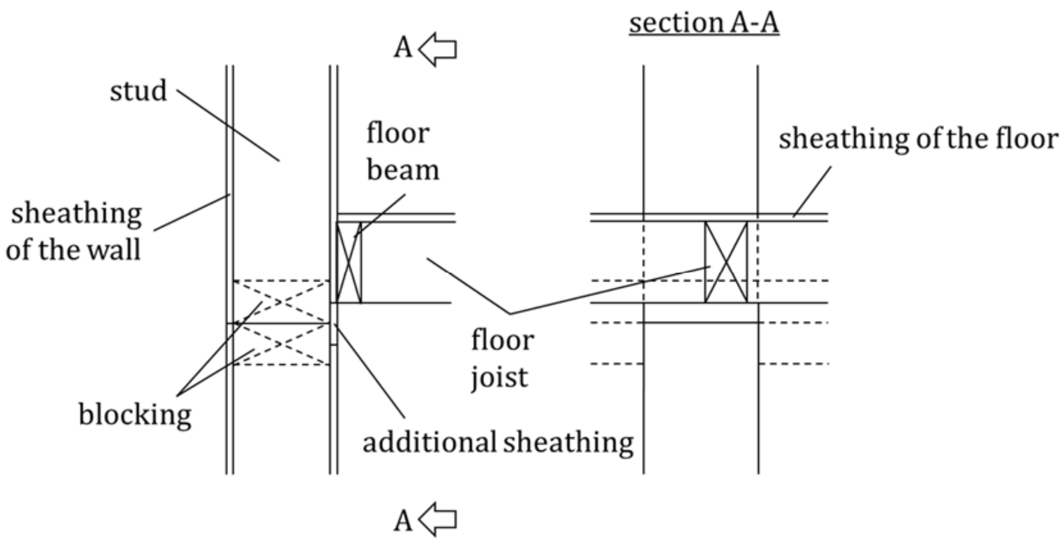


Figure 6.4: Sketch of the connection between the floor and the wall in the panel construction

In contrast to the frame construction, where loads are concentrated in the main members, the forces are more distributed within the panel walls. This makes it possible to achieve smaller dimensions. On the other hand, however, it also makes the load transfer less clear and the design much more complex.

To ensure load transfer between the modules, the connections between adjoining modules play a crucial role. To achieve a clear load transfer, it was distinguished between shear stresses and normal stresses.

The shear stresses are transferred by the structural sheathing. To achieve the load transfer, the edges of the modules are left without sheathing when manufactured in the factory, the missing sheathing is added on the site, thus effectively connecting adjoining modules.

The compression stresses are transferred via direct contact between the end grains of the studs, thus stresses perpendicular to the grain are avoided. The blocking is arranged in-between the studs and the floor beam is attached to the inside of the wall, so that the cross-section of the studs is not weakened.

To transfer tensile forces between modules above each other, special hold-down devices are necessary, e. g. bolts that connect the module walls. While for smaller structures with lower loads, it is possible to only use the nails of the sheathing to transfer all forces, for high tensile forces, as they naturally occur in multi-storey buildings, this is not recommended. A clear separation between the transfer of shear and tension is important.

The four larger apartments in each storey consist of two modules. At the open sides of those modules, beams and columns are added to carry the loads from the floors. The columns of the two modules are connected after they have been brought in place to reach a higher buckling resistance.

As can already be guessed from those descriptions, the panel construction turned out to be the costliest variant concerning the load-bearing structure. Many different components and connections were necessary and multiple iteration steps were needed to find a structure that is both capable of bearing the loads and as economic as possible at the same time. However, the major part of work can be done in the factory, which makes this structure economic again concerning the execution of the construction.

6.2.4. CLT Construction

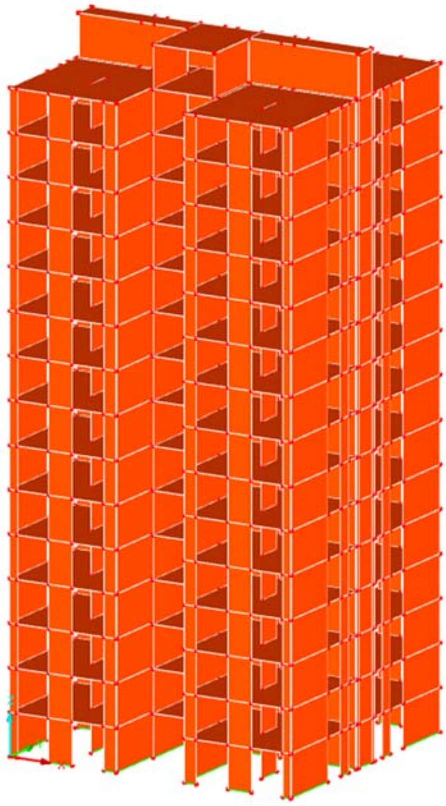


Figure 6.5: RFEM model of the CLT-construction

In this design, CLT was used in form of plates for both the walls and the floor slabs, the model is shown in Figure 6.5. Again, to avoid stresses perpendicular to the grain, instead of placing the slabs in-between the walls of the two storeys above and beneath, as is normally done in the design of smaller buildings, the slabs are attached to the walls laterally. A groove is cut into the wall elements in which the slabs are placed, cf. Figure 6.6.

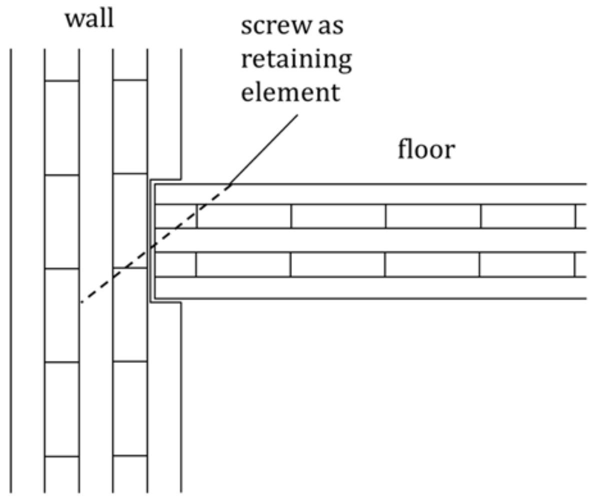


Figure 6.6: Sketch of the connection between wall and floor in the CLT-construction

Similar to the loadbearing behaviour of the panel construction, the CLT walls carry the vertical loads and the shear, while the floors act as diaphragms that transfer the horizontal loads into the walls.

One advantage of the CLT construction is its simplicity, it only consists of CLT elements and one kind of connection. This connection is required to transfer the tensile loads between walls above each other.

6.3. Preliminary Design

The goal of the preliminary design is to find a structure that fulfils the requirements from the intended usage, is able to bear the acting loads, is possible to build and as economic as possible. Since the design of a complex structure like a multi-storey building takes several iteration steps, the preliminary design can be understood as the first step in this iteration (although the preliminary design itself already takes more than one step, too). After every step, the structural elements are adapted, the corresponding loads changed, and the load transfer recalculated.

The first step in any iteration always uses some assumptions and estimates, the same is true for the preliminary design. Some examples have already been given in chapter 3.2 concerning the material properties and the loads. Another simplification is the formulae that are used, e. g. to quickly calculate the capacity of a connection, since the EC formulae can be very complex (the formulae that have been used for the design in this master thesis are described in the preface of the preliminary design documentation, see appendix A). The motivation is to find a feasible structure as efficiently as possible.

During the preliminary design phase, the most engineering competence is required. Concerning the topic of this master thesis, the most important findings were made during this phase. Those findings shall be described in the following.

The preliminary design of the frame construction was relatively straightforward, most structural elements could be designed independently. One exception were the frames in the outer walls that carry the global bending moment due to the wind. Since the frames are statically indeterminate, the cross-sections directly affect the load distribution. Therefore, the forces must be estimated as a first sub-step. A simplified system was used to determine those first estimates, cf. Figure 6.7. The whole frame was divided into two half frames, which were considered to act like Bernoulli-beams. This allowed for the simple calculation of the reaction forces in the columns, which functioned as the tension and compression chords of the beams. Based on the forces in the columns, approximate forces in the diagonals could be determined. With those estimated forces, the required cross-sections were calculated, which could then be used to model the frame with the correct stiffnesses.

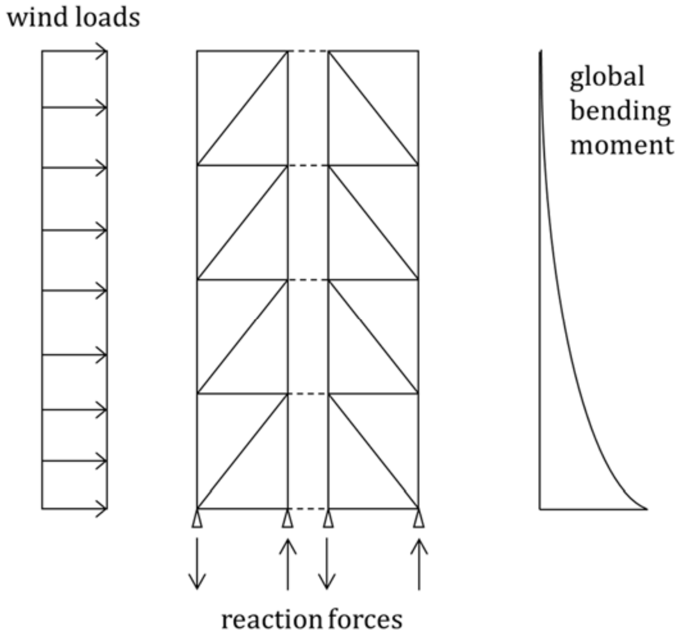


Figure 6.7: Simplified system of the frames

A big part of the preliminary design of the frame construction were the connections. The beams are connected to the columns with bolts. The connection between the diagonal and the columns features internal steel plates and dowels. To carry the wind suction loads that act on the façade, fully threaded screws were used. The sheathing, the joists and the beams were connected to each other via nails. However, although there are many different connections, all of them could be designed relatively easily using the simplified preliminary design formulae.

For both the panel construction and the CLT-construction, the stabilisation of the building was an important and not trivial part of the preliminary design. The function of the frames in the frame construction is here taken over by an interaction of the walls. In the stability analysis, the share of every wall in the global horizontal shear load and the global bending moment is calculated. Both have their maximum in the lowest storey.

In the context of the preliminary design, some assumptions were made for the stability analysis:

- The building is idealised as a vertical cantilever beam.
- The floor diaphragms are perfectly rigid.
- The stiffness of each wall is proportional to its length, i. e. all walls have the same height, thickness and shear modulus.

Based on those assumptions, the shear force is distributed over all walls according to their bending stiffnesses, cf. formula (4.1). If the shear force acts at a distance to the shear centre (which it does for wind from the side), an additional torsional moment must be considered according to formula (4.2).

$$V_k = V \cdot \frac{EI_k}{\sum_i EI_i} \tag{4.1}$$

$$V_k = M_T \cdot \frac{EI_k \cdot e_k}{\sum_i EI_i \cdot e_i^2} \tag{4.2}$$

where	V_k	= shear force of the current wall k
	V	= global acting shear force
	EI	= stiffness of a wall
	M_T	= torsional moment due to eccentricity of the global acting shear force
	e	= distance of a wall to the shear centre

To carry the global bending moment, each wall carries a share of this bending moment depending on its bending stiffness (cf. formula (4.3)) and a normal force depending on its extensional stiffness and distance to the centre of gravity (cf. formula (4.4)).

$$M_k = M \cdot \frac{EI_k}{EI_{tot}} \tag{4.3}$$

$$N_k = M \cdot \frac{EA_k \cdot e_k}{EI_{tot}} \tag{4.4}$$

where	M_k	= bending moment of the current wall k
	M	= global acting bending moment
	EI_{tot}	= total stiffness of all walls according to Steiner's theorem with respect to the centre of gravity
	N_k	= normal force of the current wall k
	EA	= extensional stiffness of a wall
	e	= distance of a wall to the centre of gravity

Finally, the vertical loads due to the self-weight of the structure and the life load could be added to obtain the total loads for which to design the walls.

Concerning the panel construction, during the design of the walls, some challenges had to be overcome. As could be expected, the loads in the walls were very high, but they also varied significantly between the individual walls, which was due to their different positions and lengths. Those variations made an efficient and economic design difficult. Dimensioning all studs according to the highest occurring loads would have been highly uneconomic, while adapting every single stud to its individual load is not practical. Finally, a solution in-between those two extremes was chosen. Only a selected number of studs was considered to be loadbearing in every wall and thus given a respective larger cross-section (up to 28/28 cm), while the non-loadbearing studs could have a much smaller cross-section (e. g. 8/28 cm). The number and arrangement of loadbearing studs could be adapted according to the load distribution in each individual wall.

This method proved to be most effective, because it helped to convert the vastly different loads on the walls to more equal loads on the studs. Short walls with relatively low loads could e. g. only have two loadbearing studs, one at each end, while in the walls with very high loads, almost every stud was loadbearing (the spacing of the studs is 61 cm).

Because this still led to considerable load differences between the studs, the method was taken one step further, introducing two different wall thicknesses. In the first floor, the outer walls running parallel to the bearing direction of the floors have a thickness of 20 cm (with the cross-section of the loadbearing studs being 20/20 cm and the one of the non-loadbearing studs 8/20 cm) while all other walls have a thickness of 28 cm (here the loadbearing studs are 28/28 cm and the non-loadbearing studs 8/28 cm).

Another challenge for the panel design was the anchorage. The problem were the comparably small cross-sections of the studs, which did not allow much space for the connections. Finally, this problem was solved by using internal steel plates in the studs in question which, together with steel bolts, form a tension resistant connection between the studs. The steel plates can be pre-attached to the lower studs, including the bolts. When the next module is placed on top, the remaining bolts are added. The bolts are located above the floor, so that this work can be done independently from the assembly of the additional sheathing, which is located underneath the floor.

The anchorage was also a challenge for the CLT design. For this construction, long steel plates are used, that are placed in grooves which are cut into the long top edges of the wall elements. Equally spaced dowels ensure a continuous load transfer along the whole length of the walls. Using glue instead of dowels would also have been possible (cf. chapter 5.4), but EC5 does not yet include rules for the design of this kind of connection.

The shear in the walls due to wind loads was not decisive in comparison to the normal forces.

6.4. Eurocode Design

In this chapter, first some general findings from the EC design shall be presented, after that, again the experiences with the three different construction variants are described.

If the preliminary design was the first iteration step, one to two more steps were required in the EC design.

The second step included a detailed load calculation and the building of the FEM models according to the dimensions that had been found in the preliminary design. With the help of those models, more accurate internal forces could be calculated. The necessary EC checks were performed for all relevant structural components. The evaluation of the results showed that some components did not fulfil the requirements, while other components were only used to a small part of their capacity.

The adaption of those components was the third iteration step. The cross-sections and connections were changed so that all checks were satisfied. But because the internal forces again are influenced by these changes, the models had to be changed, the loads recalculated, and the EC checks repeated. This time, all components still fulfilled the requirements and the design could be finished.

During the evaluation of the results of the checks, it could be confirmed that the serviceability limit state, i. e. the deformations, became decisive in some cases. Concerning the serviceability, it is important to clearly define the criterion for the EC verification. For the floors in all three construction variants, the deformations are mainly connected to the comfort of the residents. There is also a risk of damage of other components like non-loadbearing walls inside the apartments. In compliance with the Norwegian national annex to EC5, the limit values for the deformations were defined as l/300 for the instantaneous deformation and l/150 for the final deformation (including creep and other time-dependent effects), cf. [3, NA.7.2].

As indicated before, only the most relevant elements from each construction were checked in the EC design, some details are only considered in the preliminary design. An example is the connection of the sheathing of the floors to the joists in the frame construction. Because of the low loads, this connection is clearly not decisive and has therefore also no influence on the remaining structure.

The design of the frame construction could be performed without the help of finite element programs, merely the simple two-dimensional framework analysis program Stab2d was used to solve statically indetermined sub-systems like the frames on the sides. Aside from that, the checks during the second iteration step revealed that the preliminary design had already produced quite accurate results. Both facts demonstrate well the clear load-transfer via mainly one-dimensional members, which makes it easier to interpret the structure and allows for a flexible design. Only for some components (e. g. the columns), the dimensions could be decreased in order to achieve a higher degree of efficiency.

In the panel construction, the connection of the floor joists to the floor beam had to be changed. The fully threaded screws, which were chosen in the preliminary design, were not able to transfer the loads because of the large required distance to the end grain. Instead, joist hangers were used that could transfer the loads without problems. A disadvantage of this solution is the price, because it can be expected that such manufacturer-specific products are more expensive than standard wood screws. Moreover, the dimensions of some components that all had the same dimensions before, were changed to different dimensions to make the design more economic. E. g. the joists with the smallest span were adapted to have a smaller cross-section than the other joists. The solution with the differentiation between loadbearing and non-loadbearing studs was very effective also with the more accurate calculations, so that the cross-sections of the studs did not have to be changed.

For the CLT construction, because of the more complicated load distribution, an FEM analysis was inevitable. This analysis showed, however, that the estimation of the forces acting on the walls in the preliminary design had been faulty (the details will be discussed in chapter 6.5.1). Therefore, the dimensions of the walls had to be increased. This can be seen as an indication for the more complex behaviour of massive timber constructions and especially the CLT material.

6.5. FEM Analysis

6.5.1. Global FEM Analysis

FEM analyses have already been mentioned several times, most modern structures cannot be designed any more without the help of FEM. It also was an important part of the design performed in the context of this master thesis. Therefore, one entire chapter is dedicated to this topic to be able to look at some important aspects of the FEM analysis in more detail.

Besides helping to verify the loadbearing capability of the overall structures, the creation of the different models also already gave an idea of how complex and costly concerning the real erection the structures are.

Before solving the models, a convergence analysis was conducted for each model to find a suitable size for the finite elements. This is of central importance since an element size that is too large can lead to considerable deviations of the solution, while too small elements lead to a high computation time. In Figure 6.8 to Figure 6.10, the results of the convergence analysis of all three structures are visualised. The selected element size for the subsequent main analysis is marked in each diagram. For the convergence analysis, as for all subsequent calculations, a linear first order theory was used.

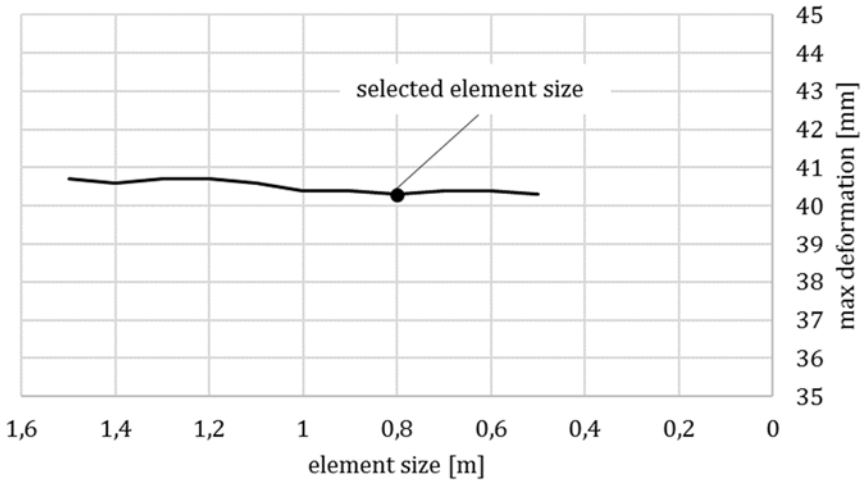


Figure 6.8: Convergence analysis of the frame construction

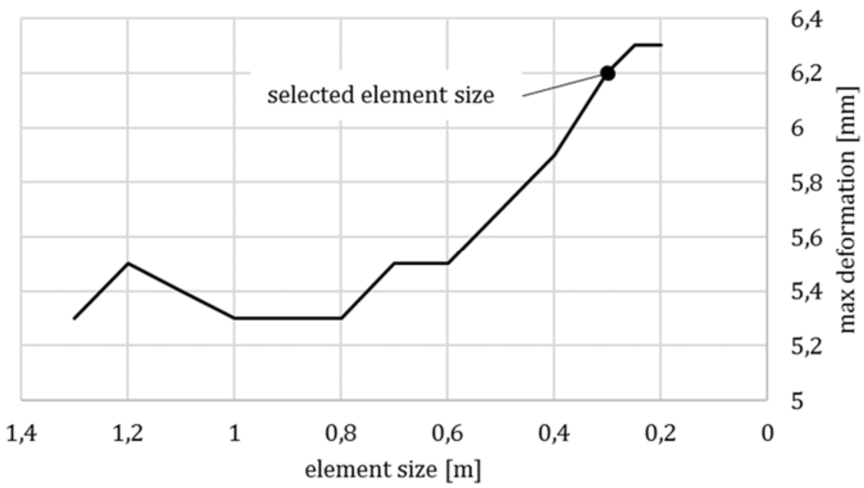


Figure 6.9: Convergence analysis of the panel construction

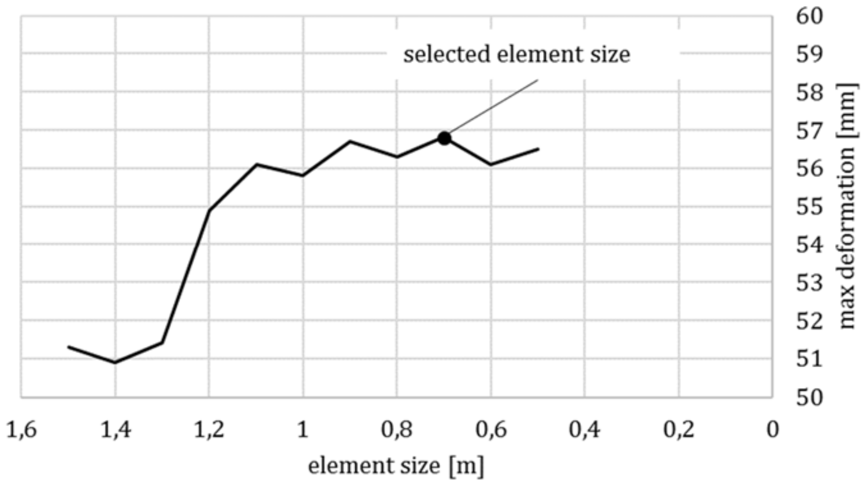


Figure 6.10: Convergence analysis of the CLT construction

For the CLT construction, the number of finite elements had a significant effect, whereas for the frame construction, the influence of the number of elements on the result was neglectable. A possible explanation for this is that the frame construction's main load-bearing structure consists of one-dimensional members, the influence of the sheathing of the floors is only secondary. The calculation of the internal forces for this framework is comparable simple and does not depend on the number of finite elements. Indeed, this structure could also be solved as a three-dimensional framework without the need for FEM. The CLT construction on the other hand consists solely of two-dimensional surface elements, which can only be solved using FEM.

As mentioned before, the FEM model was not strictly necessary for the design of the frame construction, the EC checks could also be performed using simpler sub-models. But the 3D model allows to take a closer look at the global behaviour and gives an impression of the overall structure. A comparison of the results from the design with those from the FEM-model showed that the design generally was on the safe side.

While the frame and the CLT constructions could be modelled in one piece, the panel construction consisted of too many structural elements so that the whole model could not be solved in a practical manner. Some smaller changes were made to avoid nodes that lie close to each other, resulting in an unnecessary fine mesh at those points. Still, the number of finite elements exceeded the number that could be solved in a reasonable time. To be able to solve the model, it had to be divided into smaller sub-models. The best size for the sub-models was that of one storey, the consistent numbering of the model nodes allowed for a straight-forward load transfer between the individual sub-models. The convergence analysis could be conducted for the highest storey (cf. Figure 6.9).

Since the modules consist of the walls and the floor (cf. chapter 6.2.3), the same composition was taken for the individual sub-models. Especially for the horizontal wind loads, however, the upper floor was needed for the load transfer. Because of that, the upper floor was added to each sub-model (without adding the corresponding loads again). The concept for subdivision of the panel model is illustrated in Figure 6.11.

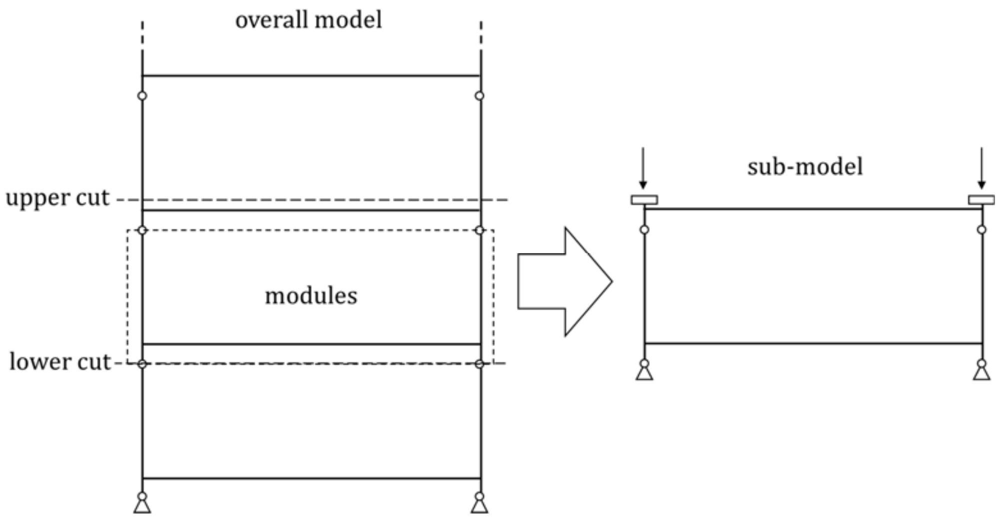


Figure 6.11: Concept for dividing the panel model into sub-models

Some additional consideration was necessary to determine the correct conditions at the transitions of the sub-models. The building as a whole was idealised as a cantilever beam. Performing cuts through this beam, equivalent to dividing the model into sub-models, will release both moments, shear forces and normal forces. Since the studs are considered to be connected to each other between storeys via hinges (i. e. without transferring bending moments), each sub-model is supported at its lower end, i. e. the lower cut, with hinged supports. The support reactions (which are equal to the internal forces in the studs) could then be transferred to the sub-model of the next storey as single loads. Because the upper floor had been added to the sub-models, the upper cut went through the studs themselves (instead of through the connection between the studs). To assure correct boundary conditions, supports were needed at the upper ends of the elements of each sub-model that prevent rotation perpendicular to their vertical axis, but allowing for free displacement in all directions. As an example, Figure 6.12 shows the model of the first storey with the loads for LC1 (self-weight) including the single forces that are transferred from the storey above.

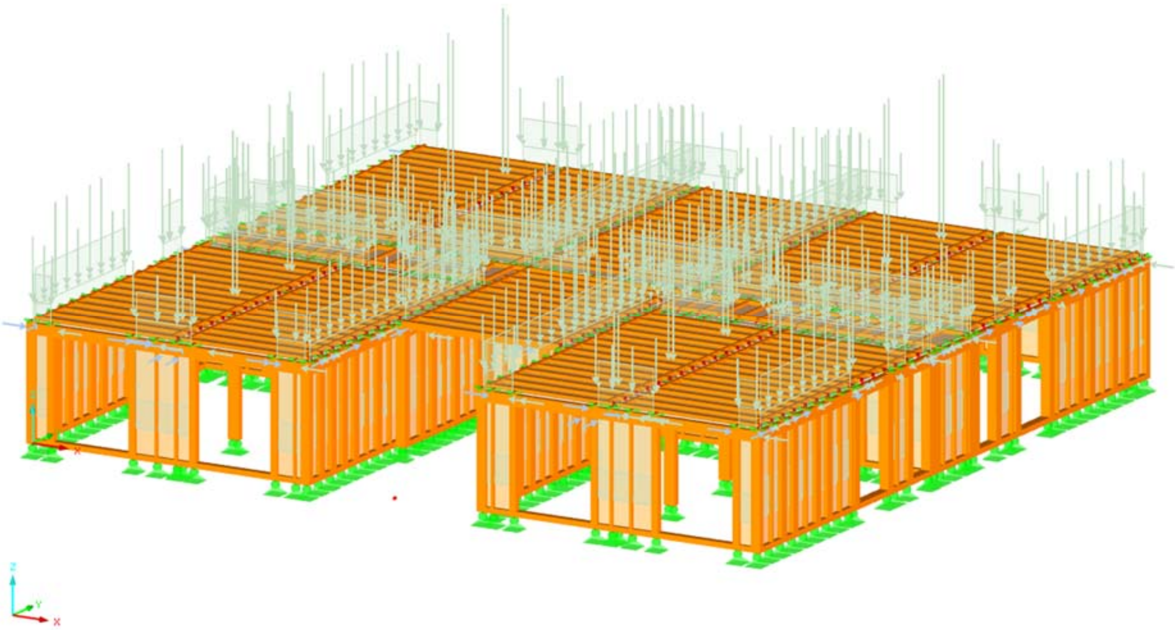


Figure 6.12: Sub-model in RFEM of the first storey of the panel construction with the loads for LC1 (self-weight)

With this method, it was finally possible to calculate reliable results.

For the CLT construction, it was possible in RFEM to define the composition of the CLT elements using the add-on RF-LAMINATE and the data from the technical approval of the selected manufacturer (see [11]). The add-on allowed for a detailed definition of the composition of the CLT-elements, it could e. g. also be specified that the individual planks are not glued together along the narrow edges.

To avoid FEM-related singularities due to surfaces that intersect each other, the model had to be adapted to leave small gaps between the walls in order to assure that the overall loadbearing behaviour correlated with the real behaviour as well as possible.

Concerning the results of the CLT model, it has already been mentioned that those results differed significantly from the results of the preliminary design. Some of the assumptions for the stability analysis had not been correct. Firstly, the building does not behave like a beam based on the Bernoulli-theory. Shear deformation has a large influence, which is probably also be an effect of the CLT-material. This can be seen from the deformation in Figure 6.13, which is not curved as one would expect for the wind loads (for a Bernoulli-beam, constant distributed loads lead to a deformation in the shape of a third order polynomial function). The floor diaphragms are also not rigid, especially because each diaphragm consists of several CLT-slabs which have hinged supports.

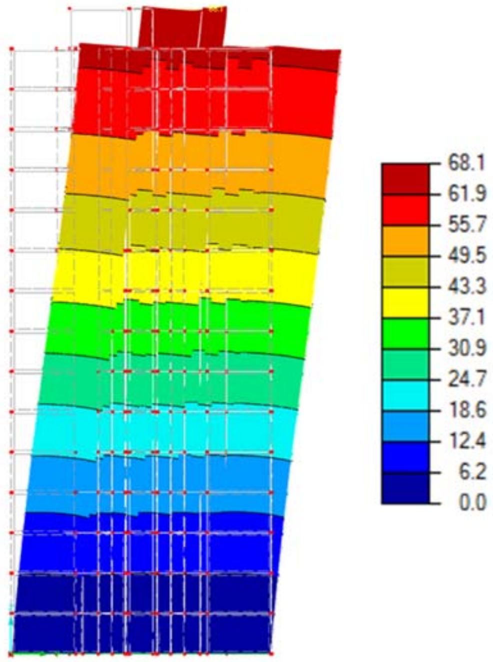


Figure 6.13: Deformation u in [mm] of the CLT-construction for LC3 (wind from the front), view from the side

When evaluating the results of the FEM analysis, precautions had to be taken because of unrealistic stress concentrations, which occurred especially at the foundation where the supports at the bottom lines of the surfaces created rigid boundary conditions. The results had to be corrected to exclude such stress concentrations.

In general, the minimum internal force per wall was selected for the design (i. e. the highest compression force). To detect stress concentrations, the difference between the minimum and the maximum internal force, the middle in-between the two, the median and the standard deviation were calculated for each wall. A high standard deviation means that the individual values on one wall deviate from each other, this is mainly the case at walls that carry a part of the bending moment from the wind loads. The middle between the two extreme values is, together with the median, best suited to detect the stress concentrations. If there are stress concentrations in a wall, only a few values will differ from the rest, so that the median is only slightly affected, while the middle value will change proportionally. Assuming negative values for compression, a middle value that is much higher than the median will mean that there are tensile stress concentrations.

As an example, wall x3 had for LC1 (self-weight) a median vertical internal force of −87,8 kN/m, with the extreme values being 32,9 and −121,1 kN/m, which means the middle value was −44,1 kN/m [9]. The large difference between median and middle value suggests that there are tensile stress concentrations. A look at the diagram in Figure 6.14 proves this hypothesis, the reason for the stress concentrations is probably the support which does not allow any deformations along the bottom line. In this case, the minimum value (−121,1 kN/m) was selected for the design.

[9] The evaluation of the internal forces was conducted at specific grid points on the surface, which were evenly distributed. Since the positions of those points were static, the results do not necessarily cover the whole range of internal forces and slight deviations can occur in comparison to the diagrams. This is beneficial, because it already helps to balance stress concentrations.

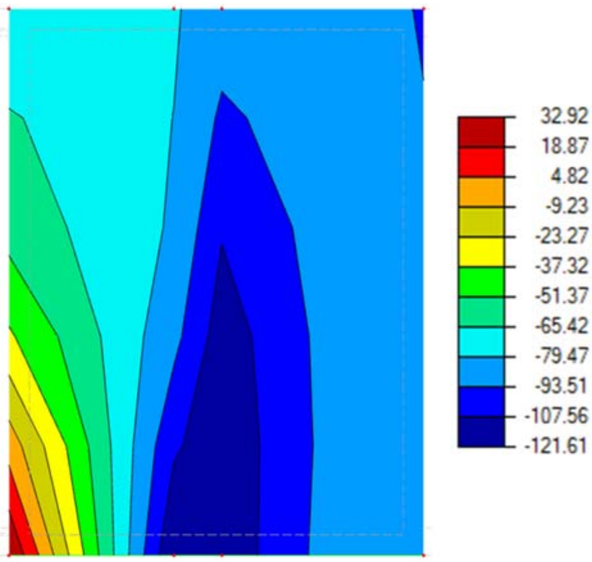


Figure 6.14: Distribution of the internal normal forces in [kN/m] in wall x3 for LC1 (self-weight)

A different example is wall x21 for LC3 (wind from the front). Here, the the minimum and the maximum value are −98,1 and −33,7 kN/m respectively, the median was −41,7 kN/m and and the middle value −65,9 kN/m, which showed the possibility of compression stress concentrations. This can again be proved by looking at Figure 6.15. Consequently, for this wall the value of the design force had to be reduced to exclude the unrealistic concentrations. A factor of 0,5 was chosen to interpolate between the extreme values, leading to the final design value of $-33{,}7 + 0{,}5 \cdot (-98{,}1 - (-33{,}7)) = -65{,}9\ \mathrm{kN/m}$.

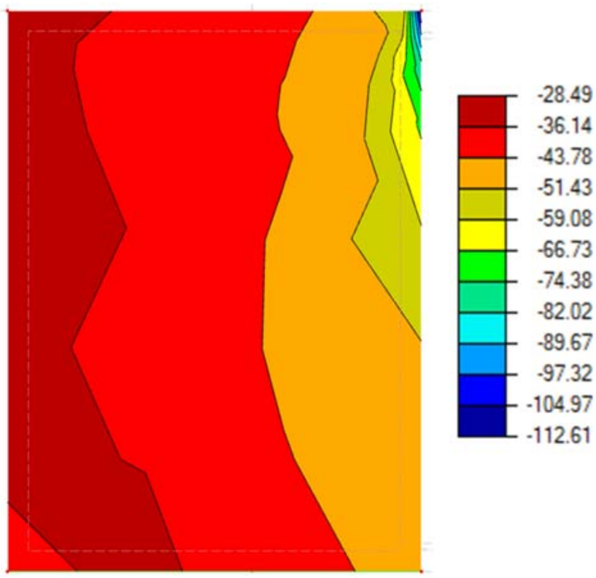


Figure 6.15: Distribution of the internal normal forces in [kN/m] in wall x21 for LC3 (wind from the front)

This procedure was used for all walls, adapting the correction factor according to how strong the concentrations were (measured by means of comparing the median and middle value).

The main results of the global FEM analysis will be presented in chapter 7.1.

6.5.2. FEM Connection Analysis

In addition to the analysis of the overall structure, one selected connection was examined to improve the understanding of the load distribution in connection details. It has already been explained that connections play an important role in timber structures, and that the rules in the EC5 are only based on single-fastener connections, using an effective number of fasteners to account for group effects. In large multi-storey timber buildings, though, connections generally consist of multiple fasteners because of the comparably high loads. In order to address this aspect, the connection between the diagonal and the column in the frame construction was selected to be analysed using FEM. It consists of the two wooden members as well as three internal steel plates together with ten steel dowels in both the column and the diagonal (cf. chapter 6.2.2). The FEM-software ANSYS was used for this purpose.

The geometry of the model is shown in Figure 6.16, in comparison to the original connection, the geometry was simplified to make a more efficient FE-solution possible, making use of the symmetry of the connection. To get a clear view of the distribution of the internal stresses, one of the middle layers of the wood components between two steel plates was modelled.

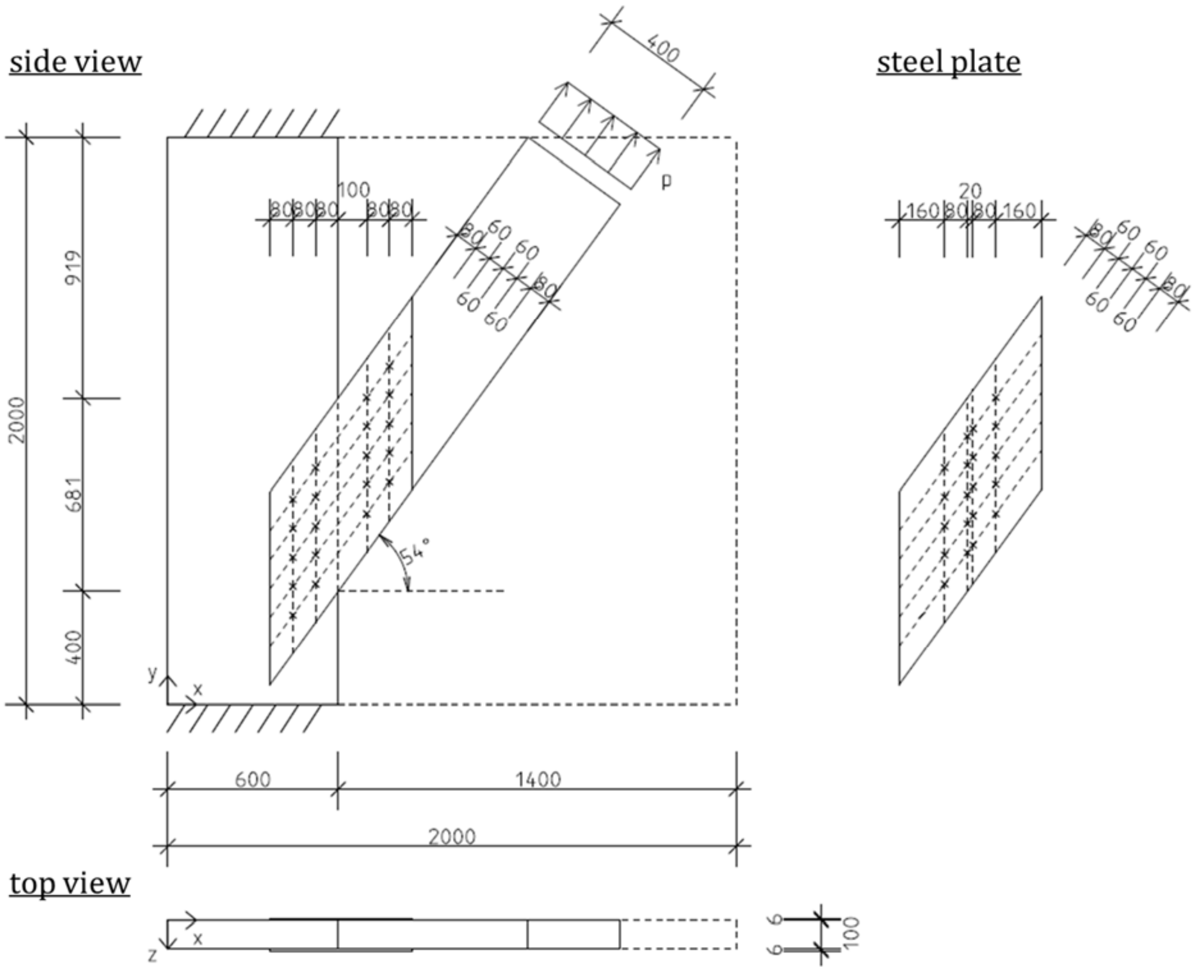


Figure 6.16: Geometry of the ANSYS model of the connection between diagonal and column in the frame construction

The wood and steel parts were modelled using the solid 8-node brick element SOLID185. To model the dowels, spring elements of type COMBIN14 were applied.

Since wood is an orthotropic material, the corresponding orthotropic material model had to be used. The values for the elasticity moduli were taken from EN 14080 for glulam GL24h. The poisson's ratios for different wood species can be found in the literature. For this application, the properties of the Douglas fir were used, which is also cultivated in Europe. The longitudinal direction of the grain was assumed to coincide with the x-axis, the radial direction with the y-axis and the transversal direction with the z-axis. All material properties are summarised in Table 6.1.

Table 6.1: Material properties for the ANSYS model [12, table 5] [16, 5-3]

wood		
E_x	11500	N/mm^2
E_y	300	N/mm^2
E_z	300	N/mm^2
G_x, G_y, G_z	650	N/mm^2
ν_{xy}	0,292	
ν_{xz}	0,449	
ν_{yz}	0,390	
ρ_{mean}	420	kg/m^3
steel		
E	210000	N/mm^2
ν	0,3	

To obtain the correct stiffness for the springs, the slip modulus of the dowels was calculated according to EC5 [2, 7.1], see equation (6.1).

$$K_{ser} = \frac{\rho_{mean}^{1,5} \cdot d}{23} \cdot 2 \tag{6.1}$$

where ρ_{mean} = mean density of the wood in [kg/m^3]

d = diameter of the dowel in [mm]

The factor 2 in equation (6.1) is due to the steel plates that are used in the connection. For M20 dowels (cf. preliminary design, see appendix A) the slip modulus per fastener and per shear plane results in:

$$K_{ser} = \frac{420^{1,5} \cdot 20}{23} \cdot 2 = 14969\ N/mm$$

Since one spring in the ANSYS model represents the action of one dowel in one shear plane, this value could be used directly as the spring stiffness of those elements.

The tensile load in the diagonal was determined in the preliminary design and amounts to 530,9 kN. Distributed over the cross-section of the diagonal of $b/h = 40/40$ cm, the resultant tensile stress is:

$$p = \frac{530900}{400^2} = 3{,}318\ N/mm^2$$

Because the wood members for the model were cut out of the whole structure, boundary conditions had to be applied at the cuts. For the column, which transfers both normal and shear forces as well as bending moments, a rigid support was modelled. The diagonal, on the other hand, transfers mostly normal forces, shear forces and bending moment can be neglected, therefore only the normal pressure p is applied to represent the action of the internal normal force. Additionally, to support the model against lateral movement, supports in the lateral directions, i. e. in global z-direction and perpendicular to the diagonal, had to be applied. This was necessary because the spring elements only transfer forces in the direction of their local x-axis, which means that movements perpendicular to this axis are not constraint. Without additional support, this causes the calculation of the model to fail because of rigid body movements.

Based on those considerations, the model could finally be solved. But it is important to keep the mentioned assumptions in mind, because the model does not represent the connection accurately enough to e. g. replace the EC check by this FE analysis. The intention is rather to gain a better understanding of the distribution of the internal stresses of a realistic connection detail.

The results will be presented in the next chapter.

7. Results

7.1. Results from the Design Analysis

The result of the design showed that it is technically possible to use all three analysed structural methods for the construction of multi-storey buildings. The design documentations and the technical plans that show the final outcome of the design are attached to this master thesis, cf. appendix A. There are, however, important differences between the three variants concerning both structural, other technical, architectural and environmental aspects. A detailed discussion of those results will follow in chapter 8. In this chapter, the results from the FEM analyses will be presented in relation to the design.

The global FEM analysis helped to verify the overall stability and loadbearing behaviour of the three structures. As an example, the internal normal forces in the front frame of the frame construction are shown in Figure 7.1. This result proves the clear load distribution of this construction which makes it a very efficient method. The overall stability is achieved relatively easily which allows for a flexible interior design.

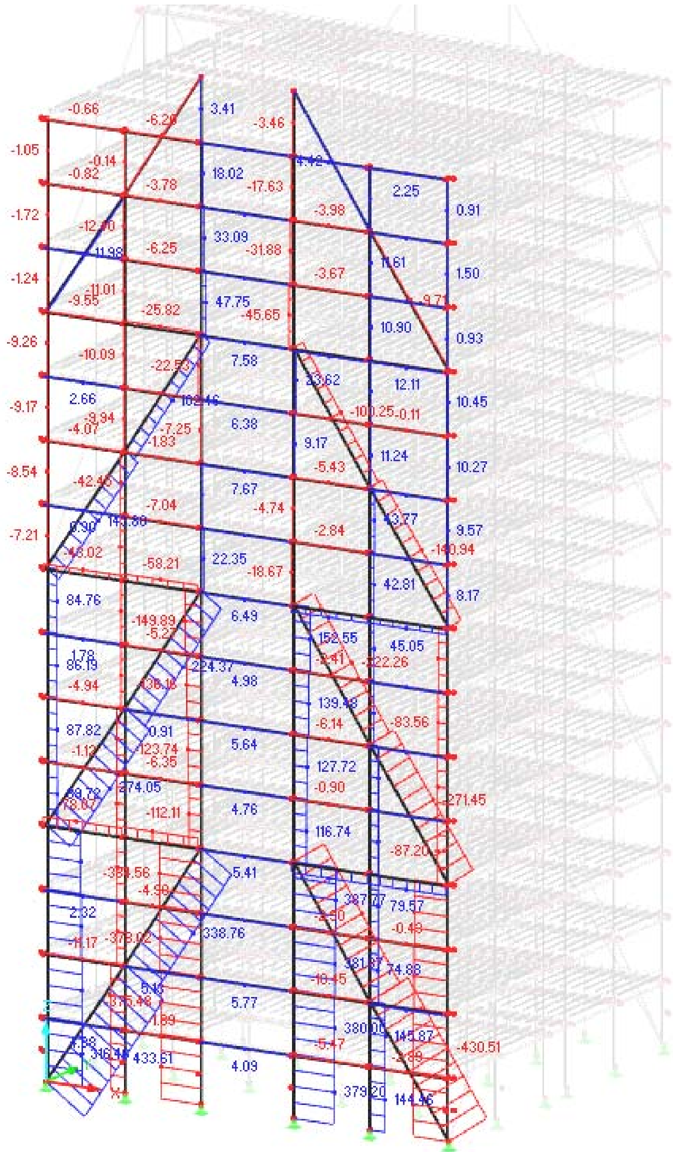


Figure 7.1: Internal normal forces in the front frame of the frame construction for LC4 (wind from the side) in [kN]

The distribution of the internal forces for the panel construction is depicted in Figure 7.2 for the first storey. Although it is not possible to read individual values, the figure still gives a good impression of the load distribution.

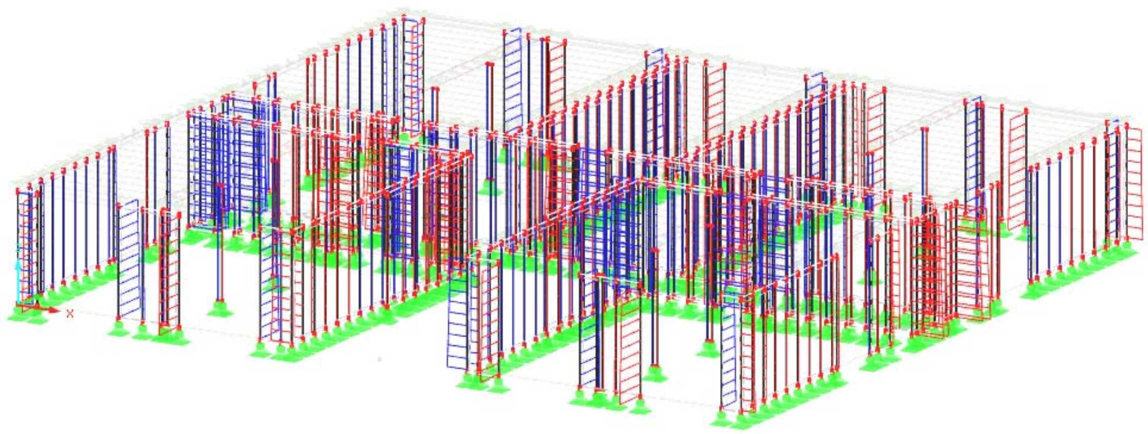


Figure 7.2: Distribution of the internal normal forces in the studs of the first storey of the panel construction for LC4 (wind from the side)

The figure shows that for pure wind loads (without self-weight or live load), the normal forces are mostly concentrated in the end studs of each wall. In contrast to that, for the preliminary design, the wind loads were distributed over the studs with the wall modelled as a beam supported the loadbearing studs, cf. Figure 7.3, which resulted in more evenly distributed loads, or rather higher loads in the middle studs. While this method is more valid for the constant vertical loads from the self-weight and the live load, it is apparently not a correct assumption for the wind loads.

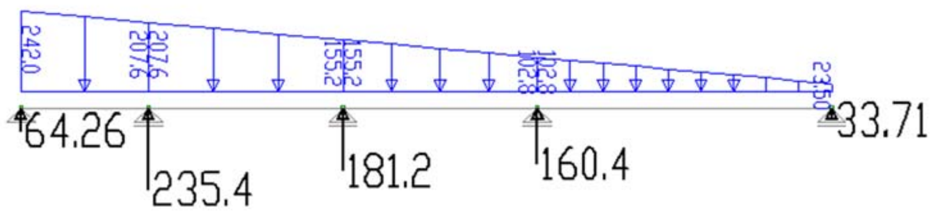


Figure 7.3: Example of the assumption of the load distribution in the loadbearing studs in a wall in the panel construction for wind from the right side according to the preliminary design

The design of the panel construction was nonetheless successful because the studs that were considered non-loadbearing provide additional load carrying capacity. This makes the design more redundant. Nevertheless, the effect of the concentrated wind loads should be considered for the design of panel constructions in multi-storey buildings with high wind loads.

To give an example of the load distribution in the CLT construction, Figure 7.4 depicts the normal forces in the walls of the first storey for wind from the side.

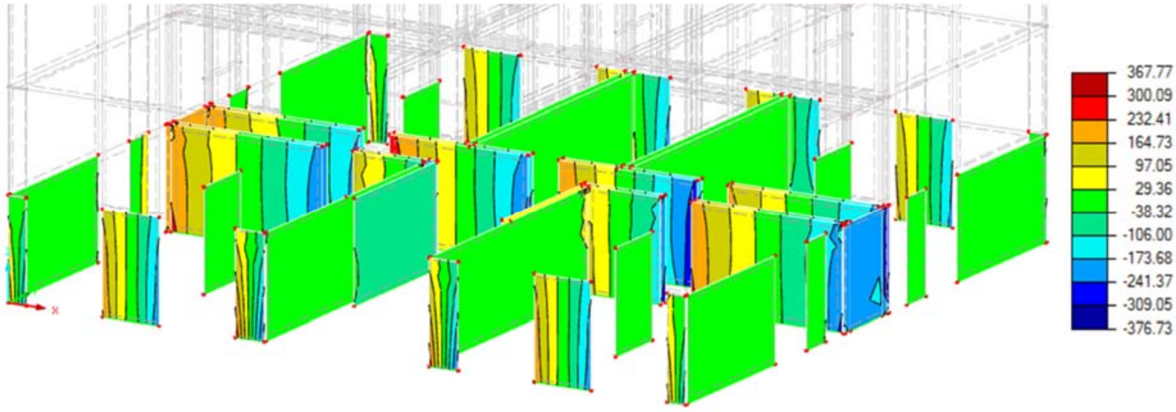


Figure 7.4: Internal normal forces in the first storey of the CLT construction for LC4 (wind from the side) in [kN/m]

It can be seen clearly that all the walls parallel to the direction of the wind carry a part of the global bending moment. The load distribution in each wall is linear according to the equation for the normal stresses $\sigma = M/I \cdot z$, where σ is the normal stresses (or, in this case, the distributed normal forces), M is the acting moment, I the second moment of inertia and z the coordinate through the cross-section (in this case along the wall). However, unlike the assumptions that had been made in the preliminary design, the walls perpendicular to the wind direction carry practically no loads. The loads in the parallel walls are accordingly higher. This is probably another reason why the EC design had to be reworked rather extensively for the CLT construction in relation to the preliminary design.

Further results for all three construction variants can be found in appendix B. The electronic files of the RFEM-analysis are also attached, see appendix A.

Another part of the design analysis was the examination of the connection detail as described in chapter 6.5.2. To get a first impression of the stress distribution, the equivalent von Mises stress is shown in Figure 7.5 [10].

[10] The deformation in all figures with ANSYS results is scaled by a factor of 100. The steel plates are not shown.

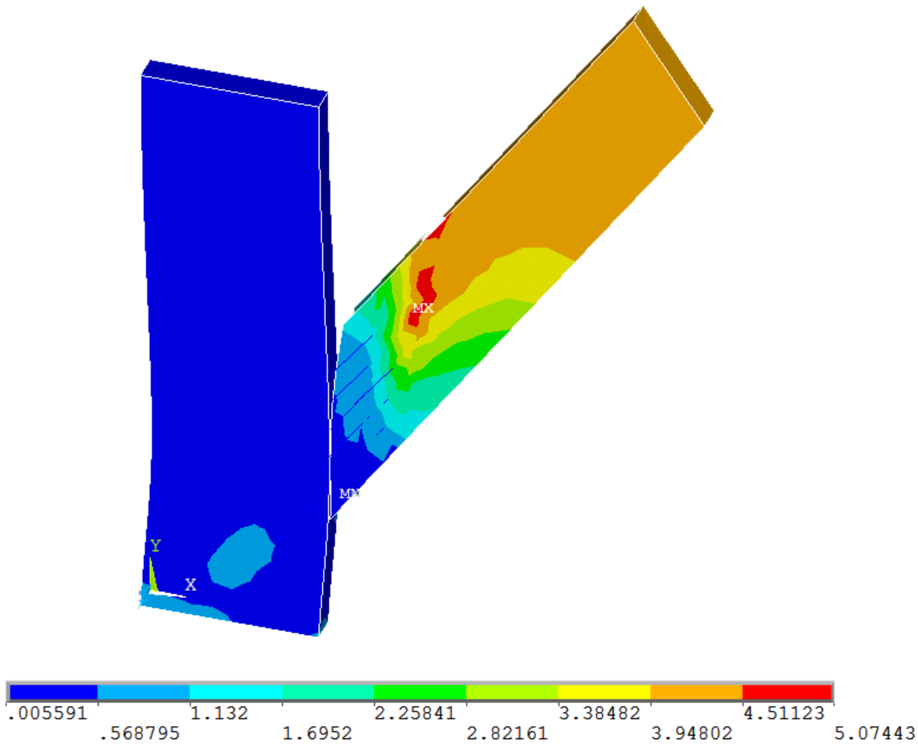


Figure 7.5: Von Mises stress σ_{VM} of the connection in [N/mm^2]

It can be seen that the stress in the upper part of the diagonal is mainly influenced by the tensile load, the von Mises stress there amounts approximately to 3,318 N/mm^2. Going down the diagonal, the stresses gradually decrease as the loads are transferred via the dowels to the steel plates and then into the column. However, stress concentrations occur in the upper range of the connection, the maximum stress there is around 1,5 times higher than the tensile load. This is most probably due to the fact that the dowels are distributed laterally along vertical lines instead of perpendicular to the grain of the diagonal. That leads to the first dowel in the upper right corner of the connection taking over most of the loads, while the dowels in the lower regions carry much less loads. The stresses in the wood develop accordingly.

The distribution of the shear stresses depicted in Figure 7.6 yields the same general result. The shear stresses are also concentrated around the upper dowels.

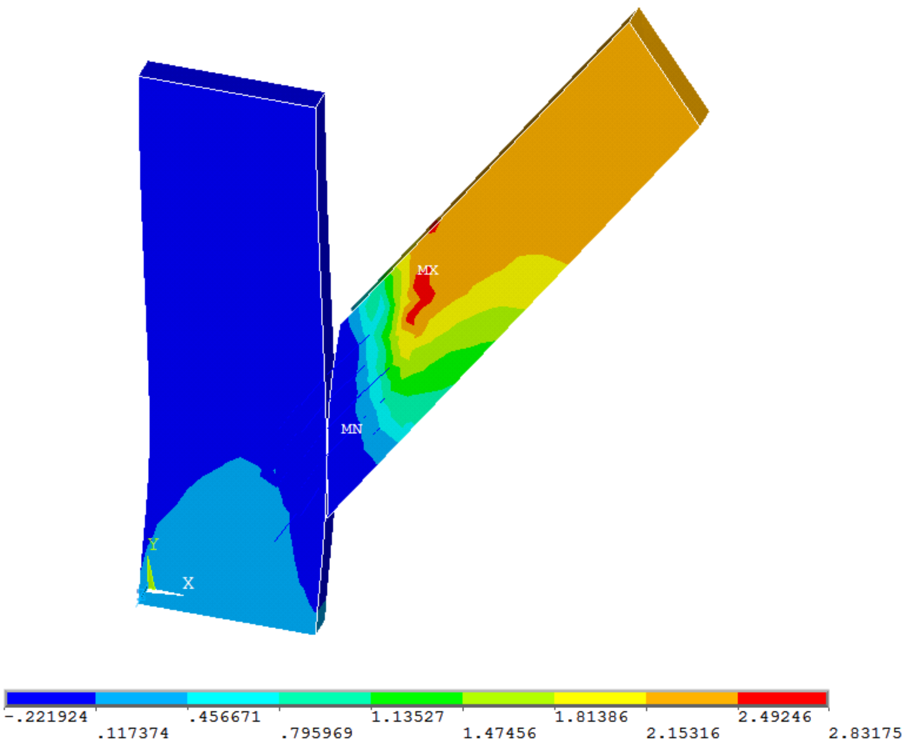


Figure 7.6: Shear stress τ_{xy} of the connection in [N/mm^2]

In addition to that, however, shear stresses also develop in the diagonal in some distance from the connection, where the local effects of the dowels have no influence any more. This is probably again due to the arrangement of the dowels, since the diagonal, as a part of the frame in the frame construction, is a pure tension member. One possible explanation is the asymmetrical load transfer from the wood into the dowels. Because higher loads are transferred in the upper region of the connection compared to the lower region, the internal forces are unbalanced. To satisfy the equilibrium of forces, additional shear forces are required.

Shear stresses along the grain are potentially dangerous because they can lead to cracks. In the EC, the check against cracks, i. e. the check for tension perpendicular to the grain, is based on the shear force in the member in question. The capacity against tension perpendicular to the grain is calculated using formula (6.2) with the geometric dimensions according to Figure 7.7.

$$F_{90,Rk} = 14 \cdot b \cdot w \cdot \sqrt{\frac{h_e}{1 - h_e/h}} \tag{6.2}$$

where w = modification factor, $w = 1$ for all connections except for nail plates

b, h_e, h = geometric dimension, see Figure 7.7

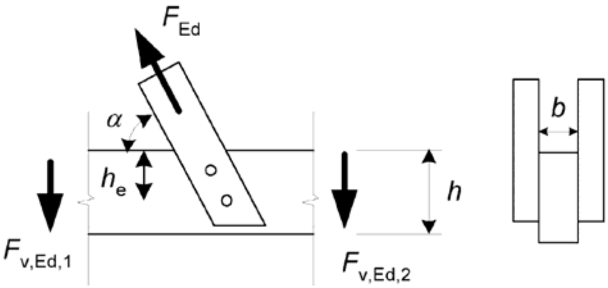


Figure 7.7: Definition of the forces and geometric dimensions for the check against tension perpendicular to the grain according to EC5 [2, 8.1.4]

Since this check is only necessary for a force that acts at an angle to the grain, it is only required for the column and not the diagonal (see Eurocode design for the execution of this check, cf. appendix A). The FE results on the other hand suggest that the shear forces are larger in the diagonal, due to the asymmetric arrangement of the dowels. This aspect is, however, not considered in the EC.

Lastly, Figure 7.8 shows the total strain inside the wood members. The results reinforce the suggestions that have been made so far. Since the strain and the stress are directly linked via Hooke's law $\sigma = E \cdot \varepsilon$, with the stresses σ, the strains ε and the Young's modulus E, the places with the highest stresses also show the highest strains. The shear deformation in the diagonal can also be seen relatively clearly when looking at the deformed shape of the model.

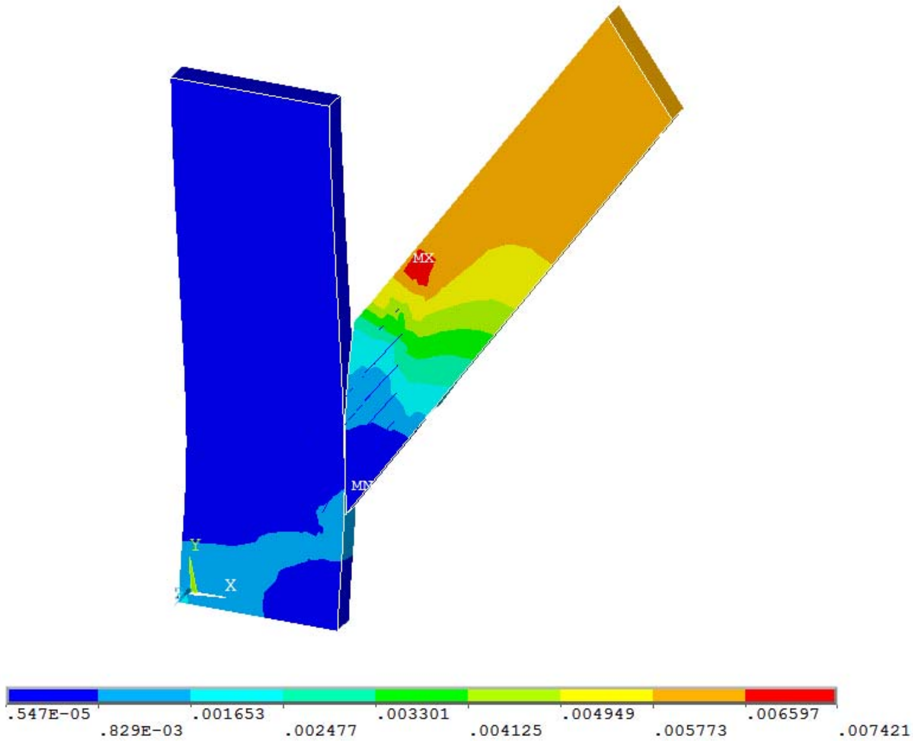


Figure 7.8: Total strain ε of the connection in [-]

Further figures can be found in appendix C, which show the same overall results. The electronic fie containing the ANSYS model is also attached, see appendix A.

To sum up, the most important insight from the ANSYS analysis is that the arrangement of the fasteners in a multi-fastener connection has a significant influence on the distribution of the stresses and thus the loadbearing behaviour and potentially also the capacity of the connection. It appears to be disadvantageous to arrange the fasteners with an offset parallel to the grain. As a result of that, the load transfer is unbalanced and additional shear stresses develop. The shear stresses raise the risk for splitting, the stress concentrations around the front dowels can also lead to cracks. But it is important to remember that this analysis was carried out without considering the splitting itself or related effects.

The Eurocode does consider the distances between the fasteners to determine an effective number of fasteners, but the offset along the grain is not included. According to EC, the effective number of fasteners for the column $n_{ef} = 6{,}74$ is lower than for the diagonal with $n_{ef} = 7{,}93$ (see Eurocode design, cf. appendix A), while the above results suggest a less beneficial stress distribution for the diagonal. In addition to the checks from the EC, the described effects should be kept in mind for the design of multi-storey timber buildings and the layout of such connections adjusted accordingly.

7.2. Comparison

To elaborate the central results from all previous chapters, a short comparison of the three variants shall be made in this chapter. As a tool for the comparison, an evaluation matrix shall be used, as was described in the context of the LCA in chapter 5.1.3. For this comparison however, a more general evaluation matrix is established, considering all the above mentioned aspects. The result is presented in Table 7.1.

First, all criteria were weighted to account for their importance. In the next step, all three variants were assigned values between 1 and 5 for each criterion, were 1 is the worst rating and 5 the best rating, that means that e. g. a low difficulty of the construction would be rated with a high value. The scale from 1 to 5 is fine enough to clearly bring out the differences between the three constructions, but not too fine, for which there is no sufficient basis. The results were calculated by multiplying the value with the weighting of the corresponding criterion.

Table 7.1: Evaluation matrix for the comparison of the three construction variants frame construction, panel construction and CLT construction

criteria	weighting	frame construction		panel construction		CLT construction	
		value	result	value	result	value	result
complexity of the structure	10 %	3	30	1	10	5	50
difficulty of the erection	20 %	2	40	5	100	4	80
difficulty of the transport	10 %	5	50	1	10	3	30
deconstruction and reuse	5 %	5	25	1	5	2	10
suitability for prefabrication	10 %	2	20	5	50	3	30
fire safety	15 %	2	30	3	45	5	75
noise protec-tion	5 %	1	5	2	10	5	25
environmental aspects of the materials	15 %	4	60	4	60	2	30
difficulty of ad-justments of the interior	5 %	5	25	1	5	3	15
architectural value	5 %	4	20	1	5	5	25
sum	100 %		305		300		370

The results reveal that the CLT construction is best suited from this all-integrating point of view. It did not get the lowest value 1 for any criterion, the worst ratings were attained for deconstruction and reuse, since the CLT elements are engineered wood products that are highly adjusted to the specific project, and environmental aspects, because most synthetic adhesives are required for the manufacturing. The frame construction and the panel construction have almost the same result, although the scoring at the individual criteria is often opposite. For the difficulty of the transport, for example, the frame construction scores high because the basic components are relatively easily available and can be transported efficiently. The panel elements, however, prefabricated walls and floors or even whole prefabricated modules, are more difficult to transport and the distance between the factory and the construction site is presumably much bigger. For the difficulty of the erection on the other hand, the panel construction gets a high value because the prefabricated elements only have to be connected on site, while the erection of the frame is more costly.

It is important to remind that this evaluation will not valid for every case, it is only intended to give a general idea of the differences, the advantages and drawbacks of the three structural methods. The decision for one or another of the methods will in addition to those results highly depend on project-specific aspects, which cannot be considered here. Moreover, the above values were only determined qualitatively based on the findings of this master thesis. For a more thorough evaluation, especially when applied to a real building project, input from experts from different fields should be considered.

8. Discussion

Apart from the aspects of the previous chapter, some more detailed results concerning the outcome of the design and the experiences with the three variants will be discussed in this chapter.

The frame construction is very effective to transfer the wind loads which create a large global bending moment at the foot of building. This is naturally a big challenge for all multi-storey buildings. Looking at existing multi-storey timber buildings today, like the original Treet construction or the Mjøstårnet (cf. chapter 1), most of them feature some kind of frame construction on the outside of the building.

Looking at the connection detail in the frame construction, one can conclude that it would have been more appropriate to separate the connection of the two diagonals that meet at the corner column. This connection is quite complex, as could be illustrated in the FEM analysis of the connection in chapter 6.5.2. Much space is needed to securely transfer the very high loads. The two-dimensional connection makes this task even more complicated. Based on those experiences, it can be recommended to always prefer one-dimensional connections, although again, the conditions can be different in a specific project.

Of all three variants, the panel construction is least suitable for multi-storey buildings. The panel construction is an efficient and lightweight construction method for smaller buildings, which has many advantages like the good suitability for prefabrication and its small weight. It is, however, not efficient for buildings with many storeys because of the very high loads that occur in multi-storey buildings. The extensive preliminary design and the issues with creating an FEM model illustrate the complexity of the construction. The load distribution is not clear, which makes an optimisation of the design difficult.

In contrast to that, the main advantage of the mass timber variant is its simple construction. Especially compared to the panel construction, but also the frame construction, less different structural elements and therefore also less connections are necessary. The mass timber elements combine the different structural functions in an effective way.

However, the preliminary design had to be reworked comparably strongly for the CLT construction. This is obviously due to inadequate assumptions for the preliminary design, e. g. concerning the strength or the Young's modulus. In the preliminary design, it was not accounted for the different material parameters in the different directions of the plates, although those actually differ significantly.

One reason for those problems is the lack of experience with this material. Adequate assumptions (which are always based on experience) are not available yet as extensively as for the other construction methods. Another issue for the design of the CLT construction is that standardised rules for the design of CLT are still missing. In the EC, CLT is not mentioned at all. Assumptions had to be made, but because of the advantages of CLT that were pointed out earlier (above all the superior homogeneity), there is potential for e. g. higher modification factors $\mathrm{k_{mod}}$ which would increase the efficiency of this material (see also [19]).

On the other hand, however, it was comparatively easy to adjust the dimensions of the members in question. In spite of the considerable changes that had to be made, the load distribution only changed relatively little. This proves again the simplicity of the structure. The structural elements are quite independent of each other concerning the load transfer. The simplicity of the connections was also beneficial, because connections often represent fixed points for the design, forming boundary conditions that limit the freedom of the design perceptibly.

To come back to the reason for those necessary adjustments, there is still some work and research to be done to gain more expertise about this material. Extensions to the standards are necessary to include CLT, especially because it is used more and more and will very likely play an important role in the future of multi-storey timber buildings, as was discussed earlier. The FEM-software RFEM already included an add-on to model CLT-materials realistically. This can be seen as a sign that the industry has already understood the importance of this material and is using it, while the research and standardisation still have to catch up. CLT has also been used in different extend for the multi-storey timber buildings Treet and Mjøstårnet.

It can be seen from the outcome of this master thesis, that all three variants have advantages and drawbacks. This is not surprising and had already been indicated in chapter 6.1. To round up the conclusions of the design analysis, a short look will be taken at how to combine the different variants effectively.

Since the frame is very well suited to take over the wind loads, it makes sense to use this structure on the outside of the building for the global stability. On the inside, modules in panel construction can be used. To minimise the loads on the panel walls, it is possible to connect the modules to the frames to transfer the vertical forces there. This can either be done with every storey or with several storeys, creating platforms at equal intervals, very much like the original structure of Treet was carried out. This makes it possible to make use of the panel construction's advantages. Nonetheless, higher loads will probably occur in the panel walls compared to smaller buildings. Concerning this, the solution with a distinction between loadbearing and non-loadbearing studs has proved to be very effective.

A CLT construction is very well suited for multi-storey timber buildings without additional structures. For even bigger buildings, constructions in the style of traditional monolithic concrete structures can be imagined, with an internal massive core, providing the stability. In this way, CLT can also be combined with other materials, e. g. a lightweight panel construction to form the outer walls. Furthermore, floor slabs made from CLT are well suited to be used in different constructions, one of their advantages compared to e. g. joist floors if their capability of bearing loads in both directions, which makes them much more efficient. Because of their massive structure and high weight, they also add a good fire and noise protection.

9. Conclusions

In this master thesis, an extensive basis for modern timber design was established and could successfully be used to perform a design analysis of three variants of the multi-storey timber building Treet. One variant was a frame construction, one a panel construction and one a CLT construction. From the experiences during the design and from the final outcome, conclusions could be made concerning the suitability of those three variants, thus giving a versatile answer to the central research question (question number 1) that was formulated in chapter 4. Some of the most important findings are:

- A frame construction is well suited to provide stability in multi-storey timber buildings.
- A panel construction should only be used in combination with other construction methods, but then it has some great advantages, above all the good suitability for prefabrication.
- CLT can be used for various purposes in multi-storey timber buildings, either on its own or in combination with other materials and structural methods.
- A clear load transfer is beneficial, especially for the optimisation of the design.
- Engineered wood products like glulam and CLT are practically inevitable for multi-storey timber buildings.
- Stresses perpendicular to the grain should be avoided.

To give answers to the secondary questions, here are the further important points, gathered from all chapters of the master thesis:

2. Wood is the oldest material used for building purposes. Much experience exists for timber constructions, but today's technologies differ considerably from those used earlier in history. There are still many reservations amongst both clients and planners in the building industry, which are connected to the disadvantages of the traditional technologies, although the new technologies solved most of those disadvantages.
3. In spite of the different historical developments, in both Norway and Germany, as in many other countries, the timber industry lacks expertise for those new technologies.
4. Supported by programs like the Norwegian strategy to promote the use of wood in public buildings, however, the timber industry is developing.
5. Wood is a natural material with all the advantages and drawbacks that are connected to this fact. Its greatest advantage compared to steel and concrete is probably its environmental properties. But wood has structural advantages, too, like the low self-weight.
6. The role connections play in the design of timber buildings depends on the construction method. For the frame and the panel construction, the connections play an important role, a large part of the design documentation is dedicated to the connections. The CLT construction on the other hand only requires a minimum of connections.
7. The design of the panel and the CLT construction, like most modern engineering constructions, required the use of FEM-software. Special caution is required when evaluating the results because of model-related effects that can influence the results unrealistically. The frame construction could be designed only making use of a simple framework analysis program.
8. The FEM-analysis of the connections showed that the layout of the fasteners in multi-fastener connections has an important effect on the stress distribution and is not completely considered in the EC. Generally, it can be recommended to arrange the fasteners on an orthogonal grid parallel and perpendicular to the grain to achieve an even load transfer.

10. Recommendations

As described in the introduction in chapter 1, today's highest timber building, Treet, has a height of 51 m, when the Mjøstårnet will be finished in 2019, it will have a height of 81 m. Thinking further into the future, it can well be imagined that there soon will be wooden skyscrapers with heights far above 100 m. The aspects discussed in this master thesis will still be able to be used in a general sense for such buildings, but the structures must of course be adapted. To obtain the necessary cross-sections, CLT will definitely play an important role. New products like moulded wood improve the wood's properties and will make such new structures possible. To make those new products applicable, further research is needed.

To economically design new constructions, improvements must be made to the current standards, e. g. to include CLT in the EC.

Apart from that, with this master thesis it could be demonstrated that there are today no technical hindrances to construct multi-storey buildings using timber. It is now the task of the legislative organ to provide modern, contemporary rules that support timber structures, especially considering the big environmental advantages that have been discussed. At the same time, the responsible planners in the building industry should be more aware of the technical possibilities that have been presented, and rid themselves of the reservations connected to earlier weaknesses of timber.

All in all, this master thesis showed that timber constructions still have much unused potential, also for tomorrow's buildings.

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Appendix

A. Attachments

- architectural plans of the original building Treet P01–P06 (PDF) [15]
- load calculation (PDF)
- preliminary design (PDF)
- Eurocode design (PDF)
- technical plans (PDF)
- EXCEL design files (XLSX)
- RFEM models (RF5)
- ANSYS model (DB)

B. Selected RFEM Results

B.a) Frame Construction

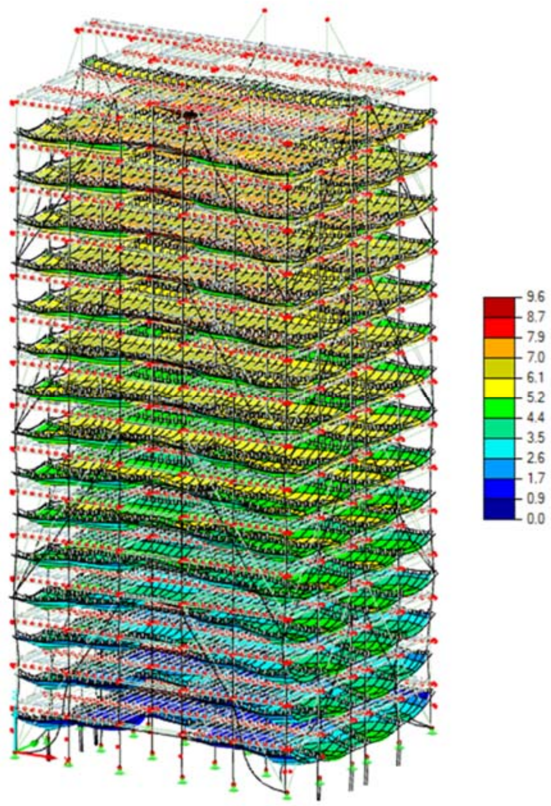


Figure B.1: Deformation of the frame construction for LC1 (self-weight) in [mm]

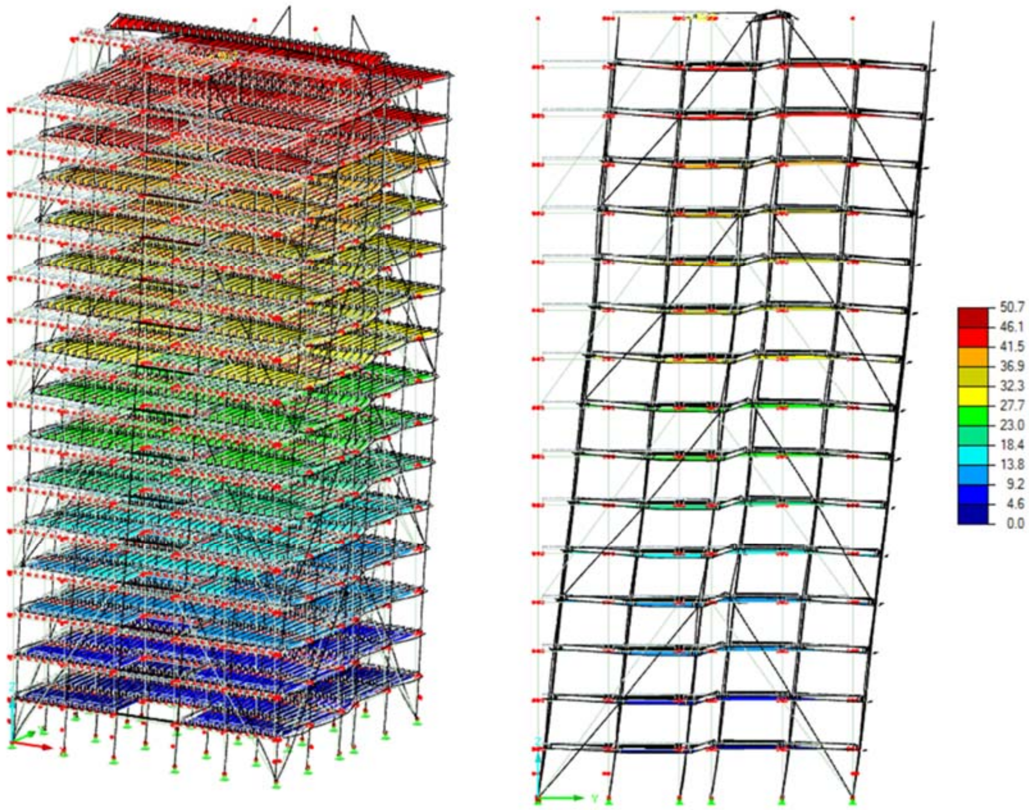


Figure B.2: Deformation of the frame construction for LC3 (wind from the front) in [mm]

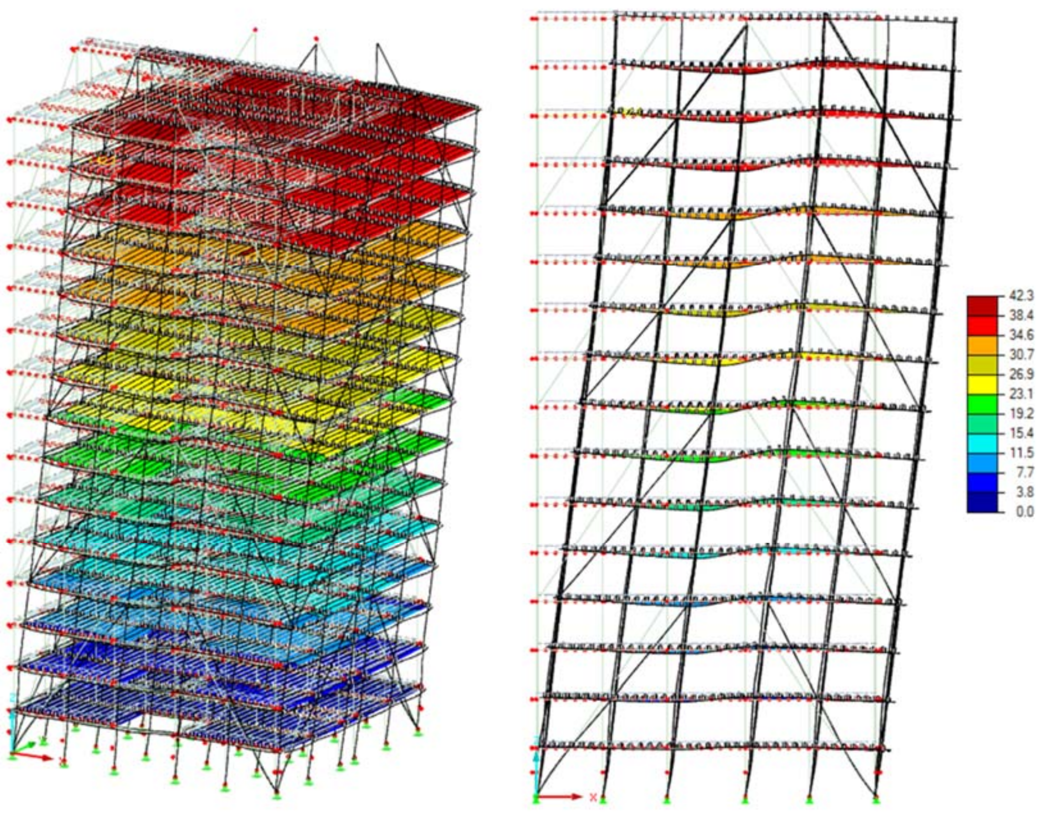


Figure B.3: Deformation of the frame construction for LC4 (wind from the side) in [mm]

Figure B.4: Internal normal forces in the columns of the frame construction in the first storey for LC1 (self-weight) in [kN]

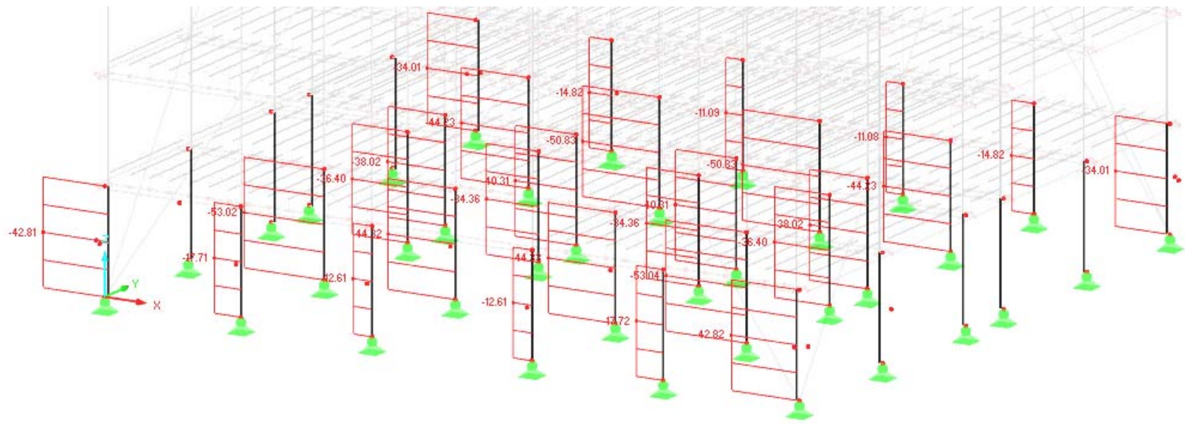


Figure B.5: Internal normal forces in the columns of the frame construction in the first storey for LC2 (snow) in [kN]

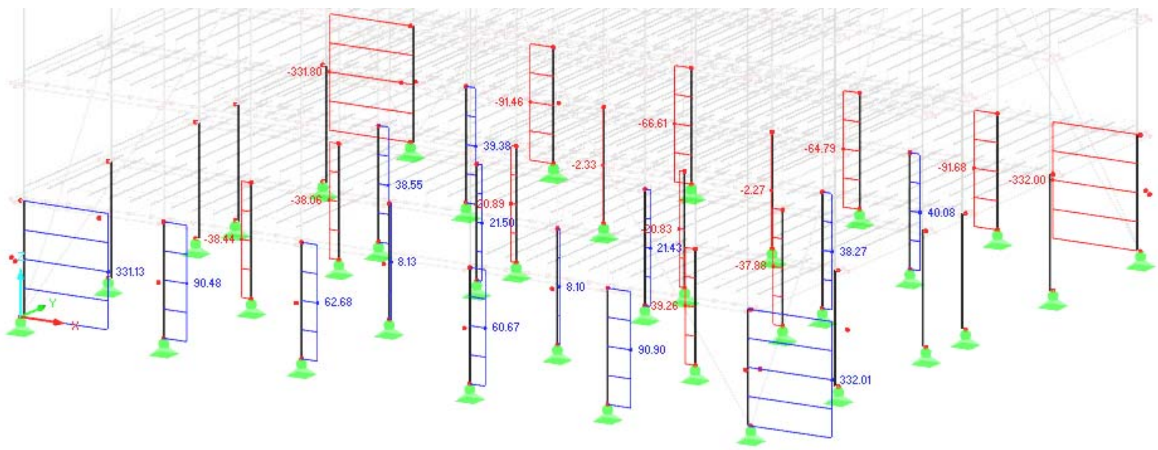


Figure B.6: Internal normal forces in the columns of the frame construction in the first storey for LC3 (wind from the front) in [kN]

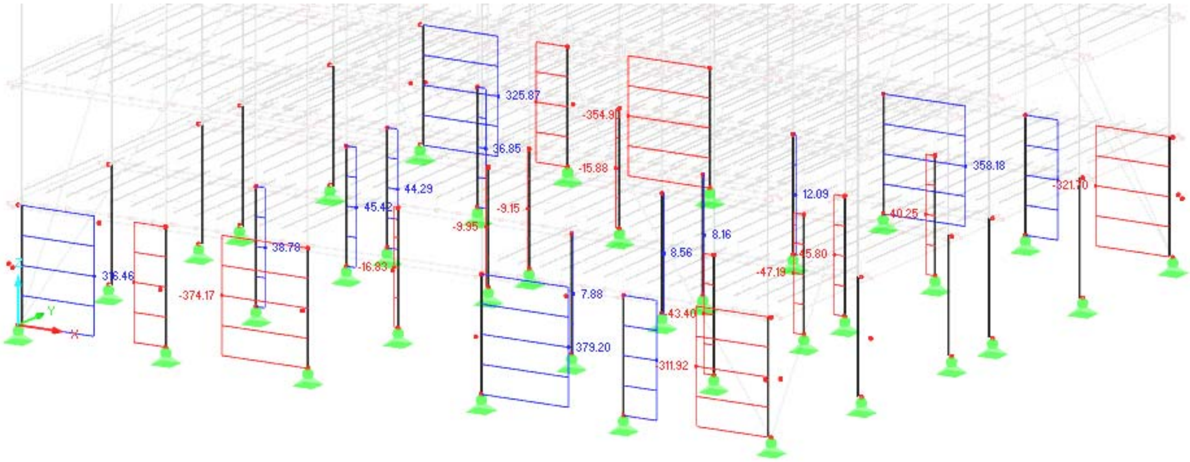


Figure B.7: Internal normal forces in the columns of the frame construction in the first storey for LC4 (wind from the side) in [kN]

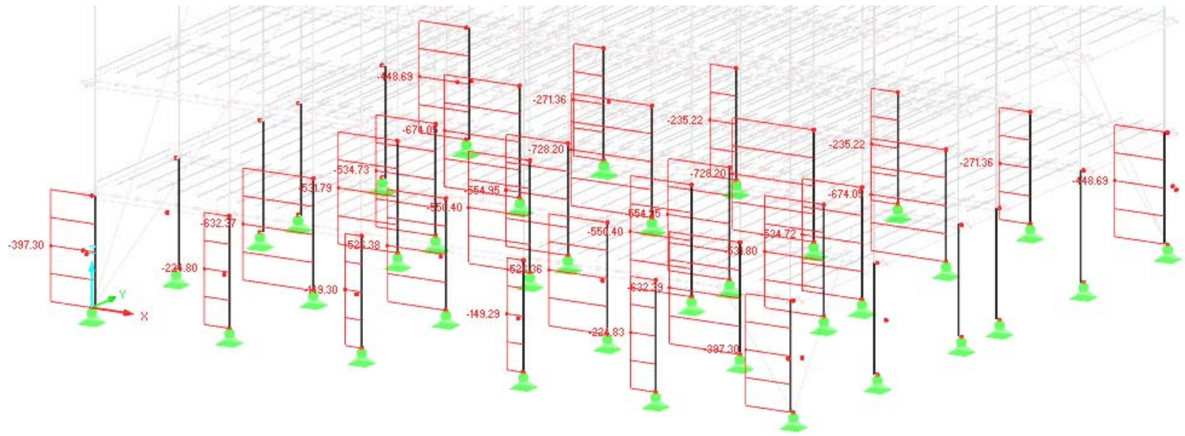


Figure B.8: Internal normal forces in the columns of the frame construction in the first storey for LC5 (life load) in [kN]

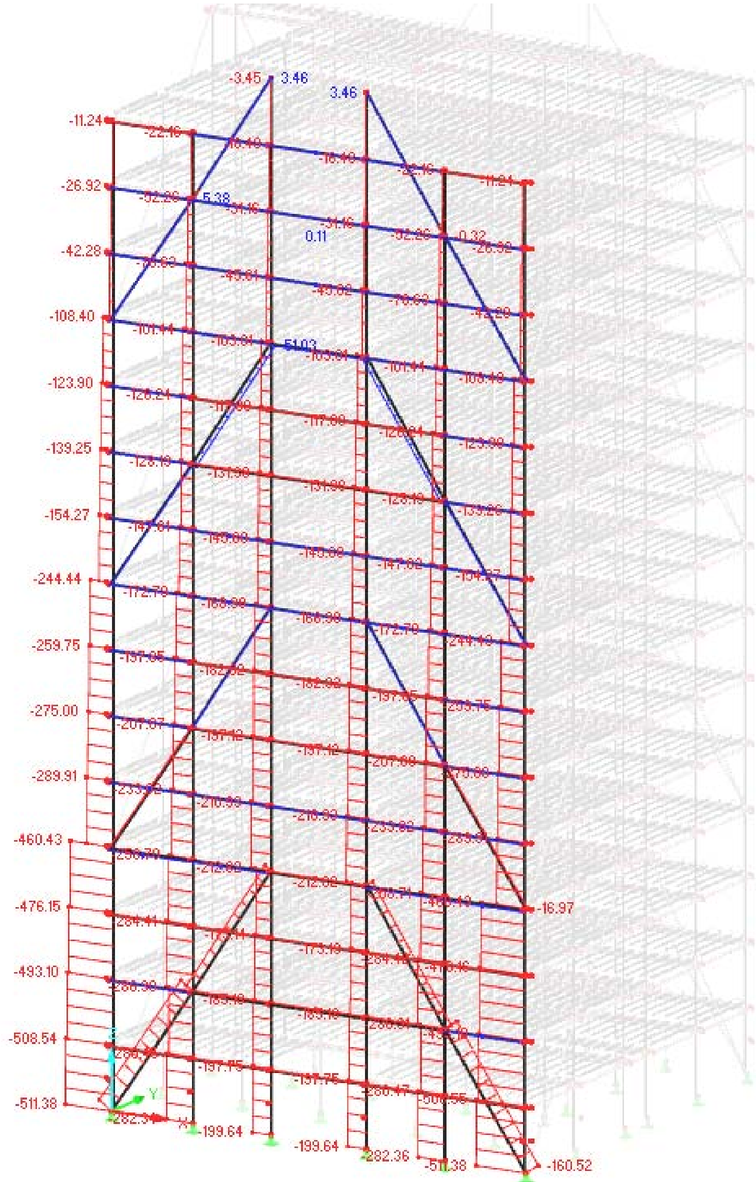


Figure B.9: Internal normal forces in the front frame of the frame construction for LC1 (self-weight) in [kN]

B.b) Panel Construction

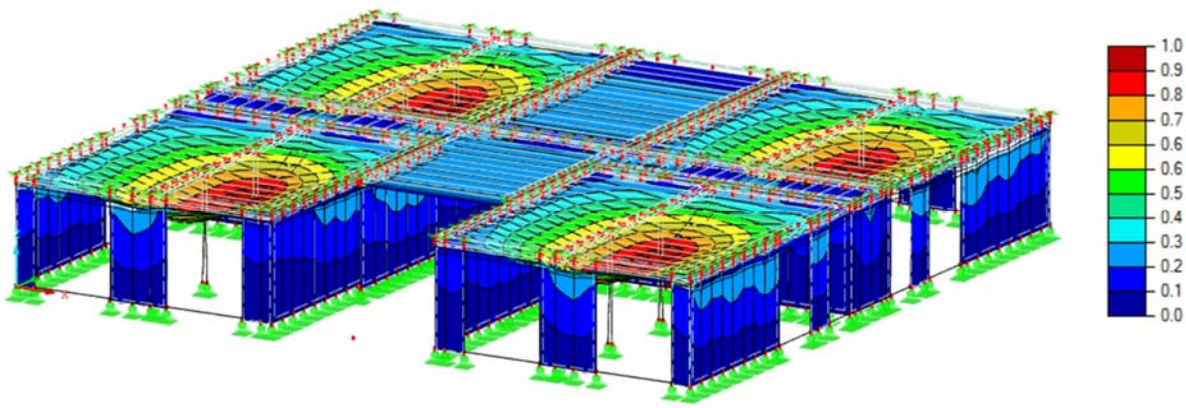


Figure B.10: Deformation of the first floor of the panel construction for LC1 (self-weight) in [mm]

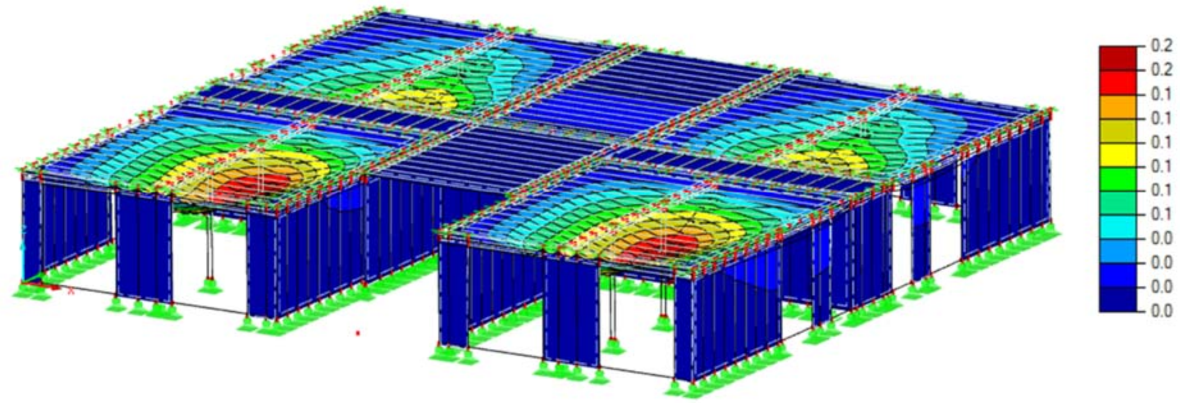


Figure B.11: Deformation of the first floor of the panel construction for LC2 (snow) in [mm]

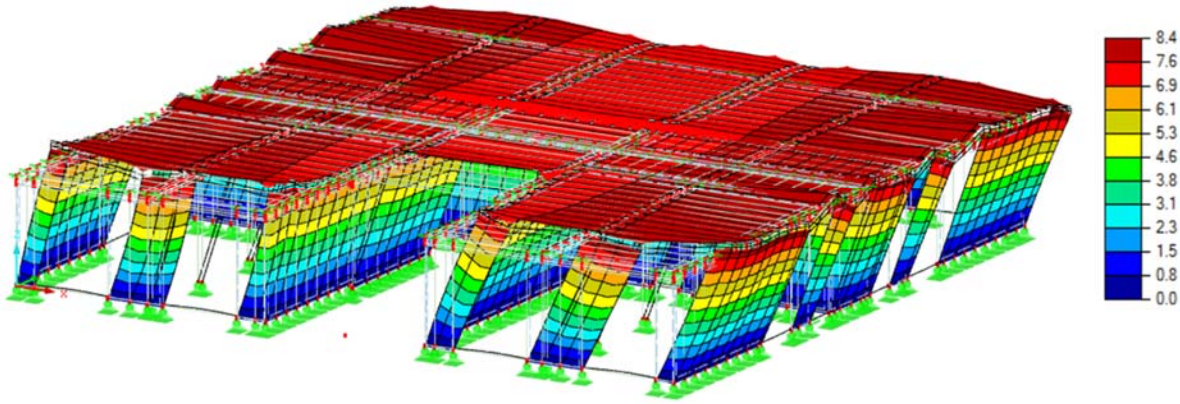


Figure B.12: Deformation of the first floor of the panel construction for LC3 (wind from the front) in [mm]

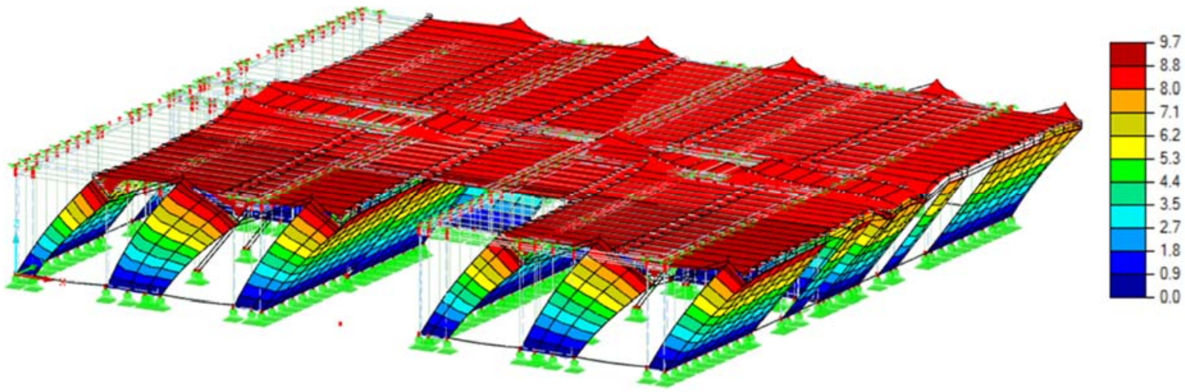


Figure B.13: Deformation of the first floor of the panel construction for LC4 (wind from the side) in [mm]

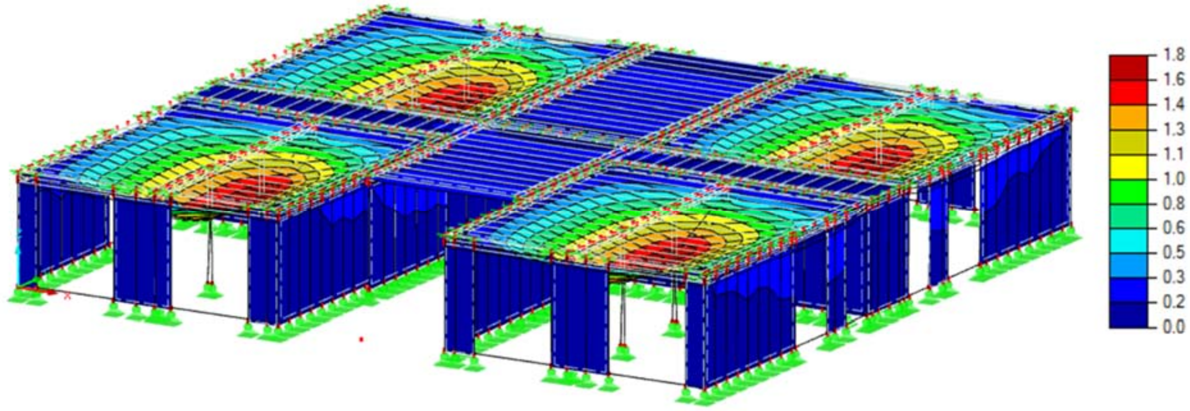


Figure B.14: Deformation of the first floor of the panel construction for LC5 (life load) in [mm]

Figure B.15: Distribution of the internal normal forces in the studs of the first storey of the panel construction for LC1 (self-weight)

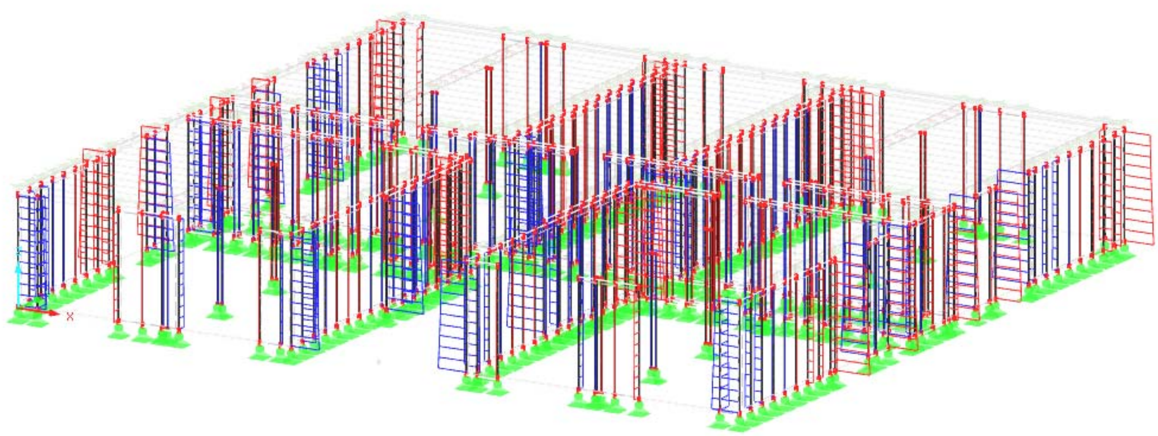


Figure B.16: Distribution of the internal normal forces in the studs of the first storey of the panel construction for LC3 (wind from the front)

B.c) CLT Construction

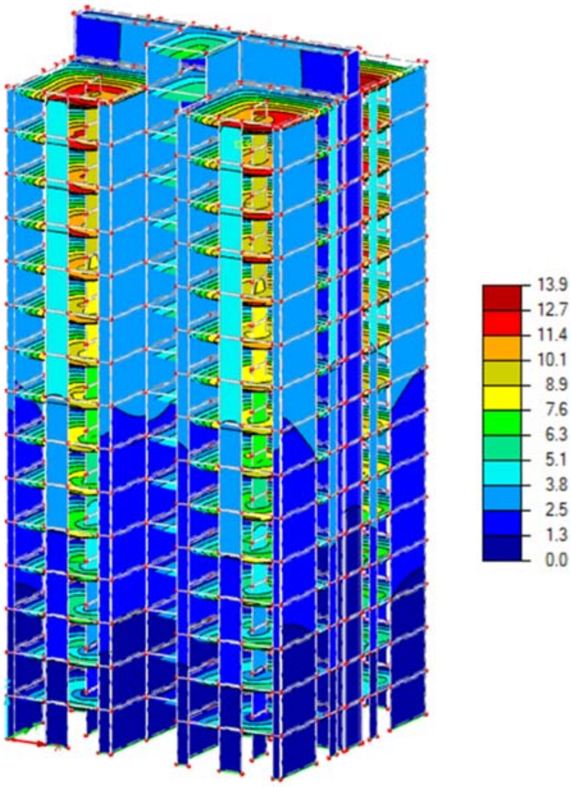


Figure B.17: Deformation of the CLT construction for LC1 (self-weight) in [mm]

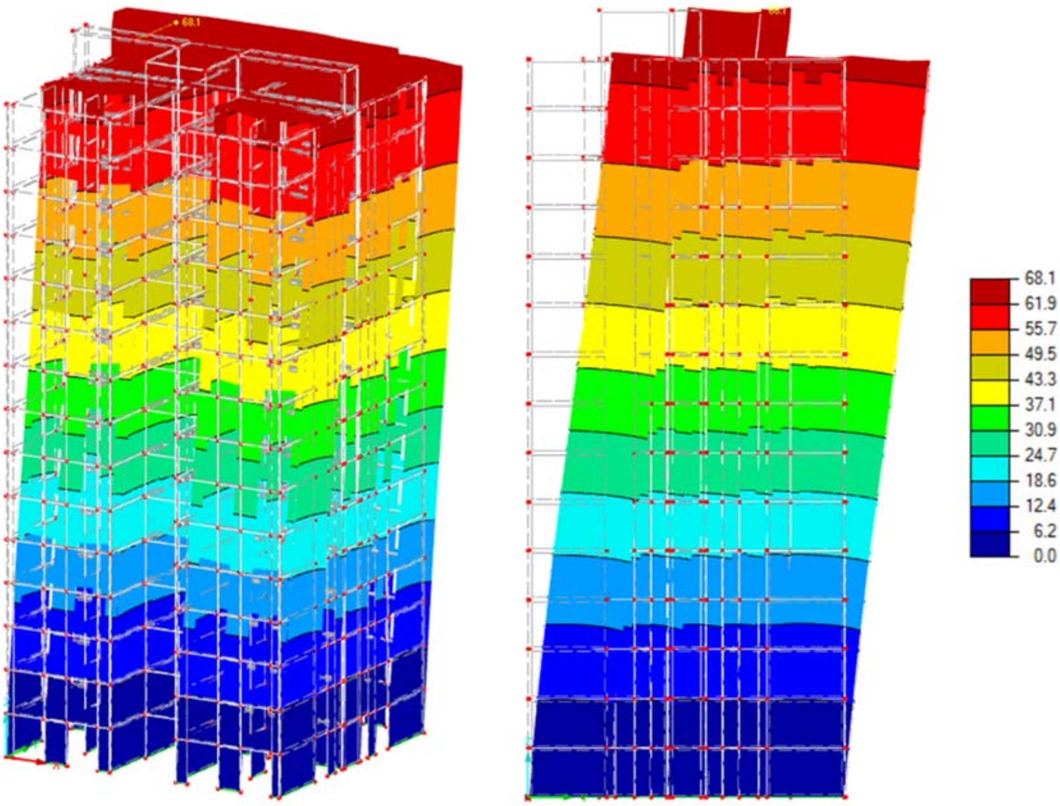


Figure B.18: Deformation of the CLT construction for LC3 (wind from the front) in [mm]

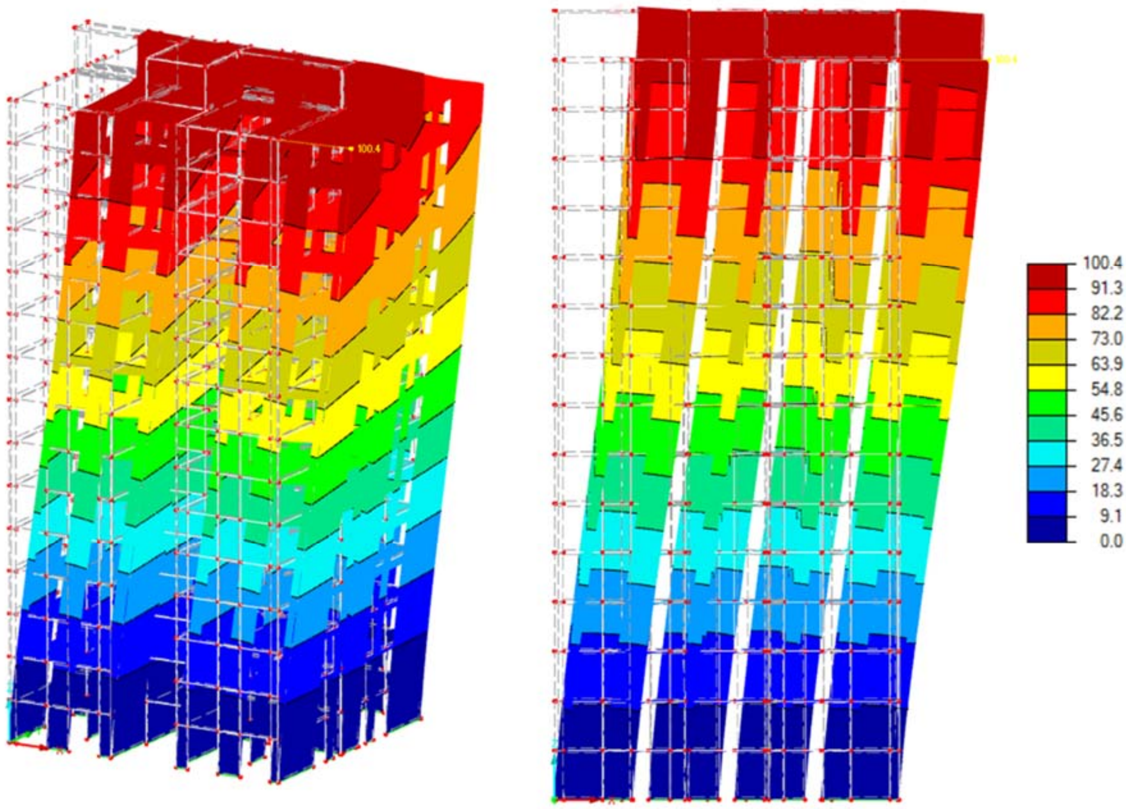


Figure B.19: Deformation of the CLT construction for LC4 (wind from the front) in [mm]

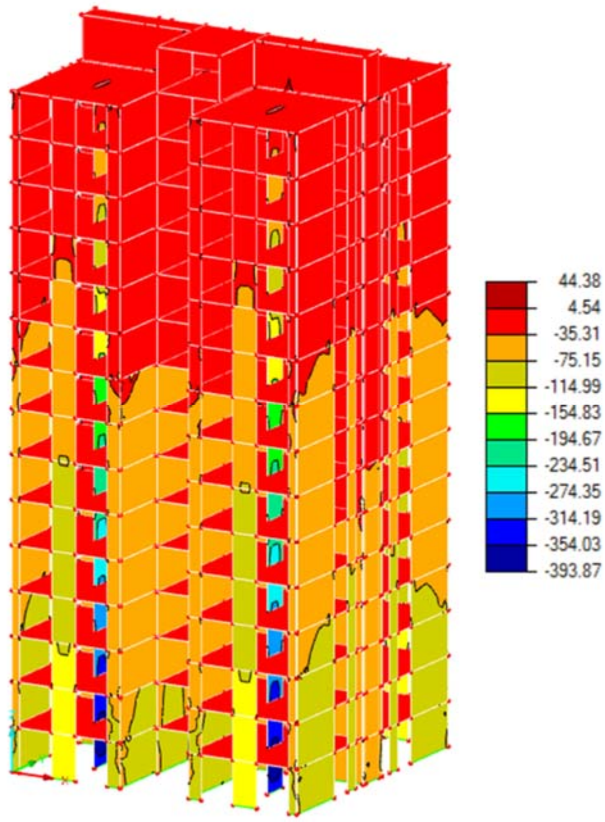


Figure B.20: Internal normal forces in the CLT construction for LC1 (self-weight) in [kN/m]

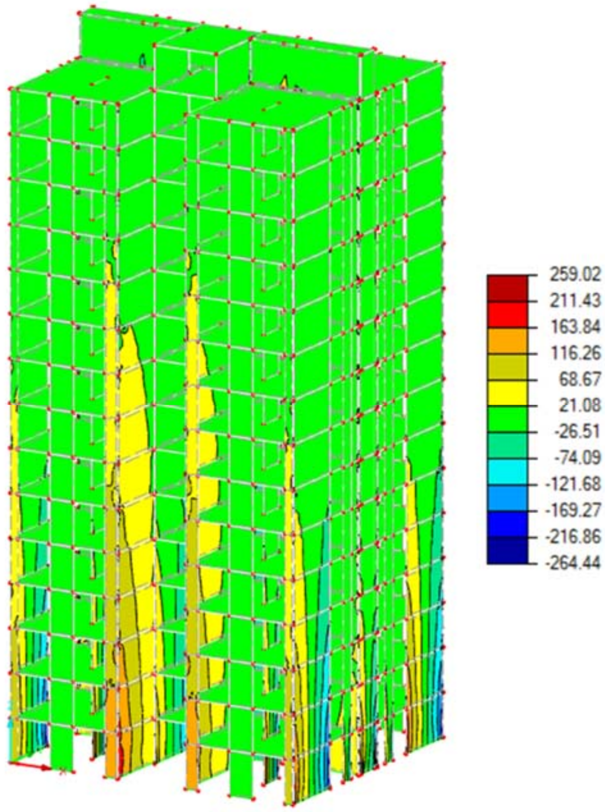


Figure B.21: Internal normal forces in the CLT construction for LC3 (wind from the front) in [kN/m]

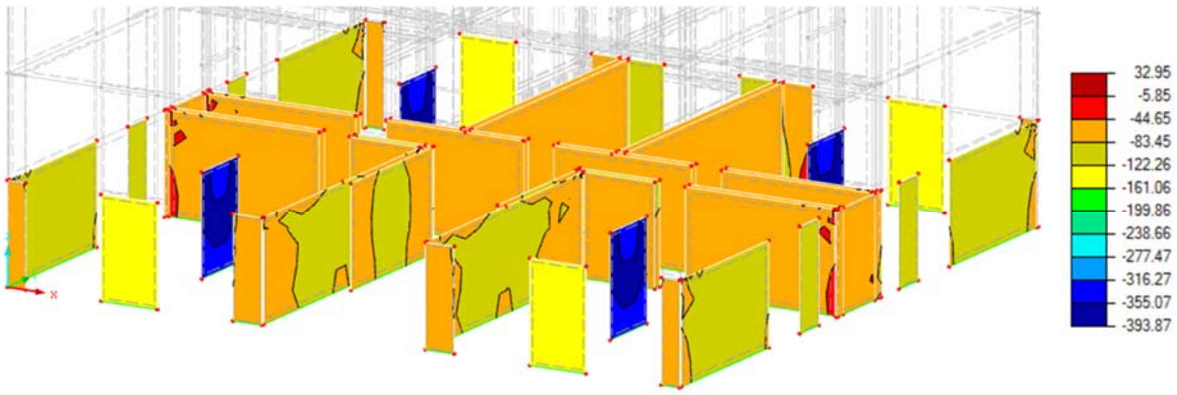


Figure B.22: Internal normal forces in the first storey of the CLT construction for LC1 (self-weight) in [kN/m]

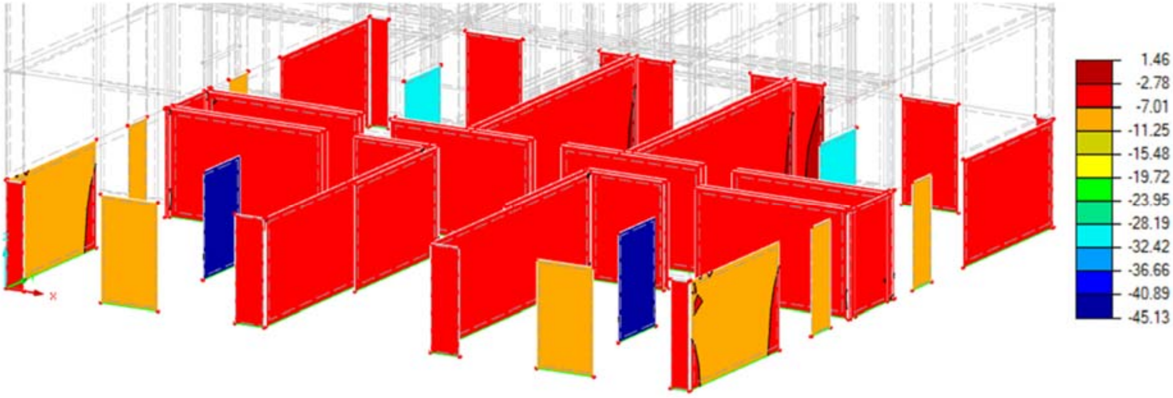


Figure B.23: Internal normal forces in the first storey of the CLT construction for LC2 (snow) in [kN/m]

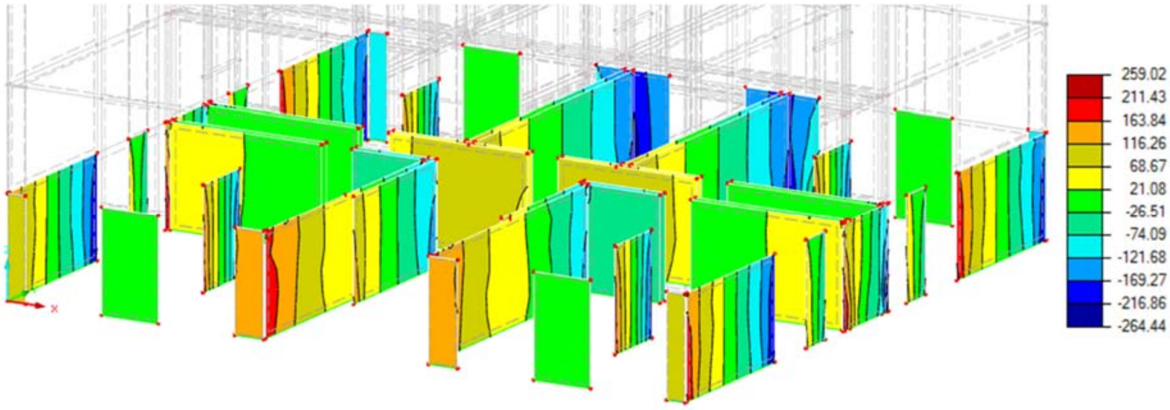


Figure B.24: Internal normal forces in the first storey of the CLT construction for LC3 (wind from the front) in [kN/m]

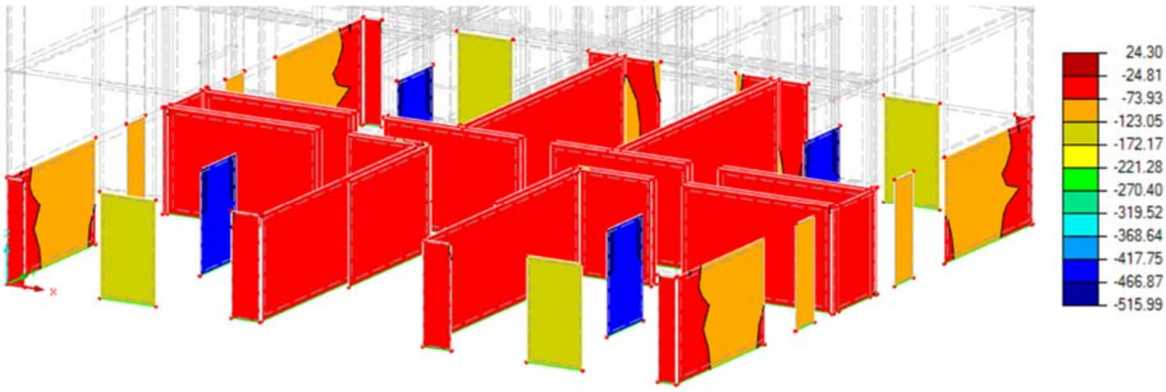


Figure B.25: Internal normal forces in the first storey of the CLT construction for LC5 (life load) in [kN/m]

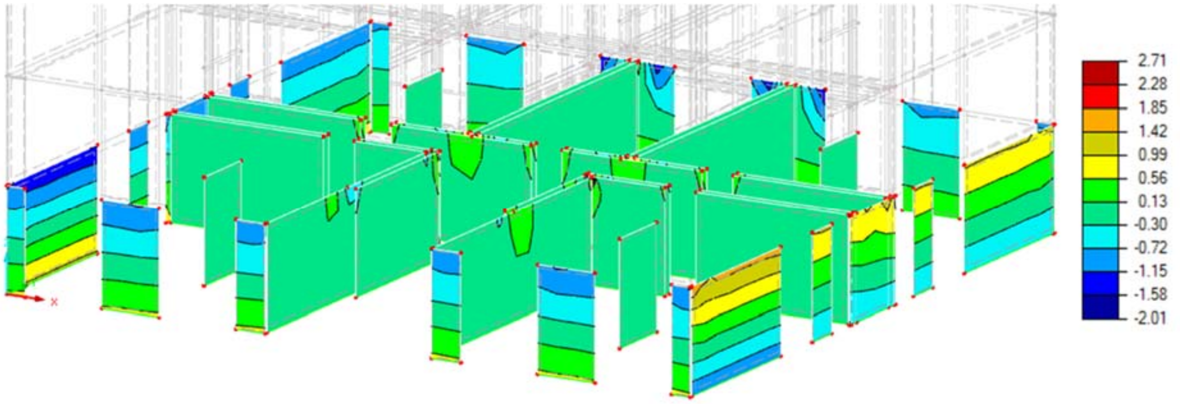


Figure B.26: Internal shear forces in the first storey of the CLT construction for LC3 (wind from the front) in [kN/m]

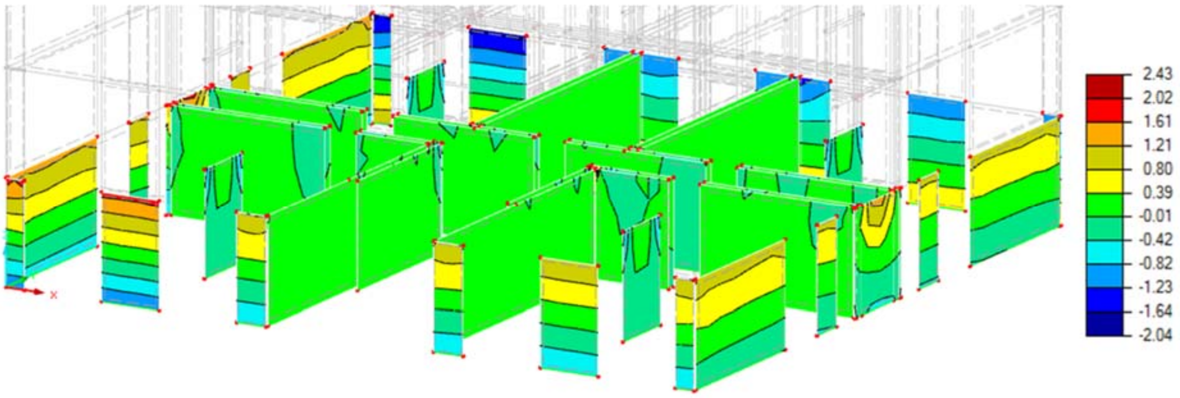


Figure B.27: Internal shear forces in the first storey of the CLT construction for LC4 (wind from the side) in [kN/m]

C. Selected ANSYS results

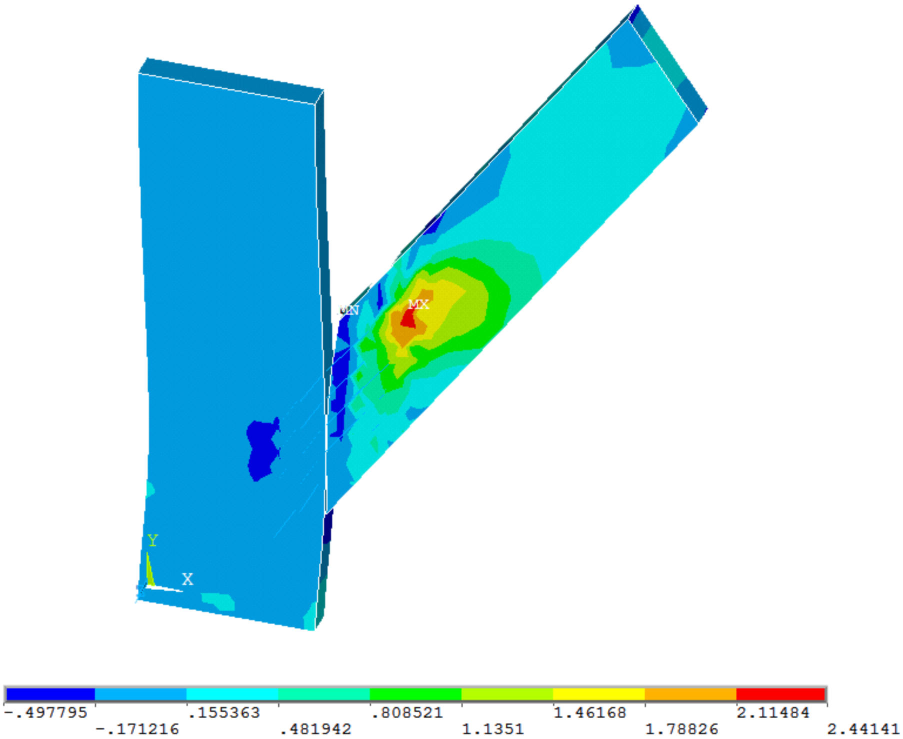


Figure B.28: Stress in global x direction σ_x of the connection in [N/mm^2]

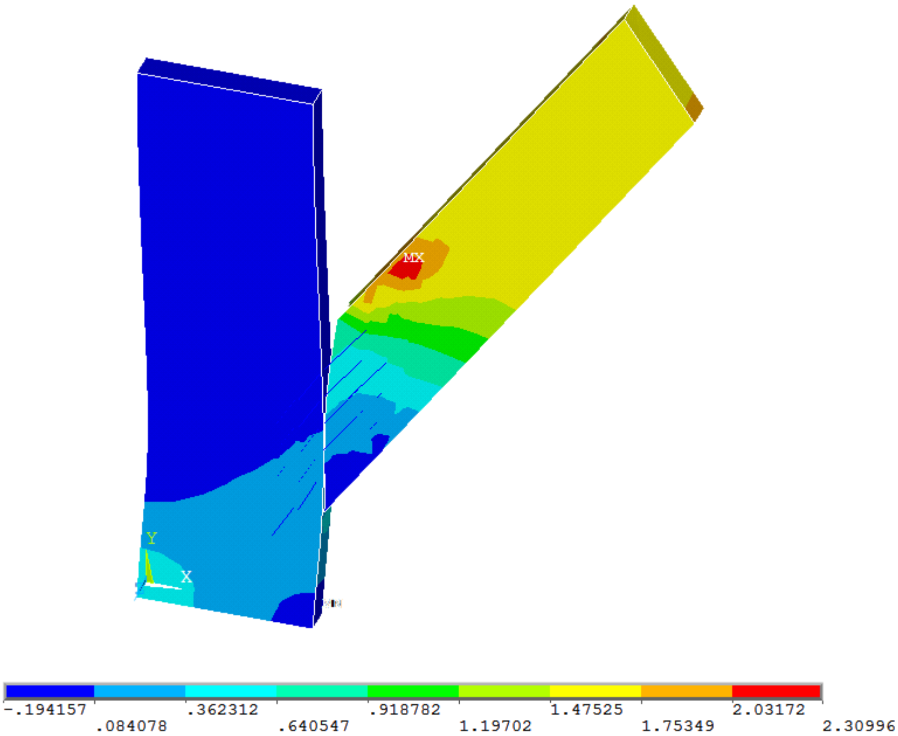


Figure B.29: Stress in global y direction σ_y of the connection in [N/mm^2]

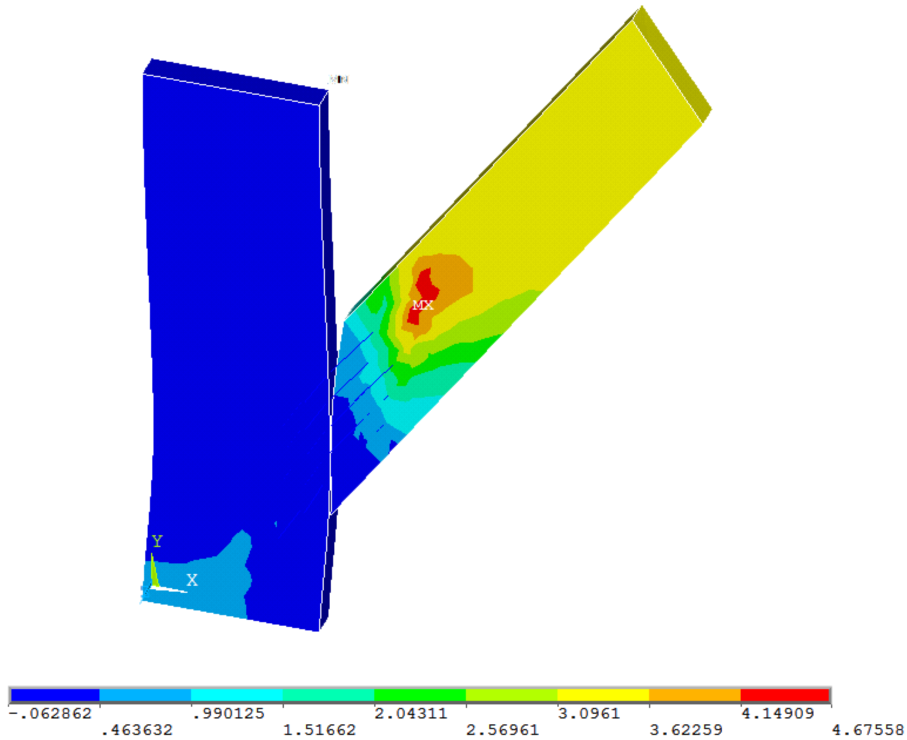


Figure B.30: First principle stress σ_1 of the connection in [N/mm^2]

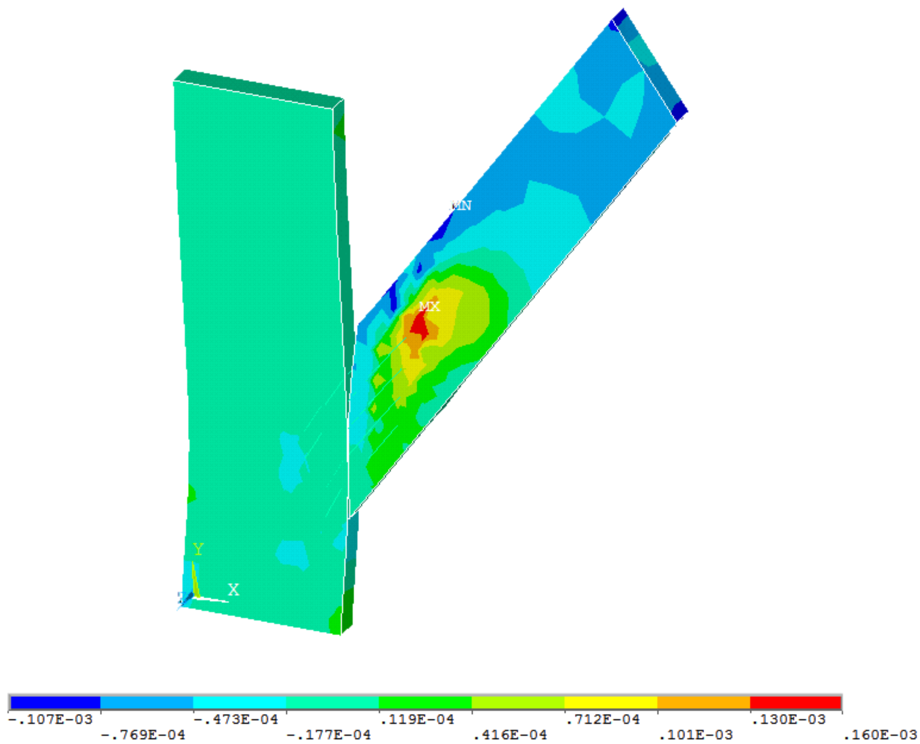


Figure B.31: Strain in global x direction ε_x of the connection in [-]

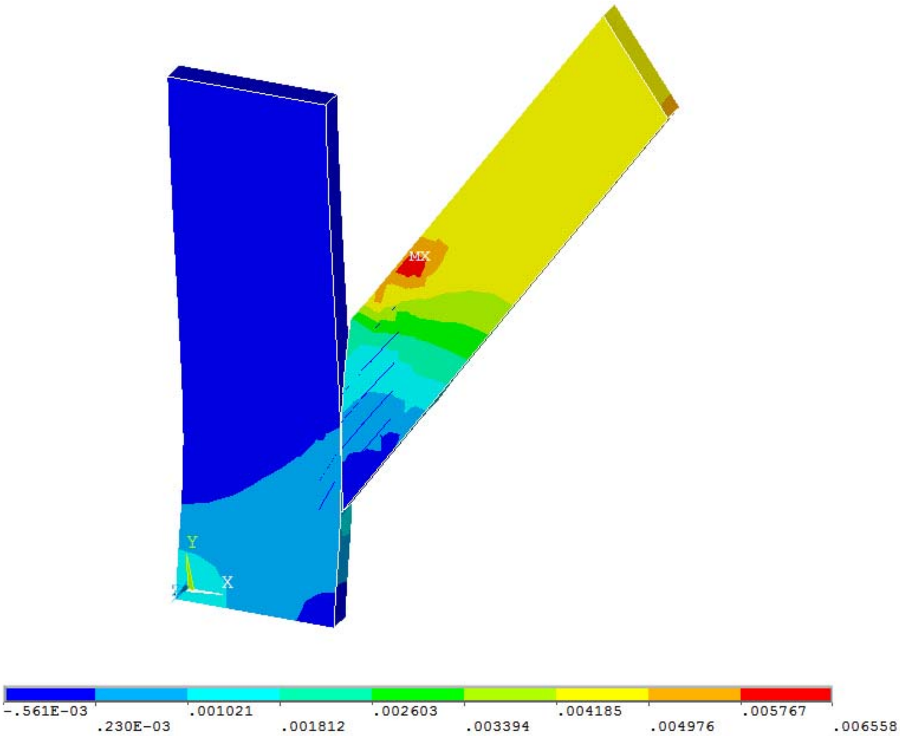


Figure B.32: Strain in global y direction ε_y of the connection in [-]

Load Calculation

Table of Contents

Self-weight

Technical drawings of the wall, floor and roof constructions can be found in the appendix.

a) Frame Construction

roof	characteristic weight γ		dimensions b		h		spacing a		load g	
gravel, protective layer	0,2	kN/m2/cm			5	cm			1	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	30	cm	0,02	kN/m2
airtight insulation, wood-fibre	0,026	kN/m2/cm			2	cm			0,047	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
horizontal carriers, C24	4,2	kN/m3	6	cm	16	cm	30	cm	0,13	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
									1,458	kN/m2
roof joists, glulam GL24h	3,7	kN/m3	14	cm	24	cm	65	cm	0,19	kN/m2
									1,65	kN/m2

floor	characteristic weight γ		dimensions b		h		spacing a		load g	
floor tiles incl. mortar	0,3	kN/m2/cm			1	cm			0,30	kN/m2
cement screed	0,22	kN/m2/cm			3	cm			0,66	kN/m2
footfall sound insulation, wood fibre	0,028	kN/m2/cm			4	cm			0,10	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
									1,15	kN/m2
floor joists, glulam GL24h	3,7	kN/m3	14	cm	24	cm	65	cm	0,19	kN/m2
									1,34	kN/m2

outer wall, facade h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
inner plywood sheathing, spruce	0,05	kN/m2/cm			1	cm			0,06	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
vertical carriers, C24	4,2	kN/m3	6	cm	16	cm	60	cm	0,07	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			1	cm			0,06	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			2	cm			0,04	kN/m2
ventilation + battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			3	cm			0,13	kN/m2
									0,44	kN/m2

b) Panel Construction

roof 1, 2	characteristic weight γ		dimensions b		h		spacing a		load g	
gravel, protective layer	0,2	kN/m2/cm			5	cm			1	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	30	cm	0,02	kN/m2
airtight insulation, wood-fibre	0,026	kN/m2/cm			2	cm			0,0468	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
horizontal carriers, C24	4,2	kN/m3	6	cm	16	cm	30	cm	0,13	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
roof joists, glulam GL24h	3,7	kN/m3	16	cm	24	cm	60	cm	0,24	kN/m2
									1,69	kN/m2

Self-weight

roof 3	characteristic weight γ	dimensions b	h	spacing a	load g
gravel, protective layer	0,2 kN/m2/cm		5 cm		1 kN/m2
plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
ventilation + counter battens, C24	4,2 kN/m3	4 cm	3 cm	30 cm	0,02 kN/m2
airtight insulation, wood-fibre	0,026 kN/m2/cm		2 cm		0,0468 kN/m2
heat insulation, wood fibre	0,005 kN/m2/cm		16 cm		0,08 kN/m2
horizontal carriers, C24	4,2 kN/m3	6 cm	16 cm	30 cm	0,13 kN/m2
plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
roof joists, glulam GL24h	3,7 kN/m3	8 cm	24 cm	60 cm	0,12 kN/m2
					1,58 kN/m2

floor 1, 2	characteristic weight γ	dimensions b	h	spacing a	load g
floor tiles incl. mortar	0,3 kN/m2/cm		1 cm		0,30 kN/m2
cement screed	0,22 kN/m2/cm		3 cm		0,66 kN/m2
footfall sound insulation, wood fibre	0,028 kN/m2/cm		4 cm		0,10 kN/m2
plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
floor joists, glulam GL24h	3,7 kN/m3	16 cm	20 cm	60 cm	0,20 kN/m2
					1,35 kN/m2

floor 3	characteristic weight γ	dimensions b	h	spacing a	load g
floor tiles incl. mortar	0,3 kN/m2/cm		1 cm		0,30 kN/m2
cement screed	0,22 kN/m2/cm		3 cm		0,66 kN/m2
footfall sound insulation, wood fibre	0,028 kN/m2/cm		4 cm		0,10 kN/m2
plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
floor joists, glulam GL24h	3,7 kN/m3	8 cm	20 cm	60 cm	0,10 kN/m2
					1,25 kN/m2

outer module wall 28/28 studs h = 3,195 m	characteristic weight γ	dimensions b	t	spacing a	load g
inner plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
heat insulation, wood fibre	0,005 kN/m2/cm		16 cm		0,08 kN/m2
studs, glulam GL24h	3,7 kN/m3	28 cm	28 cm	60 cm	0,48 kN/m2
bottom blocking, glulam GL24h	3,7 kN/m3	16 cm	28 cm		0,05 kN/m2
bottom plate, glulam GL24h	3,7 kN/m3	10 cm	24 cm		0,03 kN/m2
top blocking, glulam GL24h	3,7 kN/m3	16 cm	28 cm		0,05 kN/m2
outer plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
airtight insulation, wood-fibre	0,0235 kN/m2/cm		2 cm		0,04 kN/m2
ventilation + battens, C24	4,2 kN/m3	4 cm	3 cm	60 cm	0,01 kN/m2
cladding, wood	5 kN/m3		3 cm		0,13 kN/m2
					1,04 kN/m2

Self-weight

outer module wall 24/24 studs h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
inner plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
studs, glulam GL24h	3,7	kN/m3	24	cm	24	cm	60	cm	0,36	kN/m2
bottom blocking, glulam GL24h	3,7	kN/m3	16	cm	24	cm			0,04	kN/m2
bottom plate, glulam GL24h	3,7	kN/m3	10	cm	24	cm			0,03	kN/m2
top blocking, glulam GL24h	3,7	kN/m3	16	cm	24	cm			0,04	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			2	cm			0,04	kN/m2
ventilation + battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			3	cm			0,13	kN/m2
									0,90	kN/m2

outer module wall 20/20 studs h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
inner plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
studs, glulam GL24h	3,7	kN/m3	20	cm	20	cm	60	cm	0,25	kN/m2
bottom blocking, glulam GL24h	3,7	kN/m3	16	cm	20	cm			0,04	kN/m2
bottom plate, glulam GL24h	3,7	kN/m3	10	cm	24	cm			0,03	kN/m2
top blocking, glulam GL24h	3,7	kN/m3	16	cm	20	cm			0,04	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			2	cm			0,04	kN/m2
ventilation + battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			3	cm			0,13	kN/m2
									0,78	kN/m2

outer module wall 16/16 studs h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
inner plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
studs, glulam GL24h	3,7	kN/m3	16	cm	16	cm	60	cm	0,16	kN/m2
bottom blocking, glulam GL24h	3,7	kN/m3	16	cm	16	cm			0,03	kN/m2
bottom plate, glulam GL24h	3,7	kN/m3	10	cm	24	cm			0,03	kN/m2
top blocking, glulam GL24h	3,7	kN/m3	16	cm	16	cm			0,03	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			2	cm			0,09	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			2	cm			0,04	kN/m2
ventilation + battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			3	cm			0,13	kN/m2
									0,67	kN/m2

Self-weight

outer module wall 8/16 studs h = 3,195 m	characteristic weight γ	dimensions b	 t	spacing a	load g
inner plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
heat insulation, wood fibre	0,005 kN/m2/cm		16 cm		0,08 kN/m2
studs, glulam GL24h	3,7 kN/m3	8 cm	16 cm	60 cm	0,08 kN/m2
bottom blocking, glulam GL24h	3,7 kN/m3	16 cm	16 cm		0,03 kN/m2
bottom plate, glulam GL24h	3,7 kN/m3	10 cm	24 cm		0,03 kN/m2
top blocking, glulam GL24h	3,7 kN/m3	16 cm	16 cm		0,03 kN/m2
outer plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
airtight insulation, wood-fibre	0,0235 kN/m2/cm		2 cm		0,04 kN/m2
ventilation + battens, C24	4,2 kN/m3	4 cm	3 cm	60 cm	0,01 kN/m2
cladding, wood	5 kN/m3		3 cm		0,13 kN/m2
					0,59 kN/m2

inner module wall 28/28 studs h = 3,195 m	characteristic weight γ	dimensions b	 t	spacing a	load g
inner plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
sound insulation, wood fibre	0,005 kN/m2/cm		16 cm		0,08 kN/m2
studs, glulam GL24h	3,7 kN/m3	28 cm	28 cm	60 cm	0,48 kN/m2
bottom blocking, glulam GL24h	3,7 kN/m3	16 cm	28 cm		0,05 kN/m2
bottom plate, glulam GL24h	3,7 kN/m3	10 cm	24 cm		0,03 kN/m2
top blocking, glulam GL24h	3,7 kN/m3	16 cm	28 cm		0,05 kN/m2
outer plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
					0,88 kN/m2

inner module wall 24/24 studs h = 3,195 m	characteristic weight γ	dimensions b	 t	spacing a	load g
inner plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
sound insulation, wood fibre	0,005 kN/m2/cm		16 cm		0,08 kN/m2
studs, glulam GL24h	3,7 kN/m3	24 cm	24 cm	60 cm	0,36 kN/m2
bottom blocking, glulam GL24h	3,7 kN/m3	16 cm	24 cm		0,04 kN/m2
bottom plate, glulam GL24h	3,7 kN/m3	10 cm	24 cm		0,03 kN/m2
top blocking, glulam GL24h	3,7 kN/m3	16 cm	24 cm		0,04 kN/m2
outer plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
					0,73 kN/m2

inner module wall 20/20 studs h = 3,195 m	characteristic weight γ	dimensions b	 t	spacing a	load g
inner plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
sound insulation, wood fibre	0,005 kN/m2/cm		16 cm		0,08 kN/m2
studs, glulam GL24h	3,7 kN/m3	20 cm	20 cm	60 cm	0,25 kN/m2
bottom blocking, glulam GL24h	3,7 kN/m3	16 cm	20 cm		0,04 kN/m2
bottom plate, glulam GL24h	3,7 kN/m3	10 cm	24 cm		0,03 kN/m2
top blocking, glulam GL24h	3,7 kN/m3	16 cm	20 cm		0,04 kN/m2
outer plywood sheathing, spruce	0,05 kN/m2/cm		2 cm		0,09 kN/m2
					0,61 kN/m2

c) CLT Construction

roof 1	characteristic weight γ		dimensions b		h		spacing a		load g	
gravel, protective layer	0,2	kN/m2/cm			5	cm			1	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			1,8	cm			0,09	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	30	cm	0,02	kN/m2
airtight insulation, wood-fibre	0,026	kN/m2/cm			1,8	cm			0,0468	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
horizontal carriers, C24	4,2	kN/m3	6	cm	16	cm	30	cm	0,13	kN/m2
CLT	4	kN/m3			14,5	cm			0,58	kN/m2
									1,95	kN/m2

roof 2, 3	characteristic weight γ		dimensions b		h		spacing a		load g	
gravel, protective layer	0,2	kN/m2/cm			5	cm			1	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			1,8	cm			0,09	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	30	cm	0,02	kN/m2
airtight insulation, wood-fibre	0,026	kN/m2/cm			1,8	cm			0,0468	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
horizontal carriers, C24	4,2	kN/m3	6	cm	16	cm	30	cm	0,13	kN/m2
CLT	4	kN/m3			20,9	cm			0,84	kN/m2
									2,20	kN/m2

roof 4	characteristic weight γ		dimensions b		h		spacing a		load g	
gravel, protective layer	0,2	kN/m2/cm			5	cm			1	kN/m2
plywood sheathing, spruce	0,05	kN/m2/cm			1,8	cm			0,09	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	30	cm	0,02	kN/m2
airtight insulation, wood-fibre	0,026	kN/m2/cm			1,8	cm			0,0468	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
horizontal carriers, C24	4,2	kN/m3	6	cm	16	cm	30	cm	0,13	kN/m2
CLT	4	kN/m3			9,5	cm			0,38	kN/m2
									1,75	kN/m2

floor 1	characteristic weight γ		dimensions b		h		spacing a	load g	
floor tiles incl. mortar	0,3	kN/m2/cm			1	cm		0,30	kN/m2
cement screed	0,22	kN/m2/cm			3	cm		0,66	kN/m2
footfall sound insulation, wood fibre	0,028	kN/m2/cm			3,6	cm		0,10	kN/m2
CLT	4	kN/m3			14,5	cm		0,58	kN/m2
								1,64	kN/m2

floor 2, 3	characteristic weight γ		dimensions b		h		spacing a	load g	
floor tiles incl. mortar	0,3	kN/m2/cm			1	cm		0,30	kN/m2
cement screed	0,22	kN/m2/cm			3	cm		0,66	kN/m2
footfall sound insulation, wood fibre	0,028	kN/m2/cm			3,6	cm		0,10	kN/m2
CLT	4	kN/m3			20,9	cm		0,84	kN/m2
								1,90	kN/m2

Self-weight

floor 4	characteristic weight		dimensions			spacing		load	
	γ		b		h	a		g	
floor tiles incl. mortar	0,3	kN/m2/cm		1	cm			0,30	kN/m2
cement screed	0,22	kN/m2/cm		3	cm			0,66	kN/m2
footfall sound insulation, wood fibre	0,028	kN/m2/cm		3,6	cm			0,10	kN/m2
CLT	4	kN/m3		9,5	cm			0,38	kN/m2
								1,44	kN/m2

outer wall t = 170 mm h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
CLT	4	kN/m3			17,0	cm			0,68	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
vertical carriers, C24	4,2	kN/m3	6	cm	16	cm	60	cm	0,07	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			1,2	cm			0,06	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			1,5	cm			0,04	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			2,5	cm			0,13	kN/m2
									1,06	kN/m2

outer wall t = 145 mm h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
CLT	4	kN/m3			14,5	cm			0,58	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
vertical carriers, C24	4,2	kN/m3	6	cm	16	cm	60	cm	0,07	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			1,2	cm			0,06	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			1,5	cm			0,04	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			2,5	cm			0,13	kN/m2
									0,96	kN/m2

outer wall t = 120 mm h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
CLT	4	kN/m3			12,0	cm			0,48	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
vertical carriers, C24	4,2	kN/m3	6	cm	16	cm	60	cm	0,07	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			1,2	cm			0,06	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			1,5	cm			0,04	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			2,5	cm			0,13	kN/m2
									0,86	kN/m2

outer wall t = 95 mm h = 3,195 m	characteristic weight γ		dimensions b		t		spacing a		load g	
CLT	4	kN/m3			9,5	cm			0,38	kN/m2
heat insulation, wood fibre	0,005	kN/m2/cm			16	cm			0,08	kN/m2
vertical carriers, C24	4,2	kN/m3	6	cm	16	cm	60	cm	0,07	kN/m2
outer plywood sheathing, spruce	0,05	kN/m2/cm			1,2	cm			0,06	kN/m2
airtight insulation, wood-fibre	0,0235	kN/m2/cm			1,5	cm			0,04	kN/m2
ventilation + counter battens, C24	4,2	kN/m3	4	cm	3	cm	60	cm	0,01	kN/m2
cladding, wood	5	kN/m3			2,5	cm			0,13	kN/m2
									0,76	kN/m2

half inner dividing wall t = 170 mm h = 3,195 m	characteristic weight γ		dimensions b	t		spacing a	load g	
CLT	4	kN/m3		17	cm		0,68	kN/m2
sound insulation, wood fibre	0,005	kN/m2/cm		8	cm		0,04	kN/m2
							0,72	kN/m2

half inner dividing wall t = 145 mm h = 3,195 m	characteristic weight γ		dimensions b	t		spacing a	load g	
CLT	4	kN/m3		14,5	cm		0,58	kN/m2
sound insulation, wood fibre	0,005	kN/m2/cm		8	cm		0,04	kN/m2
							0,62	kN/m2

half inner dividing wall t = 120 mm h = 3,195 m	characteristic weight γ		dimensions b	t		spacing a	load g	
CLT	4	kN/m3		12	cm		0,48	kN/m2
sound insulation, wood fibre	0,005	kN/m2/cm		8	cm		0,04	kN/m2
							0,52	kN/m2

half inner dividing wall t = 95 mm h = 3,195 m	characteristic weight γ		dimensions b	t		spacing a	load g	
CLT	4	kN/m3		9,5	cm		0,38	kN/m2
sound insulation, wood fibre	0,005	kN/m2/cm		8	cm		0,04	kN/m2
							0,42	kN/m2

inner wall t = 246 mm h = 3,195 m	characteristic weight γ		dimensions b	t		spacing a	load g	
CLT	4	kN/m3		24,6	cm		0,98	kN/m2
							0,98	kN/m2

inner wall t = 170 mm h = 3,195 m	characteristic weight γ		dimensions b	t		spacing a	load g	
CLT	4	kN/m3		17	cm		0,68	kN/m2
							0,68	kN/m2

inner wall t = 95 mm h = 3,195 m	characteristic weight γ		dimensions b	t		spacing a	load g	
CLT	4	kN/m3		9,5	cm		0,38	kN/m2
							0,38	kN/m2

Live loads

Vertical Life Loads

category	description	q_k [kN/m^2]	Q_k [kN]
A	residential buildings, kitchen, bathroom, living and sleeping rooms, e. g. in hospitals, hotels		
	floor	2,5 *	2,0
H	roofs, not walkable, only maintenance		
	slope $\alpha = 0° < 20°$	0,75	1,5
*) including 0,5 kN/m^2 additional load for non-loadbearing walls			

reduction factor for vertical distributed live loads q_k in multiple storeys

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

$$\alpha_n = \frac{2 + (14 - 2) \cdot 0{,}7}{14} = 0{,}74$$

$n = 14$

load category $= A$ ($\psi_0 = 0{,}7$)

Snow loads

snow load on the ground for Bergen, Hordaland, South-Norway

$s_{k,0} = 2{,}0$ kN/m^2

$H_g = 150$ m

$H \approx 2$ m. o. h.

$s_k = 2{,}0$ kN/m^2

snow load on the roof

$s = \mu_i \cdot C_e \cdot C_t \cdot s_k$

exposure factor $C_e = 1{,}0$ (normal exposure)

thermal factor $C_t = 1{,}0$

$s = \mu_i \cdot s_k$

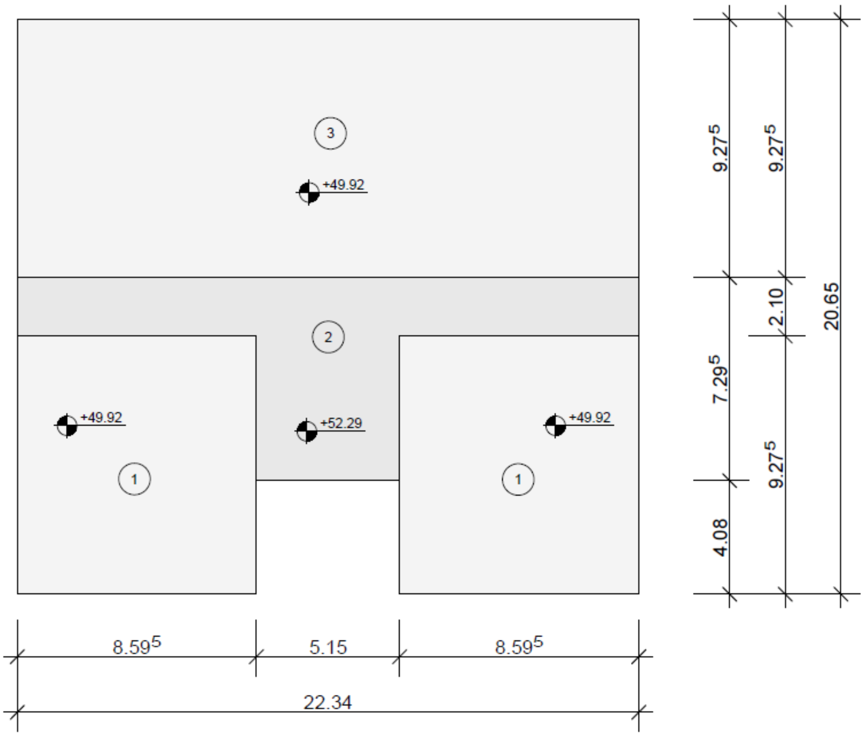


Figure 1: Roof reference

Roof 1

$\alpha = 0° \leq 15°$

→ $\mu_s = 0$

$b_1 = 2{,}10 \text{ m}$

$b_2 = 9{,}275 \text{ m}$

$h = 52{,}92 - 49{,}92 = 3{,}0 \text{ m}$

$$\mu_w = \frac{b_1 + b_2}{2 \cdot h} = \frac{2{,}10 + 9{,}275}{2 \cdot 3{,}0} = 1{,}90 \leq 2 \cdot 3{,}67/2 = 3{,}67$$

$s = 1{,}90 \cdot 2{,}0 = 3{,}80 \text{ kN/m}^2$

On the safe side, the higher snow load is considered to be constant along the length of the roof of the lift, until 4,08 m before the end of the roof, and then decrease linearly.

$l_s = 2 \cdot 3{,}0 = 6{,}0 \text{ m} > 4{,}08 \text{ m}$

$$\mu' = 0{,}8 + \frac{1{,}90 - 0{,}8}{6{,}0}(6{,}0 - 4{,}08) = 1{,}15$$

$s = 1{,}15 \cdot 2{,}0 = 2{,}30 \text{ kN/m}^2$

Roof 2

$\alpha = 0°$

→ $\mu_1 = 0{,}8$

$s = 0{,}8 \cdot 2{,}0 = 1{,}6\ \mathrm{kN/m^2}$

Roof 3

$\alpha = 0° \leq 15°$

→ $\mu_s = 0$

$$b_1 = \frac{2{,}10 + 7{,}295}{2} = 4{,}70\ \mathrm{m}$$

$b_2 = 9{,}275\ \mathrm{m}$

$h = 52{,}92 - 49{,}92 = 3{,}0\ \mathrm{m}$

$$\mu_w = \frac{4{,}70 + 9{,}275}{2 \cdot 3{,}0} = 2{,}33 \leq 2 \cdot 3{,}00/2 = 3{,}00$$

$s = 2{,}33 \cdot 2{,}0 = 4{,}66\ \mathrm{kN/m^2}$

$l_s = 2 \cdot 3{,}0 = 6{,}0\ \mathrm{m} < b_2 = 9{,}275\ \mathrm{m}$

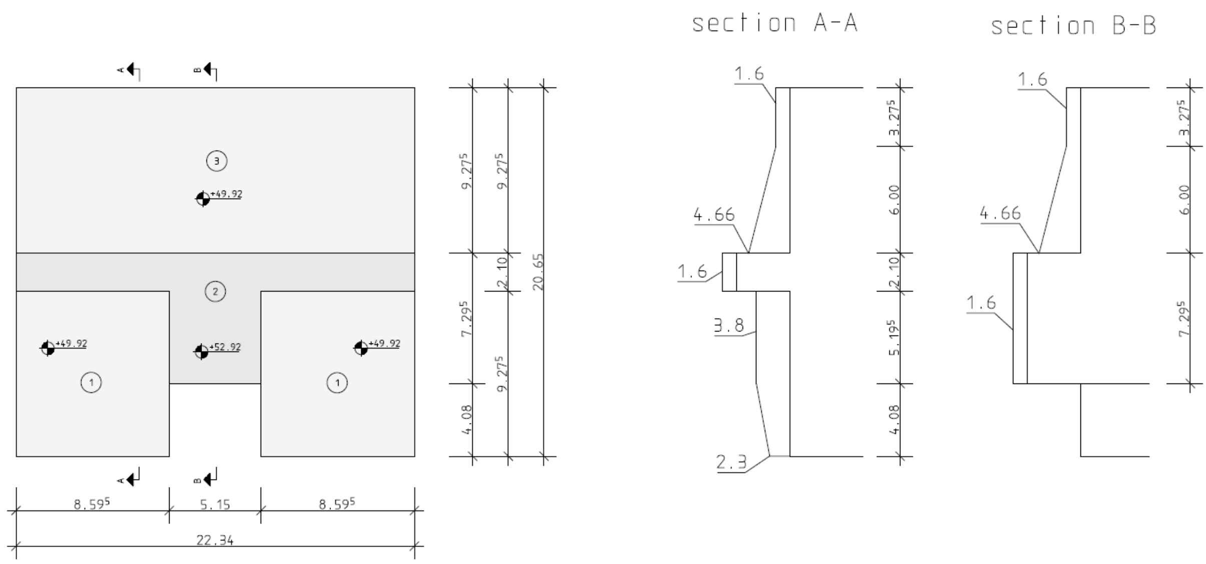


Figure 2: Summary of the snow loads on the roof

Wind Loads

reference wind speed for terrain category II (open terrain with individual houses or trees)

$v_{b,0} = 26\ \mathrm{m/s} < v_0 = 30\ \mathrm{m/s}$

basic wind speed with direction factor $c_{dir} = 1{,}0$, season factor $c_{season} = 1{,}0$ and probability factor $c_{prob} = 1{,}0$.

$H \approx 2\ \mathrm{m.o.h.}$

$H_0 = 900\ \mathrm{m}$

$H_{topp} = 1500\ \mathrm{m}$

$$c_{alt} = 1{,}0 + \frac{(30-26)(2-900)}{26(1500-900)} = 0{,}77 < 1{,}0$$

$$c_{alt} = 1{,}0$$

$$v_b = 1{,}0 \cdot 26 = 26\ \mathrm{m/s}$$

10 minutes average wind speed over the height z for terrain category IV

roughness factor: $k_r = 0{,}24$

roughness length: $z_0 = 1{,}0\ \mathrm{m}$

$z_{min} = 16\ \mathrm{m}$

$z_{max} = 200\ \mathrm{m}$

$c_r(z) = 0{,}24 \cdot \ln\left(\frac{z}{1{,}0\ \mathrm{m}}\right)$ $\qquad z_{min} \leq z \leq z_{max}$

$c_r(z) = c_r(z_{min})$ $\qquad z \leq z_{min}$

terrain form factor: $c_o(z) = 1{,}0$

$v_m(z) = 0{,}24 \cdot ln\left(\frac{z}{1{,}0\ m}\right) \cdot 26\ m/s$ $\qquad z_{min} \leq z \leq z_{max}$

$v_m(z) = v_m(z_{min})$ $\qquad z \leq z_{min}$

turbulence intensity (ratio between the standard derivation for the instantaneous wind speed (1 second) and the 10 minutes average wind speed)

turbulence factor: $k_l = 1{,}0$

$I_v(z) = \frac{1{,}0}{1{,}0 \cdot ln\left(\frac{z}{1{,}0\ m}\right)}$ $\qquad z_{min} \leq z \leq z_{max}$

$I_v(z) = I_v(z_{min})$ $\qquad z \leq z_{min}$

10 minutes average wind speed pressure

air density $\rho = 1{,}25\ kg/m^3$

$q_m(z) = 0{,}5 \cdot 1{,}25\ kg/m^3 \cdot v_m^2(z)$

gust wind speed pressure

peak factor $k_p = 3{,}5$

$q_p(z) = [1 + 2 \cdot 3{,}5 \cdot I_v(z)] \cdot q_m(z)$

general geometry

$h = 52{,}92 - 2{,}0 = 50{,}92\ m$

$b_1 = 20{,}65\ m$

$b_2 = 22{,}34\ m$

$h > 2b$

wind from the front/back

$b = 20{,}65\ m$

$z_{e0} = 0\ m$

$z_{e1} = 20{,}65\ m > z_{min} = 16\ m$

$z_{e2} = 50{,}92 - 20{,}65 = 30{,}27\ m$

$z_{e3} = 50{,}92\ m < z_{max} = 200\ m$

i	z_e	v_m	I_v	q_m	q_p
3	50,92	24,52	0,254	375,8	1044,0
2	30,27	21,28	0,293	283,0	863,4
1	20,65	18,17	0,330	206,3	683,0

wind from the sides

$b = 22{,}34\ m$

$z_{e0} = 0\ m$

$z_{e1} = 22{,}34\ m > z_{min} = 16\ m$

$z_{e2} = 50{,}92 - 22{,}34 = 28{,}58\ m$

$z_{e3} = 50{,}92\ m < z_{max} = 200\ m$

i	z_e	v_m	I_v	q_m	q_p
3	50,92	24,52	0,254	375,8	1044,0
2	28,58	20,92	0,298	273,5	844,0
1	22,34	19,38	0,322	234,7	763,7

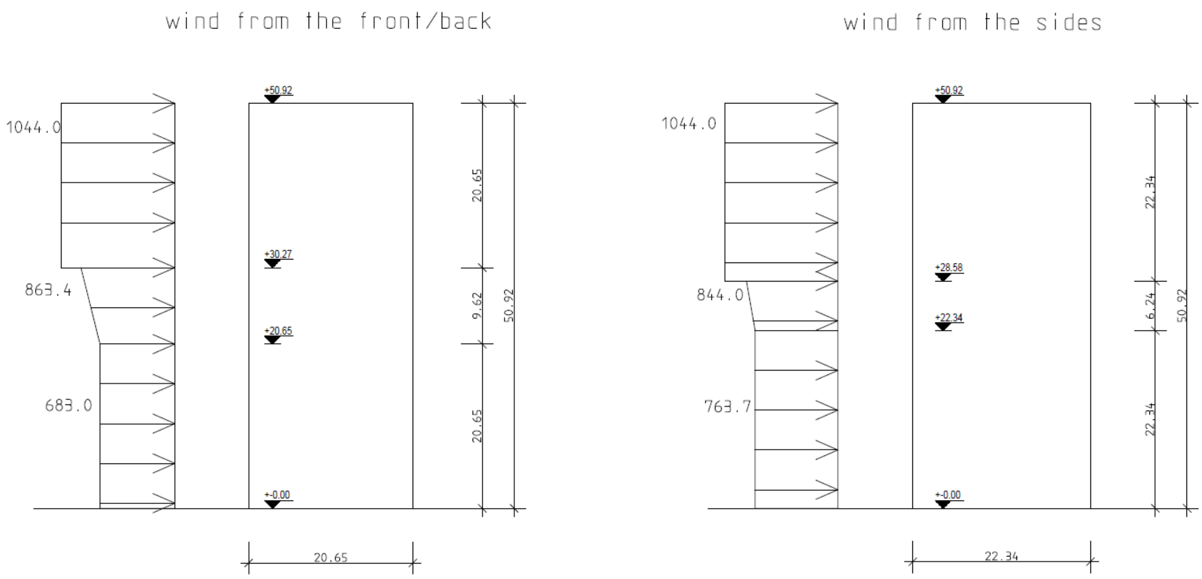


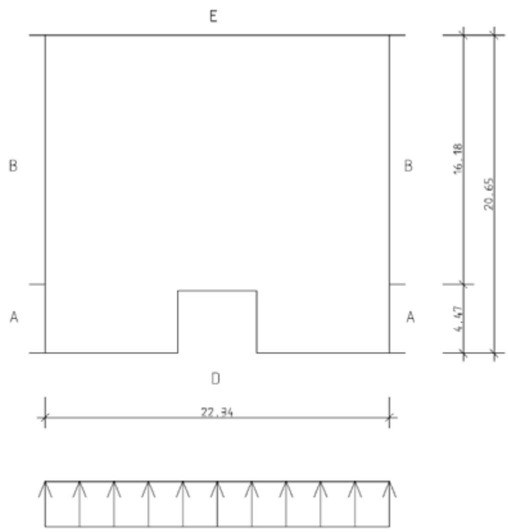
Figure 3: Summary of the gust wind speed pressure q_p

wind pressure

$w_e = q_p(z_e) \cdot c_{pe}$

wind pressure on the walls

wind from the front/back



$b = 22{,}34\ m$

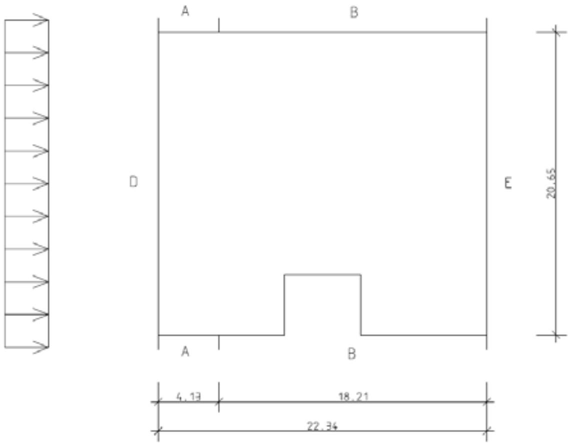
$d = 20{,}65\ m$

$$e = 22{,}34\ m \begin{cases} \geq d = 20{,}65\ m \\ < 5d = 5 \cdot 20{,}65 = 103{,}25\ m \end{cases}$$

$h/d = 50{,}92/20{,}65 = 2{,}47$

	A	B	D	E
$c_{pe,10}$	-1,2	-0,8	+0,8	-0,57
$c_{pe,1}$	-1,4	-1,1	+1,0	-0,57

wind from the sides



- area C is neglected

$b = 20{,}65\ m$

$d = 22{,}34\ m$

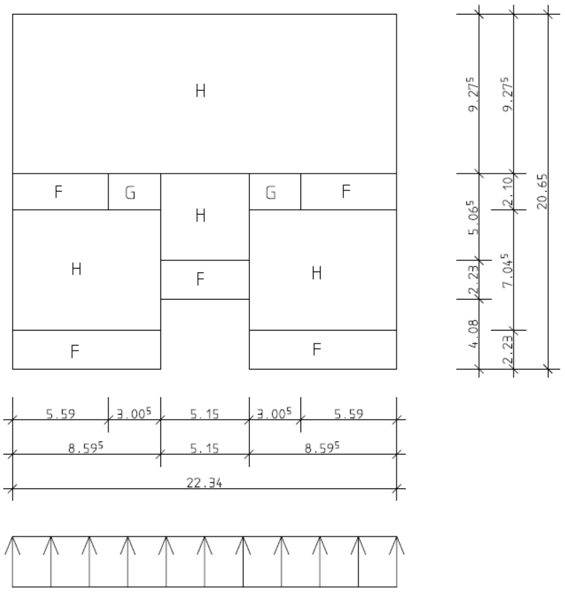
$e = 20{,}65\ m < d = 22{,}34\ m$

$h/d = 50{,}92/22{,}34 = 2{,}28$

	A	B	D	E
$c_{pe,10}$	-1,2	-0,8	+0,8	-0,56
$c_{pe,1}$	-1,4	-1,1	+1,0	-0,56

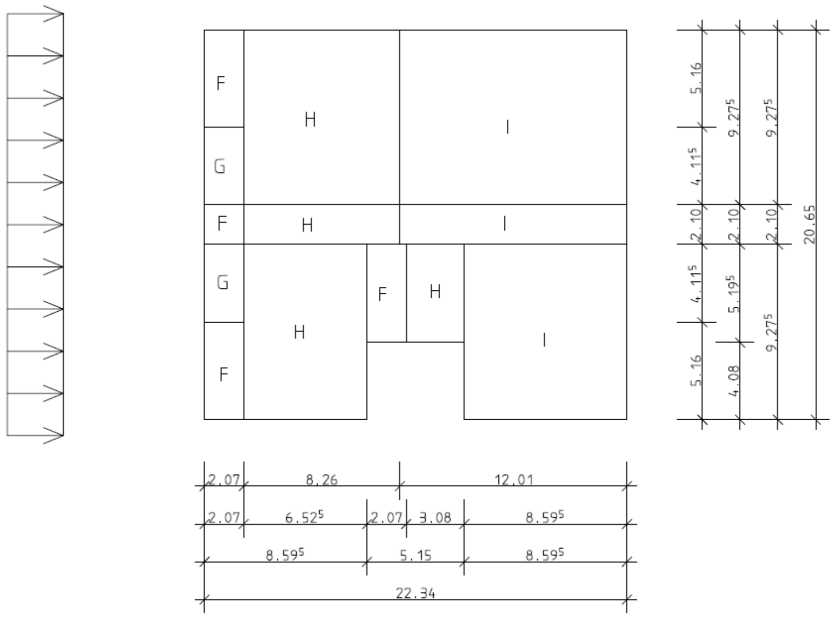
wind pressure on the roof

wind from the front/back



	F	G	H
$c_{pe,10}$	-1,8	-1,2	-0,7
$c_{pe,1}$	-2,5	-2,0	-1,2

wind from the sides



	F	G	H	I
$c_{pe,10}$	-1,8	-1,2	-0,7	+/-0,2
$c_{pe,1}$	-2,5	-2,0	-1,2	+/-0,2

Appendix

a) Frame Construction

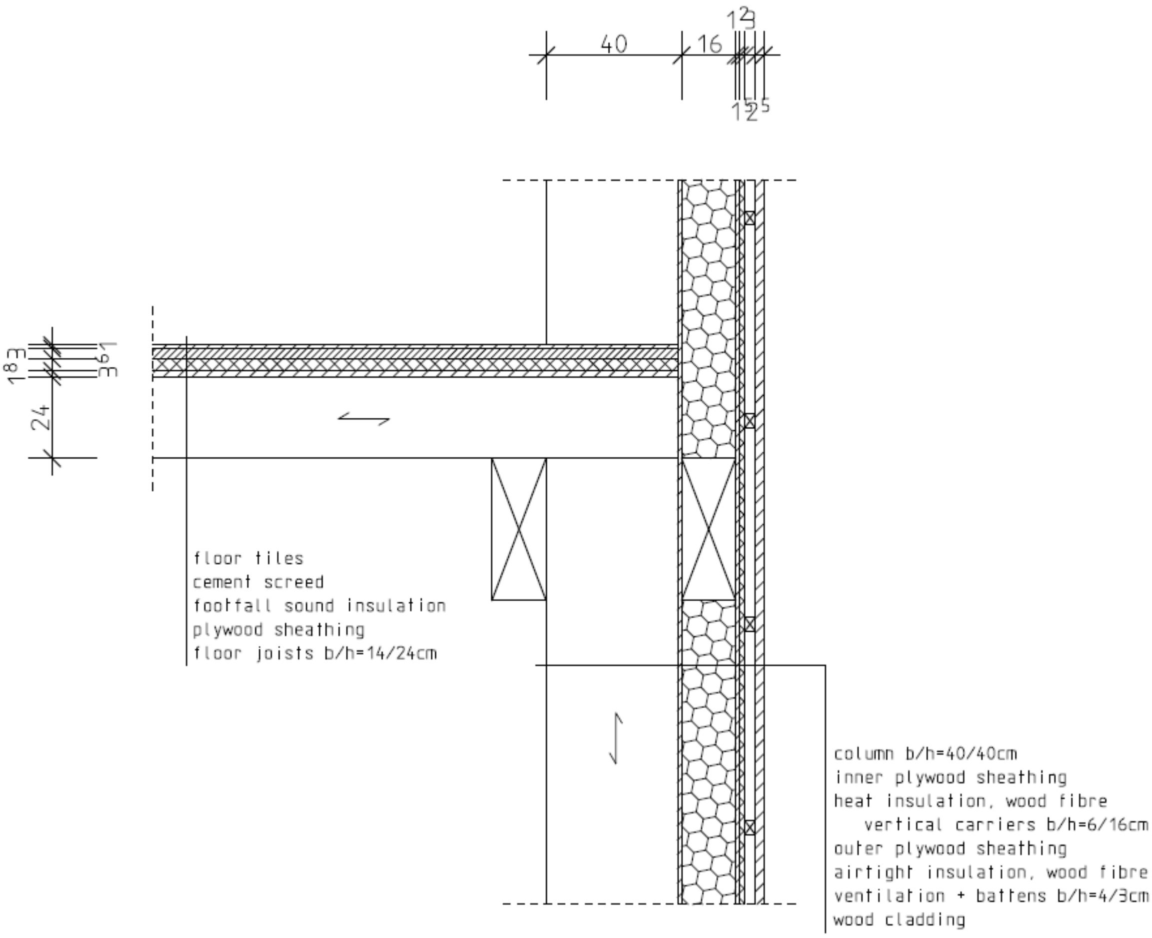


Figure 4: Vertical section through the wall and the floor of the frame construction

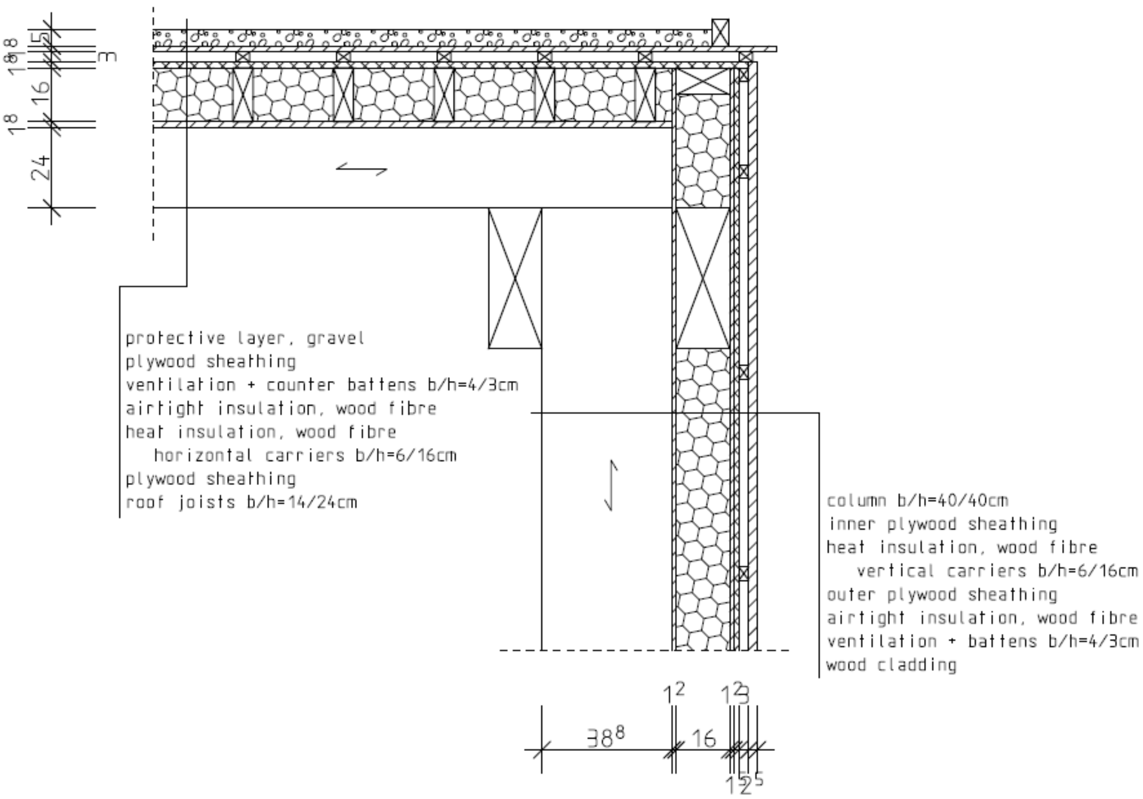


Figure 5: Vertical section through the wall and the roof of the frame construction

b) Panel Construction

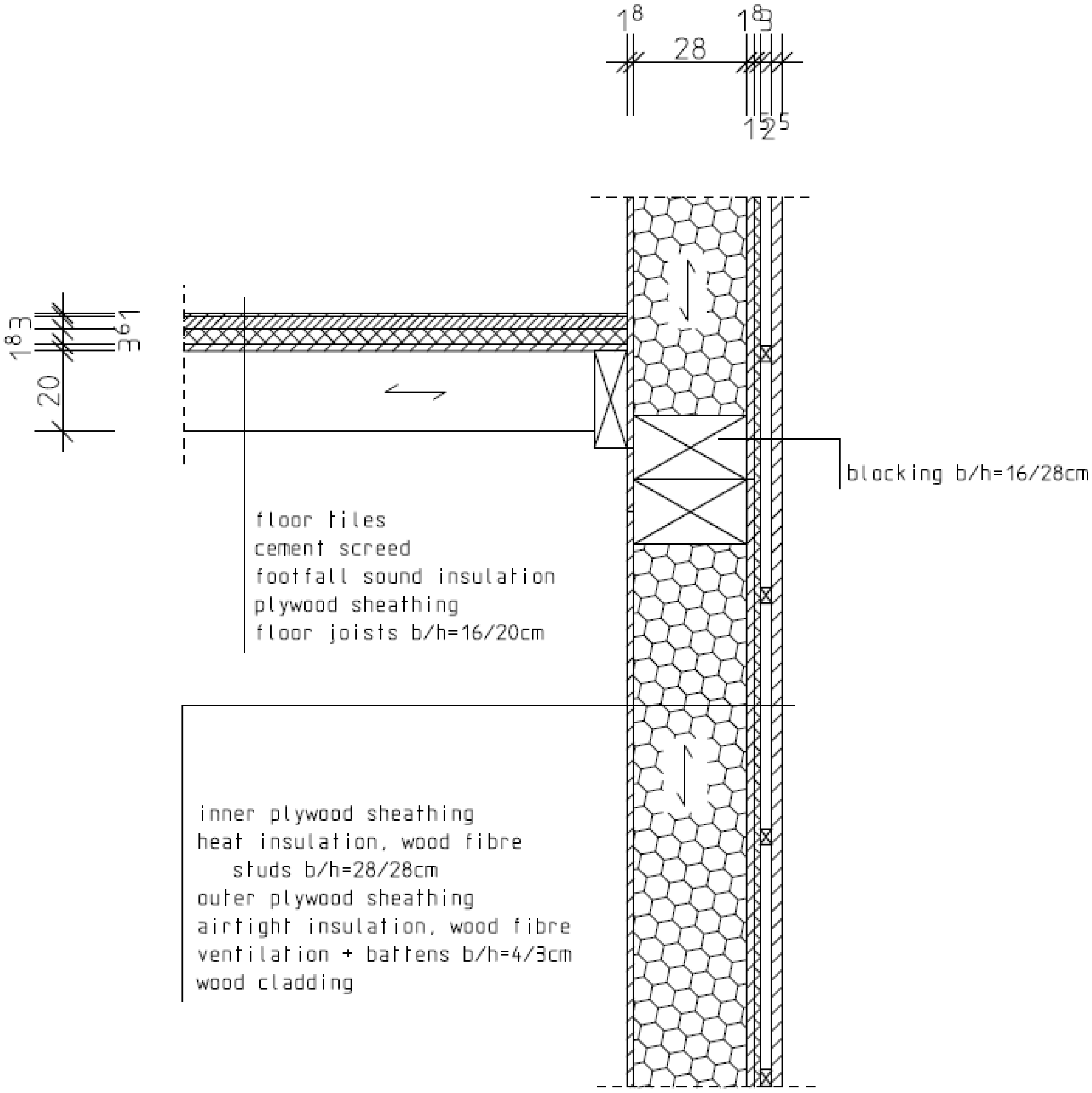


Figure 6: Vertical section through the wall and the floor of the panel construction

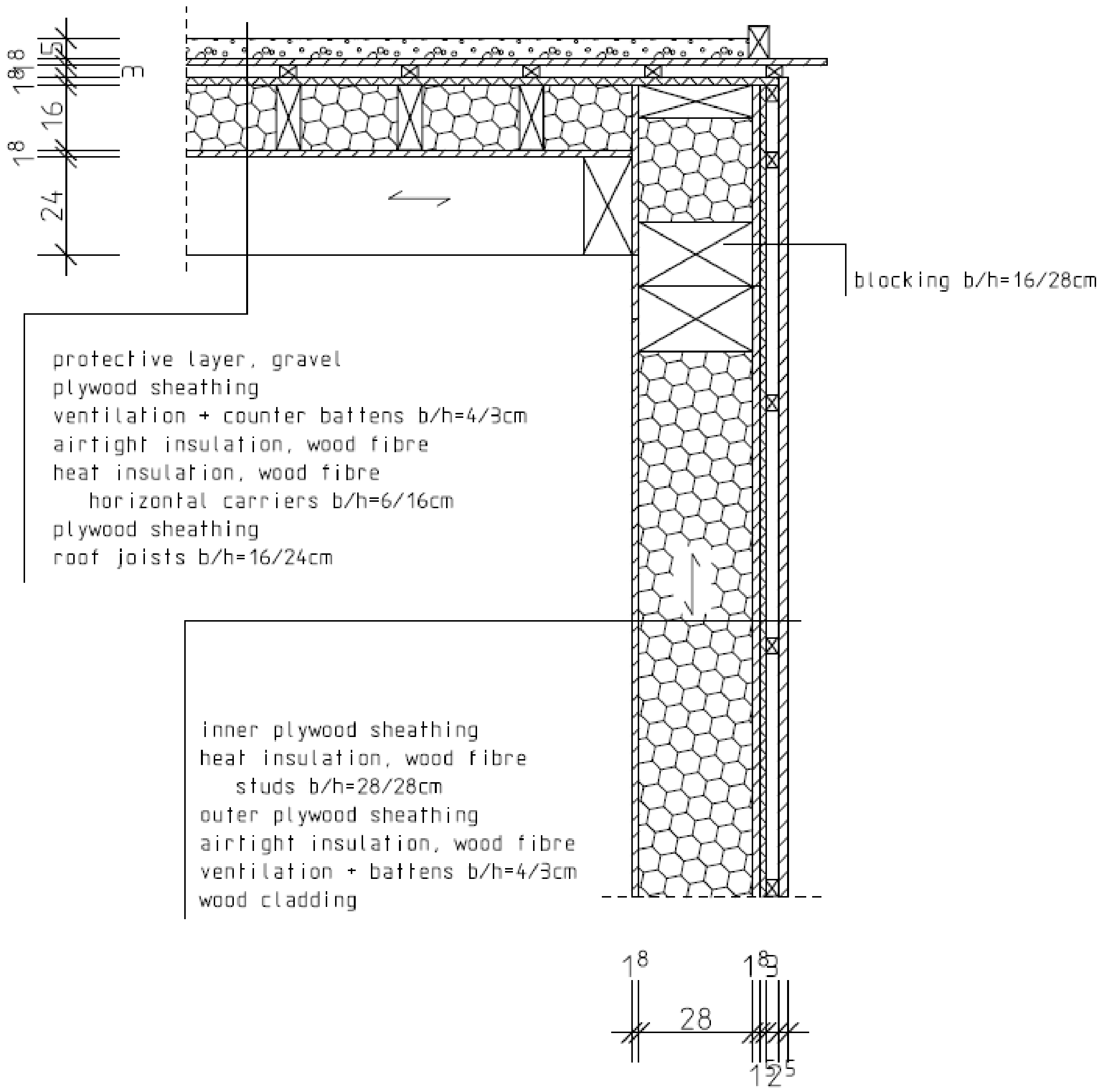


Figure 7: Vertical section through the wall and the roof of the panel construction

c) CLT Construction

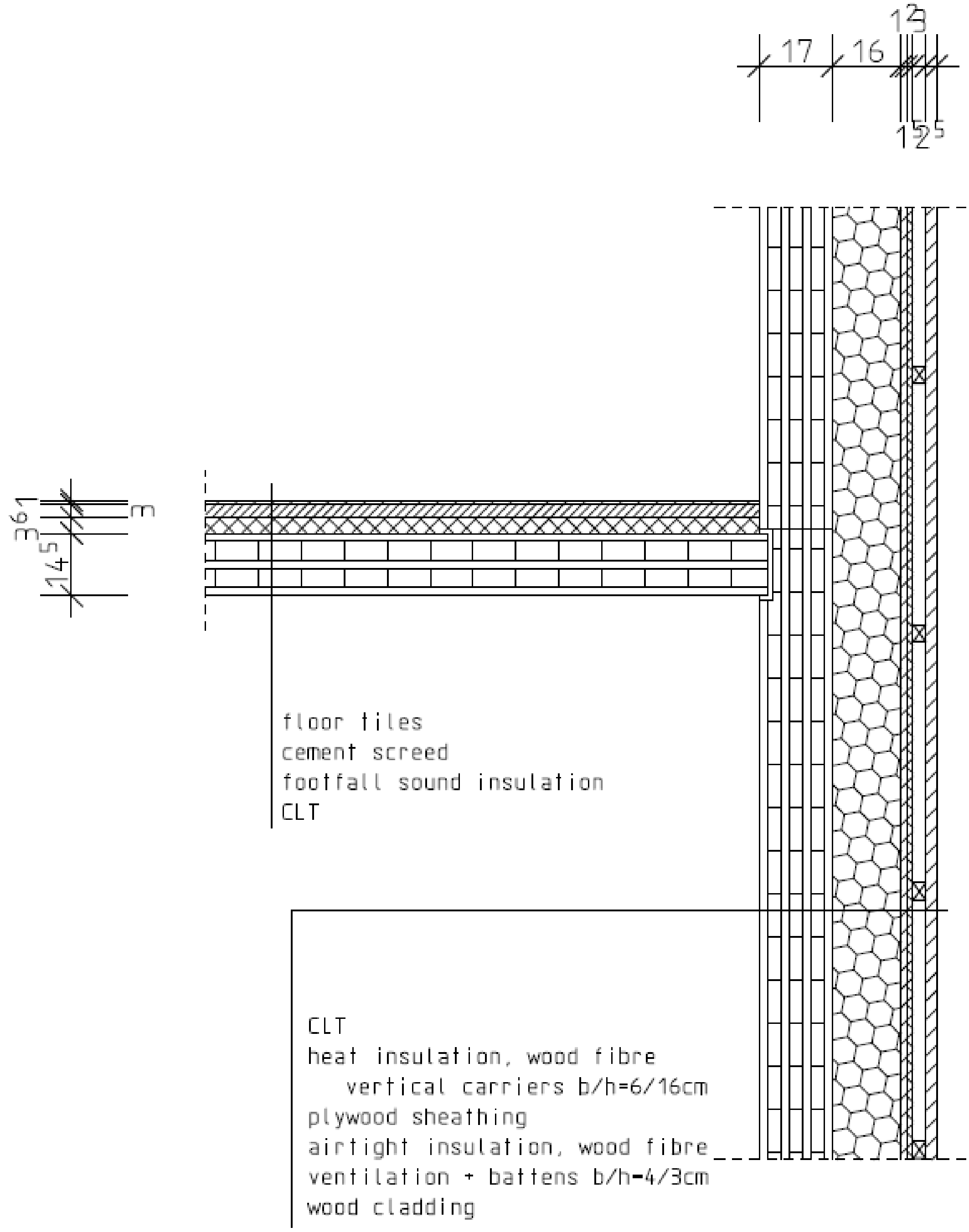


Figure 8: Vertical section through the wall and the floor of the CLT construction

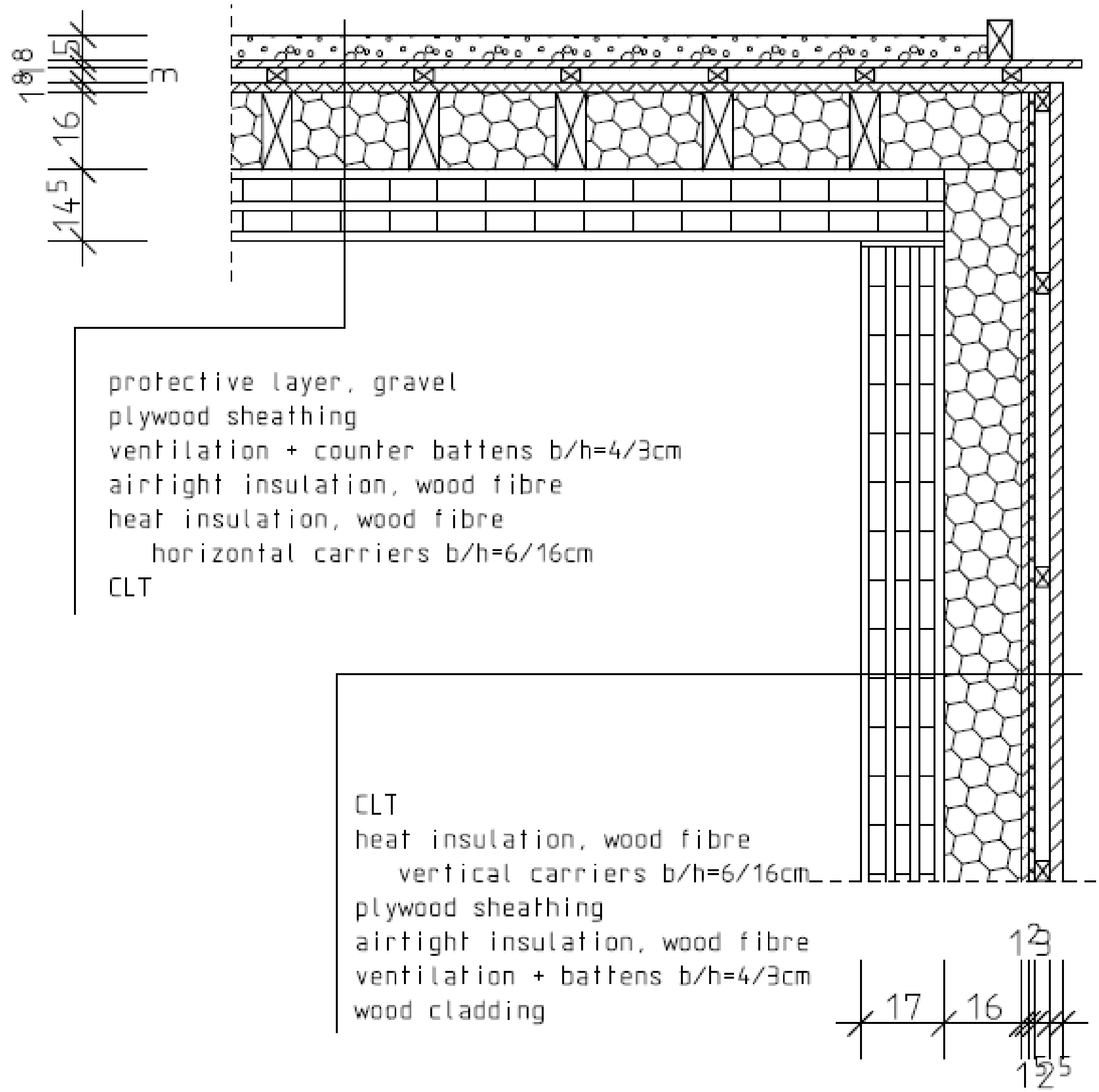


Figure 9: Vertical section through the wall and the roof of the CLT construction

Preliminary Design

Table of Contents

General

Remarks

This documentation summarises the results of the preliminary design. The relevant files are attached for further reference. Technical reference plans are also attached (cf. appendix).

Description of the Building

The subject of this design is the building Treet, located in Bergen, Norway. The exact address is:

Damsgårdsveien 99
5058 Bergen
Hordaland, South-Norway

Three different structures will be designed for this building, using three different timber construction methods:

a) frame construction
b) panel construction
c) CLT construction

The dimensions of the building are a/b/h = 22,34/20,65/47,925 m. It consists of a single level underground car park plus 14 normal storeys, each having a height of 3,195 m. The usage of the building is purely residential. Each storey consists of 5 units, 4 of them are apartments and one is either a storage room or another, smaller apartment.

Each storey has a central corridor and all storeys are connected via a large staircase and a lift in the middle and secondary stairway on one side of the corridor.

The building has flat roofs, with the area of the corridor, the staircases and the lifts protruding one storey higher than the apartments (which gives a height of $47{,}925 + 3{,}195 = 51{,}12$ m at the highest point). Therefore, snow drifts must be considered on the lower roofs.

Loadbearing Concept

a) Frame Construction

The vertical loads from the floors are carried by the floor joists (element reference number 3, cf. element reference plan Pa1.1) which run along the vertical axes. The joists are single-span beams and are supported by the beams (2) which run perpendicular to the joists, along the horizontal axes.

The beams are compound beams consisting of two identical cross-sections that are attached to the columns (1) on both sides. Two additional beams are attached to the outside of the columns in the vertical axes A and F.

To stabilise the structure against wind, diagonals (4) are added in between the outer columns, forming two frames in each direction.

The wind pressure acts on the façade, in which vertical carriers (6) carry the bending moment. The vertical carriers span one storey and transfer the wind loads to the outer beams (2.4) and into the floors. The floors act as diaphragms with the structural sheathing (5) providing the shear stiffness to pass the loads on to the frames on each side.

The diaphragms behave like simply supported beams. The frames are the supports, the beams 2.4 the flanges, taking compression and tension forces, and the sheathing the chord, taking the shear.

Wind suction on the roof was not considered, it can be carried by the battens and horizontal carriers in the roof structure, but is not decisive in comparison to the snow load and self-weight.

b) Panel Construction

The panel construction features twelve prefabricated modules per storey that consist of walls and the floor (the upper side is closed when the next module is placed on top).

The vertical loads are carried by the floor joists (1, cf. element reference plan Pb4.1). They are supported by the floor beams (2) which are attached to the inside of the walls and transfer the loads into the walls. The walls consist of vertical studs (3) which carry the loads, the structural sheathing (4) which provides the shear stiffness and horizontal blocking in between the studs.

The modules 1, 2, 6 and 7 (cf. module reference plan Pb3) are open on one of the four sides. There, the floor is supported by a beam (6) on four columns (5).

To carry the horizontal wind loads, the floors act as diaphragms and the walls as shear walls with the sheathing providing the necessary shear stiffness. To calculate the loads on the shear walls from the wind, a stability analysis must be conducted.

The non-loadbearing walls inside the apartments are not considered for stability to make later changes to the interior layout possible. The double walls where two modules meet are considered as two separate walls.

For the stability analysis, the building is idealised as a vertical cantilever beam with the maximum vertical loads, horizontal shear and global bending moment at the support at the foundation. The share of every wall in shear and in bending is calculated and the vertical loads from the self-weight and the life load are added. To carry the global bending moment, each wall carries a share of this bending moment depending on its stiffness and a normal force depending on its distance to the centre of gravity.

The studs carry the vertical forces in the walls. The studs in the outer walls also carry the horizontal load from the wind and transfer these loads into the diaphragms.

Only selected studs are considered to be loadbearing and have an accordingly larger cross-section compared to the non-loadbearing studs. All studs are arranged on a regular grid which is fitted to the sizes of the plywood panels that form the sheathing of the walls.

For tensile stresses, special hold-down devices function as anchoring. The largest tensile stresses will also occur at the support since the global bending moment, which causes the tension, increases quadratically while the self-weight, which cancels out part of the tension, only increases linearly over the height.

c) CLT Construction

The CLT-construction consists solely of mass timber panels made from CLT. Panels with different thicknesses are used for the different walls (5, cf. element reference plan Pc3.1) and floor slabs (1–4). Between different apartments, double walls are used to increase the sound protection

To carry the horizontal wind loads, the floors act as diaphragms and the walls as shear walls. To calculate the loads on the shear walls from the wind, a stability analysis must be conducted.

The non-loadbearing walls inside the apartments are not considered for stability to make later changes to the interior layout possible. The double walls are considered as two separate walls.

To avoid large compression stresses perpendicular to the grain, the floor slabs are not put in between the wall elements, but set into a groove cut into the lower wall.

To account for fire safety, only CLT-panels with at least 5 layers are used. If the outer layer is destroyed during a fire, the next layer, which is oriented perpendicular to the outer layer, is also rendered ineffective for the bearing of the loads and three layers remain, two in load direction and one perpendicular to that. Thus, the element is still able to carry a reduced amount of loads.

Assumptions

For the preliminary design, all checks are made with characteristic values and reduced preliminary design strengths. The used material properties are summarised in table …

material	property	preliminary design value
solid wood, glulam	strength parallel to the grain	$\sigma_p = 5\ N/mm^2$
	compression strength perpendicular to the grain	$\sigma_{p,c} = 1{,}5\ N/mm^2$
	Young's modulus	$E_p = 10\ 000\ N/mm^2$
plywood	shear strength	$\sigma_{p,v} = 1{,}5\ N/mm^2$
	out-of-plane bending strength	$\sigma_p = 8\ N/mm^2$
	Young's modulus	$E = 4\ 000\ N/mm^2$
CLT	strength in the strong direction	$\sigma_p = 5\ N/mm^2$
	strength in the weak direction	$\sigma_p = 4\ N/mm^2$
	in-plane bending around the strong axis	$\sigma_{p,m} = 4\ N/mm^2$
	compression strength perpendicular to the grain	$\sigma_{p,c} = 1{,}5\ N/mm^2$
	in-plane shear strength	$\sigma_{p,v} = 0{,}6\ N/mm^2$
	Young's modulus	$E_p = 4\ 000\ N/mm^2$
steel	strength	$\sigma_p = 140\ N/mm^2$

For the CLT, the values are derived from the technical approval [1]. This document also includes recommendations concerning the span length of slab elements, which are used for the preliminary design.

The loads in multi-storey buildings change strongly over the height. In this design, only the required cross-sections in the lowest storey, where the loads are highest, are determined. Assuming the forces resulting from the vertical loads to change linearly and the forces from the bending due to the wind loads to change quadratically over the height, reduced cross-sections or the upper storeys are estimated.

Materials

For the preliminary design, no concrete strength classes are defined yet, instead the preliminary design strengths are used (see table above). Only for the CLT a specific material is chosen, because the properties of this material are not defined in standards but by the manufacturer directly.

- wooden members (beams, studs, floor joists, …): glulam
- structural sheathing: plywood
- CLT by Martinsons (distributed in Norway by Splitkon AS)

Software

- Stab2d (simple analysis program for two-dimensional frameworks)
- Microsoft EXCEL ©

Extracts from the calculations with EXCEL can be found in the appendix. The EXCEL-files themselves are also attached for more detailed information.

Literature

[1] Teknisk Godkjenning Martinsons KL-trä
[2] Bautabellen für Ingenieure, 20th edition 2012

The technical approval [1] contains tables with recommended thicknesses for floor slabs that have been used for the preliminary design.

Preliminary Design Formulae

The simplified design formulae used for this preliminary design are taken from [2].

element	property	formula
floor joist	height	$h \approx l/20$
steel bolts, dowels	capacity per fastener (single shear)	$F = 2{,}0 \cdot d^2$
	capacity per fastener (double shear)	$F = 4{,}4 \cdot d^2$
	required space per fastener	$\text{req } A = 30 \cdot d^2$
	capacity at an angle to the grain	$F' = \left(1 - \frac{\alpha}{360}\right) \cdot F$
	capacity with steel plate	$F' = 1{,}25 \cdot F$
fully threaded screws	capacity per screw (axial tension)	$F = 5 \cdot d^2$
	required space per screw	$\text{req } A = 60 \cdot d^2$
nails	capacity per nail (shear)	$F = 3{,}5 \cdot d^2$
d in [cm] A in [cm^2] F in [kN]		

For continuous beams, the reaction forces are higher than for rows with simply supported beams. To account for this continuity effect, a factor of 1,15 is added in cases where it is possible that beams act as continuous beams.

To account for buckling, generally only 70 % of the strength are considered (factor 0,7). For the studs in the panel construction, a factor of 0,8 is used, because the sheathing prevents the studs from buckling around in their weak direction.

Loads

Simplified loads are used for the preliminary design.

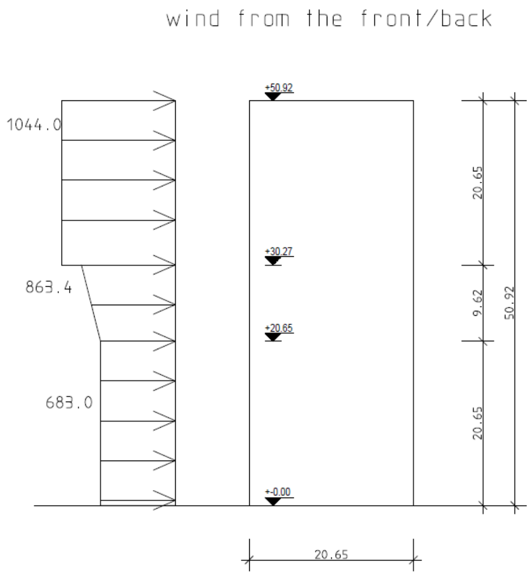
vertical loads

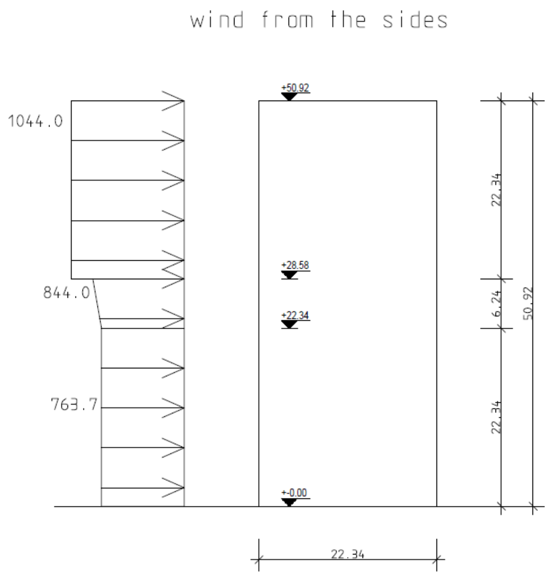
live load	$q_k = 2{,}0\ \mathrm{kN/m^2}$
self-weight floors	$g_k = 2{,}0\ \mathrm{kN/m^2}$
self-weight walls	$g_k = 1{,}0\ \mathrm{kN/m^2}$
snow load	$s_k = 2{,}0\ \mathrm{kN/m^2}$

reduction factor for life load over several storeys

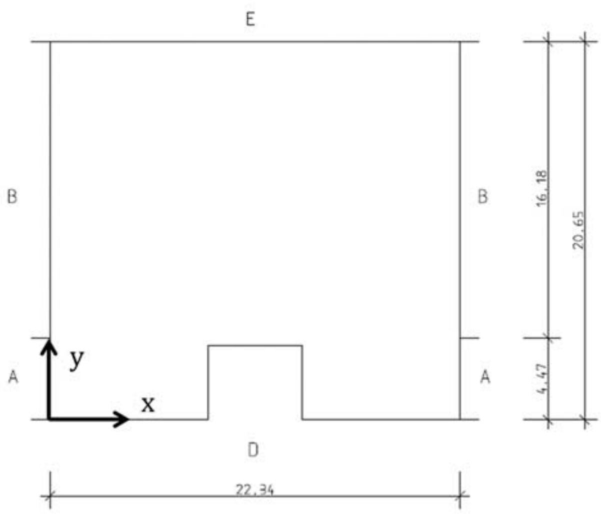
$\alpha_n = 0{,}74$

wind loads

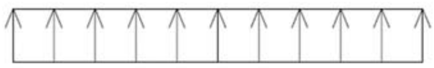




wind from the front



	$c_{pe,10}$
A	-1,2
B	-0,8
D	+0,8
E	-0,57



horizontal wind force

$$q_{p,tot} = 1{,}044 \cdot 20{,}65 + \frac{0{,}8634 + 0{,}6830}{2} \cdot 9{,}62 + 0{,}6830 \cdot 20{,}65 = 43{,}1 \text{ kN/m}$$

$$Q_{p,tot} = 43{,}1 \cdot 22{,}34 = 962{,}9 \text{ kN}$$

$$c_{pe}(D + E) = 0{,}8 + 0{,}57 = 1{,}37$$

$$W_y = 1{,}37 \cdot 962{,}9 = 1319 \text{ kN}$$

bending moment of the cantilever beam

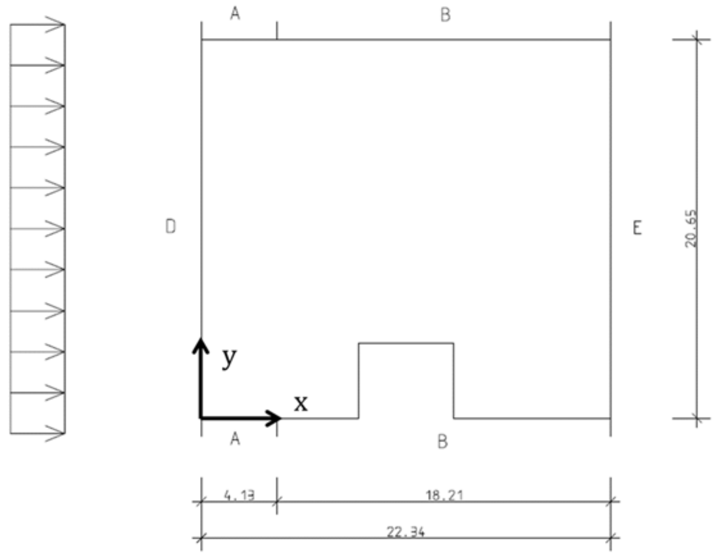
$$\begin{aligned} m_{qp,tot} &= 1{,}044 \cdot 20{,}65 \cdot \left(50{,}92 - \frac{20{,}65}{2}\right) \\ &\quad + 0{,}683 \cdot 9{,}62 \cdot \left(20{,}65 + \frac{9{,}62}{2}\right) \\ &\quad + \frac{1}{2} \cdot (0{,}8643 - 0{,}683) \cdot 9{,}62 \cdot \left(20{,}65 + \frac{2}{3} \cdot 9{,}62\right) \\ &\quad + 0{,}683 \cdot 20{,}65 \cdot \frac{20{,}65}{2} \\ &\quad = 1212 \text{ kNm/m} \end{aligned}$$

$$M_{qp,tot} = 1212 \cdot 22{,}34 = 27076 \text{ kNm}$$

$$c_{pe}(D + E) = 0{,}8 + 0{,}57 = 1{,}37$$

$$M_x = -1{,}37 \cdot 27076 = -37094 \text{ kNm}$$

wind from the sides



	$c_{pe,10}$
A	-1,2
B	-0,8
D	+0,8
E	-0,57

total horizontal wind force

$$q_{p,tot} = 1{,}044 \cdot 22{,}34 + \frac{0{,}844 + 0{,}7637}{2} \cdot 6{,}24 + 0{,}7637 \cdot 22{,}34 = 45{,}4 \text{ kN/m}$$

$$Q_{p,tot} = 45{,}4 \cdot 20{,}65 = 937{,}5 \text{ kN}$$

$$c_{pe}(D + E) = 0{,}8 + 0{,}56 = 1{,}36$$

$$W_x = 1{,}36 \cdot 937{,}5 = 1275 \text{ kN}$$

bending moment of the cantilever beam

$$m_{qp,tot} = 1{,}044 \cdot 22{,}34 \cdot \left(50{,}92 - \frac{22{,}34}{2}\right)$$

$$+0{,}7637 \cdot 6{,}24 \cdot \left(22{,}34 + \frac{6{,}24}{2}\right)$$

$$+\frac{1}{2} \cdot (0{,}844 - 0{,}7637) \cdot 6{,}24 \cdot \left(22{,}34 + \frac{2}{3} \cdot 6{,}24\right)$$

$$+0{,}7637 \cdot 22{,}34 \cdot \frac{22{,}34}{2}$$

$$= 1246 \text{ kNm/m}$$

$$M_{qp,tot} = 1246 \cdot 20{,}65 = 25722 \text{ kNm}$$

$$c_{pe}(D + E) = 0{,}8 + 0{,}56 = 1{,}36$$

$$M_y = 1{,}36 \cdot 25722 = 34982 \text{ kNm}$$

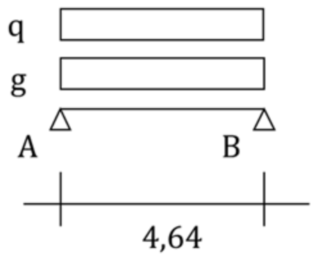
a) Frame Construction

Element Reference Overview

reference no	element	page
1	columns	
1.1	inner columns	27
1.2	corner columns	27
1.3	side columns	27
1.4	side columns	27
2	beams	
2.1	beam	16
2.2	beam	14
2.3	beam	18
2.4	beam	22
3	joists	
3.1	joists	12
3.2	joists	13
4	diagonal	31
5	structural sheathing of the floors	33
6	vertical façade carriers	35
7	diaphragm	20
8	frame	21
	connections overview	36

3.1 Joists

system



$l = \frac{9,275}{2} = 4,64 \text{ m}$

$h \approx \frac{l}{20} = \frac{464}{20} = 23,2 \text{ cm}$

- selected spacing: $e = 65 \text{ cm}$

loads

$q_k = 2,0 \text{ kN/m}^2$

$g_k = 2,0 \text{ kN/m}^2$

loads on one joist

$q_k = 2,0 \cdot 0,65 = 1,3 \text{ kN/m}^2$

$g_k = 1,3 \text{ kN/m}^2$

internal forces

$\max M_1 = 2 \cdot 0,125 \cdot 1,3 \cdot 4,64^2 = 7,0 \text{ kNm}$

$\text{req } W_y = 700/0,5 = 1400 \text{ cm}^3$

selected dimensions

$$\boxed{\begin{array}{c} b/h = 14/24 \\ e = 65 \text{ cm} \\ I_y = 16128 \text{ cm}^4 \\ W_y = 1344 \text{ cm}^3 \end{array}}$$

deformation

$\max \delta = \frac{2 \cdot 1,3/100 \cdot 464^4}{76,8 \cdot 1000 \cdot 16128} = 0,973 \text{ cm}$

$< l/250 = 464/250 = 1,86 \text{ cm}$

max reaction forces

$A_g = 0,5 \cdot 1,3 \cdot 4,64 = 3,02 \text{ kN}$

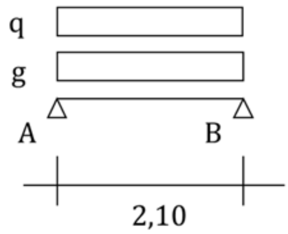
$A_q = 0,5 \cdot 1,3 \cdot 4,64 = 3,02 \text{ kN}$

$A = 6,03 \text{ kN}$

3.2 Joists

system

$l = 2{,}10$ m



loads

$q_k = 2{,}0 \ kN/m^2$

$g_k = 2{,}0 \ kN/m^2$

loads on one joist

$q_k = 2{,}0 \cdot 0{,}65 = 1{,}3 \ kN/m^2$

$g_k = 1{,}3 \ kN/m^2$

- selected spacing: $e = 65$ cm

selected dimensions

$b/h = 14/24$
$e = 65$ cm
$I_y = 16128 \ cm^4$
$W_y = 1344 \ cm^3$

- joist 3.2 is not decisive, the same dimensions are selected as for joist 3.1

max reaction forces

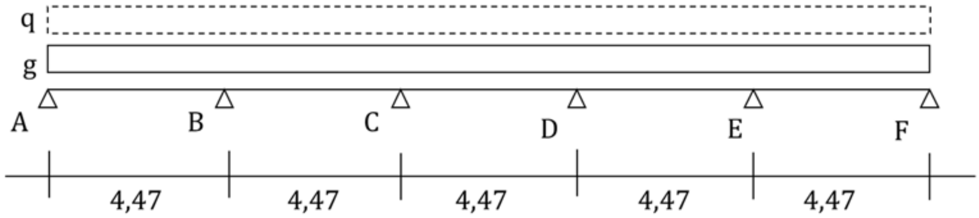
$A_g = 0{,}5 \cdot 1{,}3 \cdot 2{,}1 = 1{,}37$ kN

$A_q = 0{,}5 \cdot 1{,}3 \cdot 2{,}1 = 1{,}37$ kN

$A = 2{,}73$ kN

2.2 Beam

system



$l = 4{,}47 \text{ m}$

- for simplicity, an average span is calculated

loads

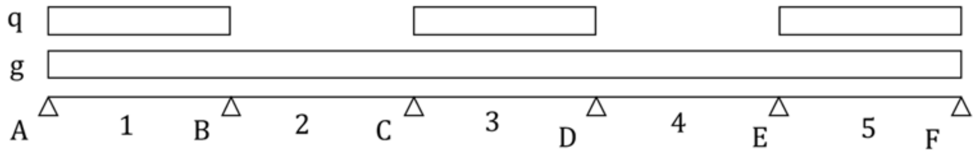
$$g_k = \frac{3{,}02}{0{,}65} = 4{,}65 \text{ kN/m}$$

$$q_k = 4{,}65 \text{ kN/m}$$

- → reaction force A @ 3.1
- 0,65 m is the spacing of the floor joists, the reaction forces are converted to distributed loads

internal forces

max moment in the first field



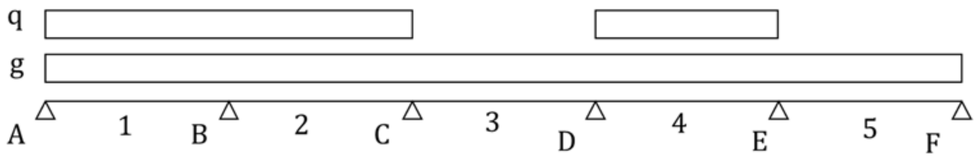
$\max M_1 = (0{,}078 + 0{,}100) \cdot 4{,}65 \cdot 4{,}47^2 = 16{,}54 \text{ kNm}$

$\text{req } W_y = 1654/0{,}5 = 3308 \text{ cm}^3$

corresponding moment at the support B

$M_B = (-0{,}105 - 0{,}053) \cdot 4{,}65 \cdot 4{,}47^2 = -14{,}7 \text{ kNm}$

max moment at the support



$\max M_B = (-0{,}105 - 0{,}12) \cdot 4{,}65 \cdot 4{,}47^2 = 20{,}9 \text{ kNm}$

$\text{req } W_y = 2090/0{,}5 = 4181 \text{ cm}^3$

selected dimensions

$b/h = 16/42$
$I_y = 98784 \text{ cm}^4$
$W_y = 4704 \text{ cm}^3$

a) Frame Construction

deformation

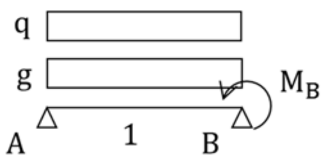
$$\max \delta = \frac{2 \cdot 4{,}65/100 \cdot 447^4}{76{,}8 \cdot 1000 \cdot 98784}$$

$$+ \frac{-1470}{16 \cdot 1000 \cdot 98784} \cdot 447^2$$

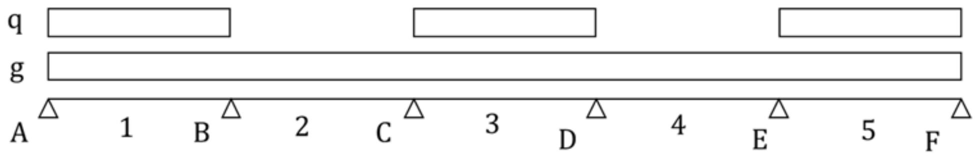
$$= 0{,}489 - 0{,}186 = 0{,}30 \text{ cm}$$

$$< l/250 = 447/250 = 1{,}79 \text{ cm}$$

substitute system:



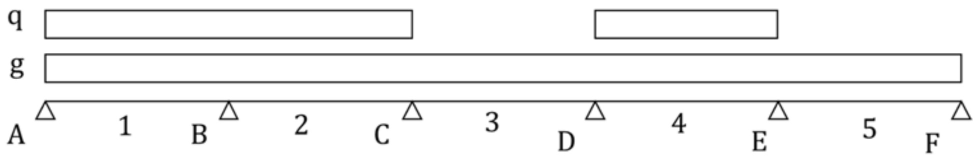
max reaction forces



$A_g = 0{,}395 \cdot 4{,}65 \cdot 4{,}47 = 8{,}2 \text{ kN}$

$A_q = 0{,}447 \cdot 4{,}65 \cdot 4{,}47 = 9{,}3 \text{ kN}$

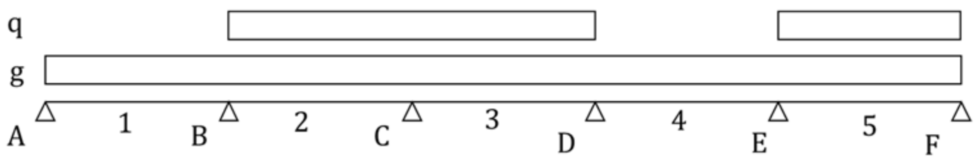
$A = 17{,}5 \text{ kN}$



$B_g = 1{,}132 \cdot 4{,}65 \cdot 4{,}47 = 23{,}5 \text{ kN}$

$B_q = 1{,}218 \cdot 4{,}65 \cdot 4{,}47 = 25{,}3 \text{ kN}$

$B = 48{,}8 \text{ kN}$



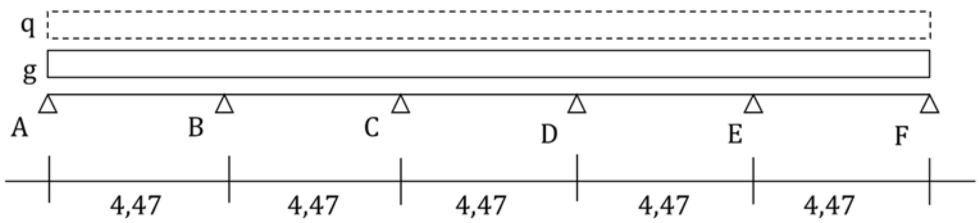
$C_g = 0{,}974 \cdot 4{,}65 \cdot 4{,}47 = 20{,}2 \text{ kN}$

$C_q = 1{,}167 \cdot 4{,}65 \cdot 4{,}47 = 24{,}3 \text{ kN}$

$C = 44{,}5 \text{ kN}$

2.1 Beam

system



l = 4,47 m

- average span

loads

$$g_k = \frac{3,02}{0,65} = 4,65 \text{ kN/m}$$

- → reaction force A @ 3.1

$q_k = 4,65$ kN/m

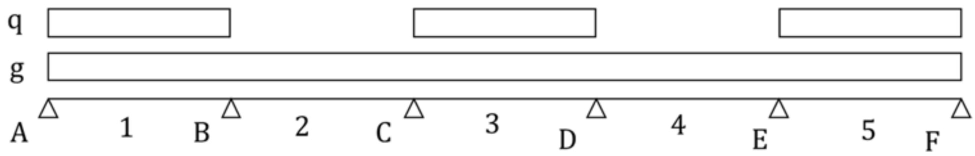
dimensions

$$\boxed{\begin{array}{c} b/h = 16/42 \\ I_y = 98784 \text{ cm}^4 \\ W_y = 4704 \text{ cm}^3 \end{array}}$$

- beam 2.1 is not decisive, the same dimensions are selected as for beam 2.2

a) Frame Construction

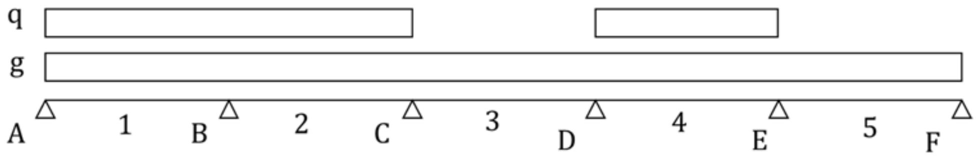
max reaction forces



$A_g = 0{,}395 \cdot 4{,}65 \cdot 4{,}47 = 8{,}2 \text{ kN}$

$A_q = 0{,}447 \cdot 4{,}65 \cdot 4{,}47 = 9{,}3 \text{ kN}$

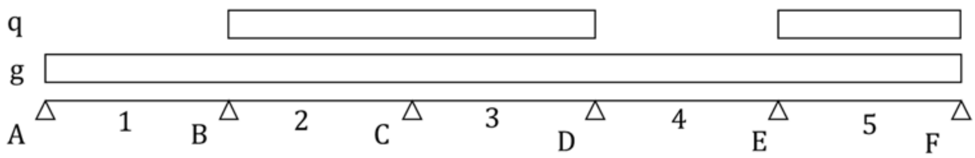
$A = 17{,}5 \text{ kN}$



$B_g = 1{,}132 \cdot 4{,}65 \cdot 4{,}47 = 23{,}5 \text{ kN}$

$B_q = 1{,}218 \cdot 4{,}65 \cdot 4{,}47 = 25{,}3 \text{ kN}$

$B = 48{,}8 \text{ kN}$



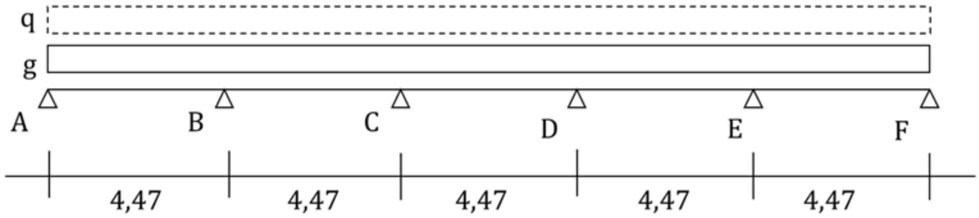
$C_g = 0{,}974 \cdot 4{,}65 \cdot 4{,}47 = 20{,}2 \text{ kN}$

$C_q = 1{,}167 \cdot 4{,}65 \cdot 4{,}47 = 24{,}3 \text{ kN}$

$C = 44{,}5 \text{ kN}$

2.3 Beam

system



$l = 4{,}47$ m

- average span

loads

$$g_k = \frac{1{,}37}{0{,}65} = 2{,}1 \text{ kN/m}$$

- → reaction force A @ 3.2

$q_k = 2{,}1$ kN/m

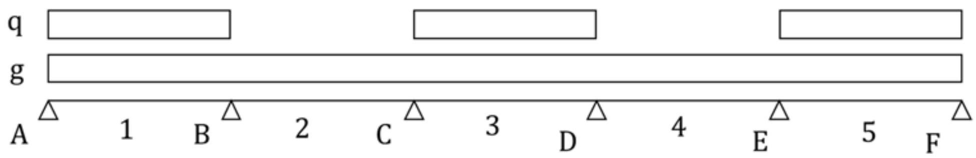
dimensions

$$\boxed{\begin{array}{c} b/h = 16/42 \\ I_y = 98784 \text{ cm}^4 \\ W_y = 4704 \text{ cm}^3 \end{array}}$$

- beam 2.3 is not decisive, the same dimensions are selected as for beam 2.2

a) Frame Construction

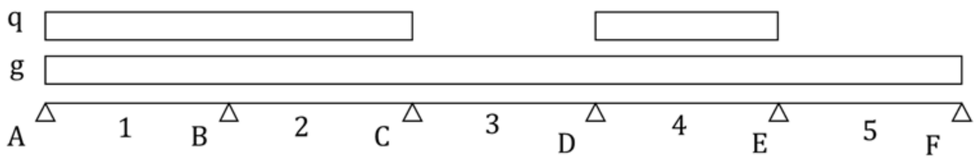
max reaction forces



$A_g = 0{,}395 \cdot 2{,}1 \cdot 4{,}47 = 3{,}7$ kN

$A_q = 0{,}447 \cdot 2{,}1 \cdot 4{,}47 = 4{,}2$ kN

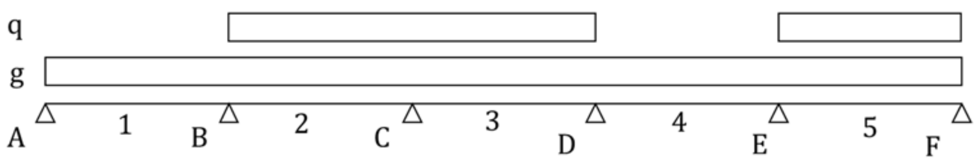
$A = 7{,}9$ kN



$B_g = 1{,}132 \cdot 2{,}1 \cdot 4{,}47 = 10{,}6$ kN

$B_q = 1{,}218 \cdot 2{,}1 \cdot 4{,}47 = 11{,}4$ kN

$B = 22{,}1$ kN



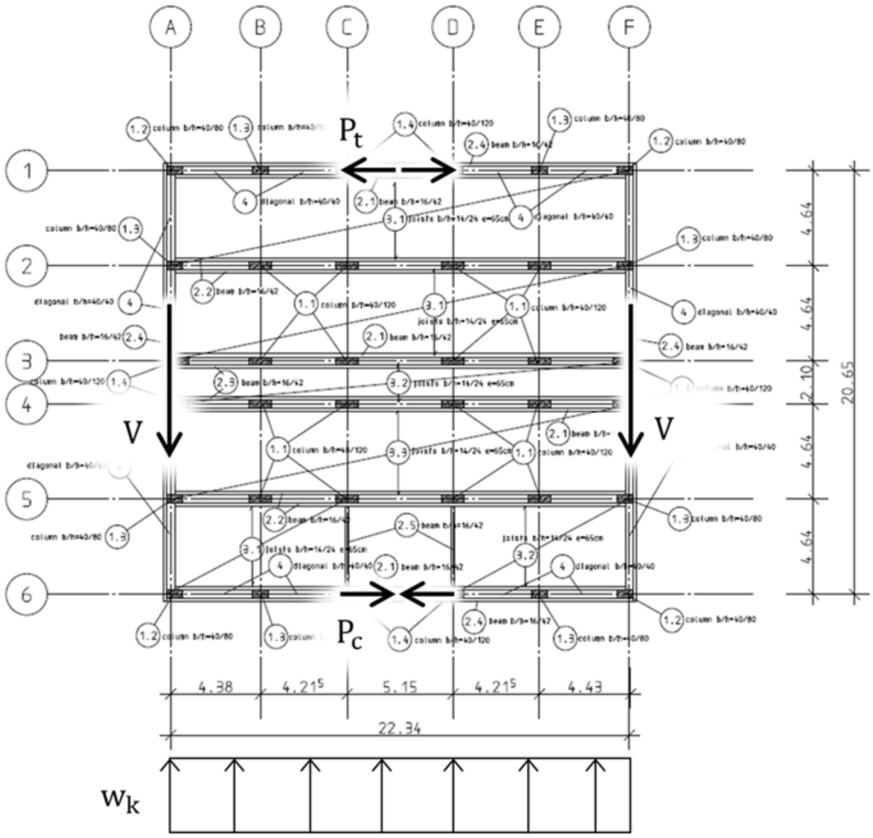
$C_g = 0{,}974 \cdot 2{,}1 \cdot 4{,}47 = 9{,}1$ kN

$C_q = 1{,}167 \cdot 2{,}1 \cdot 4{,}47 = 11{,}0$ kN

$C = 20{,}1$ kN

7 Diaphragm

system



wind load

$\max w_e(D + E) = 1{,}044 \cdot (0{,}8 + 0{,}57)$
$= 1{,}43\ \mathrm{kN/m^2}$

$w_k = 1{,}15 \cdot 1{,}43 \cdot 3{,}195 = 5{,}25\ \mathrm{kN/m}$

- wind from the front becomes decisive
- h = 3,195 m is the height of one storey
- the vertical carriers in the façade (6) in between the beams 2.4 are single-span beams; to account for continuity effects, a factor of 1,15 is added

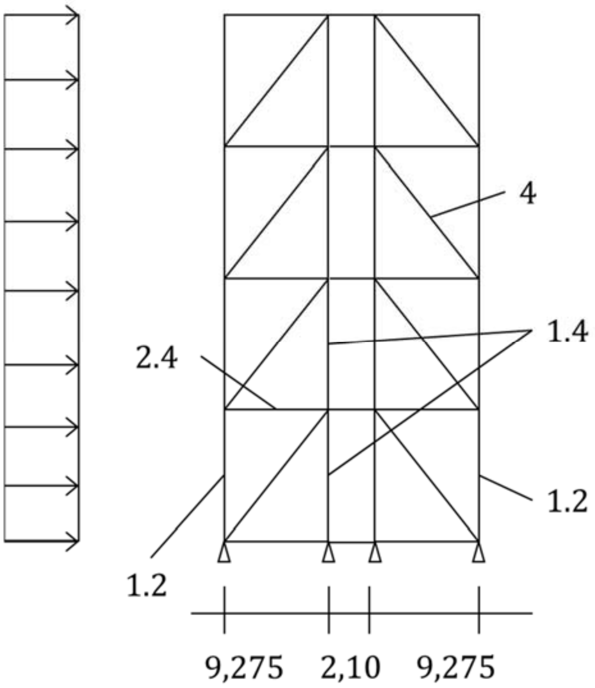
internal forces

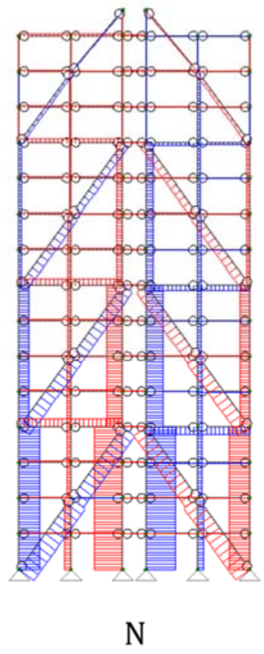
$$|P_t| = |P_c| = \frac{5{,}25 \cdot 22{,}34^2}{8} / 20{,}65 = 15{,}9\ \mathrm{kN}$$

$$V = \frac{5{,}25 \cdot 22{,}34}{2} = 58{,}6\ \mathrm{kN}$$

8 Frame

system





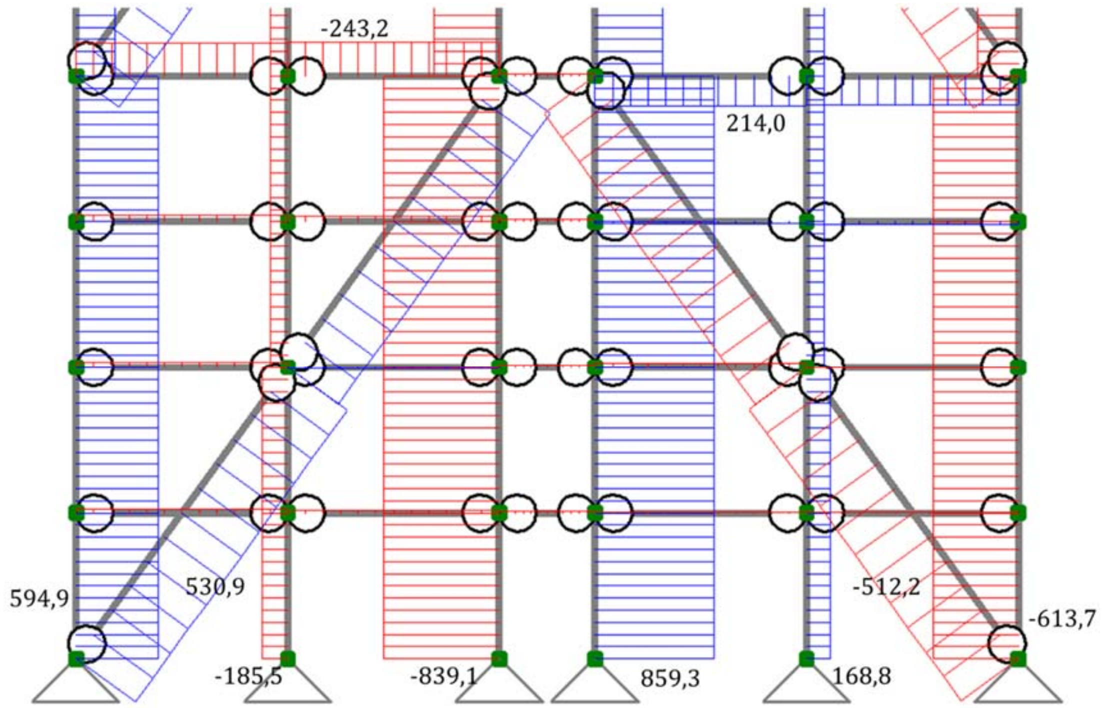
wind load

$\max w_e(D+E) = 1{,}044 \cdot (0{,}8 + 0{,}57)$
$= 1{,}43\ \mathrm{kN/m^2}$

- wind from the front becomes decisive

internal forces

normal forces [kN]



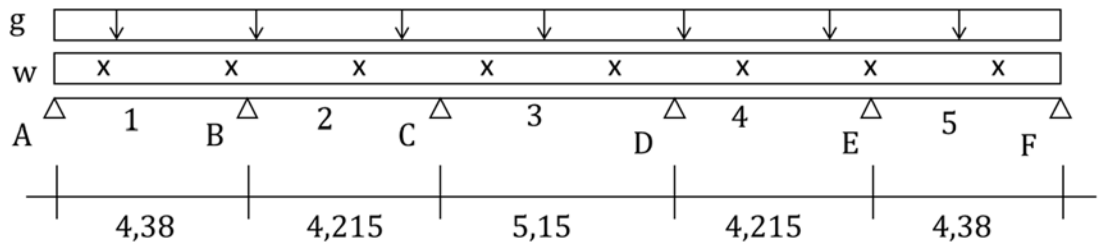
2.4 Beam (Front)

The beam 2.4 on the front of the building is subjected multiple loads:

- horizontal bending from the wind
- vertical bending from the self-weight of the façade
- compression or tension as part of the diaphragm

Wind from the front becomes decisive.

system



loads

self-weight of the façade (vertical)

$g_k = 1 \text{ kN/m}^2$

$g_k = 1{,}15 \cdot 1 \cdot 3{,}195 = 3{,}67 \text{ kN/m}$

- factor 1,15 to account for continuity effects

wind load (wind from the front, wind area D, horizontal)

$w_e(D) = 1{,}044 \cdot 0{,}8 = 0{,}84 \text{ kN/m}^2$

$w_k = 1{,}15 \cdot 0{,}84 \cdot 3{,}195 = 3{,}09 \text{ kN/m}$

compression from the diaphragm

$|P_c| = 15{,}9 \text{ kN}$

dimensions

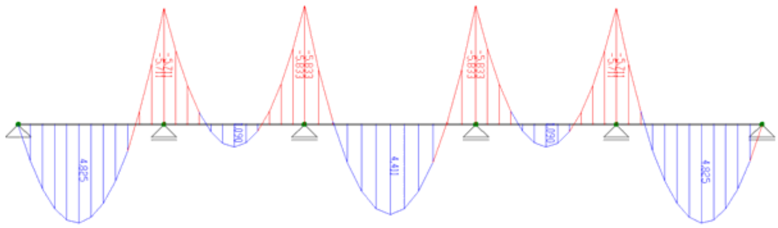
$$\boxed{\begin{array}{c} b/h = 16/42 \\ A = 672 \text{ cm}^2 \\ W_y = 4704 \text{ cm}^3 \\ W_z = 1792 \text{ cm}^3 \end{array}}$$

- the same dimensions are selected as for beam 2.2

a) Frame Construction

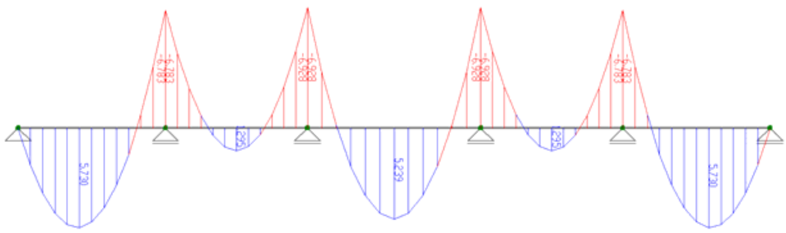
internal forces

wind load



$\max M_3 = 4{,}41 \text{ kNm}$

self-weight of the façade



$\max M_3 = 5{,}24 \text{ kNm}$

$$\sigma = \frac{441}{1792} + \frac{524}{4704} + \frac{15{,}9}{672} = 0{,}38 \text{ kN/cm}^2 < 0{,}5 \text{ kN/cm}^2$$

- For the wind load, the beam is bent around its weak axis (z) and for the self-weight of the façade around its strong axis (y).

max reaction forces

wind load

$A_w = 5{,}46 \text{ kN}$

$B_w = 14{,}55 \text{ kN}$

$C_w = 14{,}5 \text{ kN}$

self-weight of the façade

$A_g = 6{,}49 \text{ kN}$

$B_g = 17{,}3 \text{ kN}$

$C_g = 17{,}2 \text{ kN}$

2.4 Beam (Side)

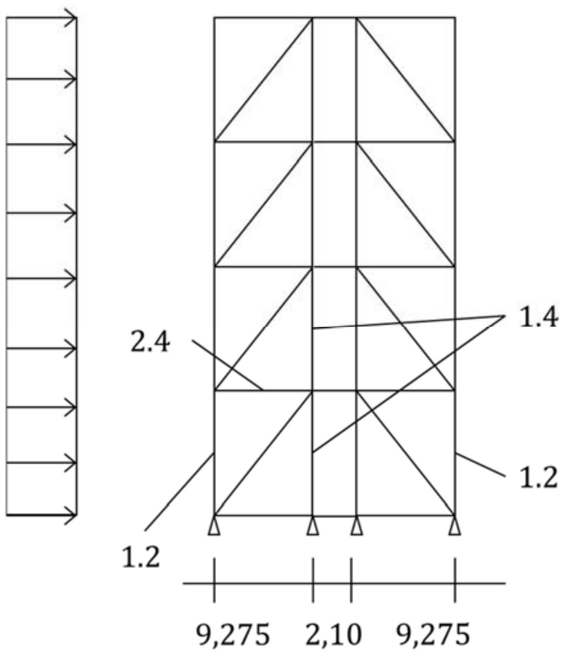
The beam 2.4 on the side of the building is subjected multiple loads:

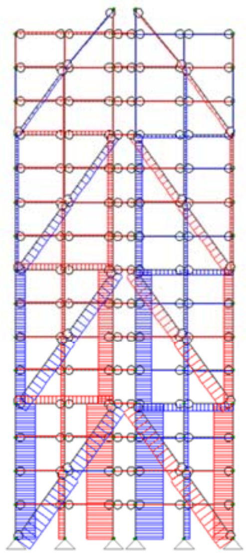
- horizontal bending from the wind
- vertical bending from the self-weight of the façade
- compression or tension as part of the frame (together with the outer columns and the diagonals)

Wind from the front becomes decisive.

system

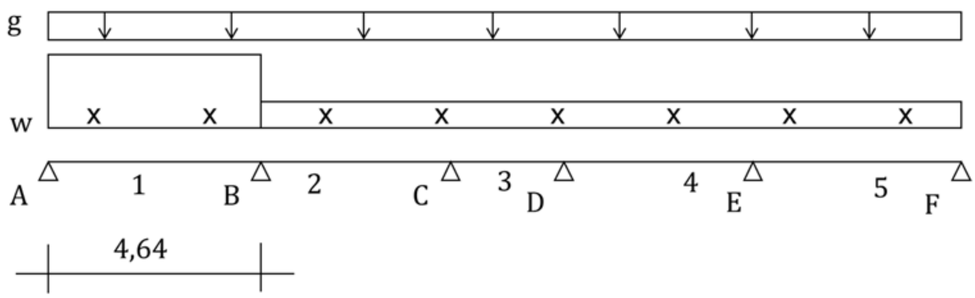
global system (frame)





N

local system



loads

self-weight of the façade (vertical)

$g_k = 1 \text{ kN/m}^2$

$g_k = 1{,}15 \cdot 1 \cdot 3{,}195 = 3{,}67 \text{ kN/m}$

- factor 1,15 to account for continuity effects

a) Frame Construction

wind load (wind from the front, wind area A and B, horizontal)

$$w_e(A) = 1{,}2 \cdot 1{,}044 = 1{,}25 \text{ kN/m}^2$$

$$w_k = 1{,}15 \cdot 1{,}25 \cdot 3{,}195 = 4{,}59 \text{ kN/m}$$

$$w_e(B) = 0{,}8 \cdot 1{,}044 = 0{,}84 \text{ kN/m}^2$$

$$w_k = 1{,}15 \cdot 0{,}84 \cdot 3{,}195 = 3{,}09 \text{ kN/m}$$

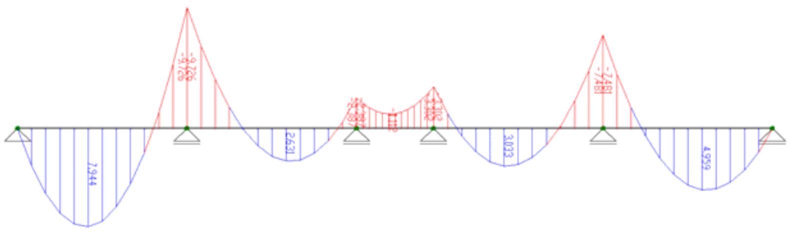
dimensions

- the same dimensions are selected as for beam 2.2

$$\boxed{\begin{gathered} b/h = 16/42 \\ A = 672 \text{ cm}^2 \\ W_y = 4704 \text{ cm}^3 \\ W_z = 1792 \text{ cm}^3 \end{gathered}}$$

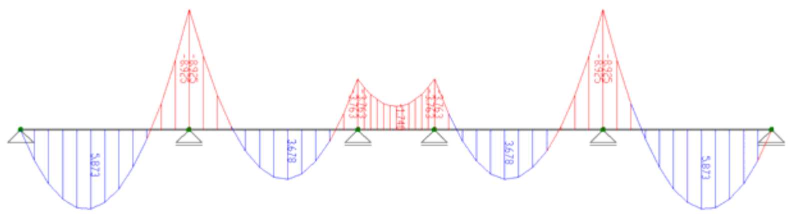
internal forces

wind load



$$\max M_1 = 7{,}94 \text{ kNm}$$

self-weight of the façade



$$\max M_1 = 5{,}87 \text{ kNm}$$

normal compression force from the frame

$$N = 243 \text{ kN}$$

check

- for the wind load, the beam is bent around its weak axis (z) and for the self-weight of the façade around its strong axis (y)

$$\sigma = \frac{794}{1792} + \frac{587}{4704} + \frac{243}{672} = 0{,}93 \frac{\text{kN}}{\text{cm}^2} > 0{,}5 \text{ kN/cm}^2$$

- wood has good stress redistribution properties, which are not considered here, the result is therefore accepted for the preliminary design

a) Frame Construction

max reaction forces

wind load

$A_w = 8{,}55 \text{ kN}$

$B_w = 21{,}5 \text{ kN}$

$C_w = 9{,}94 \text{ kN}$

self-weight of the façade

$A_g = 6{,}59 \text{ kN}$

$B_g = 20{,}1 \text{ kN}$

$C_g = 11{,}3 \text{ kN}$

a) Frame Construction

1 Columns

loads

self-weight and live load

→ reaction forces of the beams

wind load (wind from the front, wind area D + E)

horizontal wind force

$W_{tot} = 1319\ kN$

$W = 1319/2 = 659{,}5\ kN$

- one frame takes half the total wind force

bending moment of the cantilever beam

$M_{tot} = 1{,}37 \cdot 27076 = 37094\ kNm$

$M = 37094/2 = 18547\ kNm$

- one shear frame takes half the total moment

internal forces

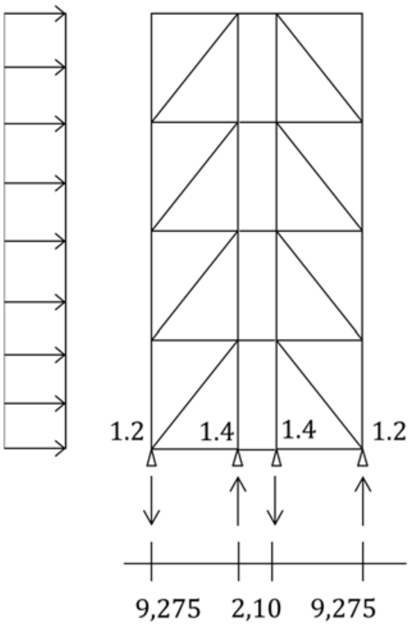
self-weight and live load

- the load per storey N_i is added up for the roof and the 14 storeys with a reduction factor $\alpha_n = 0{,}74$ for the live load

a) Frame Construction

column	load	
1.1	$N_{g,i} = 2 \cdot 23{,}5 = 47{,}0$ kN $N_{q,i} = 2 \cdot 25{,}3 = 50{,}6$ kN $N = 47{,}0 \cdot 15 + 50{,}6 \cdot (1 + 14 \cdot 0{,}74) = 1600$ kN	• 2 x reaction force B @ 2.2 (each column carries the load from two beams)
1.2	$N_{g,i} = 8{,}2 + 6{,}49 + 6{,}59 = 21{,}3$ kN $N_{q,i} = 9{,}3$ kN $N = 21{,}3 \cdot 15 + 9{,}3 \cdot (1 + 14 \cdot 0{,}74) = 425{,}1$ kN	• reaction force A @ 2.1 • reaction force A @ 2.4 (front) • reaction force A @ 2.4 (side)
1.3	$N_{g,i} = 2 \cdot 8{,}2 + 20{,}1 = 36{,}5$ kN $N_{q,i} = 2 \cdot 9{,}3 = 18{,}6$ kN $N = 36{,}5 \cdot 15 + 18{,}6 \cdot (1 + 14 \cdot 0{,}74) = 758{,}8$ kN	• 2 x reaction force A @ 2.2 • reaction force B @ 2.4 (side)
1.4	$N_{g,i} = 8{,}2 + 3{,}7 + 11{,}3 = 23{,}2$ kN $N_{q,i} = 9{,}3 + 4{,}2 = 13{,}5$ kN $N = 23{,}2 \cdot 15 + 13{,}5 \cdot (1 + 14 \cdot 0{,}74) = 501{,}4$ kN	• reaction force A @ 2.1 + A @ 2.3 • reaction force C @ 2.4 (side)

wind load



- The frames consist of two half frames with diagonals that are connected via the beams 2.4.

columns 1.2, 1.4

$$N \approx \frac{18547/2}{9{,}275} = 1000 \text{ kN}$$

column 1.3

$$N \approx 1000/2 = 500 \text{ kN}$$

- the normal forces in the columns are estimated: the moment is distributed over the two half frames, where a pair of forces creates the reaction moment

a) Frame Construction

This table sums up the results for all columns. The required cross-sections are calculated considering a factor of 0,7 for the compression capacity to account for buckling.

column	self-weight and live load [kN]	wind load [kN]	total load [kN]	A_{req} [cm^2]
1.1	1600	0	1600	4571
1.2	425,1	1000	1425,1	4072
1.3	758,8	500	1258,8	3597
1.4	501,4	1000	1501,4	4290

selected dimensions

$$\boxed{\begin{array}{c} b/h = 40/120 \\ A = 4800\ cm^2 \end{array}}$$

- in the first step, all columns have the same dimensions

With those estimated dimensions and the estimated dimensions of the diagonals (see below), a simple two-dimensional framework analysis is conducted to obtain more accurate results for the forces due to the wind. The results are shown in the following table. The dimensions of the columns are adjusted.

1. step

column	self-weight and live load [kN]	wind load [kN]	total load [kN]	A_{req} [cm^2]	selected dimensions
1.1	1600	0	1600	4571	40/120
1.2	425,1	636,6	1062	3034	40/80
1.3	758,8	232,3	991	2831	40/80
1.4	501,4	815,1	1317	3763	40/120

2. step

The analysis is repeated with the new dimensions which leads to the following final results (cf. 8 Frame).

column	self-weight and live load [kN]	wind load [kN]	total load [kN]	A_{req} [cm^2]	selected dimensions
1.1	1600	0	1600	4571	40/120
1.2	425,1	613,7	1039	2968	40/80
1.3	758,8	185,5	944	2698	40/80
1.4	501,4	839,1	1341	3830	40/120

The cross-section of the column can decrease over the height of the building because of the decreasing loads. Assuming the loads due to the self-weight and the life load (N) to decrease linearly and the loads due to the wind (M) to decrease quadratically, the following cross-sections are calculated:

a) Frame Construction

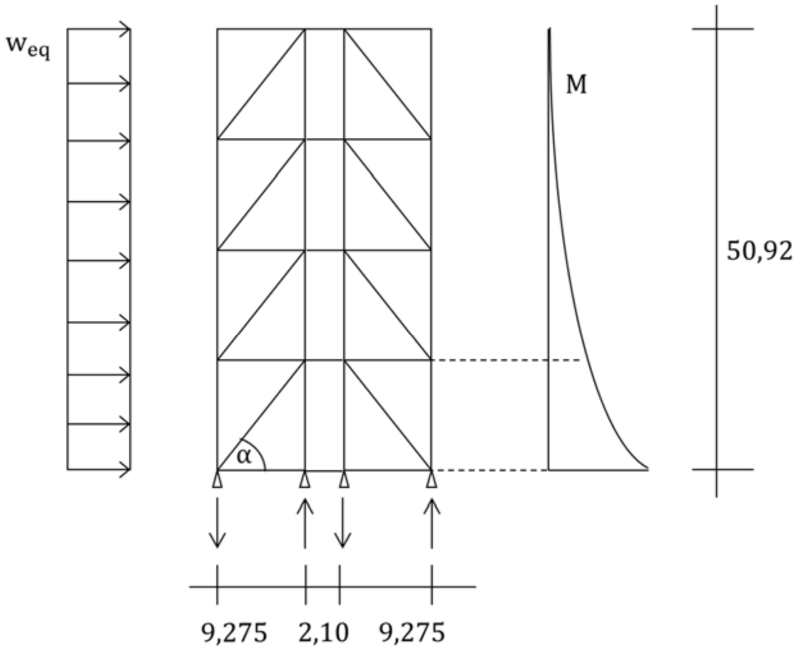
			column 1.1	column 1.2	column 1.3	column 1.4
storey no	M	N	b/h	b/h	b/h	b/h
-	[%]	[%]	[cm/cm]	[cm/cm]	[cm/cm]	[cm/cm]
0	100,00	100,00	40/120	40/80	40/80	40/120
1	87,11	93,33	40/120	40/80	40/80	40/120
2	75,11	86,67	40/120	40/80	40/80	40/120
3	64,00	80,00	40/120	40/80	40/80	40/120
4	53,78	73,33	40/100	40/60	40/60	40/80
5	44,44	66,67	40/100	40/60	40/60	40/80
6	36,00	60,00	40/100	40/60	40/60	40/80
7	28,44	53,33	40/100	40/60	40/60	40/80
8	21,78	46,67	40/60	40/40	40/40	40/60
9	16,00	40,00	40/60	40/40	40/40	40/60
10	11,11	33,33	40/60	40/40	40/40	40/60
11	7,11	26,67	40/60	40/40	40/40	40/60
12	4,00	20,00	40/40	40/40	40/40	40/40
13	1,78	13,33	40/40	40/40	40/40	40/40
14	0,44	6,67	40/40	40/40	40/40	40/40
15	0,00	0,00	40/40	40/40	40/40	40/40

Note that the columns 1.2, 1.3 and 1.4 on the sides cannot be smaller than 40/60 because the beams are attached to them.

4 Diagonal

The diagonals are part of the frames and thus responsible for the transfer of the wind loads.

system



angle of the diagonals

$$\alpha = \arctan\left(\frac{4 \cdot 3{,}195}{9{,}275}\right) = 54°$$

- h = 3,195 m is the height of one storey

loads

bending moment at the support

$$M = 18547 \text{ kNm}$$

- calculation of the wind loads → see above

simplified constant wind pressure

$$w_{eq} = 2 \cdot 18547/50{,}92^2 = 14{,}3 \text{ kN/m}$$

- for simplicity, the wind pressure is converted into an equivalent constant load

bending moment at the top end of the diagonal

$$M = \frac{1}{2} \cdot 14{,}3 \cdot \left(\frac{3}{4} \cdot 50{,}92\right)^2 = 10428 \text{ kNm}$$

internal forces

normal force in the column at the bottom end of the diagonal (→ see above)

$$N \approx \frac{18547/2}{9{,}275} = 1000 \text{ kN}$$

normal force in the column at the top end of the diagonal

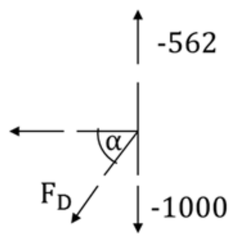
$$N \approx \frac{10428/2}{9{,}275} = 562 \text{ kN}$$

- to calculate the force in the diagonal, the normal forces in the columns at its bottom end (at the support) and at its top end are estimated

a) Frame Construction

force in the diagonal

$$F_D = \frac{1000 - 562}{\cos 54} = 745 \text{ kN}$$



$$A_{req} = \frac{745}{0{,}7 \cdot 0{,}5} = 2129 \text{ cm}^2$$

- factor 0,7 to account for buckling

selected dimensions

$$b/h = 48/48$$

The simple two-dimensional framework analysis leads to more accurate results (cf. above).

(...)

final results

$F_D = 512{,}2$ kN

$A_{req} = 1463$ cm^2

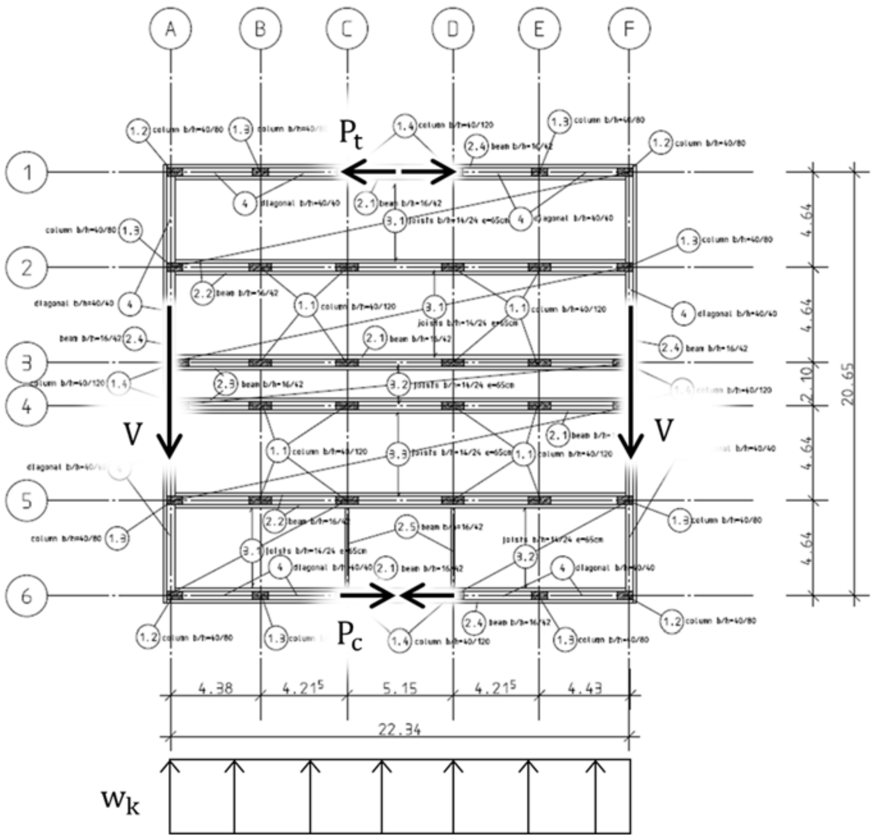
$$\boxed{b/h = 40/40}$$

5 Structural Sheathing of the Floors

The sheathing of the floors has two main functions:

- provide shear stiffness for the floor diaphragm (shear in the vertical sections, i. e. perpendicular to the surface of the sheathing)
- carry the live load from the floor, spanning between the floor joists

system



loads

- → cf. diaphragm

wind load (wind from the front)

$w_k = 5{,}25 \text{ kN/m}$

live load

$q_k = 2{,}0 \text{ kN/m}^2$

internal forces

shear

$$V = \frac{5{,}25 \cdot 22{,}34}{2} = 58{,}6 \text{ kN}$$

$$l = 20{,}65 - 6 \cdot 0{,}4 = 18{,}25 \text{ m}$$

- the shear force V is distributed over the length of the floor (columns must be subtracted)

$$v = \frac{58{,}6}{18{,}25} = 3{,}21 \text{ kN/m}$$

$$\text{req } t = \frac{3{,}21/100}{0{,}15} = 0{,}21 \text{ cm}$$

- preliminary design shear strength $\sigma_{p,v} = 1{,}5 \text{ N/mm}^2$

bending

$l = 0{,}65\ \mathrm{m}$

$$M < \frac{2 \cdot 0{,}65^2}{8} = 0{,}106\ \mathrm{kNm/m}$$

$$\mathrm{req}\ W = \frac{0{,}106 \cdot 100}{0{,}8} = 13{,}3\ \mathrm{cm^3/m}$$

$$\mathrm{req}\ t = \sqrt{\frac{13{,}3 \cdot 6}{100}} = 0{,}90\ \mathrm{cm}$$

-
- the span of the sheathing equals the spacing of the secondary beams
- preliminary design bending strength $\sigma_{p,v} = 8 \frac{N}{mm^2}$

deformation

limitation: $\delta \leq l/250$

$$\delta \leq \frac{2/100 \cdot 65^4}{76{,}8 \cdot 400 \cdot I} \leq 65/250$$

$$\mathrm{req}\ I = 44{,}7\ \mathrm{cm^4/m}$$

$$\mathrm{req}\ t = \sqrt[3]{\frac{44{,}7 \cdot 12}{100}} = 1{,}75\ \mathrm{cm}$$

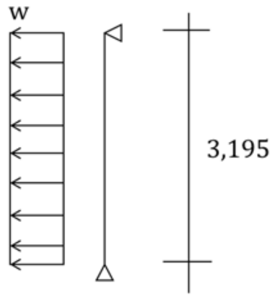
selected dimension

$$\boxed{t = 18\ \mathrm{mm}}$$

- in direct comparison, the bending is clearly decisive and the shear almost neglectable

6 Vertical Façade Carriers

system



$h = 3{,}195$ m

carriers spacing

$e = 60$ cm

loads

$q_p = 1{,}044\ \mathrm{kN/m^2}$

$c_{pe}(A) = -1{,}2$

$w = -1{,}2 \cdot 1{,}044 = -1{,}25\ \mathrm{kN/m^2}$

loads on one carrier

$w_k = 0{,}6 \cdot 1{,}25 = 2{,}0\ \mathrm{kN/m}$

internal forces

$$\max M = \frac{2{,}0 \cdot 3{,}195^2}{8} = 2{,}55\ \mathrm{kNm}$$

$\mathrm{req}\ W_y = 255/0{,}5 = 510\ \mathrm{cm^3}$

$$\mathrm{req}\ b = \frac{510 \cdot 6}{16^2} = 12{,}0\ \mathrm{cm}$$

selected dimensions

$$\boxed{\begin{array}{c} b/h = 12/16 \\ e = 60\ \mathrm{cm} \\ W_y = 512\ \mathrm{cm^3} \end{array}}$$

- the height of the carrier equals the thickness of the façade, which again equals the breadth of the outer primary beams (cf. e. g. 2.4), i. e. 16 cm

Connections Overview

This is an overview of all connections. Some connections are shown in detail in the next sections (marked in bold font), the other connections follow the same principles.

connection	b/h connection	loads	type of fastener	capacity per fastener *	number of fasteners
beam/column					
2.2/1.1 middle	120/42	$2 \cdot 48{,}8 = 97{,}6$ kN	M20 bolt	13,2 kN	11 M20
2.2/1.3 side	40/42	$2 \cdot 17{,}5 = 35{,}0$ kN	M20 bolt	13,2 kN	4 M20
2.1+2.3/1.4 side	80/42	$17{,}5 + 7{,}9 = 25{,}4$ kN	M20 bolt	13,2 kN	2 M20
2.1+2.4/1.2 front	40/42	$18{,}5 + 6{,}49 = 25$ kN	M20 bolt	13,2 kN	2 M20
2.1+2.4/1.4 front	120/42	$44{,}5 + 17{,}2 = 61{,}7$ kN	M20 bolt	13,2 kN	5 M20
2.1+2.4/1.3 front	80/42	$48{,}8 + 17{,}3 = 66{,}1$ kN	M20 bolt	13,2 kN	5 M20
2.4/1.2 side	40/42	6,59 kN (v) 8,55 kN (h)	Ø8 screw	3,2 kN	3 up + 2 down
2.4/1.3 side	40/42	20,1 kN (v) 21,5 kN (h)	Ø8 screw	3,2 kN	9 up + 5 down
2.4/1.4 side	40/42	11,3 kN (v) 9,94 kN (h)	Ø8 screw	3,2 kN	5 up + 3 down
diagonal/column					
4/1.2	-	530,9 kN	M20 dowel	17,6 kN	2 x 10 M20 + 3 steel plates
4/1.4	-	454,0 kN	M20 dowel	17,6 kN	2 x 10 M20 + 3 steel plates
sheathing/joists					
5/3	-	0,19 kN/m	Ø3 nails	0,32 kN	1 nail/m
joists/beam					
3/2	-	3,92 kN	Ø3,4 nails	0,4 kN	2 x 5 nails + steel angle
*) per 2 shear planes for bolts and dowels					

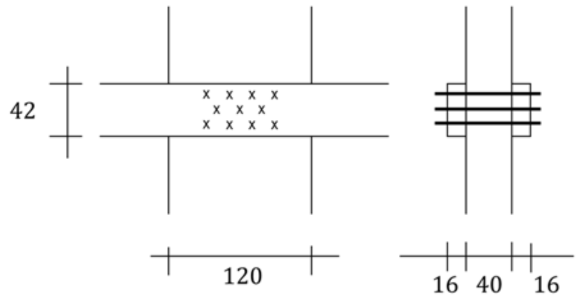
Connection Beam 2.2 to Column 1.1

For the connections between the beams and the columns, M20 bolts are used.

system

available space

$A = 120 \cdot 42 = 5040 \text{ cm}^2$



required space per bolt M20

$A_{req} = 30 \cdot d_b^2 = 30 \cdot 2{,}0^2 = 120 \text{ cm}^2$

$\max n = \frac{5040}{120} = 42$

loads

$F = 2 \cdot 48{,}8 = 97{,}6 \text{ kN}$

- 2 x reaction force B @ 2.2

check

capacity per bolt (double shear)

$F_b = 4{,}4 \cdot d_b^2 = 4{,}4 \cdot 2{,}0^2 = 17{,}6 \text{ kN}$

capacity at 90° to the grain

$F_b' = \left(1 - \frac{90}{360}\right) \cdot 17{,}6 = 13{,}2 \text{ kN}$

$\text{req } n = \frac{97{,}6}{13{,}2} = 7{,}4$

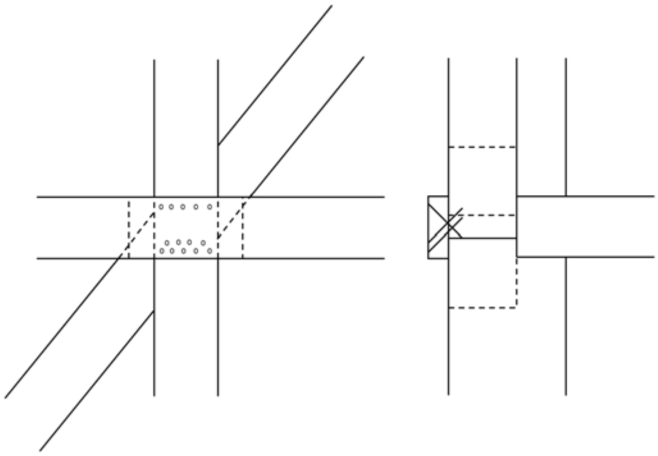
selected connectors

$\boxed{\text{11 M20}}$

Connection Outer Beam 2.4 on the Side to Column 1.3

For the connections of the outer beams on the side to the columns, fully threaded screws Ø8 mm are used. The screws carry loads in axial direction. Both the vertical loads from the self-weight of the façade and the horizontal loads from the wind suction must be considered. The screws that go at 45° upwards carry the vertical loads, while an equal amount of upward and downward screws carry the horizontal loads.

system



available space

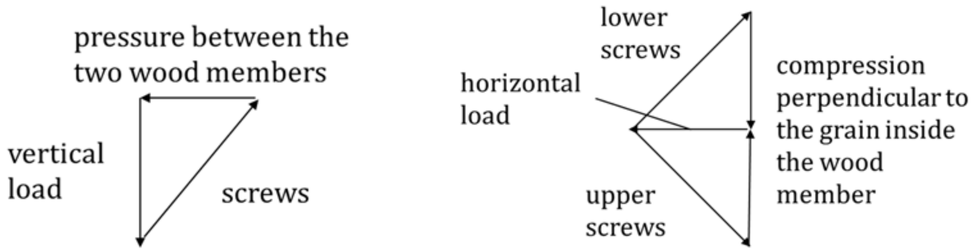
$$A = 40 \cdot 42 = 1680\ \text{cm}^2$$

required space per screw

$$A_{req} = 60 \cdot d_s^2 = 60 \cdot 0{,}8^2$$
$$= 38{,}4\ \text{cm}^2$$

$$\max n = \frac{1680}{38{,}4} = 43$$

principle of the force distribution in the connection



loads

self-weight of the façade (vertical)

- reaction force B @ 2.4 at the side for self-weight of the façade

$$F_v = 20{,}1\ \text{kN}$$

in the direction of the screw

$$F_v = \frac{20{,}1}{\sin 45} = 28{,}4\ \text{kN}$$

wind loads (horizontal)

- reaction force B @ 2.4 at the side for wind from the front

$$F_h = 21{,}5\ \text{kN}$$

in the direction of the screw

$$F_h = \frac{21{,}5}{\sin 45} = 30{,}4\ \text{kN}$$

check

a) Frame Construction

capacity per screw

$F_s = 5 \cdot d_s^2 = 5 \cdot 0{,}8^2 = 3{,}2\ \mathrm{kN}$

$\mathrm{req}\ n_v = \frac{28{,}4}{3{,}2} = 9$

$\mathrm{req}\ n_h = \frac{30{,}4}{3{,}2} = 10 = 5 + 5$

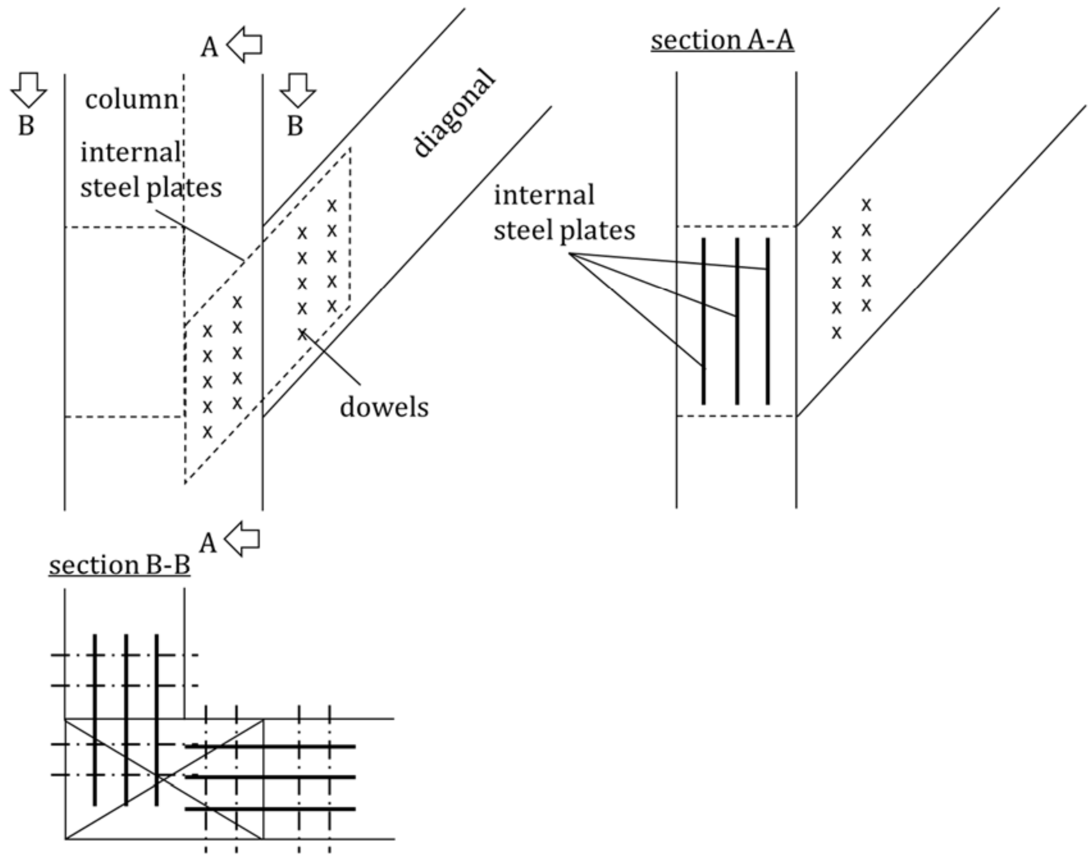
selected connectors

$\boxed{9\ \text{up} + 5\ \text{down}\ \varnothing 8}$

Connection Diagonal 4 to Column 1.2

The diagonals are in the same layer as the columns and are connected to them directly via steel plates inside the wood. As fasteners, M20 steel dowels are used.

system



required space per dowel

$A_{req} = 30 \cdot d_b^2 = 30 \cdot 2{,}0^2 = 120\ cm^2$

loads

max tensions force in the diagonal

$F = 530{,}9\ kN$

check

capacity per dowel (2 shear planes)

$F_d = 4{,}4 \cdot 2^2 = 17{,}6\ kN$

capacity for 6 shear planes

$F_d = 3 \cdot 17{,}6 = 52{,}8\ kN$

- 3 steel plates are used inside the wood, giving 6 shear planes in total
- the width of the single wood layers is $b = 40/4 = 10$ cm and should not be smaller than min $b = 5 \cdot d_b = 5 \cdot 2 = 10$ cm

capacity at an angle

- the angle of the diagonals is $\alpha = 54°$ (cf. 4 diagonal)

a) Frame Construction

$$F'_d = \left(1 - \frac{36}{360}\right) \cdot 52{,}8 = 47{,}5 \text{ kN}$$

25 % higher capacity with steel plates

$$F_d = 1{,}25 \cdot 47{,}5 = 59{,}4 \text{ kN}$$

$$\text{req n} = \frac{530{,}9}{59{,}4} = 9 \rightarrow 10$$

- with steel plates the capacity is approximately 25 % higher

$$\boxed{\text{sel n} = 10}$$

$$\text{req A} = 10 \cdot 30 \cdot 2^2 = 1200 \text{ cm}^2$$

check of the steel plates:

- preliminary design strength of steel $\sigma_p = 14 \text{ kN/cm}^2$

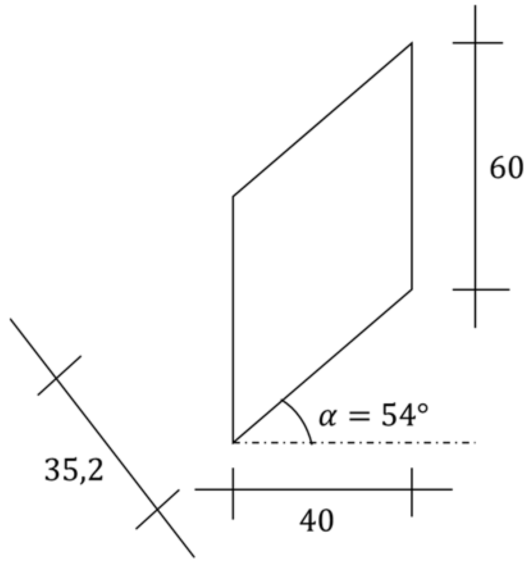
$$F = \frac{530{,}9}{3} = 177 \text{ kN}$$

$$b_{eff} = 35{,}2 - 3 \cdot 2 = 29 \text{ cm}$$

$$\text{req t} = 177/29/14 = 0{,}44 \text{ cm}$$

selected thickness of the steel plate

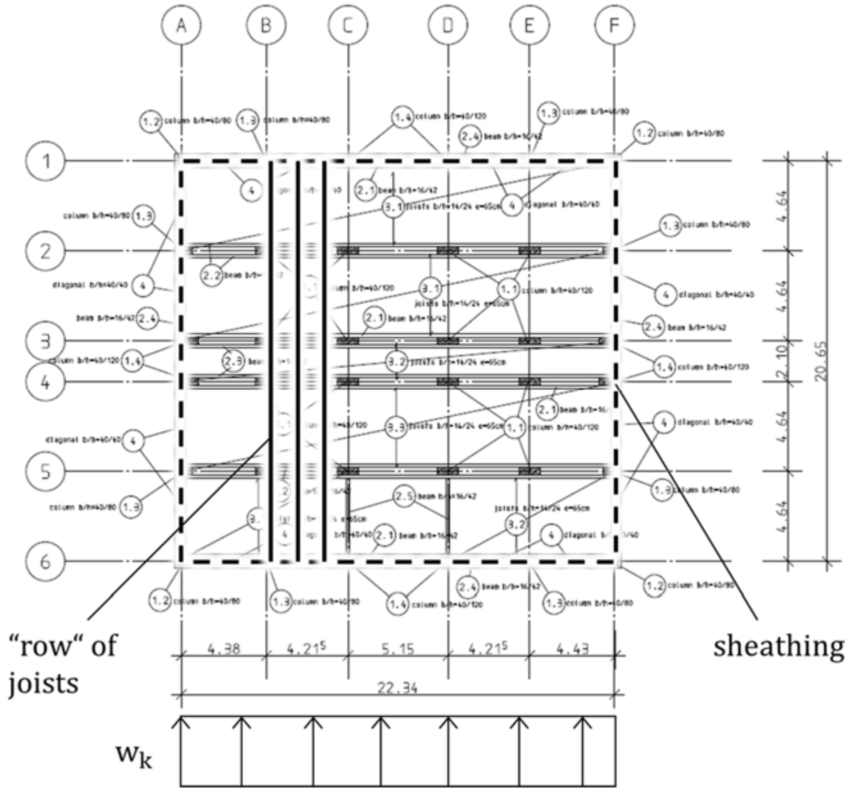
$$\boxed{t = 6 \text{ mm}}$$



Connection Sheathing 5 to Joists 3

The sheathing is connected to the joists with nails.

system



loads

wind load (wind from the front)

$w_k = 5{,}25$ kN/m

- cf. diaphragm 7
- wind from the sides is not decisive

internal forces

force in one "row" of joists

$F = 5{,}25 \cdot 0{,}65 \cdot 1{,}15 = 3{,}92$ kN

$$v = \frac{3{,}92}{20{,}65} = 0{,}19 \text{ kN/m}$$

capacity per nail Ø3,0 mm

$F_n = 3{,}5 \cdot 0{,}3^2 = 0{,}32$ kN

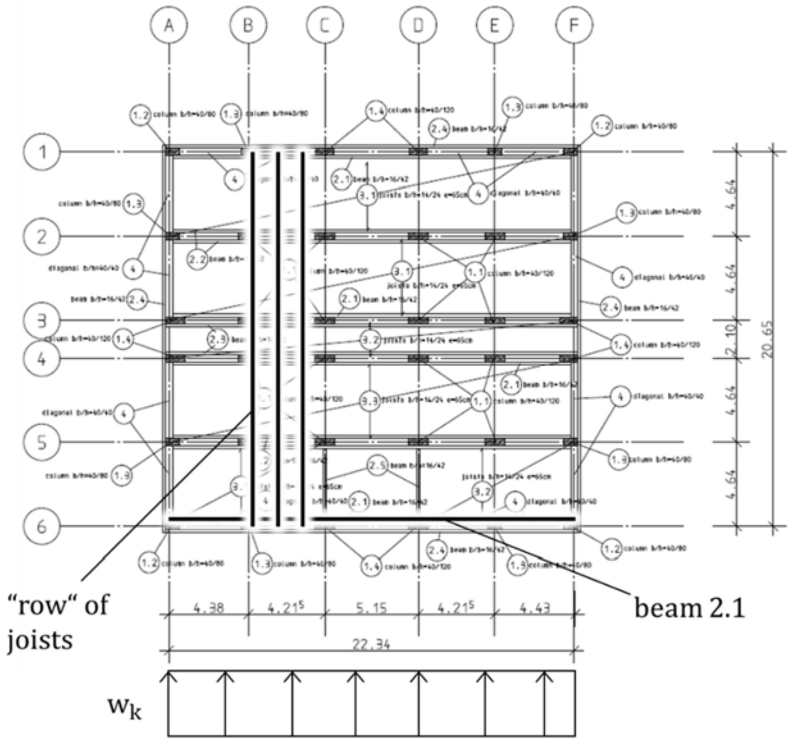
$$\boxed{\text{req n} = \frac{0{,}19}{0{,}32} = 1 \text{ /m}}$$

- spacing of the secondary beams
  e = 65 cm
- factor 1,15 to account for continuity effects

Connection Joists 3 to Beams 2

The joists are connected to the beams with steel angles and nails.

system



loads

wind load (wind from the front)

$w_k = 5{,}25 \text{ kN/m}$

- cf. diaphragm 7
- wind from the sides is not decisive

internal forces

force in one “row” of joists

$F = 5{,}25 \cdot 0{,}65 \cdot 1{,}15 = 3{,}92 \text{ kN}$

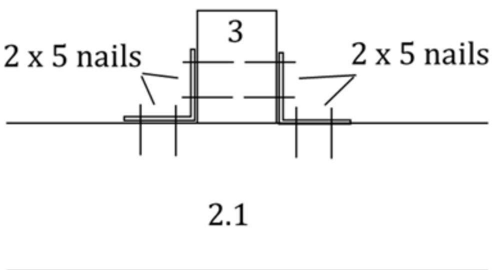
- spacing of the secondary beams e = 65 cm
- continuity factor 1,15

This force must be transferred from the outer beam 2.1 to each joist 3.

capacity per nail Ø3,4 mm

$F_n = 3{,}5 \cdot 0{,}34^2 = 0{,}4 \text{ kN}$

$$\boxed{\text{req } n = \frac{3{,}92}{0{,}4} = 10 = 5 + 5}$$



This applies to all connections of all secondary beams 3 with the outer primary beam 2.1.

At all inner connection, with the beams 2.2 and 2.3, no load transfer is necessary, only a reduced amount of nails is selected to avoid large deformation differences between the different beams.

$$\boxed{n = 6 = 3 + 3}$$

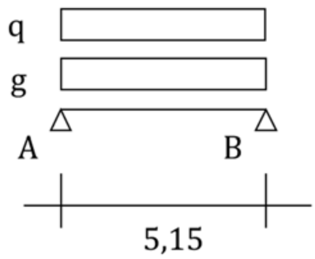
b) Panel Construction

Element Reference Overview

reference no	element	page
1	floor joists	
1.1	floor joists modules 3, 4	45
1.2	floor joists modules 1, 2, 6, 7	46
1.3	floor joists modules 5	47
2	floor beam	59
3	walls/studs	48
4	wall sheathing	57
5	columns	62
6	beam	60
	connections overview	63

1.1 Floor Joists

system



$l = 5{,}15\ \text{m}$

$h \approx \frac{l}{20} = \frac{515}{20} = 25{,}8\ \text{cm}$

loads

$q_k = 2{,}0\ \text{kN/m}^2$

$g_k = 2{,}0\ \text{kN/m}^2$

loads on one joist

$q_k = 2{,}0 \cdot 0{,}6 = 1{,}2\ \text{kN/m}$

- selected spacing: $e = 60\ \text{cm}$

$g_k = 1{,}2\ \text{kN/m}$

internal forces

$$\max M_1 = \frac{2 \cdot 1{,}2 \cdot 5{,}15^2}{8} = 7{,}96\ \text{kNm}$$

$\text{req}\ W_y = 796/0{,}5 = 1592\ \text{cm}^3$

selected dimensions

$$\boxed{\begin{gathered} b/h = 16/24 \\ e = 60\ \text{cm} \\ I_y = 18432\ \text{cm}^4 \\ W_y = 1536\ \text{cm}^3 \end{gathered}}$$

deformation

$$\max \delta = \frac{2 \cdot 1{,}2/100 \cdot 515^4}{76{,}8 \cdot 1000 \cdot 18432} = 1{,}19\ \text{cm}$$

$< l/250 = 515/250 = 2{,}06\ \text{cm}$

max reaction forces

$A_g = 1{,}2 \cdot 5{,}15/2 = 3{,}1\ \text{kN}$

$A_q = 1{,}2 \cdot 5{,}15/2 = 3{,}1\ \text{kN}$

$A = 6{,}2\ \text{kN}$

1.2 Floor Joists

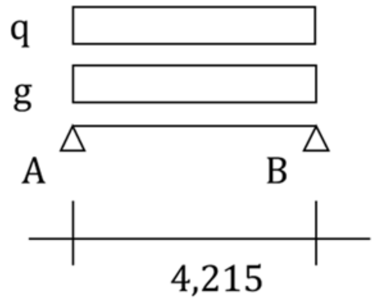
system

$l = 4{,}215$ m

loads

$q_k = 2{,}0\ kN/m^2$

$g_k = 2{,}0\ kN/m^2$



loads on one beam

$q_k = 2{,}0 \cdot 0{,}6 = 1{,}2\ kN/m$

$g_k = 1{,}2\ kN/m$

- selected spacing: e = 60 cm

selected dimensions

$$\boxed{\begin{array}{c} b/h = 16/24 \\ e = 60\ cm \\ I_y = 18432\ cm^4 \\ W_y = 1536\ cm^3 \end{array}}$$

- joists 1.2 are not decisive, the same dimensions are selected as for joist 1.1

max reaction forces

$A_g = 1{,}2 \cdot 4{,}215/2 = 2{,}5\ kN$

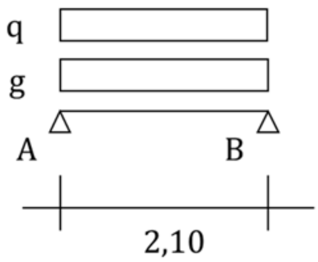
$A_q = 1{,}2 \cdot 4{,}215/2 = 2{,}5\ kN$

$A = 5{,}0\ kN$

b) Panel Construction

1.3 Floor Joists

system



$l = 2{,}1$ m

loads

$q_k = 2{,}0$ kN/m^2

$g_k = 2{,}0$ kN/m^2

loads on one beam

$q_k = 2{,}0 \cdot 0{,}6 = 1{,}2$ kN/m

$g_k = 1{,}2$ kN/m

- selected spacing: e = 60 cm

selected dimensions

b/h = 16/24
e = 60 cm
$I_y = 18432$ cm^4
$W_y = 1536$ cm^3

- joists 1.3 are not decisive, the same dimensions are selected as for joist 1.1

max reaction forces

$A_q = 1{,}2 \cdot 2{,}1/2 = 1{,}3$ kN

$A_q = 1{,}2 \cdot 2{,}1/2 = 1{,}3$ kN

$A = 2{,}6$ kN

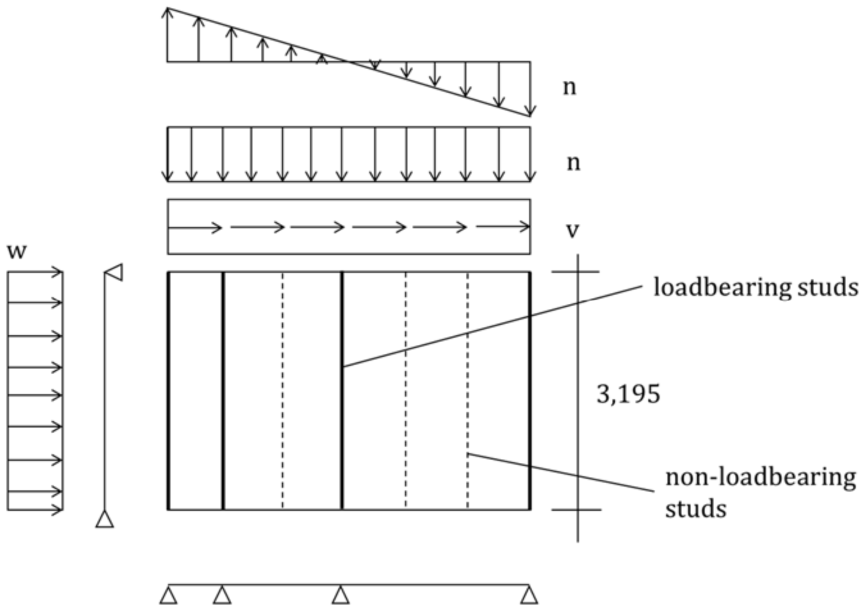
3 Walls/Studs

The full stability analysis was performed using Excel (see appendix). Here, only the walls Wy1 and Wy8 are shown. Wy8 has the highest vertical load, while Wy1 has the highest horizontal wind load.

Based on the loads on the walls, the loads on the individual studs were calculated as the reaction forces of a continuous beam (see system sketch below).

system

$h = 3{,}195$ m



loads

wind loads (wind from the front)

$W_y = 1319$

$M_x = -37094$ kNm

wind loads (wind from the side, wind area D + E)

total horizontal wind force

$W_x = 1275$ kN

bending moment of the cantilever beam

$M_y = 34982$ kNm

internal forces

wall Wy8 for wind from the front

shear force

$v = 20{,}39$ kN/m

normal force (compression)

$n = 394{,}3$ kN/m

horizontal wind load

$w = 0$

- for detailed calculations, see Excel analysis

b) Panel Construction

wall Wy1 for wind from the back

shear force

$v = 6{,}22$ kN/m

normal force (compression)

$n = 237{,}5$ kN/m

horizontal wind load

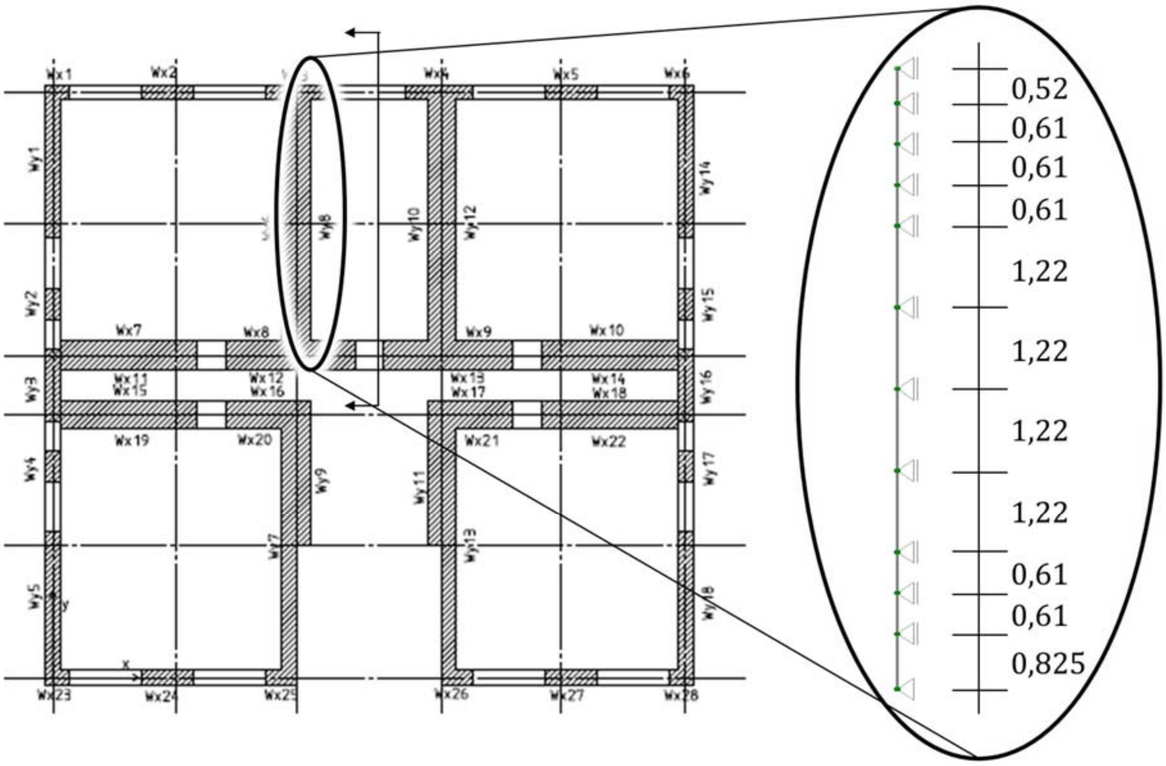
$w = 0{,}98$ kN/m

In the end of the preliminary design, two different thicknesses of the walls were determined: 20 cm (with the loadbearing studs being b/h = 20/20 cm) and 28 cm (with the loadbearing studs being b/h = 28/28 cm). Because of the decreasing loads with the height of the building, the cross-sections of the studs could be adapted in the higher storeys. Assuming the loads due to the self-weight and the life load (N) to decrease linearly and the loads due to the wind (M) to decrease quadratically, the following cross-sections are calculated:

storey no	M	N	b/h	b/h
-	[%]	[%]	[cm/cm]	[cm/cm]
0	100,00	100,00	20/20	28/28
1	87,11	93,33	20/20	28/28
2	75,11	86,67	20/20	28/28
3	64,00	80,00	20/20	28/28
4	53,78	73,33	20/20	28/28
5	44,44	66,67	16/16	24/24
6	36,00	60,00	16/16	24/24
7	28,44	53,33	16/16	24/24
8	21,78	46,67	16/16	24/24
9	16,00	40,00	16/16	24/24
10	11,11	33,33	16/8	20/20
11	7,11	26,67	16/8	20/20
12	4,00	20,00	16/8	20/20
13	1,78	13,33	16/8	20/20
14	0,44	6,67	16/8	20/20
15	0,00	0,00	16/8	20/20

3.y8 Studs in Wall y3

system



loads

self-weight of the wall

$g_k = 1{,}0 \text{ kN/m}^2$

over 14 storeys

$g_k = 1 \cdot 14 \cdot 3{,}195 = 44{,}7 \text{ kN/m}$

floor load

$$p_k = \frac{3{,}2}{0{,}6} \cdot 15 + \frac{3{,}2}{0{,}6} \cdot (1 + 14 \cdot 0{,}74)$$
$$= 136 \text{ kN/m}$$

- reaction force A @ 1.1
- the load per storey is added up for the roof and the 14 storeys with a reduction factor $\alpha_n = 0{,}74$ for the live load

total vertical load (constant)

$n = -44{,}7 - 136 = -181 \text{ kN/m}$

b) Panel Construction

wind loads (linear)

- cf. Excel analysis

wind from the front

$\max n = 178 \text{ kN/m}$

$\min n = -213 \text{ kN/m}$

wind from the back

$\max n = 213 \text{ kN/m}$

$\min n = -178 \text{ kN/m}$

wind from the left side

$\max n = \min n = 9{,}2 \text{ kN/m}$

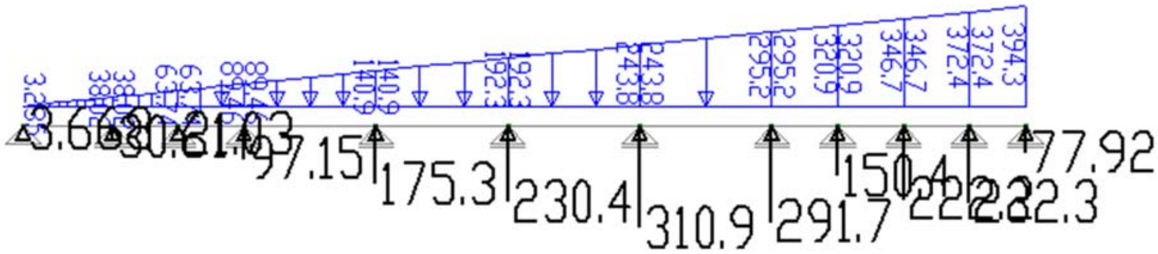
wind from the right side

$\max n = \min n = -9{,}2 \text{ kN/m}$

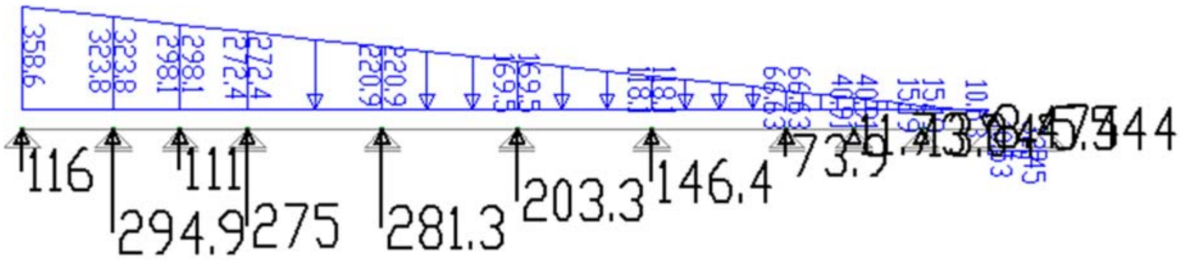
b) Panel Construction

internal forces

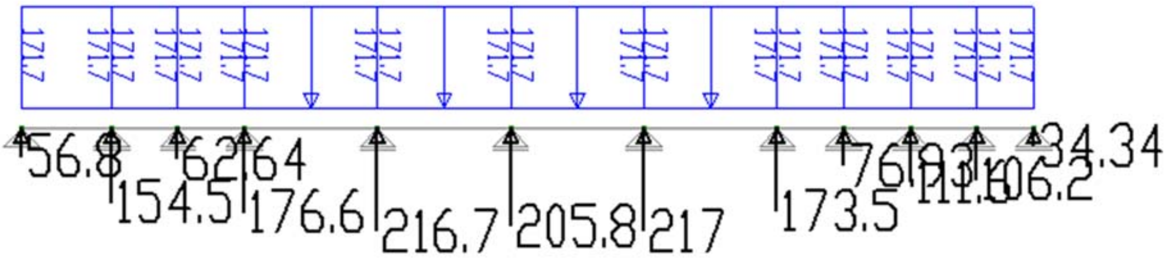
wind from the front



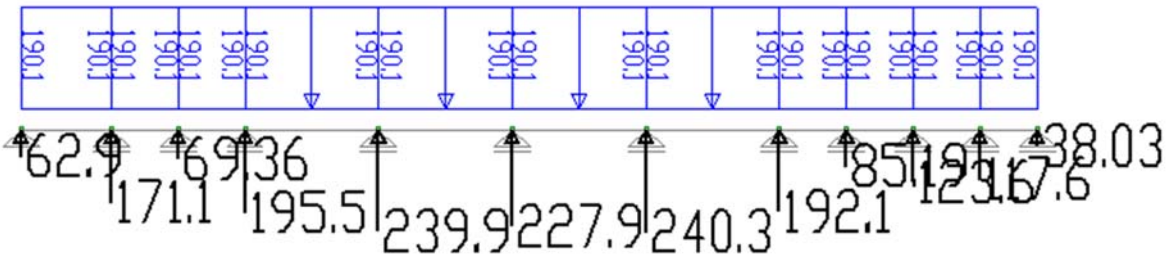
wind from the back



wind from the left side



wind from the right side



For each loadbearing studs (each support), the required cross-section is calculated using the preliminary design strength of timber of 5 N/mm^2 with a factor of 0,8 to account for buckling. Since the sheathing prevents the studs from buckling in one direction and also makes buckling in the other direction more difficult by effectively connecting the individual studs, a higher buckling factor is chosen.

selected dimensions

b/t = 28/28
A = 784 cm^2

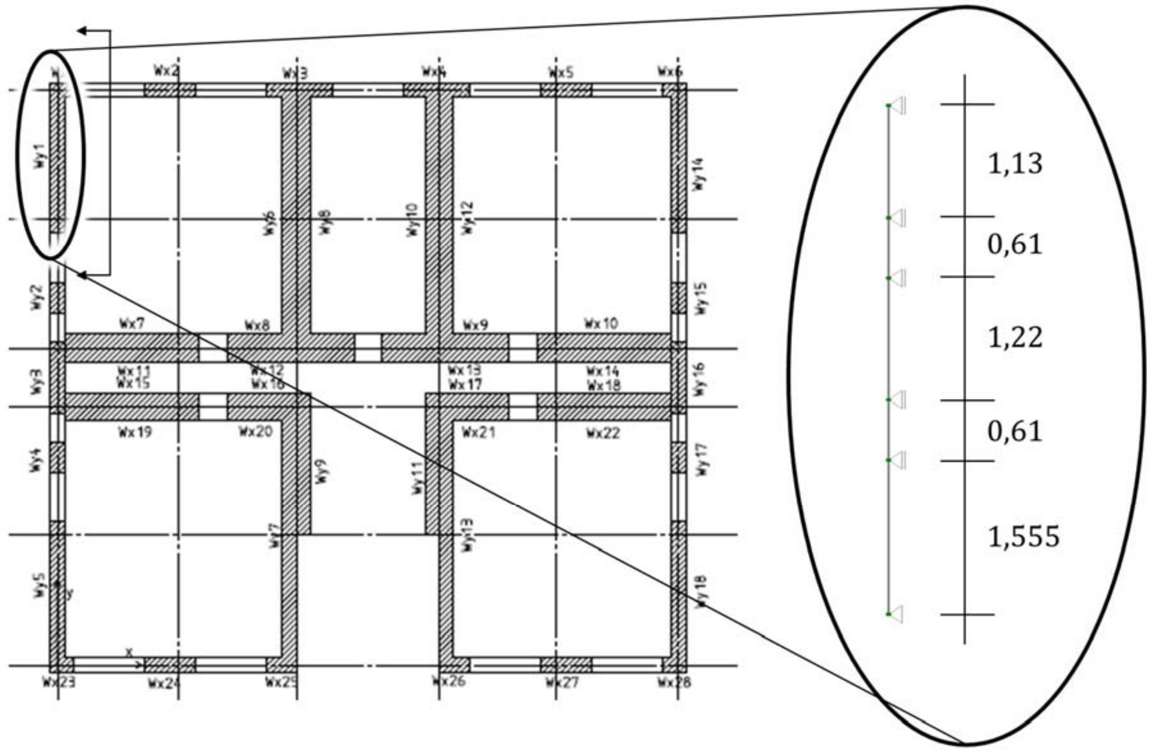
b) Panel Construction

stud no	1	2	3	4	5	6	7	8	9	10	11	12
max force [kN]	116	295	111	275	281	230	311	292	150	222	232	78
req b [cm]	10	26	10	25	25	21	28	26	13	20	21	7
selected b [cm]	28	28	28	28	28	28	28	28	28	28	28	28

b) Panel Construction

3.y1 Studs in Wall y1

system



loads

self-weight of the wall

$g_k = 1{,}0\ \mathrm{kN/m^2}$

over 14 storeys

$g_k = 1 \cdot 14 \cdot 3{,}195 = 44{,}7\ \mathrm{kN/m}$

floor load

$$p_k = \frac{2{,}5}{0{,}6} \cdot 15 + \frac{2{,}5}{0{,}6} \cdot (1 + 14 \cdot 0{,}74)$$
$$= 109{,}8\ \mathrm{kN/m}$$

- reaction force A @ 1.2
- the load per storey is added up for the roof and the 14 storeys with a reduction factor $\alpha_n = 0{,}74$ for the live load

total vertical load (constant)

$n = 44{,}7 + 109{,}8 = 154{,}5\ \mathrm{kN/m}$

wind loads (linear)

- cf. Excel analysis

wind from the front

$\max n = 83\ \mathrm{kN/m}$

$\min n = -133\ \mathrm{kN/m}$

wind from the back

$\max n = 133\ \mathrm{kN/m}$

$\min n = -83\ \mathrm{kN/m}$

b) Panel Construction

wind from the left side

$\max n = \min n = 40 \text{ kN/m}$

wind from the right side

$\max n = \min n = -40 \text{ kN/m}$

horizontal wind pressure (wind from the back)

$c_{pe}(B) = -1{,}2$

$q_p = 1{,}044 \text{ kN/m}^2$

$|w| = 1{,}2 \cdot 1{,}044 = 1{,}25 \text{ kN/m}^2$

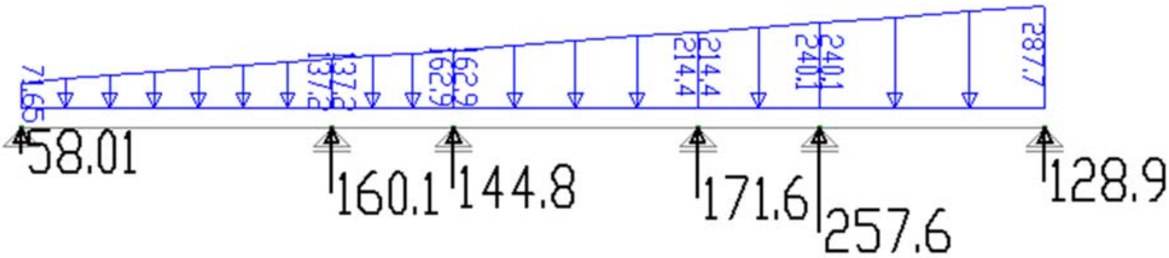
wind pressure per stud

$|w| = 1{,}15 \cdot 1{,}25 \cdot 0{,}61 = 0{,}88 \text{ kN/m}$

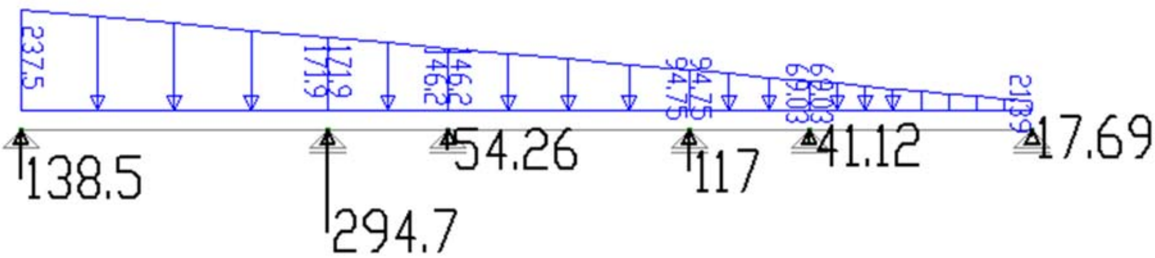
- spacing of the studs $e = 0{,}61$ cm
- continuity factor 1,15

internal forces

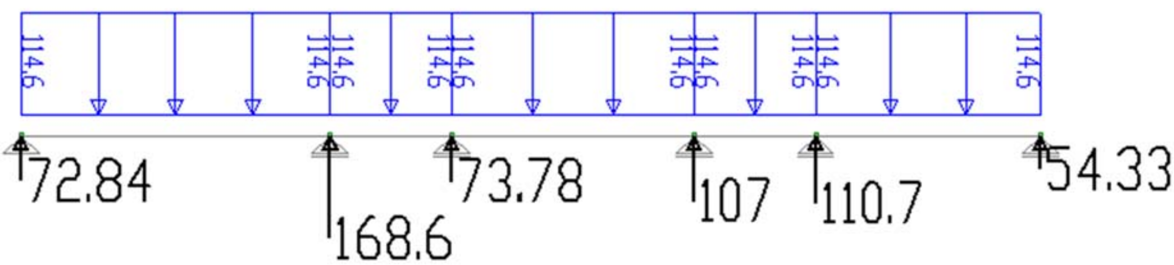
wind from the front



wind from the back

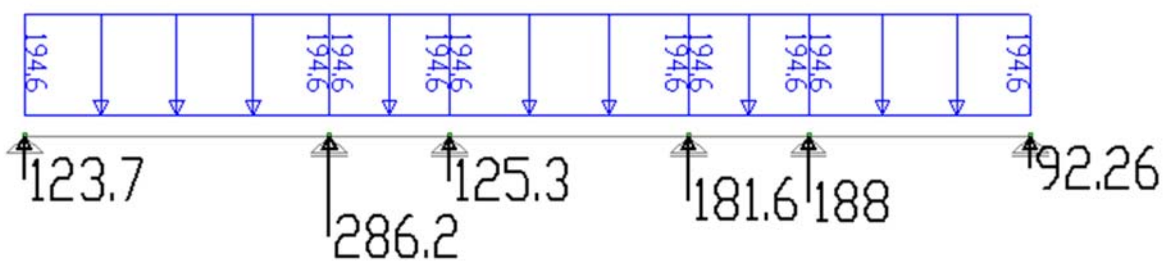


wind from the left side



wind form the right side

b) Panel Construction



bending moment from wind pressure

$M = 0{,}88 \cdot 3{,}195^2/8 \; = 1{,}12 \text{ kNm}$

selected dimensions

$b/t = 28/28$
$A = 784 \text{ cm}^2$
$W = 3659 \text{ cm}^3$

stud no	1	2	3	4	5	6
max force [kN]	139	295	145	182	258	129
req b [cm]	12	26	13	16	23	12
selected b [cm]	28	28	28	28	28	28

stress check

$$\sigma = \frac{295}{784} + \frac{112}{3659} = 0{,}38 + 0{,}03 = 0{,}41 \text{ kN/cm}^2$$

$$\approx 0{,}8 \cdot 0{,}5 = 0{,}40 \text{ kN/cm}^2$$

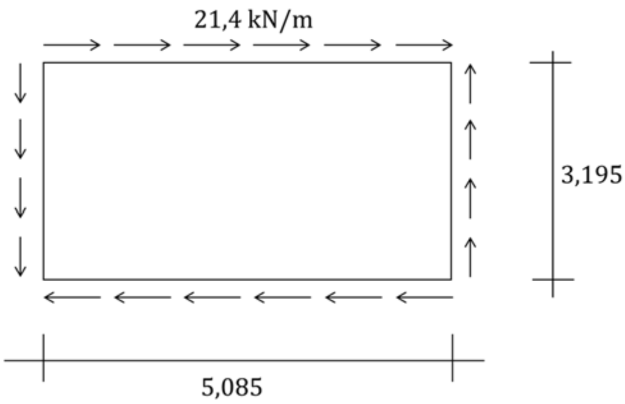
- the wind pressure has almost no effect compared to the high normal forces

4 Sheathing

The sheathing is connected to the studs and the blocking via nails.

Wall Wx15 becomes decisive for the sheathing for wind from the left side.

system



loads

- cf. Excel analysis

v = 21,4 kN/m

nails

selected: Ø5 mm

shear capacity per nail

$F_n = 3{,}5 \cdot d_n^2 = 3{,}5 \cdot 0{,}5^2 = 0{,}88$ kN

req n = 20,4/0,88 = 23,3 /m

selected nails

$\boxed{\begin{array}{c}\text{Ø5 mm } l = 100 \text{ mm} \\ e = 8 \text{ cm in 2 rows}\end{array}}$

minimum distances (α = 0°)

between nails in 1 row $a_1 = 12 \cdot 0{,}5 = 6 \text{ cm} < 8 \text{ cm}$

between the rows $a_2 = 5 \cdot 0{,}5 = 2{,}5 \text{ cm}$

to the side edges $a_4 = 5 \cdot 0{,}5 = 2{,}5 \text{ cm}$

→ required breadth of the wooden members (studs and blocking)

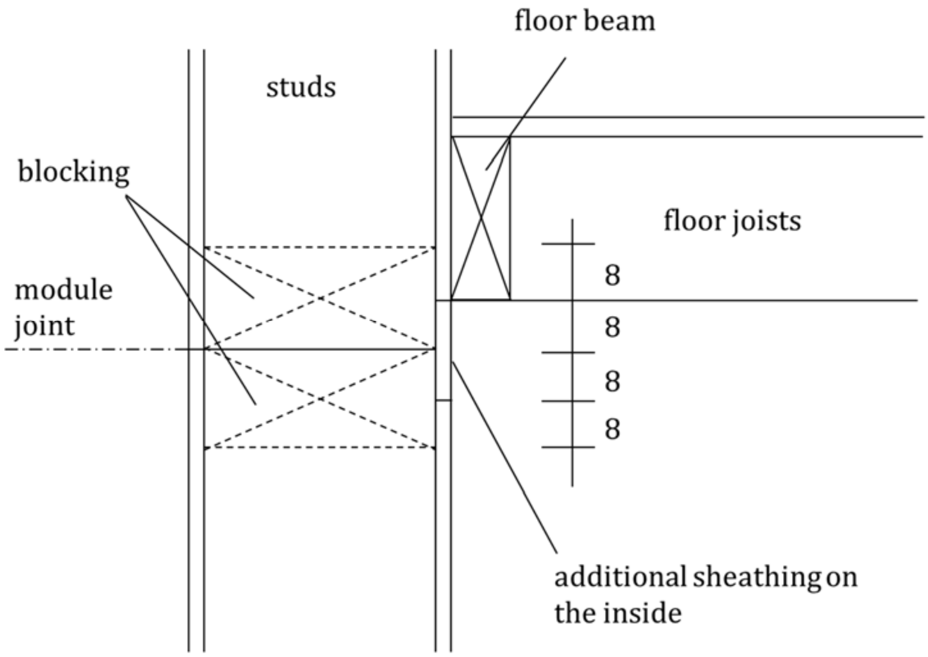
req b = 3 · 2,5 = 7,5 cm

selected blocking

$\boxed{\begin{array}{c}b = 16 \text{ cm} \\ \text{h acc. to the width of the wall}\end{array}}$

- at the seams of the sheathing panels, the blocking must be b ≥ 2 · 7,5 = 15 cm

b) Panel Construction



selected sheathing

$$\text{req t} = \frac{21{,}4/100}{0{,}15} = 1{,}43\ \text{cm}$$

$$\boxed{t = 18\ \text{mm}}$$

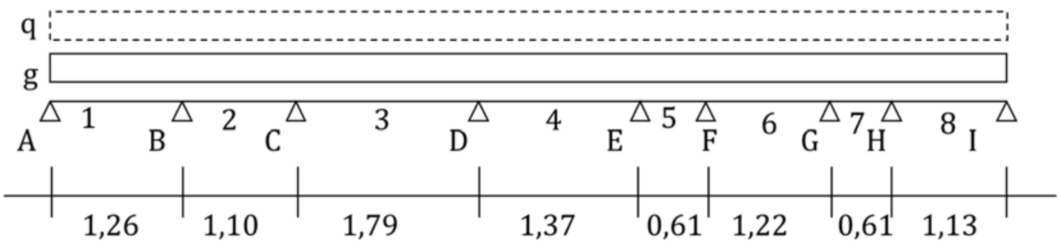
- preliminary design shear strength
  $\sigma_{p,v} = 1{,}5\ \text{N/mm}^2$

b) Panel Construction

2 Floor Beam

The floor beams are only connected to the load-bearing studs. The floor beam in axis 1–3/A becomes decisive.

system



loads

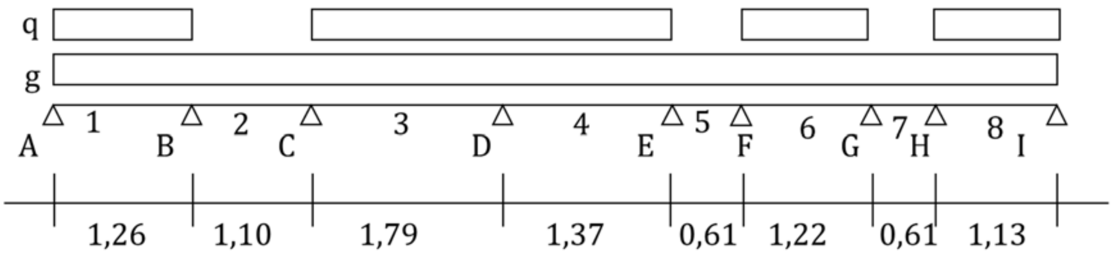
$$g_k = \frac{2,5}{0,6} = 4,2 \text{ kN/m}$$

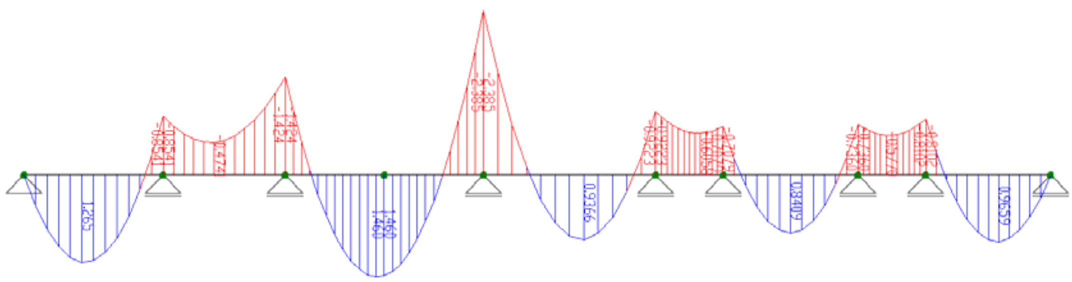
$$q_k = 4,2 \text{ kN/m}$$

- the single forces from the floor joists are converted to a constant continuous load
- reaction force A @ 1.2

internal forces

max moment at support D





$$\max M_F = -2,38 \text{ kNm}$$

$$\text{req } W_y = \frac{238}{0,5} = 476 \text{ cm}^3$$

selected dimensions

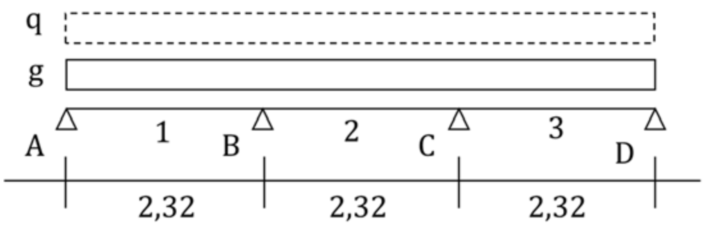
b/h = 10/24
$I_y = 11520 \text{ cm}^4$
$W_y = 960 \text{ cm}^3$
$A = 240 \text{ cm}^2$

b) Panel Construction

6 Beam

system

l = 9,275/4 = 2,32 m



loads

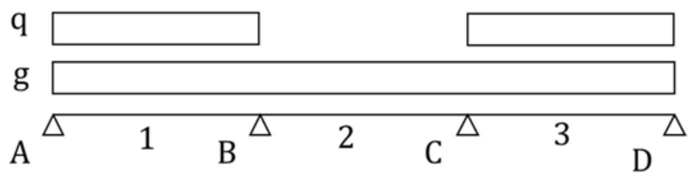
- reaction force A @ 1.2
- spacing of the floor joists = 0,6 m

$$g_k = \frac{2,5}{0,6} = 4,17 \text{ kN/m}$$

$$q_k = \frac{2,5}{0,6} = 4,17 \text{ kN/m}$$

internal forces

max moment in the first field



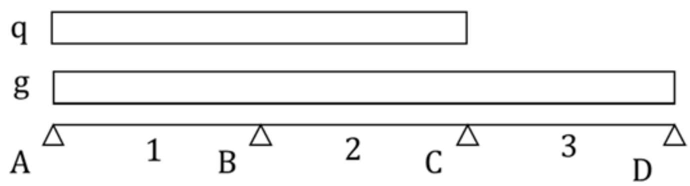
$$\max M_1 = (0,08 + 0,101) \cdot 4,17 \cdot 2,32^2 = 4,06 \text{ kNm}$$

$$\text{req } W = \frac{406}{0,5} = 812 \text{ cm}^3$$

corresponding moment at the support:

$$M_B = (-0,1 - 0,05) \cdot 4,17 \cdot 2,32^2 = -3,37 \text{ kNm}$$

max moment at the support B



$$\max M_B = (-0,10 - 0,117) \cdot 4,17 \cdot 2,32^2 = -4,87 \text{ kNm}$$

$$\text{req } W = \frac{487}{0,5} = 974 \text{ cm}^3$$

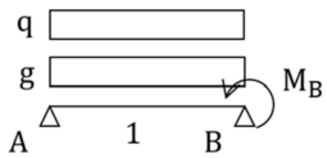
b) Panel Construction

selected dimensions

$$\boxed{\begin{array}{c} b/h = 10/24 \\ W_y = 960\ \text{cm}^3 \\ I_y = 11520\ \text{cm}^4 \end{array}}$$

deformation

substitute system:



$$\max \delta = \frac{2 \cdot 4{,}17/100 \cdot 232^4}{76{,}8 \cdot 1000 \cdot 11520}$$

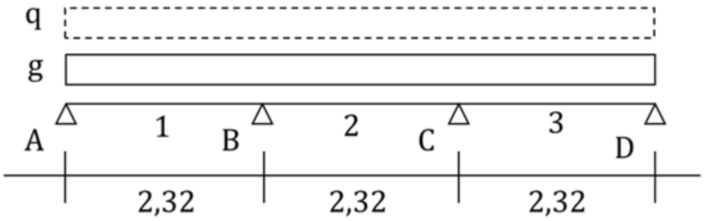
$$+ \frac{-337}{16 \cdot 1000 \cdot 11520} \cdot 232^2$$

$$= 0{,}273 - 0{,}098 = 0{,}18\ \text{cm}$$

$$< l/250 = 232/250 = 0{,}93\ \text{cm}$$

5 Columns

system



loads

$$g_k = \frac{2{,}5}{0{,}6} \cdot 15 = 62{,}5 \text{ kN/m}$$

$$q_k = \frac{2{,}5}{0{,}6} \cdot (1 + 14 \cdot 0{,}74) = 47{,}3 \text{ kN/m}$$

- reaction force A @ 1.2
- spacing of the floor joists = 0,6 m
- the loads are added up over all 15 storeys

internal forces

columns A, D

$$N = 0{,}400 \cdot 62{,}5 \cdot 2{,}32 + 0{,}45 \cdot 47{,}3 \cdot 2{,}32 = 107{,}4 \text{ kN}$$

$$\text{req } A = \frac{107{,}4}{0{,}7 \cdot 0{,}5} = 306{,}9 \text{ cm}^2$$

columns B, C

$$N = 1{,}100 \cdot 62{,}5 \cdot 2{,}32 + 1{,}2 \cdot 47{,}3 \cdot 2{,}32 = 291{,}2 \text{ kN}$$

$$\text{req } A = \frac{291{,}2}{0{,}7 \cdot 0{,}5} = 832 \text{ cm}^2$$

selected dimensions

columns A, D

$$\boxed{\begin{array}{c} 2 \times b/h = 16/28 \\ A = 2 \cdot 448 \text{ cm}^2 \end{array}}$$

columns B, C

$$\boxed{\begin{array}{c} 2 \times b/h = 28/28 \\ A = 2 \cdot 784 \text{ cm}^2 \end{array}}$$

b) Panel Construction

Connections Overview

connection	b/h overlap	loads	type of fastener	capacity per fastener	number of fasteners
anchoring					
3/3	-	134,7 kN	M20 dowels	22,0 kN *	2 × 7 M20 + steel plate
floor joists/floor beam					
1/2	16/24	6,2 kN (v)	screws Ø8	3,2 kN	3 Ø8
floor beam/studs					
2/3	24/28	26,6 kN	M20 dowels	8,0 kN **	5 M20
*) double shear **) single shear					

b) Panel Construction

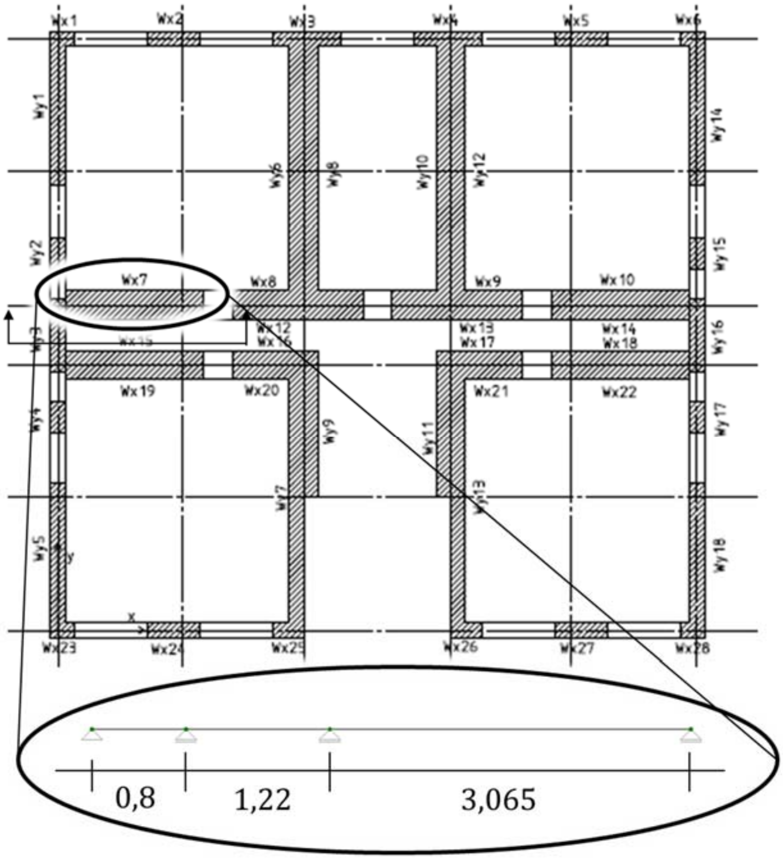
Anchoring

For tension, wall Wx7 becomes decisive for wind from the left side.

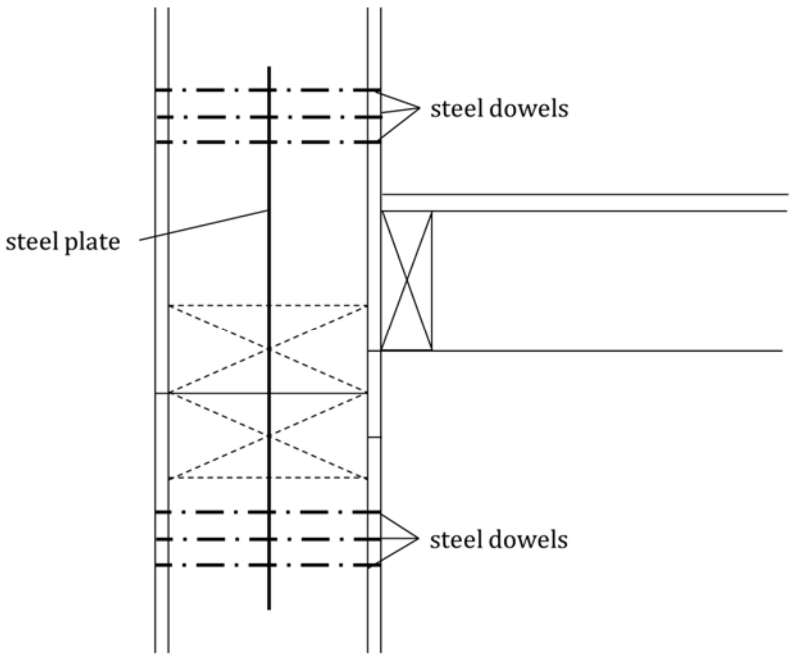
The hold-down device consists of a steel plate inside each load-bearing stud and steel dowels.

system

global system



local system



b) Panel Construction

loads

self-weight of the wall

$g_k = 1{,}0\ \mathrm{kN/m^2}$

over 14 storeys

$g_k = 1 \cdot 14 \cdot 3{,}195 = 44{,}7\ \mathrm{kN/m}$

live load

$p_k = 0$

total vertical load (constant)

$n = 0{,}5 \cdot (-44{,}7) = -22{,}4\ \mathrm{kN/m}$

- to be on the safe side, only 50 % of the self-weight and live loads are considered

wind loads (linear)

- cf. Excel analysis

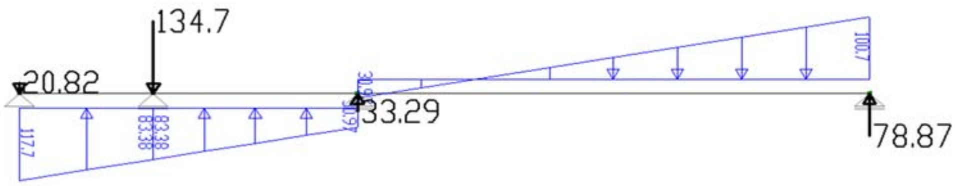
wind from the left side

$\max n = 140{,}1\ \mathrm{kN/m}$

$\min n = -78{,}3\ \mathrm{kN/m}$

internal forces

wind from the left side



$\max F_t = 134{,}7\ \mathrm{kN}$

steel dowels

capacity per dowel M20 (double shear with inner steel plate)

$F_d = 4{,}4 \cdot 2^2 = 17{,}6\ \mathrm{kN}$

25 % higher capacity with steel plates

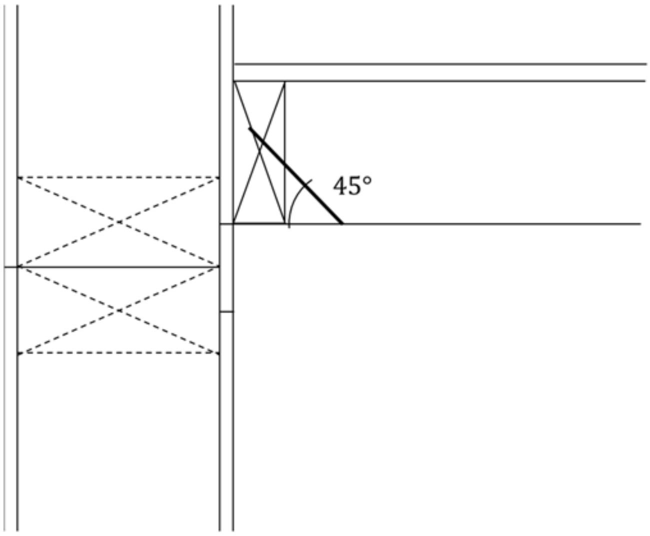
$F_d = 1{,}25 \cdot 17{,}6 = 22{,}0\ \mathrm{kN}$

$$\boxed{\text{req } n = \frac{134{,}7}{22} = 7}$$

Connection Floor Joists 1 to Floor Beam 2

This connection uses screws Ø8 mm that are subjected to tension.

system



loads

$F = 6{,}2$ kN

- reaction force A @ 1.1

internal forces

force in the screws at an angle of 45°

$$S = \frac{6{,}2}{\sin 45} = 8{,}77 \text{ kN}$$

check

capacity per screw

$F_s = 5 \cdot d_s^2 = 5 \cdot 0{,}8^2 = 3{,}2$ kN

$$\boxed{\text{req } n = \frac{8{,}77}{3{,}2} = 3}$$

required space

$$\text{req } A = 3 \cdot 60 \cdot 0{,}8^2 = 115{,}2 \text{ cm}^2$$

available space

$$A = 16 \cdot 24 = 384 \text{ cm}^2$$

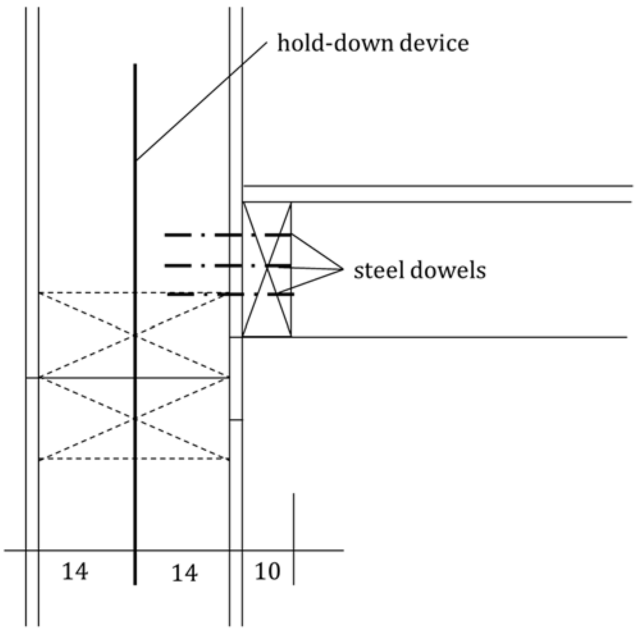
- the available space equals the cross-section of the floor joist 16/24

b) Panel Construction

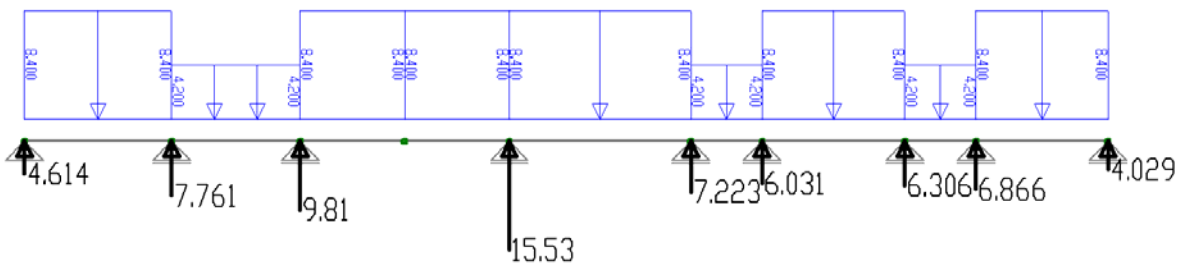
Connection Floor Beam 2 to Studs 3

For this connection, steel dowels M20 are used, that don't interfere with the hold-down device (cf. anchoring). The floor beam in axis 1–3/A is decisive.

system



loads



$F = 15{,}5 \text{ kN}$

check

capacity per dowel (single shear)

$$F_d = 2 \cdot d_d^2 = 2 \cdot 2^2 = 8 \text{ kN}$$

at 90° to the grain for the floor beam

$$F_d' = \left(1 - \frac{90}{360}\right) \cdot 8 = 6 \text{ kN}$$

$$\boxed{\text{req n} = \frac{15{,}5}{6} = 3}$$

required space

$$\text{req A} = 3 \cdot 30 \cdot 2^2 = 360 \text{ cm}^2$$

b) Panel Construction

available space

$$A = 24 \cdot 20 = 480 \text{ cm}^2$$

- floor beam: 14/24
- stud: 20/20 (in the upper storeys, the cross-section 28/28 is reduced to 20/20)

c) CLT Construction

Element Reference Overview

reference no	element	page
1	floor slab	70
2, 3	floor slab	71
4	floor slab	72
5	Walls	73
	anchoring	78

Remarks

The floor slabs are designed assuming they only span in one direction. This is on the safe side, as they actually span in both directions so that a multi-axial state of stress can be developed and loads can be better distributed.

The load transfer from the floor slabs to the walls is calculated in a simplified manner by determining a length of influence for every wall and transferring the loads from the slab within the resulting area of influence to the wall.

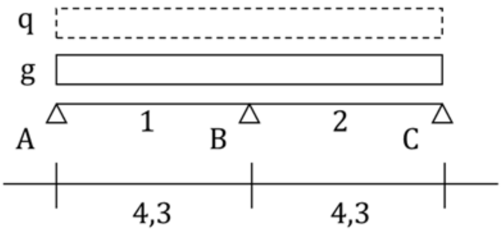
To calculate the shear forces and bending moments in the walls from the wind loads, a stability analysis was performed using Excel. Only selected walls are presented in this report.

1 Floor Slab

It could be seen from the sheathing of the diaphragm in the frame construction that shear from horizontal wind forces can be neglected for the preliminary design.

system

- the middle support is the beam 6



$$l = \frac{8{,}595}{2} = 4{,}30 \text{ m}$$

loads

$g_k = 2{,}0 \text{ kN/m}^2$

$q_k = 2{,}0 \text{ kN/m}^2$

selected dimensions

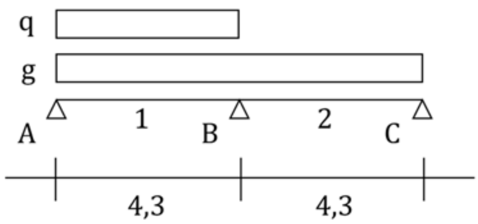
$$\boxed{\begin{aligned} t &= 145 \text{ mm} \\ &= 19 + 44 + 19 + 44 + 19 \end{aligned}}$$

max reaction forces

$A_g = 0{,}375 \cdot 2{,}0 \cdot 4{,}30 = 3{,}2 \text{ kN/m}$

$A_q = 0{,}438 \cdot 2{,}0 \cdot 4{,}30 = 3{,}8 \text{ kN/m}$

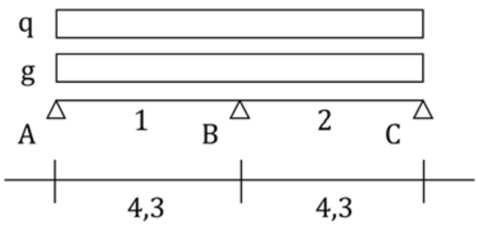
$A = 7{,}0 \text{ kN/m}$



$B_g = 1{,}25 \cdot 2{,}0 \cdot 4{,}3 = 10{,}75 \text{ kN/m}$

$B_q = 1{,}25 \cdot 2{,}0 \cdot 4{,}3 = 10{,}75 \text{ kN/m}$

$B = 21{,}5 \text{ kN/m}$



pressure perpendicular to the grain

- preliminary design compression strength perpendicular to the grain $\sigma_{p,c} = 1{,}5 \text{ N/mm}^2$

end wall supports

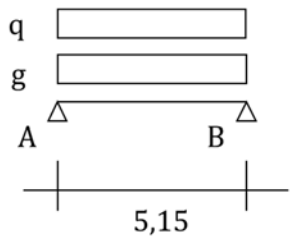
$$\text{req } t = \frac{6{,}99/100}{0{,}15} = 0{,}47 \text{ cm}$$

middle wall support

$$\text{req } t = \frac{21{,}5/100}{0{,}15} = 1{,}43 \text{ cm}$$

2, 3 Floor Slab

system



$l = 5{,}15$ m

loads

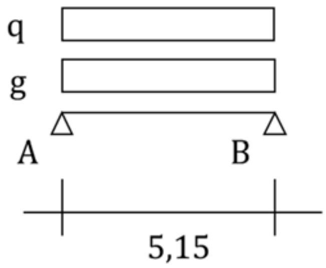
$g_k = 2{,}0\ kN/m^2$

$q_k = 2{,}0\ kN/m^2$

selected dimensions

$$\boxed{\begin{array}{c} t = 170\ mm \\ = 19 + 31{,}5 + 19 + 31{,}5 + 19 + 31{,}5 + 19 \end{array}}$$

max reaction forces



$A_g = 2{,}0 \cdot 4{,}30/2 = 4{,}3\ kN/m$

$A_q = 2{,}0 \cdot 4{,}30/2 = 4{,}3\ kN/m$

$A = 8{,}6\ kN/m$

pressure perpendicular to the grain

wall supports

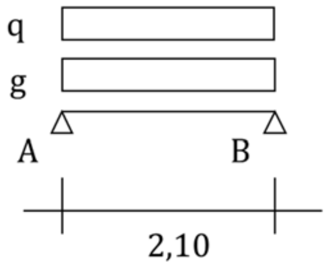
$$\text{req } t = \frac{8{,}6/100}{0{,}15} = 0{,}57\ cm$$

- preliminary design compression strength perpendicular to the grain $\sigma_{p,c} = 1{,}5\ N/mm^2$

4 Floor Slab

system

l = 2,10 m



loads

$g_k = 2{,}0\ \mathrm{kN/m^2}$

$q_k = 2{,}0\ \mathrm{kN/m^2}$

selected dimensions

$$\boxed{\begin{array}{c} t = 95\ \mathrm{mm} \\ = 19 + 19 + 19 + 19 + 19 \end{array}}$$

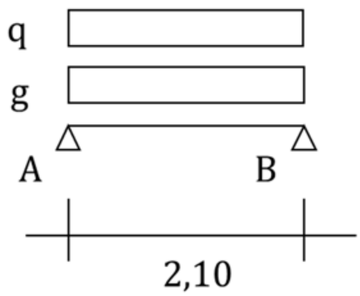
- for fire safety reasons, a slab with at least 5 layers is selected

max reaction forces

$A = 2{,}0 \cdot 2{,}1/2 = 2{,}1\ \mathrm{kN/m}$

$A = 2{,}0 \cdot 2{,}1/2 = 2{,}1\ \mathrm{kN/m}$

$A = 4{,}2\ \mathrm{kN/m}$



pressure perpendicular to the grain

wall supports

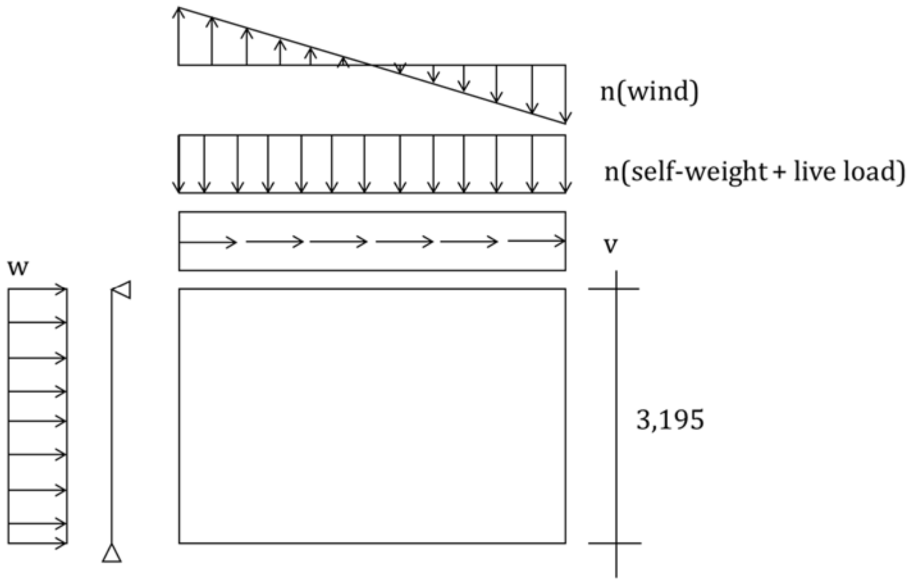
$$\text{req t} = \frac{4{,}2/100}{0{,}15} = 0{,}28\ \mathrm{cm}$$

- preliminary design compression strength perpendicular to the grain $\sigma_{p,c} = 1{,}5\ \mathrm{N/mm^2}$

5 Walls

The full stability analysis was performed using Excel (see appendix). Here, only the wall y5 is shown.

system



global loads

wind loads (wind from the front)

$W_y = 1319$

$M_x = -37094\ \text{kNm}$

wind loads (wind from the side)

$W_x = 1275\ \text{kN}$

$M_y = 34982\ \text{kNm}$

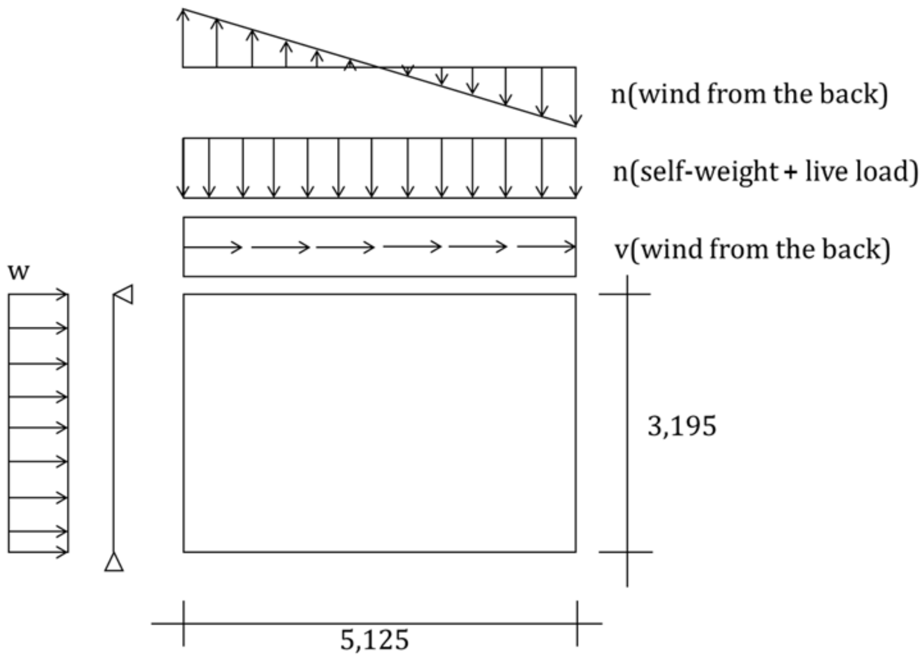
In the end of the preliminary design, walls with three different thicknesses were used: 95 mm, 120 mm and 259 mm. To achieve a more economic design, the thicknesses were decreased in the higher floors, making use of the decreasing loads. Assuming the loads due to the self-weight and the life load (N) to decrease linearly and the loads due to the wind (M) to decrease quadratically, the following thicknesses are calculated:

c) CLT Construction

storey no	M	N	t	t	t
-	[%]	[%]	[mm]	[mm]	[mm]
0	100,00	100,00	95	120	259
1	87,11	93,33	95	120	259
2	75,11	86,67	95	120	259
3	64,00	80,00	95	120	259
4	53,78	73,33	95	120	259
5	44,44	66,67	95	95	170
6	36,00	60,00	95	95	170
7	28,44	53,33	95	95	170
8	21,78	46,67	95	95	170
9	16,00	40,00	95	95	170
10	11,11	33,33	95	95	95
11	7,11	26,67	95	95	95
12	4,00	20,00	95	95	95
13	1,78	13,33	95	95	95
14	0,44	6,67	95	95	95
15	0,00	0,00	95	95	95

5.y5 Outer Wall

system



loads

self-weight of the wall

$g_k = 1{,}0 \text{ kN/m}^2$

over 15 storeys

$g_k = 1 \cdot 15 \cdot 3{,}195 = 47{,}9 \text{ kN/m}$

floor load

$g_k = 2{,}0 \text{ kN/m}^2$

$q_k = 2{,}0 \text{ kN/m}^2$

load factor to account for the openings

$$k = \frac{20{,}65}{5{,}125 + 1{,}1 + 2{,}6 + 1{,}1 + 5{,}125} = 1{,}37$$

length of influence

$$l_e \approx \frac{4{,}38}{2} = 2{,}19 \text{ m}$$

$g = 1{,}37 \cdot 2 \cdot 2{,}19 = 6{,}01 \text{ kN/m}$

$q = 1{,}37 \cdot 2 \cdot 2{,}19 = 6{,}01 \text{ kN/m}$

over 15 storeys

$g = 15 \cdot 6{,}01 = 90{,}15 \text{ kN/m}$

$q = 15 \cdot 0{,}74 \cdot 6{,}01 = 66{,}71 \text{ kN/m}$

- from floor slab 1
- the load per storey is added up for the roof and the 14 storeys with a reduction factor $\alpha_n = 0{,}74$ for the live load

total vertical load (constant)

$n = -47{,}9 - 90{,}15 - 66{,}71 = -204{,}8 \text{ kN/m}$

wind load (linear)

- cf. Excel analysis

wind from the back

max $n = 76{,}5 \text{ kN/m}$

min $n = -133{,}4 \text{ kN/m}$

$v = 9{,}2 \text{ kN/m}$

wind pressure

$c_{pe} = -0{,}8$

$q_p = 1{,}044 \text{ kN/m}^2$

$w = -0{,}8 \cdot 1{,}044 = -0{,}835 \text{ kN/m}^2$

internal forces

total normal force

$n = -204{,}8 - 133{,}4 = -338{,}2 \text{ kN/m}$

$$\text{req } t = \frac{338{,}2/100}{0{,}7 \cdot 0{,}5} = 9{,}66 \text{ cm}$$

- factor 0,7 to account for buckling

bending moment due to wind

$M_w = 0{,}835 \cdot 3{,}195^2/8 = 1{,}07 \text{ kNm/m}$

selected dimensions

$$\boxed{\begin{gathered} t = 95 \text{ mm} \\ A = 100 \cdot 9{,}5 = 950 \text{ cm}^2/\text{m} \\ W_y = 100 \cdot 9{,}5^2/6 = 1504 \text{ cm}^3/\text{m} \end{gathered}}$$

check

normal force

$$\sigma = \frac{338,2}{950} + \frac{107}{1504} = 0,36 + 0,07$$
$$= 0,43 \text{ kN/cm}^2$$

$$\approx 0,8 \cdot 0,5 = 0,40 \text{ kN/m}^2$$

shear forces

$$\tau = \frac{9,2}{100 \cdot 9,5} = 0,01 \text{ kN/cm}^2$$

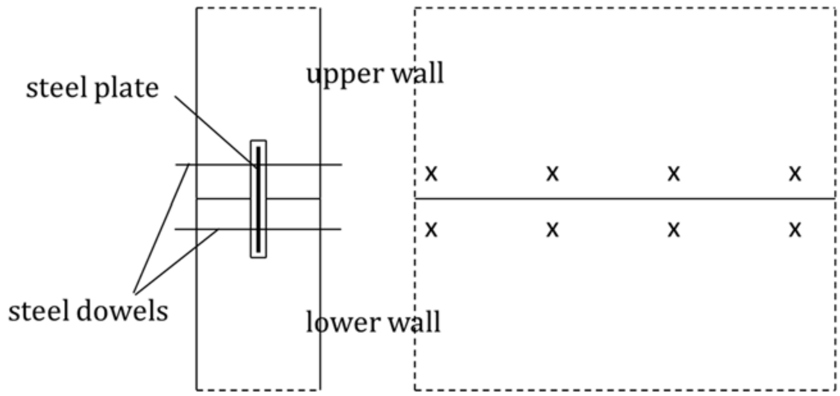
$$< 0,06 \text{ kN/cm}^2$$

- the wind pressure has almost no influence compared with the high vertical loads
- 0,8 is used as a factor to account for buckling because it is assumed that massive elements are less prone to buckling

Anchoring

For tensile stresses, wall y7 becomes decisive for wind from the front. To connect two walls on top of each other, an internal steel plate is put into a groove cut into the walls over the whole length using steel dowels M20.

system



loads

n_t = 132,4 kN/m

- to be on the safe side, only 50 % of the self-weight and live loads are considered

steel dowels

capacity per dowel M20 (double shear with steel plate)

$F_d = 1{,}25 \cdot 4{,}4 \cdot 2^2 = 22$ kN

req n = 132,4/22 = 7 /m

$\boxed{\text{7 M20 /m}}$

Appendix

Attachments

- stability analysis for the panel construction (XLSX)
- calculation of the panel studs (XLSX)
- stability analysis for the CLT construction (XLSX)
- technical plans for the frame construction Pa1.1, Pa2 (PDF)
- technical plans for the panel construction Pb1–Pb3, Pa4.1 (PDF)
- technical plans for the CLT construction Pc1–Pc2, Pc3.1 (PDF)

b) Panel Construction

Summary of the Stability Analysis

orientation	no	l	shear wind from the left side |fx|	|fy|
-	-	m	kN/m	kN/m
x	1	0,6	0,2796526	0
x	2	1,87	2,7164362	0
x	3	2,425	4,5681439	0
x	4	2,425	4,5681439	0
x	5	1,87	2,7164362	0
x	6	0,6	0,2796526	0
x	7	5,085	21,134682	0
x	8	4,57	17,070499	0
x	9	4,57	17,070499	0
x	10	5,085	21,134682	0
x	11	5,085	21,134682	0
x	12	4,57	17,070499	0
x	13	4,57	17,070499	0
x	14	5,085	21,134682	0
x	15	5,085	21,37207	0
x	16	2,5	5,1658845	0
x	17	2,5	5,1658845	0
x	18	5,085	21,37207	0
x	19	5,085	21,37207	0
x	20	2,5	5,1658845	0
x	21	2,5	5,1658845	0
x	22	5,085	21,37207	0
x	23	0,6	0,3121523	0
x	24	1,87	3,0321259	0
x	25	1,105	1,0587382	0
x	26	1,105	1,0587382	0
x	27	1,87	3,0321259	0
x	28	0,6	0,3121523	0
y	1	5,125	0	1,2826201
y	2	1,1	0	0,0590875
y	3	2,6	0	0,3301087
y	4	1,1	0	0,0590875
y	5	5,125	0	1,2826201
y	6	9,275	0	0,9684165
y	7	9,275	0	0,9684165
y	8	9,275	0	0,9684165
y	9	4,64	0	0,2423652
y	10	9,275	0	0,9684165
y	11	4,64	0	0,2423652
y	12	9,275	0	0,9684165
y	13	9,275	0	0,9684165
y	14	5,125	0	1,2826201
y	15	1,1	0	0,0590875
y	16	2,6	0	0,3301087
y	17	1,1	0	0,0590875
y	18	5,125	0	1,2826201
max/min			21,37207	1,2826201

orientation	no	wind from the front |fx|	|fy|	normal stresses wind from the left side max n	min n	|w|
-	-	kN/m	kN/m	kN/m	kN/m	kN/m
x	1	0	0	29,437788	3,6616946	0,24588
x	2	0	0	43,310339	-37,025153	0,766326
x	3	0	0	38,557871	-65,620507	0,65568
x	4	0	0	20,890507	-83,287871	0,65568
x	5	0	0	-7,7048467	-88,040339	0,510884
x	6	0	0	-48,391695	-74,167788	0,16392
x	7	0	0	**117,74776**	-100,70463	0
x	8	0	0	85,787193	-110,54072	0
x	9	0	0	65,81072	-130,51719	0
x	10	0	0	55,974635	-162,47776	0
x	11	0	0	89,191092	-129,2613	0
x	12	0	0	57,230526	-139,09739	0
x	13	0	0	37,254054	-159,07386	0
x	14	0	0	27,417968	**-191,0344**	0
x	15	0	0	89,191092	-129,2613	0
x	16	0	0	16,472078	-90,928312	0
x	17	0	0	-10,915021	-118,31541	0
x	18	0	0	27,417968	**-191,0344**	0
x	19	0	0	**117,74776**	-100,70463	0
x	20	0	0	45,028745	-62,371645	0
x	21	0	0	17,641645	-89,758745	0
x	22	0	0	55,974635	-162,47776	0
x	23	0	0	29,437788	3,6616946	0,24588
x	24	0	0	43,310339	-37,025153	0,766326
x	25	0	0	12,566977	-34,908996	0,301886
x	26	0	0	-9,8260044	-57,296977	0,301886
x	27	0	0	-7,7048467	-88,040339	0,510884
x	28	0	0	-48,391695	-74,167788	0,16392
y	1	0	6,224236	-37,292921	-37,292921	0,65568
y	2	0	0,286737	-37,292921	-37,292921	0,30052
y	3	0	1,6019354	17,623745	17,623745	0,65568
y	4	0	0,286737	-37,292921	-37,292921	0,30052
y	5	0	6,224236	-37,292921	-37,292921	0,65568
y	6	0	**20,385724**	-68,063133	-68,063133	0
y	7	0	**20,385724**	-68,063133	-68,063133	0
y	8	0	**20,385724**	-81,243133	-81,243133	0
y	9	0	5,1019274	-81,243133	-81,243133	0
y	10	0	**20,385724**	-99,6802	-99,6802	0
y	11	0	5,1019274	-99,6802	-99,6802	0
y	12	0	**20,385724**	-86,5002	-86,5002	0
y	13	0	**20,385724**	-86,5002	-86,5002	0
y	14	0	6,224236	-117,27041	-117,27041	0,458976
y	15	0	0,286737	-117,27041	-117,27041	0,210364
y	16	0	1,6019354	-62,353745	-62,353745	0,458976
y	17	0	0,286737	-117,27041	-117,27041	0,210364
y	18	0	6,224236	-117,27041	-117,27041	0,458976
max/min		0	20,385724	117,74776	-191,03443	0,766326

orientation	no	wind from the right side max n	min n	|w|
-	-	kN/m	kN/m	kN/m
x	1	-48,391695	-74,167788	0,16392
x	2	-7,7048467	-88,040339	0,510884
x	3	20,890507	-83,287871	0,65568
x	4	38,557871	-65,620507	0,65568
x	5	43,310339	-37,025153	0,766326
x	6	29,437788	3,6616946	0,24588
x	7	55,974635	-162,47776	0
x	8	65,81072	-130,51719	0
x	9	85,787193	-110,54072	0
x	10	**117,74776**	-100,70463	0
x	11	27,417968	**-191,0344**	0
x	12	37,254054	-159,07386	0
x	13	57,230526	-139,09739	0
x	14	89,191092	-129,2613	0
x	15	27,417968	**-191,0344**	0
x	16	-10,915021	-118,31541	0
x	17	16,472078	-90,928312	0
x	18	89,191092	-129,2613	0
x	19	55,974635	-162,47776	0
x	20	17,641645	-89,758745	0
x	21	45,028745	-62,371645	0
x	22	**117,74776**	-100,70463	0
x	23	-48,391695	-74,167788	0,16392
x	24	-7,7048467	-88,040339	0,510884
x	25	-9,8260044	-57,296977	0,301886
x	26	12,566977	-34,908996	0,301886
x	27	43,310339	-37,025153	0,766326
x	28	29,437788	3,6616946	0,24588
y	1	-117,27041	-117,27041	0,458976
y	2	-117,27041	-117,27041	0,210364
y	3	-62,353745	-62,353745	0,458976
y	4	-117,27041	-117,27041	0,210364
y	5	-117,27041	-117,27041	0,458976
y	6	-86,5002	-86,5002	0
y	7	-86,5002	-86,5002	0
y	8	-99,6802	-99,6802	0
y	9	-99,6802	-99,6802	0
y	10	-81,243133	-81,243133	0
y	11	-81,243133	-81,243133	0
y	12	-68,063133	-68,063133	0
y	13	-68,063133	-68,063133	0
y	14	-37,292921	-37,292921	0,65568
y	15	-37,292921	-37,292921	0,30052
y	16	17,623745	17,623745	0,65568
y	17	-37,292921	-37,292921	0,30052
y	18	-37,292921	-37,292921	0,65568
max/min		117,74776	-191,03443	

orientation	no	wind from the front max n	min n	|w|
-	-	kN/m	kN/m	kN/m
x	1	-56,502248	-56,502248	0,116793
x	2	-56,502248	-56,502248	0,3640049
x	3	-56,502248	-56,502248	0,467172
x	4	-56,502248	-56,502248	0,467172
x	5	-56,502248	-56,502248	0,3640049
x	6	-56,502248	-56,502248	0,116793
x	7	-23,913687	-23,913687	0
x	8	-23,913687	-23,913687	0
x	9	-23,913687	-23,913687	0
x	10	-23,913687	-23,913687	0
x	11	-52,470354	-52,470354	0
x	12	-52,470354	-52,470354	0
x	13	-52,470354	-52,470354	0
x	14	-52,470354	-52,470354	0
x	15	-45,091811	-45,091811	0
x	16	-45,091811	-45,091811	0
x	17	-45,091811	-45,091811	0
x	18	-45,091811	-45,091811	0
x	19	-16,535145	-16,535145	0
x	20	-16,535145	-16,535145	0
x	21	-16,535145	-16,535145	0
x	22	-16,535145	-16,535145	0
x	23	16,053416	16,053416	0,16392
x	24	16,053416	16,053416	0,510884
x	25	16,053416	16,053416	0,301886
x	26	16,053416	16,053416	0,301886
x	27	16,053416	16,053416	0,510884
x	28	16,053416	16,053416	0,16392
y	1	5,627602	-210,45827	0,65568
y	2	-62,00025	-108,37966	0,30052
y	3	34,587612	-75,086443	0,65568
y	4	-41,902507	-88,281915	0,30052
y	5	60,17611	-155,90977	0,98352
y	6	100,40673	-290,656	0
y	7	140,37384	-250,6889	0
y	8	87,226733	-303,836	0
y	9	21,32054	-174,31623	0
y	10	87,226733	-303,836	0
y	11	21,32054	-174,31623	0
y	12	100,40673	-290,656	0
y	13	140,37384	-250,6889	0
y	14	5,627602	-210,45827	0,65568
y	15	-62,00025	-108,37966	0,30052
y	16	34,587612	-75,086443	0,65568
y	17	-41,902507	-88,281915	0,30052
y	18	60,17611	-155,90977	0,98352
max/min		140,37384	-303,836	

		wind from the back		
orientation	no	max n	min n	|w|
-	-	kN/m	kN/m	kN/m
x	1	11,772248	11,772248	0,16392
x	2	11,772248	11,772248	0,510884
x	3	11,772248	11,772248	0,65568
x	4	11,772248	11,772248	0,65568
x	5	11,772248	11,772248	0,510884
x	6	11,772248	11,772248	0,16392
x	7	-20,816313	-20,816313	0
x	8	-20,816313	-20,816313	0
x	9	-20,816313	-20,816313	0
x	10	-20,816313	-20,816313	0
x	11	-49,37298	-49,37298	0
x	12	-49,37298	-49,37298	0
x	13	-49,37298	-49,37298	0
x	14	-49,37298	-49,37298	0
x	15	-56,751522	-56,751522	0
x	16	-56,751522	-56,751522	0
x	17	-56,751522	-56,751522	0
x	18	-56,751522	-56,751522	0
x	19	-28,194855	-28,194855	0
x	20	-28,194855	-28,194855	0
x	21	-28,194855	-28,194855	0
x	22	-28,194855	-28,194855	0
x	23	-60,783416	-60,783416	0,116793
x	24	-60,783416	-60,783416	0,3640049
x	25	-60,783416	-60,783416	0,2150938
x	26	-60,783416	-60,783416	0,2150938
x	27	-60,783416	-60,783416	0,3640049
x	28	-60,783416	-60,783416	0,116793
y	1	55,894942	-160,19094	0,98352
y	2	-46,183675	-92,563083	0,30052
y	3	30,306443	-79,317612	0,65568
y	4	-66,281419	-112,66083	0,30052
y	5	1,3464336	-214,73944	0,65568
y	6	136,09267	-254,97007	0
y	7	96,125564	-294,93372	0
y	8	122,91267	-268,15007	0
y	9	-6,607099	-202,24387	0
y	10	122,91267	-268,15007	0
y	11	-6,607099	-202,24387	0
y	12	136,09267	-254,97007	0
y	13	96,125564	-294,93372	0
y	14	55,894942	-160,19094	0,98352
y	15	-46,183675	-92,563083	0,30052
y	16	30,306443	-79,317612	0,65568
y	17	-66,281419	-112,66083	0,30052
y	18	1,3464336	-214,73944	0,65568
max/min		136,09267	-294,93717	

Loads and Selected Cross-Sections of the Studs

Utilising the symmetry of the building, only one half of the walls is considered.

(compression +, tension -)

	wall		load-bearing studs	selected cross-sections									
	orientation	no	b	F1	h1	F2	h2	F3	h3	F4	h4	F5	h5
	-	-	cm	kN	cm	kN	cm	kN	cm	kN	cm	kN	cm
x1	x	1	20	26,4	28	23,8	20						
x2	x	2	20	78,2	20	73,7	20						
x3	x	3	20	95,6	20	95,6	20						
x4	x	4											
x5	x	5											
x6	x	6											
x7	x	7	28	58,7	28	134	28	188	28	107	28		
x8	x	8	28	200	28	154	28						
x9	x	9											
x10	x	10											
x11	x	11	28	64,3	28	235	28	181	28	226	28	119	28
x12	x	12	28	57,2	28	196	28	153	28	206	28	93,6	28
x13	x	13											
x14	x	14											
x15	x	15	28	64,3	28	235	28	181	28	226	28	119	28
x16	x	16	28	167	28	135	28						
x17	x	17											
x18	x	18											
x19	x	19	28	58,7	28	134	28	188	28	107	28		
x20	x	20	28	95,4	28	63,2	28						
x21	x	21											
x22	x	22											
x23	x	23	20	26,4	28	24,9	20						
x24	x	24	20	78,2	20	77,7	20						
x25	x	25	20	101	20	101	20						
x26	x	26											
x27	x	27											
x28	x	28											
y1	y	1	28	139	28	295	28	145	28	182	28	258	28
y2	y	2	28	107	28	107	28						
y3	y	3	28	110	28	110	28						
y4	y	4	28	107	28	107	28						
y5	y	5	28	140	28	285	28	159	28	153	28	270	28
y6	y	6	28	107	28	271	28	101	28	248	28	248	28
y7	y	7	28	88,7	20	239	28	199	28	141	28	265	28
y8	y	8	28	116	28	295	28	111	28	275	28	281	28
y9	y	9	28	102	28	204	28	202	28	248	28	251	28
y10	y	10											
y11	y	11											
y12	y	12											
y13	y	13											
y14	y	14											
y15	y	15											
y16	y	16											
y17	y	17											
y18	y	18											

	wall		load-b										
	orientation	no	b	F6	h6	F7	h7	F8	h8	F9	h9	F10	h10
	-	-	cm	kN	cm	kN	cm	kN	cm	kN	cm	kN	cm
x1	x	1	20										
x2	x	2	20										
x3	x	3	20										
x4	x	4											
x5	x	5											
x6	x	6											
x7	x	7	28										
x8	x	8	28										
x9	x	9											
x10	x	10											
x11	x	11	28										
x12	x	12	28										
x13	x	13											
x14	x	14											
x15	x	15	28										
x16	x	16	28										
x17	x	17											
x18	x	18											
x19	x	19	28										
x20	x	20	28										
x21	x	21											
x22	x	22											
x23	x	23	20										
x24	x	24	20										
x25	x	25	20										
x26	x	26											
x27	x	27											
x28	x	28											
y1	y	1	28	129	28								
y2	y	2	28										
y3	y	3	28										
y4	y	4	28										
y5	y	5	28	129	28								
y6	y	6	28	199	28	278	28	265	28	139	28	205	28
y7	y	7	28	278	28	199	28	248	28	245	28	111	28
y8	y	8	28	230	28	311	28	292	28	150	28	222	28
y9	y	9	28	63	28								
y10	y	10											
y11	y	11											
y12	y	12											
y13	y	13											
y14	y	14											
y15	y	15											
y16	y	16											
y17	y	17											
y18	y	18											

	wall		load-b				
	orientation	no	b	F11	h11	F12	h12
	-	-	cm	kN	cm	kN	cm
x1	x	1	20				
x2	x	2	20				
x3	x	3	20				
x4	x	4					
x5	x	5					
x6	x	6					
x7	x	7	28				
x8	x	8	28				
x9	x	9					
x10	x	10					
x11	x	11	28				
x12	x	12	28				
x13	x	13					
x14	x	14					
x15	x	15	28				
x16	x	16	28				
x17	x	17					
x18	x	18					
x19	x	19	28				
x20	x	20	28				
x21	x	21					
x22	x	22					
x23	x	23	20				
x24	x	24	20				
x25	x	25	20				
x26	x	26					
x27	x	27					
x28	x	28					
y1	y	1	28				
y2	y	2	28				
y3	y	3	28				
y4	y	4	28				
y5	y	5	28				
y6	y	6	28	216	28	72,6	20
y7	y	7	28	247	28	93,8	28
y8	y	8	28	232	28	77,9	20
y9	y	9	28				
y10	y	10					
y11	y	11					
y12	y	12					
y13	y	13					
y14	y	14					
y15	y	15					
y16	y	16					
y17	y	17					
y18	y	18					

c) CLT Construction

Summary of the Stability Analysis

orientation	no	l	wind from the side |fx|	|fy|
-	-	m	kN/m	kN/m
x	1	0,6	0,2798856	0
x	2	1,87	2,7186997	0
x	3	2,425	4,5719504	0
x	4	2,425	4,5719504	0
x	5	1,87	2,7186997	0
x	6	0,6	0,2798856	0
x	7	5,085	21,135799	0
x	8	4,57	17,0714	0
x	9	4,57	17,0714	0
x	10	5,085	21,135799	0
x	11	5,085	21,135799	0
x	12	4,57	17,0714	0
x	13	4,57	17,0714	0
x	14	5,085	21,135799	0
x	15	5,085	21,36965	0
x	16	2,5	5,1652994	0
x	17	2,5	5,1652994	0
x	18	5,085	21,36965	0
x	19	5,085	21,36965	0
x	20	2,5	5,1652994	0
x	21	2,5	5,1652994	0
x	22	5,085	21,36965	0
x	23	0,6	0,3119011	0
x	24	1,87	3,029686	0
x	25	1,105	1,0578862	0
x	26	1,105	1,0578862	0
x	27	1,87	3,029686	0
x	28	0,6	0,3119011	0
y	1	5,125	0	1,2635106
y	2	1,1	0	0,0582072
y	3	2,6	0	0,3251905
y	4	1,1	0	0,0582072
y	5	5,125	0	1,2635106
y	6	9,275	0	0,9539883
y	7	9,275	0	0,9539883
y	8	9,275	0	0,9539883
y	9	4,64	0	0,2387543
y	10	9,275	0	0,9539883
y	11	4,64	0	0,2387543
y	12	9,275	0	0,9539883
y	13	9,275	0	0,9539883
y	14	5,125	0	1,2635106
y	15	1,1	0	0,0582072
y	16	2,6	0	0,3251905
y	17	1,1	0	0,0582072
y	18	5,125	0	1,2635106
y	19	2,1	0	0,1289574
y	20	2,1	0	0,1289574
y	21	2,1	0	0,1289574
y	22	2,1	0	0,1289574
max/min			21,36965	1,2635106

		wind from the front		wind from the left side		
orientation	no	|fx|	|fy|	n1	n2	max |M|
-	-	kN/m	kN/m	kN/m	kN/m	kNm/m
x	1	0	0	-274,29784	-299,09129	1,5985767
x	2	0	0	-260,95414	-338,22706	1,5985767
x	3	0	0	-265,52543	-365,73229	1,0657178
x	4	0	0	-282,51927	-382,72614	1,0657178
x	5	0	0	-310,02451	-387,29743	1,0657178
x	6	0	0	-349,16028	-373,95373	1,0657178
x	7	0	0	-53,185022	-263,30952	0
x	8	0	0	-83,927179	-272,77063	0
x	9	0	0	-103,1421	-291,98555	0
x	10	0	0	-112,60322	-322,72771	0
x	11	0	0	23,435914	-186,68858	0
x	12	0	0	-79,574003	-268,41745	0
x	13	0	0	-98,788927	-287,63238	0
x	14	0	0	-35,982279	-246,10677	0
x	15	0	0	24,737978	-185,38652	0
x	16	0	0	-45,208822	-148,51487	0
x	17	0	0	-71,551864	-174,85791	0
x	18	0	0	-34,680215	-244,80471	0
x	19	0	0	-50,309663	-260,43412	0
x	20	0	0	-120,25643	-223,56247	0
x	21	0	0	-146,59947	-249,90552	0
x	22	0	0	-109,72782	-319,85232	0
x	23	0	0	-289,09791	-313,89136	1,5985767
x	24	0	0	-275,75421	-353,02713	1,5985767
x	25	0	0	-305,32556	-350,98684	1,0657178
x	26	0	0	-326,86487	-372,52615	1,0657178
x	27	0	0	-324,82458	-402,0975	1,0657178
x	28	0	0	-363,96035	-388,7538	1,0657178
y	1	0	6,1830854	-166,31565	-166,31565	1,0657178
y	2	0	0,2848412	-166,31565	-166,31565	1,0657178
y	3	0	1,5913445	-84,665135	-84,665135	1,0657178
y	4	0	0,2848412	-166,31565	-166,31565	1,0657178
y	5	0	6,1830854	-166,31565	-166,31565	1,0657178
y	6	0	20,250947	-149,0694	-149,0694	0
y	7	0	20,250947	-149,0694	-149,0694	0
y	8	0	20,250947	-173,4729	-173,4729	0
y	9	0	5,0681967	-234,30727	-234,30727	0
y	10	0	20,250947	-191,2071	-191,2071	0
y	11	0	5,0681967	-252,04148	-252,04148	0
y	12	0	20,250947	-166,8036	-166,8036	0
y	13	0	20,250947	-166,8036	-166,8036	0
y	14	0	6,1830854	-243,24421	-243,24421	0,7460025
y	15	0	0,2848412	-243,24421	-243,24421	0,7460025
y	16	0	1,5913445	-161,5987	-161,5987	0,7460025
y	17	0	0,2848412	-243,24421	-243,24421	0,7460025
y	18	0	6,1830854	-243,24421	-243,24421	0,7460025
y	19	0	1,0381404	-1015,332	-1015,332	0
y	20	0	1,0381404	-1015,332	-1015,332	0
y	21	0	1,0381404	-1062,0952	-1062,095	0
y	22	0	1,0381404	-1062,0952	-1062,095	0
max/min		0	20,250947	24,737978	-1062,0952	1,5985767
				24,737978	-402,0975	

		wind from the right side			wind from the front		
orientation	no	n1	n2	max |M|	n1	n2	max |M|
-	-	kN/m	kN/m	kNm/m	kN/m	kN/m	kNm/m
x	1	-373,95373	-349,16028	1,0657178	-357,37616	-357,37616	0,7593239
x	2	-387,29743	-310,02451	1,0657178	-357,37616	-357,37616	0,7593239
x	3	-382,72614	-282,51927	1,0657178	-357,37616	-357,37616	0,7593239
x	4	-365,73229	-265,52543	1,0657178	-357,37616	-357,37616	0,7593239
x	5	-338,22706	-260,95414	1,5985767	-357,37616	-357,37616	0,7593239
x	6	-299,09129	-274,29784	1,5985767	-357,37616	-357,37616	0,7593239
x	7	-322,72771	-112,60322	0	-189,55288	-189,55288	0
x	8	-291,98555	-103,1421	0	-189,55288	-189,55288	0
x	9	-272,77063	-83,927179	0	-189,55288	-189,55288	0
x	10	-263,30952	-53,185022	0	-189,55288	-189,55288	0
x	11	-246,10677	-35,982279	0	-112,93194	-112,93194	0
x	12	-287,63238	-98,788927	0	-185,1997	-185,1997	0
x	13	-268,41745	-79,574003	0	-185,1997	-185,1997	0
x	14	-186,68858	23,435914	0	-112,93194	-112,93194	0
x	15	-244,80471	-34,680215	0	-104,46297	-104,46297	0
x	16	-174,85791	-71,551864	0	-104,46297	-104,46297	0
x	17	-148,51487	-45,208822	0	-104,46297	-104,46297	0
x	18	-185,38652	24,737978	0	-104,46297	-104,46297	0
x	19	-319,85232	-109,72782	0	-179,51058	-179,51058	0
x	20	-249,90552	-146,59947	0	-179,51058	-179,51058	0
x	21	-223,56247	-120,25643	0	-179,51058	-179,51058	0
x	22	-260,43412	-50,30968	0	-179,51058	-179,51058	0
x	23	-388,7538	-363,96035	1,0657178	-301,7016	-301,7016	1,0657178
x	24	-402,0975	-324,82458	1,0657178	-301,7016	-301,7016	1,0657178
x	25	-372,52615	-326,86487	1,0657178	-301,7016	-301,7016	1,0657178
x	26	-350,98684	-305,32556	1,0657178	-301,7016	-301,7016	1,0657178
x	27	-353,02713	-275,75421	1,5985767	-301,7016	-301,7016	1,0657178
x	28	-313,89136	-289,09791	1,5985767	-301,7016	-301,7016	1,0657178
y	1	-243,24421	-243,24421	0,7460025	-124,3409	-334,22904	1,0657178
y	2	-243,24421	-243,24421	0,7460025	-190,02906	-235,07822	1,0657178
y	3	-161,5987	-161,5987	0,7460025	-67,902559	-174,3824	1,0657178
y	4	-243,24421	-243,24421	0,7460025	-170,50776	-215,55692	1,0657178
y	5	-243,24421	-243,24421	0,7460025	-71,356941	-281,24508	1,5985767
y	6	-166,8036	-166,8036	0	14,563224	-365,28312	0
y	7	-166,8036	-166,8036	0	53,383997	-326,46234	0
y	8	-191,2071	-191,2071	0	-9,8402765	-389,68662	0
y	9	-252,04148	-252,04148	0	-134,69053	-324,71609	0
y	10	-173,4729	-173,4729	0	-9,8402765	-389,68662	0
y	11	-234,30727	-234,30727	0	-134,69053	-324,71609	0
y	12	-149,0694	-149,0694	0	14,563224	-365,28312	0
y	13	-149,0694	-149,0694	0	53,383997	-326,46234	0
y	14	-166,31565	-166,31565	1,0657178	-124,3409	-334,22904	1,0657178
y	15	-166,31565	-166,31565	1,0657178	-190,02906	-235,07822	1,0657178
y	16	-84,665135	-84,665135	1,0657178	-67,902559	-174,3824	1,0657178
y	17	-166,31565	-166,31565	1,0657178	-170,50776	-215,55692	1,0657178
y	18	-166,31565	-166,31565	1,0657178	-71,356941	-281,24508	1,5985767
y	19	-1062,0952	-1062,0952	0	-1013,1356	-1099,139	0
y	20	-1062,0952	-1062,0952	0	-974,31482	-1060,3178	0
y	21	-1015,332	-1015,332	0	-1013,1356	-1099,139	0
y	22	-1015,332	-1015,332	0	-974,31482	-1060,3178	0
max/min		-84,665135	-1062,0952	1,5985767	53,383997	-1099,1385	1,5985767
		-84,665135	-363,96035		53,383997	-389,68662	

		wind from the back				
orientation	no	n1	n2	max |M|	req t	selected t
-	-	kN/m	kN/m	kNm/m	mm	mm
x	1	-290,87541	-290,87541	1,0657178	93,49	120
x	2	-290,87541	-290,87541	1,0657178	96,82	120
x	3	-290,87541	-290,87541	1,0657178	95,68	120
x	4	-290,87541	-290,87541	1,0657178	95,68	120
x	5	-290,87541	-290,87541	1,0657178	96,82	120
x	6	-290,87541	-290,87541	1,0657178	93,49	120
x	7	-186,35985	-186,35985	0	80,68	95
x	8	-186,35985	-186,35985	0	73,00	95
x	9	-186,35985	-186,35985	0	73,00	95
x	10	-186,35985	-186,35985	0	80,68	95
x	11	-109,73891	-109,73891	0	61,53	95
x	12	-182,00667	-182,00667	0	71,91	95
x	13	-182,00667	-182,00667	0	71,91	95
x	14	-109,73891	-109,73891	0	61,53	95
x	15	-115,60376	-115,60376	0	61,20	95
x	16	-115,60376	-115,60376	0	43,71	95
x	17	-115,60376	-115,60376	0	43,71	95
x	18	-115,60376	-115,60376	0	61,20	95
x	19	-190,65137	-190,65137	0	79,96	95
x	20	-190,65137	-190,65137	0	62,48	95
x	21	-190,65137	-190,65137	0	62,48	95
x	22	-190,65137	-190,65137	0	79,96	95
x	23	-376,15011	-376,15011	0,7593239	97,19	120
x	24	-376,15011	-376,15011	0,7593239	100,52	120
x	25	-376,15011	-376,15011	0,7593239	94,04	120
x	26	-376,15011	-376,15011	0,7593239	94,04	120
x	27	-376,15011	-376,15011	0,7593239	100,52	120
x	28	-376,15011	-376,15011	0,7593239	97,19	120
y	1	-285,21896	-75,330823	1,5985767	83,56	95
y	2	-219,5308	-174,48164	1,0657178	60,81	95
y	3	-178,35628	-71,876442	1,0657178	44,59	95
y	4	-239,0521	-194,00294	1,0657178	60,81	95
y	5	-338,20292	-128,31478	1,0657178	84,55	95
y	6	-330,43622	49,410115	0	91,32	120
y	7	-369,257	10,589341	0	92,31	120
y	8	-354,83972	25,006615	0	97,42	120
y	9	-351,65822	-161,63266	0	87,91	120
y	10	-354,83972	25,006615	0	97,42	120
y	11	-351,65822	-161,63266	0	87,91	120
y	12	-330,43622	49,410115	0	91,32	120
y	13	-369,257	10,589341	0	92,31	120
y	14	-285,21896	-75,330823	1,5985767	83,56	95
y	15	-219,5308	-174,48164	1,0657178	60,81	95
y	16	-178,35628	-71,876442	1,0657178	44,59	95
y	17	-239,0521	-194,00294	1,0657178	60,81	95
y	18	-338,20292	-128,31478	1,0657178	84,55	95
y	19	-1064,2917	-978,28871	0	274,78	259
y	20	-1103,1124	-1017,1095	0	275,78	259
y	21	-1064,2917	-978,28871	0	274,78	259
y	22	-1103,1124	-1017,1095	0	275,78	259
max/min		-109,73891	-1017,1095	1,5985767		
		-109,73891	-376,15011			

Eurocode Design

Table of Contents

General

Remarks

The intention of the EC design is to prove the feasibility of the structures. A description of the building and the loadbearing concept of the three structures is included in the documentation of the preliminary design.

Only the decisive load combinations are shown in this document, a complete summary of all the checks was done in Excel. An extract of the Excel document can be found in the appendix, the files themselves are attached electronically. Technical reference plans are also attached (cf. appendix).

Materials

- wooden members (beams, studs, floor joists, …): glulam GL24h
  - properties according to EN 14080
- structural sheathing: plywood Finnforest Spruce konstruksjonskryssfiner (distributed under the name "Gran III/III" by Fritzøe Engros AS)
  - type EN 636-1 (plywood for use in dry climate)
  - properties according to the technical approval [3]
- mass timber: CLT KL-trä by Martinsons (distributed by Splitkon AS)
  - properties according to the technical approval [1]

Software

- Stab2d (simple analysis program for two-dimensional frameworks)
- Microsoft EXCEL ©
- Dlubal RFEM ©

Extracts from the calculations with EXCEL and RFEM can be found in the appendix. The files themselves are attached electronically.

Literature

[1] Teknisk Godkjenning Martinsons KL-trä
[2] Bautabellen für Ingenieure, 20th edition 2012
[3] Teknisk Godkjenning Finnforest Spruce konstruksjonskryssfiner
[4] EN 1990
[5] EN 1991-1-1
[6] EN 1995-1-1
[7] Technical Data Sheet Simpson Strongtie BT4 Bjelkebærere

Load cases and combinations

For consistent numbering between this design and the RFEM-models, always two numbers are reserved for load cases and combinations that include wind loads, one for wind from the front (or back) and one for wind from the side. In the FEM-analysis with RFEM, only wind from the front and from the left side was examined, making use of the symmetry of the building.

load case	description	symbol	duration
LC1	self-weight	G	permanent
LC2	snow	S	short-term
LC3/4	wind	W	instantaneous
LC5	live load	Q	medium-term

load combination no	combination rule	duration
CO1	$E_d = 1{,}2 \cdot G$	permanent
CO2	$E_d = 1{,}2 \cdot G + 1{,}5 \cdot Q$	medium-term
CO3	$E_d = 1{,}2 \cdot G + 1{,}5 \cdot S + 1{,}5 \cdot 0{,}7 \cdot Q$	short-term
CO4	$E_d = 1{,}2 \cdot G + 1{,}5 \cdot Q + 1{,}5 \cdot 0{,}7 \cdot S$	short-term
CO5/6	$E_d = 1{,}2 \cdot G + 1{,}5 \cdot W + 1{,}5 \cdot 0{,}7 \cdot S + 1{,}5 \cdot 0{,}7 \cdot Q$	instantaneous
CO7/8	$E_d = 1{,}2 \cdot G + 1{,}5 \cdot Q + 1{,}5 \cdot 0{,}7 \cdot S + 1{,}5 \cdot 0{,}6 \cdot W$	instantaneous
CO9/10	$E_d = 1{,}2 \cdot G + 1{,}5 \cdot S + 1{,}5 \cdot 0{,}6 \cdot W + 1{,}5 \cdot 0{,}7 \cdot Q$	instantaneous
CO11/12	$E_d = 1{,}0 \cdot G + 1{,}5 \cdot W$	instantaneous

Design Factors

Since CLT is not considered in the EC, the same design factors are applied as for glulam.

material safety factors γ_M

material	γ_M
glulam	1,25
CLT	1,25
plywood	1,2
connections	1,3

modification factors k_{mod}

material	load duration	climate class		
		1	2	3
glulam, CLT, plywood	permanent	0,6	0,6	0,5
	medium-term	0,8	0,8	0,65
	short-term	0,9	0,9	0,7
	instantaneous	1,1	1,1	0,9

deformation factors k_{def}

material	climate class		
	1	2	3
glulam, CLT	0,6	0,8	2,0
plywood	0,8	-	-

a) Frame Construction

Element Reference Overview

reference no	element	page
1	columns	
1.1	inner columns	36
1.2	corner columns	-
1.3	side columns	-
1.4	side columns	38
2	beams	
2.1	beam	20
2.2	beam	12
2.3	beam	22
2.4	beam	25
3	joists	
3.1	joists	7
3.2	joists	11
4	diagonal	40
5	structural sheathing of the floors	42
6	vertical façade carriers	45
7	diaphragm	23
8	frame	24
	connections	47

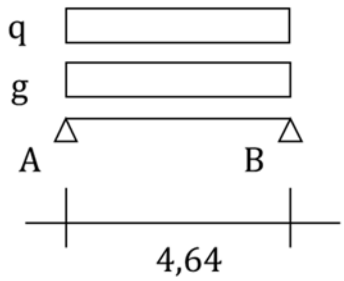
Remarks

In comparison to the preliminary design, the cross-sections of the columns were reduced. All columns are now b/h = 40/40 cm, except for those at the sides. Because the beams are attached to them, they need a larger cross-section: b/h = 40/60 cm. Because the connections to the beams require this space, the cross-sections are not decreased over the height of the building.

a) Frame Construction

3.1 Roof Joists

system



$l = \frac{9{,}275}{2} = 4{,}64\ \text{m}$

cross-section

$b/h = 14/24$

$e = 65\ \text{cm}$

$A = 336\ \text{cm}^2$

$I_y = 16128\ \text{cm}^4$

$W_y = 1344\ \text{cm}^3$

material

glulam GL24h

$f_{mk} = 24\ \text{N/mm}^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ \text{N/mm}^2$

$E_{0,mean} = 11500\ \text{N/mm}^2$

$k_{def} = 0{,}6$

loads

$g = 1{,}65\ \text{kN/m}^2$ $\quad g_k = 1{,}65 \cdot 0{,}65 = 1{,}07\ \text{kN/m}$

$s = 3{,}8\ \text{kN/m}^2$ $\quad s_k = 3{,}8 \cdot 0{,}65 = 2{,}47\ \text{kN/m}$

$q_p = 1044\ \text{N/m}^2$

$c_{pe}(I) = +0{,}2$

$w = 0{,}2 \cdot 1{,}044 = 0{,}21\ \text{kN/m}^2$ $\quad w_k = 0{,}21 \cdot 0{,}65 = 0{,}14\ \text{kN/m}$

$q(H) = 0{,}75\ \text{kN/m}^2$ $\quad q_k = 0{,}75 \cdot 0{,}65 = 0{,}49\ \text{kN/m}$

$Q_k = 1{,}5\ \text{kN}$

ULS CO3

$k_{mod} = 0{,}9$

$p_d = 1{,}20 \cdot 1{,}07 + 1{,}50 \cdot 2{,}47 = 4{,}99\ \text{kN/m}$

$Q_d = 1{,}50 \cdot 0{,}70 \cdot 1{,}5 = 1{,}58\ \text{kN}$

- the single load Q becomes decisive

$M_d = 0{,}125 \cdot 4{,}99 \cdot 4{,}64^2 + 0{,}250 \cdot 1{,}58 \cdot 4{,}64$
$= 15{,}26\ \text{kNm}$

$\sigma_{md} = \frac{1526}{1344} = 1{,}14\ \text{kN/cm}^2$

$f_{md} = 0{,}9 \cdot \frac{2{,}4}{1{,}15} = 1{,}88\ \text{kN/cm}^2$

$$\eta = \frac{1{,}14}{1{,}88} = 0{,}60 < 1{,}0$$

- no lateral torsional buckling, because the sheathing secures the compression flange of the joists

$$V_d = 0{,}500 \cdot 4{,}99 \cdot 4{,}64 + 1{,}58 = 13{,}16 \text{ kN}$$

$$\tau_d = 1{,}5 \cdot \frac{13{,}16}{0{,}67 \cdot 336} = 0{,}088 \text{ kN/cm}^2$$

$$\tau_d = 1{,}5 \cdot \frac{V_d}{k_{cr} \cdot A}$$

(for rectangular cross-sections)

$$f_{vd} = 0{,}9 \cdot \frac{0{,}35}{1{,}15} = 0{,}27 \text{ kN/cm}^2$$

$$\eta = \frac{0{,}088}{0{,}27} = 0{,}32 < 1{,}0$$

SLS CO9/10

$$w_{inst,g} = \frac{\frac{1{,}07}{100} \cdot 464^4}{76{,}8 \cdot 1150 \cdot 16128} = 0{,}349 \text{ cm}$$

$$w = \frac{p \cdot l^4}{76{,}8 \cdot EI}$$

(for single span beams)

$$w_{inst,s} = 0{,}804 \text{ cm}$$

$$w_{inst,w} = 0{,}044 \text{ cm}$$

$$w_{inst,Q} = \frac{1{,}5 \cdot 464^3}{48 \cdot 1150 \cdot 16128} = 0{,}168 \text{ cm}$$

instantaneous deformation

characteristic combination acc. to EC5-1-1 2.2.3(2):

$$w = w_g + w_{Q,1} + \sum_{i>1} \psi_{0,i} w_{Q,i}$$

$$w_{inst} = 0{,}349 + 0{,}804 + 0{,}6 \cdot 0{,}044 + 0{,}7 \cdot 0{,}168 = 1{,}30 \text{ cm}$$

$$\max w_{inst} = \frac{464}{300} = 1{,}55 \text{ cm}$$

$$\eta = \frac{1{,}30}{1{,}55} = 0{,}84 < 1{,}0$$

final deformation

simplified approach acc. to EC5-1-1 2.2.3(5):

$$w_{fin} = w_{fin,G} + w_{fin,Q,1} + \sum_{i>1} w_{fin,Q,i}$$

$$w_{fin,G} = w_{inst,G}(1 + k_{def})$$

$$w_{fin,Q,1} = w_{inst,Q,1}(1 + \psi_{2,1} k_{def})$$

$$w_{fin,Q,i} = w_{inst,Q,i}(\psi_{0,i} + \psi_{2,i} k_{def})$$

$$w_{fin} = 0{,}349 \cdot (1 + 0{,}6) + 0{,}804 \cdot (1 + 0{,}2 \cdot 0{,}6) + 0{,}044 \cdot (0{,}6 + 0 \cdot 0{,}6) + 0{,}168 \cdot (0{,}7 + 0{,}3 \cdot 0{,}6) = 1{,}63 \text{ cm}$$

$$\max w_{fin} = \frac{464}{150} = 3{,}09 \text{ cm}$$

$$\eta = \frac{1{,}63}{3{,}09} = 0{,}53 < 1{,}0$$

max reaction forces

$$A_{gk} = 0{,}500 \cdot 1{,}07 \cdot 4{,}64 = 2{,}49 \text{ kN}$$

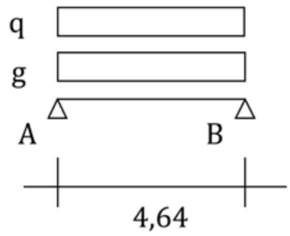
$$A_{sk} = 0{,}500 \cdot 2{,}47 \cdot 4{,}64 = 5{,}73 \text{ kN}$$

$$A_{wk} = 0{,}500 \cdot 0{,}14 \cdot 4{,}64 = 0{,}31 \text{ kN}$$

$$A_{qk} = 0{,}500 \cdot 0{,}49 \cdot 4{,}64 = 1{,}13 \text{ kN}$$

3.1 Floor Joists

system



$l = \dfrac{9{,}275}{2} = 4{,}64 \text{ m}$

cross-section

$b/h = 14/24$

$e = 65 \text{ cm}$

$A = 336 \text{ cm}^2$

$I_y = 16128 \text{ cm}^4$

$W_y = 1344 \text{ cm}^3$

material

glulam GL24h

$f_{mk} = 24 \text{ N/mm}^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5 \text{ N/mm}^2$

$E_{0,mean} = 11500 \text{ N/mm}^2$

$k_{def} = 0{,}6$

loads

$g = 1{,}34 \text{ kN/m}^2$ $\qquad g_k = 1{,}34 \cdot 0{,}65 = 0{,}87 \text{ kN/m}$

$s = 0$

$q_p = 0$

$q(A) = 2{,}5 \text{ kN/m}^2$ $\qquad q_k = 2{,}5 \cdot 0{,}65 = 1{,}63 \text{ kN/m}$

$Q_k = 2 \text{ kN}$

ULS CO2

$k_{mod} = 0{,}8$

$p_d = 1{,}20 \cdot 0{,}87 = 1{,}05 \text{ kN/m}$

$q_d = 1{,}5 \cdot 1{,}63 = 2{,}44 \text{ kN/m}$

$M_d = 0{,}125 \cdot (1{,}05 + 2{,}44) \cdot 4{,}64^2 = 9{,}37 \text{ kNm}$

$\sigma_{md} = \dfrac{937}{1344} = 0{,}70 \text{ kN/cm}^2$

$f_{md} = 0{,}8 \cdot \dfrac{2{,}4}{1{,}15} = 1{,}67 \text{ kN/cm}^2$

$\eta = \dfrac{0{,}70}{1{,}67} = 0{,}42 < 1{,}0$

- no lateral torsional buckling, because the sheathing secures the compression flange of the joists

$V_d = 0{,}500 \cdot (1{,}05 + 2{,}44) \cdot 4{,}64 = 8{,}08 \text{ kN}$

a) Frame Construction

$$\tau_d = 1{,}5 \cdot \frac{8{,}08}{0{,}67 \cdot 336} = 0{,}054\ \text{kN/cm}^2$$

$$f_{vd} = 0{,}8 \cdot \frac{0{,}35}{1{,}15} = 0{,}24\ \text{kN/cm}^2$$

$$\eta = \frac{0{,}054}{0{,}24} = 0{,}22 < 1{,}0$$

SLS CO7/8

$$w_{inst,g} = \frac{\frac{0{,}87}{100} \cdot 464^4}{76{,}8 \cdot 1150 \cdot 16128} = 0{,}283\ \text{cm}$$

$$w_{inst,s} = 0$$

$$w_{inst,w} = 0$$

$$w_{inst,q} = 0{,}529\ \text{cm}$$

instantaneous deformation

$$w_{inst} = 0{,}283 + 0{,}529 + 0 + 0 = 0{,}81\ \text{cm}$$

$$\max w_{inst} = \frac{464}{300} = 1{,}55\ \text{cm}$$

$$\eta = \frac{0{,}81}{1{,}55} = 0{,}53 < 1{,}0$$

final deformation

$$w_{fin} = 0{,}283 \cdot (1 + 0{,}6) + 0{,}529 \cdot (1 + 0{,}3 \cdot 0{,}6) + 0 + 0 = 1{,}08\ \text{cm}$$

$$\max w_{fin} = \frac{464}{150} = 3{,}09\ \text{cm}$$

$$\eta = \frac{1{,}08}{3{,}09} = 0{,}35 < 1{,}0$$

max reaction forces

$$A_{gk} = 0{,}500 \cdot 0{,}87 \cdot 4{,}64 = 2{,}02\ \text{kN}$$

$$A_{sk} = 0$$

$$A_{wk} = 0$$

$$A_{qk} = 0{,}500 \cdot 1{,}63 \cdot 4{,}64 = 3{,}77\ \text{kN}$$

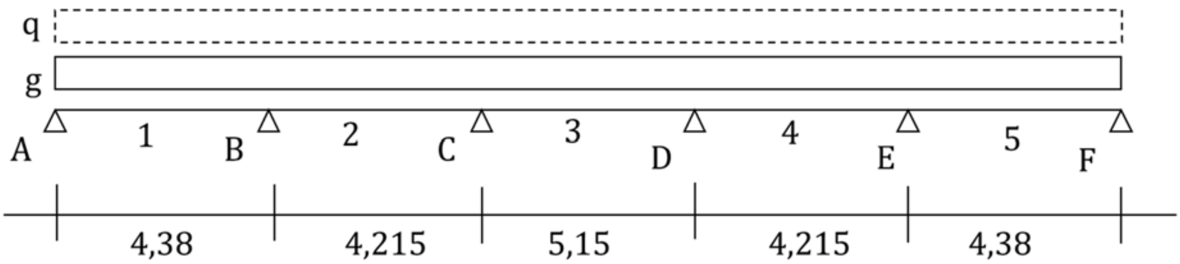
a) Frame Construction

3.2 Joists

The joists 3.2 have the same dimensions and loads as the corresponding joists 3.1, but their span is only 2,10 m. A check is therefore not necessary.

2.2 Roof Beam

system



$\max l = 5{,}15\ \text{m}$

cross-section

$b/h = 16/42$

$A = 672\ \text{cm}^2$

$I_y = 98784\ \text{cm}^4$

$W_y = 4704\ \text{cm}^3$

material

glulam GL24h

$\gamma = 3{,}7\ \text{kN/m}^3$

$f_{mk} = 24\ \text{N/mm}^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ \text{N/mm}^2$

$E_{0,mean} = 11500\ \text{N/mm}^2$

$k_{def} = 0{,}6$

loads

$$g_k = \frac{2{,}49}{0{,}65} + 3{,}7 \cdot 0{,}16 \cdot 0{,}42 = 4{,}08\ \text{kN/m}$$

$$s_k = \frac{5{,}73}{0{,}65} = 8{,}82\ \text{kN/m}$$

$$w_k = \frac{0{,}31}{0{,}65} = 0{,}48\ \text{kN/m}$$

$$q_k = \frac{1{,}13}{0{,}65} = 1{,}74\ \text{kN/m}$$

- reaction force A @ 3.1
- the single forces from the joists are converted into a constant distributed load

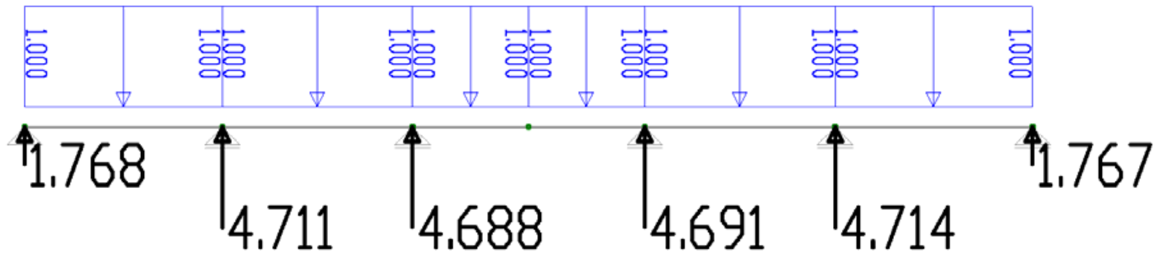
reaction forces, internal forces and deformations

- the reaction forces, internal forces and deformations are calculated by means of dimensionless "1"-loads
- the actual internal forces are then calculated by multiplying the real load with the corresponding result from the "1"-loads

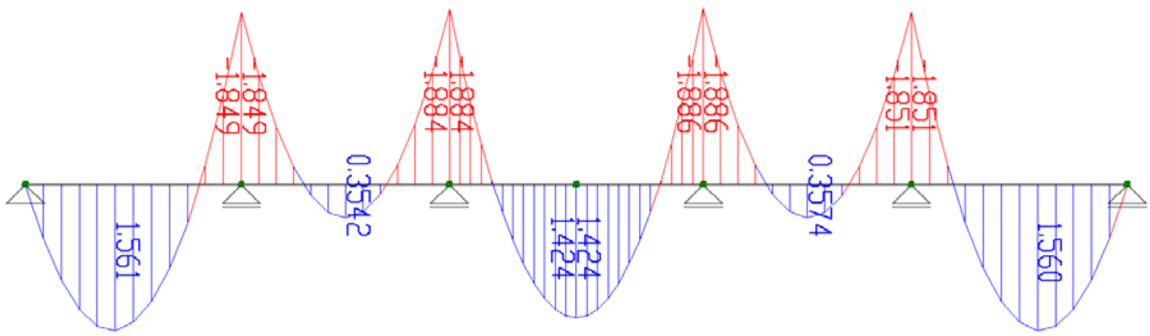
a) Frame Construction

constant load

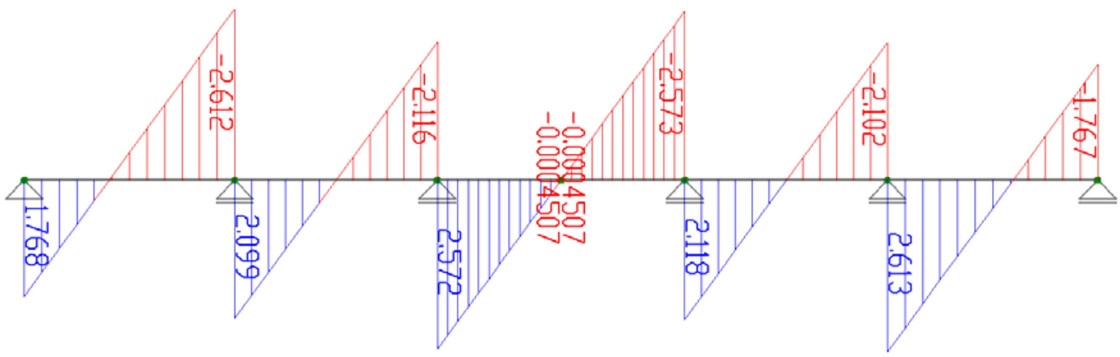
loading and reaction forces



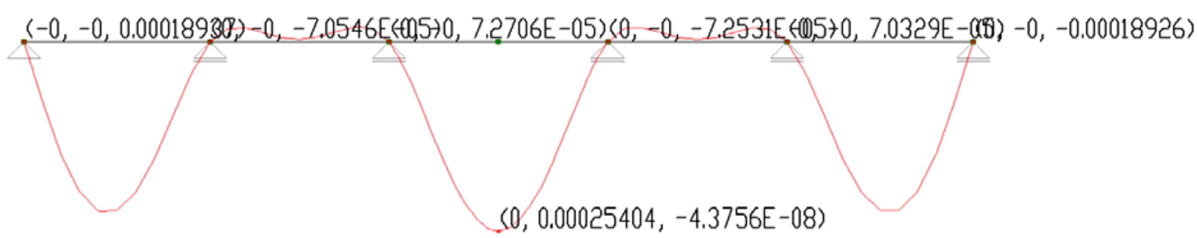
bending moment



shear force



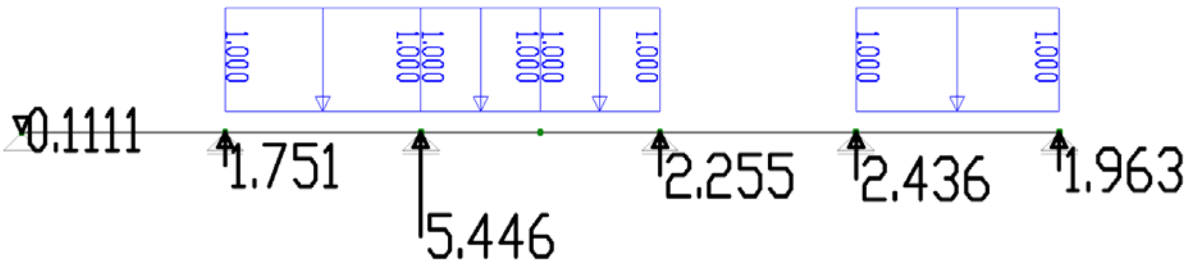
deformations



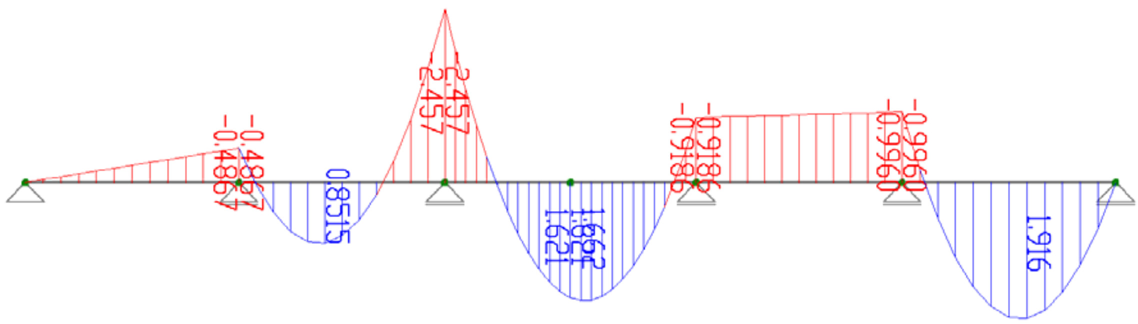
a) Frame Construction

unfavourable loads

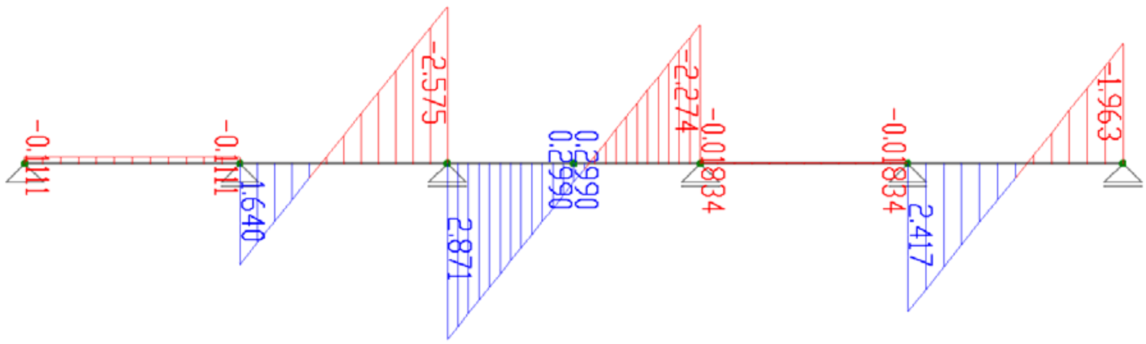
loading and reaction forces



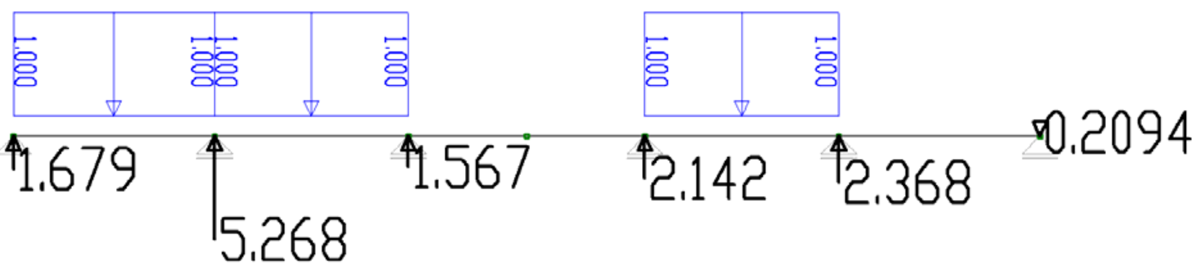
bending moment



shear force

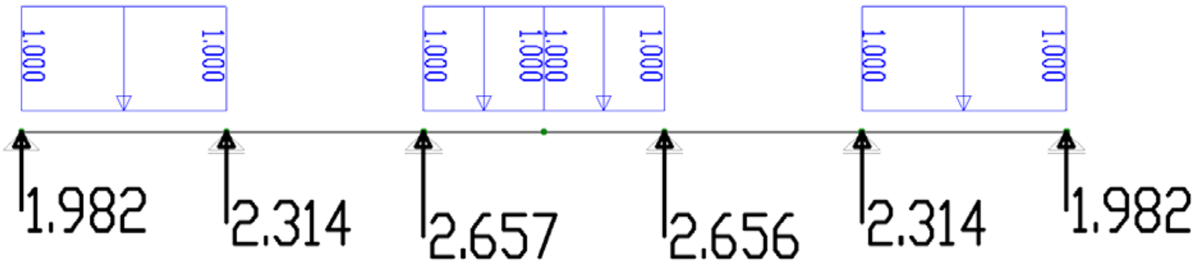


loading and reaction forces

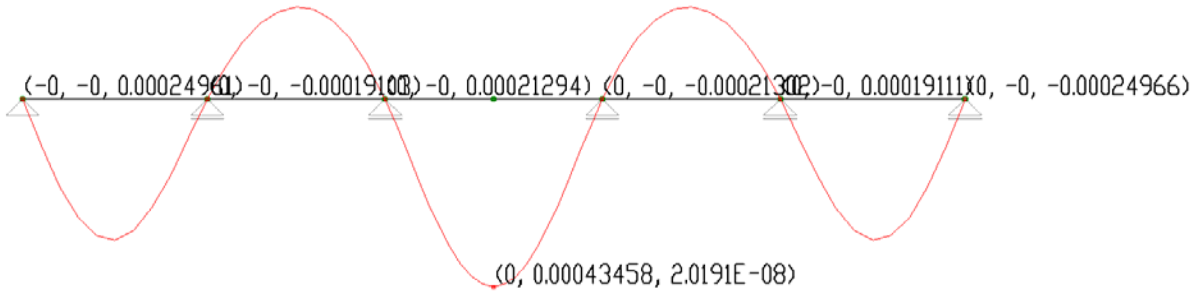


a) Frame Construction

loading and reaction forces



deformations



ULS CO3

$k_{mod} = 0{,}9$

$p_d = 1{,}20 \cdot 4{,}08 + 1{,}50 \cdot 8{,}82 = 18{,}12 \text{ kN/m}$

$q_d = 1{,}5 \cdot 0{,}7 \cdot 1{,}74 = 1{,}83 \text{ kN/m}$

$M_d = 1{,}884 \cdot 18{,}12 + 2{,}457 \cdot 1{,}83 = 38{,}66 \text{ kNm}$

$$\sigma_{md} = \frac{3866}{4704} = 0{,}82 \text{ kN/cm}^2$$

$$f_{md} = 0{,}9 \cdot \frac{2{,}4}{1{,}15} = 1{,}88 \text{ kN/cm}^2$$

$$\eta = \frac{0{,}82}{1{,}88} = 0{,}44 < 1{,}0$$

- no lateral torsional buckling because the joists secure the compression flange of the beam

$V_d = 2{,}572 \cdot 18{,}12 + 2{,}871 \cdot 1{,}83 = 51{,}86 \text{ kN}$

$$\tau_d = 1{,}5 \cdot \frac{51{,}86}{0{,}67 \cdot 672} = 0{,}17 \text{ kN/cm}^2$$

$$f_{vd} = 0{,}9 \cdot \frac{0{,}35}{1{,}15} = 0{,}27 \text{ kN/cm}^2$$

$$\eta = \frac{0{,}17}{0{,}27} = 0{,}64 < 1{,}0$$

SLS CO9/10

$w_{inst,g} = 4{,}08 \cdot 0{,}0254 = 0{,}104 \text{ cm}$

$w_{inst,s} = 8{,}82 \cdot 0{,}0254 = 0{,}224 \text{ cm}$

$w_{inst,w} = 0{,}48 \cdot 0{,}0254 = 0{,}012 \text{ cm}$

a) Frame Construction

$w_{inst,q} = 1{,}74 \cdot 0{,}0435 = 0{,}076$ cm

instantaneous deformation

$$w_{inst} = 0{,}104 + 0{,}224 + 0{,}6 \cdot 0{,}012 + 0{,}7 \cdot 0{,}076 = 0{,}39 \text{ cm}$$

$$\max w_{inst} = \frac{515}{300} = 1{,}72 \text{ cm}$$

$$\eta = \frac{0{,}39}{1{,}72} = 0{,}23 < 1{,}0$$

final deformation

$$w_{fin} = 0{,}104 \cdot (1 + 0{,}6) + 0{,}224 \cdot (1 + 0{,}2 \cdot 0{,}6) + 0{,}012 \cdot (0{,}6 + 0 \cdot 0{,}6) + 0{,}076 \cdot (0{,}7 + 0{,}3 \cdot 0{,}6) = 0{,}49 \text{ cm}$$

$$\max w_{fin} = \frac{515}{150} = 3{,}43 \text{ cm}$$

$$\eta = \frac{0{,}49}{3{,}43} = 0{,}14 < 1{,}0$$

max reaction forces

$A_{gk} = 4{,}08 \cdot 1{,}768 = 7{,}21$ kN

$A_{sk} = 8{,}82 \cdot 1{,}768 = 15{,}59$ kN

$A_{wk} = 0{,}48 \cdot 1{,}768 = 0{,}86$ kN

$A_{qk} = 1{,}74 \cdot 1{,}892 = 3{,}29$ kN

$B_{gk} = 4{,}08 \cdot 4{,}714 = 19{,}22$ kN

$B_{sk} = 8{,}82 \cdot 4{,}714 = 41{,}56$ kN

$B_{wk} = 0{,}48 \cdot 4{,}714 = 2{,}28$ kN

$B_{qk} = 1{,}74 \cdot 5{,}268 = 9{,}17$ kN

$C_{gk} = 4{,}08 \cdot 4{,}691 = 19{,}12$ kN

$C_{sk} = 8{,}82 \cdot 4{,}691 = 41{,}36$ kN

$C_{wk} = 0{,}48 \cdot 4{,}691 = 2{,}27$ kN

$C_{qk} = 1{,}74 \cdot 5{,}446 = 9{,}48$ kN

max internal forces

$V_{Bl,gk} = 2{,}612 \cdot 4{,}08 = 10{,}65$ kN

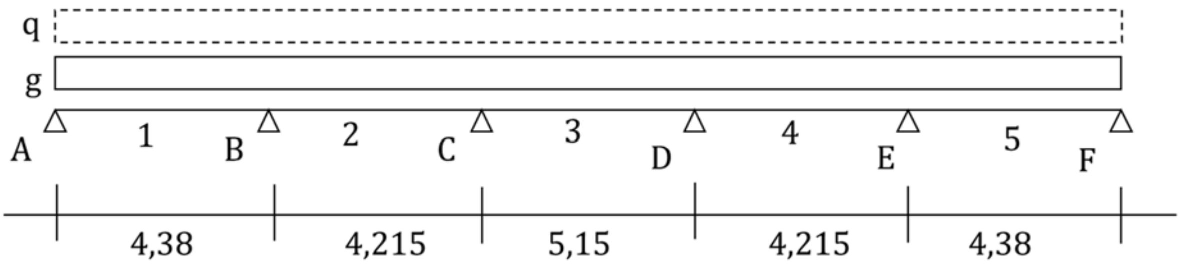
$V_{Bl,sk} = 2{,}612 \cdot 8{,}82 = 23{,}03$ kN

$V_{Bl,wk} = 2{,}612 \cdot 0{,}48 = 1{,}27$ kN

$V_{bl,qk} = 2{,}701 \cdot 1{,}74 = 4{,}70$ kN

2.2 Floor Beam

system



max l = 5,15 m

cross-section

b/h = 16/42

$A = 672\ cm^2$

$I_y = 98784\ cm^4$

$W_y = 4704\ cm^3$

material

glulam GL24h

$\gamma = 3{,}7\ kN/m^3$

$f_{mk} = 24\ N/mm^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ N/mm^2$

$E_{0,mean} = 11500\ N/mm^2$

$k_{def} = 0{,}6$

loads

- reaction force A @ 3.1

$$g_k = \frac{2{,}02}{0{,}65} + 3{,}7 \cdot 0{,}16 \cdot 0{,}42 = 3{,}36\ kN/m$$

$s_k = 0$

$w_k = 0$

$$q_k = \frac{3{,}77}{0{,}65} = 5{,}8\ kN/m$$

internal forces

- cf. 2.2 Roof Beam

ULS CO2

$k_{mod} = 0{,}8$

$p_d = 1{,}20 \cdot 3{,}36 = 4{,}03\ kN/m$

a) Frame Construction

$q_d = 1{,}5 \cdot 5{,}8 = 8{,}7 \text{ kN/m}$

$M_d = 1{,}884 \cdot 4{,}03 + 2{,}457 \cdot 8{,}7 = 29{,}0 \text{ kNm}$

$\sigma_{md} = \frac{2900}{4704} = 0{,}62 \text{ kN/cm}^2$

$f_{md} = 0{,}8 \cdot \frac{2{,}4}{1{,}15} = 1{,}67 \text{ kN/cm}^2$

$\eta = \frac{0{,}62}{1{,}67} = 0{,}37 < 1{,}0$

- no lateral torsional buckling because the joists secure the compression flange of the beam

$V_d = 2{,}572 \cdot 4{,}03 + 2{,}871 \cdot 8{,}7 = 35{,}34 \text{ kN}$

$\tau_d = 1{,}5 \cdot \frac{35{,}34}{0{,}67 \cdot 672} = 0{,}12 \text{ kN/cm}^2$

$f_{vd} = 0{,}8 \cdot \frac{0{,}35}{1{,}15} = 0{,}24 \text{ kN/cm}^2$

$\eta = \frac{0{,}12}{0{,}24} = 0{,}49 < 1{,}0$

SLS CO7/8

$w_{inst,g} = 3{,}36 \cdot 0{,}0254 = 0{,}085 \text{ cm}$

$w_{inst,s} = 0$

$w_{inst,w} = 0$

$w_{inst,q} = 5{,}8 \cdot 0{,}0435 = 0{,}252 \text{ cm}$

instantaneous deformation

$w_{inst} = 0{,}085 + 0{,}252 + 0 + 0 = 0{,}34 \text{ cm}$

$\max w_{inst} = \frac{515}{300} = 1{,}72 \text{ cm}$

$\eta = \frac{0{,}34}{1{,}72} = 0{,}20 < 1{,}0$

final deformation

$$w_{fin} = 0{,}085 \cdot (1 + 0{,}6) + 0{,}252 \cdot (1 + 0{,}3 \cdot 0{,}6) + 0 + 0 = 0{,}434 \text{ cm}$$

$\max w_{fin} = \frac{515}{150} = 3{,}43 \text{ cm}$

$\eta = \frac{0{,}43}{3{,}43} = 0{,}13 < 1{,}0$

max reaction forces

$A_{gk} = 3{,}36 \cdot 1{,}768 = 5{,}94 \text{ kN}$

$A_{sk} = 0$

$A_{wk} = 0$

a) Frame Construction

$A_{qk} = 5{,}8 \cdot 1{,}892 = 10{,}97 \text{ kN}$

$B_{gk} = 3{,}36 \cdot 4{,}714 = 15{,}83 \text{ kN}$

$B_{sk} = 0$

$B_{wk} = 0$

$B_{qk} = 5{,}8 \cdot 5{,}268 = 30{,}55 \text{ kN}$

$C_{gk} = 3{,}36 \cdot 4{,}691 = 15{,}75 \text{ kN}$

$C_{sk} = 0$

$C_{wk} = 0$

$C_{qk} = 5{,}8 \cdot 5{,}446 = 31{,}59 \text{ kN}$

a) Frame Construction

2.1 Roof Beam

- cf. 2.2 Roof Beam

a) Frame Construction

2.1 Floor Beam

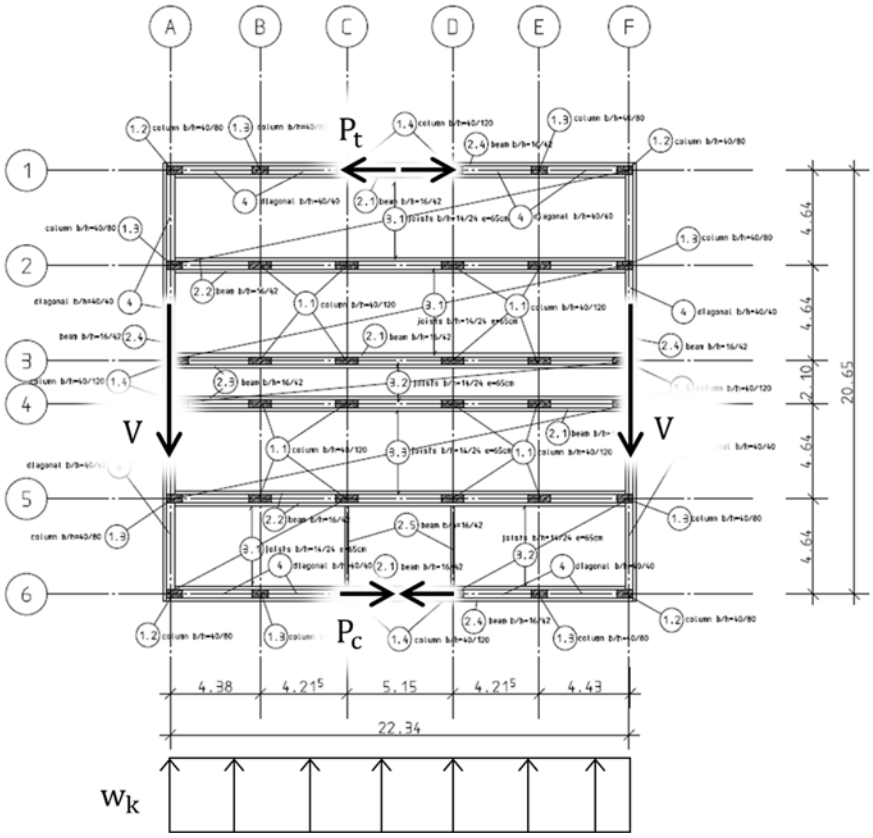
- cf. 2.2 Floor Beam

2.3 Beams

The beams 2.3 have the same dimensions and spans as the corresponding beams 2.2, but their load is smaller because they get the loads from the joists 3.2 which have a smaller span. A check is therefore not necessary.

7 Diaphragm

system



wind load

$\max w_e(D + E) = 1{,}044 \cdot (0{,}8 + 0{,}57)$
$= 1{,}43\ \text{kN/m}^2$

$w_k = 1{,}15 \cdot 1{,}43 \cdot 3{,}195 = 5{,}25\ \text{kN/m}$

- wind from the front becomes decisive
- h = 3,195 m is the height of one storey
- the vertical carriers in the façade (6) in between the beams 2.4 are single-span beams; to account for continuity effects, a factor of 1,15 is added

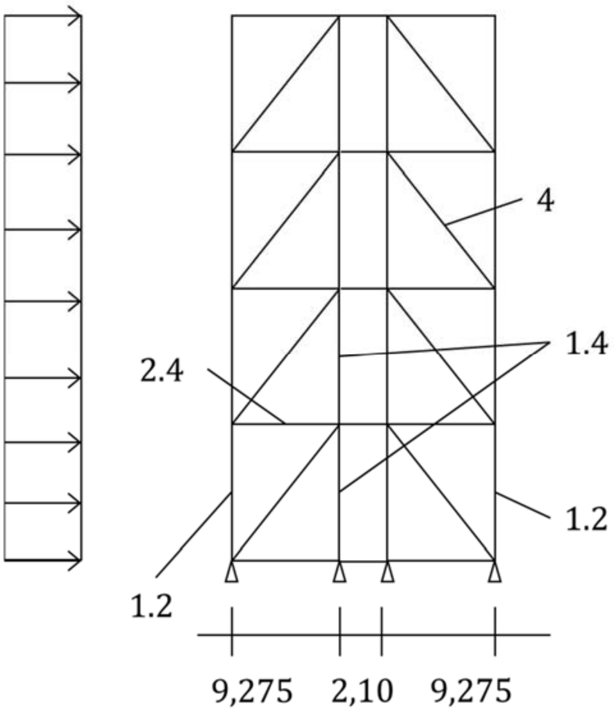
internal forces

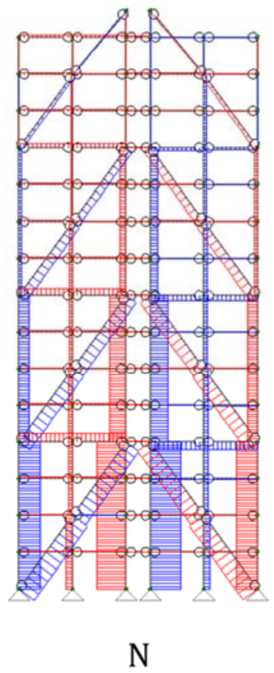
$$|P_t| = |P_c| = \frac{5{,}25 \cdot 22{,}34^2}{8}/20{,}65 = 15{,}9\ \text{kN}$$

$$V = \frac{5{,}25 \cdot 22{,}34}{2} = 58{,}6\ \text{kN}$$

8 Frame

system





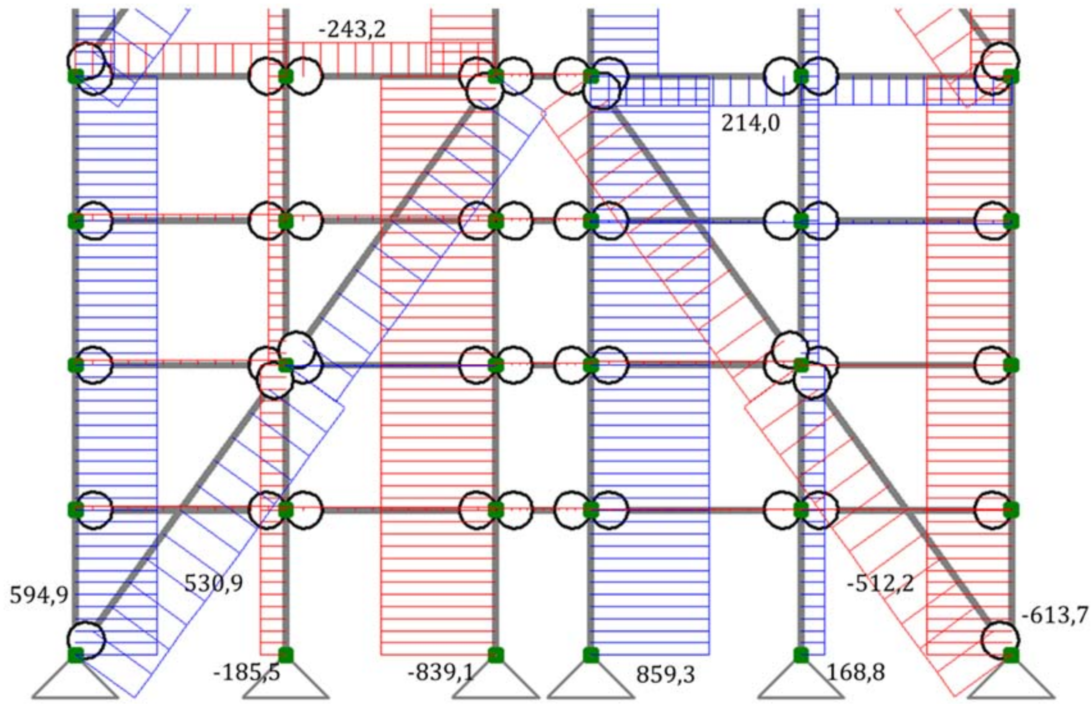
wind load

$\max w_e(D + E) = 1{,}044 \cdot (0{,}8 + 0{,}57)$
$= 1{,}43 \text{ kN/m}^2$

- wind from the front becomes decisive

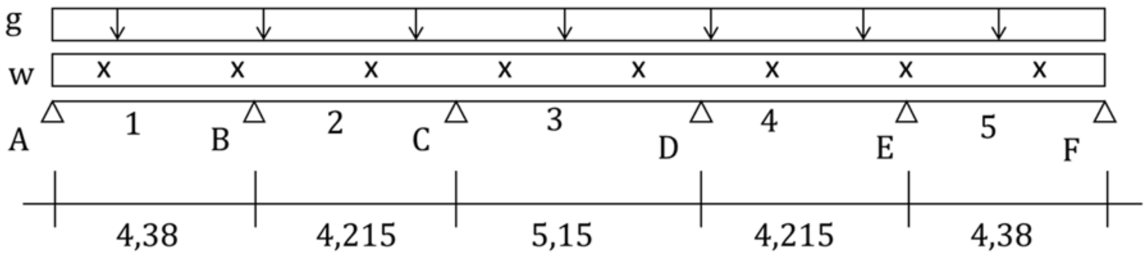
internal forces

normal forces N [kN]



2.4 Beam (front)

system



max l = 5,15 m

cross-section

b/h = 16/42

$A = 672\ cm^2$

$I_y = 98784\ cm^4$

$W_y = 4704\ cm^3$

$i_y = \frac{42}{\sqrt{12}} = 12{,}1\ cm$ $\quad i = \frac{h}{\sqrt{12}}$

$I_z = 14336\ cm^4$

$W_z = 1792\ cm^3$

$i_z = \frac{16}{\sqrt{12}} = 4{,}6\ cm$

$I_{tor} = 0{,}140 \cdot 42 \cdot 16^3 = 24084\ cm^4$ $\quad I_{tor} = 0{,}140 \cdot h \cdot b^3$

material $\quad$ (for rectangular cross-sections)

glulam GL24h

$\gamma = 3{,}7\ kN/m^3$

$f_{mk} = 24\ N/mm^2$

$f_{ck} = 24 N/mm^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ N/mm^2$

$E_{0,mean} = 11500\ N/mm^2$

$E_{0,05} = 9600\ N/mm^2$

$G_{0,05} = 540\ N/mm^2$

$k_{def} = 0{,}6$

a) Frame Construction

loads

$g = 0{,}5\ kN/m^2$

$g_k = 1{,}15 \cdot 3{,}195 \cdot 0{,}5 + 3{,}7 \cdot 0{,}16 \cdot 0{,}42$
$= 2{,}09\ kN/m$

$s_k = 0$

$q_p = 1044\ N/m^2$

- wind from the front

$c_{pe}(D) = 0{,}8$

$w = 0{,}8 \cdot 1{,}044 = 0{,}84\ kN/m^2$

$w_k = 1{,}15 \cdot 3{,}195 \cdot 0{,}84 = 3{,}07\ kN/m^2$

$N_{c,wk} = 15{,}9\ kN$

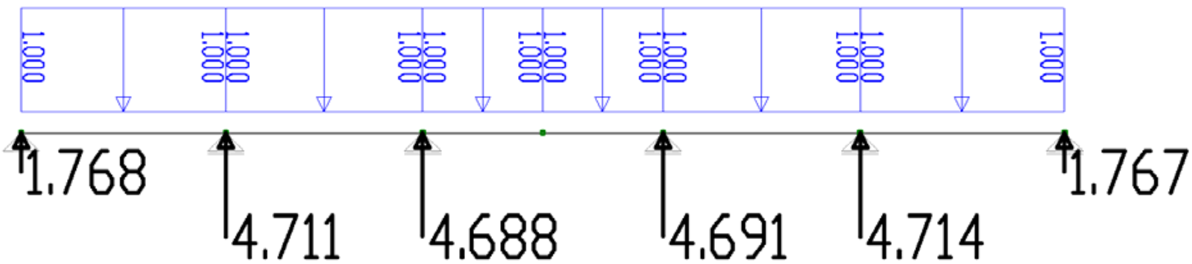
$q_k = 0$

reaction forces, internal forces and deformations

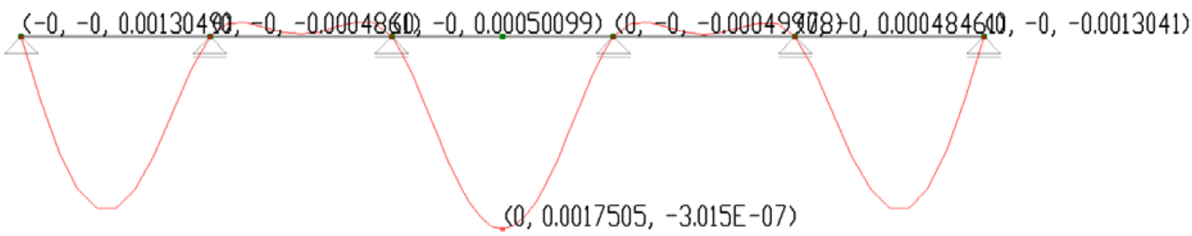
- cf. 2.2 Roof Beam for vertical loads
- because of the smaller moment of inertia for horizontal loads (I_z), the deformations due to horizontal loads must be calculated separately

constant load

loading and reaction forces



deformation



ULS CO5/6

$k_{mod} = 1{,}1$

$p_d = 1{,}20 \cdot 2{,}09 + 0 = 2{,}50\ kN/m$

$w_d = 1{,}50 \cdot 3{,}07 = 4{,}60\ kN/m$

$N_{cd} = 1{,}50 \cdot 15{,}9 = 23{,}85\ kN$

$q_d = 0$

$M_{yd} = 1{,}884 \cdot 2{,}50 + 0 = 4{,}72\ kNm$

a) Frame Construction

$$\sigma_{myd} = \frac{472}{474} = 0{,}10 \text{ kN/cm}^2$$

$$M_{zd} = 1{,}884 \cdot 4{,}60 = 8{,}68 \text{ kNm}$$

$$\sigma_{mzd} = \frac{868}{1792} = 0{,}48 \text{ kN/cm}^2$$

$$\sigma_{cd} = \frac{23{,}85}{672} = 0{,}035 \text{ kN/cm}^2$$

$$f_{md} = 1{,}1 \cdot \frac{2{,}4}{1{,}15} = 2{,}30 \text{ kN/cm}^2$$

$$f_{cd} = 1{,}1 \cdot \frac{2{,}4}{1{,}15} = 2{,}30 \text{ kN/cm}^2$$

lateral torsional buckling

$$l_{ef} < 5{,}15 \text{ m}$$

$$\sigma_{m,crit} = \frac{\pi \cdot \sqrt{960 \cdot 14336 \cdot 54 \cdot 24084}}{515 \cdot 4704} = 5{,}49 \text{ kN/cm}^2$$

$$\lambda_{rel,m} = \sqrt{\frac{2{,}4}{5{,}49}} = 0{,}66 \leq 0{,}75$$

$$k_{crit} = 1{,}0$$

flexural buckling

$$\lambda_y = \frac{515}{12{,}1} = 42{,}6$$

$$\lambda_{rel,y} = \frac{42{,}6}{\pi} \cdot \sqrt{\frac{24}{9600}} = 0{,}677$$

$$k_y = 0{,}5 \cdot (1 + 0{,}1 \cdot (0{,}677 - 0{,}3) + 0{,}677^2) = 0{,}748$$

$$k_{c,y} = \frac{1}{0{,}748 + \sqrt{0{,}748^2 - 0{,}677^2}} = 0{,}938$$

$$\lambda_z = \frac{515}{4{,}6} = 112{,}0$$

$$\lambda_{rel,z} = \frac{112}{\pi} \cdot \sqrt{\frac{24}{9600}} = 1{,}78$$

$$k_z = 0{,}5 \cdot (1 + 0{,}1 \cdot (1{,}78 - 0{,}3) + 1{,}78^2) = 2{,}16$$

$$k_{c,z} = \frac{1}{2{,}16 + \sqrt{2{,}16^2 - 1{,}78^2}} = 0{,}296$$

a) Frame Construction

$$\eta_1 = \frac{0{,}035}{0{,}938 \cdot 2{,}30} + \frac{0{,}10}{1{,}0 \cdot 2{,}30} + \left(\frac{0{,}48}{2{,}30}\right)^2 = 0{,}10 < 1{,}0$$

$$\frac{\sigma_{cd}}{k_{c,y} \cdot f_{cd}} + \frac{\sigma_{myd}}{k_{crit} \cdot f_{md}} + \left(\frac{\sigma_{mzd}}{f_{md}}\right)^2 \leq 1$$

and

$$\eta_2 = \frac{0{,}035}{0{,}296 \cdot 2{,}30} + \left(\frac{0{,}10}{1{,}0 \cdot 2{,}30}\right)^2 + \frac{0{,}48}{2{,}30} = 0{,}27 < 1{,}0$$

$$\frac{\sigma_{cd}}{k_{c,z} \cdot f_{cd}} + \left(\frac{\sigma_{myd}}{k_{crit} \cdot f_{md}}\right)^2 + \frac{\sigma_{mzd}}{f_{md}} \leq 1$$

cf. [2, 9.33]

$$V_{zd} = 2{,}572 \cdot 2{,}50 + 0 = 6{,}54 \text{ kN}$$

$$\tau_{zd} = 1{,}5 \cdot \frac{6{,}54}{0{,}67 \cdot 672} = 0{,}022 \text{ kN/cm}^2$$

$$V_{yd} = 2{,}572 \cdot 4{,}60 = 12{,}02 \text{ kN}$$

$$\tau_{yd} = 1{,}5 \cdot \frac{12{,}02}{0{,}67 \cdot 672} = 0{,}040 \text{ kN/cm}^2$$

$$f_{vd} = 1{,}1 \cdot \frac{0{,}35}{1{,}15} = 0{,}33 \text{ kN/cm}^2$$

$$\eta = \frac{\sqrt{0{,}022^2 + 0{,}040^2}}{0{,}33} = 0{,}14 < 1{,}0$$

SLS CO9/10

$$w_{inst,z,g} = 2{,}09 \cdot 0{,}0254 = 0{,}053 \text{ cm}$$

$$w_{inst,z,s} = 0$$

$$w_{inst,y,w} = 3{,}07 \cdot 0{,}1751 = 0{,}537 \text{ cm}$$

$$w_{inst,z,q} = 0$$

instantaneous deformation

$$w_{z,inst} = 0{,}053 + 0 + 0 = 0{,}053 \text{ cm}$$

$$w_{y.inst} = 0{,}537 \text{ cm}$$

$$w_{inst} = \sqrt{0{,}053^2 + 0{,}537^2} = 0{,}54 \text{ cm}$$

$$\max w_{inst} = \frac{515}{300} = 1{,}72 \text{ cm}$$

$$\eta = \frac{0{,}54}{1{,}72} = 0{,}31 < 1{,}0$$

final deformation

$$w_{z,fin} = 0{,}053 \cdot (1 + 0{,}6) + 0 + 0 = 0{,}085 \text{ cm}$$

$$w_{y,fin} = 0{,}537 \cdot (1 + 0) = 0{,}537 \text{ cm}$$

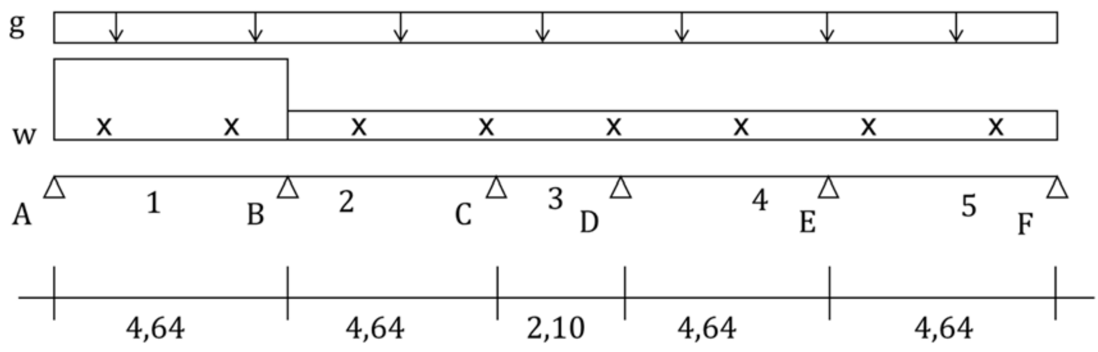
$$w_{fin} = \sqrt{0{,}085^2 + 0{,}537^2} = 0{,}54 \text{ cm}$$

$$\max w_{fin} = \frac{515}{150} = 3{,}43 \text{ cm}$$

$$\eta = \frac{0{,}54}{3{,}43} = 0{,}16 < 1{,}0$$

2.4 Beam (side)

system



max l = 5,15 m

cross-section

b/h = 16/42

$A = 672\ cm^2$

$I_y = 98784\ cm^4$

$W_y = 4704\ cm^3$

$i_y = \frac{42}{\sqrt{12}} = 12{,}1\ cm$

$I_z = 14336\ cm^4$

$W_z = 1792\ cm^3$

$i_z = \frac{16}{\sqrt{12}} = 4{,}6\ cm$

$I_{tor} = 0{,}140 \cdot 42 \cdot 16^3 = 24084\ cm^4$

material

glulam GL24h

$\gamma = 3{,}7\ kN/m^3$

$f_{mk} = 24\ N/mm^2$

$f_{ck} = 24 N/mm^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ N/mm^2$

$E_{0,mean} = 11500\ N/mm^2$

$E_{0,05} = 9600\ N/mm^2$

$G_{0,05} = 540\ N/mm^2$

$k_{def} = 0{,}6$

a) Frame Construction

loads

$g = 0{,}5\ \mathrm{kN/m^2}$

$g_k = 1{,}15 \cdot 3{,}195 \cdot 0{,}5 + 3{,}7 \cdot 0{,}16 \cdot 0{,}42$
$= 2{,}09\ \mathrm{kN/m}$

$s_k = 0$

$q_p = 1044\ \mathrm{N/m^2}$

- wind from the front

$c_{pe}(A) = 1{,}2$

$c_{pe}(B) = 0{,}8$

$w(A) = 1{,}2 \cdot 1{,}044 = 1{,}25\ \mathrm{kN/m^2}$

$w_k(A) = 1{,}15 \cdot 3{,}195 \cdot 1{,}25 = 4{,}60\ \mathrm{kN/m^2}$

$w(B) = 0{,}8 \cdot 1{,}044 = 0{,}84\ \mathrm{kN/m^2}$

$w_k(B) = 1{,}15 \cdot 3{,}195 \cdot 0{,}84 = 3{,}07\ \mathrm{kN/m^2}$

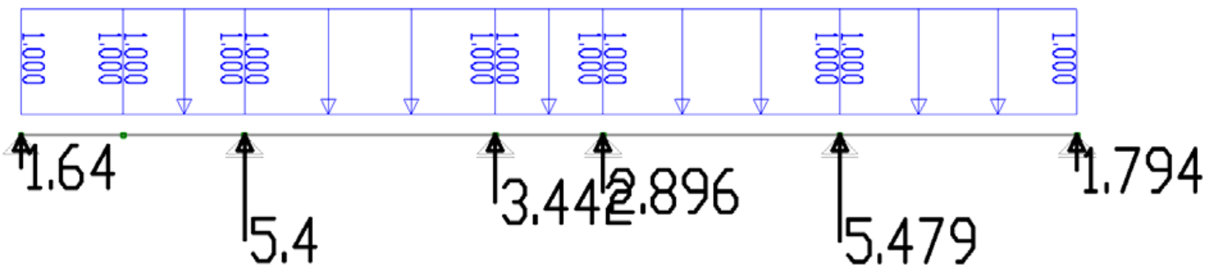
$N_{c,wk} = 243{,}2\ \mathrm{kN}$

$q_k = 0$

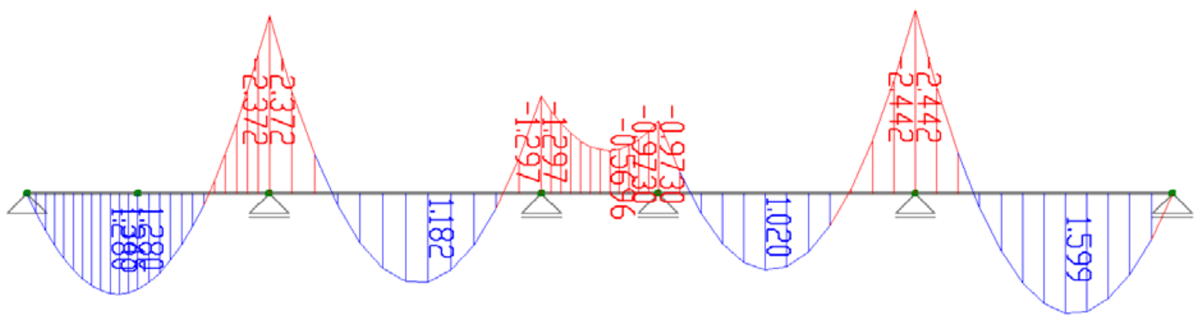
reaction forces, internal forces and deformations

vertical constant load

loading and reaction forces

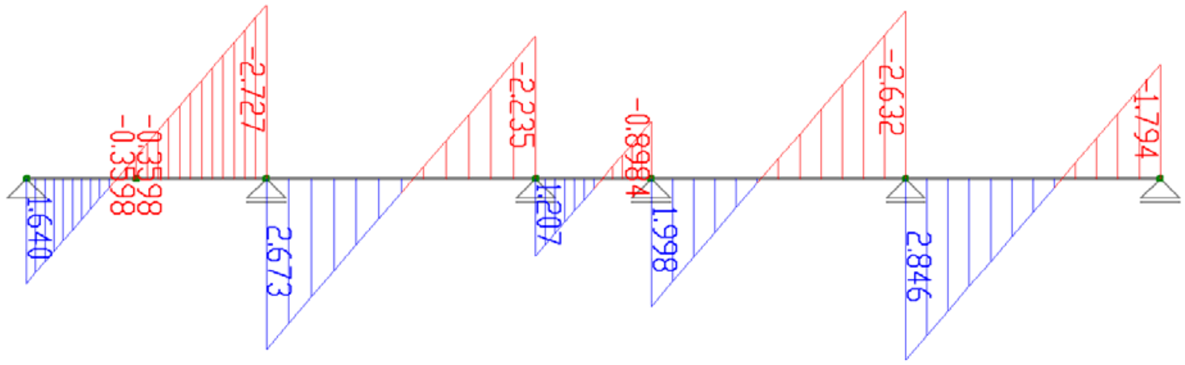


bending moment

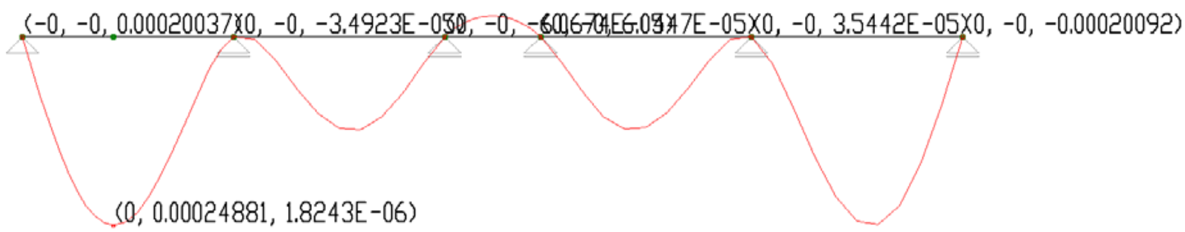


a) Frame Construction

shear force



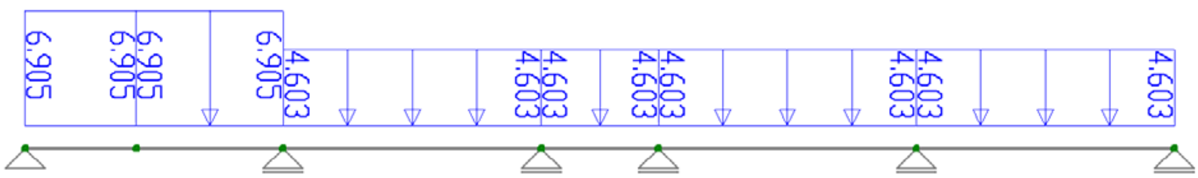
deformations



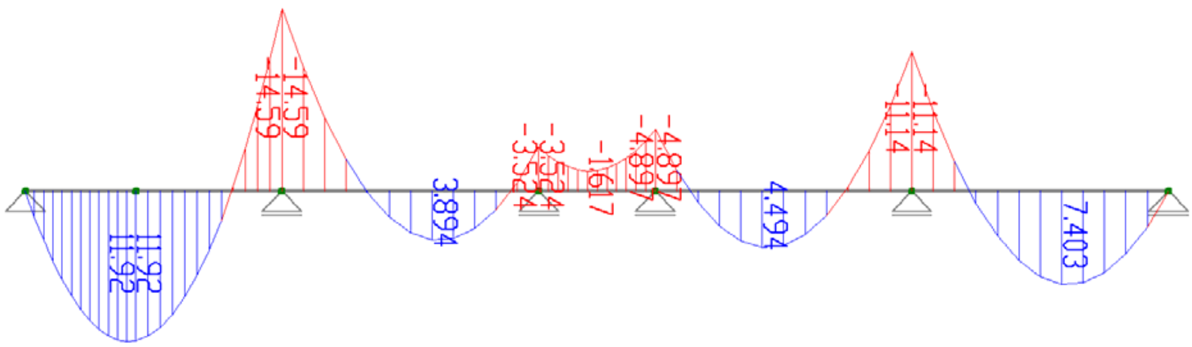
horizontal wind load

- because the wind load is not constant, it must be calculated separately

loading

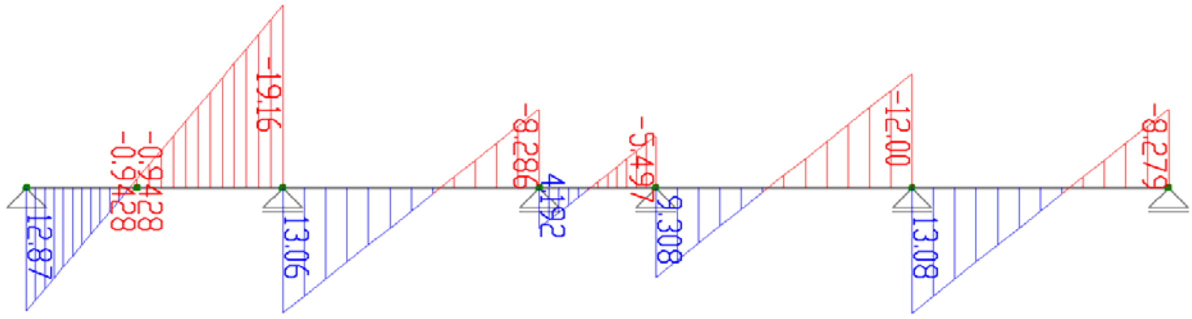


bending moment

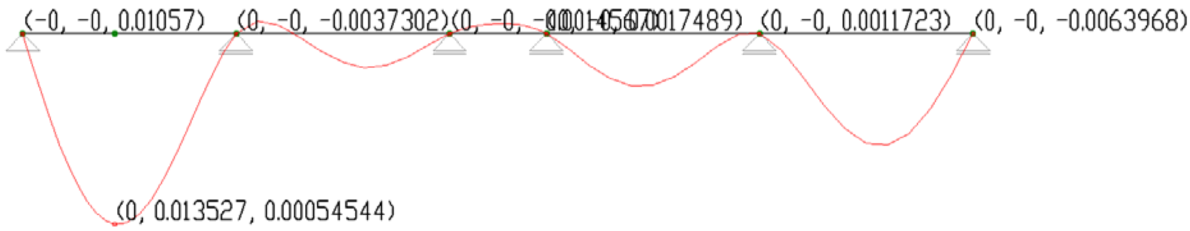


a) Frame Construction

shear force



deformation



ULS CO5/6

$k_{mod} = 1,1$

$p_d = 1,20 \cdot 2,09 + 0 = 2,50\ \text{kN/m}$

$w_d(A) = 1,50 \cdot 4,60 = 6,90\ \text{kN/m}$

$w_d(B) = 1,50 \cdot 3,07 = 4,60\ \text{kN/m}$

$N_{cd} = 1,50 \cdot 243,2 = 364,8\ \text{kN}$

$q_d = 0$

$M_{yd} = 2,442 \cdot 2,50 + 0 = 6,11\ \text{kNm}$

$$\sigma_{myd} = \frac{611}{474} = 0,13\ \text{kN/cm}^2$$

$M_{zd} = 14,59\ \text{kNm}$

$$\sigma_{mzd} = \frac{1459}{1792} = 0,81\ \text{kN/cm}^2$$

$$\sigma_{cd} = \frac{364,8}{672} = 0,54\ \text{kN/cm}^2$$

$$f_{md} = 1,1 \cdot \frac{2,4}{1,15} = 2,30\ \text{kN/cm}^2$$

$$f_{cd} = 1,1 \cdot \frac{2,4}{1,15} = 2,30\ \text{kN/cm}^2$$

lateral torsional buckling

$l_{ef} < 4,64\ \text{m}$

a) Frame Construction

$$\sigma_{m,crit} = \frac{\pi \cdot \sqrt{960 \cdot 14336 \cdot 54 \cdot 24084}}{464 \cdot 4704}$$
$$= 6{,}09 \text{ kN/cm}^2$$

$$\lambda_{rel,m} = \sqrt{\frac{2{,}4}{6{,}09}} = 0{,}63 \leq 0{,}75$$

$$k_{crit} = 1{,}0$$

flexural buckling

$$\lambda_y = \frac{464}{12{,}1} = 38{,}3$$

$$\lambda_{rel,y} = \frac{38{,}3}{\pi} \cdot \sqrt{\frac{24}{9600}} = 0{,}61$$

$$k_y = 0{,}5 \cdot (1 + 0{,}1 \cdot (0{,}61 - 0{,}3) + 0{,}61^2)$$
$$= 0{,}702$$

$$k_{c,y} = \frac{1}{0{,}702 + \sqrt{0{,}702^2 - 0{,}61^2}} = 0{,}953$$

$$\lambda_z = \frac{464}{4{,}6} = 100{,}9$$

$$\lambda_{rel,z} = \frac{100{,}9}{\pi} \cdot \sqrt{\frac{24}{9600}} = 1{,}61$$

$$k_z = 0{,}5 \cdot (1 + 0{,}1 \cdot (1{,}61 - 0{,}3) + 1{,}61^2)$$
$$= 1{,}86$$

$$k_{c,z} = \frac{1}{1{,}86 + \sqrt{1{,}86^2 - 1{,}61^2}} = 0{,}358$$

$$\eta_1 = \frac{0{,}54}{0{,}953 \cdot 2{,}30} + \frac{0{,}13}{1{,}0 \cdot 2{,}30} + \left(\frac{0{,}81}{2{,}30}\right)^2 = 0{,}43$$
$$< 1{,}0$$

$$\eta_2 = \frac{0{,}54}{0{,}358 \cdot 2{,}30} + \left(\frac{0{,}13}{1{,}0 \cdot 2{,}30}\right)^2 + \frac{0{,}81}{2{,}30} = 1{,}01$$
$$\approx 1{,}0$$

$$V_{zd} = 2{,}727 \cdot 2{,}50 + 0 = 6{,}82 \text{ kN}$$

$$\tau_{zd} = 1{,}5 \cdot \frac{6{,}82}{0{,}67 \cdot 672} = 0{,}024 \text{ kN/cm}^2$$

$$V_{yd} = 19{,}16 \text{ kN}$$

$$\tau_{yd} = 1{,}5 \cdot \frac{19{,}16}{0{,}67 \cdot 672} = 0{,}065 \text{ kN/cm}^2$$

$$f_{vd} = 1{,}1 \cdot \frac{0{,}35}{1{,}15} = 0{,}33 \text{ kN/cm}^2$$

a) Frame Construction

$$\eta = \frac{\sqrt{0{,}024^2 + 0{,}065^2}}{0{,}33} = 0{,}21 < 1{,}0$$

SLS CO5/6

$w_{inst,z,g} = 2{,}09 \cdot 0{,}02488 = 0{,}053 \text{ cm}$

$w_{inst,z,s} = 0$

$w_{inst,y,w} = 1{,}353 \text{ cm}$

$w_{inst,z,q} = 0$

instantaneous deformation

$w_{z,inst} = 0{,}053 + 0 + 0 = 0{,}053 \text{ cm}$

$w_{y.inst} = 1{,}353 \text{ cm}$

$w_{inst} = \sqrt{0{,}053^2 + 1{,}353^2} = 1{,}35 \text{ cm}$

$\max w_{inst} = \frac{464}{300} = 1{,}55 \text{ cm}$

$\eta = \frac{1{,}35}{1{,}55} = 0{,}88 < 1{,}0$

final deformation

$w_{z,fin} = 0{,}053 \cdot (1 + 0{,}6) + 0 + 0 = 0{,}085 \text{ cm}$

$w_{y,fin} = 1{,}353 \cdot (1 + 0) = 0{,}537 \text{ cm}$

$w_{fin} = \sqrt{0{,}085^2 + 1{,}353^2} = 1{,}36 \text{ cm}$

$\max w_{fin} = \frac{464}{150} = 3{,}09 \text{ cm}$

$\eta = \frac{1{,}36}{3{,}09} = 0{,}44 < 1{,}0$

max reaction forces

vertical

$B_{gk} = 2{,}09 \cdot 5{,}48 = 11{,}43 \text{ kN}$

$B_{sk} = 0$

$B_{wk} = 0$

$B_{qk} = 0$

$C_{gk} = 2{,}09 \cdot 3{,}442 = 7{,}18 \text{ kN}$

$C_{sk} = 0$

$C_{wk} = 0$

$C_{qk} = 0$

horizontal

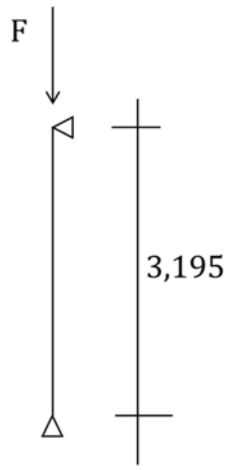
a) Frame Construction

$B_{wk} = 20{,}88\ \mathrm{kN}$

1.1 Column

The column 1.1 carries only the vertical self-weight and life load, but does not take part in carrying the global bending moment from the wind loads. The column in axis B becomes decisive.

system



cross-section

$b/h = 40/40$

$A = 1600\ cm^2$

$I_y = I_z = 213333\ cm^4$

$i_y = i_z = \frac{40}{\sqrt{12}} = 11{,}5\ cm$

material

glulam GL24h

$\gamma = 3{,}7\ kN/m^3$

$f_{ck} = 24\ N/mm^2$

loads

$g_k = 3{,}7 \cdot 0{,}4 \cdot 0{,}4 = 0{,}592\ kN/m$

- self-weight of the column

$N_{gk} = 2 \cdot 19{,}22 + 14 \cdot 2 \cdot 15{,}83 = 481{,}6\ kN$

- reaction force B @ 2.2
- 2 beams are attached to 1 column
- the loads are added up for the roof and 14 floors

$s_k = 0$

$N_{sk} = 2 \cdot 41{,}56 = 83{,}12\ kN$

$w_k = 0$

$N_{wk} = 2 \cdot 2{,}28 = 4{,}57\ kN$

$q_k = 0$

$N_{qk} = 2 \cdot 9{,}17 + 14 \cdot 0{,}74 \cdot 2 \cdot 30{,}55 = 651{,}4\ kN$

ULS CO2

$k_{mod} = 0{,}8$

$g_d = 1{,}2 \cdot 0{,}592 = 0{,}71\ kN/m$

$$N_{cd} = 1{,}2 \cdot 481{,}6 + 0{,}71 \cdot 16 \cdot 3{,}195 + 1{,}5 \cdot 651{,}4 = 1591{,}4\ kN$$

- on the safe side, the self-weight of the column is added up over 16 storeys

$\sigma_{cd} = \frac{1591{,}4}{1600} = 0{,}99\ kN/cm^2$

$f_{cd} = 0{,}8 \cdot \frac{2{,}4}{1{,}15} = 1{,}67\ kN/cm^2$

flexural buckling

$\lambda = \frac{319{,}5}{11{,}5} = 27{,}8$

a) Frame Construction

$$\lambda_{rel} = \frac{27{,}8}{\pi} \cdot \sqrt{\frac{24}{9600}} = 0{,}44$$

$$k = 0{,}5 \cdot (1 + 0{,}1 \cdot (0{,}44 - 0{,}3) + 0{,}44^2)$$
$$= 0{,}604$$

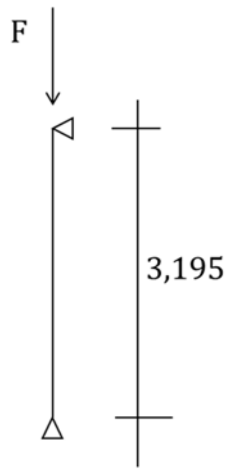
$$k_c = \frac{1}{0{,}604 + \sqrt{0{,}604^2 - 0{,}44^2}} = 0{,}983$$

$$\eta = \frac{0{,}99}{0{,}983 \cdot 1{,}67} = 0{,}61 < 1{,}0$$

1.4 Column

The column 1.4 is part of the frame that carries the global bending moment from the wind loads.

system



cross-section

$b/h = 40/40$

$A = 1600\ cm^2$

$I_y = I_z = 213333\ cm^4$

$i_y = i_z = \frac{40}{\sqrt{12}} = 11{,}5\ cm$

material

glulam GL24h

$\gamma = 3{,}7\ kN/m^3$

$f_{ck} = 24\ N/mm^2$

loads

$g_k = 3{,}7 \cdot 0{,}4 \cdot 0{,}4 = 0{,}592\ kN/m$

$N_{gk} = 19{,}12 + 7{,}18 + 14 \cdot (15{,}75 + 7{,}18)$
$= 347{,}3\ kN$

- reaction force C @ 2.1 and 2.4 (side)
- 1 beam 2.1 and 1 beam 2.4 are attached to the column

$s_k = 0$

$N_{sk} = 41{,}36\ kN$

$w_k = 0$

$N_{wk} = 839{,}1\ kN$

$q_k = 0$

$N_{qk} = 9{,}48 + 14 \cdot 0{,}74 \cdot 31{,}59 = 336{,}7\ kN$

ULS CO5/6

$k_{mod} = 1{,}1$

$g_d = 1{,}2 \cdot 0{,}592 = 0{,}71\ kN/m$

$$N_{cd} = 1{,}2 \cdot 347{,}3 + 0{,}71 \cdot 16 \cdot 3{,}195 + 1{,}5 \cdot 839{,}1 + 1{,}5 \cdot 0{,}7 \cdot 41{,}36 + 1{,}5 \cdot 0{,}7 \cdot 336{,}7 = 2108{,}7\ kN$$

$\sigma_{cd} = \frac{2108{,}7}{1600} = 1{,}32\ kN/cm^2$

$f_{cd} = 1{,}1 \cdot \frac{2{,}4}{1{,}15} = 2{,}30\ kN/cm^2$

a) Frame Construction

flexural buckling

$$\lambda = \frac{319{,}5}{11{,}5} = 27{,}8$$

$$\lambda_{\mathrm{rel}} = \frac{27{,}8}{\pi} \cdot \sqrt{\frac{24}{9600}} = 0{,}44$$

$$\mathrm{k} = 0{,}5 \cdot (1 + 0{,}1 \cdot (0{,}44 - 0{,}3) + 0{,}44^2)$$
$$= 0{,}604$$

$$\mathrm{k_c} = \frac{1}{0{,}604 + \sqrt{0{,}604^2 - 0{,}44^2}} = 0{,}983$$

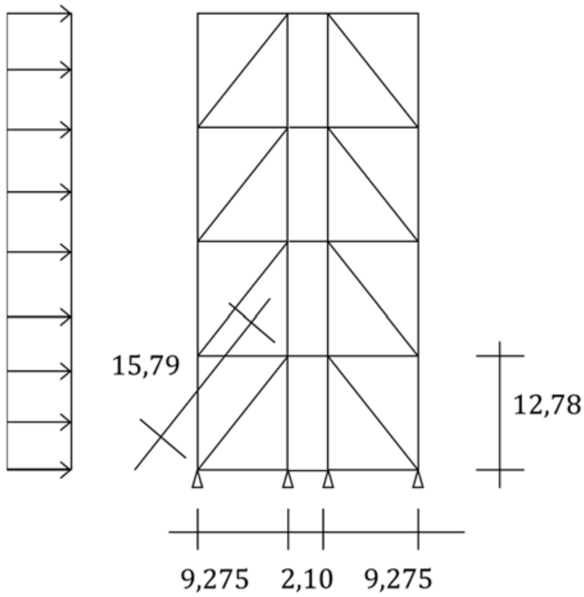
$$\eta = \frac{1{,}32}{0{,}983 \cdot 2{,}30} = 0{,}58 < 1{,}0$$

4 Diagonal

The diagonal is part of the frame that carries the global bending moment from the wind loads. Although the tensile forces in the diagonals are slightly higher, compression becomes decisive because of the flexural buckling.

system

$\max l = \frac{15{,}79}{2} = 7{,}90 \text{ m}$



cross-section

$b/h = 40/40$

$A = 1600 \text{ cm}^2$

$I_y = I_z = 213333 \text{ cm}^4$

$i_y = i_z = \frac{40}{\sqrt{12}} = 11{,}5 \text{ cm}$

material

glulam GL24h

$\gamma = 3{,}7 \text{ kN/m}^3$

$f_{ck} = 24 \text{ N/mm}^2$

- each diagonal consists of two individual members which span two storeys

loads

$g_k = 0$

$s_k = 0$

$w_k = 0$

$N_{wk} = 512{,}2 \text{ kN}$

- see 8 Frame

$q_k = 0$

ULS CO5/6

$k_{mod} = 1{,}1$

$N_{cd} = 1{,}5 \cdot 512{,}2 = 768{,}3 \text{ kN}$

$\sigma_{cd} = \frac{768{,}3}{1600} = 0{,}48 \text{ kN/cm}^2$

$f_{cd} = 1{,}1 \cdot \frac{2{,}4}{1{,}15} = 2{,}30 \text{ kN/cm}^2$

flexural buckling

$\lambda = \frac{790}{11{,}5} = 68{,}7$

$\lambda_{rel} = \frac{68{,}7}{\pi} \cdot \sqrt{\frac{24}{9600}} = 1{,}09$

a) Frame Construction

$$\mathrm{k} = 0{,}5 \cdot (1 + 0{,}1 \cdot (1{,}09 - 0{,}3) + 1{,}09^2)$$
$$= 1{,}14$$

$$\mathrm{k_c} = \frac{1}{1{,}14 + \sqrt{1{,}14^2 - 1{,}09^2}} = 0{,}678$$

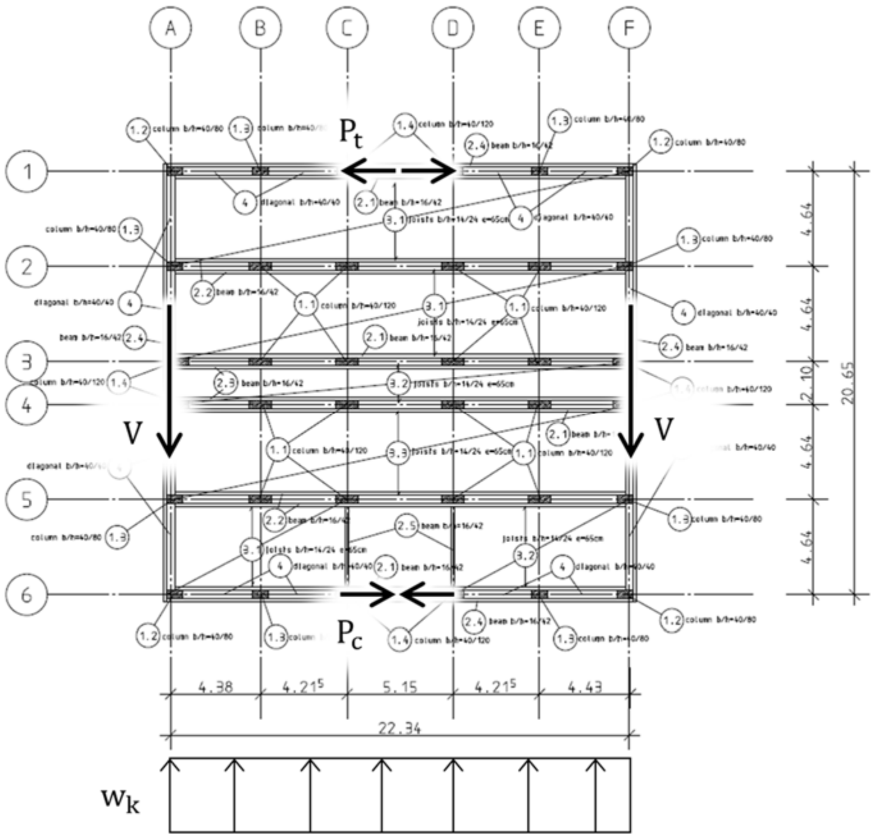
$$\eta = \frac{0{,}48}{0{,}678 \cdot 2{,}30} = 0{,}31 < 1{,}0$$

5 Structural Sheathing of the Floors

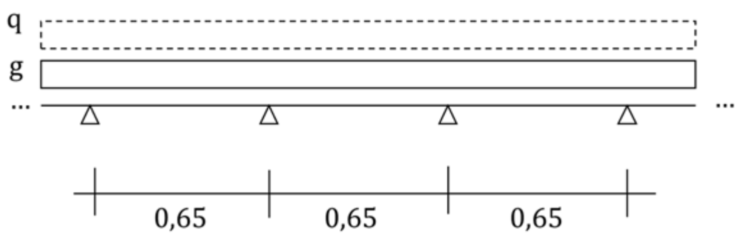
In the roof, the vertical loads are carries by the horizontal carriers and the joists, the sheathing does not carry any loads. The horizontal carriers run perpendicular to the roof joists, so that the loads from the roof do not lie on the sheathing. Only in the floors the sheathing must carry the vertical loads, self-weight and life load, between the joists. Additionally, it also is responsible for the shear stiffness of the diaphragm.

system

global system (diaphragm)



local system



$l = 0{,}65$ m

$t = 18$ mm

$A = 180$ cm/m

$I_y = 48{,}6\ cm^4/m$

$W_y = 54\ cm^3/m$

material

plywood

$m_0 = 1200$ Nmm/mm

$k_{cr} = 1{,}0$

$EI_0 = 4320$ kNmm²/mm

$k_{def} = 0{,}8$

- the material properties are taken from the technical approval [3]

loads

$g_k = 1{,}34$ kN/m²

$s_k = 0$

$w_k = 0$

shear from the diaphragm

$l = 20{,}65 - 6 \cdot 0{,}4 = 18{,}25$ m

$$v = \frac{58{,}6}{18{,}25} = 3{,}21 \text{ kN/m}$$

- the shear force is distributed over the whole length of the building, the columns must be subtracted because they go through the sheathing

$q_k(A) = 2{,}5$ kN/m²

ULS CO2

$k_{mod} = 0{,}8$

$g_d = 1{,}20 \cdot 1{,}34 = 1{,}61$ kN/m²

$q_d = 1{,}5 \cdot 2{,}5 = 3{,}75$ kN/m²

$$M_d < 0{,}125 \cdot (1{,}61 + 3{,}75) \cdot 0{,}65^2 = 0{,}28 \text{ kNm/m}$$

$$M_{Rd} = 0{,}8 \cdot \frac{1200/1000}{1{,}15} = 0{,}83 \text{ kNm/m}$$

$$\eta = \frac{0{,}28}{0{,}83} = 0{,}34 < 1{,}0$$

The shear loads from the diaphragm are not decisive, cf. preliminary design.

SLS CO7/8

$$w_{inst,g} = \frac{\frac{1{,}34}{100^2} \cdot 65^4}{76{,}8 \cdot 4320/10} = 0{,}072 \text{ cm}$$

$w_{inst,s} = 0$

$w_{inst,w} = 0$

$$w_{inst,q} = \frac{\frac{2{,}5}{100^2} \cdot 65^4}{76{,}8 \cdot 4320/10} = 0{,}135 \text{ cm}$$

instantaneous deformation

$w_{inst} = 0{,}072 + 0{,}135 + 0 + 0 = 0{,}21$cm

a) Frame Construction

$$\max w_{inst} = \frac{65}{300} = 0,22 \text{ cm}$$

$$\eta = \frac{0,21}{0,22} = 0,95 < 1,0$$

final deformation

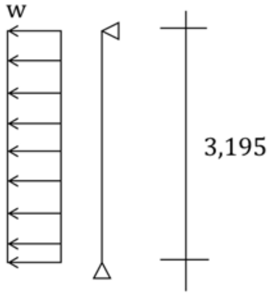
$$w_{fin} = 0,072 \cdot (1 + 0,8) + 0,135 \cdot (1 + 0,3 \cdot 0,8) + 0 + 0 = 0,30 \text{ cm}$$

$$\max w_{fin} = \frac{65}{150} = 0,43 \text{ cm}$$

$$\eta = \frac{0,30}{0,43} = 0,68 < 1,0$$

6 Vertical Façade Carriers

system



$h = 3{,}195$ m

cross-section

$b/h = 6/16$

$e = 60$ cm

$A = 96\ cm^2$

$I_y = 2048\ cm^4$

$W_y = 256\ cm^3$

material

glulam GL24h

$f_{mk} = 24\ N/mm^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ N/mm^2$

$E_{0,mean} = 11500\ N/mm^2$

$k_{def} = 0{,}6$

loads

$g_k = 0$

$q_p = 1044\ N/m^2$

$c_{pe}(A) = 1{,}2$

$w = 1{,}2 \cdot 1{,}044 = 1{,}25\ kN/m^2$

$w_k = 1{,}25 \cdot 0{,}6 = 0{,}75\ kN/m$

ULS CO5/6

$k_{mod} = 1{,}1$

$w_d = 1{,}50 \cdot 0{,}75 = 1{,}13\ kN/m$

$M_d = 0{,}125 \cdot 1{,}13 \cdot 3{,}195^2 = 1{,}44\ kNm$

$$\sigma_{md} = \frac{144}{256} = 0{,}56\ kN/cm^2$$

$$f_{md} = 1{,}1 \cdot \frac{2{,}4}{1{,}15} = 2{,}30\ kN/cm^2$$

$$\eta = \frac{0{,}56}{2{,}30} = 0{,}24 < 1{,}0$$

- no lateral torsional buckling, because the sheathing of the walls secures the carriers from both sides

$V_d = 0{,}500 \cdot 1{,}13 \cdot 3{,}195 = 1{,}80\ kN$

$$\tau_d = 1{,}5 \cdot \frac{1{,}80}{0{,}67 \cdot 96} = 0{,}042\ kN/cm^2$$

$$f_{vd} = 1{,}1 \cdot \frac{0{,}35}{1{,}15} = 0{,}33\ kN/cm^2$$

a) Frame Construction

$$\eta = \frac{0,42}{0,33} = 0,13 < 1,0$$

SLS CO5/6

$$w_{inst,g} = 0$$

$$w_{inst,s} = 0$$

$$w_{inst,w} = \frac{\frac{0,75}{100} \cdot 319,5^4}{76,8 \cdot 1150 \cdot 2048} = 0,433 \text{ cm}$$

$$w_{inst,Q} = 0$$

instantaneous deformation

$$w_{inst} = 0,433 \text{ cm}$$

$$\max w_{inst} = \frac{319,5}{300} = 1,07 \text{ cm}$$

$$\eta = \frac{0,433}{1,07} = 0,41 < 1,0$$

final deformation

$$w_{fin} = 0,433 \cdot (1 + 0 \cdot 0,6) = 0,433 \text{ cm}$$

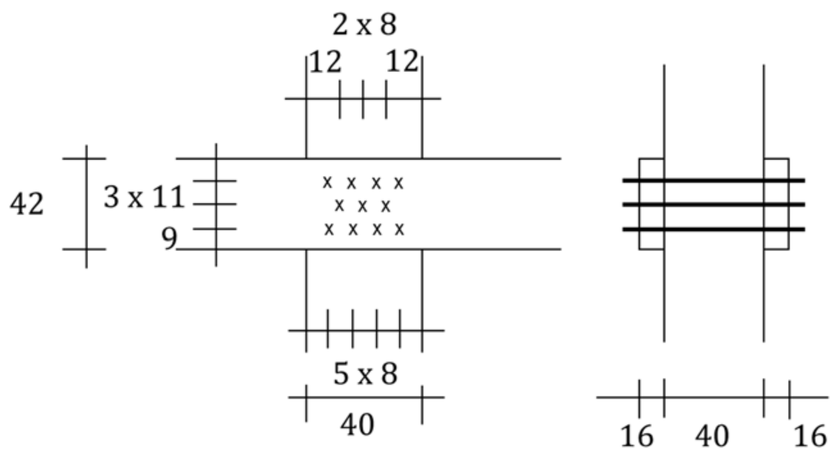
$$\max w_{fin} = \frac{319,5}{150} = 2,13 \text{ cm}$$

$$\eta = \frac{0,433}{2,13} = 0,20 < 1,0$$

Connection Beam 2.2 to Column 1.1

The connection under the roof becomes decisive.

system



angle of the force to the grain

beam: $\alpha = 90°$

column: $\alpha = 0°$

fasteners

11 M20 bolts

$f_{yk} = 240\ \mathrm{N/mm^2}$

- strength class 4.6

$f_{uk} = 400\ \mathrm{N/mm^2}$

material

glulam GL24h

$\rho_k = 385\ \mathrm{kg/m^3}$

minimum distances

beam

$a_1 = (4 + \cos 90) \cdot 2{,}0 = 8\ \mathrm{cm}$

$a_2 = 4 \cdot 2{,}0 = 8\ \mathrm{cm}$

$$a_{4t} = \max\big((2 + 2\sin 90) \cdot 2{,}0 = 8\ ; 3 \cdot 2{,}0 = 6\big)$$
$$= 8\ \mathrm{cm}$$

$a_{4c} = 3 \cdot 2{,}0 = 6\ \mathrm{cm}$

column

$a_1 = (4 + \cos 0) \cdot 2{,}0 = 10\ \mathrm{cm}$

$a_2 = 4 \cdot 2{,}0 = 8\ \mathrm{cm}$

$a_{4c} = 3 \cdot 2{,}0 = 6\ \mathrm{cm}$

capacity per fastener per shear plane

a) Frame Construction

$M_{y,Rk} = 0{,}3 \cdot 400 \cdot 20^{2{,}6}/1000 = 289{,}6 \text{ Nm}$

beam (member 1)

$f_{h,0,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot 20) \cdot 385$
$= 25{,}26 \text{ N/mm}^2$

$k_{90} = 1{,}35 + 0{,}015 \cdot 20 = 1{,}65$

$$f_{h,90,k} = \frac{25{,}26}{1{,}65 \cdot \sin^2 90 + \cos^2 90} = 15{,}3 \text{ N/mm}^2$$

column (member 2)

$f_{h,0,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot 20) \cdot 385$
$= 25{,}26 \text{ N/mm}^2$

$$\beta = \frac{25{,}26}{15{,}3} = 1{,}65$$

$$F_{v,Rk} = 1{,}15 \cdot \sqrt{\frac{2 \cdot 1{,}65}{1 + 1{,}65}} \cdot \sqrt{2 \cdot 289{,}6 \cdot 1000 \cdot 15{,}3 \cdot 20}/1000 = 17{,}1 \text{ kN}$$

- failure mode k becomes decisive
- the axial capacity of the fasteners is neglected

loads

- reaction force B @ Roof Beam 2.2

$F_{gk} = 2 \cdot 19{,}22 = 38{,}4 \text{ kN}$

$F_{sk} = 2 \cdot 41{,}56 = 83{,}1 \text{ kN}$

$F_{wk} = 2 \cdot 2{,}28 = 4{,}6 \text{ kN}$

$F_{qk} = 2 \cdot 9{,}17 = 18{,}3 \text{ kN}$

shear forces in the beam at the connection

$V_{gk} = 2 \cdot 10{,}65 = 21{,}3 \text{ kN}$

$V_{sk} = 2 \cdot 23{,}03 = 46{,}1 \text{ kN}$

$V_{wk} = 2 \cdot 1{,}27 = 2{,}53 \text{ kN}$

$V_{qk} = 2 \cdot 4{,}70 = 9{,}4 \text{ kN}$

ULS CO3

$k_{mod} = 0{,}9$

$F_d = 1{,}2 \cdot 38{,}4 + 1{,}5 \cdot 83{,}1 + 1{,}5 \cdot 0{,}7 \cdot 18{,}3$
$= 190{,}0 \text{ kN}$

$F_{v.Rk} = 17{,}1 \cdot 11 \cdot 2 = 376{,}0$

- 11 bolts and 2 shear planes

$$F_{v,Rd} = 0{,}9 \cdot \frac{376{,}0}{1{,}3} = 260{,}3 \text{ kN}$$

$$\eta = \frac{190{,}0}{260{,}3} = 0{,}73 < 1{,}0$$

a) Frame Construction

check for tension perpendicular to the grain in the beam

$$V_d = 1{,}2 \cdot 21{,}3 + 1{,}5 \cdot 46{,}1 + 1{,}5 \cdot 0{,}7 \cdot 9{,}4$$
$$= 104{,}5 \text{ kN}$$

$$F_{90Rk} = 14 \cdot (2 \cdot 160) \cdot 1 \cdot \sqrt{\frac{330}{1 - \frac{33}{46}}} / 1000$$
$$= 153{,}1 \text{ kN}$$

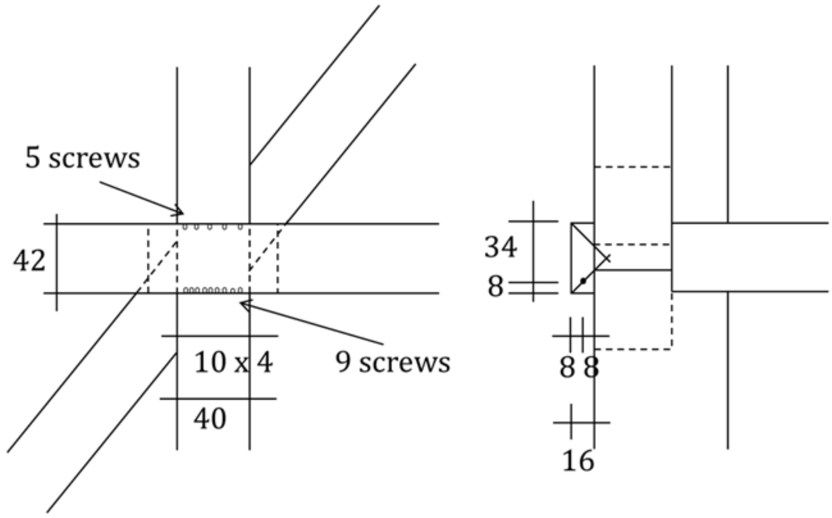
$$F_{90Rd} = 0{,}9 \cdot \frac{153{,}1}{1{,}3} = 106{,}0 \text{ kN}$$

$$\eta = \frac{104{,}5}{106{,}0} = 0{,}99 < 1{,}0$$

a) Frame Construction

Connection Outer Beam 2.4 on the Side to Column 1.3

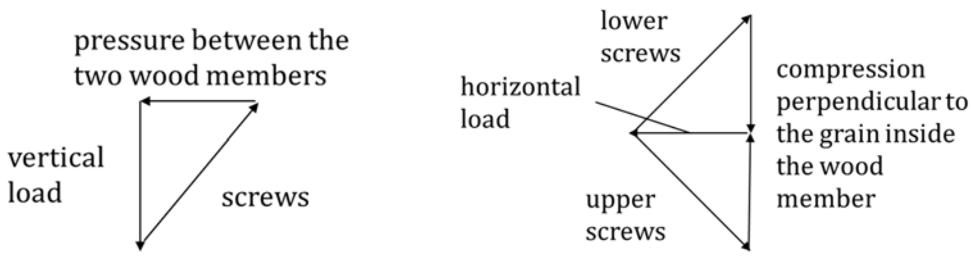
system



angle of the screws

$\alpha = 45°$

principle of the force distribution in the connection



angle of the force to the grain

beam: $\alpha = 90°$

column: $\alpha = 0°$

fasteners

for vertical loads

9 screws Ø8 mm

$n_{ef} = 9^{0,9} = 7{,}22$

for horizontal loads

10 screws Ø8 mm

$n_{ef} = 10^{0,9} = 7{,}94$

- the 9 screws at the bottom that go upwards carry the vertical loads, the 5 upper screws together with 5 of the lower screws carry the horizontal loads

$f_{yk} = 240\ \mathrm{N/mm^2}$

$f_{uk} = 400\ \mathrm{N/mm^2}$

- strength class 4.6

material

glulam GL24h

a) Frame Construction

$\rho_k = 385 \text{ kg/m}^3$

minimum distances

$a_1 = 7 \cdot 0{,}8 = 5{,}6 \text{ cm}$

$a_2 = 5 \cdot 0{,}8 = 4 \text{ cm}$

$a_{2CG} = 4 \cdot 0{,}8 = 3{,}2 \text{ cm}$

capacity of the screws

- fully threaded screws are used, therefore pull-through of the head does not need to be considered

pull-out of the shaft

(not decisive)

tensile capacity of the screws

$f_{tens,k} = f_{yk} = 240 \text{ N/mm}^2$

for vertical loads

$$F_{t,Rk} = 7{,}22 \cdot 240 \cdot \pi \cdot \left(\frac{8}{2}\right)^2 /1000 = 87{,}2 \text{ kN}$$

for horizontal loads

$$F_{t,Rk} = 7{,}94 \cdot 240 \cdot \pi \cdot \left(\frac{8}{2}\right)^2 /1000 = 95{,}8 \text{ kN}$$

loads

- beam 2.4 on the side

vertical

$F_{gk} = 11{,}43 \text{ kN}$

horizontal

$F_{wk} = 20{,}88 \text{ kN}$

internal forces

vertical

$V_{gk} = 5{,}94 \text{ kN}$

horizontal

$V_{wk} = 12{,}15 \text{ kN}$

ULS CO5/6

$k_{mod} = 1{,}1$

vertical

$$F_d = 1{,}2 \cdot \frac{11{,}43}{\sin 45} = 19{,}4 \text{ kN}$$

$$F_{t,Rd} = 1{,}1 \cdot \frac{87{,}2}{1{,}3} = 73{,}7 \text{ kN}$$

$$\eta = \frac{19{,}4}{73{,}7} = 0{,}26$$

a) Frame Construction

horizontal

$$F_d = 1{,}5 \cdot \frac{20{,}88}{\sin 45} = 44{,}3 \text{ kN}$$

$$F_{t,Rd} = 1{,}1 \cdot \frac{95{,}8}{1{,}3} = 81{,}1 \text{ kN}$$

$$\eta = \frac{44{,}3}{81{,}1} = 0{,}55$$

→ $\eta_{tot} = 0{,}26 + 0{,}55 = 0{,}81 < 1{,}0$

check for tension perpendicular to the grain in the beam

vertical

$$V_d = 1{,}2 \cdot 5{,}94 = 7{,}13 \text{ kN}$$

$$F_{90Rk} = 14 \cdot 160 \cdot 1 \cdot \sqrt{\frac{340}{1 - \frac{34}{46}}} / 1000 = 80{,}9 \text{ kN}$$

$$F_{90Rd} = 1{,}1 \cdot \frac{80{,}9}{1{,}3} = 68{,}4 \text{ kN}$$

$$\eta = \frac{7{,}13}{68{,}4} = 0{,}10 < 1{,}0$$

horizontal

$$V_d = 1{,}5 \cdot 12{,}15 = 18{,}23 \text{ kN}$$

$$F_{90Rk} = 14 \cdot 460 \cdot 1 \cdot \sqrt{\frac{80}{1 - \frac{8}{16}}} / 1000 = 81{,}5 \text{ kN}$$

$$F_{90Rd} = 1{,}1 \cdot \frac{81{,}5}{1{,}3} = 68{,}9 \text{ kN}$$

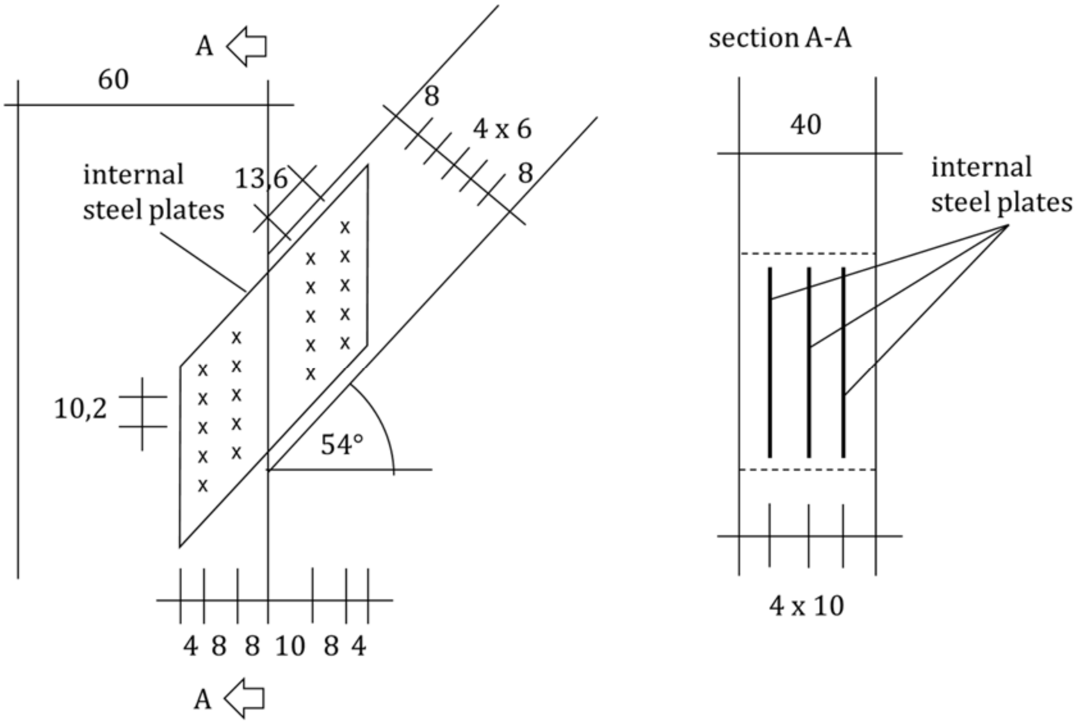
$$\eta = \frac{18{,}23}{68{,}9} = 0{,}26 < 1{,}0$$

→ $\eta_{tot} = 0{,}10 + 0{,}26 = 0{,}37 < 1{,}0$

Connection Diagonal 4 to Column 1.2 or 1.4

The principle that is shown here can be applied to all connections of the diagonal with a column.

system



fasteners

10 M20 dowels

in the diagonal

$\alpha = 0°$

$$n_{ef} = 5 \cdot 2^{0,9} \cdot \sqrt[4]{\frac{136}{13 \cdot 20}} = 7,93$$

in the column

$\alpha = 54°$

$$n_{ef} = 5 \cdot 2^{0,9} \cdot \sqrt[4]{\frac{102}{13 \cdot 20}} = 6,74$$

$f_{yk} = 240\ N/mm^2$

- strength class 4.6

$f_{uk} = 400\ N/mm^2$

material

glulam GL24h

$\rho_k = 385\ kg/m^3$

minimum distances

diagonal

a) Frame Construction

$a_1 = (3 + 2 \cdot \cos 0) \cdot 2{,}0 = 10\ \text{cm}$

$a_2 = 3 \cdot 2{,}0 = 6\ \text{cm}$

$a_{3t} = \max(7 \cdot 2{,}0 = 14\,;8) = 14\ \text{cm}$

$a_{4c} = 3 \cdot 2{,}0 = 6\ \text{cm}$

column

$a_1 = (3 + 2 \cdot \cos 54) \cdot 2{,}0 = 9{,}24\ \text{cm}$

$a_2 = 3 \cdot 2{,}0 = 6\ \text{cm}$

$$a_{4t} = \max\big((2 + 2 \cdot \sin 54) \cdot 2{,}0 = 6{,}51\,; 3 \cdot 2{,}0 = 6\big) = 6{,}51\ \text{cm}$$

capacity per fastener per shear plane

$M_{y,Rk} = 0{,}3 \cdot 400 \cdot 20^{2{,}6}/1000 = 289{,}6\ \text{Nm}$

$$f_{h,0,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot 20) \cdot 385 = 25{,}26\ \text{N/mm}^2$$

$k_{90} = 1{,}35 + 0{,}015 \cdot 20 = 1{,}65$

$$f_{h,54,k} = \frac{25{,}26}{1{,}65 \cdot \sin^2 54 + \cos^2 54} = 20{,}62\ \text{N/mm}^2$$

diagonal

- failure mode g becomes decisive
- dowels have no axial capacity

$$F_{v,Rk} = 25{,}26 \cdot 100 \cdot 20 \cdot \left[\sqrt{2 + \frac{4 \cdot 289{,}6 \cdot 1000}{25{,}26 \cdot 20 \cdot 100^2}} - 1\right]/1000 = 24{,}9\ \text{kN}$$

column

$F_{v,Rk} = 21{,}0\ \text{kN}$

loads

- see 8 Frame

$F_{w,t} = 530{,}9\ \text{kN}$

$V_k = 20{,}2\ \text{kN}$

ULS CO5/6

$k_{mod} = 1{,}1$

$F_d = 1{,}5 \cdot 530{,}9 = 796{,}4\ \text{kN}$

diagonal

- 10 bolts and 6 shear planes

$F_{v.Rk} = 7{,}93 \cdot 6 \cdot 24{,}9 = 1185{,}8\ \text{kN}$

$$F_{v,Rd} = 1{,}1 \cdot \frac{1185{,}8}{1{,}3} = 1003{,}4\ \text{kN}$$

a) Frame Construction

$$\eta = \frac{796{,}4}{1003{,}4} = 0{,}79 < 1{,}0$$

column

$$F_{v.Rk} = 6{,}74 \cdot 6 \cdot 21{,}0 = 850{,}9 \text{ kN}$$

$$F_{v,Rd} = 1{,}1 \cdot \frac{850{,}9}{1{,}3} = 720{,}0 \text{ kN}$$

$$\eta = \frac{796{,}4}{720{,}0} = 1{,}11 > 1{,}0$$

- additional strengthening is required, e. g. by fully threaded screws perpendicular to the grain to increase the capacity of the connection

check for tension perpendicular to the grain in the column

$$V_d = 1{,}5 \cdot 20{,}2 = 30{,}3 \text{ kN}$$

$$F_{90Rk} = 14 \cdot 400 \cdot 1 \cdot \sqrt{\frac{160}{1 - \frac{16}{60}}}/1000$$

$$= 82{,}7 \text{ kN}$$

$$F_{90Rd} = 1{,}1 \cdot \frac{82{,}7}{1{,}3} = 70{,}0 \text{ kN}$$

$$\eta = \frac{30{,}3}{70{,}0} = 0{,}43 < 1{,}0$$

b) Panel Construction

Element Reference Overview

Remarks

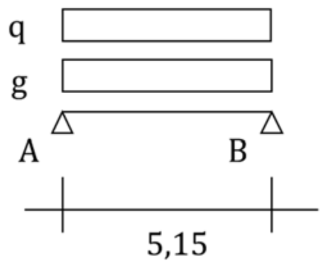
For the calculation of the internal forces in the studs, the FE software RFEM was used.

Compared to the preliminary design, the cross-sections of the joists, the beams and the columns were adapted. Under the roof, larger cross-sections were required, while the cross-sections in the floors could be decreased. The studs were not changed.

b) Panel Construction

1.1 Roof Joists

system



$l = 5{,}15$ m

cross-section

$b/h = 16/24$

$e = 60$ cm

$A = 384\ \text{cm}^2$

$I_y = 18432\ \text{cm}^4$

$W_y = 1536\ \text{cm}^3$

material

glulam GL24h

$f_{mk} = 24\ \text{N/mm}^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ \text{N/mm}^2$

$E_{0,mean} = 11500\ \text{N/mm}^2$

$k_{def} = 0{,}6$

loads

$g = 1{,}69\ \text{kN/m}^2$ $\quad g_k = 1{,}69 \cdot 0{,}6 = 1{,}01\ \text{kN/m}$

$s = 4{,}66\ \text{kN/m}^2$ $\quad s_k = 4{,}66 \cdot 0{,}6 = 2{,}80\ \text{kN/m}$

$q_p = 1044\ \text{N/m}^2$

$c_{pe}(I) = +0{,}2$

$w = 0{,}2 \cdot 1{,}044 = 0{,}21\ \text{kN/m}^2$ $\quad w_k = 0{,}21 \cdot 0{,}6 = 0{,}13\ \text{kN/m}$

$q(H) = 0{,}75\ \text{kN/m}^2$ $\quad q_k = 0{,}75 \cdot 0{,}6 = 0{,}45\ \text{kN/m}$

$Q_k = 1{,}5\ \text{kN}$

ULS CO3

$k_{mod} = 0{,}9$

$p_d = 1{,}20 \cdot 1{,}01 + 1{,}50 \cdot 2{,}80 = 5{,}41\ \text{kN/m}$

$Q_d = 1{,}50 \cdot 0{,}70 \cdot 1{,}5 = 1{,}58\ \text{kN}$

$$M_d = 0{,}125 \cdot 5{,}41 \cdot 5{,}15^2 + 0{,}250 \cdot 1{,}58 \cdot 5{,}15 = 19{,}97\ \text{kNm}$$

$$\sigma_{md} = \frac{1997}{1536} = 1{,}30\ \text{kN/cm}^2$$

$$f_{md} = 0{,}9 \cdot \frac{2{,}4}{1{,}15} = 1{,}88\ \text{kN/cm}^2$$

b) Panel Construction

$$\eta = \frac{1,30}{1,88} = 0,69 < 1,0$$

- no lateral torsional buckling, because the sheathing secures the compression flange of the joists

$$V_d = 0,500 \cdot 5,41 \cdot 4,64 + 1,58 = 15,51 \text{ kN}$$

$$\tau_d = 1,5 \cdot \frac{15,51}{0,67 \cdot 384} = 0,09 \text{ kN/cm}^2$$

$$f_{vd} = 0,9 \cdot \frac{0,35}{1,15} = 0,27 \text{ kN/cm}^2$$

$$\eta = \frac{0,09}{0,27} = 0,33 < 1,0$$

SLS CO9/10

$$w_{inst,g} = \frac{\frac{1,04}{100} \cdot 515^4}{76,8 \cdot 1150 \cdot 18432} = 0,438 \text{ cm}$$

$$w_{inst,s} = 1,208 \text{ cm}$$

$$w_{inst,w} = 0,054 \text{ cm}$$

$$w_{inst,Q} = \frac{1,5 \cdot 515^3}{48 \cdot 1150 \cdot 18432} = 0,201 \text{ cm}$$

instantaneous deformation

$$w_{inst} = 0,438 + 1,208 + 0,6 \cdot 0,054 + 0,7 \cdot 0,201 = 1,82 \text{ cm}$$

$$\max w_{inst} = \frac{515}{300} = 1,72 \text{ cm}$$

$$\eta = \frac{1,82}{1,72} = 1,06 \approx 1,0$$

final deformation

$$w_{fin} = 0,438 \cdot (1 + 0,6) + 1,208 \cdot (1 + 0,2 \cdot 0,6) + 0,054 \cdot (0,6 + 0) + 0,201 \cdot (0,7 + 0,3 \cdot 0,6) = 2,26 \text{ cm}$$

$$\max w_{fin} = \frac{515}{150} = 3,43 \text{ cm}$$

$$\eta = \frac{2,26}{3,43} = 0,66 < 1,0$$

max reaction forces

$$A_{gk} = 0,5 \cdot 1,01 \cdot 5,15 = 2,61 \text{ kN}$$

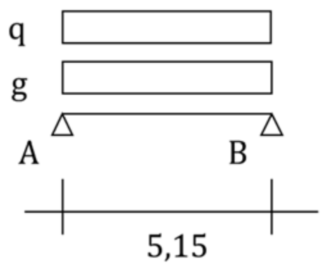
$$A_{sk} = 0,5 \cdot 2,80 \cdot 5,15 = 7,20 \text{ kN}$$

$$A_{wk} = 0,5 \cdot 0,13 \cdot 5,15 = 0,32 \text{ kN}$$

$$A_{qk} = 0,5 \cdot 0,45 \cdot 5,15 = 1,16 \text{ kN}$$

1.1 Floor Joists

system



$l = 5{,}15$ m

cross-section

$b/h = 16/20$

$e = 60$ cm

$A = 320\ \text{cm}^2$

$I_y = 10667\ \text{cm}^4$

$W_y = 1067\ \text{cm}^3$

material

glulam GL24h

$f_{mk} = 24\ \text{N/mm}^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ \text{N/mm}^2$

$E_{0,mean} = 11500\ \text{N/mm}^2$

$k_{def} = 0{,}6$

loads

$g = 1{,}39\ \text{kN/m}^2$ — $g_k = 1{,}39 \cdot 0{,}6 = 0{,}83\ \text{kN/m}$

$s = 0$

$w = 0$

$q(A) = 2{,}5\ \text{kN/m}^2$ — $q_k = 2{,}5 \cdot 0{,}6 = 1{,}5\ \text{kN/m}$

$Q_k = 1{,}5\ \text{kN}$

ULS CO2

$k_{mod} = 0{,}8$

$p_d = 1{,}20 \cdot 0{,}83 + 1{,}50 \cdot 1{,}5 = 3{,}25\ \text{kN/m}$

$M_d = 0{,}125 \cdot 3{,}25 \cdot 5{,}15^2 = 10{,}78\ \text{kNm}$

$$\sigma_{md} = \frac{1078}{1067} = 1{,}01\ \text{kN/cm}^2$$

$$f_{md} = 0{,}8 \cdot \frac{2{,}4}{1{,}15} = 1{,}67\ \text{kN/cm}^2$$

$$\eta = \frac{1{,}01}{1{,}67} = 0{,}61 < 1{,}0$$

- no lateral torsional buckling, because the sheathing secures the compression flange of the joists

$V_d = 0{,}500 \cdot 3{,}25 \cdot 4{,}64 = 8{,}37\ \text{kN}$

$$\tau_d = 1{,}5 \cdot \frac{8{,}37}{0{,}67 \cdot 320} = 0{,}06\ \text{kN/cm}^2$$

b) Panel Construction

$$f_{vd} = 0{,}8 \cdot \frac{0{,}35}{1{,}15} = 0{,}24 \text{ kN/cm}^2$$

$$\eta = \frac{0{,}06}{0{,}24} = 0{,}24 < 1{,}0$$

SLS CO7/8

$$w_{inst,g} = \frac{\frac{0{,}83}{100} \cdot 515^4}{76{,}8 \cdot 1150 \cdot 10667} = 0{,}623 \text{ cm}$$

$$w_{inst,s} = 0$$

$$w_{inst,w} = 0$$

$$w_{inst,q} = 1{,}120 \text{ cm}$$

instantaneous deformation

$$w_{inst} = 0{,}623 + 1{,}120 + 0 + 0 = 1{,}74 \text{ cm}$$

$$\max w_{inst} = \frac{515}{300} = 1{,}72 \text{ cm}$$

$$\eta = \frac{1{,}74}{1{,}72} = 1{,}02 \approx 1{,}0$$

- this is still OK, because amongst others the beneficial effect of the sheathing was not considered yet

final deformation

$$w_{fin} = 0{,}623 \cdot (1 + 0{,}6) + 1{,}120 \cdot (1 + 0{,}3 \cdot 0{,}6) + 0 + 0 = 2{,}32 \text{ cm}$$

$$\max w_{fin} = \frac{515}{150} = 3{,}43 \text{ cm}$$

$$\eta = \frac{2{,}32}{3{,}43} = 0{,}68 < 1{,}0$$

max reaction forces

$$A_{gk} = 0{,}5 \cdot 0{,}83 \cdot 5{,}15 = 2{,}15 \text{ kN}$$

$$A_{sk} = 0$$

$$A_{wk} = 0$$

$$A_{qk} = 0{,}5 \cdot 1{,}5 \cdot 5{,}15 = 3{,}86 \text{ kN}$$

1.2 Roof Joists

The roof joists 1.2 have the same loads and dimensions as the roof joists 1.1 but a smaller span, they are therefore not decisive, and a check is not necessary.

max reaction forces

$A_{gk} = 0{,}5 \cdot 1{,}01 \cdot 4{,}38 = 2{,}22 \text{ kN}$

$A_{sk} = 0{,}5 \cdot 2{,}80 \cdot 4{,}38 = 6{,}12 \text{ kN}$

$A_{wk} = 0{,}5 \cdot 0{,}13 \cdot 4{,}38 = 0{,}27 \text{ kN}$

$A_{qk} = 0{,}5 \cdot 0{,}45 \cdot 4{,}38 = 0{,}99 \text{ kN}$

1.2 Floor Joists

The floor joists 1.2 have the same loads and dimensions as the floor joists 1.1 but a smaller span, they are therefore not decisive, and a check is not necessary.

max reaction forces

$A_{gk} = 0{,}5 \cdot 0{,}83 \cdot 4{,}38 = 1{,}83 \text{ kN}$

$A_{sk} = 0$

$A_{wk} = 0$

$A_{qk} = 0{,}5 \cdot 1{,}5 \cdot 4{,}38 = 3{,}29 \text{ kN}$

b) Panel Construction

1.3 Roof Joists

The dimensions for the roof joists 1.3 could be decreased to $b/h = 8/24$.

b) Panel Construction

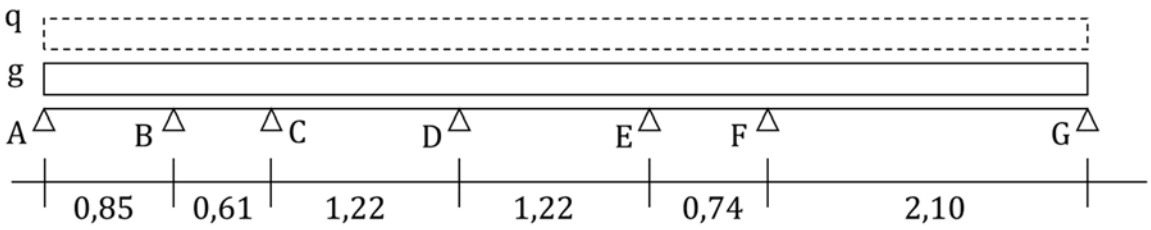
1.3 Floor Joists

The dimensions for the floor joists 1.3 could be decreased to $b/h = 8/20$.

2 Roof Beam

The beam under the roof in axis 3–5/C becomes decisive.

system



max l = 2,10 m

cross-section

b/h = 12/24
$A = 288\ cm^2$
$I_y = 13824\ cm^4$
$W_y = 1152\ cm^3$
material

glulam GL24h
$f_{mk} = 24\ N/mm^2$
$k_{cr} = 0{,}67$
$f_{vk} = 3{,}5\ N/mm^2$
$E_{0,mean} = 11500\ N/mm^2$
$k_{def} = 0{,}6$

loads

- reaction force A @ 1.1

$$g_k = \frac{2{,}61}{0{,}6} = 4{,}35\ kN/m$$

$$s_k = \frac{7{,}20}{0{,}6} = 12{,}0\ kN/m$$

$$w_k = \frac{0{,}32}{0{,}6} = 0{,}54\ kN/m$$

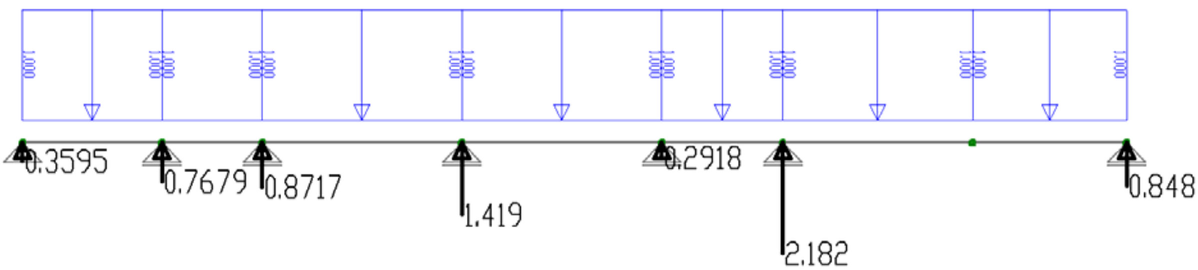
$$q_k = \frac{1{,}16}{0{,}6} = 1{,}93\ kN/m$$

b) Panel Construction

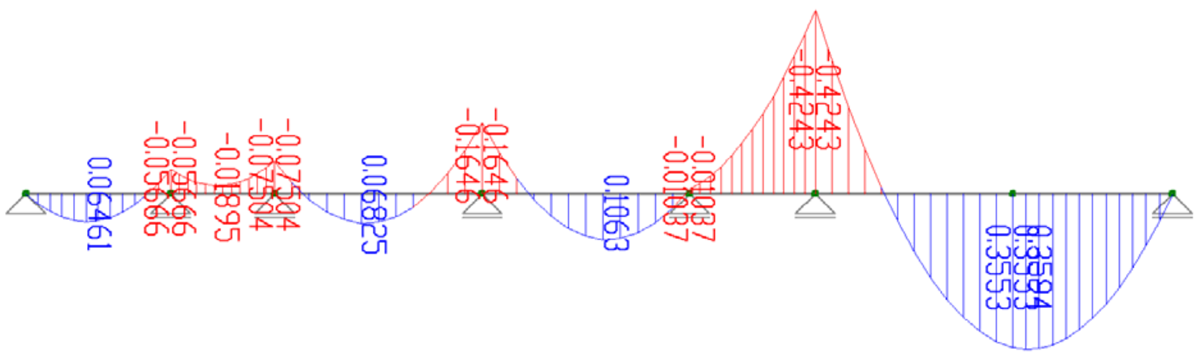
reaction forces, internal forces and deformations

constant load

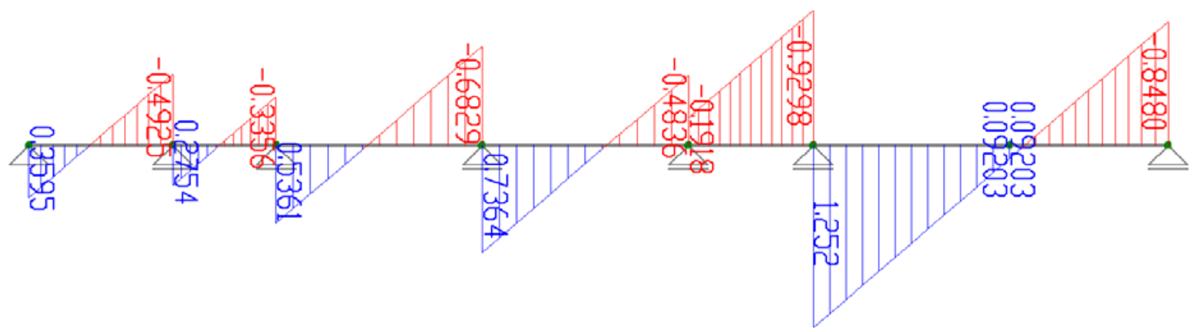
loading and reaction forces



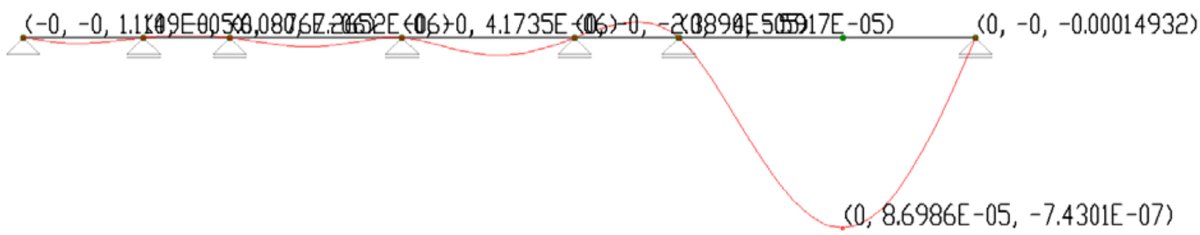
bending moment



shear force



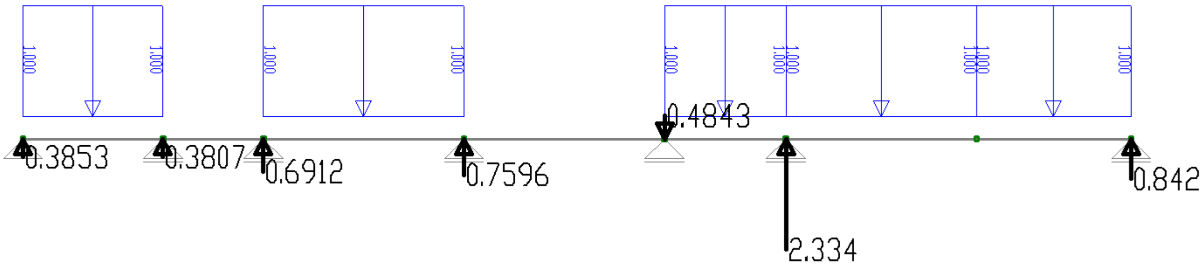
deformations



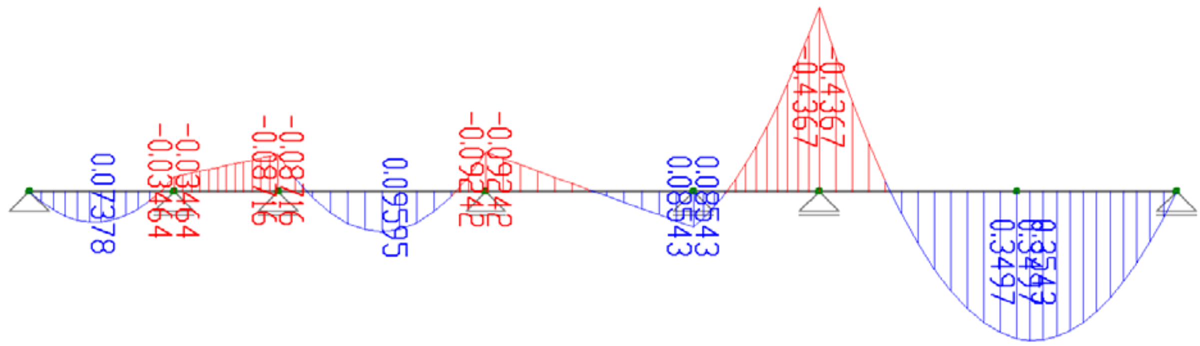
b) Panel Construction

unfavourable loads

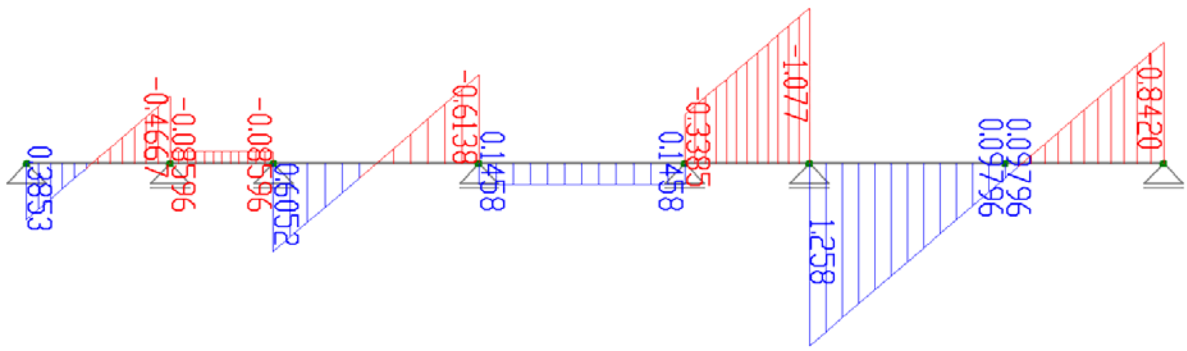
loading and reaction forces



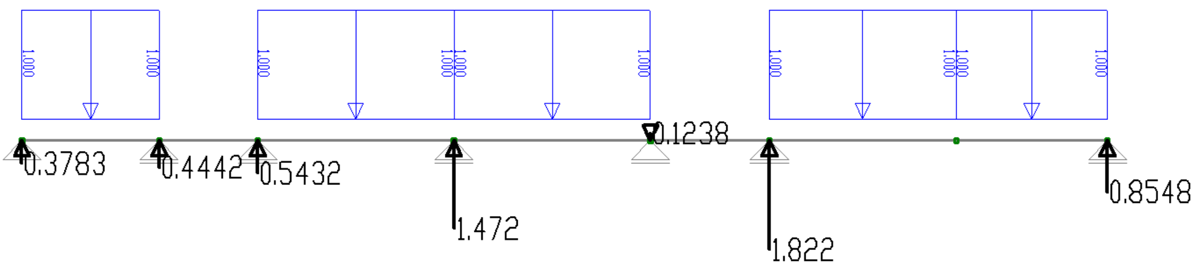
bending moment



shear force

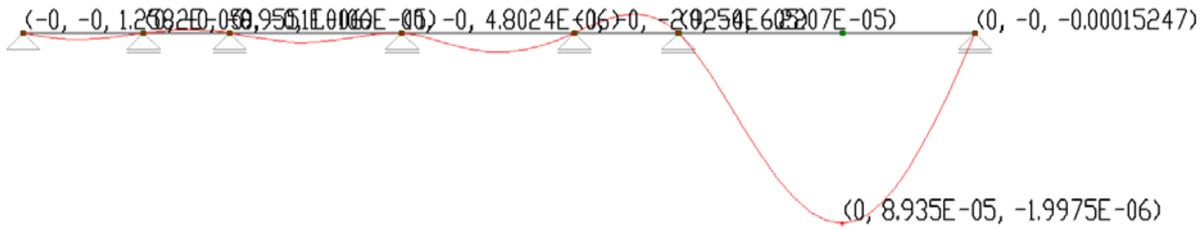


loading and reaction forces



b) Panel Construction

deformations



ULS CO3

$k_{mod} = 0,9$

$p_d = 1,20 \cdot 4,35 + 1,50 \cdot 12,0 = 23,2 \text{ kN/m}$

$q_d = 1,5 \cdot 0,7 \cdot 1,93 = 2,03 \text{ kN/m}$

$$M_d = 23,2 \cdot 0,4243 + 2,03 \cdot 0,4367$$
$$= 10,74 \text{ kNm}$$

$$\sigma_{md} = \frac{1074}{1152} = 0,93 \text{ kN/cm}^2$$

$$f_{md} = 0,9 \cdot \frac{2,4}{1,15} = 1,88 \text{ kN/cm}^2$$

$$\eta = \frac{0,93}{1,88} = 0,50 < 1,0$$

- no lateral torsional buckling because the joists secure the compression flange of the beam

$V_d = 23,2 \cdot 1,252 + 2,03 \cdot 1,258 = 31,6 \text{ kN}$

$$\tau_d = 1,5 \cdot \frac{31,6}{0,67 \cdot 288} = 0,25 \text{ kN/cm}^2$$

$$f_{vd} = 0,9 \cdot \frac{0,35}{1,15} = 0,27 \text{ kN/cm}^2$$

$$\eta = \frac{0,25}{0,27} = 0,90 < 1,0$$

SLS CO9/10

$w_{inst,g} = 4,35 \cdot 0,0087 = 0,038 \text{ cm}$

$w_{inst,s} = 12,0 \cdot 0,0087 = 0,104 \text{ cm}$

$w_{inst,w} = 0,54 \cdot 0,0087 = 0,005 \text{ cm}$

$w_{inst,q} = 1,93 \cdot 0,0089 = 0,017 \text{ cm}$

instantaneous deformation

$$w_{inst}$$
$$= 0,038 + 0,104 + 0,6 \cdot 0,005 + 0,7 \cdot 0,017$$
$$= 0,157$$

$$\max w_{inst} = \frac{210}{300} = 0,70 \text{ cm}$$

$$\eta = \frac{0,157}{0,70} = 0,22 < 1,0$$

b) Panel Construction

final deformation

$$w_{fin} = 0{,}038 \cdot (1 + 0{,}6) + 0{,}104 \cdot (1 + 0{,}2 \cdot 0{,}6) + 0{,}005 \cdot (0{,}6 + 0) + 0{,}017 \cdot (0{,}7 + 0{,}3 \cdot 0{,}6) = 0{,}195 \text{ cm}$$

$$\max w_{fin} = \frac{210}{150} = 1{,}40 \text{ cm}$$

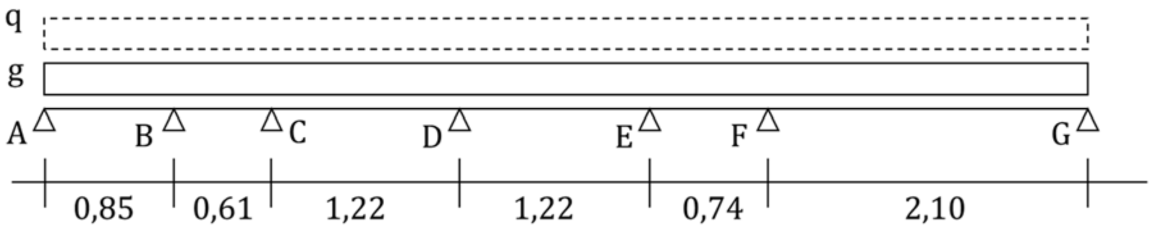
$$\eta = \frac{0{,}195}{1{,}40} = 0{,}14 < 1{,}0$$

b) Panel Construction

2 Floor Beam

The beam in axis 3–5/C becomes decisive.

system



max l = 2,10 m

cross-section

b/h = 8/24
$A = 192\ cm^2$
$I_y = 9216\ cm^4$
$W_y = 768\ cm^3$
material

glulam GL24h
$f_{mk} = 24\ N/mm^2$
$k_{cr} = 0{,}67$
$f_{vk} = 3{,}5\ N/mm^2$
$E_{0,mean} = 11500\ N/mm^2$
$k_{def} = 0{,}6$

loads

- reaction force A @ 1.1

$$g_k = \frac{2{,}15}{0{,}6} = 4{,}35\ kN/m$$

$s_k = 0$

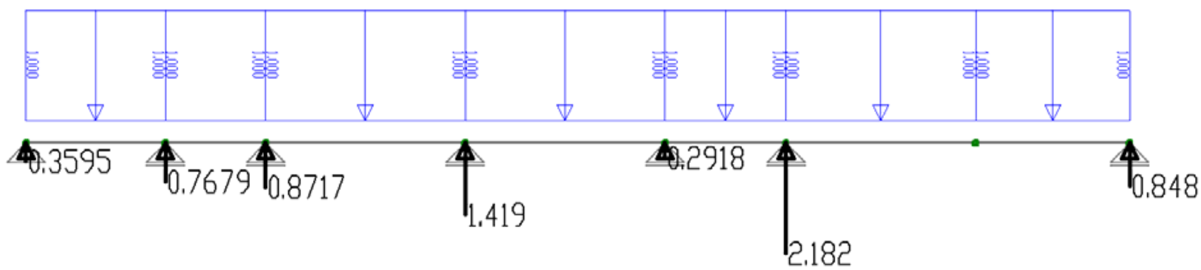
$w_k = 0$

$$q_k = \frac{3{,}86}{0{,}6} = 6{,}44\ kN/m$$

reaction forces, internal forces and deformations

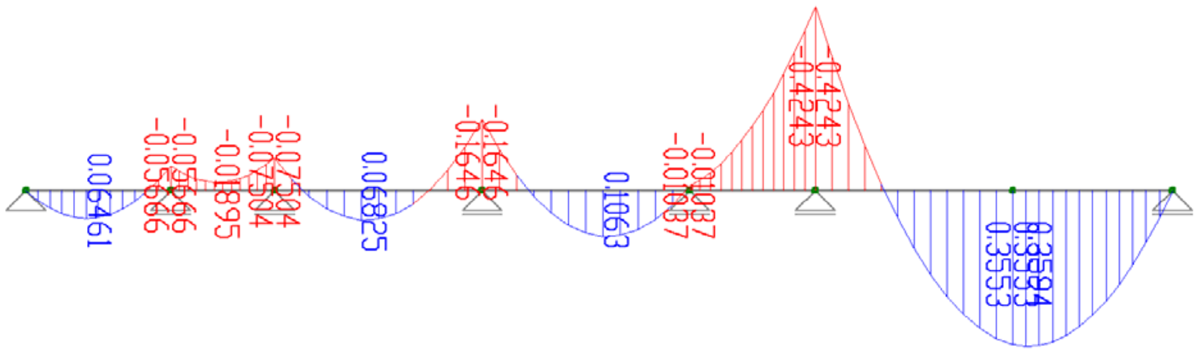
constant load

loading and reaction forces

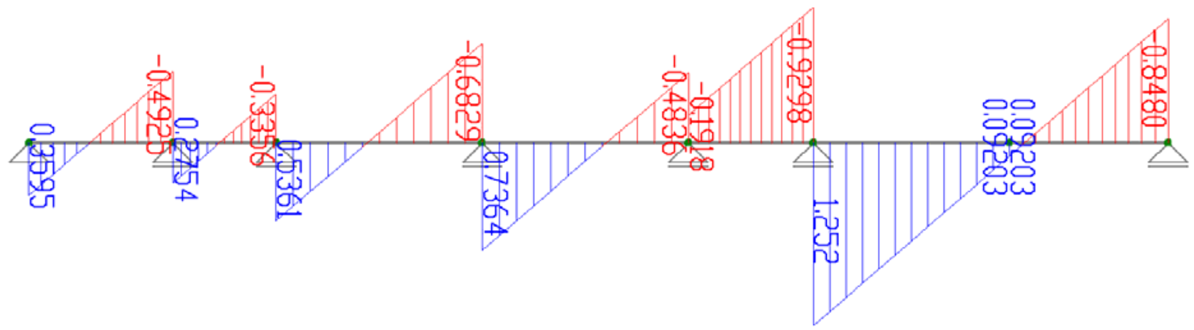


b) Panel Construction

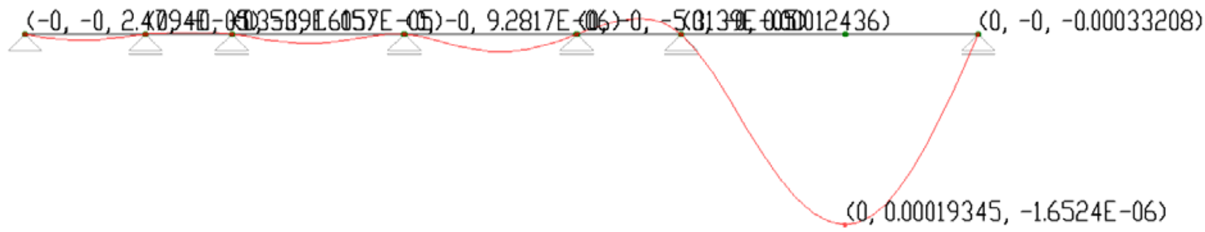
bending moment



shear force

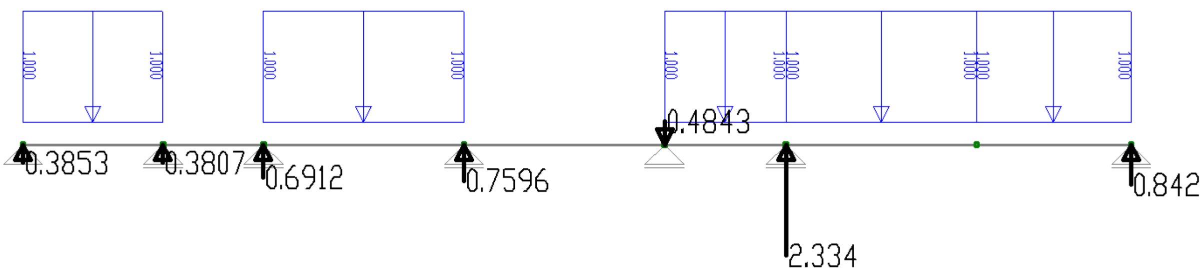


deformations



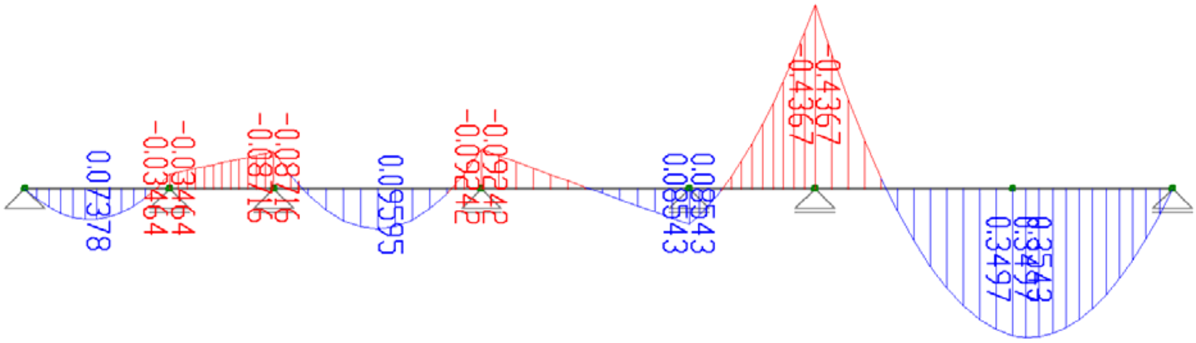
unfavourable loads

loading and reaction forces

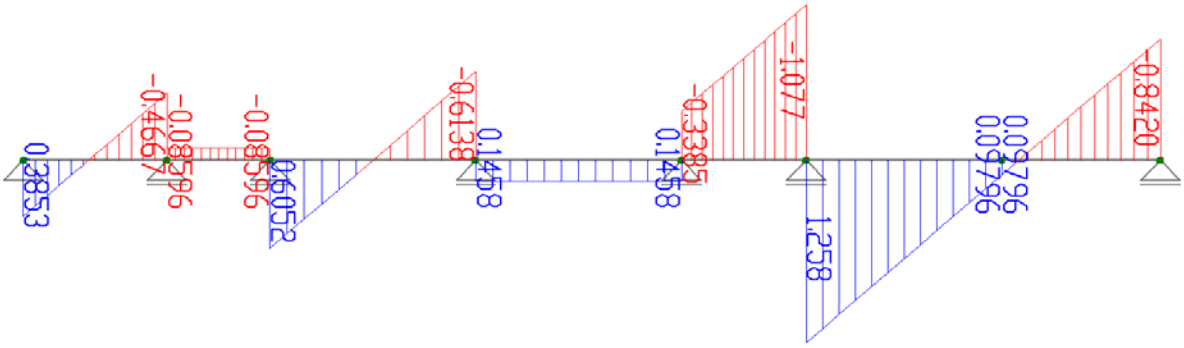


b) Panel Construction

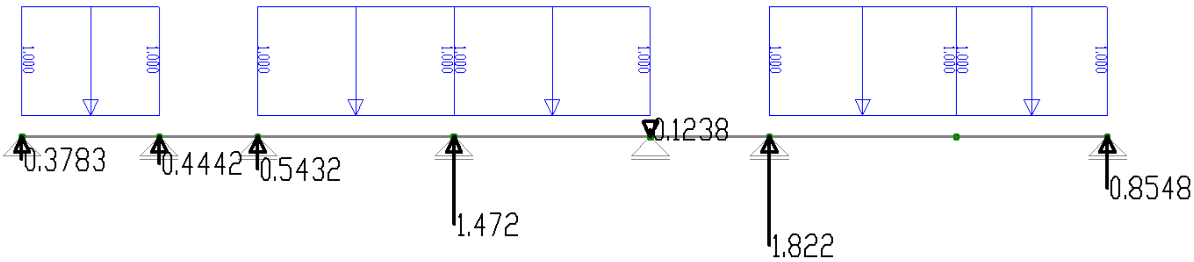
bending moment



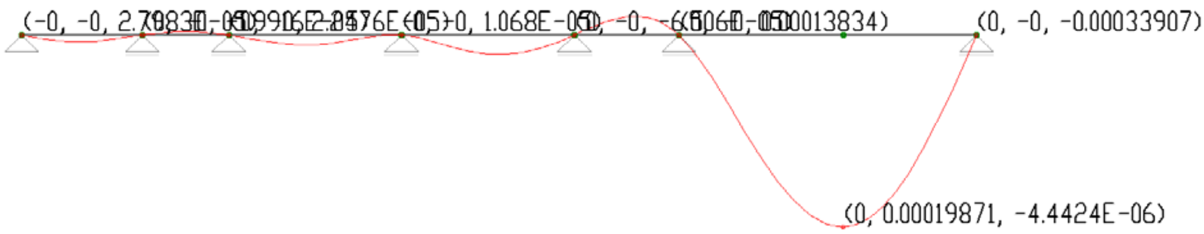
shear force



loading and reaction forces



deformations



ULS CO2

$k_{mod} = 0,8$

$g_d = 1,20 \cdot 3,58 = 4,30\ kN/m$

$q_d = 1,5 \cdot 6,44 = 9,66\ kN/m$

$M_d = 4,30 \cdot 0,4243 + 9,66 \cdot 0,4367 = 6,04\ kNm$

b) Panel Construction

$$\sigma_{md} = \frac{604}{768} = 0{,}79\ \text{kN/cm}^2$$

$$f_{md} = 0{,}8 \cdot \frac{2{,}4}{1{,}15} = 1{,}67\ \text{kN/cm}^2$$

$$\eta = \frac{0{,}79}{1{,}67} = 0{,}47 < 1{,}0$$

- no lateral torsional buckling because the joists secure the compression flange of the beam

$$V_d = 4{,}30 \cdot 1{,}252 + 9{,}66 \cdot 1{,}258 = 17{,}53\ \text{kN}$$

$$\tau_d = 1{,}5 \cdot \frac{17{,}53}{0{,}67 \cdot 192} = 0{,}20\ \text{kN/cm}^2$$

$$f_{vd} = 0{,}8 \cdot \frac{0{,}35}{1{,}15} = 0{,}24\ \text{kN/cm}^2$$

$$\eta = \frac{0{,}20}{0{,}24} = 0{,}84 < 1{,}0$$

SLS CO7/8

$$w_{inst,g} = 3{,}58 \cdot 0{,}01935 = 0{,}069\ \text{cm}$$

$$w_{inst,s} = 0$$

$$w_{inst,w} = 0$$

$$w_{inst,q} = 6{,}44 \cdot 0{,}01987 = 0{,}128\ \text{cm}$$

instantaneous deformation

$$w_{inst} = 0{,}069 + 0{,}128 + 0 + 0 = 0{,}20\ \text{cm}$$

$$\max w_{inst} = \frac{210}{300} = 0{,}70\ \text{cm}$$

$$\eta = \frac{0{,}20}{0{,}70} = 0{,}28 < 1{,}0$$

final deformation

$$w_{fin} = 0{,}069 \cdot (1 + 0{,}6) + 0{,}128 \cdot (1 + 0{,}3 \cdot 0{,}6) + 0 + 0 = 0{,}26\ \text{cm}$$

$$\max w_{fin} = \frac{210}{150} = 1{,}40\ \text{cm}$$

$$\eta = \frac{0{,}26}{1{,}40} = 0{,}19 < 1{,}0$$

The floor beam in axis 1–3/A becomes decisive for the connection to the studs (cf. Connection Floor Beam 2 to Studs 3).

max reaction forces

$$A_{gk} = 3{,}04 \cdot 1{,}817 = 5{,}53\ \text{kN}$$

$$A_{sk} = 0$$

b) Panel Construction

$A_{wk} = 0$

$A_{qk} = 5{,}475 \cdot 1{,}88 = 10{,}29 \text{ kN}$

max internal forces

$V_{Cl,gk} = 0{,}9393 \cdot 3{,}04 = 2{,}86 \text{ kN}$

$V_{Cl,sk} = 0$

$V_{Cl,wk} = 0$

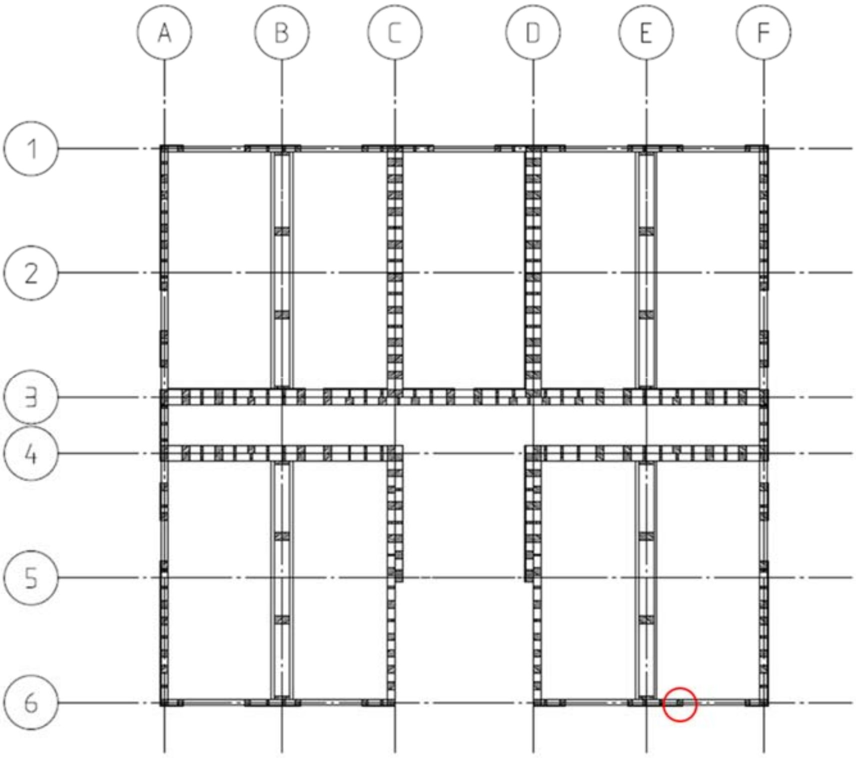
$V_{Cl.qk} = 0{,}9785 \cdot 5{,}475 = 5{,}36 \text{ kN}$

3 Studs

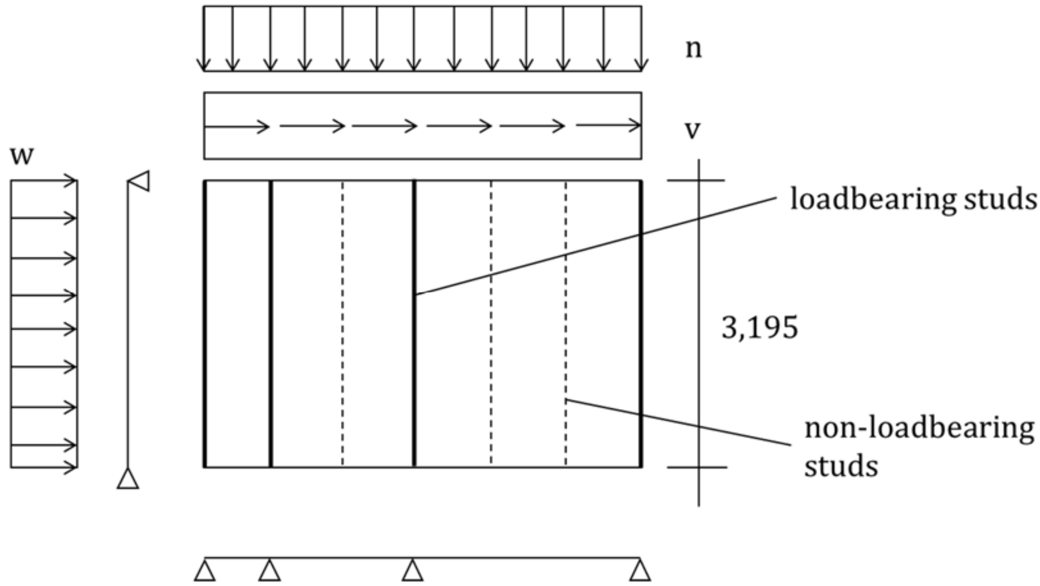
Because the cross-section of the studs decreases in the higher storeys, checks must be performed at every transition, i. e. in the 11., the 6. and the 1. floor. Still, the 1. floor becomes decisive. The internal forces were determined using the FEM software RFEM.

system

decisive stud



general global system (wall)



cross-section

b/h = 20/20

b) Panel Construction

$A = 400\ cm^2$

$I_y = I_z = 13333\ cm^4$

$i_y = i_z = \frac{16}{\sqrt{12}} = 5{,}77\ cm$

material

glulam GL24h

$f_{ck} = 24\ N/mm^2$

ULS CO6

$k_{mod} = 1{,}1$

$N_{cd} = 460{,}8\ kN$

$\sigma_{cd} = \frac{460{,}8}{400} = 1{,}27\ kN/cm^2$

$f_{cd} = 1{,}1 \cdot \frac{2{,}4}{1{,}15} = 2{,}30\ kN/cm^2$

flexural buckling

$\lambda = \frac{319{,}5}{5{,}77} = 55{,}3$

$\lambda_{rel} = \frac{55{,}3}{\pi} \cdot \sqrt{\frac{24}{9600}} = 0{,}88$

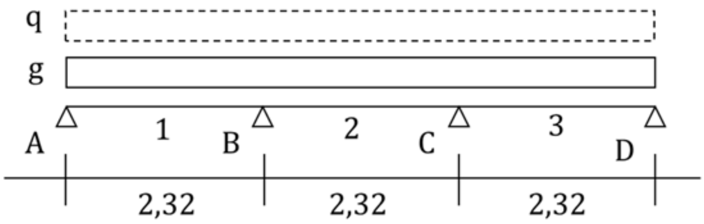
$k = 0{,}5 \cdot (1 + 0{,}1 \cdot (0{,}88 - 0{,}3) + 0{,}88^2)$
$= 0{,}92$

$k_c = \frac{1}{0{,}92 + \sqrt{0{,}92^2 - 0{,}88^2}} = 0{,}853$

$\eta = \frac{1{,}27}{0{,}853 \cdot 2{,}30} = 0{,}67 < 1{,}0$

6 Beam (Roof)

system



max $l = 2{,}32$ m

cross-section

$b/h = 12/24$

$A = 288\ cm^2$

$I_y = 13824\ cm^4$

$W_y = 1152\ cm^3$

material

glulam GL24h

$f_{mk} = 24\ N/mm^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ N/mm^2$

$E_{0,mean} = 11500\ N/mm^2$

$k_{def} = 0{,}6$

loads

- reaction force A @ 1.2

$$g_k = \frac{2{,}22}{0{,}6} = 3{,}70\ kN/m$$

$$s_k = \frac{6{,}12}{0{,}6} = 10{,}21\ kN/m$$

$$w_k = \frac{0{,}27}{0{,}6} = 0{,}46\ kN/m$$

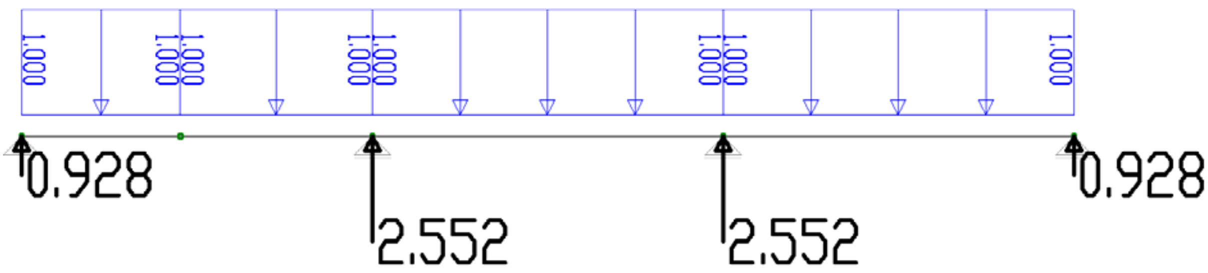
$$q_k = \frac{0{,}99}{0{,}6} = 1{,}64\ kN/m$$

reaction forces, internal forces and deformations

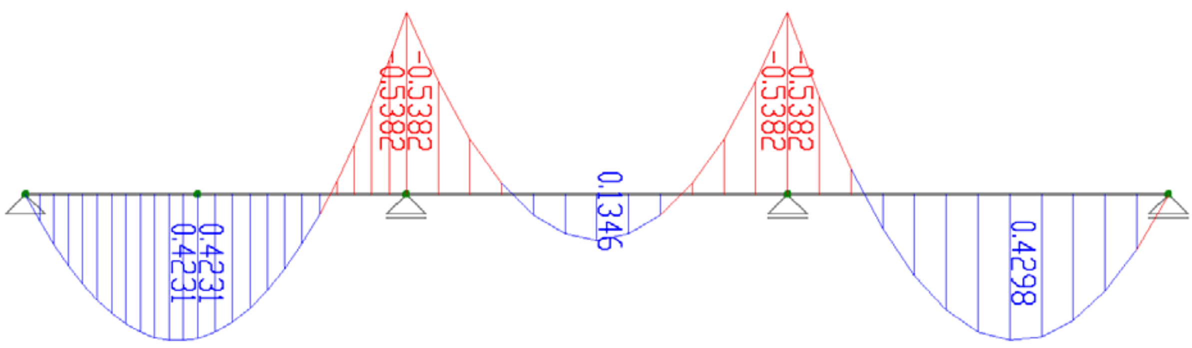
b) Panel Construction

constant load

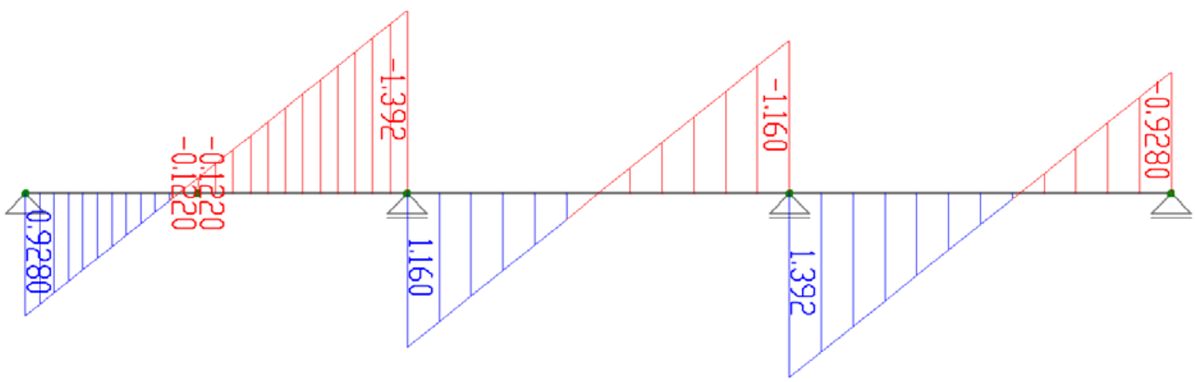
loading and reaction forces



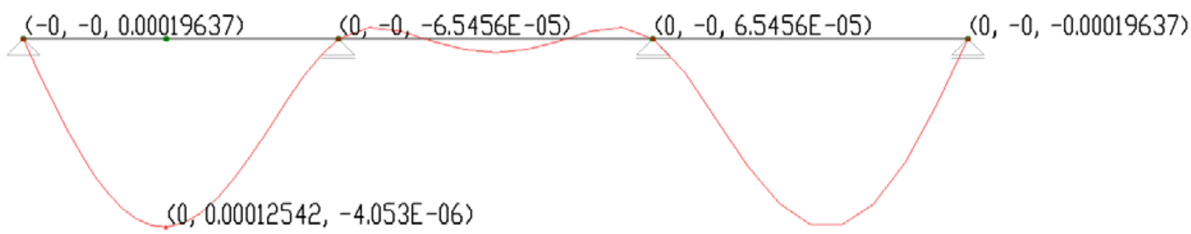
bending moment



shear force



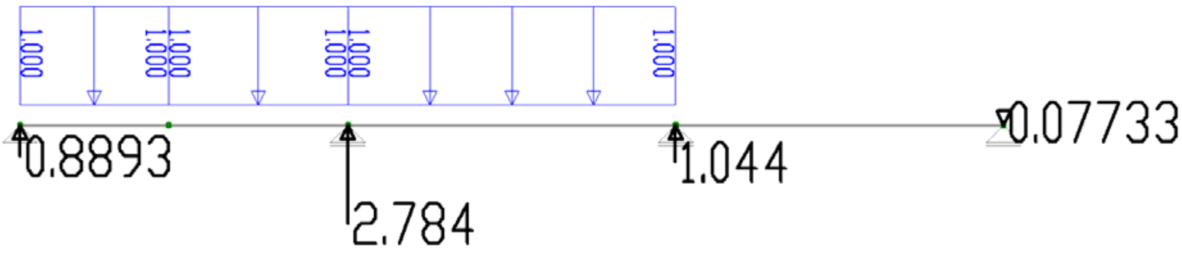
deformations



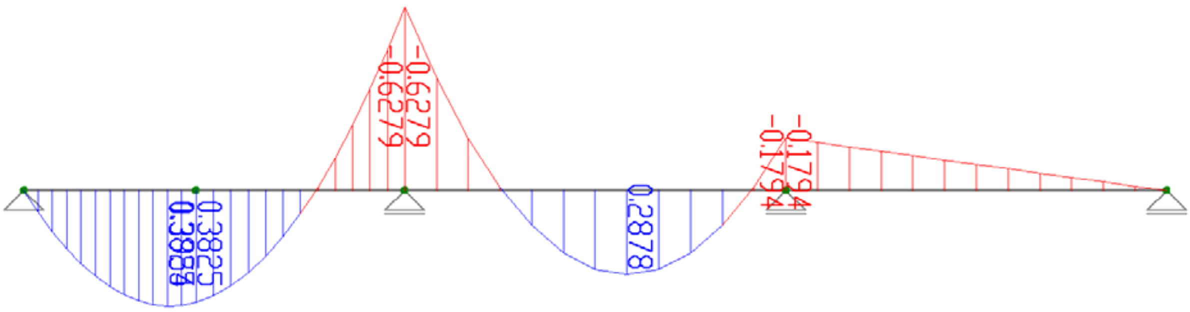
b) Panel Construction

unfavourable loads

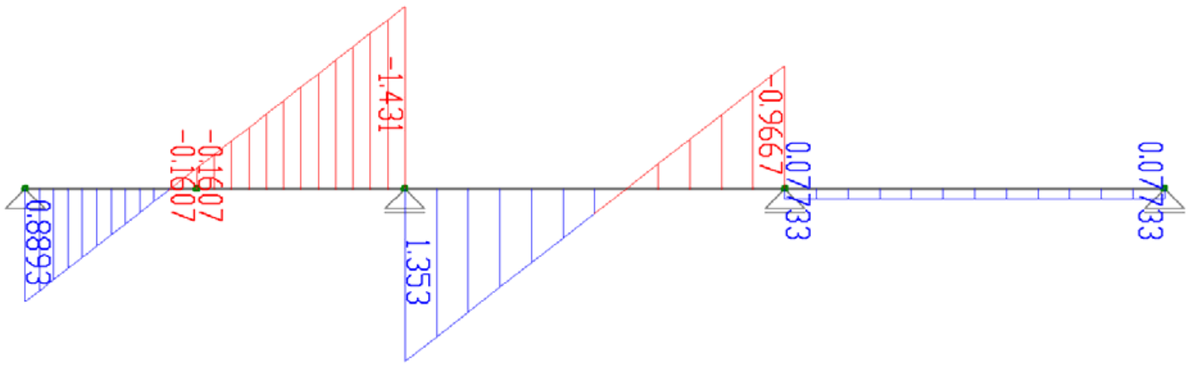
loading and reaction forces



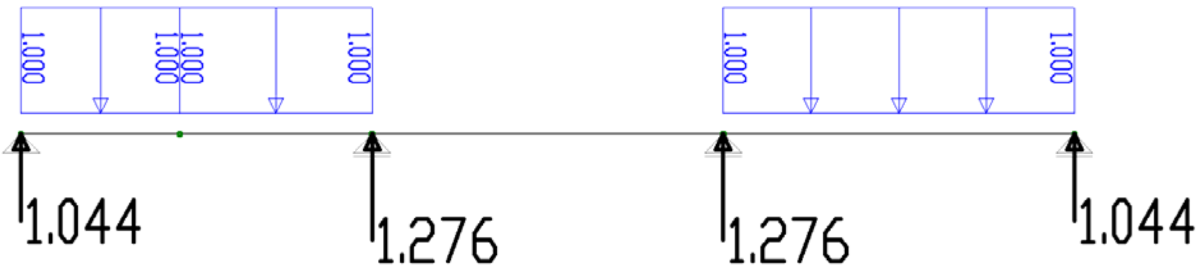
bending moment



shear force

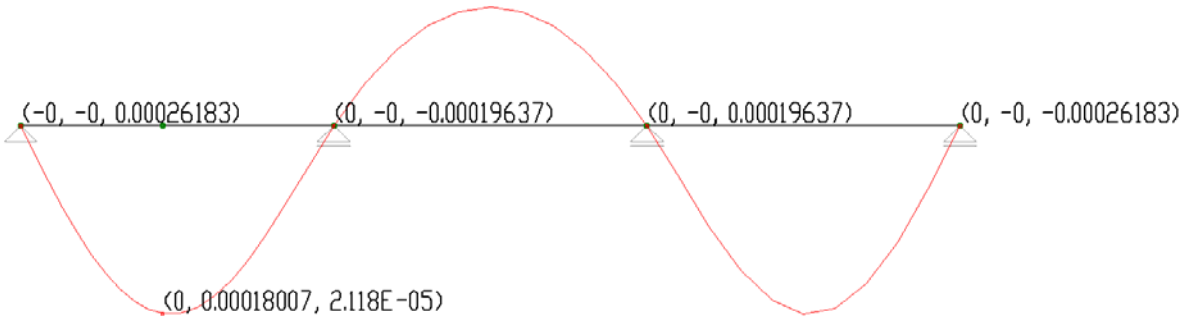


loading and reaction forces



b) Panel Construction

deformations



ULS CO3

$k_{mod} = 0{,}9$

$p_d = 1{,}20 \cdot 3{,}70 + 1{,}50 \cdot 10{,}21 = 19{,}75\ kN/m$

$q_d = 1{,}5 \cdot 0{,}7 \cdot 1{,}64 = 1{,}72\ kN/m$

$M_d = 19{,}75 \cdot 0{,}5382 + 1{,}72 \cdot 0{,}6297$
$= 11{,}72\ kNm$

$$\sigma_{md} = \frac{1172}{1152} = 1{,}02\ kN/cm^2$$

$$f_{md} = 0{,}9 \cdot \frac{2{,}4}{1{,}15} = 1{,}88\ kN/cm^2$$

$$\eta = \frac{1{,}02}{1{,}88} = 0{,}54 < 1{,}0$$

- no lateral torsional buckling because the joists secure the compression flange of the beam

$V_d = 19{,}75 \cdot 1{,}392 + 1{,}72 \cdot 1{,}431 = 29{,}96\ kN$

$$\tau_d = 1{,}5 \cdot \frac{29{,}96}{0{,}67 \cdot 288} = 0{,}23\ kN/cm^2$$

$$f_{vd} = 0{,}9 \cdot \frac{0{,}35}{1{,}15} = 0{,}27\ kN/cm^2$$

$$\eta = \frac{0{,}23}{0{,}27} = 0{,}85 < 1{,}0$$

SLS CO9/10

$w_{inst,g} = 3{,}70 \cdot 0{,}01254 = 0{,}046\ cm$

$w_{inst,s} = 0{,}128\ cm$

$w_{inst,w} = 0{,}006\ cm$

$w_{inst,q} = 1{,}64 \cdot 0{,}01801 = 0{,}030\ cm$

instantaneous deformation

$w_{inst} = 0{,}046 + 0{,}128 + 0{,}6 \cdot 0{,}006 + 0{,}7 \cdot 0{,}30$
$= 0{,}199\ cm$

$$\max w_{inst} = \frac{232}{300} = 0{,}77\ cm$$

b) Panel Construction

$$\eta = \frac{0,199}{0,77} = 0,26 < 1,0$$

final deformation

$$\begin{aligned} w_{fin} &= 0,046 \cdot (1 + 0,6) + 0,128 \cdot (1 + 0,2 \cdot 0,6) \\ &+ 0,006 \cdot (0,6 + 0) + 0,030 \cdot (0,7 + 0,3 \cdot 0,6) \\ &= 0,247 \text{ cm} \end{aligned}$$

$$\max w_{fin} = \frac{232}{150} = 1,55 \text{ cm}$$

$$\eta = \frac{0,247}{1,55} = 0,16 < 1,0$$

max reaction forces

$A_{gk} = 3,70 \cdot 0,928 = 3,43$ kN

$A_{sk} = 9,47$ kN

$A_{wk} = 0,42$ kN

$A_{qk} = 1,64 \cdot 1,044 = 1,71$ kN

$B_{gk} = 3,70 \cdot 2,552 = 9,45$ kN

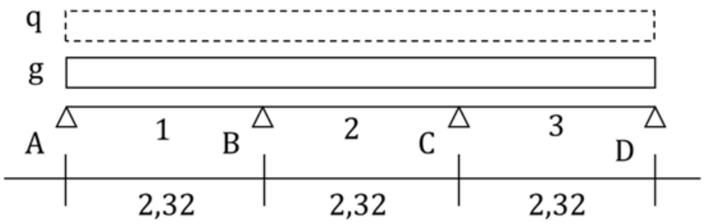
$B_{sk} = 26,04$ kN

$B_{wk} = 1,17$ kN

$B_{qk} = 1,64 \cdot 2,784 = 4,57$ kN

6 Beam (Floor)

system



$\max l = 2{,}32\ \text{m}$

cross-section

$b/h = 8/24$

$A = 192\ \text{cm}^2$

$I_y = 9216\ \text{cm}^4$

$W_y = 768\ \text{cm}^3$

material

glulam GL24h

$f_{mk} = 24\ \text{N/mm}^2$

$k_{cr} = 0{,}67$

$f_{vk} = 3{,}5\ \text{N/mm}^2$

$E_{0,mean} = 11500\ \text{N/mm}^2$

$k_{def} = 0{,}6$

loads

- reaction force A @ 1.2

$$g_k = \frac{1{,}83}{0{,}6} = 3{,}04\ \text{kN/m}$$

$s_k = 0$

$w_k = 0$

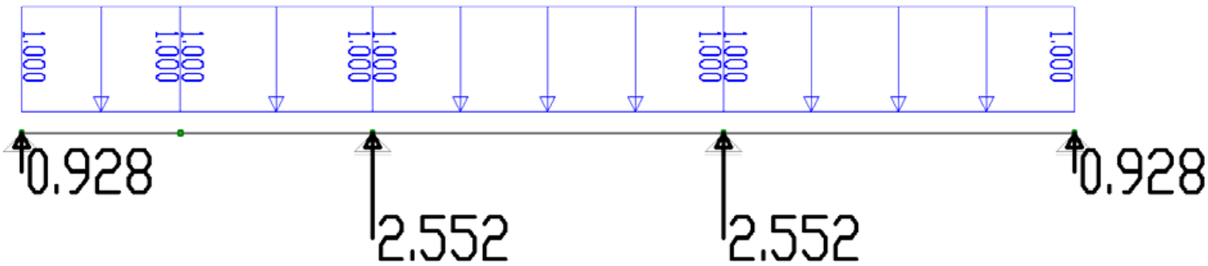
$$q_k = \frac{3{,}29}{0{,}6} = 5{,}48\ \text{kN/m}$$

b) Panel Construction

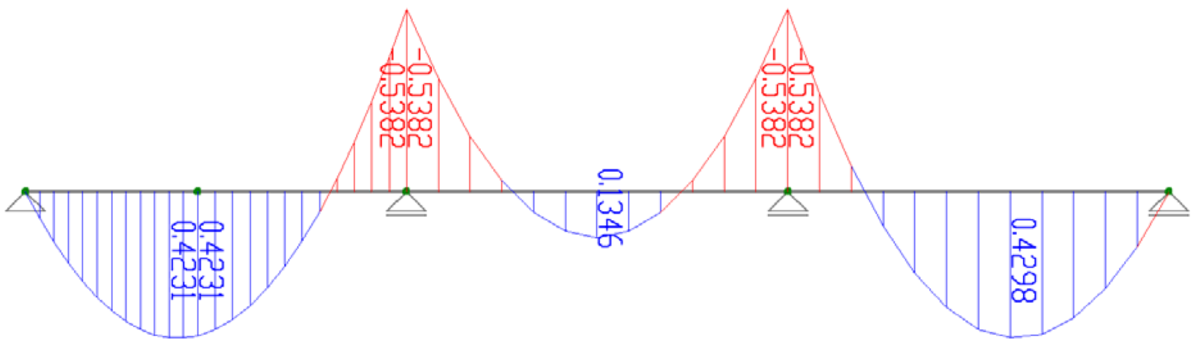
reaction forces, internal forces and deformations

constant load

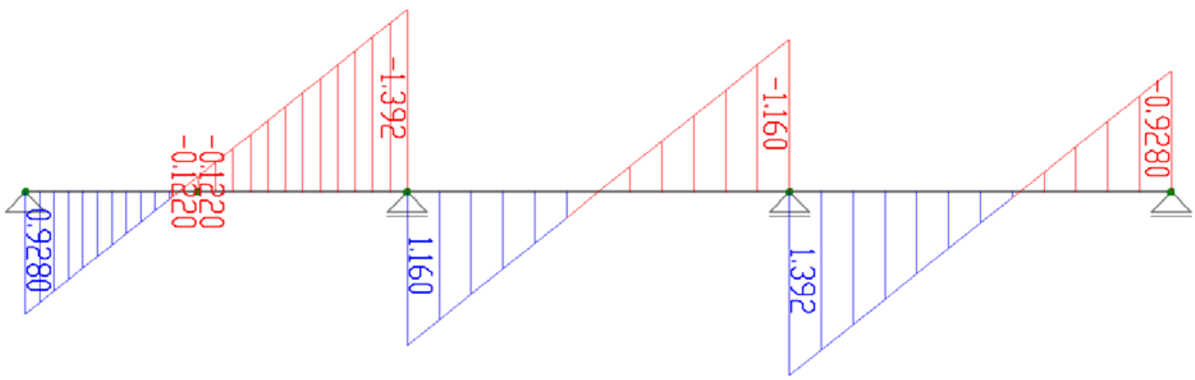
loading and reaction forces



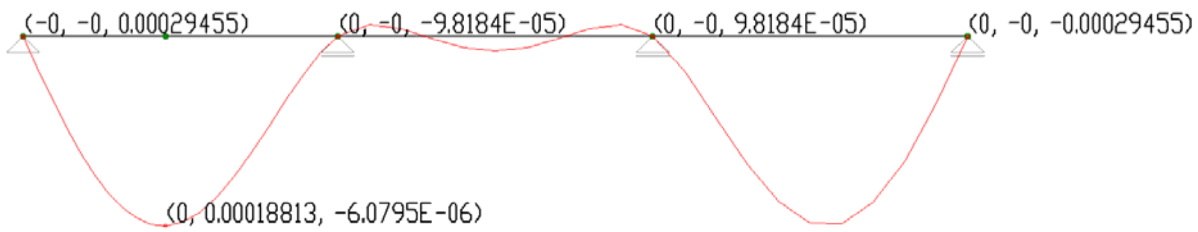
bending moment



shear force



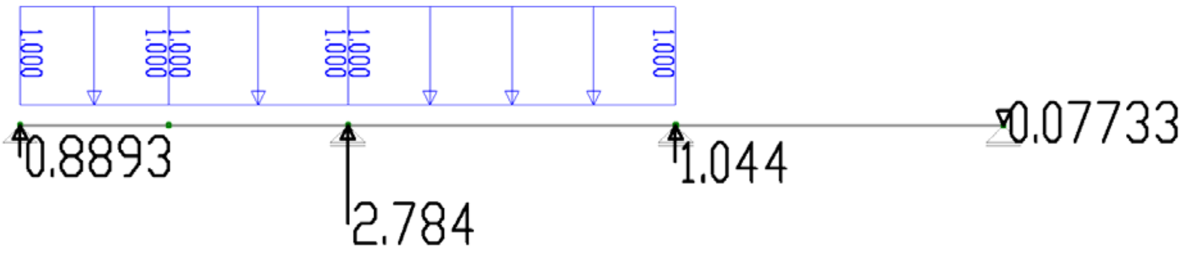
deformations



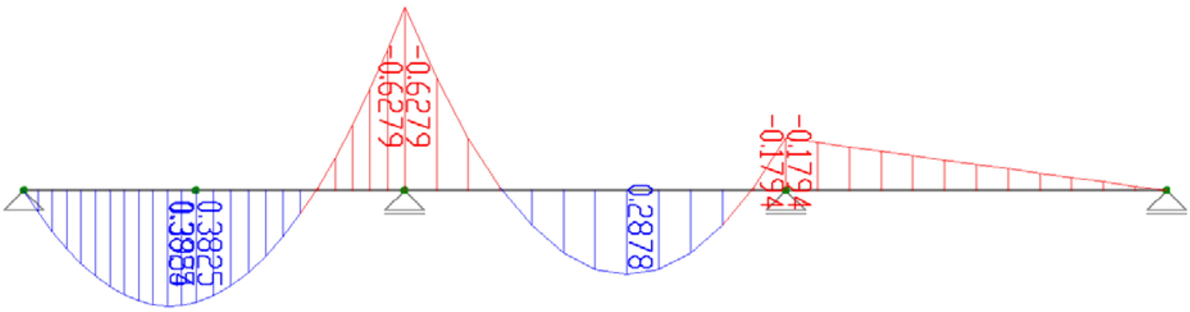
b) Panel Construction

unfavourable loads

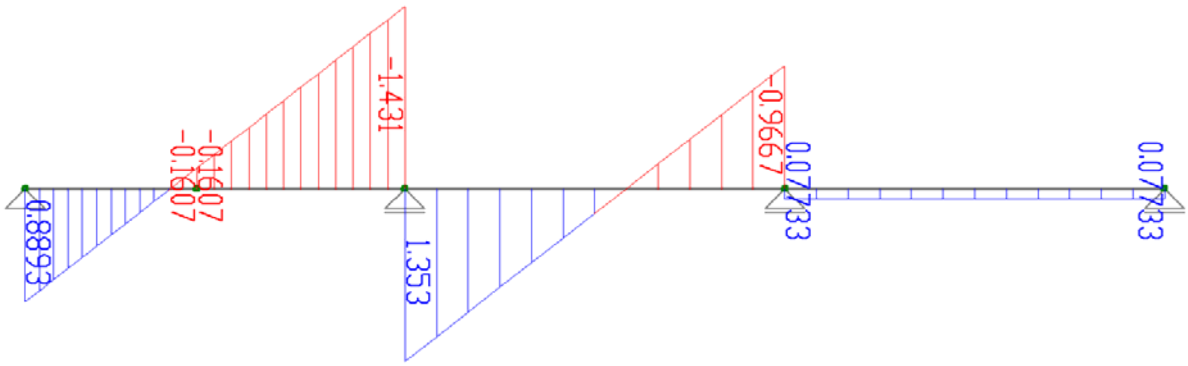
loading and reaction forces



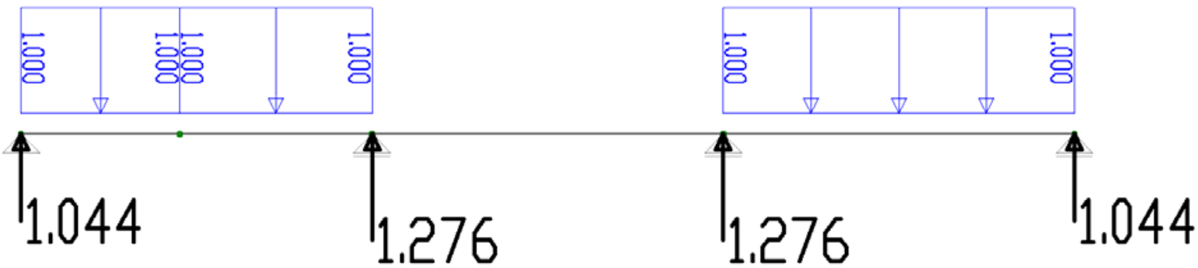
bending moment



shear force

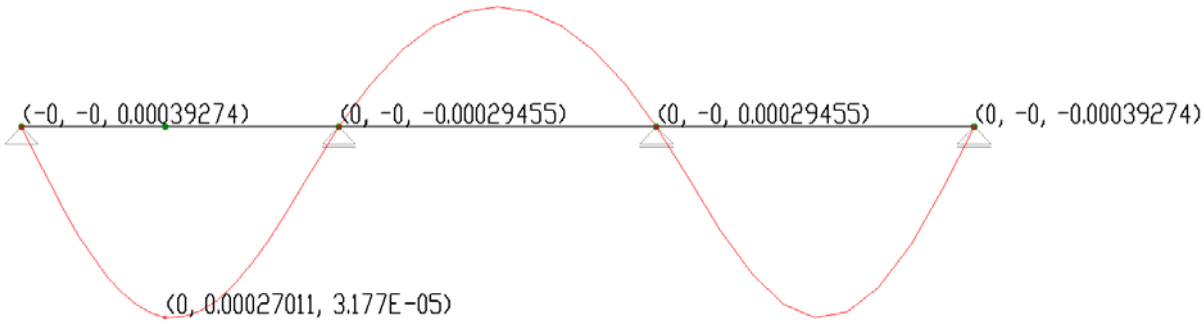


loading and reaction forces



b) Panel Construction

deformations



ULS CO2

$k_{mod} = 0{,}8$

$g_d = 1{,}20 \cdot 3{,}04 = 3{,}65 \text{ kN/m}$

$q_d = 1{,}5 \cdot 5{,}48 = 8{,}21 \text{ kN/m}$

$M_d = 3{,}65 \cdot 0{,}5382 + 8{,}21 \cdot 0{,}6297 = 7{,}14 \text{ kNm}$

$\sigma_{md} = \frac{714}{768} = 0{,}93 \text{ kN/cm}^2$

$f_{md} = 0{,}8 \cdot \frac{2{,}4}{1{,}15} = 1{,}67 \text{ kN/cm}^2$

$\eta = \frac{0{,}93}{1{,}67} = 0{,}56 < 1{,}0$

- no lateral torsional buckling because the joists secure the compression flange of the beam

$V_d = 3{,}65 \cdot 1{,}392 + 8{,}21 \cdot 1{,}431 = 16{,}84 \text{ kN}$

$\tau_d = 1{,}5 \cdot \frac{16{,}84}{0{,}67 \cdot 192} = 0{,}20 \text{ kN/cm}^2$

$f_{vd} = 0{,}8 \cdot \frac{0{,}35}{1{,}15} = 0{,}24 \text{ kN/cm}^2$

$\eta = \frac{0{,}20}{0{,}24} = 0{,}81 < 1{,}0$

b) Panel Construction

SLS CO7/8

$w_{inst,g} = 3{,}04 \cdot 0{,}0188 = 0{,}057 \text{ cm}$

$w_{inst,s} = 0$

$w_{inst,w} = 0$

$w_{inst,q} = 5{,}48 \cdot 0{,}02701 = 0{,}148 \text{ cm}$

instantaneous deformation

$w_{inst} = 0{,}057 + 0{,}148 + 0 + 0 = 0{,}205 \text{ cm}$

$\max w_{inst} = \frac{232}{300} = 0{,}77 \text{ cm}$

$\eta = \frac{0{,}205}{0{,}77} = 0{,}26 < 1{,}0$

final deformation

$$w_{fin} = 0{,}057 \cdot (1 + 0{,}6) + 0{,}148 \cdot (1 + 0{,}3 \cdot 0{,}6) + 0 + 0 = 0{,}266 \text{ cm}$$

$\max w_{fin} = \frac{232}{150} = 1{,}55 \text{ cm}$

$\eta = \frac{0{,}266}{1{,}55} = 0{,}17 < 1{,}0$

max reaction forces

$A_{gk} = 3{,}04 \cdot 0{,}928 = 2{,}82 \text{ kN}$

$A_{sk} = 0$

$A_{wk} = 0$

$A_{qk} = 5{,}48 \cdot 1{,}044 = 5{,}72 \text{ kN}$

$B_{gk} = 3{,}04 \cdot 2{,}552 = 7{,}77 \text{ kN}$

$B_{sk} = 0$

$B_{wk} = 0$

$B_{qk} = 5{,}48 \cdot 2{,}784 = 15{,}24 \text{ kN}$

5 Columns

system

cross-sections

column A, D

$b/h = 16/20$

$A = 320\ cm^2$

$I_y = I_z = 10667\ cm^4$

$$i_y = i_z = \frac{16}{\sqrt{12}} = 4{,}62\ cm$$

column B, C

$b/h = 20/20$

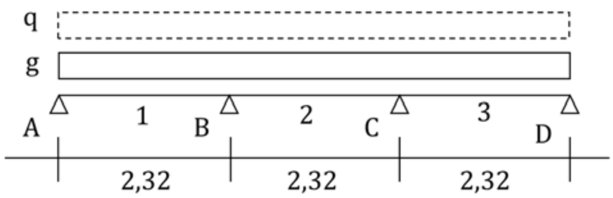
$A = 400\ cm^2$

$I_y = I_z = 13333\ cm^4$

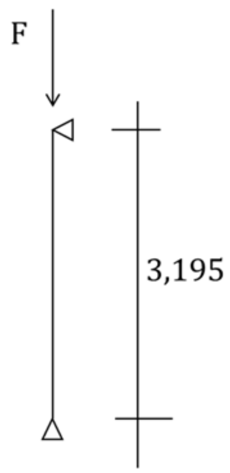
$$i_y = i_z = \frac{20}{\sqrt{12}} = 5{,}77\ cm$$

material
glulam GL24h
$\gamma = 3{,}7\ kN/m^3$
$f_{ck} = 24\ N/mm^2$

global system (beam 6)



local system



loads

column A, D

$g_k = 3{,}7 \cdot 0{,}16 \cdot 0{,}2 = 0{,}037\ kN/m$

- self-weight of the column

$N_{gk} = 3{,}43 + 14 \cdot 2{,}82 = 43{,}0\ kN$

- reaction force A @ 6
- the load is added up for the roof and 14 storeys

$s_k = 0$

$N_{sk} = 9{,}47\ kN$

$w_k = 0$

$N_{wk} = 0{,}42\ kN$

$q_k = 0$

$N_{qk} = 1{,}71 + 14 \cdot 0{,}74 \cdot 5{,}72 = 60{,}9\ kN$

column B, C

$g_k = 3{,}7 \cdot 0{,}2 \cdot 0{,}2 = 0{,}046\ kN/m$

- self-weight of the column

$N_{gk} = 9{,}45 + 14 \cdot 7{,}77 = 118{,}2\ kN$

- reaction force B @ 6
- the load is added up for the roof and 14 storeys

$s_k = 0$

$N_{sk} = 26{,}04\ kN$

$w_k = 0$

b) Panel Construction

$N_{wk} = 1{,}17\ \mathrm{kN}$

$q_k = 0$

$N_{qk} = 4{,}57 + 14 \cdot 0{,}74 \cdot 15{,}24 = 162{,}5\ \mathrm{kN}$

ULS CO2

$k_{mod} = 0{,}8$

column A, D

$g_d = 1{,}2 \cdot 0{,}037 = 0{,}044\ \mathrm{kN/m}$

$$N_{cd} = 1{,}2 \cdot 43{,}0 + 0{,}044 \cdot 16 \cdot 3{,}195 + 1{,}5 \cdot 60{,}9 = 145{,}2\ \mathrm{kN}$$

$$\sigma_{cd} = \frac{145{,}2}{320} = 0{,}45\ \mathrm{kN/cm^2}$$

$$f_{cd} = 0{,}8 \cdot \frac{2{,}4}{1{,}15} = 1{,}67\ \mathrm{kN/cm^2}$$

flexural buckling

$$\lambda = \frac{319{,}5}{4{,}62} = 69{,}2$$

$$\lambda_{rel} = \frac{69{,}2}{\pi} \cdot \sqrt{\frac{24}{9600}} = 1{,}10$$

$$k = 0{,}5 \cdot (1 + 0{,}1 \cdot (1{,}10 - 0{,}3) + 1{,}10^2) = 1{,}15$$

$$k_c = \frac{1}{1{,}15 + \sqrt{1{,}15^2 - 1{,}10^2}} = 0{,}683$$

$$\eta = \frac{0{,}45}{0{,}683 \cdot 1{,}67} = 0{,}40 < 1{,}0$$

column B, C

$g_d = 1{,}2 \cdot 0{,}046 = 0{,}055\ \mathrm{kN/m}$

$$N_{cd} = 1{,}2 \cdot 118{,}2 + 0{,}055 \cdot 16 \cdot 3{,}195 + 1{,}5 \cdot 162{,}5 = 388{,}4\ \mathrm{kN}$$

$$\sigma_{cd} = \frac{388{,}4}{400} = 0{,}97\ \mathrm{kN/cm^2}$$

$$f_{cd} = 0{,}8 \cdot \frac{2{,}4}{1{,}15} = 1{,}67\ \mathrm{kN/cm^2}$$

flexural buckling

$$\lambda = \frac{319{,}5}{5{,}77} = 55{,}3$$

b) Panel Construction

$$\lambda_{rel} = \frac{55{,}3}{\pi} \cdot \sqrt{\frac{24}{9600}} = 0{,}88$$

$$k = 0{,}5 \cdot (1 + 0{,}1 \cdot (0{,}88 - 0{,}3) + 0{,}88^2)$$
$$= 0{,}92$$

$$k_c = \frac{1}{0{,}92 + \sqrt{0{,}92^2 - 0{,}88^2}} = 0{,}853$$

$$\eta = \frac{0{,}97}{0{,}853 \cdot 1{,}67} = 0{,}68 < 1{,}0$$

b) Panel Construction

Anchoring

The hold-down device consists of two internal steel plates and dowels inside the corresponding studs. At studs with smaller tensile forces, the connection can be adapted, e. g. by using only one internal steel plate and/or less dowels.

system

angle of the force to the grain

$\alpha = 0°$

fasteners

8 M20 dowels

$$n_{ef} = 2 \cdot 4^{0,9} \cdot \sqrt[4]{\frac{200}{13 \cdot 20}} = 6{,}52$$

strength class 4.6

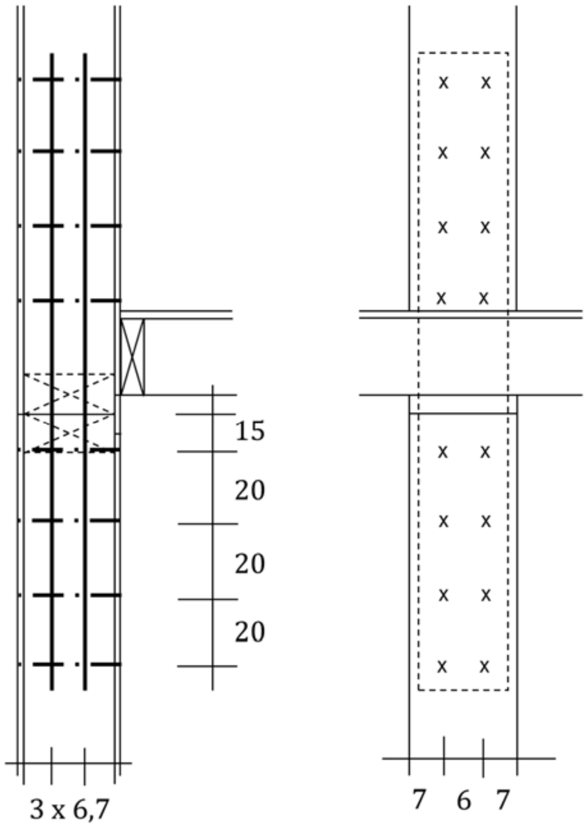
$f_{yk} = 240\ \mathrm{N/mm^2}$

$f_{uk} = 400\ \mathrm{N/mm^2}$

material

glulam GL24h

$\rho_k = 385\ \mathrm{kg/m^3}$



minimum distances

$a_1 = (3 + 2 \cdot \cos 0) \cdot 2{,}0 = 10\ \mathrm{cm}$

$a_2 = 3 \cdot 2{,}0 = 6\ \mathrm{cm}$

$a_{3t} = \max(7 \cdot 2{,}0 = 14\,; 8) = 14\ \mathrm{cm}$

$a_{4c} = 3 \cdot 2{,}0 = 6\ \mathrm{cm}$

capacity per fastener per shear plane

$M_{y,Rk} = 0{,}3 \cdot 400 \cdot 20^{2,6}/1000 = 289{,}6\ \mathrm{Nm}$

$$f_{h,0,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot 20) \cdot 385 = 25{,}26\ \mathrm{N/mm^2}$$

$$F_{v,Rk} = 25{,}26 \cdot 67 \cdot 20 \cdot \left[\sqrt{2 + \frac{4 \cdot 289{,}6 \cdot 1000}{25{,}26 \cdot 20 \cdot 67^2}} - 1\right]/1000 = 19{,}8\ \mathrm{kN}$$

- failure mode g becomes decisive
- dowels have no axial capacity

b) Panel Construction

ULS CO11/12

$k_{mod} = 1,1$

$F_d = 448,6 \text{ kN}$

- cf. RFEM analysis

$F_{v.Rk} = 19,8 \cdot 6,52 \cdot 4 = 516,2 \text{ kN}$

- 6,52 bolts and 4 shear planes

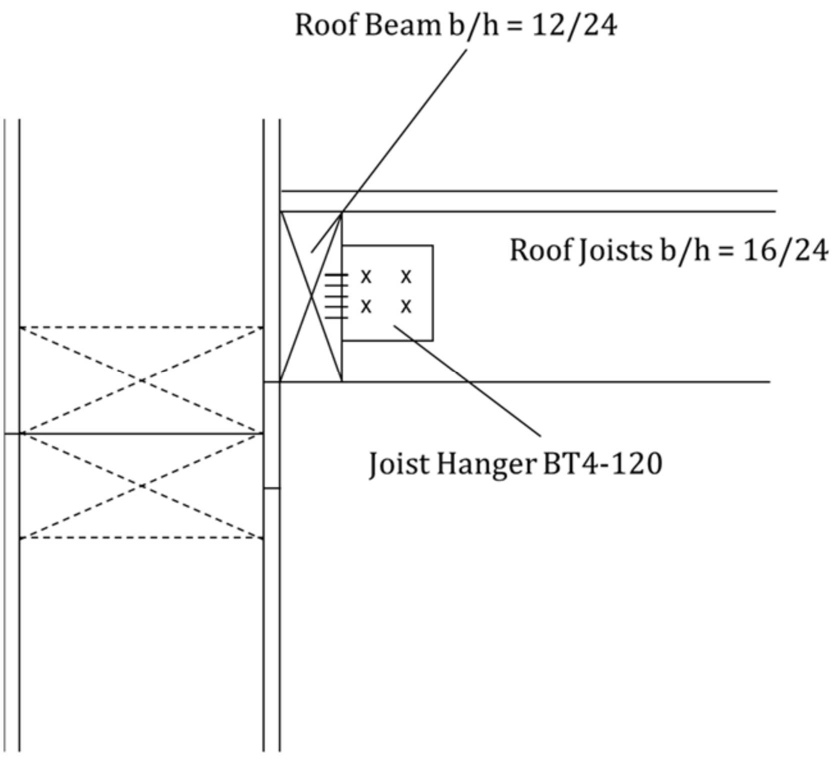
$$F_{v,Rd} = 1,1 \cdot \frac{516,2}{1,3} = 436,8 \text{ kN}$$

$$\eta = \frac{448,6}{436,8} = 1,03 \approx 1,0$$

Connection Roof Joists 1 to Roof Beam 2

This connection had to be changed in comparison to the preliminary design. Instead of diagonal fully threaded screws, joist hangers are used.

system



angle of the force to the grain

$\alpha = 90°$

fasteners

Joist Hanger Simpson Strongtie BT4-120 or similar

material

glulam GL24h

$\rho_k = 385\ kg/m^3$

capacity

- cf. technical data sheet [7]

$F_{v,Rk} = 23{,}9\ kN$

loads

- reaction force A @ 1.1
- the shear forces in the joist are equal to the support forces

$F_{gk} = 2{,}61\ kN$

$F_{sk} = 7{,}20\ kN$

$F_{wk} = 0{,}32\ kN$

$F_{qk} = 1{,}16\ kN$

ULS CO3

b) Panel Construction

$k_{mod} = 0{,}9$

$F_d = 1{,}2 \cdot 2{,}61 + 1{,}5 \cdot 7{,}20 + 1{,}5 \cdot 0{,}7 \cdot 1{,}16$
$\quad = 15{,}1 \text{ kN}$

$F_{v.Rk} = 23{,}9 \text{ kN}$

- 11 bolts and 2 shear planes

$$F_{v,Rd} = 0{,}9 \cdot \frac{23{,}9}{1{,}3} = 16{,}5 \text{ kN}$$

$$\eta = \frac{15{,}1}{16{,}5} = 0{,}92 < 1{,}0$$

check for tension perpendicular to the grain in the beam

$V_d = F_d = 15{,}1 \text{ kN}$

$$F_{90Rk} = 14 \cdot 160 \cdot 1 \cdot \sqrt{\frac{165}{1 - \frac{16{,}5}{24}}}/1000$$

$$= 51{,}5 \text{ kN}$$

$$F_{90Rd} = 0{,}9 \cdot \frac{51{,}5}{1{,}3} = 35{,}6 \text{ kN}$$

$$\eta = \frac{15{,}1}{35{,}6} = 0{,}43 < 1{,}0$$

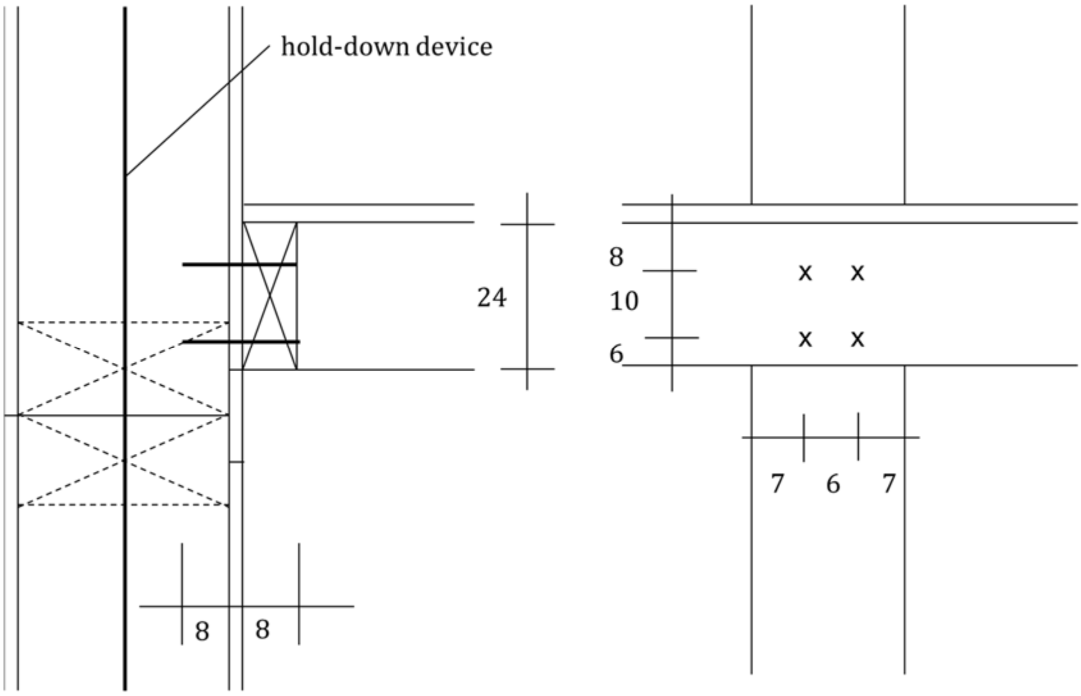
Connection Floor Joists 1 to Floor Beam 2

For this connection, the same system is used, but the loads are lower from the floor than from the roof, so that this connection is not decisive.

Connection Floor Beam 2 to Studs 3

The floor beam in axis 1–3/A becomes decisive at support C.

system



angle of the force to the grain

floor beam

$\alpha = 90°$

stud

$\alpha = 0°$

fasteners

4 M20 dowels

$$n_{ef} = 2 \cdot 2^{0,9} \cdot \sqrt[4]{\frac{100}{13 \cdot 20}} = 2,94$$

$f_{yk} = 240\ \text{N/mm}^2$

- strength class 4.6

$f_{uk} = 400\ \text{N/mm}^2$

material

glulam GL24h

$\rho_k = 385\ \text{kg/m}^3$

minimum distances

floor beam

$a_1 = (3 + 2 \cdot \cos 90) \cdot 2,0 = 6\ \text{cm}$

b) Panel Construction

$a_2 = 3 \cdot 2{,}0 = 6 \text{ cm}$

$$a_{4t} = \max\big((2 + 2 \cdot \sin 90) \cdot 2{,}0 = 8\,;\; 3 \cdot 2{,}0 = 6\big) = 8 \text{ cm}$$

$a_{4c} = 3 \cdot 2{,}0 = 6 \text{ cm}$

stud

$a_1 = (3 + 2 \cdot \cos 0) \cdot 2{,}0 = 10 \text{ cm}$

$a_2 = 3 \cdot 2{,}0 = 6 \text{ cm}$

$a_{4c} = 3 \cdot 2{,}0 = 6 \text{ cm}$

capacity per fastener per shear plane

$M_{y,Rk} = 0{,}3 \cdot 400 \cdot 20^{2{,}6}/1000 = 289{,}6 \text{ Nm}$

floor beam (member 1)

$$f_{h,0,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot 20) \cdot 385 = 25{,}26 \text{ N/mm}^2$$

$k_{90} = 1{,}35 + 0{,}015 \cdot 20 = 1{,}65$

$$f_{h,90,k} = \frac{25{,}26}{1{,}65 \cdot \sin^2 90 + \cos^2 90} = 15{,}3 \text{ N/mm}^2$$

stud (member 2)

$$f_{h,0,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot 20) \cdot 385 = 25{,}26 \text{ N/mm}^2$$

$$\beta = \frac{25{,}26}{15{,}3} = 1{,}65$$

- failure mode d becomes decisive
- dowels have no axial capacity

$$F_{v,Rk} = 1{,}05 \cdot \frac{15{,}3 \cdot 80 \cdot 20}{2 + 1{,}65} \cdot \left[\sqrt{2 \cdot 1{,}65 \cdot (1 + 1{,}65) + \frac{4 \cdot 1{,}65 \cdot (2 + 1{,}65) \cdot 289{,}64 \cdot 1000}{15{,}3 \cdot 20 \cdot 80^2}} - 1{,}65\right] = 13{,}1 \text{ kN}$$

loads

- reaction force C @ 2

$F_{gk} = 5{,}53 \text{ kN}$

$F_{qk} = 10{,}29 \text{ kN}$

shear forces in the beam at the connection

$V_{gk} = 2{,}86 \text{ kN}$

$V_{qk} = 5{,}36 \text{ kN}$

ULS CO2

b) Panel Construction

$k_{mod} = 0,8$

$F_d = 1,2 \cdot 5,53 + 1,5 \cdot 10,29 = 22,08 \text{ kN}$

$F_{v.Rk} = 2,93 \cdot 1 \cdot 13,09 = 38,5 \text{ kN}$

- 2,94 bolts and 1 shear plane

$F_{v,Rd} = 0,8 \cdot \frac{38,5}{1,3} = 23,7 \text{ kN}$

$\eta = \frac{22,08}{23,7} = 0,93 < 1,0$

tension perpendicular to the grain

$V_d = 1,2 \cdot 2,86 + 1,5 \cdot 5,36 = 11,5 \text{ kN}$

$F_{90Rk} = 14 \cdot 80 \cdot 1 \cdot \sqrt{\frac{180}{1 - \frac{18}{24}}} / 1000 = 30,1 \text{ kN}$

$F_{90Rd} = 0,8 \cdot \frac{30,1}{1,3} = 18,5 \text{ kN}$

$\eta = \frac{11,5}{18,5} = 0,62 < 1,0$

Connection Roof Beam 2 to Studs 3

This connection follows the same principle as the connection of the floor beam to the studs. Because of the higher loads from the roof than from the floors, 6 dowels are required instead of 4. Therefore, the studs in the highest storey must be 28/28 (like the studs in storey 1 to 5) to allow for enough space for the 6 dowels.

c) CLT Construction

Element Reference Overview

reference no	element	page
1	floor slab	100
2, 3	floor slab	-
4	floor slab	-
5	Walls	106
	Anchoring	108

Remarks

Because of good lateral distribution of the loads in the slabs, the single concentrated force Q will not become decisive.

For the calculation of the internal forces of the walls, the FE software RFEM was used.

Because of missing rules for CLT in the EC, the same design factors as for glulam are applied.

Compared to the preliminary design, the thicknesses of the walls had to be increased. Now walls with the thicknesses 145 mm, 170 mm and 246 mm are used. The reduction of the thickness over the height of the building was adapted.

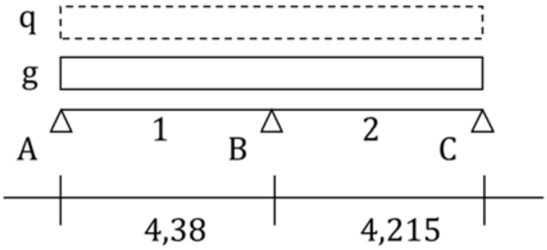
storey no	M	N	t	t	t
-	[%]	[%]	[mm]	[mm]	[mm]
0	100,00	100,00	145	170	246
1	87,11	93,33	145	170	246
2	75,11	86,67	145	170	246
3	64,00	80,00	145	170	246
4	53,78	73,33	145	170	246
5	44,44	66,67	120	120	170
6	36,00	60,00	120	120	170
7	28,44	53,33	120	120	170
8	21,78	46,67	120	120	170
9	16,00	40,00	120	120	170
10	11,11	33,33	95	95	95
11	7,11	26,67	95	95	95
12	4,00	20,00	95	95	95
13	1,78	13,33	95	95	95
14	0,44	6,67	95	95	95
15	0,00	0,00	95	95	95

c) CLT Construction

1 Roof Slab

system

max l = 4,38 m



- see technical approval [1]

cross-section

t = 145 mm

$A = 1450\ cm^2/m$

$I_y = 25405\ cm^4/m$

$W_y = 3504\ cm^3/m$

material

CLT KL-trä 145

$f_{mk} = 10{,}1\ N/mm^2$

$k_{cr} = 0{,}67$

$f_{vk} = 0{,}7\ N/mm^2$

$E_{0,mean} = 5290\ N/mm^2$

$k_{def} = 0{,}6$

loads

$g_k = 1{,}97\ kN/m^2$

$s_k = 4{,}41\ kN/m^2$

$q_p = 1044\ N/m^2$

$c_{pe}(I) = +0{,}2$

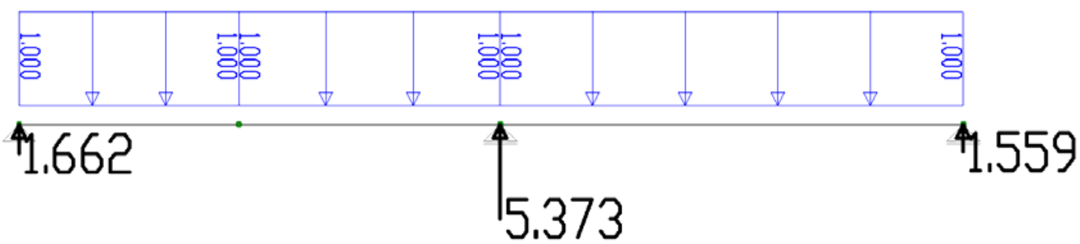
$w_k = 0{,}2 \cdot 1{,}044 = 0{,}21\ kN/m^2$

$q(H) = 0{,}75\ kN/m^2$

reaction forces, internal forces and deformations

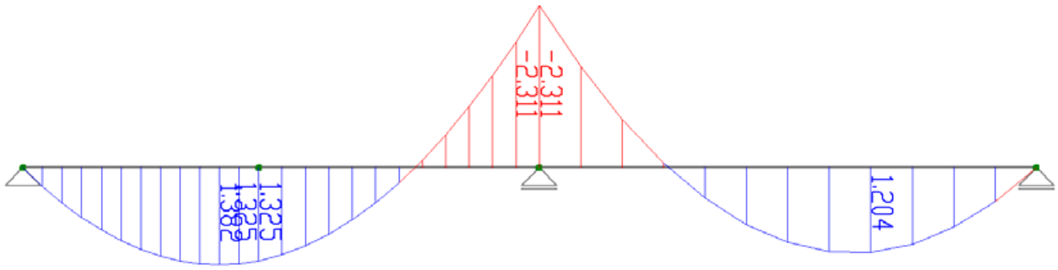
constant load

loading and reaction forces

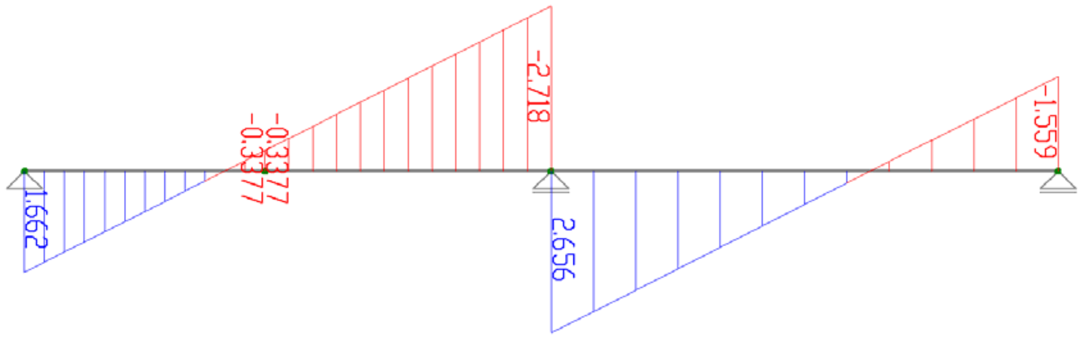


c) CLT Construction

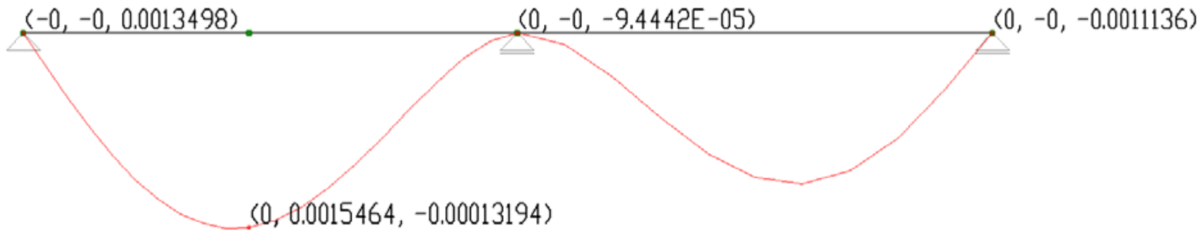
bending moment



shear force

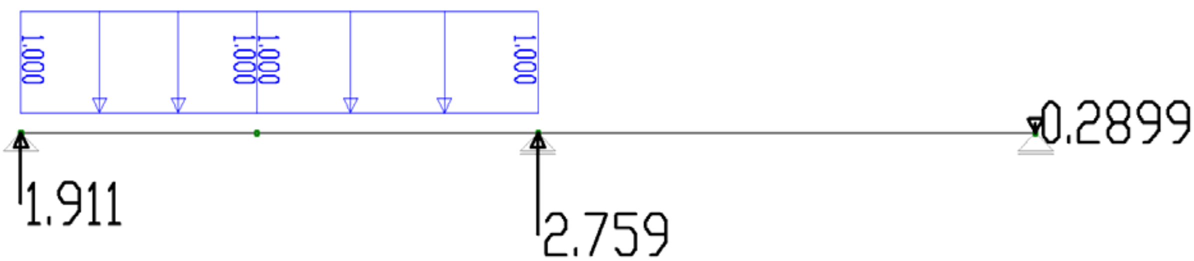


deformations



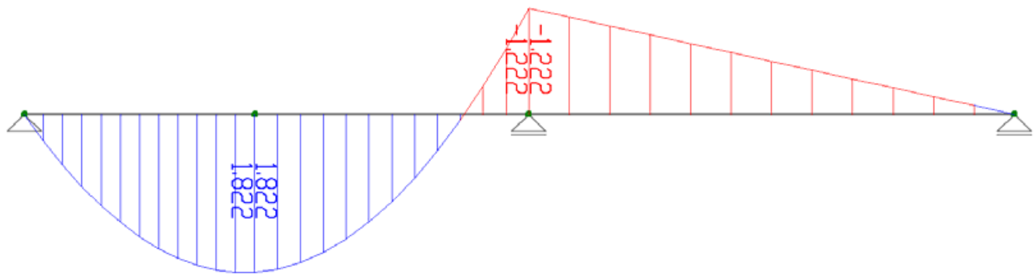
unfavourable loads

loading and reaction forces

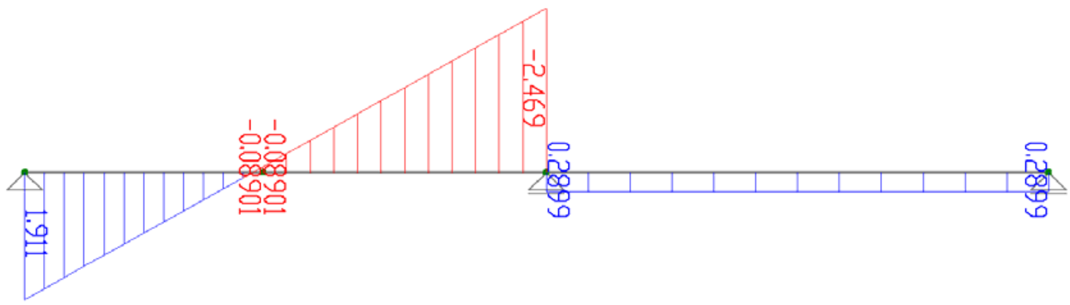


c) CLT Construction

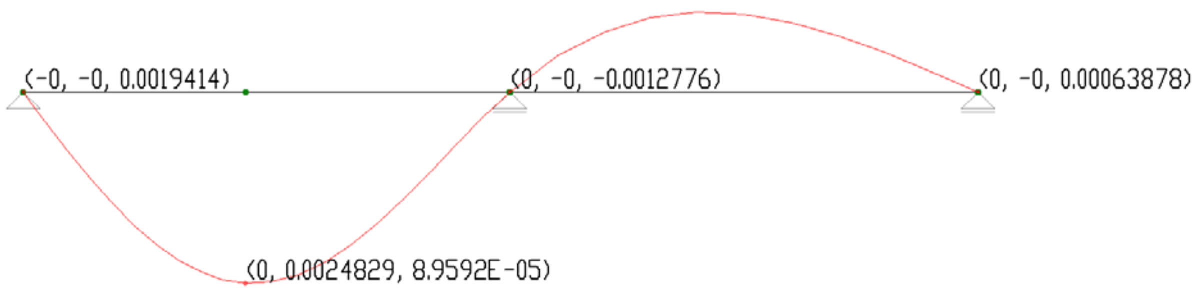
bending moment



shear force



deformations



ULS CO3

$k_{mod} = 0{,}9$

$p_d = 1{,}20 \cdot 1{,}97 + 1{,}50 \cdot 4{,}41 = 8{,}97\ kN/m^2$

$q_d = 1{,}50 \cdot 0{,}7 \cdot 0{,}75 = 0{,}79\ kN/m^2$

$$M_d = 8{,}97 \cdot 2{,}311 + 0{,}79 \cdot 2{,}311$$
$$= 22{,}55\ kNm/m$$

$$\sigma_{md} = \frac{2255}{3504} = 0{,}64\ kN/cm^2$$

$$f_{md} = 0{,}9 \cdot \frac{1{,}01}{1{,}15} = 0{,}79\ kN/cm^2$$

$$\eta = \frac{0{,}64}{0{,}79} = 0{,}81 < 1{,}0$$

$$V_d = 8{,}97 \cdot 2{,}718 + 0{,}79 \cdot 2{,}718 = 26{,}52\ kN$$

$$\tau_d = 1{,}5 \cdot \frac{26{,}52}{0{,}67 \cdot 1450} = 0{,}041\ kN/cm^2$$

c) CLT Construction

$$f_{vd} = 0,9 \cdot \frac{0,07}{1,15} = 0,055 \text{ kN/cm}^2$$

$$\eta = \frac{0,041}{0,055} = 0,75 < 1,0$$

SLS CO9/10

$$w_{inst,g} = 1,97 \cdot 0,1546 = 0,305 \text{ cm}$$

$$w_{inst,s} = 0,681 \text{ cm}$$

$$w_{inst,w} = 0,032 \text{ cm}$$

$$w_{inst,q} = 0,75 \cdot 0,2483 = 0,186 \text{ cm}$$

instantaneous deformation

$$\begin{aligned} w_{inst} & \\ &= 0,305 + 0,681 + 0,6 \cdot 0,032 + 0,7 \cdot 0,186 \\ &= 1,14 \text{ cm} \end{aligned}$$

$$\max w_{inst} = \frac{438}{300} = 1,46 \text{ cm}$$

$$\eta = \frac{1,14}{1,46} = 0,78 < 1,0$$

final deformation

$$\begin{aligned} w_{fin} & \\ &= 0,305 \cdot (1 + 0,6) + 0,681 \cdot (1 + 0,2 \cdot 0,6) \\ &+ 0,032 \cdot (0,6 + 0) + 0,186 \cdot (0,7 + 0,3 \cdot 0,6) \\ &= 1,43 \text{ cm} \end{aligned}$$

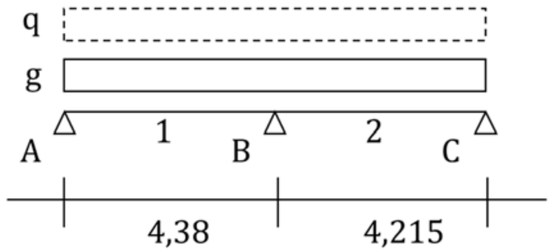
$$\max w_{fin} = \frac{438}{150} = 2,92 \text{ cm}$$

$$\eta = \frac{1,43}{2,92} = 0,49 < 1,0$$

c) CLT Construction

1 Floor Slab

system



max l = 4,38 m

cross-section

t = 145 mm

$A = 1450\ cm^2/m$

$I_y = 25405\ cm^4/m$

$W_y = 3504\ cm^3/m$

- see technical approval [1]

material

CLT KL-trä 145

$f_{mk} = 10,1\ N/mm^2$

$k_{cr} = 0,67$

$f_{vk} = 0,7\ N/mm^2$

$E_{m,50} = 5290\ N/mm^2$

$k_{def} = 0,6$

loads

$g_k = 1,64\ kN/m^2$

$s_k = 0$

$w_k = 0$

$q(A) = 2,5\ kN/m^2$

reaction forces, internal forces and deformations

- cf. 1.1 Roof Slab

ULS CO2

$k_{mod} = 0,8$

$g_d = 1,20 \cdot 1,64 = 2,0\ kN/m^2$

$q_d = 1,50 \cdot 2,5 = 3,75\ kN/m^2$

$M_d = 2,0 \cdot 2,311 + 3,75 \cdot 2,311 = 13,21\ kNm/m$

$$\sigma_{md} = \frac{1321}{3504} = 0,38\ kN/cm^2$$

$$f_{md} = 0,8 \cdot \frac{1,01}{1,15} = 0,70\ kN/cm^2$$

$$\eta = \frac{0,38}{0,70} = 0,54 < 1,0$$

$V_d = 2,0 \cdot 2,718 + 3,75 \cdot 2,718 = 15,54\ kN$

c) CLT Construction

$$\tau_d = 1{,}5 \cdot \frac{15{,}54}{0{,}67 \cdot 1450} = 0{,}024 \text{ kN/cm}^2$$

$$f_{vd} = 0{,}8 \cdot \frac{0{,}07}{1{,}15} = 0{,}049 \text{ kN/cm}^2$$

$$\eta = \frac{0{,}024}{0{,}049} = 0{,}50 < 1{,}0$$

SLS CO7/8

$$w_{inst,g} = 1{,}64 \cdot 0{,}1546 = 0{,}253 \text{ cm}$$

$$w_{inst,s} = 0$$

$$w_{inst,w} = 0$$

$$w_{inst,q} = 2{,}5 \cdot 0{,}2483 = 0{,}621 \text{ cm}$$

instantaneous deformation

$$w_{inst} = 0{,}253 + 0 + 0 + 0{,}7 \cdot 0{,}621 = 0{,}69 \text{ cm}$$

$$\max w_{inst} = \frac{438}{300} = 1{,}46 \text{ cm}$$

$$\eta = \frac{0{,}69}{1{,}46} = 0{,}47 < 1{,}0$$

final deformation

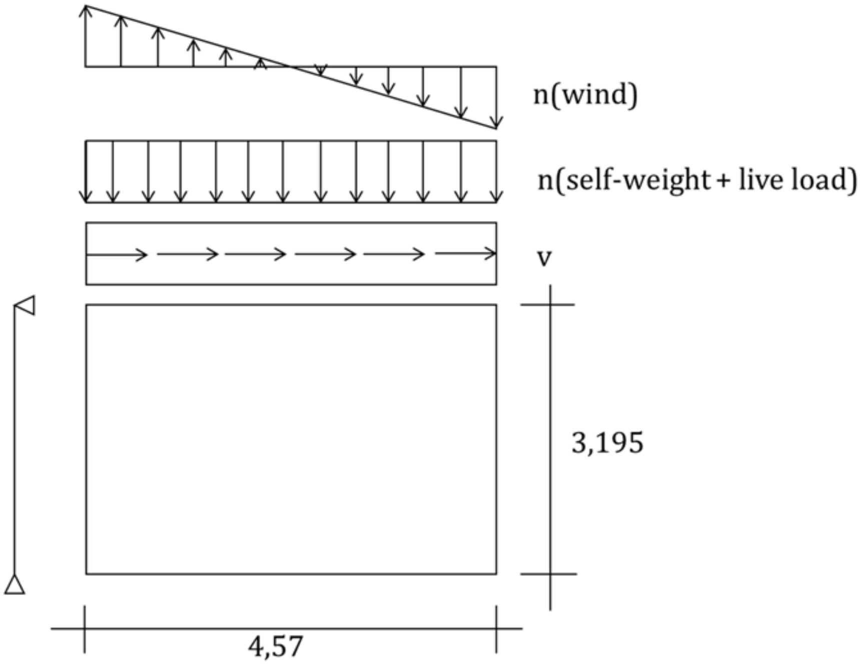
$$w_{fin} = 0{,}253 \cdot (1 + 0{,}6) + 0 + 0 + 0{,}621 \cdot (0{,}7 + 0{,}3 \cdot 0{,}6) = 0{,}95 \text{ cm}$$

$$\max w_{fin} = \frac{438}{150} = 2{,}92 \text{ cm}$$

$$\eta = \frac{0{,}95}{2{,}92} = 0{,}33 < 1{,}0$$

5.x9 Outer Wall

system



cross-section

$t = 145\ \mathrm{mm}$

$A = 1450\ \mathrm{cm^2/m}$

$I_y = 25405\ \mathrm{cm^4/m}$

$W_y = 3504\ \mathrm{cm^3/m}$

$i_y = \frac{14{,}5}{\sqrt{12}} = 4{,}19\ \mathrm{cm}$

material

- see technical approval [1]

CLT KL-trä 145

$f_{ck} = 8{,}3\ \mathrm{N/mm^2}$

$f_{mk} = 10{,}1\ \mathrm{N/mm^2}$

$k_{cr} = 0{,}67$

$E_{m,50} = 5290\ \mathrm{N/mm^2}$

$E_{c,05} = 2300\ \mathrm{N/mm^2}$

$k_{def} = 0{,}6$

internal forces

$n_{gk} = -68{,}6\ \mathrm{kN/m}$

$n_{sk} = -4{,}9\ \mathrm{kN/m}$

c) CLT Construction

$n_{wk} = -222{,}1 \text{ kN/m}$

- wind from the side becomes decisive

$n_{qk} = -50{,}7 \text{ kN/m}$

ULS CO6

$k_{mod} = 1{,}1$

$$n_{cd} = 1{,}2 \cdot 68{,}6 + 1{,}5 \cdot 222{,}1 + 1{,}5 \cdot 0{,}7 \cdot 4{,}9 + 1{,}5 \cdot 0{,}7 \cdot 50{,}7 = 473{,}7 \text{ kN/m}$$

$$\sigma_{cd} = \frac{473{,}7}{100 \cdot 14{,}5} = 0{,}33 \text{ kN/cm}^2$$

$$f_{cd} = 1{,}1 \cdot \frac{0{,}83}{1{,}15} = 0{,}79 \text{ kN/cm}^2$$

flexural buckling

$$\lambda = \frac{319{,}5}{4{,}19} = 76{,}3$$

$$\lambda_{rel} = \frac{76{,}3}{\pi} \cdot \sqrt{\frac{8{,}3}{2300}} = 1{,}46$$

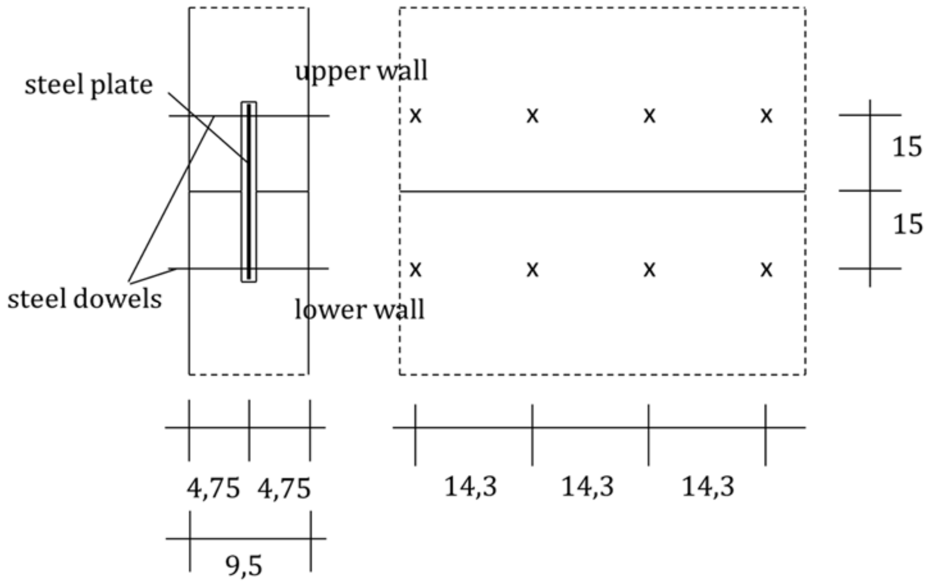
$$k = 0{,}5 \cdot (1 + 0{,}1 \cdot (1{,}46 - 0{,}3) + 1{,}46^2) = 1{,}62$$

$$k_c = \frac{1}{1{,}62 + \sqrt{1{,}62^2 - 1{,}46^2}} = 0{,}429$$

$$\eta = \frac{0{,}33}{0{,}429 \cdot 0{,}79} = 0{,}96 < 1{,}0$$

Anchoring Wall 5.y3

system



angle of the force to the grain

$\alpha = 90°$

fasteners

7 M20 dowels /m

$f_{yk} = 240\ \text{N/mm}^2$

- strength class 4.6

$f_{uk} = 400\ \text{N/mm}^2$

material

CLT KL-trä 170

$\rho_k = 385\ \text{kg/m}^3$

minimum distances

$a_2 = 4 \cdot 2{,}0 = 8\ \text{cm}$

$a_{3t} = \max(7 \cdot 2{,}0 = 14\ ;8) = 14\ \text{cm}$

capacity per fastener per shear plane

$M_{y,Rk} = 0{,}3 \cdot 400 \cdot 20^{2{,}6}/1000 = 289{,}6\ \text{Nm}$

$$f_{h,0,k} = 0{,}082 \cdot (1 - 0{,}01 \cdot 20) \cdot 385$$
$$= 25{,}26\ \text{N/mm}^2$$

$k_{90} = 1{,}35 + 0{,}015 \cdot 20 = 1{,}65$

$$f_{h,90,k} = \frac{25{,}26}{1{,}65 \cdot \sin^2 90 + \cos^2 90}$$
$$= 15{,}3\ \text{N/mm}^2$$

$$F_{v,Rk} = 15{,}3 \cdot 170/2 \cdot 20 \cdot \left[\sqrt{2 + \frac{4 \cdot 289{,}6 \cdot 1000}{15{,}3 \cdot 20 \cdot (170/2)^2}} - 1\right]/1000 = 15{,}3\ \text{kN}$$

- failure mode g becomes decisive
- the axial capacity of the fasteners is neglected

ULS CO11/12

$k_{mod} = 1{,}1$

$F_d = 138{,}4\ \text{kN/m}$

- cf. RFEM analysis

$F_{v.Rk} = 15{,}3 \cdot 7 \cdot 2 = 214{,}4\ \text{kN/m}$

- 7 bolts and 2 shear planes per m

$$F_{v,Rd} = 1{,}1 \cdot \frac{214{,}4}{1{,}3} = 181{,}5\ \text{kN}$$

$$\eta = \frac{138{,}4}{181{,}5} = 0{,}76 < 1{,}0$$

check for tension perpendicular to the grain in the beam

$V_d \approx 0{,}61 \cdot 138{,}4 \cdot 0{,}143 = 12{,}1\ \text{kN}$

- the shear force is estimated assuming the wall to be a beam supported by the dowels with the uniform load F_d

$$F_{90Rk} = 14 \cdot 95 \cdot 1 \cdot \sqrt{\frac{150}{1 - \frac{150}{319{,}5}}}/1000 = 16{,}7\ \text{kN}$$

$$F_{90Rd} = 1{,}1 \cdot \frac{16{,}7}{1{,}3} = 14{,}1\ \text{kN}$$

$$\eta = \frac{12{,}1}{14{,}1} = 0{,}85 < 1{,}0$$

Appendix

Attachments

- Eurocode design (XLSX)
- RFEM results for the panel construction (XLSX)
- RFEM results for the CLT construction (XLSX)
- RFEM models for the frame, panel and CLT construction (RF5)
- technical plans for the frame construction Pa1.2, Pa2 (PDF)
- technical plans for the panel construction Pb1–Pb3, Pa4.2 (PDF)
- technical plans for the CLT construction Pc1–Pc2, Pc3.2 (PDF)

a) Frame Construction

Elements

		joists 3.1 roof	joists 3.1 floor	joists 3.2 roof	joists 3.2 floor	beam 2.2 roof	beam 2.2 floor
decisive CO ULS	-	CO3	CO2	CO4	CO2	CO3	CO2
max eta ULS	-	0,604562492	0,417695102	0,115496973	0,112132553	0,639329492	0,485770077
decisive CO SLS	-	CO9/10	CO7/8	CO7/8	CO7/8	CO9/10	CO7/8
max eta SLS	-	0,838635072	0,52514617	0,058997644	0,048683764	0,225926291	0,196513867
system							
max l	m	4,64	4,64	2,1	2,1	5,15	5,15
e	cm	65	65	65	65	-	-
b	cm	14	14	14	14	16	16
h	cm	24	24	24	24	42	42
A	cm^2	336	336	336	336	672	672
Wy	cm^3	1344	1344	1344	1344	4704	4704
Iy	cm^4	16128	16128	16128	16128	98784	98784
Wz	cm^3	-	-	-	-	-	-
Iz	cm^4	-	-	-	-	-	-
material	-	glulam	glulam	glulam	glulam	glulam	glulam
kcr	-	0,67	0,67	0,67	0,67	0,67	0,67

		beam 2.1 roof	beam 2.1 floor	beam 2.3 roof	beam 2.3 floor	beam 2.4 front wind f the front	beam 2.4 side wind f the front
decisive CO ULS	-	CO3	CO2	CO3	CO2	CO5/6	CO5/6
max eta ULS	-	0,639329492	0,485770077	0,18445999	0,225690353	0,267572152	1,018398698
decisive CO SLS	-	CO9/10	CO7/8	CO9/10	CO7/8	CO5/6	CO5/6
max eta SLS	-	0,225926291	0,196513867	0,070080913	0,090953667	0,314440053	0,875424484
system							
max l	m	5,15	5,15	5,15	5,15	5,15	4,64
e	cm	-	-	-	-	-	-
b	cm	16	16	16	16	16	16
h	cm	42	42	42	42	42	42
A	cm^2	672	672	672	672	672	672
Wy	cm^3	4704	4704	4704	4704	4704	4704
Iy	cm^4	98784	98784	98784	98784	98784	98784
Wz	cm^3	-	-	-	-	1792	1792
Iz	cm^4	-	-	-	-	14336	14336
material	-	glulam	glulam	glulam	glulam	glulam	glulam
kcr	-	0,67	0,67	0,67	0,67	0,67	0,67

		column 1.1 axis B	column 1.1 axis C	column 1.2	column 1.3	column 1.4	diagonal 4
decisive CO ULS	-	CO2	CO2	CO5/6	CO2	CO5/6	CO5/6
max eta ULS	-	0,606023779	0,620680368	0,239213538	0,379254424	0,584022458	0,308514138
decisive CO SLS	-						
max eta SLS	-						
system							
max l	m	3,195	3,195	3,195	3,195	3,195	3,195
e	cm	-	-	-	-	-	-
b	cm	40	40	40	40	40	40
h	cm	40	40	60	40	40	40
A	cm^2	1600	1600	2400	1600	1600	1600
Wy	cm^3	10666,66667	10666,66667	24000	10666,66667	10666,66667	10666,66667
Iy	cm^4	213333,3333	213333,3333	720000	213333,3333	213333,3333	213333,3333
Wz	cm^3	10666,66667	10666,66667	16000	10666,66667	10666,66667	10666,66667
Iz	cm^4	213333,3333	213333,3333	320000	213333,3333	213333,3333	213333,3333
material	-	glulam	glulam	glulam	glulam	glulam	glulam
kcr	-	0,67	0,67	0,67	0,67	0,67	0,67

		facade carriers 6	sheathing floor 5
decisive CO ULS	-	CO5/6	CO2
max eta ULS	-	0,244810443	0,33897373
decisive CO SLS	-	CO5/6	CO7/8
max eta SLS	-	0,406609784	0,953559028
system			
max l	m	3,195	0,65
e	cm	60	-
b	cm	6	100
h	cm	16	1,8
A	cm^2	96	180
Wy	cm^3	256	54
Iy	cm^4	2048	48,6
Wz	cm^3	96	-
Iz	cm^4	288	-
material	-	glulam	plywood
kcr	-	0,67	1

Connections

		2.2 (B) / 1.1 roof	2.2 (C) / 1.1 roof	2.2 (B) / 1.1 floor
decisive CO ULS	-	CO3	CO3	CO2
max eta ULS	-	0,98606405	0,97825065	0,722277848
system				
no of wood members	-	3	3	3
no of steel plates	-	0	0	0
no of shear planes	-	2	2	2
type of connection	-	wood-wood	wood-wood	wood-wood
t1	cm	16	16	16
material 1	-	glulam	glulam	glulam
rhok1	kg/m^3	385	385	385
t2	cm	40	40	40
material 2	-	glulam	glulam	glulam
rhok2	kg/m^3	385	385	385
treq	cm			
no of fasteners	-	11	11	11
fastener type	-	bolts	bolts	bolts
d	mm	20	20	20
dhead	mm			
l	mm	-	-	-
inclination	°	-	-	-
lef	mm	-	-	-
req lef	mm	-	-	-
strength class	-	4,6	4,6	4,6
fyk	N/mm^2	240	240	240
fuk	N/mm^2	400	400	400
nef	-	11	11	11
force angle to the grain	°	90	90	90
b(shear members)	cm	32	32	32
he(shear members)	cm	33	33	33
h(shear members)	cm	46	46	46
minimum distances				
a1	cm	8	8	8
a3t	cm	14	14	14
a3c/a1c	cm	14	14	14
a2	cm	8	8	8
a4t	cm	8	8	8
a4c/a2c	cm	6	6	6

		2.2 (C) / 1.1 floor	2.4 side (B) / 1.3 vertical loads	2.4 side (B) / 1.3 horizontal loads
decisive CO ULS	-	CO2	CO1	CO5/6
max eta ULS	-	0,750255082	0,482120575	0,546268705
system				
no of wood members	-	3	2	2
no of steel plates	-	0	0	0
no of shear planes	-	2	1	1
type of connection	-	wood-wood	wood-wood	wood-wood
t1	cm	16	16	16
material 1	-	glulam	glulam	glulam
rhok1	kg/m^3	385	385	385
t2	cm	40	40	40
material 2	-	glulam	glulam	glulam
rhok2	kg/m^3	385	385	385
treq	cm		9,6	9,6
no of fasteners	-	11	9	10
fastener type	-	bolts	screws	screws
d	mm	20	8	8
dhead	mm		16	16
l	mm	-	450	450
inclination	°	-	45	45
lef	mm	-	223,72583	223,72583
req lef	mm	-	48	48
strength class	-	4,6	4,6	4,6
fyk	N/mm^2	240	240	240
fuk	N/mm^2	400	400	400
nef	-	11	7,224674056	7,943282347
force angle to the grain	°	90	90	90
b(shear members)	cm	32	16	46
he(shear members)	cm	33	34	8
h(shear members)	cm	46	46	16
minimum distances				
a1	cm	8	5,6	5,6
a3t	cm	14	-	-
a3c/a1c	cm	14	8	8
a2	cm	8	4	4
a4t	cm	8	-	-
a4c/a2c	cm	6	3,2	3,2

		2.4 side (B) / 1.3 combined	4 / 1.2 diagonal	4 / 1.2 column
decisive CO ULS	-	CO5/6	CO5/6	CO5/6
max eta ULS	-	0,809243565	0,793653072	1,10610114
system				with screws OK
no of wood members	-	2	4	4
no of steel plates	-	0	3	3
no of shear planes	-	1	6	6
type of connection	-	wood-wood	wood-steel	wood-steel
t1	cm	16	10	10
material 1	-	glulam	glulam	glulam
rhok1	kg/m^3	385	385	385
t2	cm	40		
material 2	-	glulam		
rhok2	kg/m^3	385		
treq	cm	9,6		
no of fasteners	-	9 + 5	10	10
fastener type	-	screws	dowels	dowels
d	mm	8	20	20
dhead	mm	16		
l	mm	450	-	-
inclination	°	45	-	-
lef	mm	223,72583	-	-
req lef	mm	48	-	-
strength class	-	4,6	4,6	4,6
fyk	N/mm^2	240	240	240
fuk	N/mm^2	400	400	400
nef	-		7,934843957	6,737669525
force angle to the grain	°		0	36
b(shear members)	cm			40
he(shear members)	cm			16
h(shear members)	cm			60
minimum distances				
a1	cm		10	9,236067977
a3t	cm		14	14
a3c/a1c	cm		6	16,45798706
a2	cm		6	6
a4t	cm		6	6,513274123
a4c/a2c	cm		6	6

		3 / 2	
decisive CO ULS	-	CO5/6	
max eta ULS	-		**0,752491153**
system			
no of wood members	-		1
no of steel plates	-		1
no of shear planes	-		1
type of connection	-	wood-steel	
t1	cm		8
material 1	-	glulam	
rhok1	kg/m^3		385
t2	cm		
material 2	-		
rhok2	kg/m^3		
treq	cm		
no of fasteners	-		10
fastener type	-	nails	
d	mm		3,4
dhead	mm		
l	mm	-	
inclination	°	-	
lef	mm	-	
req lef	mm	-	
strength class	-		
fyk	N/mm^2		
fuk	N/mm^2		600
nef	-		10
force angle to the grain	°		90
b(shear members)	cm		16
he(shear members)	cm		33
h(shear members)	cm		46
minimum distances			
a1	cm		1,7
a3t	cm		3,4
a3c/a1c	cm		3,4
a2	cm		1,7
a4t	cm		2,38
a4c/a2c	cm		1,7

b) Panel Construction

Elements

		joists 1.1 roof	joists 1.2 roof	joists 1.3 roof	joists 1.1 floor	joists 1.2 floor	joists 1.3 floor
decisive CO ULS	-	CO3	CO3	CO4	CO2	CO2	CO2
max eta ULS	-	0,692070891	0,509530317	0,195227212	0,605177273	0,437740141	0,238837368
decisive CO SLS	-	CO9/10	CO9/10	CO7/8	CO7/8	CO7/8	CO7/8
max eta SLS	-	1,060073588	0,661014086	0,09912381	1,015210287	0,624534816	0,139068916
system							
max l	m	5,15	4,38	2,1	5,15	4,38	2,1
e	cm	60	60	60	60	60	60
b	cm	16	16	8	16	16	8
h	cm	24	24	24	20	20	20
A	cm^2	384	384	192	320	320	160
Wy	cm^3	1536	1536	768	1066,666667	1066,666667	533,3333333
Iy	cm^4	18432	18432	9216	10666,66667	10666,66667	5333,333333
Wz	cm^3						
Iz	cm^4						
min i	cm						
lambda	-						
lambdarel	-						
k	-						
kc	-						
material	-	glulam	glulam	glulam	glulam	glulam	glulam
kcr	-	0,67	0,67	0,67	0,67	0,67	0,67

		roof beam 2 axis 4/A-C	floor beam 2 axis 4/A-C	roof beam 2 axis 1-3/A	floor beam 2 axis 1-3/A	roof beam 2 axis 3-5/C
decisive CO ULS	-	CO3	CO2	CO3	CO2	CO3
max eta ULS	-	0,230297691	0,405485628	0,574357926	0,549173227	0,897490131
decisive CO SLS	-	CO9/10	CO7/8	CO9/10	CO7/8	CO9/10
max eta SLS	-	0,076162969	0,076854053	0,055425855	0,033768636	0,224454457
system						
max l	m	2,5	2,5	1,79	1,79	2,1
e	cm	-	-	-	-	-
b	cm	12	8	12	8	12
h	cm	24	24	24	24	24
A	cm^2	288	192	288	192	288
Wy	cm^3	1152	768	1152	768	1152
Iy	cm^4	13824	9216	13824	9216	13824
Wz	cm^3					
Iz	cm^4					
min i	cm					
lambda	-					
lambdarel	-					
k	-					
kc	-					
material	-	glulam	glulam	glulam	glulam	glulam
kcr	-	0,67	0,67	0,67	0,67	0,67

		floor beam 2 axis 3-5/C	beam 6 roof	beam 6 floor	columns 5 A/D	columns 5 B/C
decisive CO ULS	-	CO2	CO3	CO2	CO2	CO2
max eta ULS	-	0,839293288	0,850237914	0,806340575	0,398129788	0,681499837
decisive CO SLS	-	CO7/8	CO9/10	CO7/8	CO5/6	CO5/6
max eta SLS	-	0,281657363	0,256758536	0,265285187	0	0
system						
max l	m	2,1	2,32	2,32	3,195	3,195
e	cm	-	-	-	-	-
b	cm	8	12	8	16	20
h	cm	24	24	24	20	20
A	cm^2	192	288	192	320	400
Wy	cm^3	768	1152	768	1066,666667	1333,333333
Iy	cm^4	9216	13824	9216	10666,66667	13333,33333
Wz	cm^3				853,3333333	1333,333333
Iz	cm^4				6826,666667	13333,33333
min i	cm				4,618802154	5,773502692
lambda	-				69,17377913	55,3390233
lambdarel	-				1,100934888	0,88074791
k	-				1,146075558	0,916895836
kc	-				0,682799502	0,853380864
material	-	glulam	glulam	glulam	glulam	glulam
kcr	-	0,67	0,67	0,67	0,67	0,67

Connections

		anchoring	1 / 2 roof	1 / 2 floor
decisive CO ULS	-	CO11/12	CO3	CO2
max eta ULS	-	1,027131903	0,915590272	0,441283577
system				
no of wood members	-	3	2	2
no of steel plates	-	2	0	0
no of shear planes	-	4	1	1
type of connection	-	wood-steel	wood-wood *joist hanger **	wood-wood *joist hanger **
t1	cm	6,7		
material 1	-	glulam		
rhok1	kg/m^3	385		
t2	cm	6,7		
material 2	-	glulam		
rhok2	kg/m^3	385		
treq	cm			
no of fasteners	-	8	1	1
fastener type	-	dowels		
d	mm	20		
dhead	mm			
l	mm	-		
inclination	°	-		
lef	mm	-		
req lef	mm	-		
strength class	-	4,6		
fyk	N/mm^2	240		
fuk	N/mm^2	400		
nef	-	6,522260638	1	1
force angle to the grain	°	0	90	90
b(shear members)	cm		16	16
he(shear members)	cm		16,5	16,5
h(shear members)	cm		24	24
minimum distances				
a1	cm	10		
a3t	cm	14		
a3c/a1c	cm	6		
a2	cm	6		
a4t	cm	6		
a4c/a2c	cm	6		

		2 / 3 roof axis 4/A-C support B	2 / 3 floor axis 4/A-C support B
decisive CO ULS	-	CO3	CO2
max eta ULS	-	0,434154318	0,68171065
system			
no of wood members	-	2	2
no of steel plates	-	0	0
no of shear planes	-	1	1
type of connection	-	wood-wood	wood-wood
t1	cm	12	8
material 1	-	glulam	glulam
rhok1	kg/m^3	385	385
t2	cm	12	8
material 2	-	glulam	glulam
rhok2	kg/m^3	385	385
treq	cm		
no of fasteners	-	4	4
fastener type	-	dowels	dowels
d	mm	20	20
dhead	mm		
l	mm	-	-
inclination	°	-	-
lef	mm	-	-
req lef	mm	-	-
strength class	-	4,6	4,6
fyk	N/mm^2	240	240
fuk	N/mm^2	400	400
nef	-	2,939095209	2,939095209
force angle to the grain	°	90	90
b(shear members)	cm	12	8
he(shear members)	cm	18	18
h(shear members)	cm	24	24
minimum distances			
a1	cm	6	6
a3t	cm	14	14
a3c/a1c	cm	14	14
a2	cm	6	6
a4t	cm	10,28318531	10,28318531
a4c/a2c	cm	6	6

		2 / 3 roof axis 1-3/A support C	2 / 3 floor axis 1-3/A support C
decisive CO ULS	-	CO3	CO2
max eta ULS	-	0,839082531	0,932453464
system			
no of wood members	-	2	2
no of steel plates	-	0	0
no of shear planes	-	1	1
type of connection	-	wood-wood	wood-wood
t1	cm	12	8
material 1	-	glulam	glulam
rhok1	kg/m^3	385	385
t2	cm	12	8
material 2	-	glulam	glulam
rhok2	kg/m^3	385	385
treq	cm		
no of fasteners	-	6	4
fastener type	-	dowels	dowels
d	mm	20	20
dhead	mm		
l	mm	-	-
inclination	°	-	-
lef	mm	-	-
req lef	mm	-	-
strength class	-	4,6	4,6
fyk	N/mm^2	240	240
fuk	N/mm^2	400	400
nef	-	4,408642813	2,939095209
force angle to the grain	°	90	90
b(shear members)	cm	12	8
he(shear members)	cm	18	18
h(shear members)	cm	24	24
minimum distances			
a1	cm	6	6
a3t	cm	14	14
a3c/a1c	cm	14	14
a2	cm	6	6
a4t	cm	8	8
a4c/a2c	cm	6	6

c) CLT Construction

Elements

		roof slab 1	floor slab 1	roof slab 2, 3	floor slab 2, 3	roof slab 4	floor slab 4
decisive CO ULS	-	CO3	CO2	CO3	CO2	CO3	CO2
max eta ULS	-	0,814243812	0,536717582	0,43774764	0,291731817	0,18616872	0,216985585
decisive CO SLS	-	CO9/10	CO7/8	CO9/10	CO7/8	CO9/10	CO7/8
max eta SLS	-	0,777777773	0,598859315	0,733897709	0,437945625	0,292693209	0,288282631
system							
max l	m	4,38	4,38	5,15	5,15	2,1	2,1
e	cm	-	-	-	-	-	-
b	cm	100	100	100	100	100	100
h	cm	14,5	14,5	20,9	20,9	9,5	9,5
A	cm^2	1450	1450	2090	2090	950	950
Wy	cm^3	3504,166667	3504,166667	7280,166667	7280,166667	1504,166667	1504,166667
Iy	cm^4	25405,20833	25405,20833	76077,74167	76077,74167	7144,791667	7144,791667
iy	cm						
Wz	cm^3						
Iz	cm^4						
min i	cm						
lambda	-						
lambdarel	-						
k	-						
kc	-						
material	-	CLT	CLT	CLT	CLT	CLT	CLT
kcr	-	0,67	0,67	0,67	0,67	0,67	0,67

		walls	
			x9
decisive CO ULS	-	x9	CO6
max eta ULS	-	0,96008	0,96008
decisive CO SLS	-		
max eta SLS	-		
system			
max l	m		3,195
e	cm		
b	cm		100
h	cm		14,5
A	cm^2		1450
Wy	cm^3		3504,17
Iy	cm^4		25405,2
iy	cm		4,18579
Wz	cm^3		
Iz	cm^4		
min i	cm		4,18579
lambda	-		76,3297
lambdarel	-		1,45955
k	-		1,62312
kc	-		0,42859
material	-		CLT
kcr	-		0,67

Connections

		anchoring y3	anchoring x26	anchoring x23
decisive CO ULS	-	CO11/12	CO11/12	CO11/12
max eta ULS	-	0,854177482	0,63966675	0,346353301
system				
no of wood members	-	2	2	2
no of steel plates	-	1	1	1
no of shear planes	-	2	2	2
type of connection	-	wood-steel	wood-steel	wood-steel
t1	cm	8,5	8,5	7,25
material 1	-	CLT	CLT	CLT
rhok1	kg/m^3	385	385	385
t2	cm	8,5	8,5	7,25
material 2	-	CLT	CLT	CLT
rhok2	kg/m^3	385	385	385
treq	cm			
no of fasteners	-	7	7	7
spacing e	cm	14,28571429	14,28571429	14,28571429
fastener type	-	bolts	bolts	bolts
d	mm	20	20	20
dhead	mm			
l	mm	-	-	-
inclination	°	-	-	-
lef	mm	-	-	-
req lef	mm	-	-	-
strength class	-	4,6	4,6	4,6
fyk	N/mm^2	240	240	240
fuk	N/mm^2	400	400	400
nef	-	7	7	7
force angle to the grain	°	90	90	90
b(shear members)	cm	9,5	9,5	9,5
he(shear members)	cm	15	15	15
h(shear members)	cm	319,5	319,5	319,5
minimum distances				
a1	cm	8	8	8
a3t	cm	14	14	14
a3c/a1c	cm	14	14	14
a2	cm	8	8	8
a4t	cm	8	8	8
a4c/a2c	cm	6	6	6