# Timber stadium engineering

## a feasibility study

Master's Thesis

## Final Report

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bam utiliteitsbouw

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# Preface

This thesis is all about the feasibility of timber stadium engineering. It concludes the Master of Science programme at the section of Building Engineering of the Faculty of Civil Engineering and Geosciences at Delft University of Technology.

The research carried out in this thesis should be considered as a first step towards a football stadium structure that is fully composed of timber elements. To the author's knowledge no such stadium has ever been built yet; something that will hopefully change in the near future.

The research has been carried out at BAM Advies & Engineering in Bunnik, where I had all facilities required at my disposal. I am grateful for the support and guidance that was offered to me by the company and its employees.

I would like to express my gratitude to my supervisors, Prof. Dr. Ir. J.W.G. van de Kuilen, Ir. G.J.P. Ravenshorst, Ir. P.A. de Vries, Ir. S. Pasterkamp and Ir. V. van der Wal, for their supervision.

In addition, I would like to thank all my colleagues at BAM Advies & Engineering; in particular Wilbert Diepenhorst for the interesting discussions concerning the subject; Arend Rutgers for his advise on the architectural design of the Euroborg stadium and Mark Spanenburg for sharing his knowledge on structural mechanics.

Finally, I would like to thank Hanneke, my family and my friends for their curiosity, their support and their patience.

Eindhoven, October 2011,

T. van den Boogaard

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# Abstract

Having a society that is continuously demanding for higher standards towards sustainability and sustainable construction, the building industry has adopted this responsible approach. An important aspect is found in the choice for material, and that is where the naturally renewable material timber comes into play.

In recent years, various types of plate-like timber materials have been introduced to the building industry, offering a wide range of new possibilities. Due to a lack of knowledge on these possibilities, in the Netherlands, the material is not used as often as could be expected.

To explore the boundaries of timber engineering, in this thesis, it is investigated whether it is feasible to engineer a stadium structure that is composed of timber elements to the utmost extent. The focus within this feasibility study is on the structural system, the floors and the grandstands. The roof structure is not included, since it appeared that such structures are already common in practice. As a reference, use is made of the architectural design of the Euroborg stadium, which was designed and engineered by BAM Advies & Engineering in recent years.

The first part of the thesis provides an introduction to the most important aspects concerning the design and engineering of a stadium structure. Next to that, reference projects are discussed to provide insight in the possibilities of timber as a structural material.

The preliminary design is elaborated on in the second part of the thesis. Investigating several structural systems it is found that a timber core system proves to be most beneficial. Decisive factors are the excellent fitting to the existing architectural design and the ease of erection. The proposed cores have a height of 19.22 m and are compiled from 4 walls, having a width of 5 m each, which are made from LVL elements.

Analysing various floor products that are available on the market, timber hollow core elements called Lignatur appeared to be most beneficial to be implemented in the design. These span 14.4 m between the front and the back façades of the stadium structure and are supported by glued laminated timber support beams. Accounting for the required free height at the distinct floors, the span-width of these beams is limited to 9.65 m.

When the grandstands are concerned, 4-tier elements made from LVL are introduced. Accounting for grandstand elements with an increased riser height the maximum span-width becomes 10.4 m, where vibrations are governing and under the condition that the grandstand support beams are infinitesimally stiff. Accounting for Glulam support beams, the maximum span-width decreases to a certain extent.

In the detailed design phase, the feasibility of the proposed structural system is considered more in detail. At first, the structural behaviour lintels that are present in the core walls are investigated. It appears that these comply with all requirements.

Evaluating the preliminary design phase, an expert on construction technology posed questions on the erection of timber cores with the proposed dimensions. It has therefore been investgated what structural consequences it holds when the core is divided in storey-high segments. It appeared that several measures are required to transfer the acting stresses at the horizontal joints. A feasible solution is acquired by applying a combined system consisting of glued-in rods and shear plates.

Subsequently research has been performed on the distinct connections that are present at the LVL core. Due to the limited dimensions of the LVL elements (max 2.5 m), a large number of fasteners is required to obtain the required wall width of 5 m.

The above is considered undesirable, wherefore it was investigated whether the core walls could be compiled from CLT elements, which are available in widths up to 4.8 m. Accounting for the results

obtained, it was concluded that the structural behaviour of these elements is less beneficial than for LVL, but that they do comply with all requirements. The CLT core is therefore considered most beneficial.

As a final check, the CLT core walls have been checked on buckling between two storeys and on their behaviour under fire conditions. It appeared that the proposed core complies with all requirements.

With regard to the research question it is concluded that, from a structural point of view, a stadium of which the structure is compiled from timber elements is a feasible solution. Accounting for the functional requirements and the results obtained in this thesis, the optimal span-width of a timber stadium structure is set to 7.8 m.

The results as presented in this thesis are still preliminary, wherefore several recommendations are made to improve the design.

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# Introduction

## Background and motivation

These days, probably one of the most heard terms when it comes to overall development is sustainability or sustainable development. These terms can be best explained by the definition as stated by the Brundtland Commission, which explained sustainable development as development which "*meets the needs of the present without compromising the ability of future generations to meet their own needs*"[ 1 ]. To come to such development, there should be a more or less ideal equilibrium between ecological, economical and social interests.

In the building industry, the focus is merely on sustainable construction and sustainable materials. An important role at this is taken by the government which stimulates the industry to build 'greener' by offering allowances: *"the government wants to stop non-sustainable trends by supporting sustainable developers, by national and international cooperation and with a strong willingness towards innovation".* [ 2 ]. As a result, in recent years various initiatives have been set up to promote sustainable construction.

This responsible attitude has been taken up by various important players in the building industry, resulting in a changing approach towards projects. Initiators expect and demand their projects to be demonstrably sustainable, which obliges other parties to join them. On the other hand, engineering offices and contractors are well-aware of their social function and the impact of their moral behaviour on their reputation.

Sustainable construction is described by Prof. Dr. Ir. Hendriks [ 3 ] as *"a way of designing and constructing buildings that support human health (physical, psychological and social) and which is in harmony with nature, both animate and inanimate"*. On the field of materials, this approach can be enhanced by making use of naturally renewable materials and/or high-grade reusable materials.

When naturally renewable materials are at stake, one often comes up with wood. And indeed, the material can be regarded as being sustainable, but only if it is harvested both legally and with a sustainable approach. With the latter is meant, that the timber should be harvested from forests which are managed in such a way that harvesting never exceeds natural growth. As a result, the total amount of trees will 'never' diminish.

Despite that, the material wood is not being used so often in the building industry as one would expect. There are various reasons for that, but for now only the main reasons will be mentioned.

In the past, there was a lack of knowledge on quality grading of the material. As a result, the majority of the wood was being cultivated for low-grade applications, such as the paper industry. With the help of several trade associations these problems have now been overcome.

Another reason lies in the fact that people often tend to think that a lot of the wood is harvested not-sustainable or at least illegal. Especially when (tropical) hardwoods are concerned, this is one of the biggest concerns.

The last reason that is mentioned is found at the field of the engineers. Although timber is one of the oldest building materials, for example in The Netherlands, only few timber structures can be found. Engineers often tend to choose for commonly-used materials like steel and concrete. As a result, also the knowledge about structural timber engineering is less spread, i.e. only few offices are able to (and often specialised in) design timber structures.

In response to the earlier described call for sustainability, this approach has become one of the major aspects when the organisation of a major sports event is at stake. The Olympic games of Sydney in 2000

were the first Olympics in which sustainability played a vital role. From then on, the demands of the International Olympic Committee have only become higher.
In recent years, also the Fédération International de Football Association (abb. FIFA) has adopted this sustainable approach. As a consequence, the organising committees for upcoming World Championships of football are obliged to include sustainability within their plans.

To prepare their selves to this upcoming future, the idea was developed at BAM Utiliteitsbouw to design a timber football stadium. Since no such stadium has been built according to modern standards, it should be investigated whether such a stadium is structurally feasible.

## Problem definition and research goal

As may be clear from the analysis above, this thesis is about the feasibility of timber being the main construction material in stadium engineering. Since no such structure has ever been designed in timber, there is a lack of knowledge on the possibilities of using the material for this purpose.

### Research goal

It is aimed to set the first step towards a stadium structure that is completely composed of structural timber. Formulated in other words, the main goal of this thesis is defined as follows:

*The main goal of this thesis is to gain knowledge, and to obtain a first impression, on the possibilities of utilising structural timber in football stadium engineering.*

The finished thesis is meant to provide BAM Advies & Engineering (abb. BAM A&E) with a first impression on the boundaries of timber stadium engineering. Due to the limited time span, it proved to be impossible to cover all relevant aspects.

### Research question

Making use of the above defined research goal, the main research question can be formulated:

*To what extent is the utilisation of structural timber in football stadium engineering feasible?*

As a stadium structure covers a wide range of aspects, this research question is only answered partially in this thesis: it proved to be impossible to provide a complete answer on all aspects within the given time span. Therefore, the focus is laid on several main aspects, which showed up during the graduation process.

## Research subquestions

The main research question is answered by discussing several subquestions, which are presented here together with a short explanation:

- § *What kind of timber products are available to use? What are their main applications and what are their characteristics?*
  Research is carried out on the products that are available on the market, their strength properties and their availability. The research is carried out as a literature study, of which the results are presented in Annex A.

- § *How does the structural design of a 'traditional' football stadium look and what are the backgrounds on the design?*
  A proper structural design is, from the authors' point of view, only feasible, if the designer has some knowledge on the backgrounds of the architectural design. As examples, the theory behind sight lines, the guidelines on escape routes, etc. are mentioned.

  In addition, at BAM A&E, the structural design of most stadia is more or less the same. The backgrounds behind the choices within this standardized design, can obviously be used when making a design in timber.

  This question is answered by studying literature, by consulting architects and engineers at BAM A&E and by research on reference projects. The results are shown in chapter 1, chapter 2 and Annex B.

- § *What kind of structures can be achieved when making use of structural timber?*
  Research is carried out on reference projects to explore the structural possibilities of timber. During this research, the focus was laid on those structures that could function as a reference for the timber stadium. The results, which provide the reader with insight in the boundaries of structural timber, can be found in chapter 3.

- § *Which structural system in timber fits a stadium at best?*
  It is investigated which structural systems are suitable to be used in a stadium structure. For each of these systems a first design is made, making use of realistic loading conditions and calculation methods.

  Evaluating these systems and the obtained results, the most beneficial solution is determined. The results are shown in chapter 4 Structural system.

- § *What kind of structural configuration is most beneficial for the floors within a stadium structure?*
  Accounting for the functional requirements and the specific loading conditions that apply to a stadium structure, several floor configurations are investigated and evaluated. This results in a proposal for the floor configuration that suits the timber stadium structure. The results are shown in chapter 5 Floors.

- § *What span can be obtained when a timber grandstand is concerned?*
  A first design of a grandstand structure is made while accounting for the required shape of the grandstand and the specific load conditions. Subsequently, the maximum span-width of the structure is increased by improving the design. This results in a range of reachable span-widths, which is elaborated on in chapter 6 Grandstands.

To found the feasibility of re proposed structural system, the most beneficial system has been investigated more into detail. The results of this deepening are presented in the Detailed design part of this thesis.

By answering the presented subquestions, a (partial) answer is formulated to the research question as shown above. The answer to the research question is discussed in the Discussion.

## Scope

In this section the scope of the thesis is defined, i.e. which aspects are taken into consideration and which ones are neglected. These boundaries are defined since it is impossible to perform a complete, in-detail design of a timber stadium within the given period of time. Besides, not all aspects considering the design of a stadium are relevant for the purpose of this thesis. Therefore, it is defined here what is considered relevant and what is not.

### General

§ Due to the authors' background and education, the focus in this thesis is on the structural design of a football stadium.

§ All structural calculations are based on the Eurocodes. When the Eurocodes do not provide a satisfying procedure, use is made of the Dutch NEN. All functional requirements follow from the Dutch Building Decree.

§ Basis for the design is that the structure consists of timber elements to the utmost extent. Every member is designed in timber, unless it proves impossible. An exception can be found at the foundation, which is made in concrete.

### Architectural design

§ As a starting point for the structural design there is accounted for the functional plan of a reference stadium, since there is a lack of an architectural design. This reference stadium is considered a typical BAM stadium and is elaborated on in chapter 2 Design reference stadium.

§ This reference stadium has been designed by implementing knowledge gained by years of experience, and can therefore be considered 'optimised'. As a consequence, no thorough research is performed on the aspects involving the architectural design.

§ Arend Rutgers, architect at BAM A&E Architecten, is consulted when the architectural design of the stadium is at stake. He is assigned the fictive role of being the architect on one hand and the project's principal on the other hand, depending on the nature of the decision to be taken.

### Design focus

§ The design is focussed on a small part of the stadium structure, being a typical building part and a typical cross-section, rather than the stadium as a whole. Due to the repetitive character, the final proposal represents the whole stadium quite well.

§ The considered building part, or typical section, is generalised to a great extent: areas having very specific requirements (e.g. VIP-rooms, dressing rooms, etc.) and local 'disturbances', such as vomitories (entrances to the stands) and floor openings, are simply neglected.

§ The goal is to provide a first impression on the feasibility of a timber stadium structure, rather than to make a complete design. Therefore, not all relevant design aspects (e.g. temperature effects, etc.) are taken into consideration.

### Subjects of interest outside the scope

§ Costs
§ Sustainability of timber
§ Construction method
§ Timber specific aspects (difficult purification, water tightness, weathering effects, etc.)

## Thesis outline

This thesis is globally divided in three distinct phases, namely the research phase, the preliminary design phase and the detailed design phase.

### § Research phase

In the research part of this thesis, an introduction is given to football stadium engineering, the reference stadium that has been used is presented and finally, a first impression is provided on the feasibility of timber football stadium engineering.

### § Preliminary design phase

The part concerning the preliminary design provides a first global design of the stadium structure. In chapter 4 Structural system, various stabilizing systems are investigated, after which the most beneficial system is determined for this situation. In chapter 5 the focus is on the design of the floors within the stadium structure. This part of the thesis is enclosed by chapter 6, which elaborates on the grandstands.

### § Detailed design phase

The last part of this thesis concerns the detailed design. The focus is mainly on the chosen structural system. A more detailed analysis is performed on the system and several aspects that were neglected in the preliminary design phase are taken into consideration.

In chapter 7, there is elaborated on the 'lintels' above the openings of the proposed the stabilizing element. In chapter 8, the structural consequences of dividing the cores in segments are investigated. Chapter 9 deals with the influence of the required connections for the core walls on the structural behaviour of the proposed stabilizing system. In chapter 10, the proposed structural system is reconsidered, accounting for a different timber product. Finally, in chapter 11, some additional checks are performed concerning buckling of individual core walls and the fire resistance of the core.

The thesis is concluded by a design proposal and recommendations, which are presented in the Discussion part of the thesis.

For those with interest in the material timber itself, its behaviour and its appearances, reference is made to Annex A Study on literature.

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# Research

The main goal of the thesis is to determine whether a timber football stadium is structurally feasible. To be able to do so, at first, insight should be obtained on the requirements corresponding to a stadium structure and the possibilities of timber as a structural material.

In this part of the thesis, at first, a short introduction is given to stadium design and engineering, see chapter 1. This chapter provides the reader with the backgrounds on some major aspects that are at stake.

Thereafter, a reference stadium is investigated thoroughly in chapter 2 Design reference stadium. The architectural design of this reference stadium is used later on as a basis for the structural design in timber.

To provide insight in the possibilities of structural timber, several reference projects are mentioned in chapter 3. All of these projects show resemblance to the structural elements in the stadium structure in one way or another.

At the end of the research part of the thesis, the reader has been made aware of the backgrounds on stadium engineering and the possibilities of timber as a structural material.

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# 1 Introduction to stadium design and engineering

In this chapter, a short introduction is given on the design and engineering of a football stadium. Since it is not a purpose of this thesis to provide a complete survey on the recommendations and guidelines concerning stadium design, only the most important topics are discussed concisely.

Considering the earlier mentioned sub-questions, it is stated that this chapter provides an answer to the latter part of the following question:

§ *How does the structural design of a 'traditional' football stadium look and <u>what are the backgrounds on the design?</u>*

At first, there is elaborated on the history of modern stadium development. A short overview is provided on the development over the last era and the expected future. Furthermore, the most specific spatial requirements are shown, which follow directly from the FIFA requirements. In addition, something is said about the requirements on seating and visibility and their direct consequences.

Next to the architectural design, attention is paid to the structural design. Several structural solutions for stadia are mentioned first. Finally, the most important structural requirements are summarized.

## 1.1 Stadium generations

To understand the current developments in stadium design, one has to gain some insight in the history of modern stadia. In this section, an overview is provided on the various generations of stadia, as extracted from Rod Sheard's [ 4 ] theory on stadium generations. At first, the most important characteristics for each generation are shown.

According to Sheard, the first generation of stadia was built in the second half of the nineteenth century. They were meant to accommodate large numbers of spectators, without putting much effort in creating comfort and providing qualitative facilities. As an excellent example, White City stadium in London is mentioned, see figure 1.

Figure 1: White city stadium, 1908, London

These stadia had the main function to provide an enclosed podium to sport matches, by which income could be generated which facilitated the founding costs of the stadia. These did generally not offer any protection against weathering and wind, while the seating was provided by benches made of concrete or wood.

The second generation, from the 1930s, was highly influenced by television broadcasting. Though these stadia were still largely concrete bowls, facilities and comfort levels improved. Even today still a lot of these second generation stadia can be found. As an example the Olympic stadium of Berlin is referred at, see figure 2.

Figure 2: Olympic stadium, 1936, Berlin

These 2nd generation stadia were mainly 1st generation stadia which were improved to accommodate television broadcasting. This development influenced the attractiveness to visit the stadium in a bad manner. To change the tendency of decreasing spectator numbers, improvements were made on the facilities. This resulted in the stadia becoming social meeting places, instead of podia for sport games only.

Starting from the early 1990s, the third generation stadia were designed to attract whole families to the stadium. Next to a sport venue, numerous other facilities, such as bars and shops, were implemented. Since the major income shifted from ticketing to merchandise and television, the quality of spectator facilities rose enormously. One of the first examples is Stadio delle Alpi in Turin (Italy), see figure 3.

Figure 3: Stadio delle Alpi, 1990, Turin

Developments in the leisure industry made that entertainment had become available for the whole family: they could enjoy an event all together. Since most sport events only attracted mature males, stadia had to be accommodated with facilities for children, shopping malls, restaurants etc., to prevent visitor numbers to keep falling back.

Few years later, the design, funding and management of a stadium were integrated, which led to the stadium being a money maker by itself. Innovative techniques such as a sliding field, a retractable roof or moving tiers are excellent examples of the fourth generation's stadia. One of the first stadia which incorporated all these aspects was the Gelredome in Arnhem, see figure 4.

Figure 4: Gelredome, 1996, Arnhem

Due to the excessive amount of possibilities to be informed (live) about matches at any given location, a stadium visit will only remain interesting when it offers clear extras above being at home. The stadium should therefore incorporate all facilities that are present at home, while the ambiance and the overall experience take care of the desired extra dimension.

The fifth generation of stadia has finally taken possession of their place in society. These are the accelerators in the development of new urban areas as they attract all kinds of activities to their areas. To speak with Sheard, they *"have become powerful symbols of our culture, our aspirations and, sometimes, of our failures"*. One of the most striking examples, is the new Wembley in London, see figure 5.

Figure 5: Wembley, 2007, London

From the authors' point of view, even the sixth generation of stadia can already be distinguished. When big events like the World Cup of Football or the Olympic Games are considered, more and more effort is being put in the development of sustainable stadia. These stadia should be $CO_2$-neutral and should be self-supplying when energy is at stake. Another important aspect for these stadia, is the adaptability. The stadium should be able to ‘grow’ or ‘shrink’ depending on the specific event that it hosts. For the upcoming years, these will be the aspects which are governing during the design.

Taking the above in consideration, it could be concluded that the development in stadium design has been highly inspired by the main goal for its developers, which is generating as much money as possible. In chronological order, the following aspects have been implemented in the design of stadia: attracting as much spectators as possible; providing facilities for television broadcasting; combination of functions to attract whole families; multifunctional use; attracting trade and industry to the stadium’s direct environment.

On the other hand, one could say that the development is just in line with the demands of society. People have become used to a higher standard, which they expect to find in a stadium as well. In addition, society does not accept to-be-built stadia anymore, which will only be utilised twice a month. The development of stadia could therefore also be explained by the development of society.

In reality, the development as it happened followed from a combination of both: by adapting to the demands of society, larger, more varied, crowds could be attracted, leading to higher revenues. In addition, by implementing new ideas in the design of a stadium, also changes in society have appeared. As an example of the latter, multifunctional stadia can be named which turned the whole idea of the traditional stadium upside down.

Another striking aspect that can be extracted from the above is the fact that in recent years the development has accelerated. The first and the third generation followed a little 100 years after each other, while between the third and the fifth generation there were only about 15 years. At this time, the fifth generation is just a few years present, while one might already speak of a sixth generation.

As a result, one could argue that the demands of society towards stadia shift so quick, that it is of no use to design stadia for a period of 50 years anymore. This idea is even strengthened by the speed of technological development. What is state-of-the-art today can already be history tomorrow.

Without elaborating any further on this subject, one should note that these aspects do play a vital part in stadium design and engineering these (and presumably upcoming) days. Although not even directly applied, within this thesis, these aspects are kept in mind throughout the whole design stage.

## 1.2 Spatial requirements

### 1.2.1 Pitch dimensions

Pitch dimensions have a huge influence when making a stadium design. For example, a lawn tennis pitch requires much less space than a football field, and thus a totally different approach.

When a football field is concerned, the applying dimensions follow from the FIFA regulations. Though there is some margin in the exact dimensions, the purpose of the stadium is governing. For example, it is allowed to vary the pitch's length between 90 m and 120 m and the width between 45 m and 90 m. But if the stadium will be hosting international games, the margin becomes smaller, namely 100-110 m for the length and 64-75 m for the width. If the purpose of the stadium will be to host World Cup matches or International Cup finals, the appropriate dimensions become 105 m by 68 m. It goes without saying, that the purpose of the stadium should be considered before starting the design.

In addition, it is obligatory to keep an area adjacent to the pitch clear for means of safety. Behind the goals these areas should have a width of 6 m, on the sides only 3 m is required. The pitch surface should continue in these areas for at least 3 m and 2 m respectively.
Nevertheless, the FIFA recommends providing safety areas with a width of 10 m on the ends and 8.5 m on the sides. This would lead to an area of 125 m by 85 m, to be occupied by the pitch as a whole.

To complement this section, figure 6 shows the preferred lay-out of the pitch including safety zones, according to the FIFA. [ 5 ]

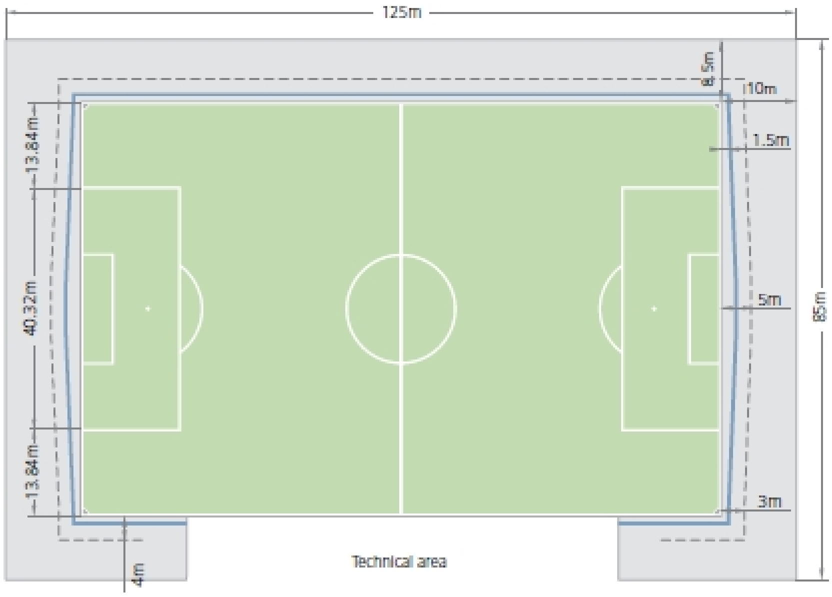


Figure 6: Pitch lay-out according to FIFA regulations

### 1.2.2 Commercial areas

As follows from the earlier explained stadium generation theory, today's stadia are more than just structures which facilitate spectators. Besides the provision of adequate seating, space has to be reserved for commercial display, sponsors, VIP's, media etc. Throughout the years, various instances have glanced at this subject leading to various design recommendations. Since most new stadia are being built to host a certain event, FIFA regulations have to be followed at least. A short impression on the spatial requirements for commercial areas in a FIFA world cup stadium is provided in table 1.

Table 1: Spatial requirements for commercial areas according to FIFA regulations

| | Stadium capacity | 80.000 | 60.000 | 40.000 | 40.000 | |
|---|---|---|---|---|---|---|
| | *Functional area* | *Opening and final* | *Semi-final* | *Quarter-final* | *Last 16 or group* | *Preferred location* |
| 1 | Commercial display [$m^2$] | 3.500 | 2.500 | 2.500 | 2.500 | Public area |
| 2 | VIP lounge [$m^2$] | 1.300 | 1.300 | 750 | 650 | Behind VIP seats |
| 3 | Sponsor village [$m^2$] | 35.000 | 20.000 | 10.000 | 8.000 | Close to the stadium |
| 4 | Commercial hospitality [$m^2$] | 50.000 | 20.000 | 10.000 | 9.000 | Inside stadium |
| 5 | Stadium media centre [$m^2$] | 6.000 | 4.000 | 4.000 | 4.000 | Behind media seats |
| 6 | TV compound [$m^2$] | 6.000 | 4.000 | 4.000 | 4.000 | Behind main stand |
| 7 | Accreditation centre [$m^2$] | 1.200 | 1.200 | 1.200 | 1.200 | Outside security area |
| 8 | Volunteers centre [$m^2$] | 400 | 400 | 400 | 400 | Inside stadium |
| | Total amount of $m^2$ | 103.400 | 53.400 | 32.850 | 29.750 | |

As can be seen, large areas should be reserved for commercial hospitality. Since these areas are revenue generating, this is beneficial to the stadium's viability. It should be noted that the sponsor village is also allowed to be situated nearby the stadium, reducing a considerable amount of the required space within the stadium.

### 1.2.3 Functional arrangement

Underneath and behind the grandstands of a stadium, often secondary functions are being housed, such as offices or shops. In addition, within the stadiums' structure, space for parking is sometimes required. All these spaces have specific demands on their free heights and their minimal dimensions. The exact requirements for various other functions follow from the Dutch Building Decree and these are elaborated on when they apply.

### 1.2.4 Capacity

When it comes to planning the capacity of a new stadium a lot of factors come into play. One of the most important steps is to perform studies on potential growth and recent visitor numbers. It is a golden rule never to increase a stadium's capacity beyond the expected (and demonstrable) number of future visitors. Besides, it has to be proved that a stadium with a certain capacity is affordable, both in founding costs and in running costs.

Another aspect that should be considered is the future purpose of the stadium. For example, the FIFA has strict demands on the stadium's capacity when hosting World Cup matches, see table 1. In case these demands do not comply with the expected amount of visitors for the stadium's future resident, a solution can be found by designing a (partly) demountable stadium.

As mentioned before, it should be kept in mind that the stadium remains affordable. The bigger the capacity of a stadium, the higher the costs per seat. For example, when constructing two tiers instead of one, the heavier the structure should be and the more cranes there are required. The relationship between capacity and costs is therefore not a linear one. The same can be said about operational and maintenance costs.

## 1.3 Structural solutions

### 1.3.1 Roof solutions

In this section, the most commonly used roof structures for stadia and sport venues are being discussed. The focus is on systems that could possibly be executed in timber. As a result, a lot of systems are already abandoned.

Subsequently, a post and beam system, a goal post system, a cantilever roof, a space frame, an arch-type roof and a grid shell are mentioned. After a short description, the (dis-)advantages of the system are named.

### Post and beam system

The post and beam systems consist of roof beams which are supported by a series of columns parallel to the pitch, both at the back of the grandstand and at the front. A schematisation can be found in figure 7.

- \+ Cheapest solution possible
- \+ Simple structure
- \- Columns at the front obstruct spectator viewing to a great extent

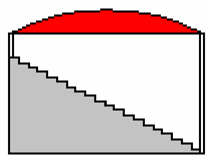

Figure 7: Post and Beam system

When modern time stadia are at stake, the latter is considered unacceptable. Therefore, these kind of roof structures are neglected in this thesis.

### Goal post system

When a goal post system is concerned, a large roof girder (or arch) spans the whole length of the grandstand parallel to the field. The girder (arch) is only supported at the ends of the grandstand. When this solution is applied to all four grandstands, the support beams do not intersect at the corners, i.e. each beam is supported by itself. A schematisation of this solution is shown in figure 8.

- \+ Unobstructed spectator viewing for most of the grandstand (near the corners the view might be disturbed somewhat)
- \+ Moderate costs
- \- Large bearings required at the corners
- \- Visually dividing the stadium in four distinct parts

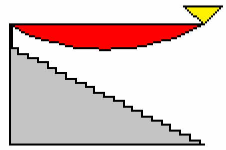

Figure 8: Goal post system

The main beam is usually an (arched) truss, under the requirement that it can span the length of the grandstand directly. With reference to section 1.2, it can be found that this length should be at least 125 m to comply with FIFA requirements.

### Cantilever system

The roof beam is fixed at one end, while its other end, facing the playing field, hangs unsupported. Similar systems can be obtained by restraining the cantilever with cables, or by adding an additional support near the back of the stand. A schematisation of the solution is shown in figure 9.

- \+ Unobstructed spectator viewing, independent on the grandstand size
- \+ Both suitable for bowl-shaped stadiums and isolated stands
- \- Costs increase rapidly with increasing cantilevering length
- \- Susceptible to wind uplift

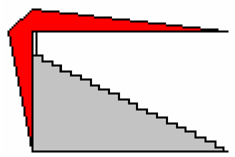

Figure 9: Cantilever system

The cantilever is usually obtained by making use of a truss. Examples are available of (steel) trusses that cantilever over 45 m (although against considerable costs). For smaller stadia which require grandstand enclosure only, this solution can be considered the most beneficial solution, since it is both cheap and aesthetically appealing. That being said, one can imagine that this solution is the most-widely used structural solution for stadium roofs.

### Space frames

A space frame is a three-dimensional grid composed of light-weight elements. The frame spans in two directions and is supported at its edges only. The top and bottom side of the grid are usually flat. A difference can be found between a true and a simulated space frame, see figure 10 and figure 11 respectively. The simulated system shows resemblance to the goal post system, except for that the girders do intersect at the corners.

- \+ Unobstructed spectator viewing
- \+ Suitable to span large distances in two directions

- \- Efficient design requires a length-to-width ratio smaller than 1.5
- \- Expensive
- \- Shade influences pitch quality in a bad manner

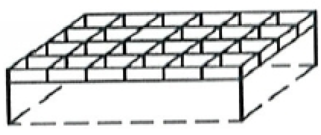

Figure 10: 'True' space frame

Due to the ratio between pitch (and thus stadium) length and width, space frames are usually not used for stadia. Despite that, there are some examples. In this thesis, this solution is neglected.

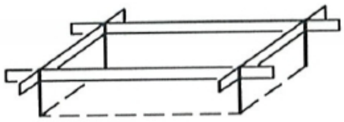

Figure 11: 'Simulated' space frame

### Arch-type roofs

Arched roofs can be found in various forms, such as barrel vaults or domes, but they all share the characteristic of being composed of several (half-)arches. The arches are often stabilized by adding purlins in between. On top of the arches a cladding is applied to enclose the roof. A schematisation of this system can be found in figure 12.

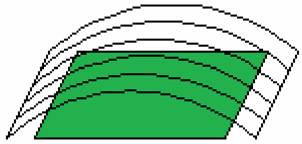

Figure 12: Arch-type roof

+ Highly efficient structural shape
+ Suitable to span large distances (in two directions)

- Fully enclosed roof influences pitch quality in a bad manner

Arch-type roofs are not commonly used for large-size stadium structures. Despite that, the structural shape is executed in timber regularly at small-size (indoor) sports venues. Due to the relatively low costs, it could be an interesting solution when a fully enclosed roof structure is required.

### Grid shell

A grid shell structure consists from a grid of relatively small and light-weight elements, deriving its strength from its double curvature. The strength-to-weight ratio is therefore very high. A schematisation of this solution can be found in figure 13.

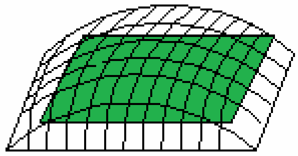

Figure 13: Grid shell

+ Unobstructed spectator viewing
+ Large spans in two directions possible

- Difficult design and construction process
- Fully enclosed roof influences grass quality in a bad manner
- Expensive

The grid is composed from light-weight materials, such as timber or steel. Although grid shells are not very common in practice, there are some impressive examples.

### Conclusions

Considering the various roof systems, it is concluded that the cantilever roof is the most beneficial solution when a partly enclosed stadium is concerned. In case a fully enclosed roof is required, the most beneficial solution, from the viewpoint of costs, is the arch-type roof.

## 1.3.2 Grandstand entrances

To enter the grandstands, two kind of entrances are distinguished, being a vomitory at the terrace level, see figure 14, or being the concourse behind the upper tiers, see figure 15.

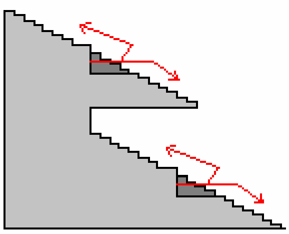

Figure 14: Vomitory-principle

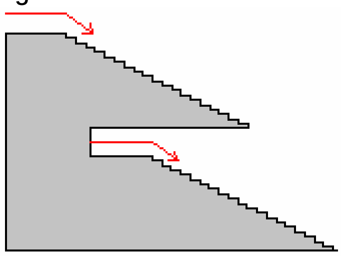

Figure 15: Concourse-principle

When a vomitory is concerned, the stands are reached directly at the lower tiers. A negative consequence is the occupation of precious space that could be used for seating as well. Entrance from the concourse is normally integrated in the gap between the lower and upper tier, except for when this space is occupied by hospitality areas. For the upper grandstand this implies that a concourse is required at the level of the top tiers, which is rather unpractical and expensive.

Considering both solutions, it is concluded that the Vomitory-principle offer the most beneficial solution.

## 1.4 Seating and visibility

Presumably, one of the most important aspects while experiencing a stadium visit, is spectator comfort. To a great extent, these comfort levels are obtained by providing comfortable seating and satisfying visibility, i.e. offering a clear and unobstructed view from every seat within the stadium.

In the last decades studies have been carried out on this subject, resulting in regulations and various design guidelines. As an introduction, some of these are shown in this section.

For seating, it is recommended by the FIFA to make use of the tip-up type of seating. Since the seating area will tip up when not being used, a wider space becomes available for the passageway. This increases safety and may decrease the space required for passing.

The NEN-EN 13200:1:2003 states that the length of one seating row, the so called bench length, is allowed to be 15 m at maximum. With the demand that each seat has a width of 500 mm, this leads to a maximum number of 28 seats per row.

Accounting for a seating block size of maximum 30 rows with 28 seats each, the required escape route will have a minimal width of 1.70 m (1.20 m per 600 persons required). It is mentioned that a width of 1.50 m per escape route is recommended. Above all, the width of the escape routes may never be decreased towards the outside of the stadium.

To avoid spectators being seated too distant from the action, the FIFA recommends that the majority of the seats is being located within a distance of 150 m from the most distant corner of the playing field. As a maximum, 190 m is accepted, see figure 16. [ 5] [ 6 ]

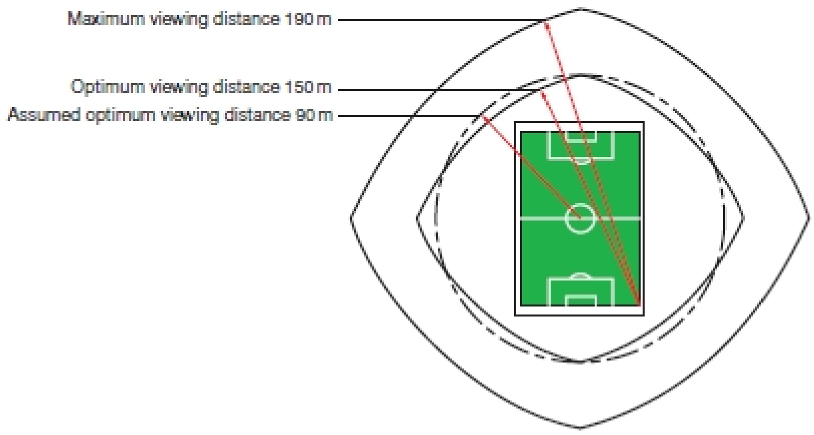


Figure 16: Guideline on viewing distances

Other important aspects which influence spectator viewing to a great extent, are the sightlines and the viewing angles. For the latter, in general, an angle of more than 34 degrees is regarded being too steep and thus unacceptable.

The sightlines are usually determined by making use of the so-called C-value. This value represents the distance (in mm) between one's sightline and the centre of the eye of the spectator in front, see figure 17.

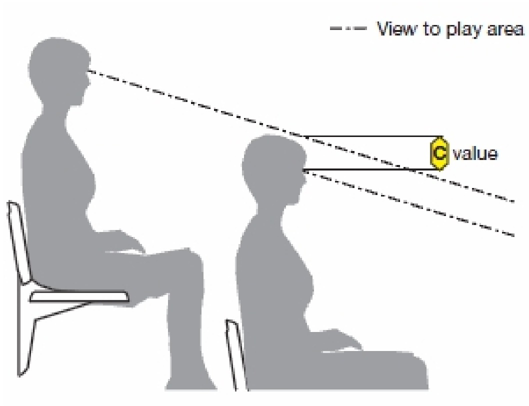


Figure 17: Explanation of the C-value

For the design of a to-be-built stadium, a C-value of 150 is classified as excellent, 120 as very good and 90 as an optimal design minimum. It has to be noticed that a higher C-value results into steeper stands, especially when the tiers are close to the playing field. The effect of the different C-values and various distances to the point of focus is shown in figure 18.

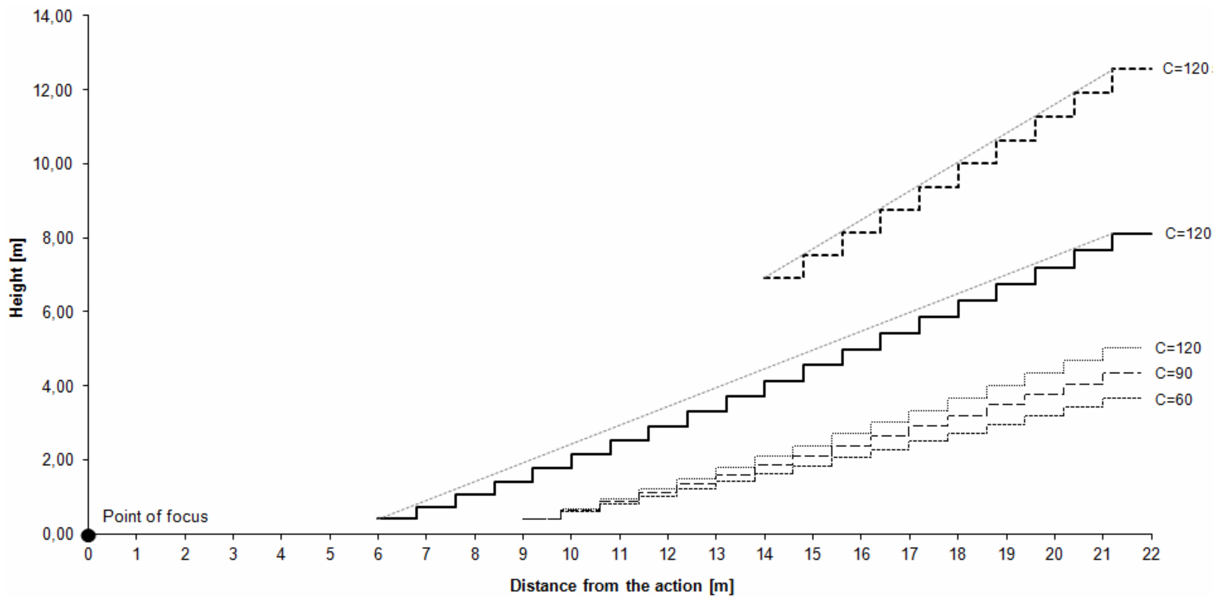


Figure 18: Effects of C-value and distance to point of focus on the shape of the grandstand

As it is preferred in Western Europe to be, as a spectator, as close to the field as possible, it is advised not to implement a running track within the stadium perimeter. Obviously, with the exception when planning to host the Olympics in the (near) future.

When the upward sightline is taken into consideration, the NEN-EN 13200:1:2003 demands that every spectator has a free sight to a height up to 15 m in the air, measured directly above the centre of the pitch.

## 1.5 Structural requirements

In this section, the most important backgrounds on the structural design of a stadium are mentioned. This overview does not claim completeness. Additional and in-detail information can be found in Annex B.

### 1.5.1 General

- § A stadium structure is classified as a life span class 4 building (reference period of 50 years)
- § Replaceable parts of the structure are classified as class 2 (reference period of 15 years)
- § A stadium is classified in reliability (and consequence) class 3, which implies high safety factors

- § For the grandstand and concourse, an imposed load of 5.0 $kN/m^2$ should be accounted for (large crowds)

### 1.5.2 Fire requirements

- § The main load-bearing structure should have a fire resistance of at least 90 minutes and should comply with Euro-Class B.
- § When the roof structure is concerned, no demands of fire resistance are made.
- § The evacuation time for a grandstand is set to 60 minutes.
- § The fire resistance of a grandstand should be at least 30 minutes.
- § All public floors should comply with Euro-class $C_{fl}$.
- § All other floors should comply with Euro-class $D_{fl}$.

### 1.5.3 Vibrations

- § The building code provides no clear guidance on the requirements concerning vibrations.
- § Consulting literature and experts, the minimum fundamental frequency of a grandstand is set to 5 Hz (no concerts allowed).

## 1.6 Conclusions

In the beginning of this chapter, it was stated that this chapter should provide an answer to the latter part of the following sub question:

*How does the structural design of a 'traditional' stadium look and <u>what are the backgrounds on the design?</u>*

In this chapter the following conclusions have been found:

- § Attracting as much spectators as possible is the main goal in stadium development.

- § A cantilever system is the most beneficial roof solution for partly enclosed stadium roofs
- § An arch-type roof is the most beneficial roof solution for fully enclosed stadium roofs
- § Vomitories are the most beneficial solution to provide access to the grandstands

- § Pitch dimensions and safety areas influence spectator visibility to a great extent.
- § To provide optimum viewing, all seats should be within a radius of 90 m from the centre of the pitch.
- § The maximum acceptable viewing angle is 34 degrees.
- § A C-factor of at least 90 should be accounted for, but 120 is recommended

- § The main structural requirements are summarized in section 1.5.

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# 2 Design reference stadium

## 2.1 Introduction

The great majority of current generation stadia are designed in concrete, steel or a combination of the above. Large scale timber stadia have not been designed yet. Therefore, at this certain moment, it is impossible to make a substantiated statement on the timber products to be used and optimal spans.

To investigate these affairs, use is made of the architectural design of a reference stadium. As mentioned earlier, when was elaborated on the scope of this thesis, the architectural design of the reference stadium is considered to be more or less unviable. In case the design in timber interferes with the architectural design of the reference stadium to a great extent, the project's principal is consulted.

Considering the research questions, it is stated that this chapter provides an answer to the first part of the following sub question:

- *<u>How does the structural design of a 'traditional' stadium look</u> and what are the backgrounds on the design?*

Therefore, at first, there is elaborated on suitable reference stadia. Thereafter, the chosen reference stadium is analysed, whereby the functional requirements, the structural elements and the structural system is elaborated on.

## 2.2 Reference Stadium

Conversations with various experts at BAM Advies & Engineering learned that three stadia could be considered as a reference project: the Gelredome stadium in Arnhem, the Veltins Arena in Gelsenkirchen and the Euroborg stadium in Groningen.

All three of them being, more or less, recently built stadia having a multifunctional character. As discussed in section 1.1, today's stadia should be more than just podia for sport events; something that is also be expected from a future timber stadium.

Another aspect that all three stadia have in common, is that they are engineered by BAM Advies & Engineering (back then known as HBG engineering). As a result, all the information which could possibly be of use is within hands.

Since the goal of this thesis is to obtain insight on the feasibility of timber as a structural material in a stadium, it is acceptable to make use of a simplified (typical) structure. It is therefore of no use, to include a retractable field or roof in the design at this stage. This only complicates the process, without improving the results. It could even be said that these complex parts of the structure influence the results in a bad way. Their presence puts high demands on the overall structural build-up, which is incompatible with a stadium that lacks them.

The choice is therefore made to make use of the Euroborg stadium as a reference, see figure 19. The stadium was completed only a few years ago, which implies that all data is still available at the company.

The stadium can be considered medium-size, which makes it feasible to investigate, while still a wide variety of aspects is present within the stadium. This leads to the Euroborg being the ideal reference stadium.

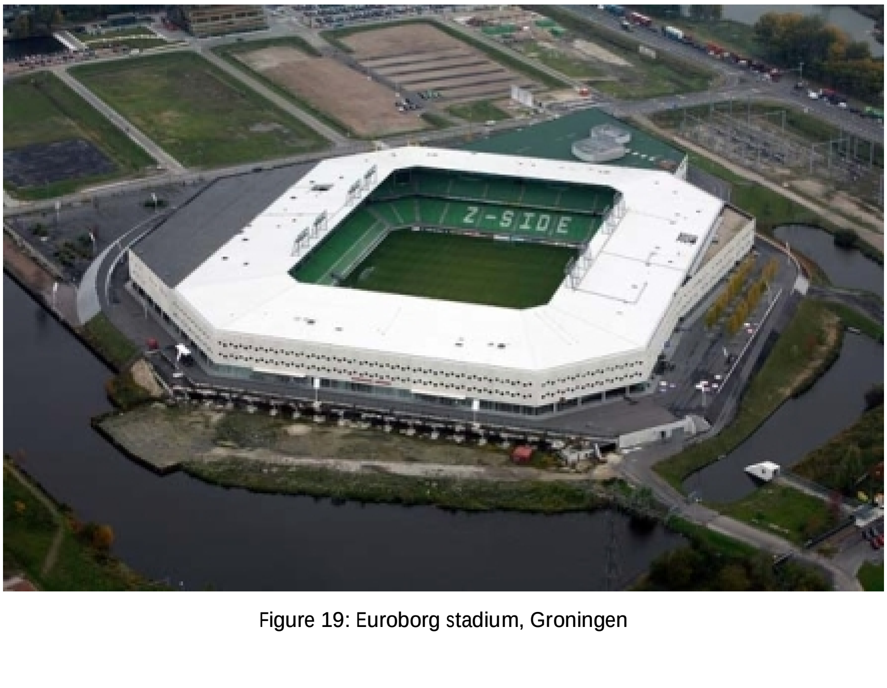


Figure 19: Euroborg stadium, Groningen

## 2.3 Analysis of the Euroborg stadium

Although the Euroborg stadium is not accompanied with a sliding field or a retractable roof, the stadium can still be considered highly multifunctional. Instead of making use of the central playing area, various functions have been located within the circumference of the stadium. One can mention a school, a cinema, a restaurant and offices for example. In addition, the lower two floors (from now on referred to as -2$^{nd}$ floor and -1$^{st}$ floor) accommodate parking lots.

All these specific functions make their own demands on the stadium's structure, and they should therefore be accounted for in the design from the beginning. Before further elaborating on these functional requirements, a standardized section of the Euroborg is shown in figure 20.

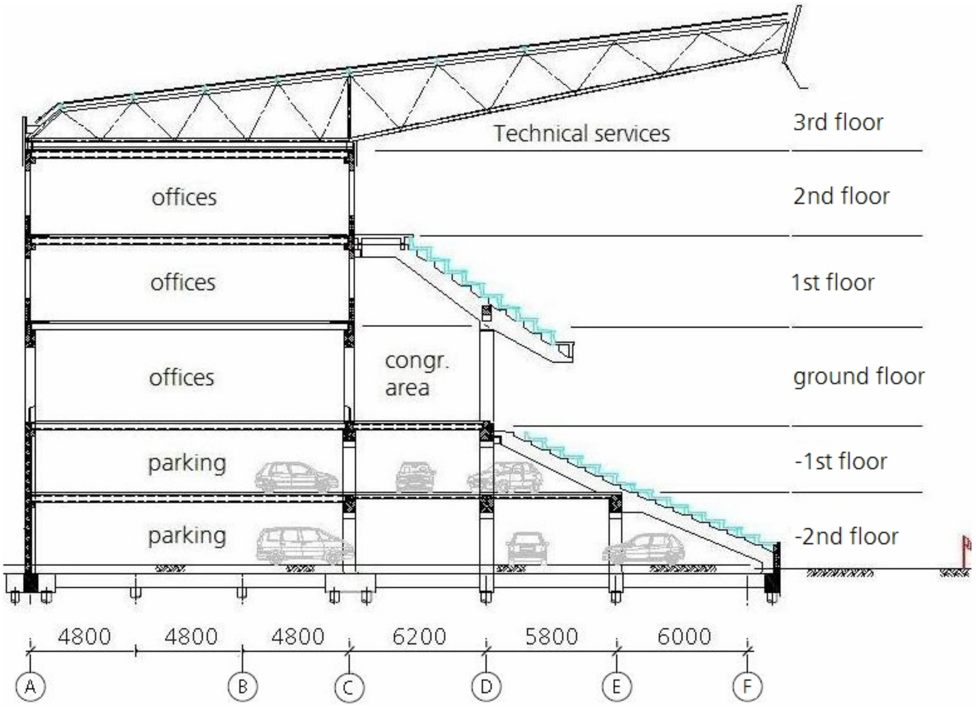


Figure 20: Standardized section of the Euroborg stadium

This section is considered to be representative for the general structure of the stadium and is referred to as the typical section for the Euroborg stadium. This typical section follows from the East stand, which is referred to as building section 5 or the Eastern building part, see figure 21.

Considering figure 20, technical areas, offices, parking lots, congregation areas and grandstands can be distinguished. For all of these functional areas, the most important requirements are discussed in 2.3.1.

The main building (the West stand), differs to a large extent from the typical section as discussed above. The reason for this can be found in the specific function of this part of the stadium. The main building contains a restaurant, a cinema, VIP boxes, player facilities etc., which put totally different demands on its structure than for the rest of the stadium. Since the main building only comprises about a fourth of the stadium structure as a whole, this section is not considered typical within the design.

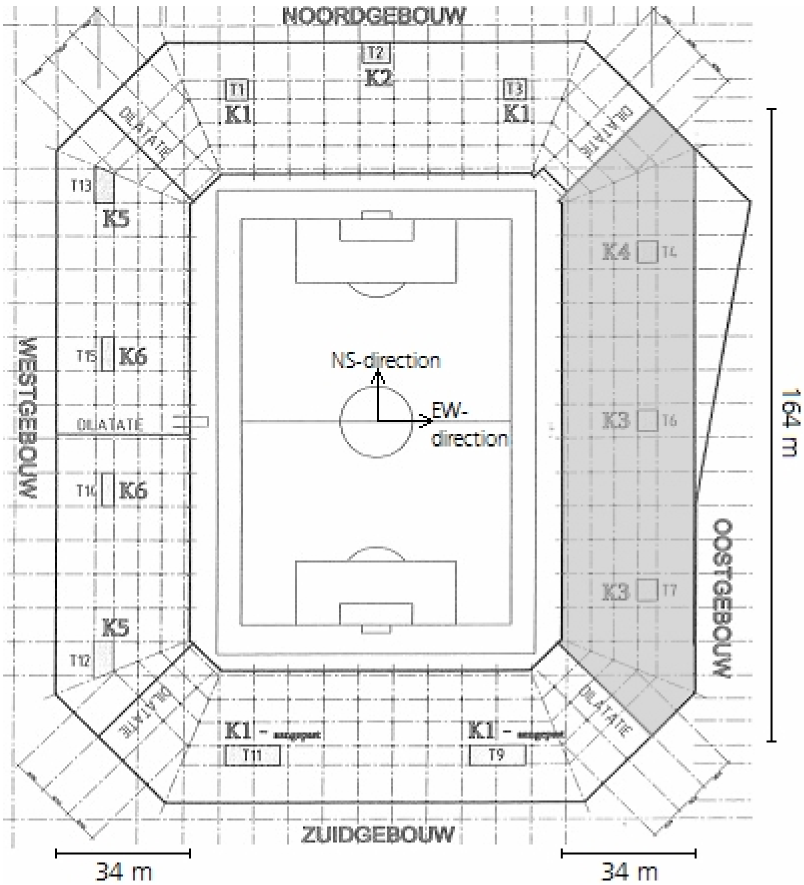


Figure 21: Distinct building parts at the Euroborg stadium

### 2.3.1 Functional requirements

§ Technical area (3rd floor)
  - Not accessible for public
  - No additional requirements

§ Offices (2nd and 1st floor)
  - Building practice prefers spans which are multiples of 1.80 m
  - Spans of 3.60 and 5.40 m are commonly used in practice
  - Building Decree demands a free height of at least 2.40 m
  - Project's principal desires a free height of 2.60 m

§ Parking lots
  - Architectural design requires parking under an angle of 60°
  - According to NEN 2443 this results in a nominal width of 2.50 m and a depth of 5.15 m
  - For single lane access, the road width should be at least 4.0 m
  - Additional safety margins are required when columns are located near the access road
  - Passable areas should have a free height of at least 2.30 m
  - For access roads and local disturbances (i.e. beams, ducts), the free height should exceed 2.10 m

§ Congregation area
  - This area is used as a circulation area and has to be obstacle-free
  - The width towards the exits may never decrease for safety reasons

§ Grandstand
  - The shape of the grandstand is directly dependent on the applied C-factor, which is about 120 mm in this case.

## 2.3.2 Structural design

The main functional requirements being known, the structural design of the Euroborg stadium is elaborated on. Considering figure 20, a first impression is obtained on the main elements within the structural design.

In this section, the most important elements are discussed and their build-up and dimensions are shown.

### Roof

- o Trusses (see figure 22) placed at a c.t.c.-distance of 8.75 m
- o Each truss spans about 36.2 m of which 20.4 m is cantilevering
- o Inclination of 5° to accommodate free flow of rain water
- o Roof decking supported by HE180A purlins
- o Bracing: hollow tubular sections Ø177.8*6.3

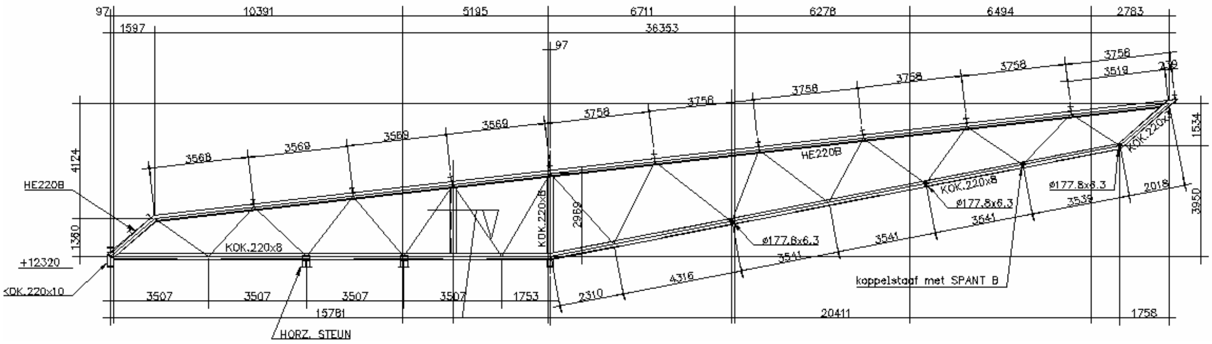


Figure 22: Typical section for the roof structure at the Euroborg stadium

### Floors

The concrete structure of the Euroborg stadium has a height of 19.22 m and holds 6 floors, being referred to as the -2nd, -1st, ground, 1st, 2nd and 3rd floor. The location of these floors can be found in figure 20.

- § -2nd floor (soil level at 6900 mm beneath standardized level)
  - o Main use function: light-weight parking
  - o Paving directly applied on the subsoil
  - o Foundation beams applied in between foundation blocks

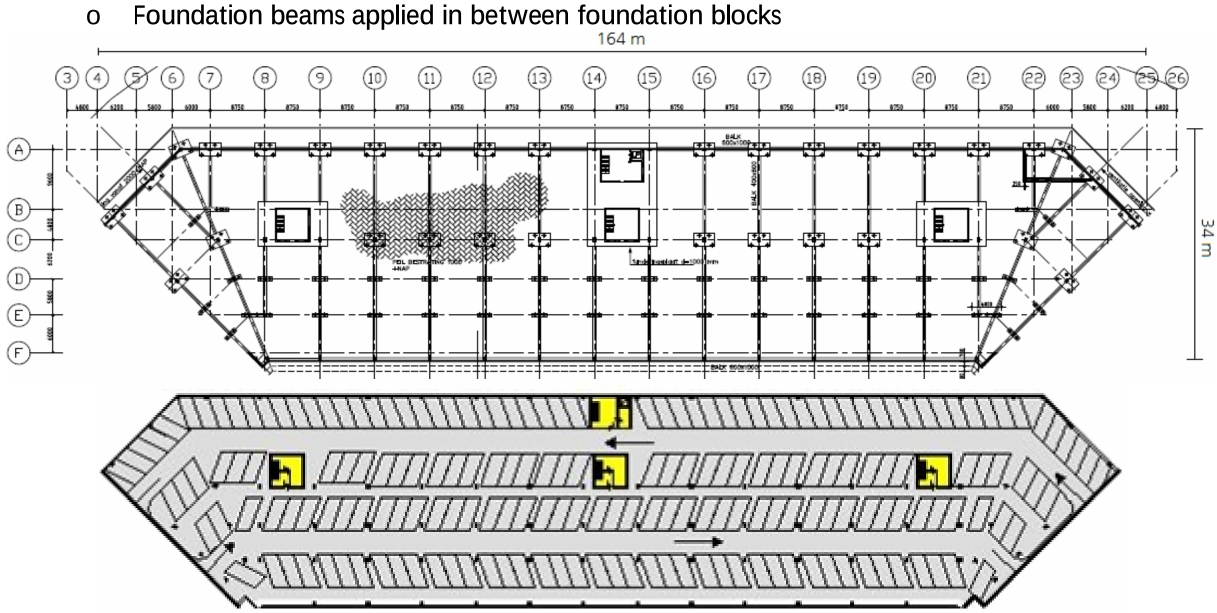


Figure 23: Structural and functional plan for the -2nd floor

- -1st floor (top at -3250 mm)
  - Main use function: light-weight parking
  - Three distinct floor areas: area 1 (axis A-C), area 2 (axis C-D) and area 3 (axis D-E)
  - Area 1: Hollow Core Slabs (abb. HCS) 320 mm, finishing layer of 80 mm
  - Area 2 and 3: HCS 260 mm, finishing layer of 80 mm
  - Slabs supported by pre-cast beams (600 by 800 mm) at axis C, D and E
  - Slabs supported by cast in-situ concrete walls at axis A

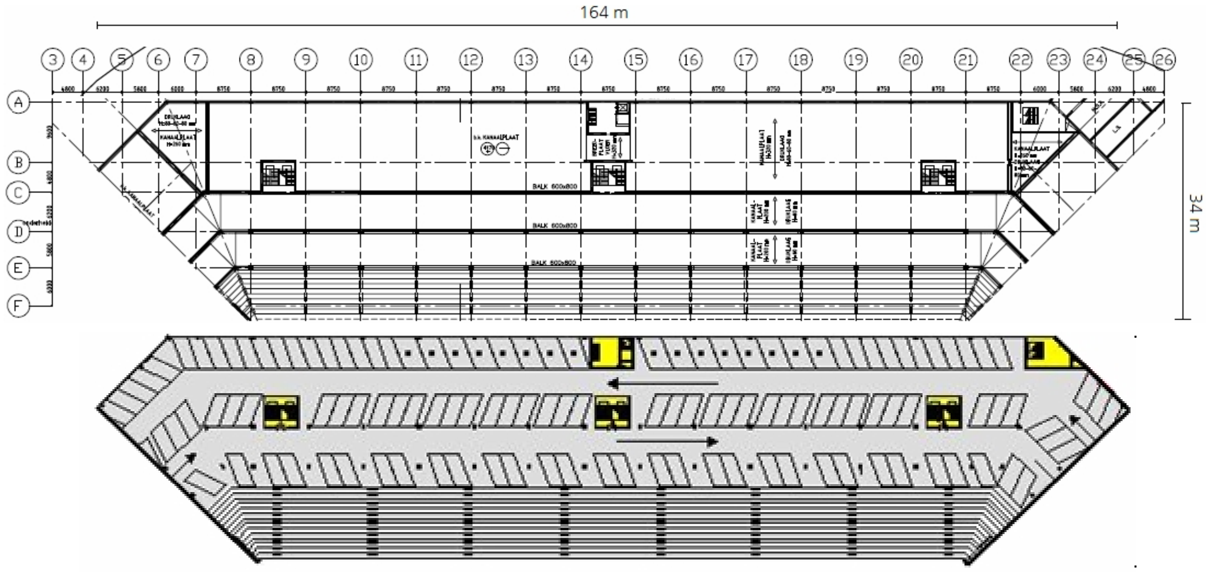


Figure 24: Structural and functional plan for the -1st floor

- Ground floor (top at +0 mm)
  - Main use function: congregation area
  - Two distinct floor areas: area 1 (axis A-C) and area 2 (axis C-D)
  - Area 1: HCS 320 mm, finishing layer of 80 mm
  - Area 2: HCS 260 mm, finishing layer of 80 mm
  - Slabs supported by pre-cast beams (600 by 950 mm) at axis C
  - Slabs supported by pre-cast beams (600 by 800 mm) at axis D
  - Slabs supported by cast in-situ concrete walls at axis A

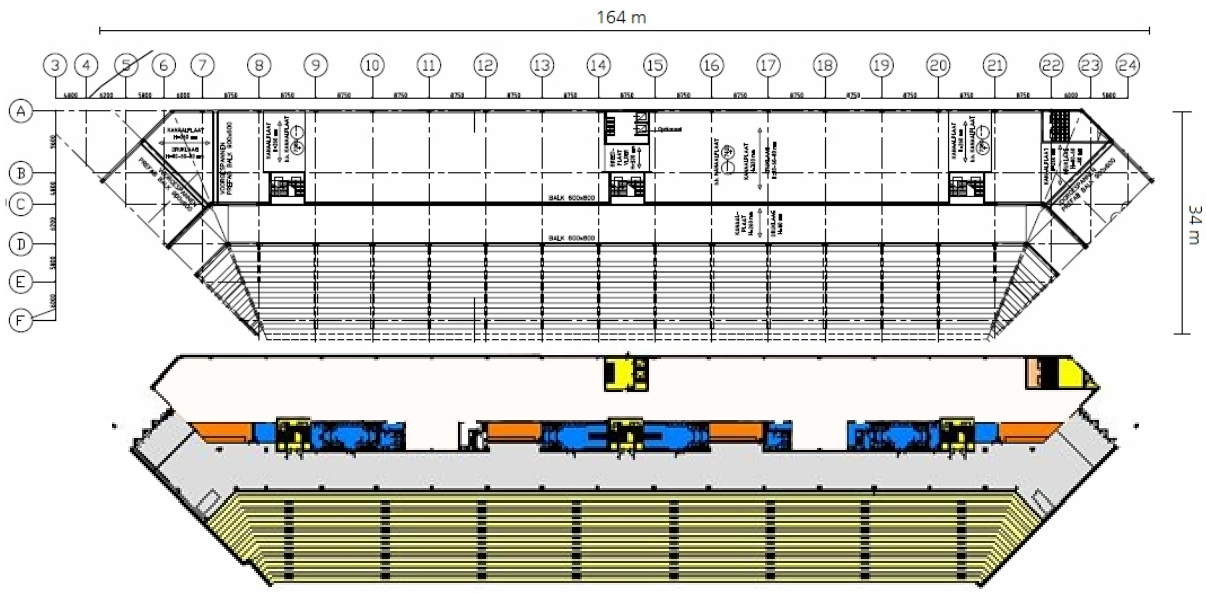


Figure 25: Structural and functional plan for the ground floor

- § 1st floor (top at +4520 mm)
    - o Main use function: offices
    - o HCS 320 mm, finishing layer of 80 mm
    - o Supported at both edges by π-frames

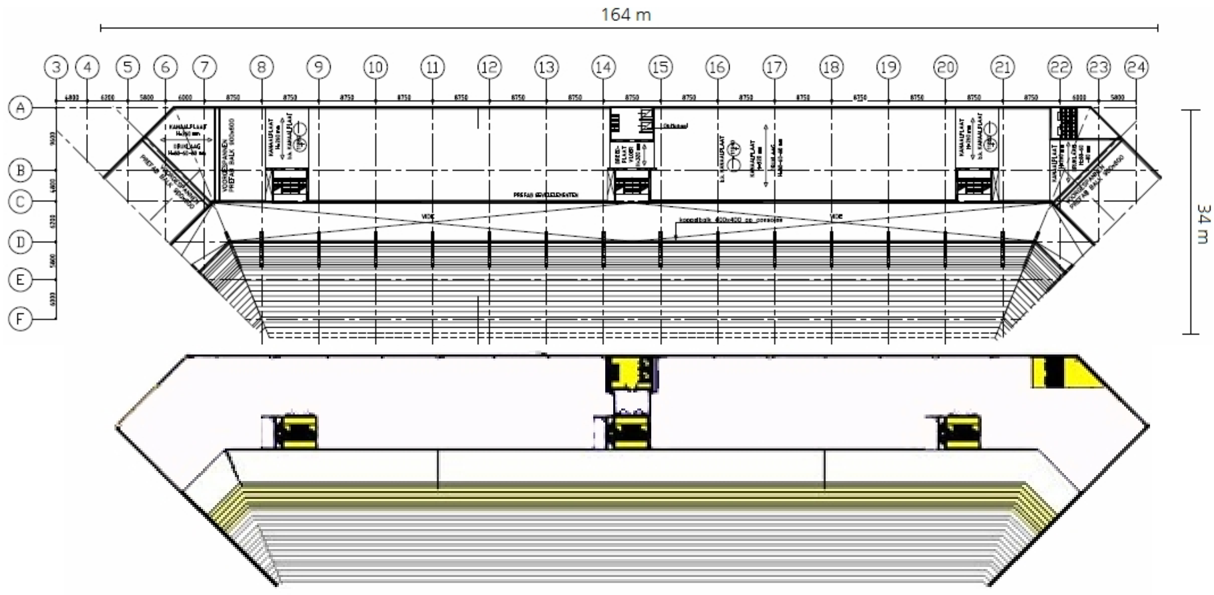


Figure 26: Structural and functional plan for the 1st floor

- § 2nd floor (top at +8420 mm)
    - o Main use function: offices
    - o HCS 320 mm, finishing layer of 80 mm
    - o Supported at both edges by prefabricated façade elements

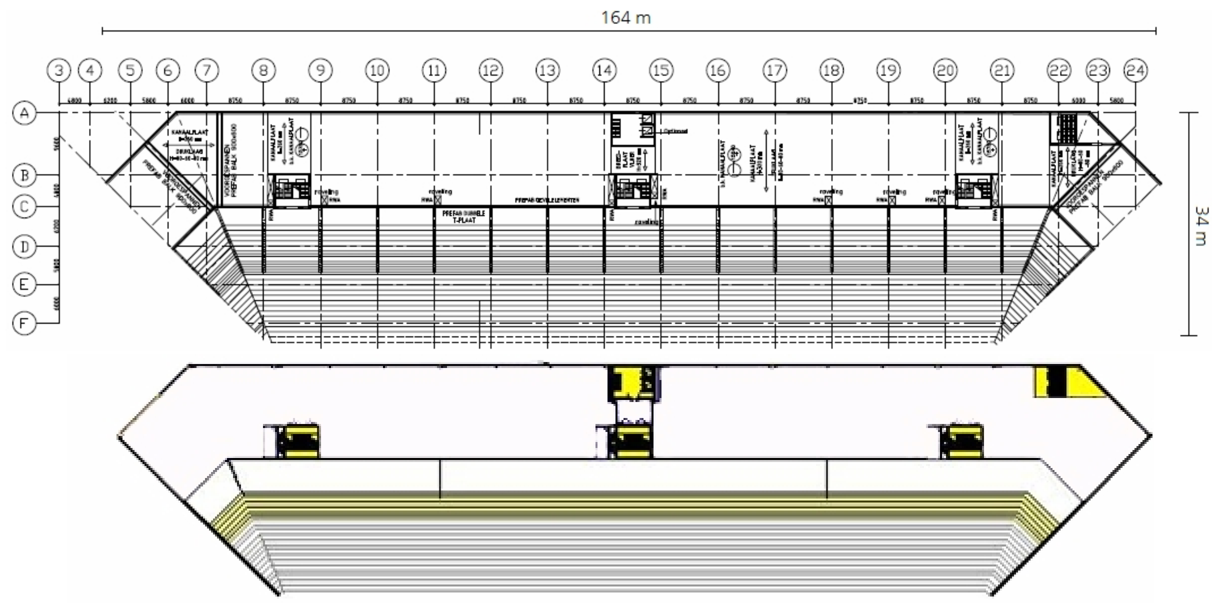


Figure 27: Structural and functional plan for the 2nd floor

- § 3rd floor (top at +12320 mm)
    - o Main use function: technical services
    - o HCS 320 mm, finishing layer of 80 mm
    - o Supported at both edges by prefabricated façade elements

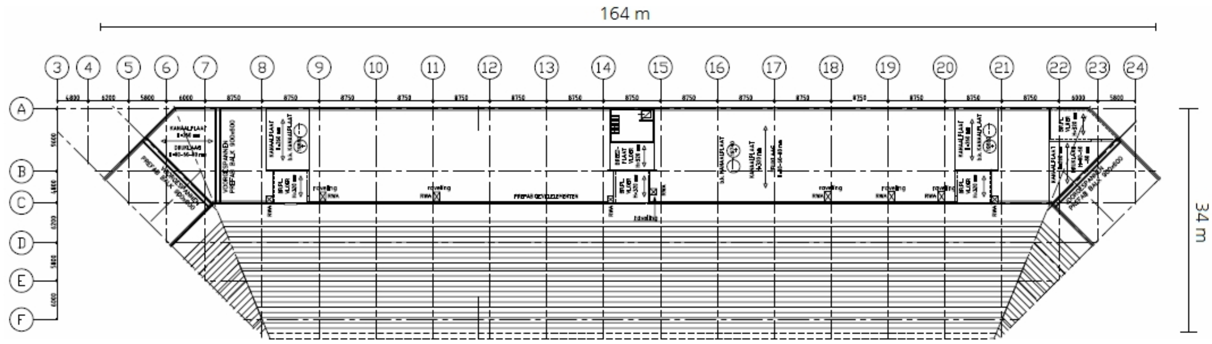


Figure 28: Structural plan for the 3rd floor

## Grandstand

When the typical section (see figure 20) is considered, one can distinguish an upper and a lower grandstand. The upper grandstand consists of 10 tiers, while the lower grandstand holds 16 tiers.

The crow-stepped shape of the grandstand is provided by the *grandstand elements*, which consist of both vertical (risers) and horizontal (treads) faces, see figure 29. The height of the risers is dependent on the required C-factor and the distance from the action, as elaborated on in the section 1.4. The width of the tread follows directly from safety regulations and is set to 800 mm

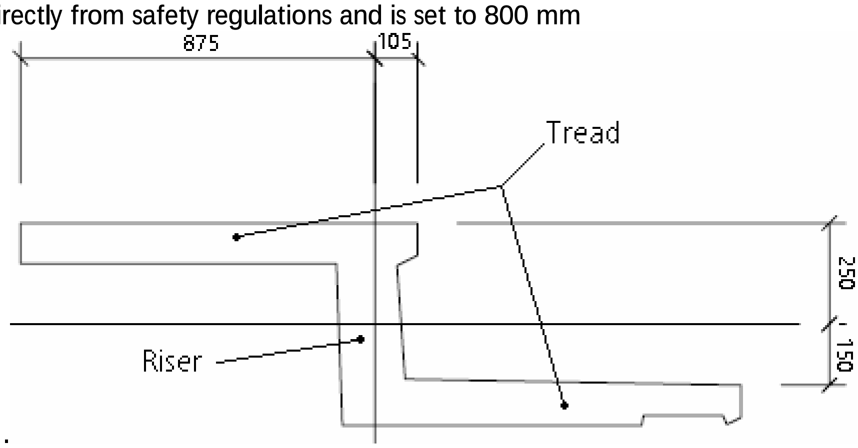


Figure 29: Example of a typical grandstand element

- Upper grandstand elements:
  - All 10 tiers: risers 540 mm; tread 800 mm

- Lower grandstand elements:
  - Upper 8 tiers: risers 400 mm; tread 800 mm
  - Lower 8 tiers: risers 335 mm; tread 800 mm

The grandstand elements span 8.75 m and are supported by the *grandstand support beams*. The shape of these beams is directly dependent on the shape of the grandstand itself.

- Upper grandstand support beams (see figure 30):
  - Supported by columns at axis C and D
  - Cantilevering after axis D

- Lower grandstand support beams (see figure 31):
  - Supported by columns at axis D and E
  - Supported by the foundation at axis F

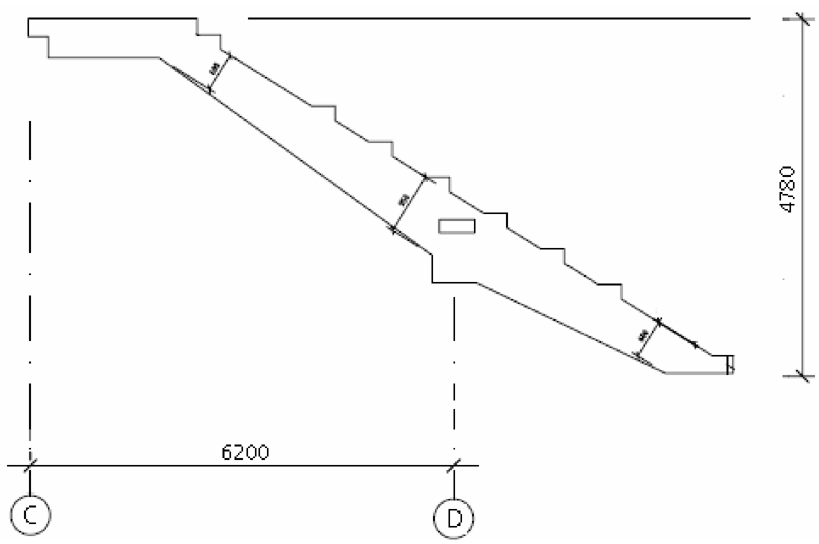


Figure 30: Shape of the upper grandstand support beam

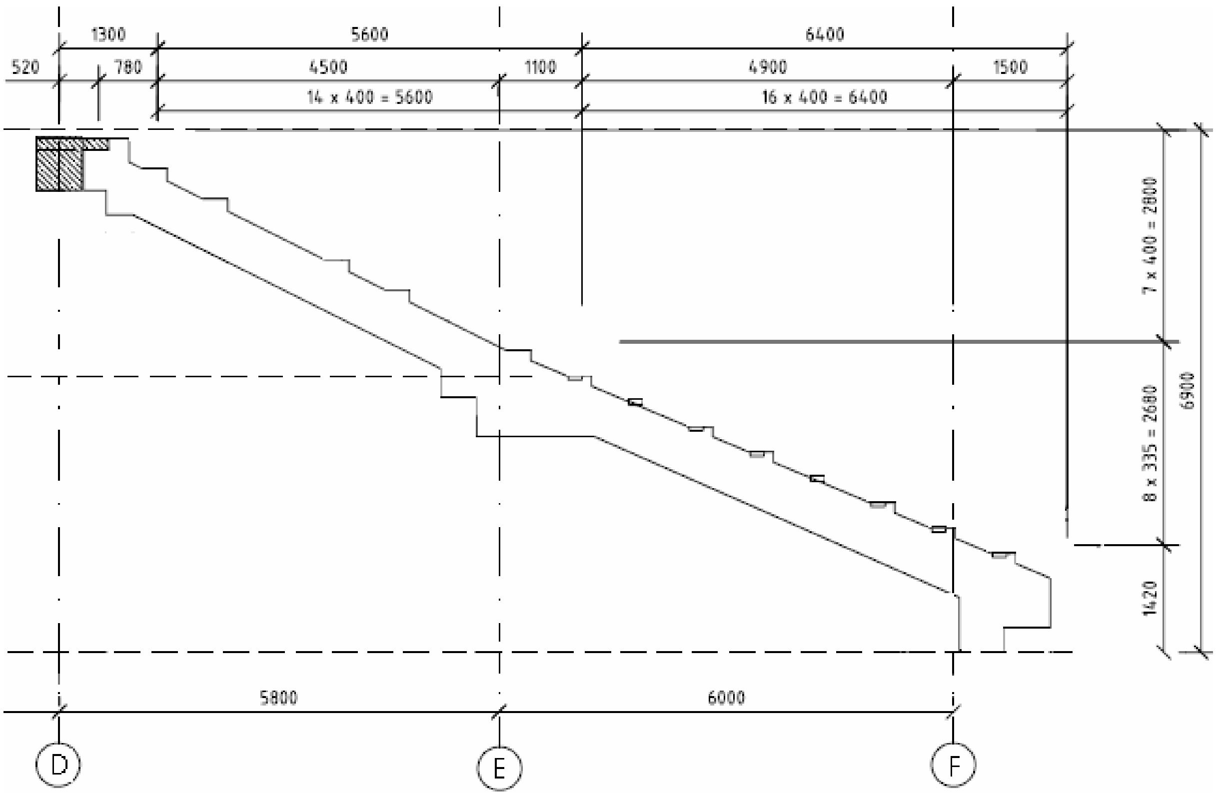


Figure 31: Shape of the lower grandstand support beam

### Vertical support elements

- At axis A, walls are available over the total height to transfer the vertical loads to the foundation
  - From the foundation to the 1st floor, cast in-situ walls (d = 250 mm) are used
  - From the 1st floor to the top, prefab sandwich elements (d = 220 mm) are used

- At axis C, columns (600 by 400 mm) are used
  - The columns are placed on a c.t.c.-distance of 8.75 m
  - Columns run from the foundation to the top

- At axis D and E, columns (800 by 600 mm) are used to support the grandstand support beams:
  - The columns are placed on a c.t.c.-distance of 8.75 m

Foundation (see figure 32)

- The columns are founded on foundation blocks, which are supported by concrete driven piles
- 1224 piles pre-cast piles are used for the whole stadium (see figure 32 for the Eastern part)
- Foundation blocks under the main columns (axis A and C) are 3400 by 2200 mm
- Each of these 'main' foundation blocks is supported by 5 piles (450 mm square)
- Foundation blocks under other columns (axis D and E) are 2200 by 800 mm
- Each of these blocks is supported by 2 piles (450 mm square)
- In between the distinct foundation blocks, beams are applied with dimensions 400 by 800 mm
- Underneath each of these beams, at axis B and F, an additional pile is applied

The piles are driven to a depth varying between 13 m and 27 m under the standardized level, depending on the exact location. As a result, the piles have lengths varying between 6 and 20 m.

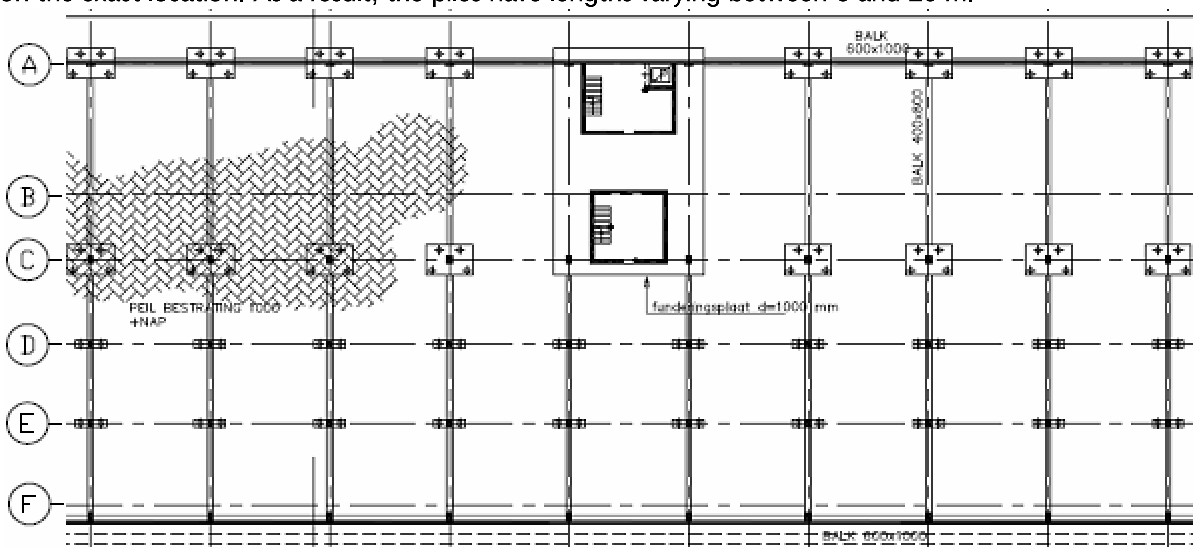


Figure 32: Overview foundation

## 2.3.3 Structural system

To be able to cope with deformations caused by effects like shrinkage, creep and uneven settlements, the concrete structure of the Euroborg is divided into several building parts. At the edges of these building parts, joints ('Dilatatie') are made. For the Euroborg stadium, these can be found at the corners of the stadium, see figure 33.

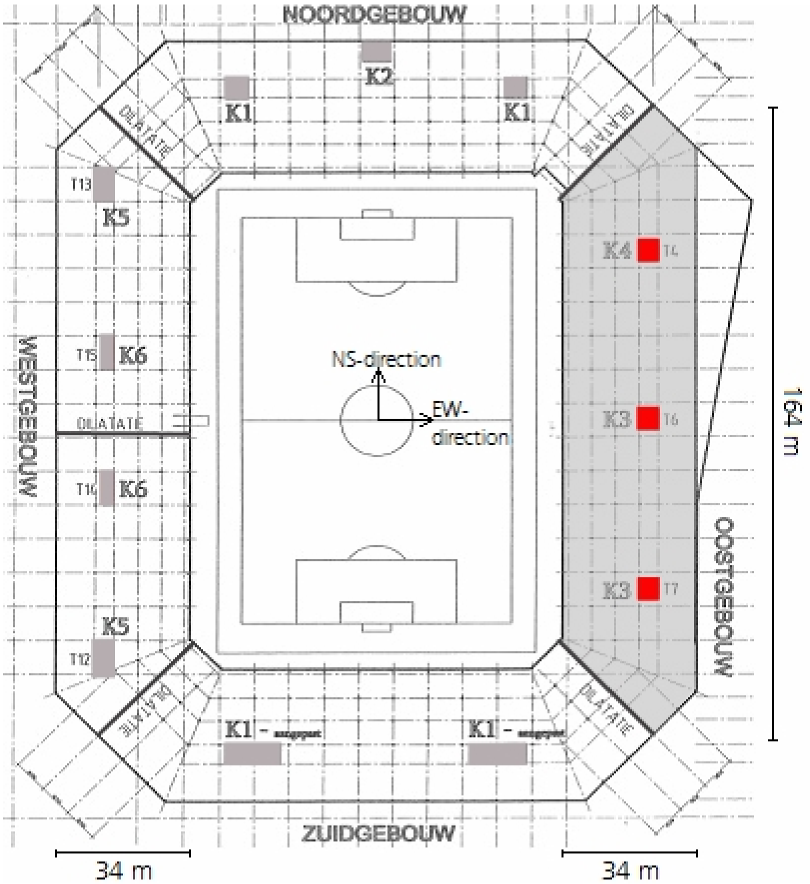


Figure 33: Structural system of the Euroborg stadium

The stability of the stadium structure is provided for every individual building part, by means of multiple cores. Each core is situated around the central stairs of that specific part of the stadium. For the Eastern building part three pre-cast concrete cores are incorporated, see figure 33 and figure 34. Each of these cores takes up more or less 33% of the acting horizontal forces, which are being introduced by the wind and the imposed grandstand loads.

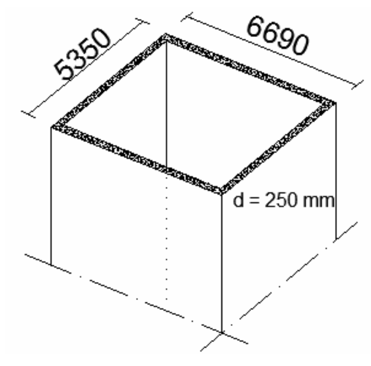


Figure 34: Dimensions of the pre-cast concrete cores at the Eastern building part

Due to the presence of the player facilities in the Western part, it was not possible to incorporate a core in the centre of this building part. As a consequence, an additional joint was introduced, see figure 33.

When timber structures are concerned, no such joints are required since the material is able to cope with these effects by itself. Despite that, the division in parts is adopted in this thesis for reasons of ease.

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# 3 Reference projects

Presenting reference projects from practice, which are related to the timber stadium structure in one way or another, the reader is provided with insight on the structural possibilities of timber and engineered wood products. Therefore, this chapter provides an answer to the following sub question:

§ *What kind of structures can be achieved when making use of structural timber?*

Consecutively, roof solutions, floor systems, a grandstand and a structural system are considered.

## 3.1 Structural roof solutions

Timber is especially suitable as a structural material when roof structures are concerned. The main reasons are the following:

§ Durability is generally not an issue, since the roof structure is protected and stays dry.
§ For roof structures there is no requirement on fire resistance.
§ Timber roof structures have a high strength-to-weight ratio

In section 1.3.1, there has been elaborated on several type of roof structures that are commonly used (or suitable) for stadium structures. Since one of these types will probably be used in a timber stadium, research has been carried out on these types of structures being executed in timber. This has resulted in the following overview.

### 3.1.1 Goal post system

Since the goal post system is generally used in stadium structures only (and such structures are not carried out in timber), there are not much references available on such timber structures. Considering the goal post system, the main element of the structure is the (arched) roof beam. Since such elements are readily available in timber, an overview is provided on the range of spans that can be reached. [ 7 ]

| | |
|---|---|
| Glulam truss (two-pinned arches): | 20 to 100 m |
| Glulam truss (three-pinned arches): | 20 to 60 m |
| Arched truss: | 40 to 120 m |

### 3.1.2 Cantilever beams

Cantilever beams in timber are generally made from solid timber, LVL or glulam (for information on these products see Annex A.2). The span ranges between 8 and 30 m. A typical example of a cantilevered roof can be found at the Autzen Stadium in Eugene, USA. [ 8 ]

| *Autzen Stadium roof* | |
|---|---|
| Location | Eugene, USA |
| Completion date | 2002 |
| Client | University of Oregon |
| Architect | Ellerbe Becket |
| Structural engineer | Ellerbe Becket |
| Main contractor | Hunt Constr. Group |
| Glulam suppliers | Wood Tech Services Inc. |
| Glulam installer | John Hyland Construction Inc. |
| Building time | 2 months |

Figure 35: Cantilevered roof at the Autzen stadium (Eugene, USA)

The roof of the Autzen Stadium is supported by a pre-cambered cantilever. The total beam length amounts 52 m, of which nearly 30 m is cantilevering. Parallel to the pitch, a small curvature is applied to the roof.

The cantilevering beams are made by bolting two distinct glulam members to each other. Each of these combined glulam members is made from Douglas Fir, having a width of 311 mm, resulting in a total width of 622 mm. The maximum height of the beam is 1830 mm. The purlins are also made from Douglas Fir, having dimensions of 130 mm by 380 mm.

### 3.1.3 Arches

Arches are most efficient when it comes to load-bearing spans: the whole cross-section is loaded in compression uniformly. To prevent in-plane buckling and snow accumulation at one side of the arch, the maximum span-to-rise ratio is 8:1 and the maximum span-to-(beam)depth ratio is 50:1. The depth-to-width ratio for the beam is about 5:1. Out of plane buckling can be prevented by designing the roof decking as a diaphragm. Arches are usually made from glulam. [ 7 ]

In this section, the three main arch-type roofs are discussed: portal arches, barrel vaults and domes.

#### Portal arches

Portals are a special kind of arches, since their shape is rather curved than arched. Main benefit is that the shape of the portal follows the traditional building envelope in a better way. This implies a rather extreme curvature at the corners.

Since only relatively small spans can be reached, this solution will be neglected. An excellent example can be found at the Tobias Grau office in Rellingen (Germany).

#### Barrel Vaults

A barrel vault consists of several arches which are aligned in series. In between the arches, a rigid deck is applied which stabilizes the structure. For large-span timber structures, the barrel vault is assumed to be the most economic solution. An excellent example can be found at the Atlantico Pavilion, see underneath.

| *Atlantico Pavilion* | |
|---|---|
| Location | Lisbon |
| Completion date | 1998 |
| Client | Parque Expo 98 SA |
| Architect | Skidmore, Owings & Merrill Inc., London |
| Structural engineer | SOM Inc., London |
| Glulam contractor | Weisrock |
| Building time | about 1 year |

Figure 36: Atlantico Pavilion (Lisbon, Portugal)

The roof structure of the main hall, with a length up to 115 m, is the longest one-way span made with glulam in a seismic zone. Swedish spruce was used as a structural material due to the environmental theme of the Expo and since it was considered the most cost-effective solution.

A total of 16 truss arches are implemented in the design, being hinged at both ends and spaced at a c.t.c.-distance of 9.0 m. The trusses are composed from a top and bottom chord, with a glulam Warren truss in between. The dimensions of the individual members can be extracted from figure 37. [ 9 ]

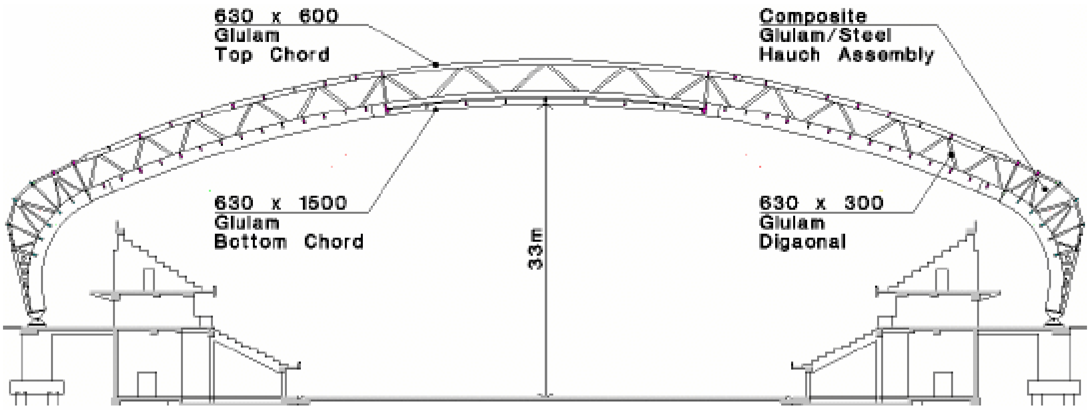


Figure 37: Typical section of the Atlantico Pavilion

### Arched domes

An arched dome consists of several half-arches, which are connected at the centre of the dome. Purlins are used to stabilize the arches. For very large roof structures truss arches are used, which can be assembled on site. An excellent example can be found at the Joensuu Multipurpose Arena.

| *Joensuu Multipurpose Arena* | |
|---|---|
| Location | Joensuu, Finland |
| Completion date | 2004 |
| Client | City of Joensuu |
| Architect | Pro-Ark Oy Marjatta |
| Structural engineer | Finnmap Consulting Oy |
| Timber supplier | Finnforest, Late-Rakenteet Oy |
| Building time | 2 years |

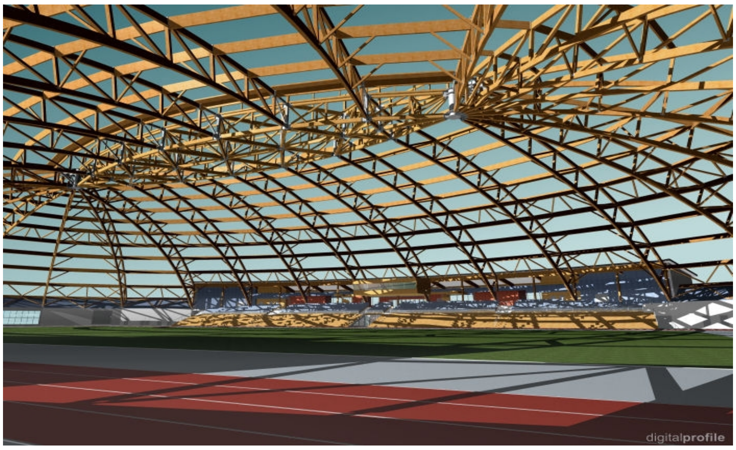


Figure 38: Joensuu Multipurpose Arena (Joensuu, Finland)

The Joensuu Multipurpose Arena was the largest timber structure in Europe when built. The oval-shaped dome has a length of 150 m, with a maximum height of 24 m. The arena can accommodate a maximum of 7000 spectators.

A total of 28 (prefabricated) latticed half-arched frames are implemented, spanning up to 100 m. The main chords are composed from glulam, while the webs and purlins are LVL. The majority of the connections is made with dowel-type fasteners.

At the centre of the dome, the arches are connected to a lens-shaped space frame. The length is about 55 m, with a maximum width of 10 m. This so-called ‘crown’ is fully made of LVL.

In total, about 1300 m3 of glulam and LVL has been used in combination with 39000 dowel-type fasteners and 150 tons of steelwork. [ 10 ]

## 3.1.4 Grid shells

### True grid shell

A (true) grid shell is designed according to the principle that an inverted ‘hanging chain’ forms a stable arch. Making use of relatively small timber elements, double curvature can be reached, accommodating large spans. An excellent example can be found at the Savill Building, see underneath.

| *Savill Building* | |
|---|---|
| Location | Windsor |
| Completion date | 2006 |
| Client | Crown Estates |
| Architect | Glenn Howells Architects |
| Structural engineer | Buro Happold |
| Roof contractor | Green Oak |

Figure 39: Savill Building (Windsor, United Kingdom)

The roof of the Savill Building has a three-domed sinusoidal shaped structure, consisting of four layers of high quality larch elements, with spacing blocks in between. These elements have dimensions of 50 by 80 mm, and are arranged in a grid of 1 by 1 m. The roof is supported by a tubular steel ring beam, following the entire edge of the roof. Due to the presence of this beam, the curvature of the timber elements is reduced. In between the tubular ring and the grid shell, Kerto is used to transfer the acting loads.

The length of the structure is about 90 m, having a maximum width of 25 m. The total structure has been built flat on the ground, and was then hoisted to its final position. Birch plywood has been used in between the layers. All connections are machine-driven screws. [ 11 ]

### Rhomboid grid shell

When a rhomboid grid shell is concerned relatively small and uniform elements are used, which are interconnected by means of screws. Almost without exception, these kind of structures are made from glulam. An excellent example can be found at the Neue Messe in Friedrichshafen (Germany). [ 7 ]

| *Multipurpose hall, Neue Messe* | |
|---|---|
| Location | Friedrichshafen, Germany |
| Completion date | 2002 |
| Client | Int. Bodensee-Messe |
| Architect | gmp |
| Structural engineer | Merz & Kaufmann Bauingenieure |
| Timber construction | Holzbau Amann, Kaufmann |
| Building time | 2 years |

Figure 40: Multipurpose hall, Neue Messe (Friedrichshafen, Germany)

The roof structure of the multipurpose hall at the Messe in Friedrichshafen was constructed from lamella rhomboids, supported at its edges by steel beams on concrete panels. All lamellas have dimensions of 200 by 800 mm and are made from glulam GL28h.

The total length of the hall amounts 150 m, having a width of 68 m. The maximum height above ground level is 26.3 m, while the edges are supported at a height of 16.6 m. The horizontal forces are taken by steel tension chords.

## 3.2 Floors systems

When large-scale structures are concerned, usually, use is made of engineered wood products (EWP's) rather than traditional beam and planking systems. This approach is therefore adopted in this thesis as well. To provide some insight on the possibilities of these EWP's, two interesting examples are shown.

### 3.2.1 Lignatur

Lignatur element can be considered timber hollow core slabs. As a result their weight-to-span ratio is relatively small. More information on Lignatur elements can be found in section 5.2.1 and Annex A.2. An interesting reference project is found at culture house 'De Kamers'.

| *De Kamers Vathorst* | |
|---|---|
| Location | Amersfoort, The Netherlands |
| Completion date | 2007 |
| Client | Stichting De Kamers |
| Architect | Korteknie Stuhlmacher |
| Structural engineer | Pieters Bouwtechniek |
| Timber floors | Lignatur AG |
| Building time | 1 year |

Figure 41: De Kamers (Amersfoort, The Netherlands)

Due to the large spans of about 7 to 9 m, the Lignatur hollow core slabs promised to be the only economic solution in timber. The elements have a width of 1 m and a length of 9 m at most. Their height is taken 320 mm. They are composed from pine beams, which are glued together under compression.

The distinct panels are connected to each other by means of a spring and slot connection. To provide diaphragm-action, steel bracings are applied on top of (some of) the floors. [ 12 ]

### 3.2.2 Cross-laminated timber

Cross-laminated timber is made by bonding single layered panels, which are in turn made from softwood laminates. More information on CLT elements can be found in section 5.2.2 and Annex A.2. An interesting reference project is found at the Norwich Open Academy.

| *Norwich Open Academy* | |
|---|---|
| Location | Amersfoort, The Netherlands |
| Completion date | 2010 |
| Client | Norfolk County Council |
| Architect | Sheppard Robson |
| Structural engineer | Ramboll UK |
| Timber floors | KLH UK |
| Building time | 18 weeks |

Figure 42: Norwich Open Academy (Norwich, United Kingdom)

The CLT floor elements span 7.55 m and have a thickness of 230 m. In case larger spans were required, glulam support beams were used. Horizontal loads are transferred through diaphragm-action of the floors. In case the spans were very large, a hybrid variant with steel trusses has been used. [ 13 ]

## 3.3 Grandstands

Timber grandstand structures are not very common in modern building practice. When timber is used as a building material, the space underneath the grandstand is generally not accessible. Examples of such structures in the Netherlands are found at the Silverdome in Zoetermeer and at the CineMec infotainment centre in Ede. The latter one is discussed more in detail since it shows good resemblance to a stadium-like grandstand.

### 3.3.1 Timber grandstand

| *Expotheater, Cinemec* | |
|---|---|
| Location | Ede, The Netherlands |
| Completion date | 2009 |
| Client | CineMec |
| Architect | DP6 |
| Structural engineer | ABT bv |
| Timber structure | Welling Didam |

Figure 43: Expotheater CineMec, Ede (The Netherlands)

The 'ExpoTheater' of CineMec in Ede is a multipurpose auditorium, which is generally used as a cinema. The theatre can be transformed into an exhibition room by making use of an inventive system: parts of the grandstands are simply hoisted just underneath the ceiling. To provide a successful system, a light-weight grandstand structure was required.

The grandstand of the ExpoTheater consists of a fixed steel part and two movable parts which are mainly composed from timber elements. The movable parts hold 19 tiers whereas a different C-factor is applied for the upper and the lower tiers. The weight of each movable part is about 32 ton.

The crow-stepped shape was obtained by connecting timber (pine) plates to each other. These are in turn supported by timber support beams which follow the shape of the grandstand, see figure 44.

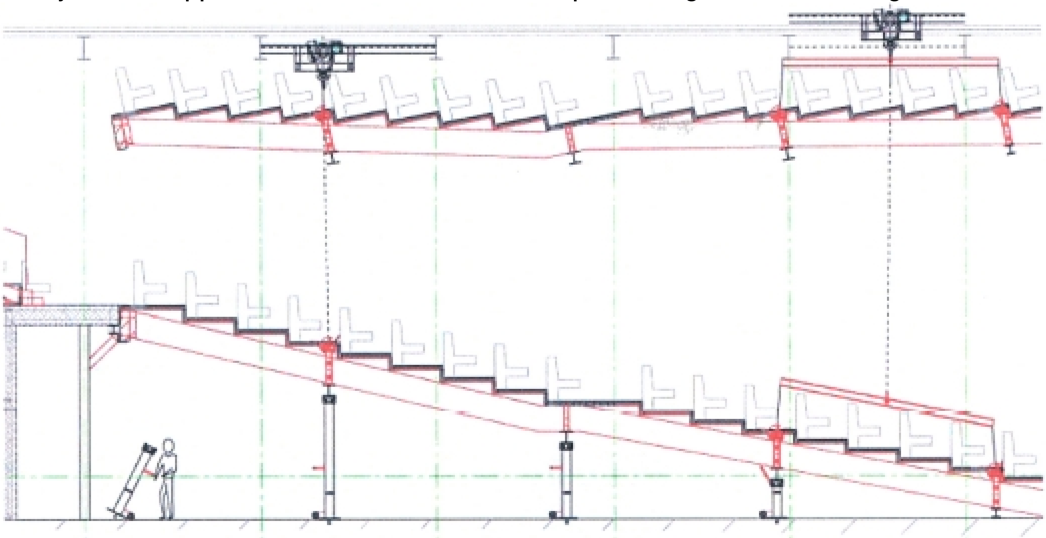

Figure 44: Section of the grandstand at the Expotheater of CineMec

To avoid hindrance due to vibrations, the design was based on a fundamental frequency of at least 8 Hz. To comply with this requirement, the support beams were placed at a c.t.c.-distance of 2.5 m. These grandstand support beams are in turn supported by steel beams which provide the connection with the (removable) propping. [ 14 ]

## 3.4 Structural systems

Stability in timber engineering can be achieved in various manners. To provide the reader with insight in the possibilities, two solutions are discussed here, being portal frames and shear walls. For the latter solution, an example is shown underneath.

### 3.4.1 Portal frames

Portal frames are composed from columns and beams, which are rigidly connected at the corners. They accommodate clear spans ranging from 15 to 30 m. The elements can be made from solid timber, Glulam, LVL and thin webbed beams. [ 11 ]

One should bear in mind that these kind of connections require a lot of attention and are therefore quite expensive. A wide range of good examples can be found on the internet.

### 3.4.2 Shear walls

Until recent, wall diaphragms were generally stud walls with a plywood sheeting. But due to the availability of large plate-like timber products (such as LVL or CLT), these diaphragms can now be made from massive timber elements. An excellent example can be found at the Stadthaus in London.

| *Stadthaus, 24 Murray Grove* | |
|---|---|
| Location | Londen, UK |
| Completion date | 2009 |
| Client | Telford Homes PLC and Metropolitan Housing Trust |
| Architect | Waugh Thistleton |
| Structural engineer | Techniker |
| Main contractor | Telford Homes |

Figure 45: Stadthaus, Murray Grove, London (UK)

The Stadthaus, with its nine stories, is often referred to as the world's tallest residential timber building. The apartments are located around a central core containing several elevators, which provides private access to the inhabitants. In total, 29 apartments are present in the building.

The load-bearing structure is mainly composed from CLT elements (see Annex A.2), which are used for both flooring and walls. Each of these elements was prefabricated, including all required cut-outs. Together, these walls provide stability to the building. [ 15 ]

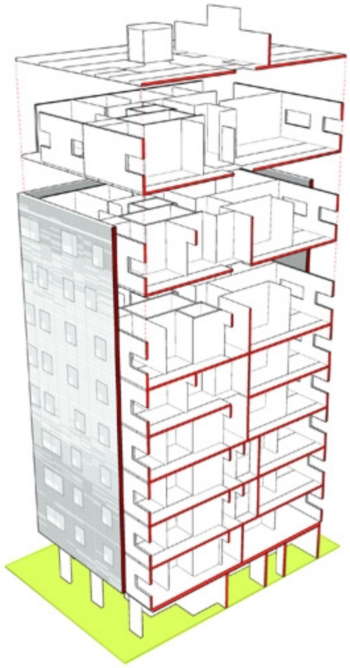

Figure 46: Axonometric view of the Stadthaus

# Conclusions Research

## Chapter 1 Introduction to stadium design and engineering

§ Attracting as much spectators as possible is the main goal in stadium development (section 1.1)
§ Pitch dimensions and safety areas influence spectator visibility to a great extent (section 1.2.1 and 1.4)
§ A cantilever system is the most beneficial roof solution for partly enclosed stadium roofs (section 1.3.1)
§ An arch-type roof is the most beneficial roof solution for fully enclosed stadium roofs (section 1.3.1)
§ Vomitories are the most beneficial solution to provide access to the grandstands (section 1.3.2)
§ To provide optimum viewing, all seats should be within a radius of 90 m from the centre of the pitch (section 1.4)
§ The maximum acceptable viewing angle is 34 degrees (section 1.4).
§ A C-factor of at least 90 mm should be accounted for, but 120 mm is recommended (section 1.4)

The main structural requirements are summarized in section 1.5.

## Chapter 2 Design reference stadium

§ The Euroborg stadium is most suitable to be used as a reference stadium (section 2.2)

The main functional requirements are summed up in section 2.3.1, the structural build-up can be found in section 2.3.2 and the structural system is elaborated on in section 2.3.3.

## Chapter 3 Reference projects

§ Structural timber is commonly used for various types of roof structures (section 3.1)
§ For large or heavy-loaded floors, purpose-made floor products are used in practice (section 3.2)
§ Timber grandstands are no commonly used structures (section 3.3)
§ Vibrations appeared to be governing for the timber grandstand structure considered (section 3.3)
§ Plate-like timber products can be used as shear walls to stabilize large structures (section 3.4)

## Considerations

§ Since timber roof structures are already common in practice, from this point on, this structural element is not considered in this thesis.

§ Most interesting structural elements to investigate are the grandstands, since there is a lack of knowledge on their feasibility.

§ It has to be investigated which kind of structural systems in timber can be applied, to stabilize the stadium structure.

§ As the stadium building is an important part of the structure, attention should be paid to possible floor solutions.

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# Preliminary design

In the preceding part of this thesis, knowledge has been acquainted on the backgrounds of stadium engineering and the possibilities of timber engineering. A reference stadium has been introduced, which is used as a basis for the upcoming preliminary design phase.

The preceding part of this thesis was concluded by several conclusions and considerations, which identified the most interesting aspects of the stadium structure. It is therefore decided to consider these subjects more in detail within this preliminary design phase.

## Chapter 4 Structural system

In this chapter, several structural systems are investigated. At first, a timber core system is considered (section 4.4). Subsequently, a system consisting of shear walls and trusses (section 4.5) is accounted for. At the end of this chapter, these systems are evaluated and a proposal is made for the final design phase.

## Chapter 5 Floors

From the research phase, it became clear that engineered wood products are probably most beneficial when heavily loaded, large span floors are concerned. In section 5.2 attention is paid to several floor products which are readily available on the market. Subsequently it is investigated how these products can be used within the stadium structure, while considering several structural configurations (sections 5.3 to 5.5). At the end of this chapter, in section 5.6, the most beneficial solution is determined and a proposal is made for the final design phase.

## Chapter 6 Grandstands

Since the grandstands are essential in a stadium structure, a preliminary design is made of the timber grandstands. After elaborating on the boundary conditions, a first design is presented (section 6.3) Accounting for the results, several adaptations are made to improve the design (section 6.4). This results in a proposal for the final design.

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# 4 Structural system

## 4.1 Introduction

Goal of this chapter is to explore the possibilities of several structural systems, and to determine the most suitable system for a timber stadium structure.

As an example of a structural system, one can think of central cores which are often used in multi-storey buildings, see figure 47. Other commonly used stabilizing elements are so-called (wind) bracings, see figure 48.

Figure 47: Central core solution as structural system

Figure 48: Stability by means of bracings

Start assumption is that the main structural material of the investigated solutions is timber. As a result, in this chapter an answer is provided to the following question:

§ *Which structural system in timber fits a stadium structure at best?*

This chapter only applies to one of the building parts of the Euroborg stadium, being the Eastern building part. The functional plan for this building part is considered to be more or less unviable, which means that the proposed stabilizing system has to be adapted to the existing plan. It goes beyond the scope of this thesis to consider extensive adaptations on the functional plan.

### 4.1.1 Approach

In this chapter, consecutively, a system with central cores and a system with small stabilizing walls are discussed. For the direction transverse to the governing wind direction a system with trusses is investigated. After discussing all systems, the (dis-)advantages are discussed and the most beneficial solution is determined by making use of expert opinions.

For each structural system, the following aspects are discussed:

- Principle behind the structural system
- Implementation in the functional plan of the Euroborg stadium
- Material use
- Structural build-up and behaviour
- Proposed design

The results follow from structural calculations, which are shown in Annex C. Before elaborating on the distinct systems, the stabilizing principle for the Euroborg stadium is recalled. Also the functional plans of the distinct storeys of this stadium are presented here again.

## 4.2 Structural system of the Euroborg

With reference to section 2.3.3, it is stated that each building part of the Euroborg stadium has its own structural system. For the sake of clarity, an overview of the structural principle is shown here again, see figure 49.

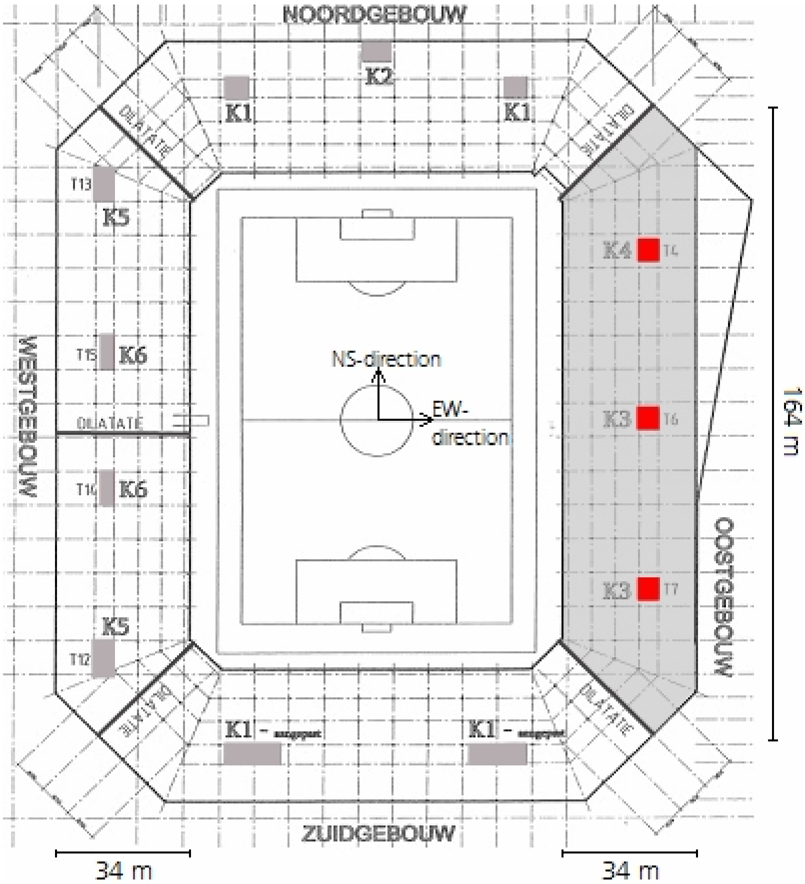



Figure 49: Overview structural system at the Euroborg stadium

Although the joints ('Dilataties') are not required for timber structures, the division in building parts is adopted in this thesis for reasons of simplicity.

### 4.2.1 Functional requirements

-2nd and -1st floor: Car park

- § Parking under an angle of 60º (width = 2.50 m, length = 5.15 m)
- § Obstacle-free span of at least 4.0 m for circulation
- § Parking lots at both sides of the driving lane
- § Free height of at least 2.30 m (may be reduced locally to 2.1 m)

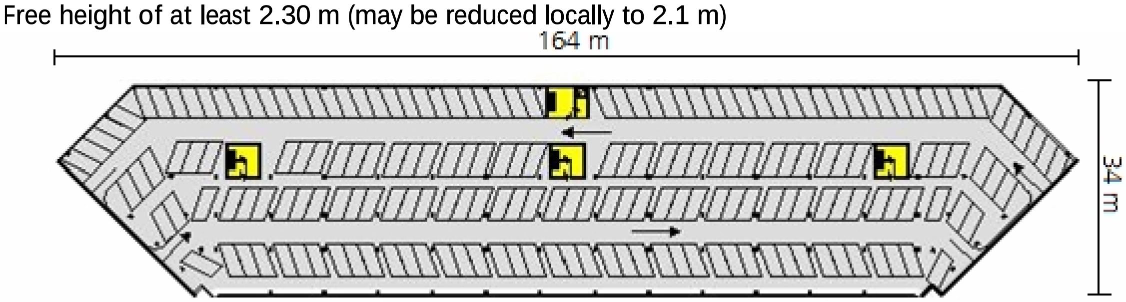



Figure 50: Functional plan for the -2nd floor

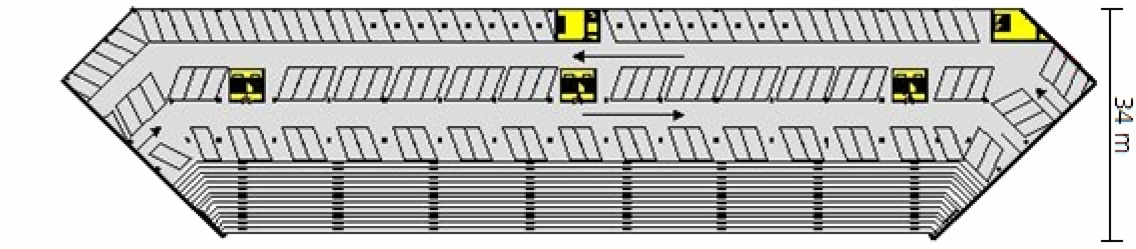



Figure 51: Functional plan for the -1st floor

- o Car park should not be considered inviolable at all costs (according to the project's principal)

Ground floor: Congregation area and commercial space

- § Obstacle-free to the greatest extent
- § A large freedom of functional arrangement to the end-user
- § Free height of at least 2.30 m, 2.60 m desired by project's principal

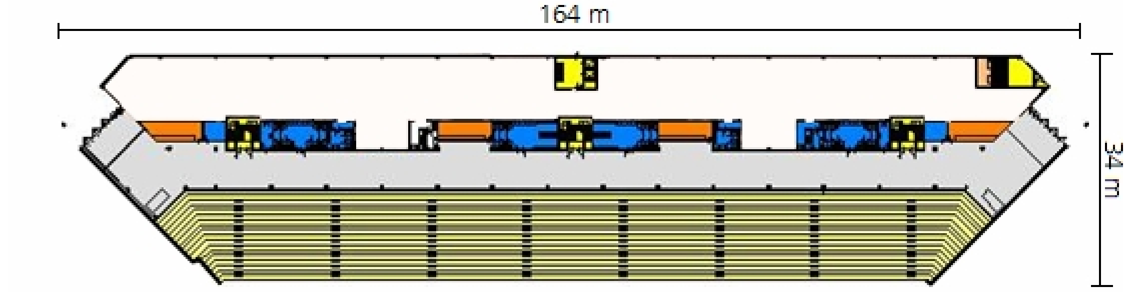



Figure 52: Functional plan for the ground floor

- o Congregation area can be divided in independent compartments, with own evacuation routes

1st and 2nd floor: Offices

- § A large freedom of functional arrangement to the end-user
- § These areas should have dimensions which are multiples of 1.80 m (Building Decree)
- § Free height of at least 2.30 m, 2.60 m desired by project's principal

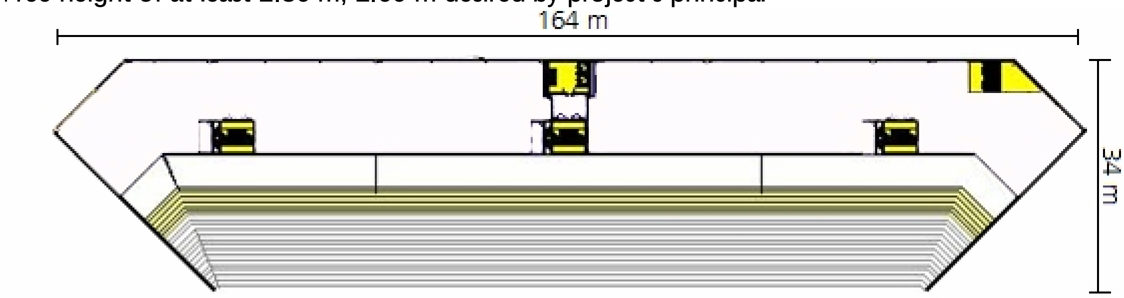



Figure 53: Functional plan for the 1st and 2nd floor

- o Free span of 14.4 m is desirable (project's principal)
- o The implementation of a large amount of walls is not desirable (project's principal)

At the backside of the grandstands, the façade could be used to act as a stabilizing element. Since offices and stadium entrances are located next to this façade, one should account for (large) openings in these elements.

Conclusion

It is desirable to implement as little stabilizing elements as possible, or at least at a relatively large centre to centre distance to each other. Main requirement for the -2nd and -1st floor is the passageway of at least 4.0 m wide. For the ground, 1st and the 2nd floor, it is required that the stabilizing elements do not put too much limits on the freedom of arrangement. It is therefore desirable to implement the stabilizing elements near the facades as much as possible, without limiting natural lightning to a large extent.

## 4.3 Suitable structural systems

To provide a stable structure, the stabilizing elements should be able to transfer the horizontal loads to the foundation. In this thesis, it is assumed that the floors act as diaphragms, transferring the horizontal loads that act on the façade to the stabilizing elements. In section 3.4.2, an excellent example of a structural system consisting of shear walls has already been discussed.

The Eastern building part of the Euroborg stadium is under influence of wind from various directions. For the structural design, the focus is laid on two major wind directions being the North-South (NS-) and the East-West (EW-) direction, see figure 54.

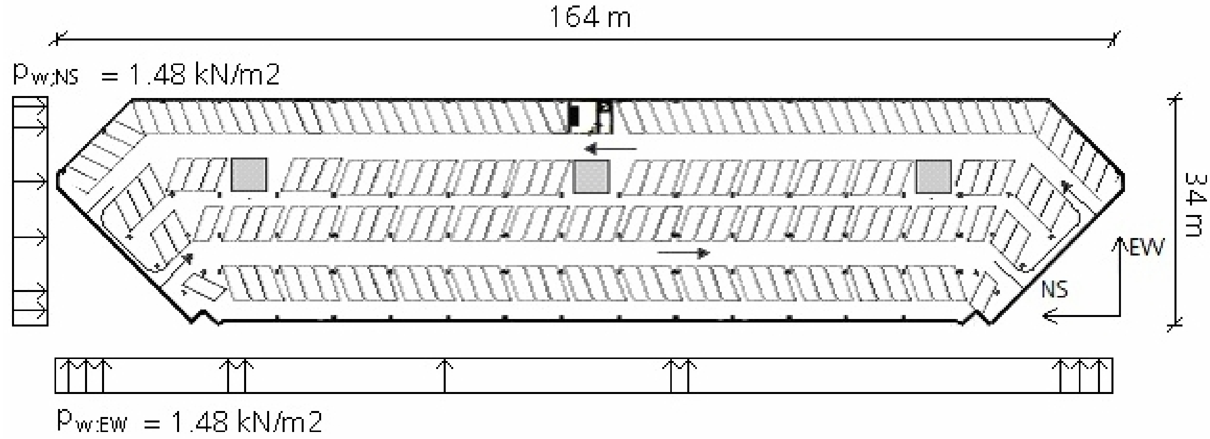



Figure 54: Main wind directions at the Eastern building part of the Euroborg stadium

The background and derivation of the loads as depicted in figure 54, can be found in Annex B and C.1.2.

Three kinds of stabilizing systems are considered in this thesis: a central core system, a system with shear walls and a system with trusses.

1. The central core system consists of multiple cores, whereby each core has walls in two directions. As a result, the cores are able to withstand horizontal loads in both directions. This solution is discussed more into detail in section 4.4.

2. A shear wall is able to withstand horizontal loads in one direction only. To provide a stable structure in two directions, at least three distinct stabilizing elements should be introduced, on the conditions that the three of them do not share the same point of rotation, and that they are not all parallel to each other. Therefore, shear walls should be introduced in both directions. This solution will be discussed in section 4.5.1.

3. The same is generally valid for a system consisting of trusses: they only provide stability in one direction. Preliminary calculations (which are not included in this thesis) have shown that the principle with trusses does not offer a feasible solution when the EW-direction is concerned. The required dimensions or amount of trusses exceed any realistic value. This solution is therefore only considered for the NS-direction. This solution is discussed in section 4.5.2.

## 4.4 Timber core system

### 4.4.1 Timber central core

#### Principle

When a core system is being used as a stabilizing element, the structure shall be considered as being braced. For braced systems, the core supports all other structural elements, such as floors and columns, horizontally. The floors are designed to act as diaphragms, transferring horizontal loads from the façades directly to the cores.

#### Implementation

When a timber core system is at stake, preliminary calculations (not included in this report) have shown that three cores should be implemented in the Eastern building part. For now, it is assumed that these cores are equally divided over the width of the building, being placed at a centre-to-centre distance of about 55 m to each other, see figure 55. Within the cores, the central stair cases are located. The cores have dimensions of about 5.5 (EW-direction) by 5.0 m (NS-direction) and a length of 19.22 m.

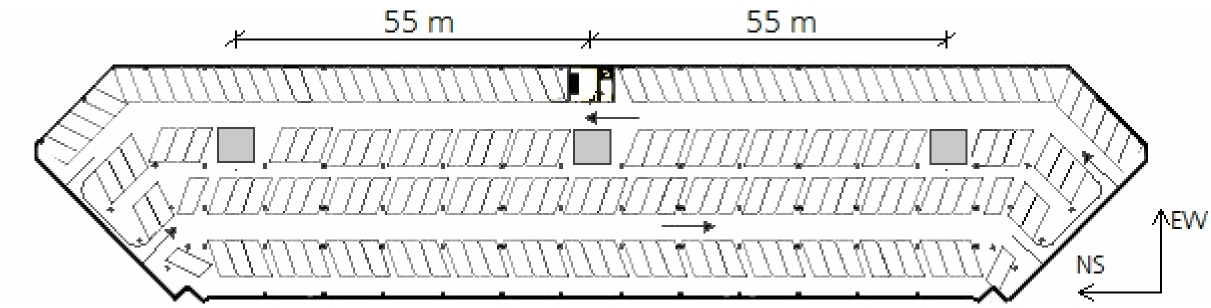



Figure 55: Proposed locations of the central cores

It is assumed that each of these cores takes an equal amount of the loads and that the three cores have equal dimensions.

To provide the reader with some insight on the acting loads, the governing load combinations (in EW- and NS- direction) are visualised in figure 56 and figure 57. The required dimensions of the cores follow directly from the combinations shown. For a complete overview on the considered load combinations, reference is made to C.1.3.

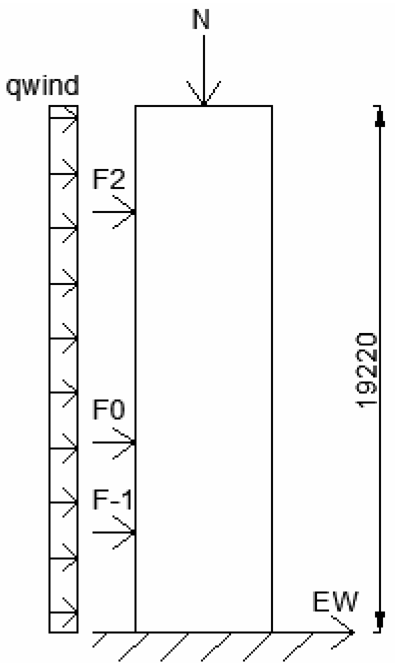



Figure 56: Governing load combinations EW-direction

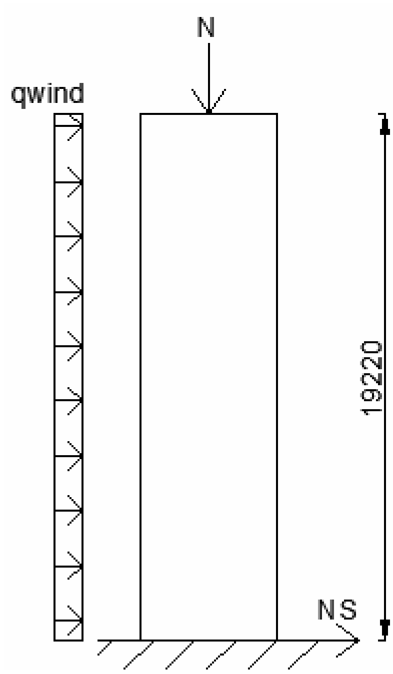



Figure 57: Governing load combination NS-direction

| | ULS | SLS |
|---|---|---|
| $q_{wind}$ [kN/m] | 134 | 81 |
| $F_2$ [kN] | 64 | 39 |
| $F_0$ [kN] | 52 | 32 |
| $F_{-1}$ [kN] | 52 | 32 |
| N [kN] | 4632 | 3245 |

| | ULS | SLS |
|---|---|---|
| $q_{wind}$ [kN/m] | 28 | 17 |
| N [kN] | 4632 | 3245 |

### Material

The proposed cores are composed of a plate-like timber product called Kerto-LVL, see Annex A.2. As is explained later on, the majority of the core walls are made from Kerto-S (Straight), which has all veneers running in the same direction. Another variant of the product is Kerto-Q (Quer), which has approximately 20% of its veneers running perpendicular to its longitudinal direction.

This composition can directly be regarded the main reason to choose for LVL: for Kerto-S all veneers add up to the strength of the element in the same direction. Making use of Kerto-Q or CLT, see Annex A.2, only a certain percentage of the cross-sectional area is addressed. Since the load direction is mainly in longitudinal direction, Kerto-S wall elements are regarded most beneficial.

§ The LVL elements are available in lengths up to 24 m, which is sufficient to cover the required height of 19.22 m directly. As a consequence, there is no need for intermediate connections in the longitudinal direction.

§ The elements are available in widths up to about 2.5 m. The required core width to accommodate stair cases and elevator shafts, though, is 5.0 m. Several panels should therefore be interconnected to provide the required width.

§ The thickness is limited to 90 mm for Kerto-S elements, and only 69 mm for Kerto-Q elements. According to the manufacturer, thicker elements can be obtained by gluing several elements on top of each other.

For the time being, the connections between the elements are neglected. It goes without saying that, when the timber core system appears to be the most beneficial system, these are accounted for retrospectively.

### Structural build-up

The layout of the core walls (e.g. location of openings) follows directly from the Euroborg stadium. The required dimensions of the core are determined by an iterative process. In this section, only the results of the last cycle of the iterative process are shown. The design calculations can be found in C.1.6.

The cores have a height of 19.22 m, which equals the distance between the ground floor and the top of the 3rd floor. As mentioned in the previous section, LVL elements having an equal length are used for the core walls. Each core consists of 4 walls, which are interconnected at their edges, see figure 58.

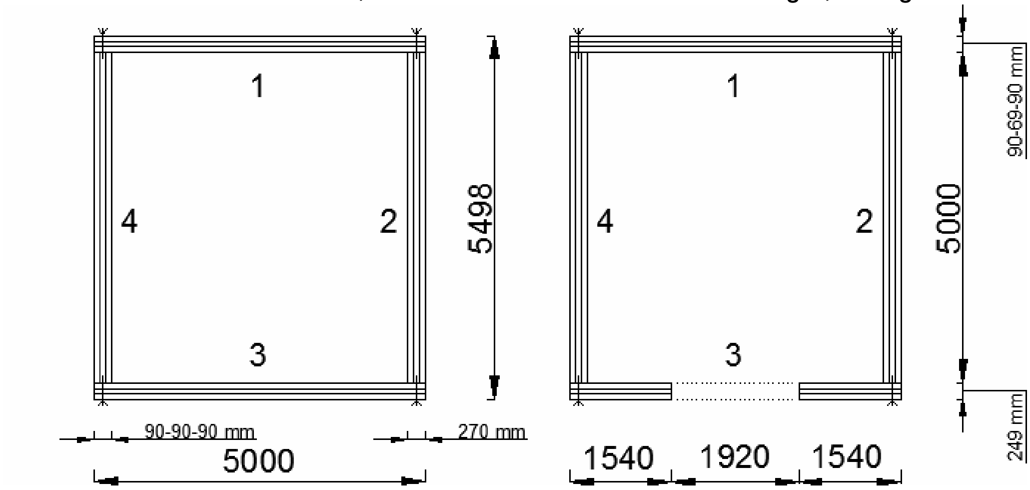



Figure 58: Sections of the fully enclosed core and the core with openings

Due to the maximum element width of 2500 mm, at least two LVL elements are required to provide the required wall width of 5000 mm. As a result, these two elements have to be connected in the middle of the wall.

When figure 58 is considered, one can find that walls 2 and 4 have a thickness of 270 mm, which is obtained by gluing 3 Kerto-S elements of 90 mm on top of each other. Such a connection should be made at the manufacturer, since it will be hard to obtain a high quality glued connection at the building site. Consultation with the manufacturer (Finnforest) proved that such a solution is feasible and commonly used in practice.

For walls 1 and 3 the design thickness was taken 249 mm only, due to the introduction of a Kerto-Q layer (maximum thickness 69 mm). The reason to implement a Q-layer follows from the appearance of openings in these walls, see figure 59. The Kerto-S elements simply fail to transfer the normal and shear stresses at the ‘lintels’ above the openings.

As mentioned above, some of the core walls contain openings to provide passages to and from the stair cases or the lifts. The width of these openings is taken 1.92 m, which follows directly from the Euroborg stadium. The height of the openings follows from the functional requirements as discussed earlier.

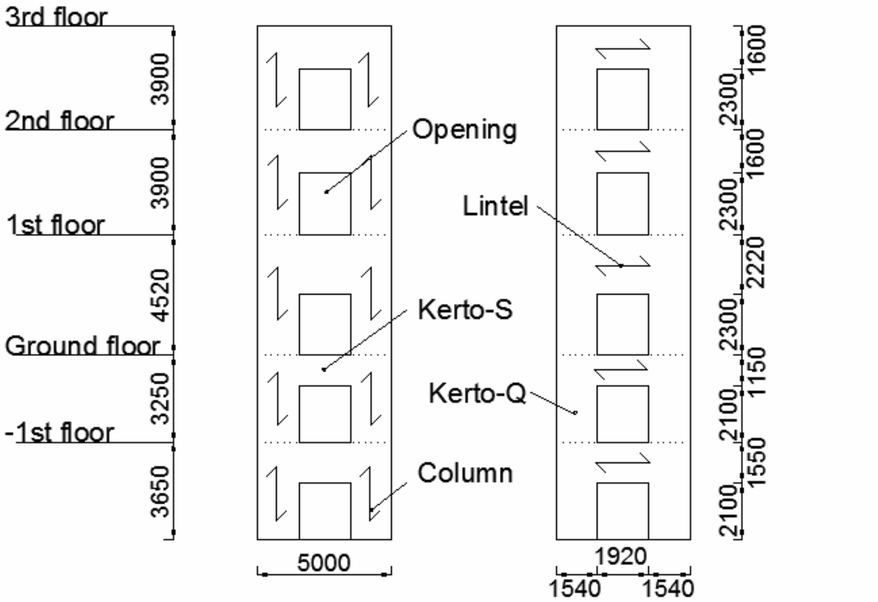



Figure 59: Kerto-S (left, 2 times 90 mm) and Kerto-Q (right, once 69 mm) layers including openings

For now, it is assumed that only one of the four core walls (see figure 58) is provided with such openings. This assumption is based on the design of the Euroborg stadium, in which the same situation applies.

Note: Up till this point, no attention has been given to the composition of the individual core walls. Since this is essential for the feasibility of the proposed core walls, a possible solution is provided in figure 60. The focus is laid on transportability and ease of the connections. A detailed analysis of this situation is provided in chapter 9: Core wall connections.

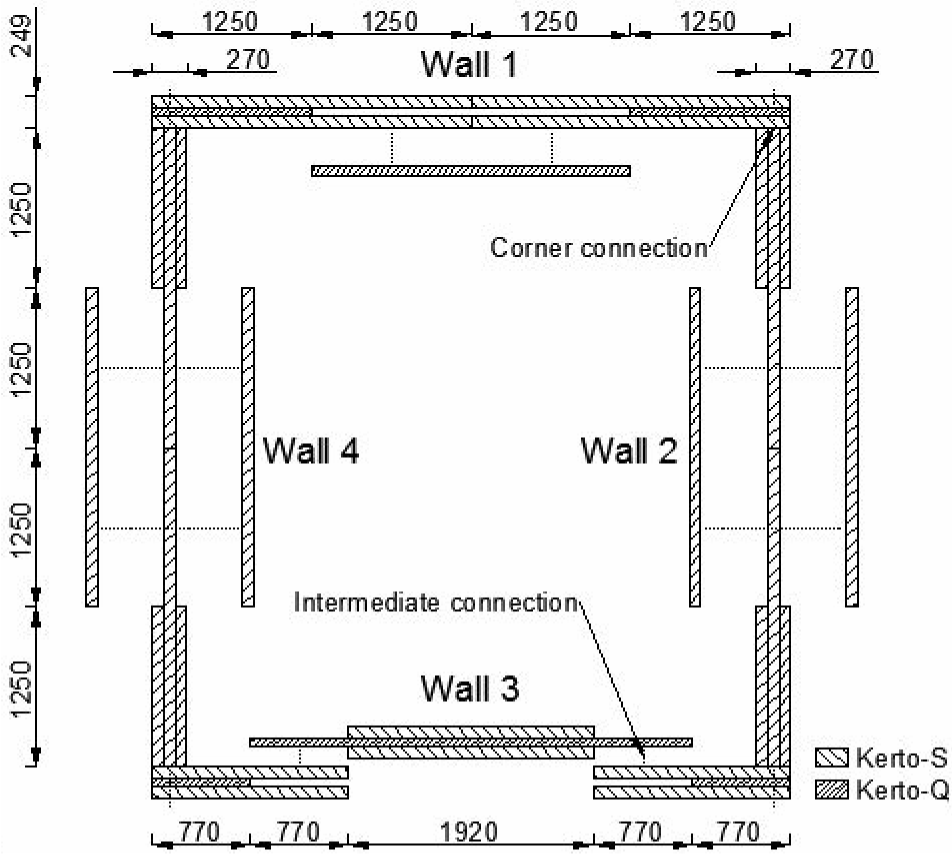



Figure 60: Proposed composition of the central core walls made from Kerto-S and Kerto-Q (indicative)

### Structural behaviour

In the preceding sections, the material and structural build-up of the three cores has been determined. It goes without saying that the proposed structural composition should comply with the requirements on strength and stiffness. Due to the preliminary character of the design, only global calculations have been performed.

### § Strength

Governing aspect in the strength calculations appeared to be the shear strength of the core walls 2 and 4. Since the unity-check for this specific situation becomes only 0.40, it is concluded that the cores hold a lot of additional safety.

### § Stiffness

Considering the governing wind direction, it is found that the bending stiffness of the proposed core amounts 17.83 * $10^{16}$ $Nmm^2$.

The maximum acceptable deflection is conservatively set to 25.6 mm (see C.1.6.1). Hereby, the minimum required bending stiffness is determined:

$$EI_{required;y} = \frac{ql^4}{8u_{max;total}} = \frac{81 * 19220^4}{8 * 25.6} = 5.398 \cdot 10^{16} Nmm^2$$

Since the bending stiffness exceeds the minimum required bending stiffness, the structure complies with the requirements.

Note: a complete and detailed version of the preliminary design calculations is provided in C.1.6.

### 4.4.2 Conclusions

Proposed structural system (see figure 58)

- Each of the three cores is composed from four individual walls, which are interconnected at their edges
- Each of the core walls has a length of 19.22 m and a width of 5 m
- Core walls 1 and 3 (see figure 58) are composed of 2 layers Kerto-S (180 mm) and 1 layer Kerto-Q (69 mm)
- Core walls 2 and 4 (see figure 58) are composed of 3 layers Kerto-S (270 mm)

Structural build-up

- At least two Kerto elements are required to obtain the required wall width of 5 m
- The required thickness is obtained by gluing layers on top of each other

Structural behaviour

- The proposed cores fulfil all requirements on global strength and stiffness (max U.C. = 0.40)

### 4.4.3 Points of attention

The following points of attention should be taken into consideration for upcoming design stages:

- Corner connections
- Composition of core walls (see figure 60)
- Structural behaviour of the lintels
- Choice of structural material
- Tensile anchorage to the foundation
- Erection method

## 4.5 System consisting of shear walls

### 4.5.1 Shear walls in EW-direction

#### Principle

Another solution is obtained by making use of so-called stabilizing walls, or shear walls. These walls are placed at a certain centre-to-centre to each other, and transfer the horizontal loads from the floors, through the walls, to the foundation. In contradistinction to the central core system, only the walls parallel to the acting wind direction act as stabilizing elements.

#### Implementation

Progressive insights and preliminary calculations (not included in this report) have shown that at least 6 walls are to be incorporated in the design to provide a reliable stabilizing system in the EW-direction. It is assumed that these shear walls are located at a regular c.t.c.-distance of about 27 m, see figure 61.

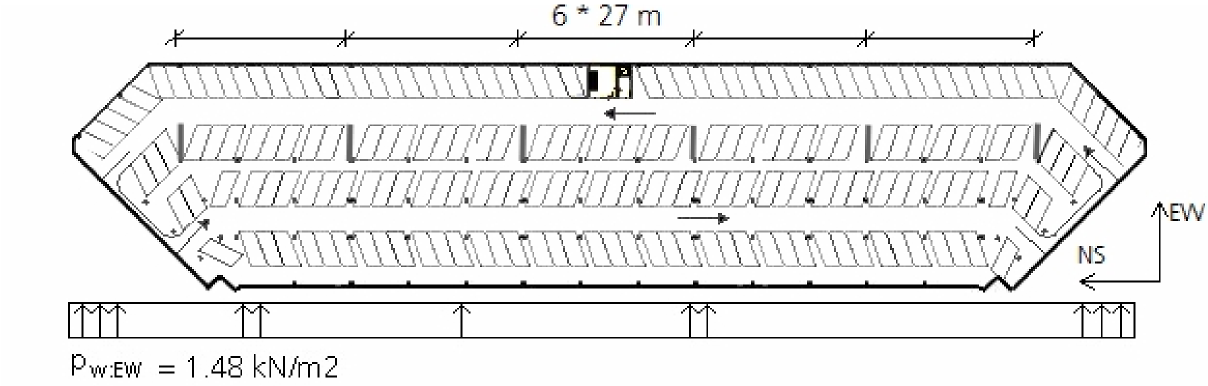



Figure 61: Proposed location of the small shear walls (c.t.c. about 27 m)

The walls are designed with a height of 19.22 m, which is equal to the distance between the top floor and the foundation. The width of the walls is taken 4.8 m which equals the distance between axis B and C, see figure 62. As a result, the walls are located at the front of the building where they do not interfere with the functional plan:

- At the lower floors, they are situated in between 2 parking lots, without limiting the passage of cars.
- For the upper floors, they are situated at the inner side of the building, not limiting commercial space nor blocking natural light.

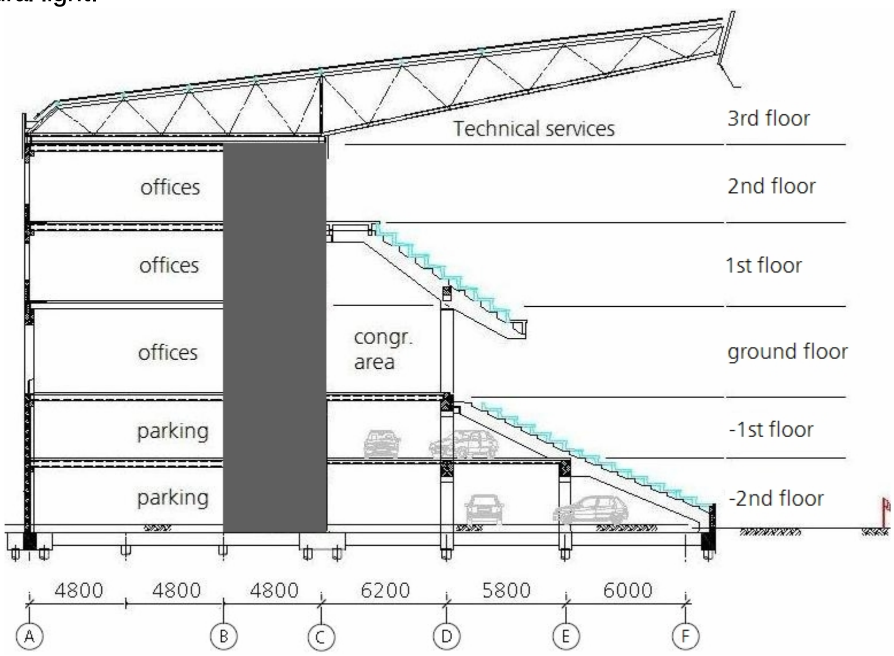
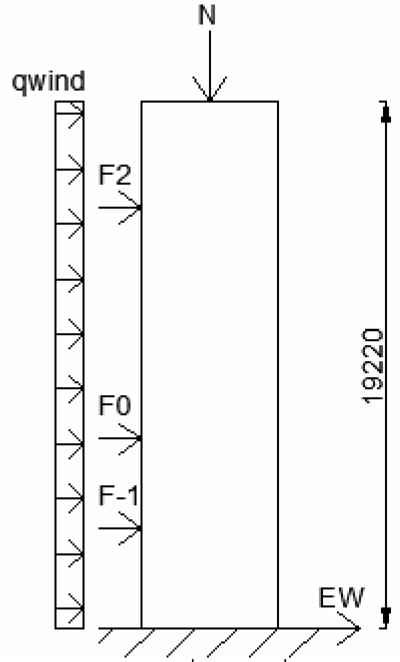



Figure 62: Small stabilizing walls spanning from top to foundation

As for the core system, it is assumed that all shear walls are identical and that each of them carries an equal amount of the loads.

To provide the reader with some insight on the acting loads, the governing load combinations (in EW-direction) are visualised in figure 63. The required wall width follows almost directly from these combinations. For a complete overview on the considered load combinations, reference is made to C.2.2.




| | ULS | SLS |
|---|---|---|
| $q_{wind}$ [kN/m] | 67 | 40.5 |
| $F_2$ [kN] | 33 | 19.5 |
| $16F_0$ [kN] | 26 | 16 |
| $F_{-1}$ [kN] | 26 | 16 |
| N [kN] | 3820 | 2651 |

Figure 63: Governing load combinations EW-direction

### Material

The shear walls are made from another plate-like timber product called Cross Laminated Timber (CLT), see Annex A.2. More specific, the product LenoTec is considered here. The reason to choose for this product is found in the dimensions in which the product is available:

§ CLT panels are readily available in lengths up to 20 m, whereby their length exceeds the required height of 19.22 m. As a result, there is no need for intermediate connections when these elements are used in longitudinal direction.

§ The elements have a maximum width of 4.8 m, which equals the required wall width and thus prevents the need for intermediate connections. In this perspective, the material CLT is beneficial over the previously mentioned Kerto-LVL.

§ The thickness of these elements varies between 50 and 500 mm, although not completely random: the available dimensions and build-up depends on the manufacturer.

### Structural build-up

The required width of the shear walls has been determined by an iterative process. In this section, only the results of the last step are shown. For the complete set of design calculations reference is made to C.2.5.

It appeared that the proposed shear walls require a thickness of at least 297 mm, when the product LenoTec is being used. In this case, the CLT element is composed of 11 laminates with a thickness of 27 mm each. Three of these laminates have their grain direction perpendicular to the load-bearing direction, whereby the effective width becomes 216 mm, see figure 64.

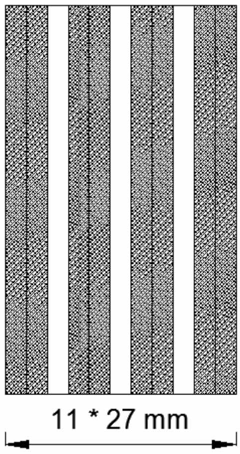



Figure 64: Build-up of the proposed CLT element

### Structural behaviour

In the preceding sections, the material and structural build-up of the shear walls have been determined. It goes without saying that the proposed structural composition should comply with the requirements on strength and stiffness. Due to the preliminary character of the design, only global calculations have been performed.

#### § Strength

Governing aspect in the strength calculations appeared to be the combination of bending moment and normal force. The unity-check for this situation amounts 0.90, whereby it is concluded that the structure holds some additional margin.

#### § Stiffness

The bending stiffness of the proposed shear walls amounts 2.22 * $10^{16}$ $Nmm^2$.

The maximum acceptable deflection is conservatively taken to be 32.8 mm (see C.2.5.1). Hereby, the minimum required bending stiffness is determined:

$$EI_{required;y} = \frac{ql^4}{8u_{max;total}} = \frac{40.5 * 19220^4}{8 * 32.8} = 2.106 \cdot 10^{16} Nmm^2$$

Since the bending stiffness exceeds the minimum required bending stiffness, it is concluded that the structure complies with the requirements.

Note: a complete and detailed version of the preliminary design calculations can be found in C.2.5.

## 4.5.2 Trusses in NS-direction

### Principle

A portal frame consists of at least two columns and a beam, which are inter-connected by a moment-resisting connection. As mentioned earlier, this kind of connection, especially in timber, puts huge demands on the dimensions of the individual members, which is obviously not desirable.

A similar system is obtained by inter-connecting these members by hinges, while introducing bracings in the corners of the frame, see figure 66. For these so-called trusses, normal forces are introduced in the bracings instead of bending moments near the connections.

### Implementation

Preliminary calculations have shown that at least 8 trusses are to be incorporated in the design to provide a suitable stabilizing system in the NS-direction, see figure 65. Since the trusses are located in the façade at the backside only, the shear walls in EW-direction are required to provide a stable structure (otherwise the structure would be able to rotate).

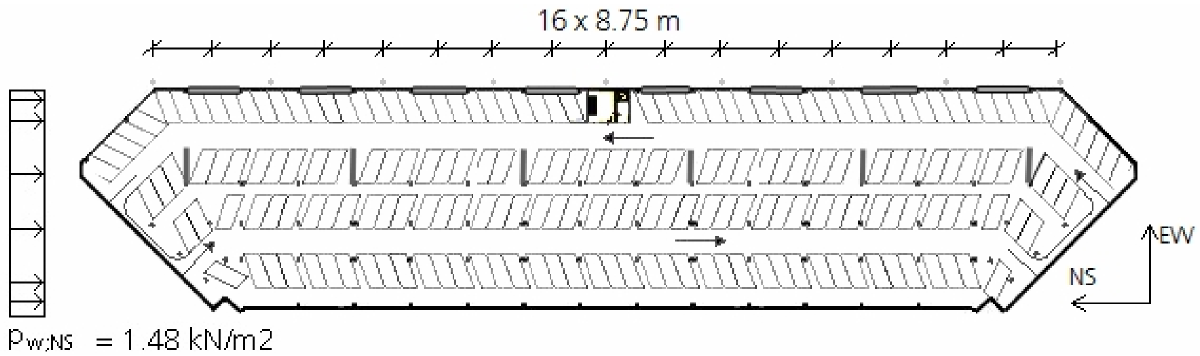
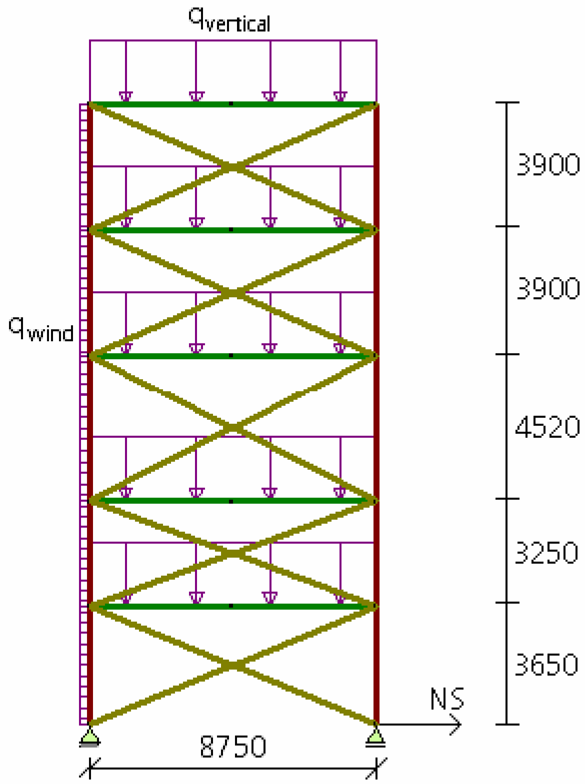



Figure 65: Proposed locations of trusses in the façade at the backside

It is assumed that each of the trusses carries an equal amount of the acting loads. Due to the large area of influence of each truss (see C.3.1), the vertical load on each truss becomes extremely large. To be able to provide a realistic structural system with trusses, additional columns are to be incorporated in between the trusses. These columns are also indicated in figure 65.

To provide the reader with some insight on the acting loads, the governing load combinations (in EW-direction) are provided in figure 66. It has to be mentioned that the acting vertical loading has been equally divided over the distinct floors. The dimensions of the individual elements follow almost directly from these combinations. For a complete overview on the considered load combinations, reference is made to C.3.2.




| | ULS | SLS |
|---|---|---|
| $q_{wind}$ [kN/m] | 10.4 | 6.3 |
| $q_{vertical}$ [kN] | 61.6 | 42.7 |

Figure 66: Governing load combinations NS-direction

## Material

All elements of the trusses are made from glued laminated timber quality class GL28h, see Annex A.2. The most common dimensions in which glulam is available, ranges between 60 and 400 mm for the width and between 100 and 2000 mm for the height. The product is available in lengths that exceed the required 19.22 m by far.

### Structural build-up

The required dimensions for the distinct elements have been determined by an iterative process. In this section, only the results of the last design step are shown. For the complete set of design calculations reference is made to C.3.4.

The columns are taken with dimensions of 200 by 625 mm. The most heavily loaded beams require dimensions of 200 by 1150 mm (or equivalent), while the bracings have dimensions of 150 by 150 mm. An overview of the build-up is provided in figure 67, where has not been accounted for a possible reduction in dimensions over the height.

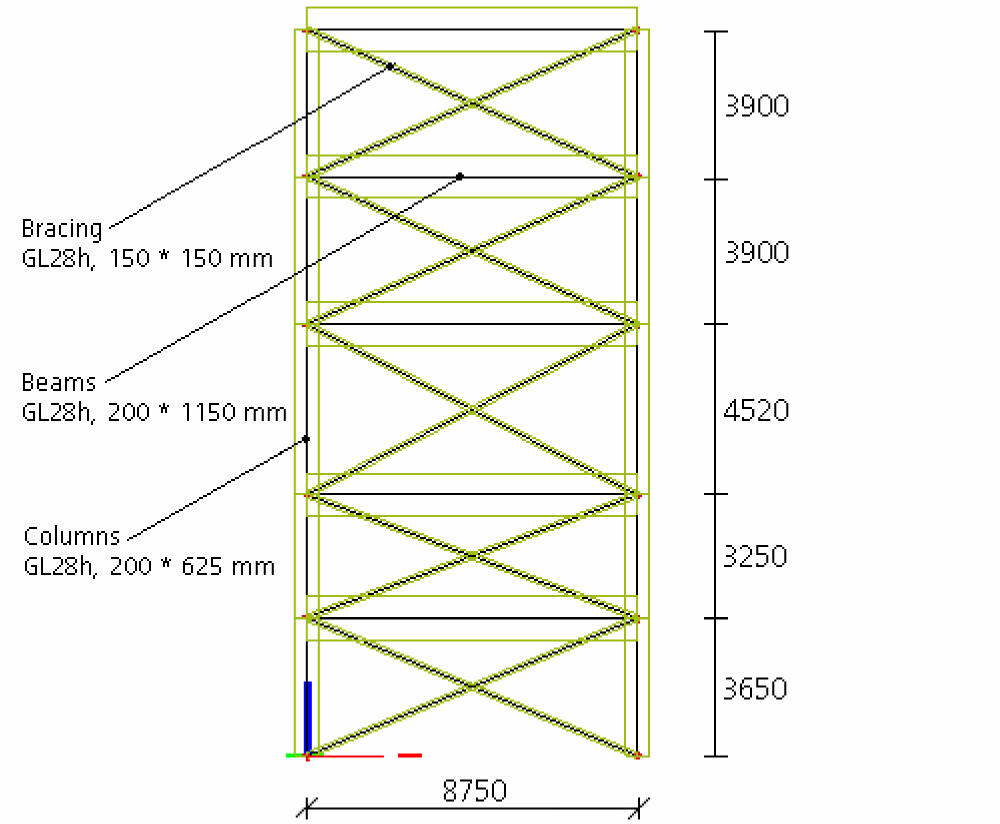



Figure 67: Build up of the proposed portal frame

### Structural behaviour

In the preceding sections, the material and structural build-up of the trusses has been determined. It goes without saying that the proposed structural composition should comply with the requirements on strength and stiffness. Due to the preliminary character of the design, only global calculations have been performed.

#### § Strength

As far as the columns and the bracings are concerned, the normal stresses are governing in the design. The proposed bracings are satisfying all requirements (U.C. = 0.52) as do the columns (U.C. = 0.67). When the beams are concerned, bending stresses due to the vertical loading are governing in the design. Also these elements satisfy the requirements (U.C. = 0.78).

#### § Stiffness

The maximum acceptable horizontal deflection was conservatively taken to be 38.1 mm (see C.3.4.1). Analysis of a computer model clarified that the deflection under the governing load situation becomes 27.5 mm at most. It is therefore concluded that the proposed system complies with the stiffness requirements.

Note: a complete and detailed version of the preliminary design calculations can be found in C.3.4.

### 4.5.3 Conclusions

#### Proposed structural system

- 6 shear walls are implemented in the design in EW-direction
- Each of these shear walls has a length of 19.22 m, a width of 4.8 m and a thickness of 297 mm

- 8 trusses are implemented in the design in NS-direction, as well as 9 intermediate columns
- Each of truss is composed of columns (200 by 625 mm), beams (200 by 1150 mm) and bracings (150 by 150 mm)

#### Structural build-up

- The shear walls are composed from LenoTec 297 elements
- Each wall is composed from one single element only when CLT is used

- All members of the truss are glued laminated timber quality class GL28h
- A lot of connections are required to create the trusses

#### Structural behaviour

- The proposed walls fulfil all requirements on global strength and stiffness (max U.C. = 0.90)
- All elements of the truss fulfil the requirements

### 4.5.4 Points of attention

The following points of attention should be taken into consideration for upcoming design stages:

- Choice of material for the shear walls (LVL instead of CLT)
- Tensile anchorage at the foundation
- Erection method

- Connections between the elements
- Dimensions of the intermediate columns

Note: One could argue that as a structural system in the NS-direction, shear walls could be used as well. This is certainly true and would result in a feasible and satisfying solution. But due to the huge resemblance to the earlier proposed wall system, working-out this solution was not considered to be interesting for this preliminary design.

## 4.6 Evaluation on structural systems

In the preceding sections two structural systems have been introduced: one consisting of timber cores (section 4.4), and one consisting of both shear walls (section 4.5.1) and trusses in the façade (section 4.5.2). In this section, these systems are evaluated and most beneficial system is determined.

Note: Stability in the NS-direction could also be provided by small shear walls instead of trusses. Since the purpose of this thesis is to show the possibilities of timber, this solution has been neglected (as shear walls are already used in EW-direction).

At first, the structural design of both systems is evaluated. Thereafter, the opinions of several experts are summarized. Combining the results, the most beneficial solution is determined.

### 4.6.1 Criteria

In sections 4.4 and 4.5, the structural design for both systems has been elaborated on. In this section, these designs are judged on various criteria.

#### Amount of timber required

§ Central core system
- 2 walls per core with dimensions 5.0 m by 19.22 m by 0.249 m = 47.86 $m^3$
- 2 walls per core with dimensions 5.0 m by 19.22 m by 0.270 m = 51.90 $m^3$

- 3 cores in total, results in a total of about 300 $m^3$

§ Combined system of shear walls and trusses
- 6 shear walls with dimensions 4.8 m by 19.22 m by 0.297 m = 164.40 $m^3$

- 8 trusses composed of columns, beams and bracings
  - 16 columns: 16 * 0.20 * 0.625 *19.22 = 38.44 $m^3$
  - 40 beams: 40 * 0.20 * 1.15 * 8.75 = 80.50 $m^3$
  - 80 bracings: 80 * 0.15 * 0.15 * 9.50 = 17.10 $m^3$

- This results in a total of about 300 $m^3$

It is concluded that, from the viewpoint of material use, none of the proposed systems is beneficial over the other.

#### Structural build-up

§ Central core system

As can be seen in figure 60, the timber core system requires quite an amount of fasteners: the Kerto elements have to be connected to form individual walls, and in turn, these walls have to be interconnected at their corners to from the core.

When the structural performance of the cores is considered, it is mentioned that the maximum U.C. amounts only 0.40. As a consequence, the proposed system holds a lot of additional safety and the dimensions may be further reduced.

§ Combined system of shear walls and trusses

The shear walls are made from a single CLT element, which implies that no intermediate connections are required. When the trusses are concerned, one can distinguish a lot of connections.

When the structural performance of the shear walls is considered, it is mentioned that the maximum U.C. is 0.90. Therefore, the walls are almost loaded to their maximum capacity. When the trusses are considered, the maximum U.C. is 0.78.

### 4.6.2 Experts' opinions

#### Functional plan

Arend Rutgers (A&E Architecten) states that the core system fits the architectural design perfectly: staircases and escapes routes have to be implemented every now and then, and these can be extracted from sight by implementing them within the cores. Having a small number of cores on a regular centre-to-centre distance, they do not interfere with the functional plan. A disadvantage of the proposed system is that the stair cases are located in the centre of the parking area: arriving at surface level (-2$^{nd}$ floor), visitors have to enter through the parking. This can be solved by locating the cores in the façade or by adding an elevation and locating the main entrance at ground floor level.

When the combined system is concerned, large bracings have to be implemented in the façade. Except for the limitations that this solution puts on the aesthetical appearance of the façade, this solution is not desirable when offices are located next to this façade (thus for the 1$^{st}$ and 2$^{nd}$ floor).

Rutgers states explicitly that, from his point of view, he would generally prefer the core system over the shear wall solution.

#### Construction method

Jaap Cederhout (BAM A&E Bouwmethodieken) states that erecting a timber core with dimensions of 5 by 5.5 m and a height of about 20 m introduces several challenges: erecting such a structure would require at least 2 cranes to support the core uniformly during erection. Next to that, additional measures would be required to prevent shear deformations near the corners: an internal bracings would be required.

When single shear walls are concerned, two cranes would be required as well. In addition, a huge scaffolding structure would be required to stabilize the walls temporarily.

From the perspective of erection, it is therefore recommended to divide the walls in sections of one or two stories for both solutions. When this solution is accounted for, the core system is regarded beneficial due to its simplicity and the speed of erection.

### 4.6.3 Evaluation

- The required amount of timber is more-or-less equal for both system
- The core system holds a lot of additional safety, while the combined system is close to its maximum capacity.

- From an architect's point of view, the core system is considered beneficial.
- From a construction technologist's viewpoint, the core system is considered beneficial although it is recommended to divide the core in storey-high segments.

## 4.7 Conclusions and recommendations

The goal of comparing the timber central core system with the combined system, was to find an answer to the following research question:

- *Which structural system in timber fits a stadium structure at best*

Considering the results of this chapter as stated in section 4.6.3, it is concluded that the central core system is the most beneficial solution when the structural system of a timber stadium is concerned. It is therefore decided to adopt such a system in the final design stage of this thesis. The complete overview of the proposed system has been given in section 4.4.

For the final design phase, it is recommended to pay attention to the following subjects:

- Corner connections
- Composition of core walls (see figure 60)
- Division of the core is smaller segments.
- Structural behaviour of the lintels
- Choice of material
- Tensile anchorage at the foundation

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# 5 Floors

## 5.1 Introduction

In this chapter, the design of the floors within the stadium is elaborated on. With reference to section 2.3, it is mentioned that a total of 5 floors, accommodating different use functions, are to be implemented in the design. Each of these floors being under the influence of distinct imposed loads.

The goal of this section is to provide insight on the possibilities of timber floors within a (stadium) structure, rather than to provide a complete and in-detail floor design. The functional arrangement of the Euroborg stadium is used as a reference. Goal of this chapter is to find an answer to the following research question:

§ *What kind of structural configuration is most beneficial for the floors within a stadium structure?*

To answer this question, various types of floors and structural configurations are investigated. Main assumption here is that the floors should be able to withstand the vertically acting forces as well as to act as a diaphragm to transfer the horizontal loads to the timber cores.

### 5.1.1 Approach

At first, a short introduction is provided on the various floor products that are considered in this thesis. This section is finished by means of an overview, where the distinct products are being compared to each other in a qualitative way.

Subsequently, several structural configurations are discussed one after another. For each of these configurations, it is determined what consequences their implementation holds.

Therefore, at first, the structural behaviour of the floor elements is investigated. Calculations on strength, stiffness and dynamic behaviour are carried out while differing loads and spans. This results in load-to-span diagrams for the floor products considered. Making use of the obtained diagrams, the most beneficial floor product is determined.

Accounting for this product, the maximum span-width is determined, which is then used to determine the dimensions of the support beams. This results in a worked-out structural plan for the governing floors within the stadium structure.

Finally, the diaphragm-action of each configuration is discussed into detail.

When all configurations are discussed, a comparison is made whereby both structural and functional aspects are considered. As a final step, a proposal is made for the final design phase of this thesis.

### 5.1.2 Design assumptions

In this section, the main design assumptions are presented. Additional information on the backgrounds is provided when required. All the assumptions presented apply to the functional plan of the Euroborg stadium.

General

§ Only the main part of the stadium building is considered, i.e. the part between axis A and C.
§ All design calculations are based on consequence class 3, since a stadium structure is concerned.
§ There has only been accounted for permanent loading caused by the dead-load of the floor products.
§ All design diagrams are made while considering $\Psi_0 = 0.5$ and $\Psi_1 = 0.5$.[1]

[1] The reason to choose for these specific Ψ-factors, is that they are considered average values. As a consequence, the diagrams provide insight in the average behaviour of the floor product.

### Serviceability

For the calculations on stiffness of the floors, the maximum additional deflection is conservatively taken $w \leq \frac{l}{500}$ mm (stiff partition walls).

When vibrations are concerned, the requirement on the fundamental frequency is taken $f_e \geq 5\text{Hz}$, as explained in B.2.7.

### Stability

Diaphragm-action is considered while accounting for the 'deep beam' theory as proposed in [ 17 ] and [ 18 ], since there is a lack of proper guidance in the building codes.

The design criteria as stated in these publications account for relatively small floors, having a width-to-span ratio (or vice versa) of 4:1 at most. In addition, it is assumed that the floor is composed of floor beams covered with sheeting on top. The theory assumes the following:

- § Shear distribution is uniform over the depth of the floor
- § It is sufficient to check the main chords and plate material
- § Horizontal deformations of the floor slab are limited and may therefore be neglected.

## 5.2 Introduction to the various floor products

In this section, the focus is on several types of floor products that are directly available on the market. Although there is a wide range of products available, the choice is made to consider the following products:

1. Timber hollow core slabs: Lignatur (section 5.2.1)
2. Cross-laminated timber: BSP Crossplan (section 5.2.2)
3. Stressed skin panels: Kerto Ripa (section 5.2.3)
4. Massive glulam floor element: Bresta (section 5.2.4)

For each of these products an overview is given on the characteristics such as build-up, available dimensions and functional characteristics. Thereafter, the distinct products are compared to each other.

Goal of this section is to introduce and to compare the distinct floor products rather than to make a deliberate choice for the most beneficial product. The scores provided in the overview at the end of this chapter should therefore be regarded as qualitative values.

### 5.2.1 Timber hollow core slabs: Lignatur

Lignatur elements can be considered as being timber hollow core slabs. Two different types are distinguished: the Lignatur Kastenelement (LK-Element) and the Flachenelement (LF-Element). The difference is found in the width of the floor element.

Lignatur elements are composed of several beams which are interconnected at their edges by means of glued connections. All the timber members are of strength class C24.

A LK-Element consists of a single box (or core) only and is available in the dimensions as shown in figure 68. A floor composed of LK-Elements consists of several boxes which are interconnected. The maximum available length is 12 m, but larger elements can be produced on request.

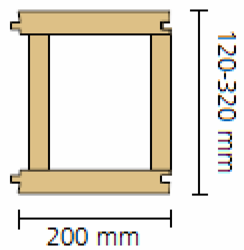



Figure 68: Lignatur Kasten Element (LK-element)

A LF-element holds several boxes (or cores) within each element, see figure 69. The maximum readily available length is 16 m, but larger elements can be produced on request.

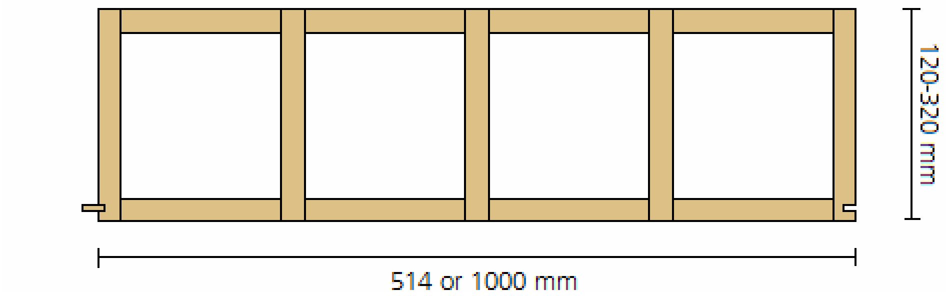



Figure 69: Lignatur Flachen Element (LF-element)

The weight of an LK-element is somewhat larger (38-68 kg/m$^2$) than that of a LF-element (34-48 kg/m$^2$), due to the larger amount of ribs.

Due to the larger available element length and width, and the lower self-weight, only the LF-element is considered in this thesis. Therefore, from this point on, when a Lignatur element is mentioned the Lignatur LF-element is at stake.

Characteristics

- § Diaphragm-action can be obtained with shear connectors or by applying an OSB-layer on the top
- § A fire protective layer or (sound-)isolation can be implemented within the cores, see figure 70.

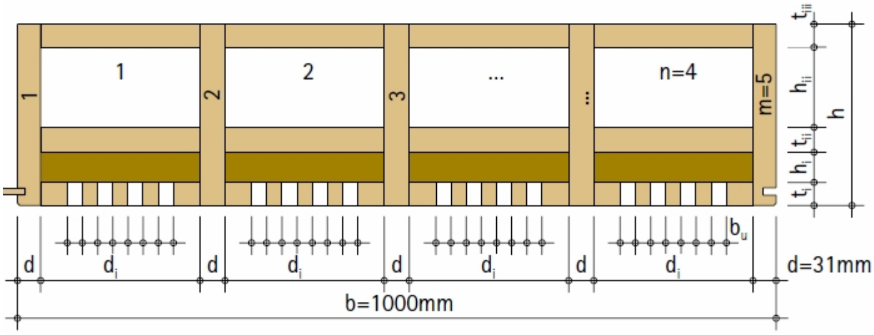



Figure 70: LF-element including a fire protective layer and a sound absorbing layer

- § A fire resistance of up to 90 minutes is reached by increasing the thickness of the bottom flange (even without a fire protective layer).
- § Ducts may be implemented within the cores, whereby the height of the floor package is limited
- § The elements lack the need for temporary strutting during the erection phase

## 5.2.2 Cross laminated timber: BSP Crossplan

Cross-laminated timber panels can also be used as a floor slab. Dimensions and build-up differs between the different manufacturers.

The BSP Crossplan elements are available in lengths up to 16.5 m and widths limited to 3000 mm. The height is ranging between 78 mm and 278 mm.

Figure 71: BSP Crossplan element

Characteristics

§ The mass of the product ranges between 35 and 135 kg/m$^2$.
§ The fire resistance exceeds 90 minutes
§ Sound isolation can be improved by applying a top layer
§ Ducts and installations increase the height of the floor package
§ The product lacks temporary strutting during erection

### 5.2.3 Stressed skin panels: Kerto-Ripa

A stressed skin panel, or a waffle floor, consists of several webs coupled by a flange at the top. The webs are in the direction of the span, while the flange spans in both directions. For the Kerto-Ripa floor, the webs are made from Kerto-S, while the flange is made from Kerto-Q. The webs and the flanges act together, thereby increasing the bending stiffness of the webs alone.

The Kerto-Ripa elements are available in lengths up to 16 m, while the thickness of the flange is 27 or 33 mm. The width of the webs varies between 31 and 69 mm.

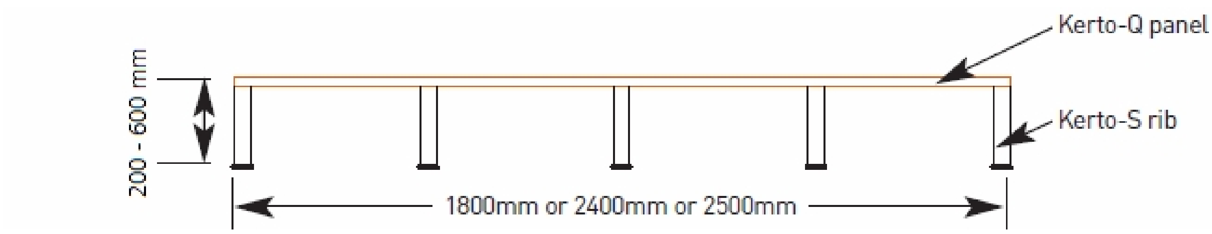



Figure 72: Kerto-Ripa element

Characteristics

§ The mass of the product is about 30 kg/m$^2$.
§ Additional measures are required to provide the required fire resistance.
§ Sound absorption of the product is minimal
§ Ducts and installations can be implemented in between the webs.

### 5.2.4 Massive Glulam Floor element: Bresta Decke

The 'Decke' floor from Bresta consists of numerous small beams which are glued together to form a massive floor element. The elements can be considered as glued laminated timber, where the laminates are connected horizontally instead of the 'usual' vertical arrangement.

The Bresta Decke elements are directly available up to 12.5 m in length, but elements with lengths up to 17.5 m can be produced on request. The optimal span-width is found around 6.5 - 7.5 m, according to the product specifications.

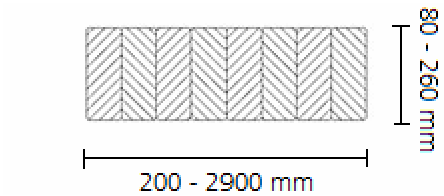



Figure 73: Decke element

The laminates should have a strength that corresponds to strength class C18 at least. In this thesis, the focus will be on Bresta Decke elements having laminates of C24 quality.

Characteristics

§ The mass of the product ranges between 30-100 kg/m$^2$.
§ For elements having larger height, a fire resistance of 90 minutes is obtained.
§ The bottom side of the element can be provided with a special acoustic profile.
§ Ducts and installations increase the height of the floor package.

### 5.2.5 Overview

In the previous sections, various floor products were introduced. The obtained information has been summarized in the overview of table 2. This table can be used as a reference when determining the best suitable floor product. Additional information on (some of) the judgement criteria is provided underneath the table.

Table 2: Overview on the characteristics of various floor types

| | Lignatur | CLT | Kerto Ripa | Bresta |
|---|---|---|---|---|
| Available dimensions | 0 | ++ | + | + |
| Mass | + | - - | ++ | - |
| Behaviour in fire | ++ | ++ | - - | ++ |
| Sound isolation | + | 0 | - | ++ |
| Ducts | ++ | - | ++ | - |
| Ease of erection | ++ | ++ | ++ | ++ |

Criteria

§ *Available dimensions*
The larger the range of dimensions, the better the score. Especially length and width are concerned here, since these are governing when the amount of shear connectors is concerned.

§ *Behaviour in fire*
Are the elements able to withstand a fire for at least 90 minutes by themselves, or are additional measures required?

§ *Ducts*
The possibility to implement ducts and small recesses is considered here. It is assumed to be beneficial when the ducts can be located within the element, since this limits the total height of the package.

## 5.3 Stretcher bond pattern

A stretcher bond pattern is a brickwork pattern whereby the location of the end joints shifts each row, as shown in figure 74. For this situation, the longitudinal axes of the floor elements point in the NS-direction. Together, the floor elements should behave as a stiff plate, since diaphragm-action is required.

In between the ‘supports’ (the cores) of the floor plate, compression and tensile forces develop. To be able to transfer these forces to the cores, a ‘tensile tie’ and a compression beam is implemented in the design, see figure 74.

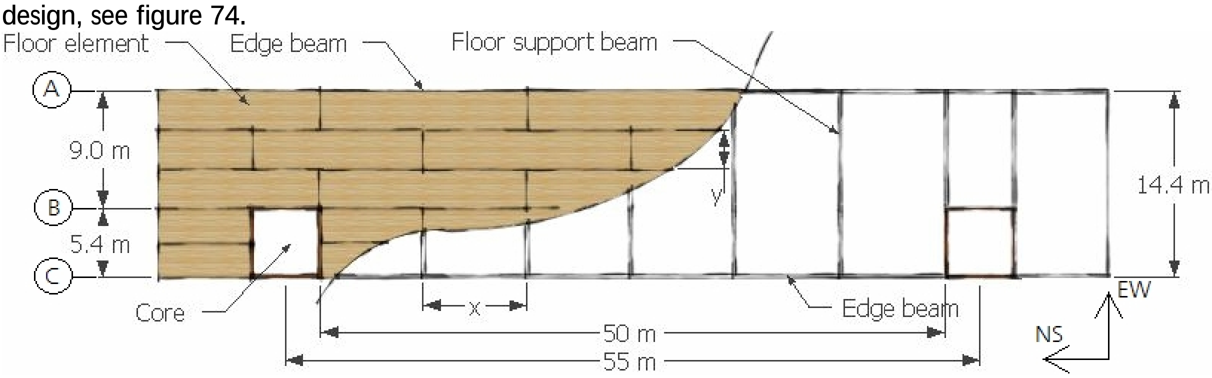



Figure 74: Support beams underneath the floor elements (indicative, not to scale)

### 5.3.1 Floor elements

Since all floor elements have to be supported at their ends, beams are implemented in the design at those locations where adjacent elements are laid against each other. Due to the stretcher bond pattern, this results into beams supporting some of the floor elements at their ends and some at mid-span. As a consequence, each of the floor elements is supported by three beams, see figure 75.

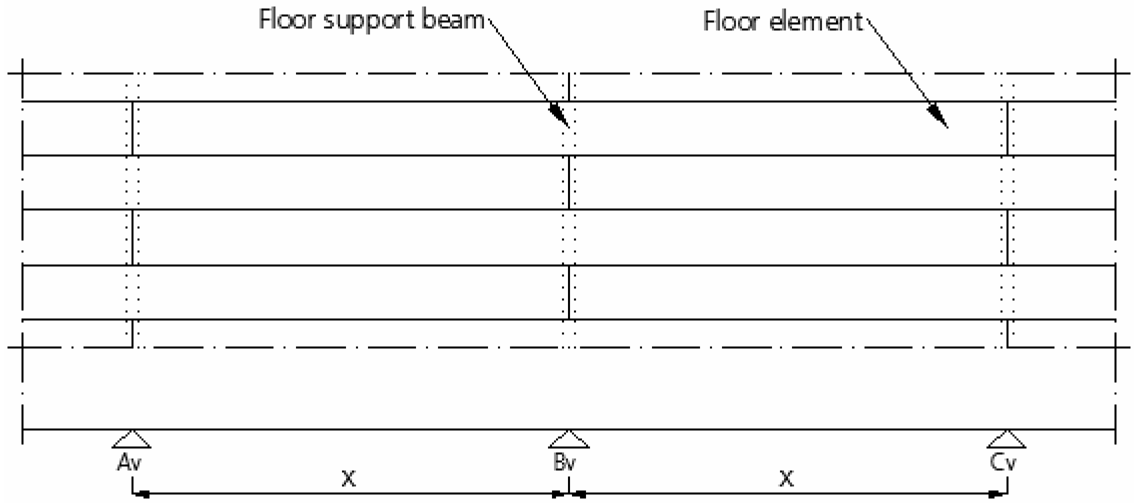



Figure 75: Floor elements supported by three support beams

Considering the earlier mentioned maximum lengths of the distinct products (section 5.2), the maximum c.t.c.-distance of the support beams is determined:

| | |
|---|---|
| Lignatur: | $x \leq 8.0$ m |
| BSP Crossplan: | $x \leq 8.25$ m |
| Bresta Decke: | $x \leq 8.75$ m |
| Kerto Ripa: | $x \leq 8.0$ m |

Accounting for these products and their characteristics, calculations are carried out on strength, stiffness and dynamic behaviour. This resulted in the height-to-span-width diagrams as shown in figure 76. For the complete set of calculations and their backgrounds, reference is made to D.2.1.

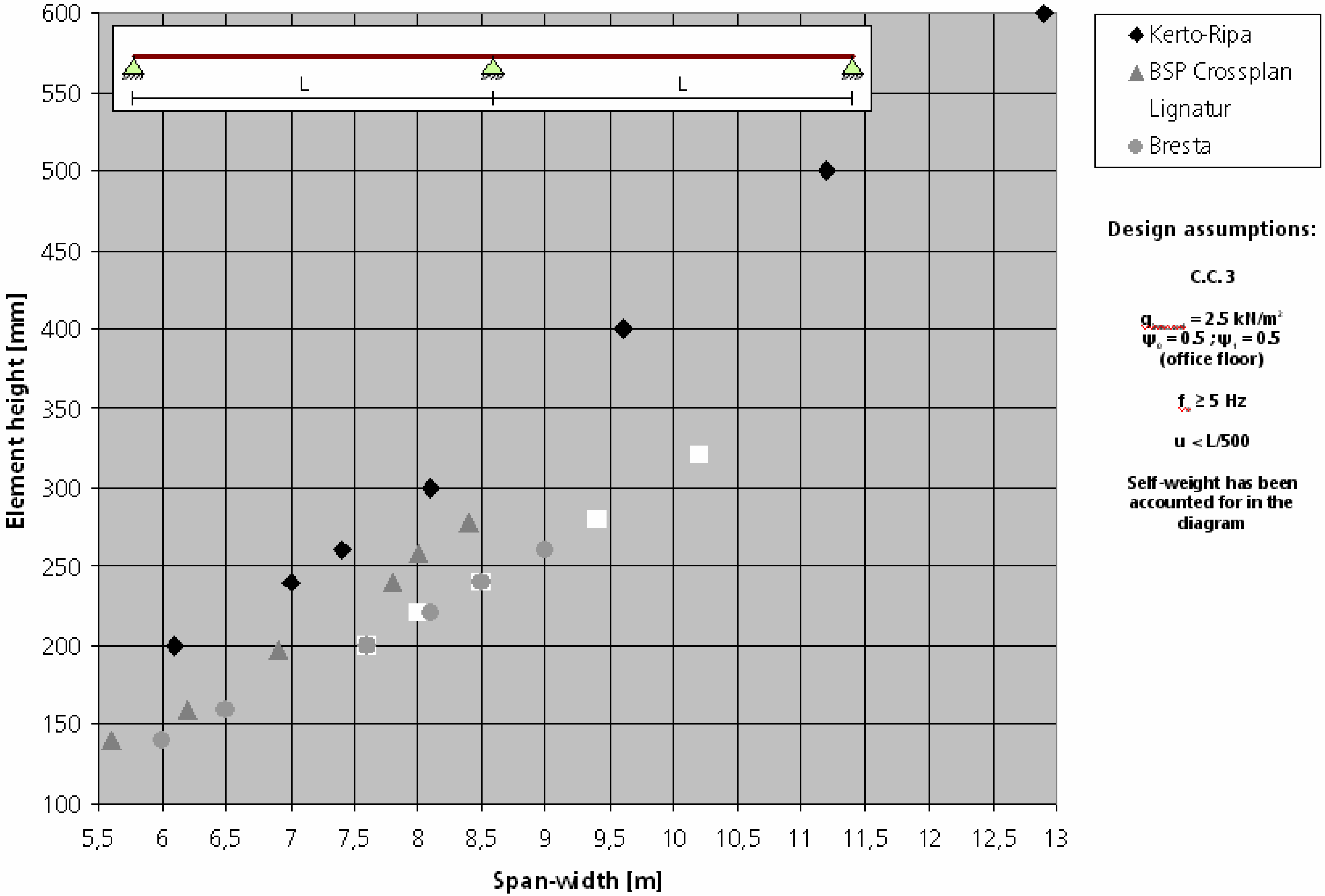



Figure 76: Height-to-span-width diagram for an element on three supports with equal spans

Considering figure 76, it is found that several products are able to span a distance that exceeds the maximum c.t.c.-distance of the support beams (due to the limited element length). This implies that the maximum element length is limiting rather than the structural behaviour. It is therefore concluded that it is not beneficial to make use of these products for this specific configuration.

This is obviously true for the situation where office floors are concerned (q = 2.5 kN/m$^2$; $\Psi_0$ = 0.5; $\Psi_1$ = 0.5), but it is questionable whether this argument holds for the other functional areas (with, for example, larger Ψ-factors and larger imposed loads).

Therefore, the same procedure as elaborated in D.2.1 is followed, while accounting for the design conditions of a congregation area (q=5.0 kN/m$^2$; $\Psi_0$ = 0.25 and $\Psi_1$ =0.7). This has resulted in the overview(1) of table 3, which provides the maximum span for both an office area and a congregation area. The maximum element length has not be accounted for here.

Table 3: Theoretical maximum spans under specific loading conditions

| Product | Office area $l_{max}$ [m] | Congregation area $l_{max}$ [m] | Max. element length $l_{max}$ [m] |
|---|---|---|---|
| Kerto Ripa 6*69*600 | 12.9 | 10.6 | 8.0 |
| Kerto Ripa 6*51*500 | 11.2 | 9.1 | 8.0 |
| Lignatur 320 | 10.2 | 8.4 | 8.0 |
| Lignatur 280 | 9.4 | 7.7 | 8.0 |
| Kerto Ripa 6*45*400 | 9.6 | 7.7 | 8.0 |
| Bresta 260 | 9.0 | 7.6 | 8.75 |
| BSP 278 | 8.4 | 7.1 | 8.25 |

(1)To prevent ambiguities, it is stated explicitly that the results concerning the congregation area can <u>not</u> be extracted from figure 8.

It appears that, when a congregation area is concerned, the maximum spans from a structural point of view are generally quite close to the maximum element length. It is therefore concluded that a floor element over three supports offers benefits for the congregation areas.

Since it is not desirable to implement a different structural plan for each storey, the governing floor is used as a basis for the design. Since the governing floor is the ground floor (the congregation area), all floor elements are designed to span over 3 supports.

From table 3, it appears that the BSP 278, Bresta 260, Kerto Ripa 6*45*400, Lignatur 280 and Lignatur 320 are the most efficient products: their maximum span is close to the maximum element length.

Before a deliberate choice between these products is made, the most important characteristics are summarized, see table 4.

Table 4: Product characteristics

| Product | Element height [mm] | Weight [kN/m$^2$] | Maximum span width [m] |
|---|---|---|---|
| Lignatur 320 | 320 | 0.72 | 8.0 |
| Lignatur 280 | 280 | 0.69 | 7.7 |
| Kerto Ripa 6*45*400 | 400 | 0.35 | 7.7 |
| Bresta 260 | 260 | 1.09 | 7.6 |
| BSP 278 | 278 | 1.33 | 7.1 |

This leads to the following conclusions:
§ Due to the relatively large element height, the Kerto Ripa product is considered unfavourable
§ The BSP 278 product has a relatively large self-weight, while the maximum span is least.

Neglecting these products, the Lignatur 320 element is considered the most beneficial: the weight is considerably low, while the span is largest. The Lignatur 320 product is therefore assumed to be the best solution when the stretcher bond configuration is considered.

In case problems concerning the free height are at stake, it is advised to consider the Lignatur 280 element rather than a Lignatur 320 element.

## 5.3.2 Support beams

Accounting for the Lignatur 320 element, the consequences of implementing it in the design are considered. At first, the focus is laid on the governing ground floor. Thereafter, the office floors are considered.

It is assumed here that the smaller the amount of support beams, the more beneficial the solution. From the previous section it appeared that the maximum span of a Lignatur 320 element over three support beams is 8 m. It is therefore decided to implement support beams at a c.t.c.-distance of 8 m. It is assumed that all support beams are glulam members of quality class GL28h.

In the most ideal situation, these support beams span between both facades of the stadium structure at once, see figure 77. In this case, no intermediate columns are required within the functional areas.

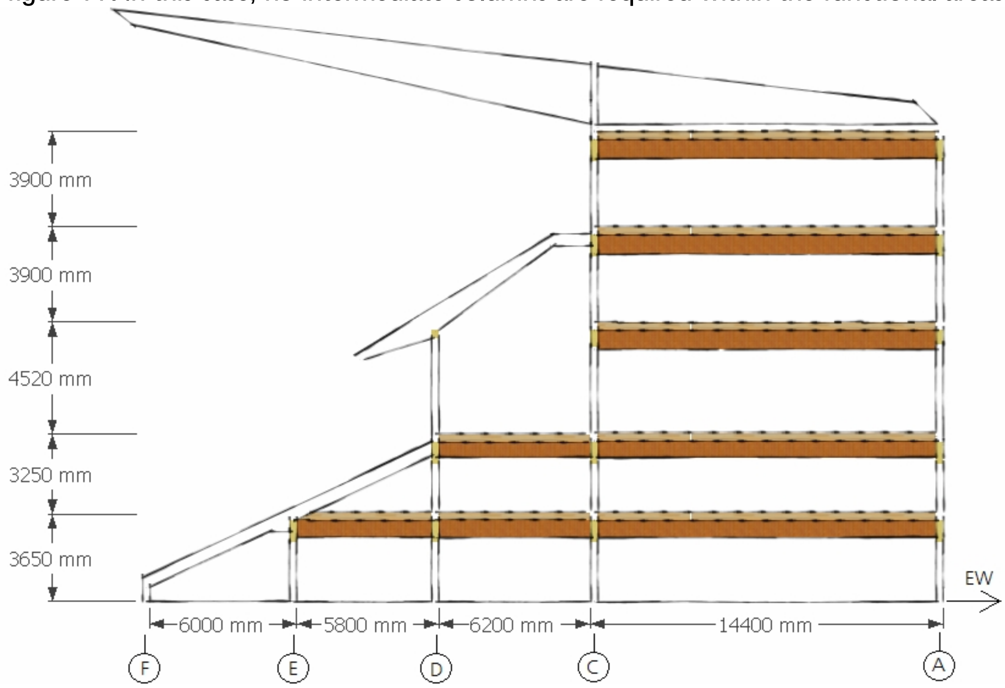



Figure 77: Support beams spanning between the façade and columns behind the grandstand (indicative)

### Ground floor

The required free height underneath the ground floor is 2.3 m, which may be locally reduced to 2.1 m. With a storey height of 3.25 m, the maximum allowed structural height for the ground floor becomes 1.15 m.

Calculations (see D.3.1) on this specific situation have shown that the dimensions of the support beams should be at least 220 by 1870 mm to span 14.4 m at once, see figure 78. Together with the Lignatur element height of 320 mm, this results in a total structural height of about 2200 mm which exceeds the maximum acceptable height by far. It is therefore concluded that an intermediate column has to be accounted for.

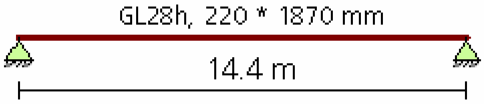



Figure 78: Support beams spanning 14.4 m at once

Accounting for the maximum structural height of 1150 mm and the Lignatur element height of 320 mm, the remaining maximum height for the beam is only 830 mm. Accounting for a single GL28h beam with dimensions of 220 by 830 mm, the maximum span-width is only 7.0 m. As a result, at least two intermediate columns are required to span the desired 14.4 m.

An acceptable solution is found by implementing one of the intermediate columns in line with the core walls, at axis B. The other column should be incorporated in between axis A and B, preferably at axis A' , at a distance of 5 m from the façade, see figure 82.

Since the spans become even smaller than 7.0 m for such an arrangement ($l_{max}$ = 5.4 m), the support beams require a height of only 640 mm for this situation, see figure 79.

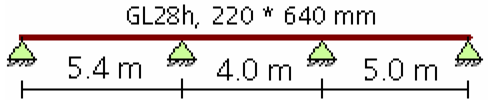



Figure 79: Support beams spanning 5.4 m at most

In case it is considered highly undesirable to incorporate intermediate columns in the design, one can opt for an alternative solution. It could, for example, be proposed to implement two support beams directly adjacent to each other, see figure 80.

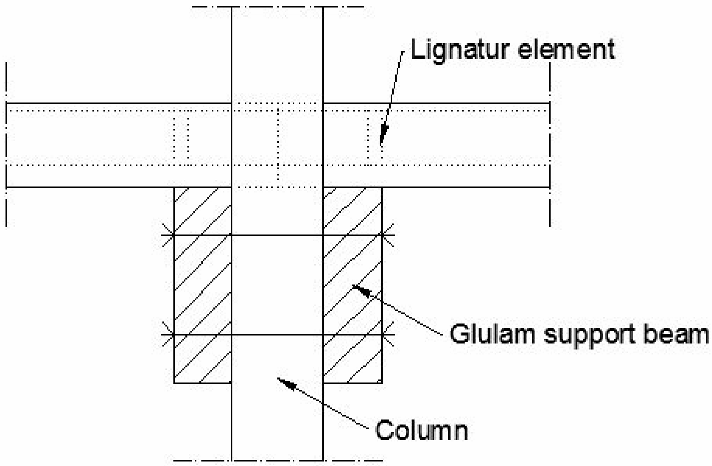



Figure 80: Alternative solution to increase the span-width of the beams

It appears that the maximum span-width for this situation becomes 9.30 m, whereby it is concluded that the intermediate column within the office areas can be left out of the design.

Note: when the Lignatur 280 element is implemented at the ground floor, the maximum beam height may become 870 mm. Such a beam is able to span 7.5 m directly, whereby only one intermediate column is required. But since an access lane is present in the middle of axes A and C, the minimum required span-width amounts 9.0 m. Implementing the Lignatur 280 elements is therefore no improvement.

### Office floors

The desirable free height underneath the governing (2nd) office floor is 2.6 m. With a storey height of 3.90 m, this results in a maximum structural height of 1.30 m for the 2nd floor.

Calculations (see D.3.1) have shown that the dimensions of the support beams should be at least 220 by 1450 mm to span 14.4 m at once, see figure 81. Accounting for Lignatur 320 elements, a structural height of 1770 mm is required, which exceeds the height available. As a result, an additional column is required.

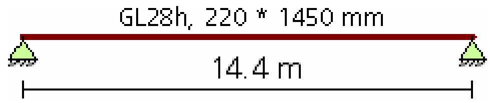



Figure 81: Support beams spanning 14.4 m at once

Implementing an additional column at axis B, see figure 82, the maximum span becomes only 9.0 m. This results in a support beam with dimensions of 220 by 800 mm.

Accounting for the solution as proposed in figure 80, the maximum span-width becomes 12.5 m. Incorporating such a solution underneath the office floors is therefore not considered beneficial.

### Overview

Resuming all the above, the following structural configuration is found, see figure 82. The introduction of 'double beams' underneath the congregation area is not accounted for.

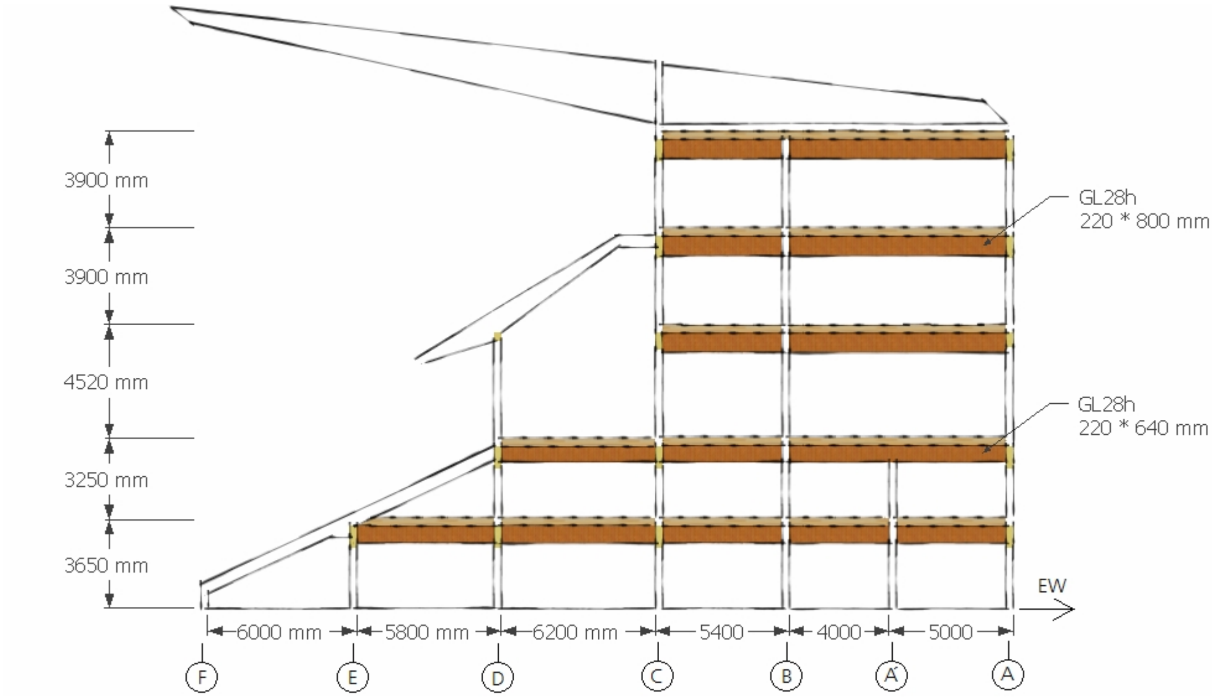



Figure 82: Proposed structural solution for the stretcher bond pattern

## 5.3.3 Columns

In the preceding section, the dimensions of the support beams were determined. These support beams are in turn supported by columns, which transfer the vertical loads to the foundation.

When the most heavily loaded column is considered, calculations (see D.4.1) have shown that the dimensions should be at least 350 by 350 $mm^2$, when timber quality GL28h is considered. Such dimensions are considered acceptable.

## 5.3.4 Diaphragm-action

When diaphragm-action is concerned, two distinct situations will be considered, being the global action and the local action. As stated earlier, use is made of the theory as presented in [ 17 ] and [ 18 ]. When global action is concerned, the focus is on the dimensions of the end and edge beams. When local action is concerned, the focus is on the transfer of shear forces in the end joints and the longitudinal joints in between distinct floor elements.

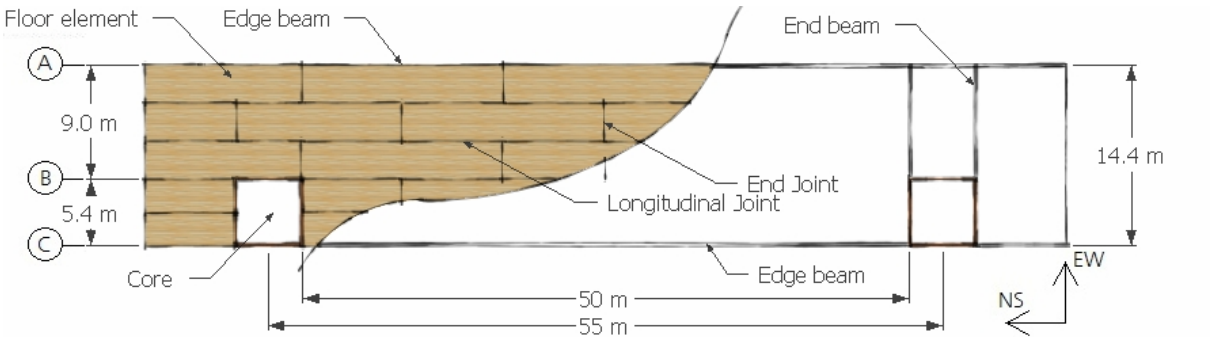



Figure 83: Indication of relevant parameter for diaphragm-action

As stated in the chapter 4, three cores are implemented in the design, being placed on a centre to centre distance of 55 m to each other. The total width of the stadium structure is 164 m. At first, the global situation is considered.

### Global diaphragm-action

The governing floor for global diaphragm-action appears to be the 2nd floor, due to the highest value for the combination of horizontal loads (wind and grandstand loading), see D.5.2.

These horizontal loads are transferred to the cores by the floor diaphragm, see figure 84.

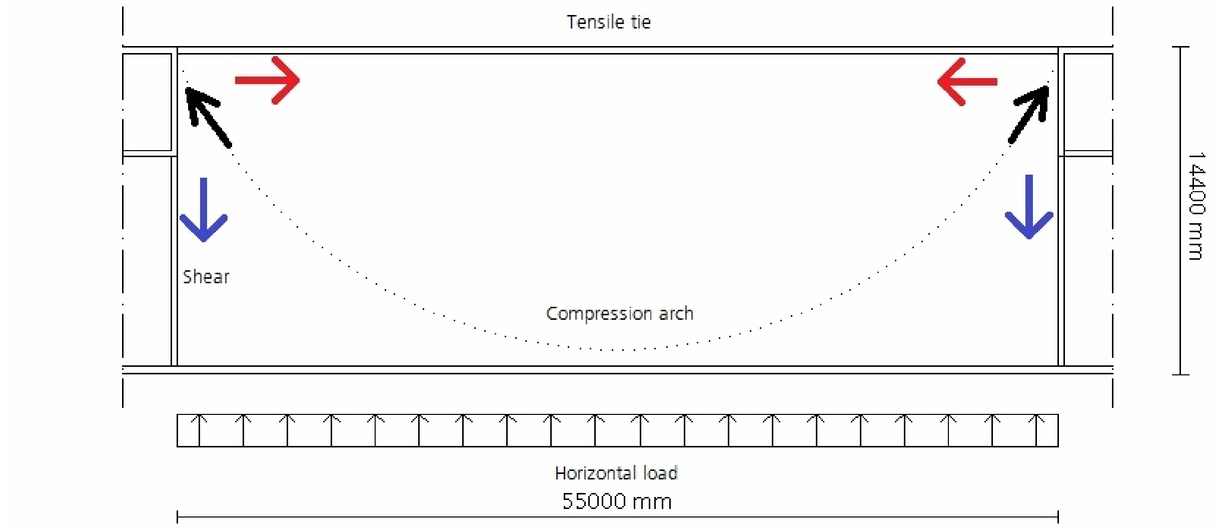



Figure 84: Distribution of loads over the floor diaphragm

Due to the large distance between two adjacent cores (about 50 m) there has to be accounted for a tensile connection at mid-span, due to the limited lengths in which these glulam beams are available.

Calculations (see Annex D.5) show that the dimensions of the edge beams should be at least 150 by 150 mm. For the end beams, the same appeared to be true: the dimensions should be at least 150 by 150 mm.

Since the dimensions of the end beams are smaller than those of the floor support beams, no adaptations are to be made on the design as proposed in the previous section, except for the implementation of the edge beams.

### Local diaphragm-action

As shown in figure 83, the chosen configuration implies joints in two directions: the longitudinal joints (along the length of the elements) and the end joints (at the ends of the elements). For diaphragm-action the elements should be able to transfer the acting shear forces from one element to another. According to the products' specification, two solutions are generally used: a connection with dowels and one by means of nailed OSB plates.

The following assumptions are accounted for in the design:

- Only the governing wind direction (the EW-direction) is considered.
- The connections are designed to resist the maximum acting shear load that acts on the whole plate.
- It is strived to design standardized connections to the utmost extent.
- Large diameter dowels (d = 30 mm) are considered due to the reduced risk of being deformed due to handling and storage operations on site.
- Smooth, non pre-drilled nails are used.
- The design is based on strength requirements only.

*Doweled connection*

- End joints

When the end joints are concerned, the Lignatur elements have to be connected to each other at their open ends. A commonly used solution, opted by the manufacturer, is the implementation of end beams at both ends of the elements, see figure 85. The end beams at both ends are subsequently connected to each other by means of dowels. The thickness of these end beams is taken 50 mm.

Calculations (see D.6.1.1) show that at least 3 dowels are to be incorporated per linear meter. This leads to a spacing of about 330 mm in between the dowels, which is acceptable. An example of the proposed connection is shown in figure 85.

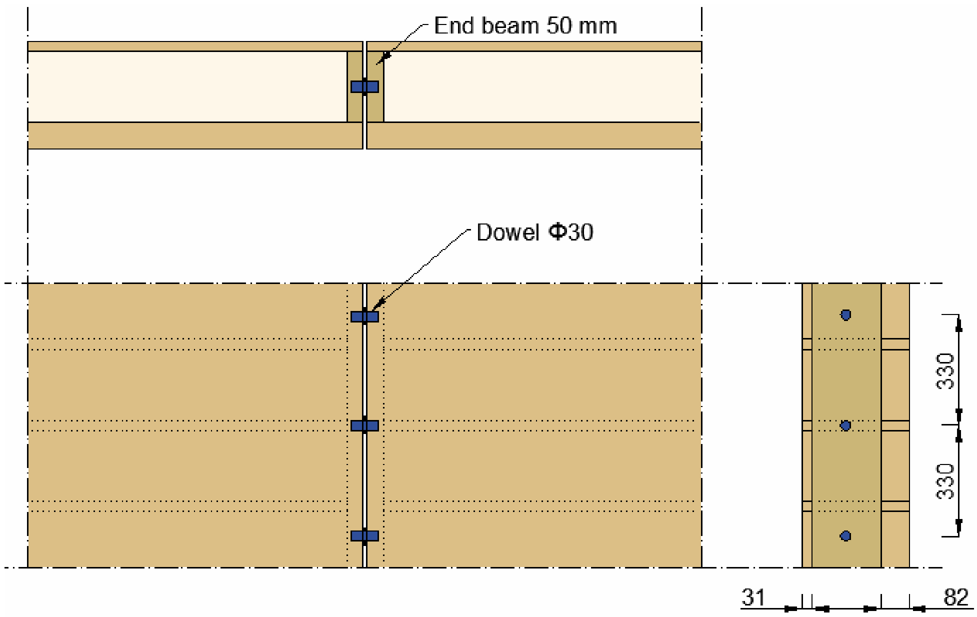



Figure 85: Proposed doweled connection at the end joints

§ Longitudinal joints

At the longitudinal joints, the connection has to be made between the outer ‘beams’ of the elements. These ‘beams’ have a thickness of only 31 mm, thereby resulting in a lower shear capacity than was found for the end beams.

Calculations (see D.6.1.1) show that at least 1.12 dowels have to be included each linear meter, which results in a spacing of about 890 mm.

Since it is desired to make use of identical elements, the spacing of the dowels is set to 800 mm. With an element length of 16 m, this leads to a total of 19 dowels per longitudinal edge, see figure 86.

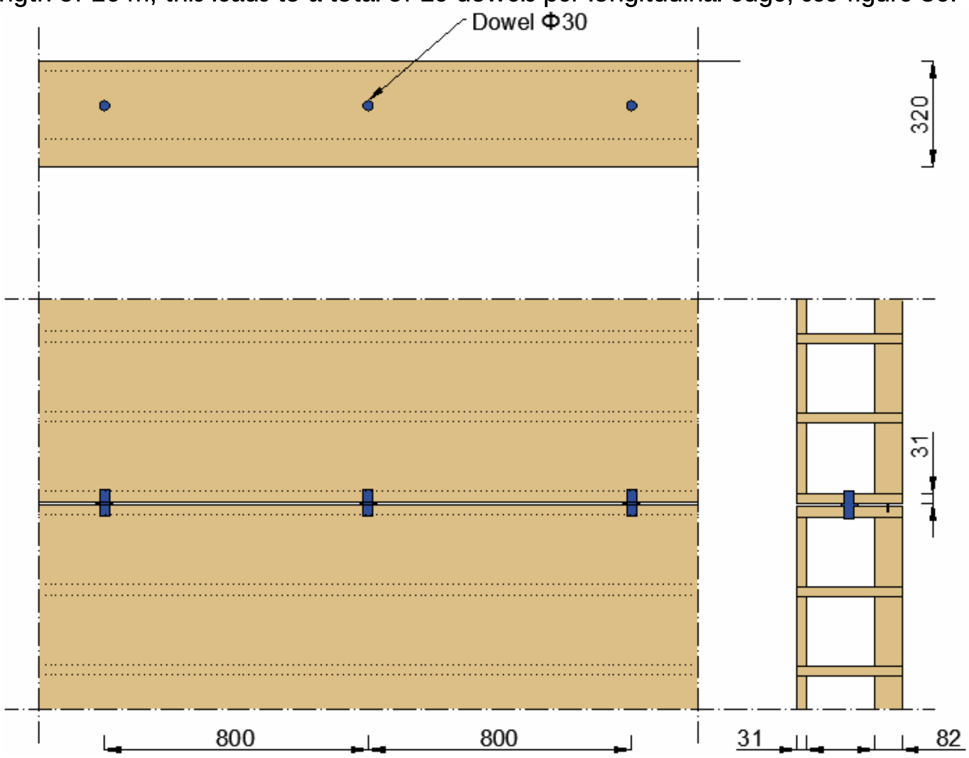



Figure 86: Proposed doweled connection for the longitudinal joints

*Nailed OSB layer*
When this solution is accounted for, one can decide to nail a large-sized layer over the whole diaphragm or to nail small strips over the joints only. Both options are commonly used in practice. In this thesis only the second option is taken into consideration, since the savings in material use are considered beneficial.

§ End joints
Also for this solution, end members having a width of 50 mm are added to the Lignatur element. Accounting for nails with a diameter of 5 mm and an OSB layer with a thickness of 20 mm, a calculation (see D.6.1.2) was carried out to determine the required amount of nails and their spacing.

It appeared that the amount of nails required becomes too large to comply with the requirements as stated in the building codes. The proposed solution is therefore not suitable to be used for the end joints.

§ Longitudinal joints
Since the ‘outer’ beams are only 31 mm wide, nails having a diameter of 3 mm are being used. With an OSB layer of 20 mm, this results in a total amount of 557 nails to be implemented over a length of 50 m. This leads to a spacing of about 80 mm, which satisfies the requirements, see figure 87.

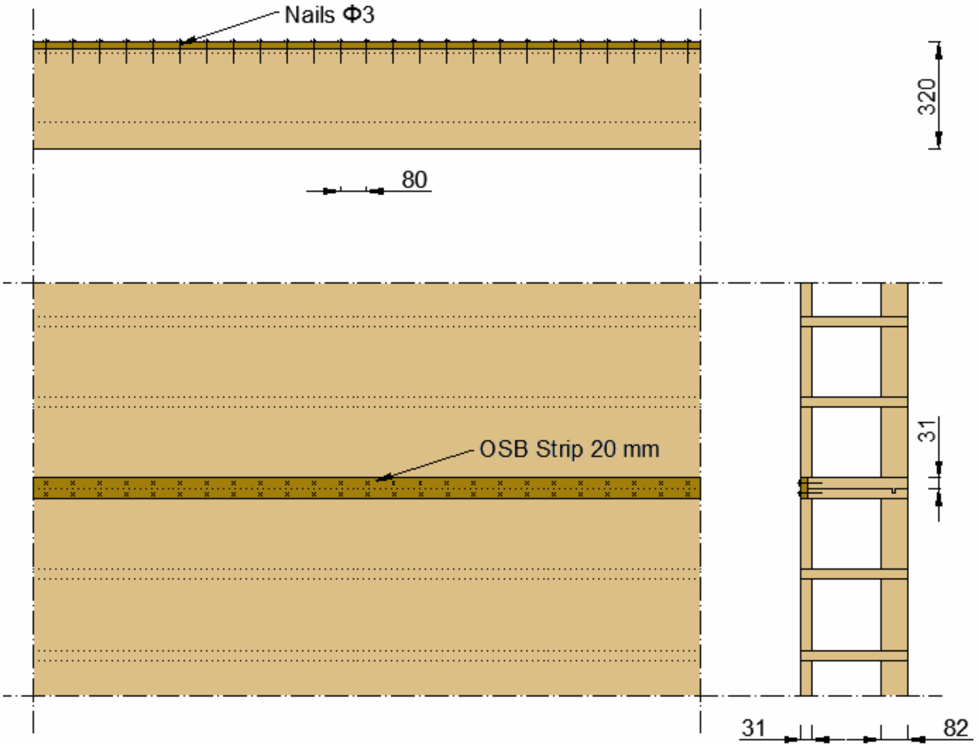


Figure 87: Proposed nailed connection for the longitudinal joints

*Conclusions and recommendations*
§ A doweled connection can be used to connect the elements in both directions.
§ The implementation of dowels in two directions leads to difficulties during the erection phase.
§ Small OSB strips nailed to the floor elements are only feasible in longitudinal direction.

§ It is recommended to use nailed OSB strips in longitudinal direction, and dowels for the end joints.
§ It is recommended to investigate alternative solutions (such as screws) for the final design

Stiffness of the diaphragm
Although it is usually not required, the stiffness of the diaphragm is checked in this section.

Since it is quite difficult to obtain an exact idea about the stiffness of the diaphragm, a highly simplified analysis is made. The diaphragm is considered a slender beam that is subjected to the combination of bending and shear. The total deformation of this combined system follows from the deflections due to shear, bending moment and the slip at the connections are summed up.

Calculations (see Annex D.7) show that the maximum deflection is in the order of 30 mm, while only 13 mm is allowed. Bending seems to be governing for the deflections.

An improvement could therefore be made by increasing the dimensions of the chords (end beams).

It is recommended that further investigations are carried out on this subject, before making use of the proposed solution.

## 5.4 Stacked bond pattern in NS-direction

A stacked bond pattern is a brickwork pattern whereby all joints are in the same line, as shown in figure 88. The longitudinal axis of the floor element points in the NS-direction. Also here, the element should be able to act as a stiff plate to provide diaphragm-action.

In between the 'supports' (the cores) of the floor plate, compression and tensile forces will develop, see figure 84. To be able to transfer these forces to the cores, a 'tensile tie' and a compression beam should be implemented in the design, see figure 88.

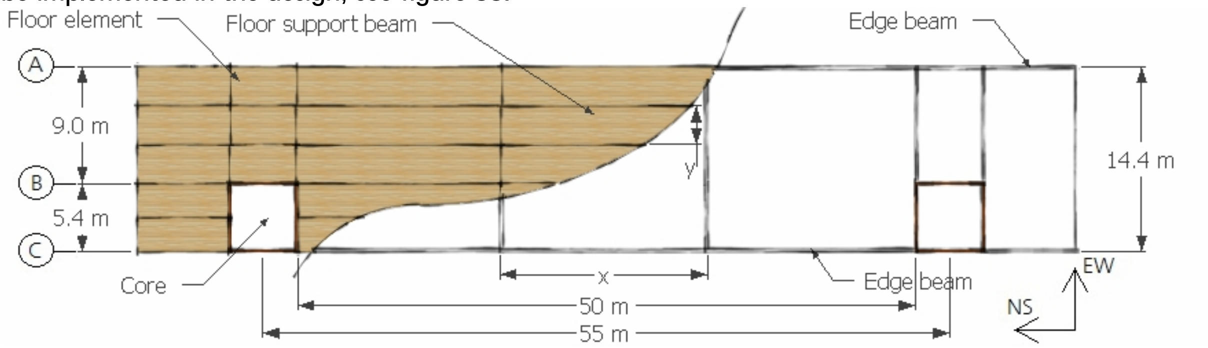



Figure 88: Support beams underneath the floor elements (indicative, not to scale)

### 5.4.1 Floor elements

Since all floor elements have to be supported at their ends, beams are implemented in the design at those locations where adjacent elements are laid against each other. Since all elements have their joints at the same location, the floor elements are all supported by two beams, see figure 89.

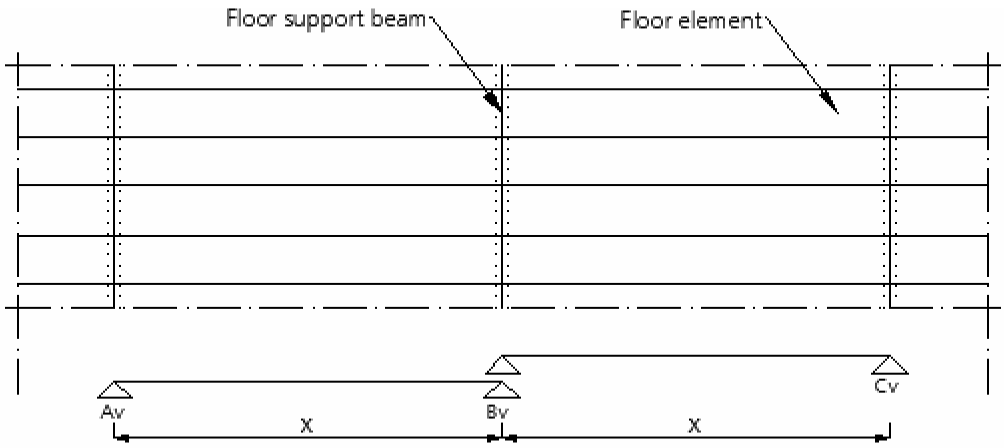



Figure 89: Floor elements supported by two beams

Calculations on strength, stiffness and dynamic behaviour are carried out, resulting in the height-to-span-width diagram as shown earlier in figure 76. Since vibrations are governing in the design, there is no difference between the diagrams of a beam on two supports and a beam over three support. For the complete set of calculations and their backgrounds, reference is made to D.2.2.

Considering figure 76, it appears that for office floors spans up to 12.9 m can be reached, which is considerably larger than the maximum span found for the stretcher bond pattern (up to 8.75 m due to the limited element length).

Before determining the most beneficial product, it is assumed that the minimum span should be 9.0 m, since otherwise it would be better to consider the stretcher bond configuration. This results in the following overview, see table 5.

Table 5: Product characteristics for various floor products

| Product | Element height [mm] | Weight [kN/m$^2$] | Maximum span width [m] |
|---|---|---|---|
| Kerto Ripa 6*69*600 | 600 | 0.64 | 12.90 |
| Kerto Ripa 6*51*500 | 500 | 0.44 | 11.20 |
| Lignatur 320 | 320 | 0.72 | 10.20 |
| Kerto Ripa 6*45*400 | 400 | 0.35 | 9.60 |
| Lignatur 280 | 280 | 0.69 | 9.40 |
| Bresta 260 | 260 | 1.09 | 9.00 |

Making use of the overview, the following conclusion is drawn:

§ Due to the relatively large element height, the Kerto Ripa products are considered unfavourable.

Since it is strived to use the same floor product at each floor, the structural behaviour of the remaining floor products has been checked under the load circumstances of a congregation area ($q = 5.0$ kN/m$^2$; $\Psi_0 = 0.25$; $\Psi_1 = 0.7$). This resulted in the overview of table 6.

Table 6: Maximum spans under different loading conditions

| Product | Office area $l_{max}$ [m] | Congregation area $l_{max}$ [m] | Maximum element length [m] |
|---|---|---|---|
| Lignatur 320 | 10.2 | 8.4 | 16 |
| Kerto Ripa 6*45*400 | 9.6 | 7.7 | 16 |
| Lignatur 280 | 9.4 | 7.7 | 16 |
| Bresta 260 | 9.0 | 7.6 | 17.5 |

It appears that, for the congregation area, the maximum span-width is more or less equal to the maximum span found for the stretcher bond pattern. This can be explained by the fact that the dynamic behaviour is governing and this behaviour is equal for beams on two and three supports.

Since it is more efficient to consider a floor element over three supports, than one over two supports, it is decided to opt for the stretcher bond pattern rather than the stacked bond pattern in NS-direction, when the ground floor is at stake.

For the office floors, the stacked bond pattern promises to be more beneficial, but is it highly questionable whether a shift in the centre to centre distance of the columns is desirable.

Since the proposed stacked bond pattern does not offer clear benefits over the stretcher bond pattern, no further attention is paid to this solution.

## 5.5 Stacked bond pattern in EW-direction

Similar to the solution as discussed in the previous section, a stacked bond pattern is investigated. The difference is found in the direction of the longitudinal axis, which is in EW-direction here. Main advantage is that the floor elements might be able to span from façade to façade at once.

In between the 'supports' (the cores) of the floor diaphragm, compression and tensile forces develop. To be able to transfer these forces to the cores, edge beams are implemented in the design, see figure 90.

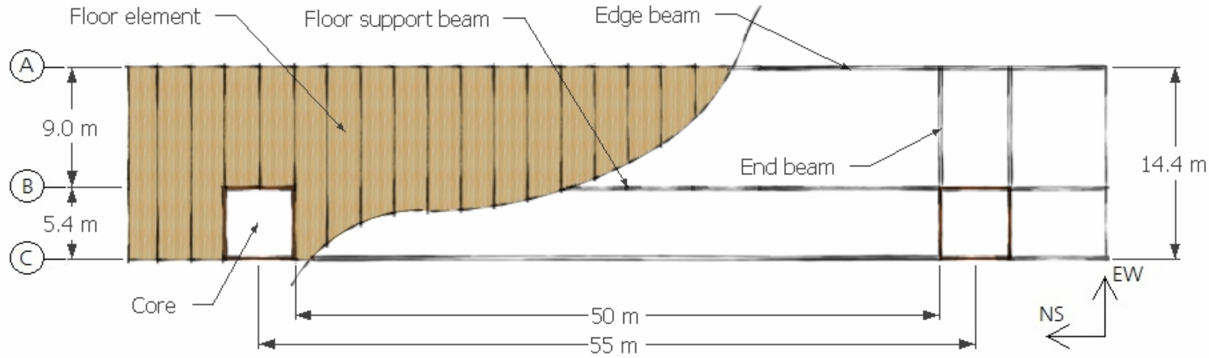



Figure 90: Support beams underneath the floor elements (indicative, not to scale)

### 5.5.1 Floor elements

Since a stacked bond pattern is at stake, the elements are to be supported at their ends only. For the 3rd, 2nd and 1st floor, this would imply that the beams span from façade to façade at once.

Unfortunately, from table 6, it is concluded that the maximum span-width for these office floors is about 10.2 m (Lignatur 320). When the functional plan of the Euroborg is considered, a span of 14.4 was incorporated here, which means that there is a loss of over 3.5 m when such an configuration is chosen. This is undesirable, and thus an additional support beam is implemented at axis B, see figure 91 and figure 92.

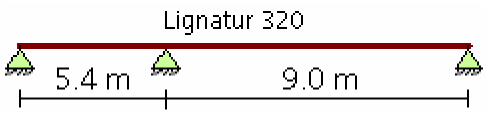



Figure 91: Lignatur 320 element spanning over 3 support beams

For the congregation area, the maximum span-width is 8.4 m (Lignatur 320), which implies that two intermediate beams (columns) are to be implemented here (no columns are allowed at mid-span due to the parking access lane). As for the stretcher bond pattern, it is decided to implement these beams (columns) at axis B and at axis A', see figure 92.

As a result, the floor elements span over four supports here. Calculations (see D.2.3) have shown that the element height may therefore be reduced to 200 mm.

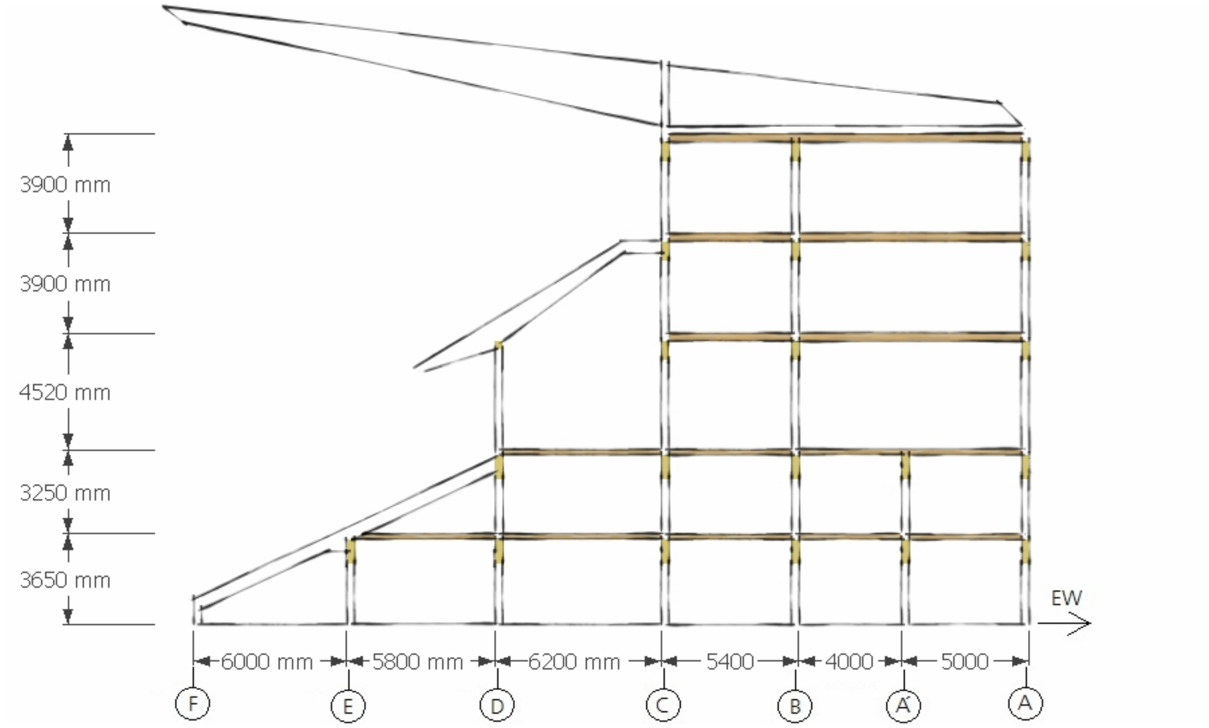



Figure 92: Support beams underneath the floor elements (indicative)

## 5.5.2 Support beams

### Ground floor

The required free height underneath the ground floor is 2.3 m, which may be locally reduced to 2.1 m. With a storey height of 3.25 m, this results in a maximum height of 1.15 m for the ground floor.

Having a floor height of 200 mm, this results into a maximum beam height of 950 mm. When a beam with dimensions 220 by 950 mm is considered, the maximum span-width of the governing beam becomes 9.65 m. Since this floor is governing in the design, it is stated that columns have to be implemented every 9.65 m, see figure 93.

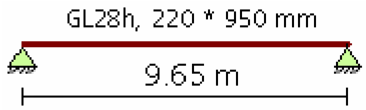



Figure 93: Ground floor support beams spanning 9.65 m at most

### Office

Since the floor elements are able to span 10.2 m at once, one additional support beam is implemented in the design. The best possible solution is found when this beam (and its support columns) is in line with the front core wall, thus at axis B. The floor elements are therefore supported by three beams.

The desirable free height underneath the governing (2$^{nd}$) office floor is 2.6 m. With a storey height of 3.90 m, this results in a maximum structural height of 1.30 m for the 2$^{nd}$ floor.

With a Lignatur 320 element, this leads to a maximum beam height of 980 mm. Considering a beam with a width of 220 mm, the maximum span-width is 11.9 m.

But again, it is desirable to make use of the same structural plan (grid) at all floors. Since the maximum span of the ground floor support beams is limited to 9.65 m, the same span is adopted here. This results in support beams with dimensions 220 by 750 mm, see figure 94.

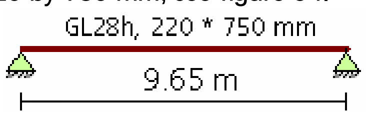



Figure 94: Office floor support beams spanning 9.65 m at most

Overview
Resuming all the above, the following structural configuration is found, see figure 95.

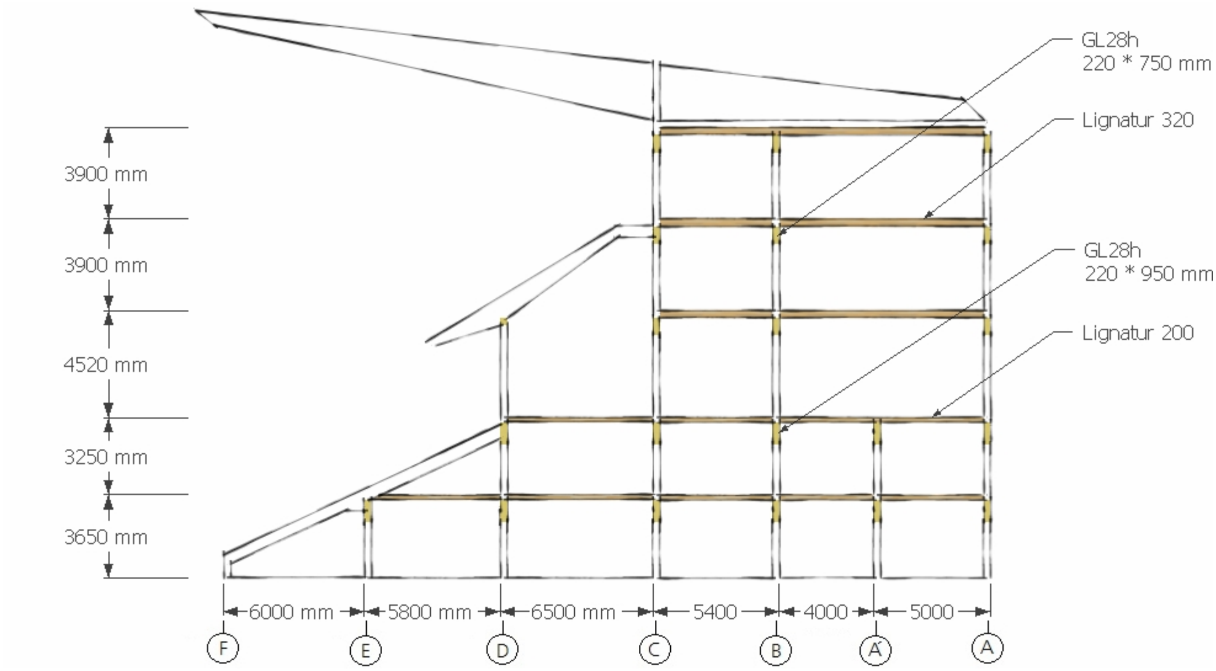



Figure 95: Proposed structural solution for the stacked bond pattern

The maximum span-width of the support beams can even be further increased, for example, by implementing multiple beams adjacent to each other (as shown in figure 80) or by accounting for materials having a higher stiffness. These possibilities are not accounted for in the proposed design.

## 5.5.3 Columns

In the preceding section, the dimensions of the support beams have been determined. These support beams are in turn supported by columns, which transfer the vertical loads to the foundation.

When the most heavily loaded column is considered, calculations (see D.4.2) have shown that the dimensions should be at least 350 by 350 $mm^2$, timber quality GL28h. Such dimensions are considered acceptable.

## 5.5.4 Diaphragm-action

Since the global diaphragm-action has already been discussed in detail in one of the previous sections, here, attention is given to the local diaphragm-action only. As stated earlier, use is made of the theory as presented [ 17 ] and [ 18 ].

An overview of the actual situation is provided in figure 96.

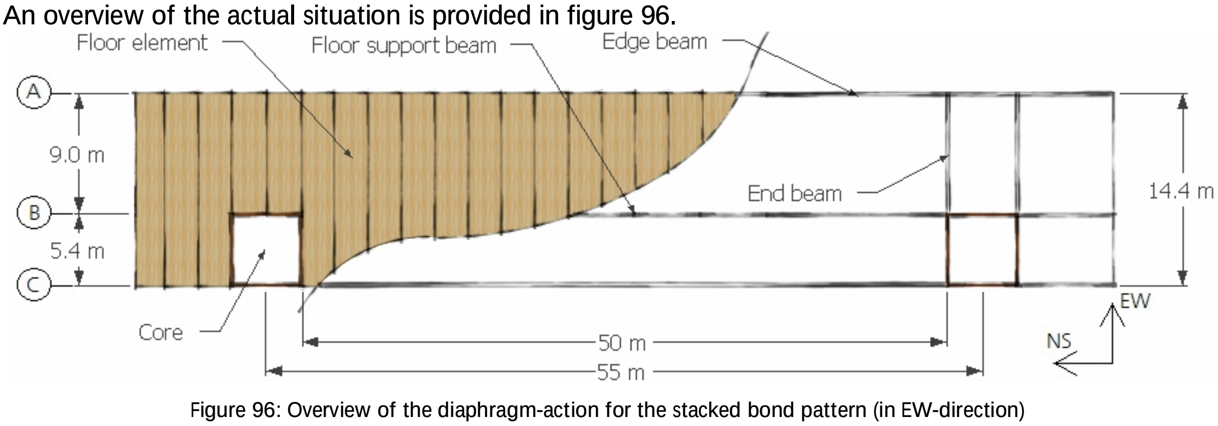



Figure 96: Overview of the diaphragm-action for the stacked bond pattern (in EW-direction)

### Local diaphragm-action

From figure 96, it appears that the joints are only present in longitudinal direction (along the length of the elements). For diaphragm-action the elements should be able to transfer the acting shear forces from one to another. According to the products' specification, two solutions are generally used: a connection with dowels and one by means of nailed OSB plates.

The following assumptions are accounted for in the design:

§ Only the governing wind direction (the EW-direction) is considered.
§ The connections are designed to resist the maximum acting shear load that acts on the whole plate.
§ It is strived to design standardized connections to the utmost extent.
§ No difference is made between the connections of individual joints
§ Large diameter dowels (d = 30 mm) are considered due to the reduced risk of being deformed due to handling and storage operations on site.
§ Smooth, non pre-drilled nails are used.
§ The design is based on strength requirements only.

*Doweled connection*

As mentioned earlier, for longitudinal joints, the connection has to be made between the outer 'beams' of the elements. These 'beams' have a thickness of only 31 mm.

Calculations (see D.6.2.1) show that at least 58 effective dowels are to be included over the span of 14.4 m. Since the dowels are placed in series, only 34 are effective. To obtain a total of 58 effective dowels, 2 rows of 58 dowels are implemented in the design. Due to the height of the outer 'beams', the choice for two rows of dowels does not lead to problems, see figure 97.

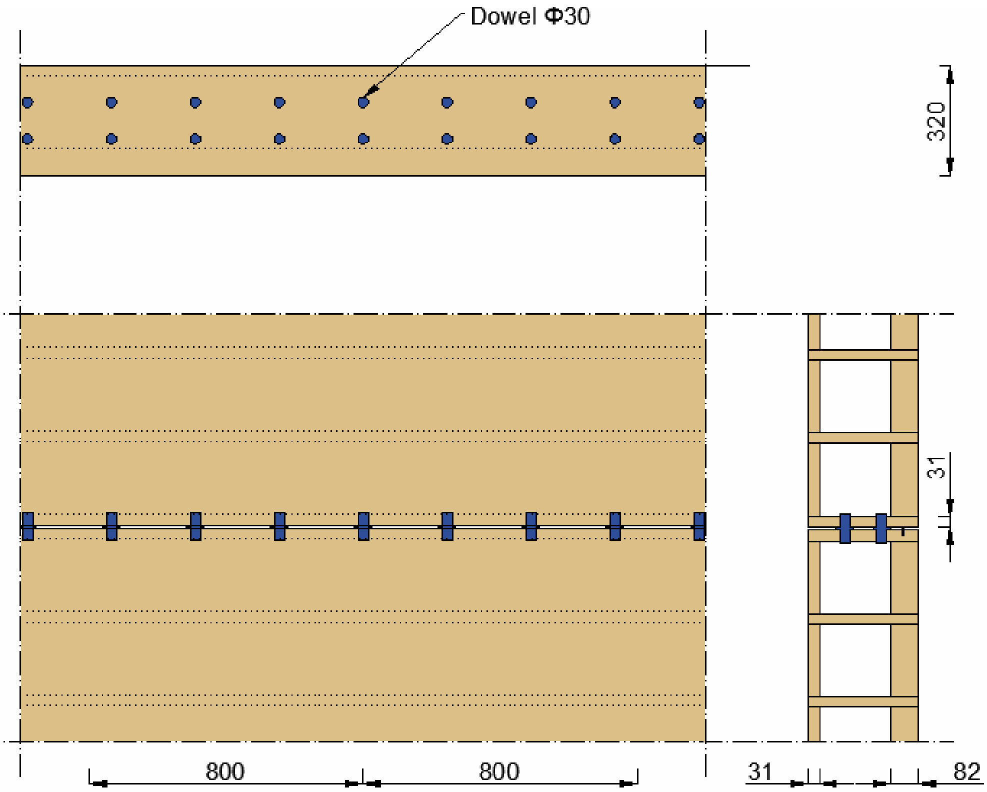



Figure 97: Proposed doweled connection for the longitudinal joints

*Nailed OSB layer*

When this solution is chosen, one can decide to nail a large-sized layer over the whole diaphragm or to nail small strips over the joints only. Both options are commonly used in practice. In this thesis only the second option is taken into consideration, since the savings in material use are considered beneficial.

Since the 'outer' beams are only 31 mm wide, nails having a diameter of 3 mm are being used. With an OSB layer of 20 mm, this results in a huge amount of nails to be implemented over a length of 14.4 m (see D.6.2.2). The spacing thereby becomes extremely small, which does not comply to the requirements as stated in the Building Codes.

Since it is not desirable to make huge adaptations on the standard floor elements, it is concluded that the nailed OSB solution is not suitable to be used for the proposed configuration.

*Conclusions and recommendations*

- § A doweled connection can be used to connect the elements.
- § Small OSB strips nailed to the floor elements do not offer a feasible solution.

- § It is recommended to consider dowels for all joints.
- § It is recommended to investigate alternative solutions (screws) for the final design.

Stiffness of the diaphragm

As for the stretcher bond pattern, the stiffness of the diaphragm is checked for this solution. Again, a highly simplified analysis is made. The diaphragm is considered a slender beam that is subjected to the combination of bending and shear. The total deformation of this combined system follows from the deflections due to shear, bending moment and the slip at the connections are summed up.

Calculations (see Annex D.7) show that the maximum deflection is in the order of 10 mm, which is smaller than the 13 mm that is allowed. It is therefore concluded that no problems are to be expected when the horizontal deformation of the floor slab is at stake.

Despite that, it is recommended that further investigations are carried out on this subject, and to consider the obtained results with some reservation.

## 5.6 Conclusions and proposal final design

In this section, the conclusions concerning the design of floors are summarized. After that, a comparison is made between the panel configurations as discussed earlier and the most beneficial solution is determined.

As stated in the introduction of the chapter on floors, the goal of this chapter was to find an answer to the following research question:

- § *What kind of structural configuration is most beneficial for floors within a stadium structure?*

Summarising the most striking results, categorised per configuration, an answer is provided to this research question.

### 5.6.1 General

- § The dynamic behaviour of the floor elements is (generally) governing for the design
- § When the support beams (GL28h) are concerned, strength requirements proved to be governing

### 5.6.2 Stretcher bond pattern

- § The available element length is the limiting factor at low loads.
- § Lignatur 320 elements with a span of 8.0 m prove to be the most beneficial solution.

- § For the ground floor, the support beams can span 7.0 m at most.
- § For the ground floor, 2 intermediate columns are implemented between axis A and C.

- § The lower floors are supported by GL28H beams with dimensions 220 by 640 mm, spanning 5.4 m.
- § The upper floors are supported by GL28H beams with dimensions 220 by 800 mm, spanning 9.0 m.
- § The governing columns have dimensions of 350 by 350 $mm^2$.
- § For the longitudinal joints, an OSB layer with a thickness of 20 mm is nailed to the elements by 557 smooth nails (d = 5mm).
- § For the end joints, end beams of at least 50 mm thickness are incorporated at both ends.
- § The connection at the ends joints is made with 3 dowels (d = 30 mm) per linear meter.

§ Horizontal deflections of the stiff plate exceed the maximum deflection (simplified analysis).

All the above is incorporated in figure 98, figure 99 and figure 100.

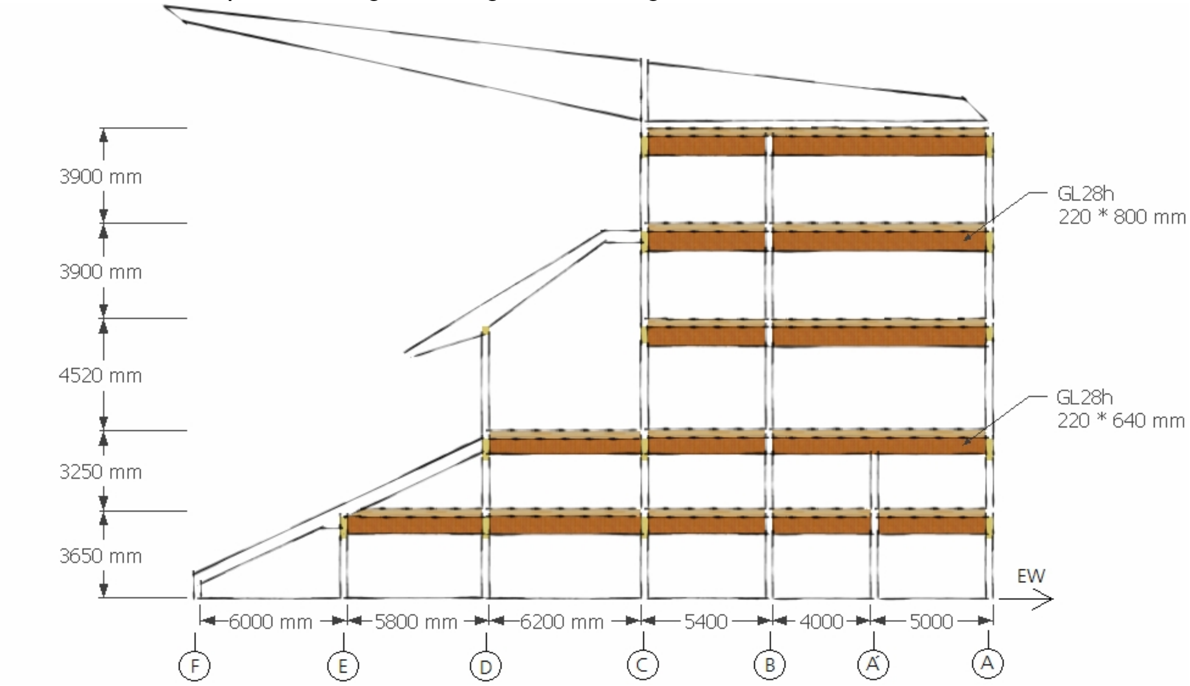



Figure 98: Section X-X' for the stretcher bond pattern

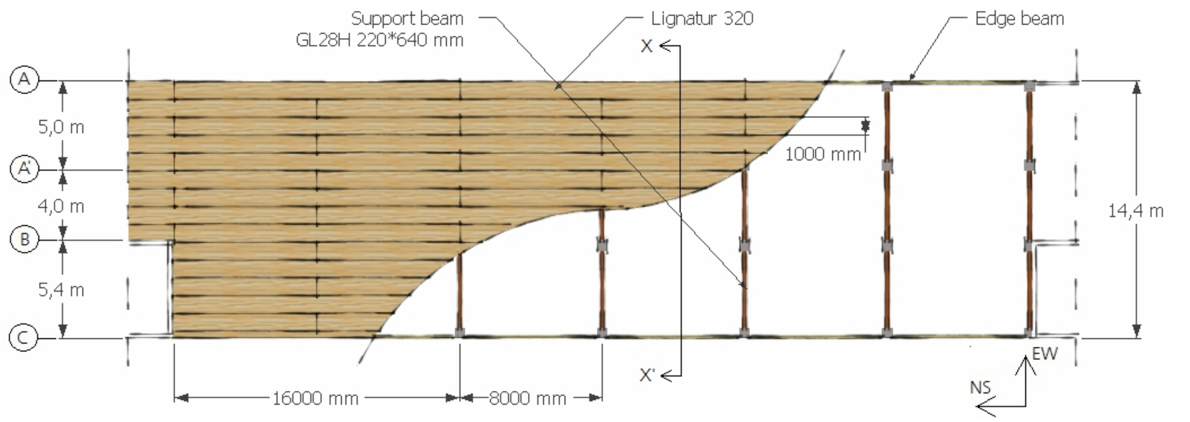



Figure 99: Typical plan of the ground floor

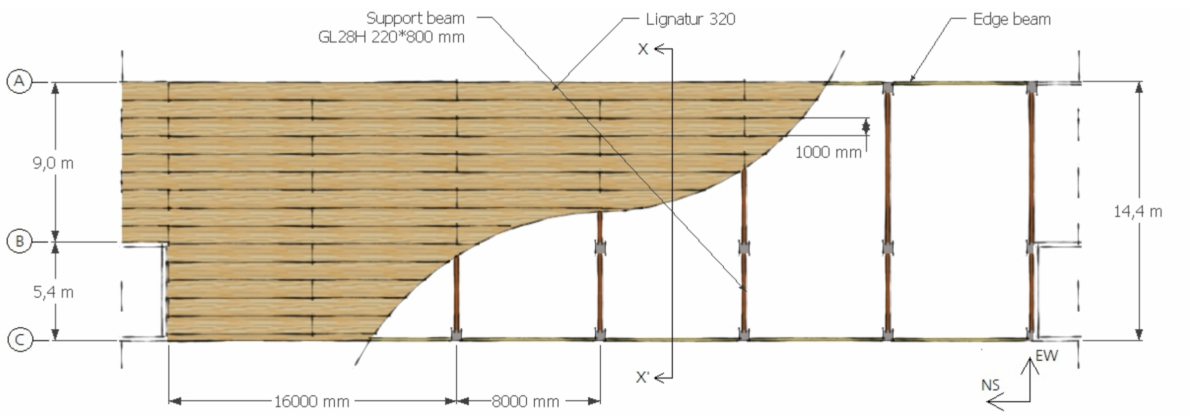



Figure 100: Typical plan of the office floors

### 5.6.3 Stacked bond pattern (in EW-direction)

- For the office floors, the Lignatur 320 elements span 14.4 m over 3 supports, at axis A, B and C.
- For the ground floor, the Lignatur 200 elements span 14.4 m over 4 supports, since an additional support is required at axis A'.

- The main support beams (axis B) of the lower floors have dimensions of 220 by 950 mm, and span 9.65 m at most.
- The main support beams of the upper floors (axis B) have dimensions of 220 by 750 mm, and span 9.65 m as well (larger dimensions and span possible)
- The governing columns have dimensions of at least 350 by 350 $mm^2$.

- For the longitudinal joints, 2 rows of 58 dowels are to be incorporated over a length of 14.4 m
- When the horizontal deflections of the diaphragm are concerned, a highly simplified analysis showed that these are limited and do not cause any problems.

Incorporating the conclusions from above in a design, figure 101, figure 102 and figure 103 are found:

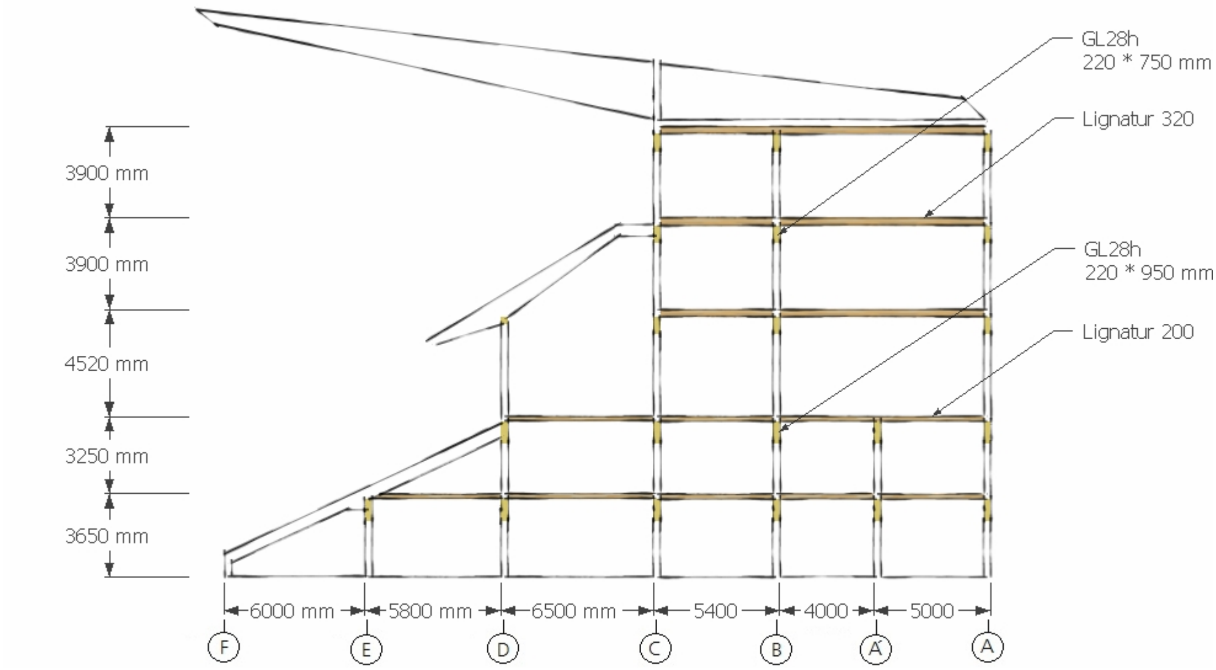



Figure 101: Section X-X' for the stacked bond pattern

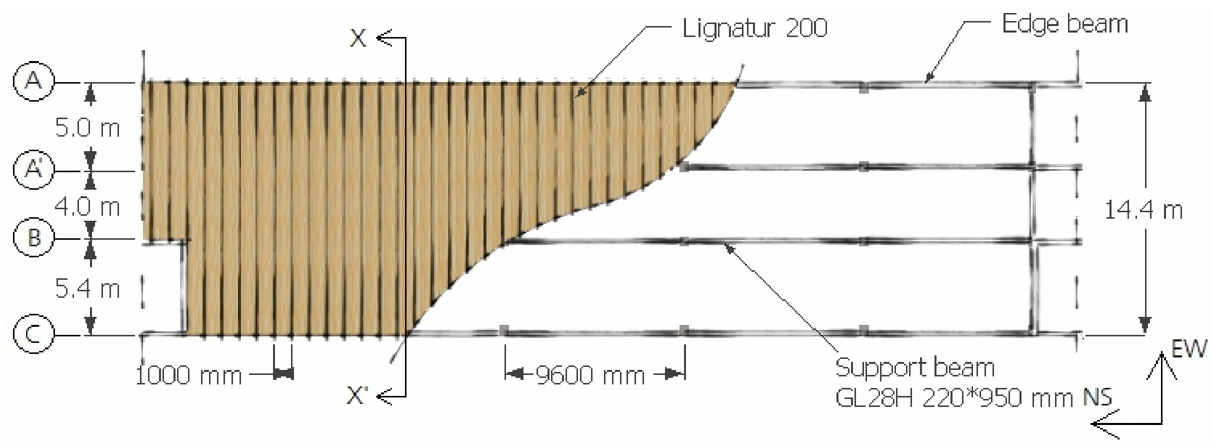



Figure 102: Typical plan of the ground floor

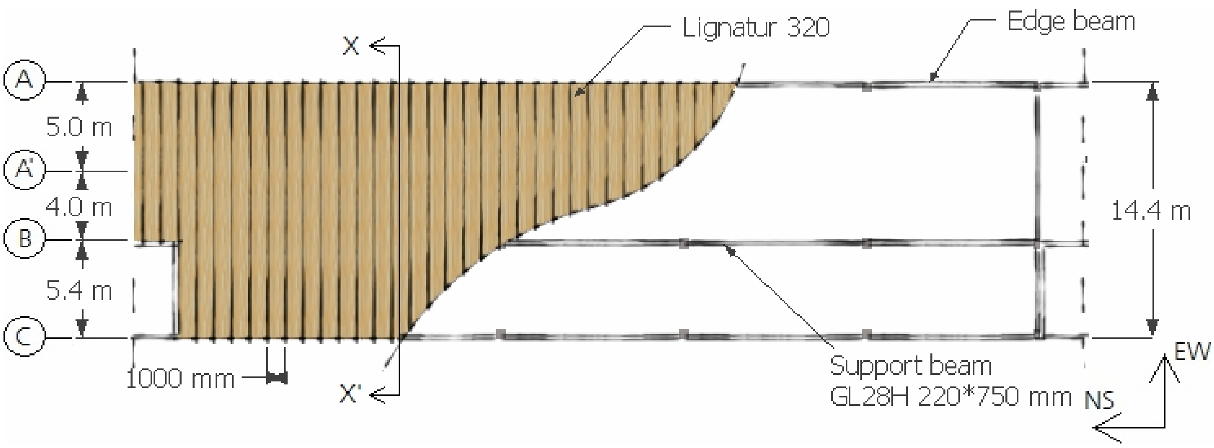



Figure 103: Typical plan of the office floors

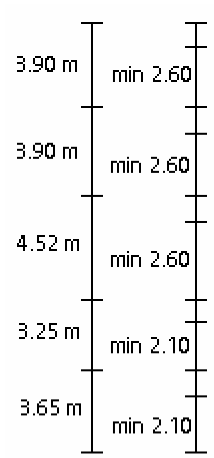
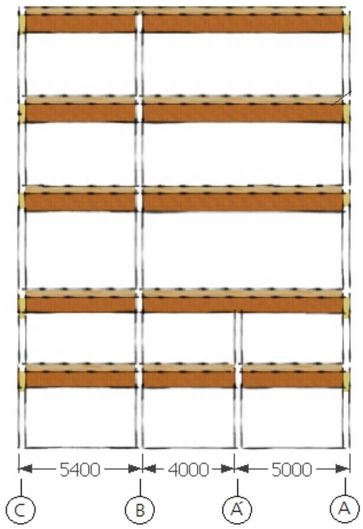
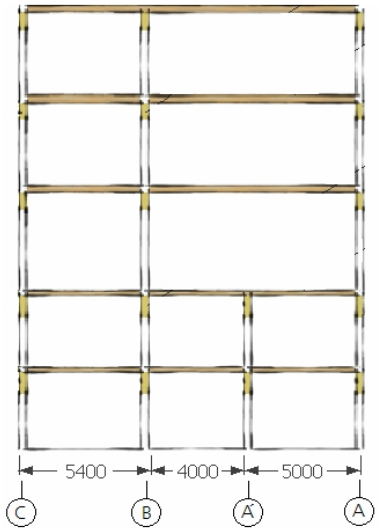
## 5.6.4 Comparison

All the above being known, the examined structural configurations are compared to each other. To be able to determine the most beneficial solution, a comparison is made on both structural and functional aspects. As a start, the structural consequences of each configuration are considered.

### Structural aspects

When both patterns are compared to each other, the following overview is obtained, see table 7.

Table 7: Comparison of different structural configurations

| | Stretcher bond | Stacked bond |
|---|---|---|
| Columns EW (c.t.c.) | | |
| - Lower floors | 5.4 – 4.0 – 5.0 [m] | 5.4 – 4.0 – 5.0 [m] |
| - Upper floors | 5.4 – 9.0 [m] | 5.4 – 9.0 [m] |
| Columns NS (c.t.c.) | Up to 8.0 [m] | Up to 9.65 [m] |
| Governing column | 350 by 350 mm$^2$ | 350 by 350 mm$^2$ |
| Support beams | | |
| - Lower floors | GL28h 220 * 640 (EW-direction) | GL28h 220 * 950[(1)] (NS-direction) |
| - Upper floors | GL28h 220 * 800 (EW-direction) | GL28h 220 * 750[(1)] (NS-direction) |
| Floor elements | | |
| - Lower floors | Lignatur 320 | Lignatur 200 |
| - Upper floors | Lignatur 320 | Lignatur 320 |
| Diaphragm-action | | |
| - Shear connectors | In two directions required | In one direction required |
| - Deflections | Exceeds limit to some extent | Within limits |
|  |  |  |







[(1)]When the governing (most heavily loaded) support beam is concerned

From the above, the following conclusions are drawn:

§ The stacked bond requires less columns (span in NS direction is 9.65 versus 8.0 for stretcher bond)

§ Considering the upper floors, the amount of material for the support beams is almost equal:
- Stretcher bond: 7 beams in between cores (c.t.c. 8 m) à $7 * 14.4 * 0.22 * 0.80 = 17.8m^3$
- Stacked bond: 1 heavy and 2 less (about 50%) loaded beams à $2 * 48 * 0.22 * 0.75 = 15.9m^3$

§ Considering the lower floors, the amount of material is less for the stretcher bond pattern:
- Stretcher bond: 7 beams in between cores (c.t.c. 8 m) à $7 * 14.4 * 0.22 * 0.64 = 14.2m^3$
- Stacked bond: 2 heavy and 2 less (about 50%) loaded beams à $3 * 48 * 0.22 * 0.95 = 30.1m^3$

§ At the lower floors, for the stacked bond, Lignatur 200 elements are used instead of Lignatur 320.

§ For the stacked bond pattern, shear connectors are only required in one direction and these comply with the requirements.

Functional aspects

§ One could argue that the appearance of several support beams of equal height at a regular c.t.c. distance (Stretcher bond) is more desirable than the implementation of a large support beam at mid span and smaller beams at the ends (Stacked bond). Dialogue with the projects' principal clarified however, that this should not be considered better or worse. As long as the required minimum free height is provided, and the support beams and columns are not located within the office area, both solutions are acceptable.

§ Within stadium structures the ducts usually run parallel to the stadium's perimeter. Consulting the projects' principal, it appeared that such a configuration is regarded highly beneficial. Therefore, it is undesirable to have large beams spanning perpendicular to this direction, i.e. spanning in the EW-direction. From this particular point of view, one could conclude that the stacked bond definitely offers benefits over the stretcher bond.

§ On the other hand, an acceptable solution for the stretcher bond could be reached by, for instance, implementing the ducts in between the cores, passing through (or underneath) the smaller-size support beams that span above the hallway. But since air conditioning and ventilation shafts are concerned (which implies large diameter pipes), the solution of ducts and shafts protruding through the support beams becomes rather difficult.

§ As mentioned earlier, the columns are placed at a larger c.t.c.-distance when the stacked bond pattern is concerned (9.65 m versus 8.0). This implies a reduced amount of columns over the width of the building.

§ From the point of view of erection, the stacked bond pattern definitely offers the most benefits: all elements span from façade to façade at once, requiring shear connectors in one direction only. This makes the erection process a lot faster and easier, in contradistinction to the situation where the elements are placed in a staggered pattern, having shear connectors in two directions.

### 5.6.5 Proposal

Considering the comparison in the previous section, it is stated that the stacked bond pattern promises to be the most beneficial solution. This configuration is therefore considered in the final design. For an overview on the structural build-up of this solution, reference is made to figure 101, figure 102 and figure 103. The centre-to-centre distance of the columns in NS-direction depends on the maximum span of the grandstand elements, which is determined in the upcoming chapter.

# 6 Grandstands

## 6.1 Introduction

When people are asked to describe a typical aspect of any random stadium, often the grandstands are named. These stands accommodate the seating, from which one can overlook the main event that is hosted within the stadium perimeter. As a consequence, from the spectators' point of view, the grandstand is regarded the most important aspect of the stadium structure.

From the perspective of engineering, the grandstands are an interesting part of the structure as well: with their crow-stepped shape, they put high demands on the overall stadium structure. Not even to mention their influence on the grid dimensions.

Goal of this chapter is to investigate the structural feasibility of timber grandstands. For the shape of these grandstands, reference is made to the Euroborg stadium, see chapter 2, which is used as a reference for the design. Although this stadium holds an upper and a lower grandstand, in this chapter, the focus is mainly on the (governing) lower grandstand. Accounting for the above, an answer is provided to the following research question:

§ *What spans can be obtained when timber grandstands are concerned?*

### 6.1.1 Approach

At first, there is elaborated on the most important boundary conditions. Consecutively, the shape of the grandstand, the restrictions on the dimensions and the support conditions are taken into consideration.

Accounting for these conditions, in section 6.3, a first design is proposed. This design is investigated by making use of finite element software, after which the results are evaluated.

Interpreting the results as obtained in section 6.3, several adaptations on the preliminary design are proposed. Considering these, it is investigated whether larger spans can be obtained.

This chapter is finished by an overview of the most important conclusions and recommendations.

### 6.1.2 General considerations

§ The grandstands are located under the cover of the stadium structure, but remain in an outside climate. Therefore, the elements are susceptible to weathering effects.

§ The grandstand elements are rather susceptible to damage, either due to the (heavy) utilization or due to intentional causes such as vandalism. To prevent problems, the grandstand elements are to be protected against these influences.

§ When the timber grandstand elements are concerned, it is assumed that these are prefabricated. As a consequence, the elements as a whole can be provided with a protective coating. With reference to the project Metropol Parasole [ 19 ], it is mentioned that a polyurethane coating can be applied to provide this protective layer. This waterproof, but vapor-permeable coating is applied as a layer of 2-3 mm thickness and can be provided with a coloured top coat. Since such physical aspects are beyond the scope of this thesis, there is not elaborated on any further.

## 6.2 Boundary conditions

### 6.2.1 Shape

In this thesis, the grandstands are considered to be composed of 2 type of elements: the *grandstand elements* and the *grandstand support beams*. The grandstand elements have a crow-stepped shape and are supported by the grandstand support beams, which are placed at a certain centre-to-centre distance.

§ The shape of the grandstand elements follows directly from the requirements on viewing angle (i.e. the C-factor, see chapter 1.4). Since these requirements are adopted in this thesis, the 'external shape' of these elements is inviolable, see figure 104.

§ For individual seating places, the NEN-EN 13200-1 recommends a minimum tread of 800 mm. The riser height depends on the required C-value, which was elaborated on in the chapter 2.3.

For the upper grandstand (10 tiers) of the Euroborg stadium, this results in a riser height of 540 mm (from now on referred to as type-3 elements). For the 8 lower tiers of the lower grandstand the riser height becomes 335 mm (type-1 elements), for the upper 7 tiers it becomes 400 mm (type-2), see figure 104.

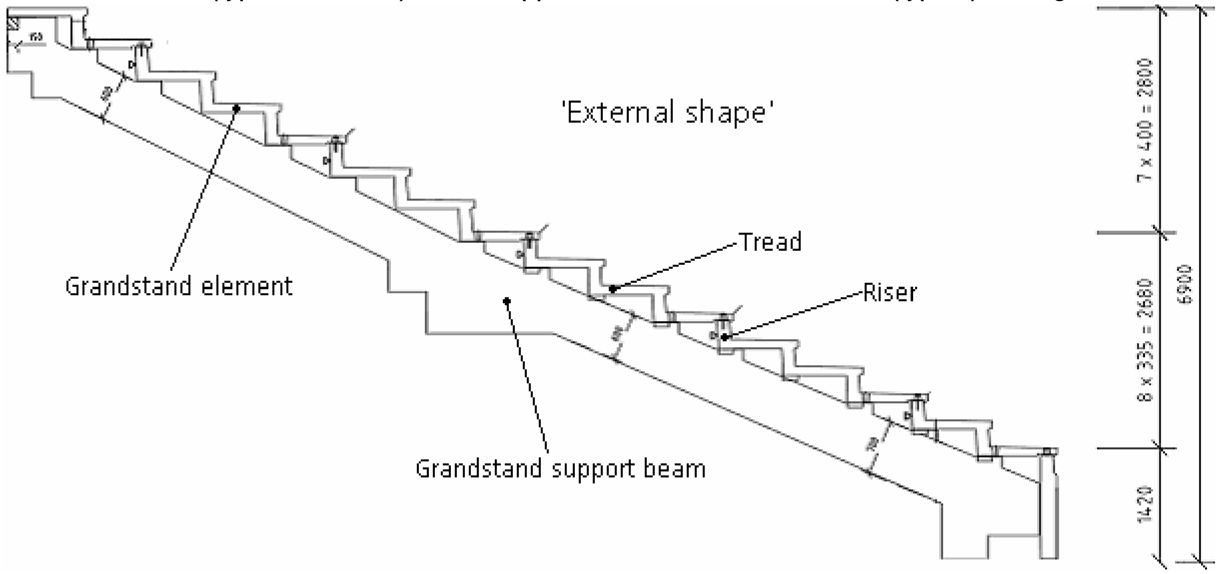



Figure 104: Shape of the lower grandstand

§ Safety regulations restrict the amount of seating places in one tier to a maximum of 28 seats, when served by a passageway on both ends of the row. With a required seating width of 500 mm per seat, this results in a maximum row width of 14 m.

§ It is assumed that the tiers are served by passageways at both ends, having a width of at least 1.2 m each. Although, this limits the row width to 14 m, the grandstand elements may exceed this value: as long as there is accounted for the passageways, the grandstand elements may have longer lengths.

### 6.2.2 Dimensions

From a practical point of view it is considered beneficial to make use of elements which are as large as possible: the larger the element, the smaller the amount of manoeuvres on site and the higher the speed of erection. Despite that, there are certain factors which limit the dimensions to which the *elements* can be designed.

The possibility of adapting the dimensions of the *support beams* is neglected here, since their shape follows directly from the shape of the grandstand *grandstand elements* (and is thus unviable).

It is therefore stated that the focus in this section is on the limits on the boundaries concerning the dimensions of the grandstand elements.

### Transport restrictions

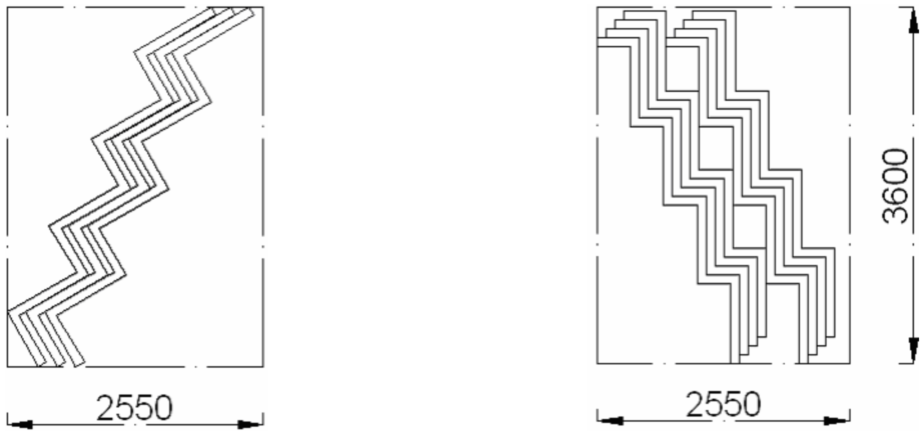
It is assumed that the transport of the grandstand elements is carried out by road, whereby the maximum dimensions are restricted by governmental bodies. With reference to the Annex F, it is stated that the maximum dimensions of the transport vehicles is limited to the following values:

§ without approval: 2.55 m * 4.0 m * 18.75 m (b*h*l), maximum weight of 40 tons
§ with approval: 3.5 m * 4.2 m * 25.25 m (b*h*l), maximum weight of 60 tons

Since the elements have to be loaded on a trailer (minimum trailer height 0.40 m), the effective height of the elements may be 3.6 m (without approval) or 3.8 m (with approval). Due to the presence of the truck, the maximum element length is smaller than the values as shown above. Since there is no well-defined information available on the maximum length of the load, the maximum element length is limited to be 16.50 m (without approval) or 23.00 m (with approval).

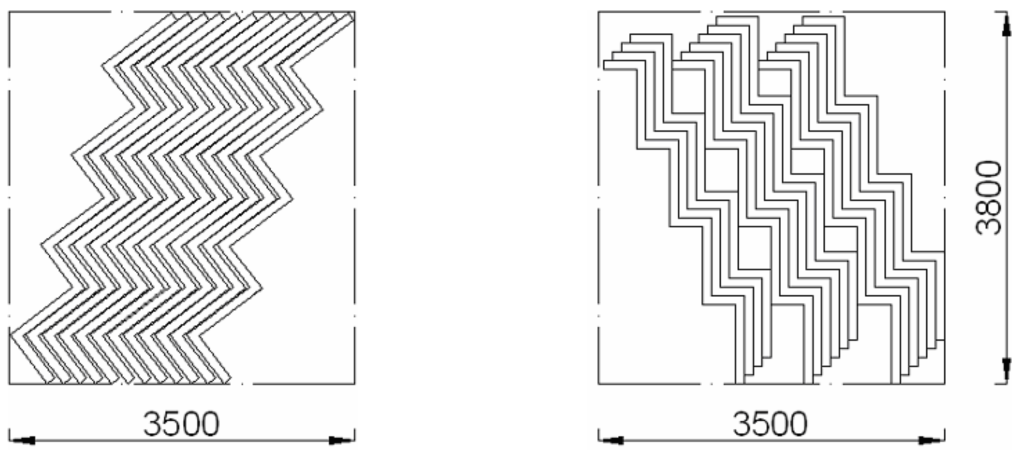
Note: when approval is required, additional restrictions are made to the transport and the costs will rise. On the other hand, the elements can have larger dimensions and the amount of trucks and hoisting operations might be reduced. It goes beyond the scope of this thesis to determine the most beneficial situation.

Considering the different types of grandstand elements, it is found that the maximum required height of the riser is 540 mm for type-3 elements (upper grandstand). The width of the treads is standardized to 800 mm. Since it is beneficial to transport as many elements as possible on one truck, it is investigated how many elements of each type can be transported under certain circumstances, see figure 105 and figure 106. To obtain a first idea, the is accounted for elements consisting of 4 tiers (4 treads and 4 risers).




Type-3 elements: 3 elements each truck    Type-1 elements: 8 elements each truck

Figure 105: Arrangement of elements during transport (without approval)




Type-3 elements: 12 elements each truck    Type-1 elements: 16 elements each truck

Figure 106: Arrangement of elements during transport (with approval)

Due to the limited width of the trucks, it appears to be beneficial to transport the elements while standing vertically to the utmost extent. The elements should then obviously be supported by some kind of frame during transport.

From figure 105 and figure 106, it is concluded that:
- It is not very efficient to transport '4-tier elements' without approval (figure 105), especially when type-3 elements are concerned.
- The transport of '4-tier elements' is achieved in a considerably efficient manner when transport with approval is concerned (figure 106). Despite that, the amount of elements that can be transported simultaneously is somewhat larger for '3-tier elements'.
- The implementation of '5-tier elements' is not considered beneficial at all.

Since, from a structural point of view, it is beneficial to design elements which are as large as possible, it is decided to account for '4-tier elements' in this thesis, instead of '3-tier elements'.

Accounting for approval, the collective weight of the elements on the truck is limited to 60 tons. With a characteristic density of 510 kg/m$^3$, the maximum length of the elements is determined, see table 8 and Annex E.1.

Table 8: Maximum element length due to weight restrictions

| Element type | Elements / truck [-] | Max. element weight [tons] | Cross-sectional area [m$^2$] | Maximum length [m] |
|---|---|---|---|---|
| Type 1 (riser = 335 mm) | 16 | 3.75 | 0.48 | 15.3 m |
| Type 3 (riser = 540 mm) | 12 | 5 | 0.55 | 17.8 m |

With reference to the allowed element length with approval (< 23 m), it is concluded that a large amount of elements per truck limits the maximum length of the elements. It goes beyond the goal of this thesis to determine the optimal ratio between element length and amount of elements.

### Hoisting restrictions
When the hoisting process of the elements is concerned, there are no limitations on the dimensions of the elements. Despite that, the weight of the elements does influence the required crane capacity. Since the weight of the elements is considerably small, no problems are to be expected.

### Practical considerations
Next to the structural behaviour of the timber grandstand elements, the physical aspects of the material is at stake as well:

- Using large elements, the amount of joints is smaller than for smaller-size elements. These joints can be considered as 'weak' spots in the structure, since these are susceptible to the infiltration of moisture and dirt. Therefore, special attention should be paid to these joints which introduces additional costs. A reduction of joints (and thus larger elements) is therefore considered beneficial.

### Conclusions
- From both a practical and a structural point of view, it is beneficial to make use of elements which are as large as possible.

- When transport with approval is concerned, the grandstand element shall consist of four tiers (i.e. 4 risers and 4 treads) at most.

- When transport without approval is concerned, it is advised to account for elements consisting of 3 tiers only, especially for the type-3 elements (riser height of 540 mm).

In the upcoming grandstand design the focus is on grandstand elements consisting of 4 tiers.

### 6.2.3 Support conditions

The grandstand elements are supported by the grandstand support beams, which are placed at a certain centre-to-centre distance. In this section, there is elaborated on the support conditions of the grandstand elements, where the focus is on the vertical support conditions only.

For timber grandstand, two kind of support conditions are interesting, see figure 107 and figure 108.

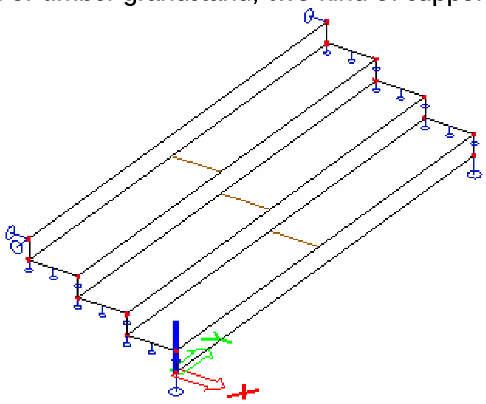


Figure 107: All edges supported

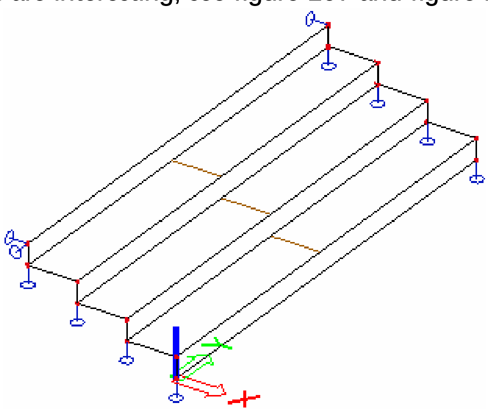


Figure 108: The edges of all risers are supported

For the situation of figure 107, all edges of the grandstand element are supported by the grandstand support beam. When figure 108 applies, only the risers are supported.

For the support conditions as presented in figure 107, the support beams should follow the shape of the grandstand elements exactly. As a consequence, the upper side of the support beam should have a crow-stepped shape, which requires a lot of additional material.

Since the risers are the main load-bearing elements it is regarded sufficient to support these only. It is therefore stated that the most beneficial situation is reached by supporting the risers of the grandstand elements only.

Conclusion

§ All risers of the grandstands are supported at their ends by the grandstand support beams

This boundary condition is therefore adopted in the preliminary design of the grandstand elements.

## 6.3 Preliminary design

As elaborated on in the previous section, the main goal of this chapter is to determine the maximum span-width of the grandstand elements.

At first, there is elaborated on the schematization used, thereafter the choice of material and the design assumptions are elaborated on.

### 6.3.1 Schematization

The grandstand element is schematized as a beam over two supports. The treads and the risers are assumed to be connected by means of fasteners, which implies a hinged connection, see figure 109. This connection is not considered any further in this thesis.

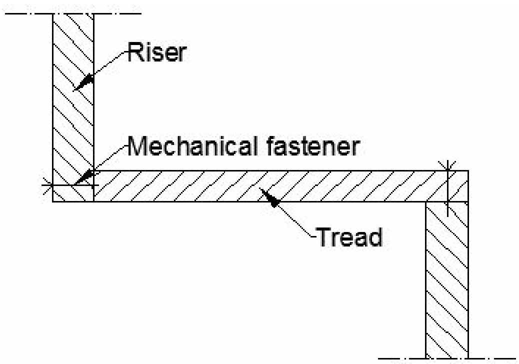


Figure 109: Proposed connection between the treads and risers

The governing load combinations are presented in E.1.2.

## 6.3.2 Material

As a start assumption for the preliminary design the LVL product Kerto, see A.2.3.3., is used. The risers are loaded in edgewise bending, parallel to the grain. As a result, no stiffness perpendicular to the grain is required. It is therefore decided to compose the risers from Kerto-S, having a thickness of 90 mm (being the maximum plate thickness for an individual Kerto-S plate).

The treads are subjected to flatwise bending in both longitudinal direction (parallel to the grain) and transverse direction (perpendicular to the grain). For the latter, stiffness perpendicular to the grain is required. Therefore, use is made of Kerto-Q, having a thickness of 69 mm (being the maximum plate thickness for an individual Kerto-Q plate).

Accounting for these products, the following values are to be used:

Kerto-S:
$E_{m;0;edge} = 13800$ N/mm$^2$

Kerto-Q:
$E_{m;0;flat} = 10500$ N/mm$^2$ $E_{m;90;flat} = 2000$ N/mm$^2$

## 6.3.3 Design assumptions

Since the goal of this thesis is to investigate the feasibility of a timber stadium, a simplified analysis is performed. In this section the most important assumptions are mentioned.

§ *Only the lower grandstand are investigated*
When the reference stadium is considered, two grandstands are distinguished: the upper and the lower grandstand. As mentioned earlier, the grandstand elements of the lower grandstand have a smaller riser height than those of the upper grandstands. As a result, the lower grandstand elements are governing in the design.

§ *The connection between the risers and the treads is assumed to be infinitesimally stiff*
When timber members are connected to each other, the stiffness of the combined element is dependent on the layout of the connection (e.g. amount of fasteners). This stiffness of the connection is generally given by a factor $\gamma$. When making use of mechanical fasteners, it is usually not possible to obtain a value of 1.0. When a glued connection is used, such a factor might be obtained. In the upcoming part of the design, it is assumed that all connections are infinitesimally stiff.

§ *The natural frequency of the grandstand should exceed 5 Hz*
Currently, there is no clear guideline on the behaviour of grandstands under human-induced vibrations, (see B.2.7.2). A minimum value that is regularly used in practice, is 5 Hz. Additional research is recommended to determine whether such a value is acceptable for timber grandstands as well.

§ *The natural frequency is determined making use of a simplified analysis*

Since the goal is to obtain a first idea on the maximum spans only, it is acceptable to make use of a simplified design procedure. When detailed design stages are at stake, it is advised to make use of detailed design procedures.

§ *Main purpose is to maximize the span that can be reached by a grandstand element*
The design of the grandstand element is based on the maximum span that can be reached under the acting loads and combinations. The element is therefore close to its limits (i.e. U.C.≈ 1). No attention is paid to the exact detailing of the proposed solution.

§ *The upper and lower part of the lower grandstand are supported by independent support beams*
When the Euroborg stadium is considered, see figure 104, it is found that the grandstand support beams span over three supports. Due to the difference in slope of the lower and the upper part, the beam is bend above the middle support. For the upcoming design, it is assumed that the upper and lower grandstand elements are supported by independent support beams, which are supported at one end by the middle support. The support beams are therefore divided in two.

The limitations of the proposed model being known, the design is elaborated on. As a start, the sectional properties are determined.

### 6.3.4 Sectional properties

The grandstand element is schematised as shown in figure 110.

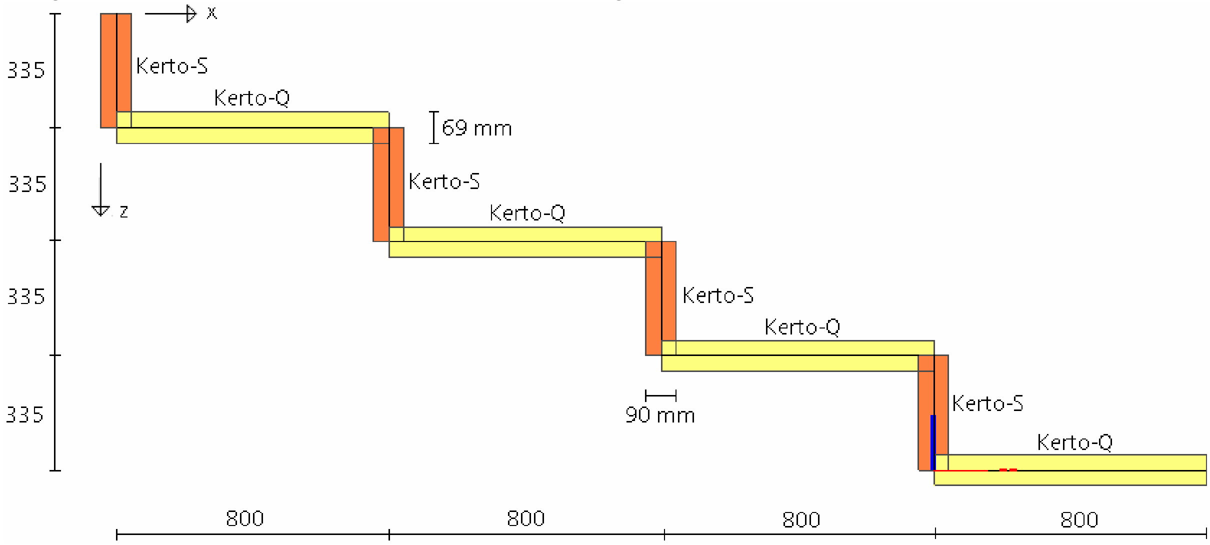



Figure 110: Cross-sectional shape of a grandstand element

It is assumed here that the grandstand element consists of 4 steps (tiers). To determine the sectional properties, the centre of gravity of the element is determined.

$$z_c = \frac{\sum EA_i * z_i}{\sum EA_i} = 767.5 \text{ mm and } x_c = \frac{\sum EA_i * x_i}{\sum EA_i} = 1477.8 \text{ mm}$$

Accounting for these values, the stiffness is determined for the main directions:

$$EI_{nn} = \sum EI_{nn;i} + \sum EA_i * n_c * n_c \qquad \text{in which n can be both x and z.}$$

Making use of this formula and the earlier mentioned centre of gravity, this results in:

$$EI_{zz} = 6.024 \cdot 10^{14} Nmm^2$$
$$EI_{xx} = 34.659 \cdot 10^{14} Nmm^2$$
$$EI_{xz} = -13.991 \cdot 10^{14} Nmm^2$$

## 6.3.5 Structural Behaviour

Since the grandstand structure has an asymmetrical shape, the structural behaviour can not be determined while making use of the 'vergeet-me-nietjes'. Therefore, use is made of the "Momentenvlakstellingen", while accounting for the values of $\kappa_x$ and $\kappa_z$ as determined by means of the following formula:

$$\begin{bmatrix} \kappa_x \\ \kappa_z \end{bmatrix} = \frac{1}{EI_{xx}EI_{zz} - (EI_{xz})^2} \begin{bmatrix} EI_{zz} & -EI_{xz} \\ -EI_{zx} & EI_{xx} \end{bmatrix} \begin{bmatrix} M_x \\ M_z \end{bmatrix}$$

The grandstand elements should comply with the requirements on strength, deflections and dynamic behaviour. Since these limit states imply different load combinations, the values of $\kappa_x$ and $\kappa_y$ are determined for each of these combinations.

The governing load combinations follow from E.1.2. and are presented here. $q_{self\text{-}weight}$ follows directly from the proposed cross-section of the element and amounts:

$$q_{self-weight} = (4 * 69 * 800 + 4 * 90 * 335) * 5.1 = 1.74 kN/m$$

As a first assumption the span-width of the grandstand element is taken 5.5 m. Accounting for this value, the structural behaviour of the proposed elements is investigated.

§ Strength

Having an element width of 3.2 m, the following combination is governing:

ULS 2a: 1.32 * (1.74 + 0.20 * 3.2)$(\downarrow)$ + 1.65 * 5.0 * 3.2$(\downarrow)$ + 1.65 * 0.25 * 0.5 * 3.2$(\leftarrow)$

This implies that:

$q_{x;d}$ = 0.66 kN/m and $M_{x;d} = 1/8 * 0.66 * 5.5^2 = 2.50$ kNm

$q_{z;d}$ = 29.54 kN/m and $M_{z;d} = 1/8 * 29.54 * 5.5^2 = 111.70$ kNm

Implementing the obtained values in the formula above, the following is found:

$$\begin{bmatrix} \kappa_x \\ \kappa_z \end{bmatrix} = \frac{1}{34.659 \cdot 10^{14} * 6.024 \cdot 10^{14} - \left(-13.991 \cdot 10^{15}\right)^2} \begin{bmatrix} 6.024 \cdot 10^{14} & 13.991 \cdot 10^{15} \\ 13.991 \cdot 10^{15} & 34.659 \cdot 10^{14} \end{bmatrix} * \begin{bmatrix} 2.50 \cdot 10^6 \\ 111.70 \cdot 10^6 \end{bmatrix} = \begin{bmatrix} 1.212 \cdot 10^{-6} \\ 2.999 \cdot 10^{-6} \end{bmatrix}$$

Accounting for these values for the curvature, the acting stresses are determined at any point of the structure by making use of the following formulae:

$$\sigma(x,z) = E\kappa_x * x - E\kappa_z * z = E * 1.212 \cdot 10^{-6} * x - E * 2.999 \cdot 10^{-6} * z$$

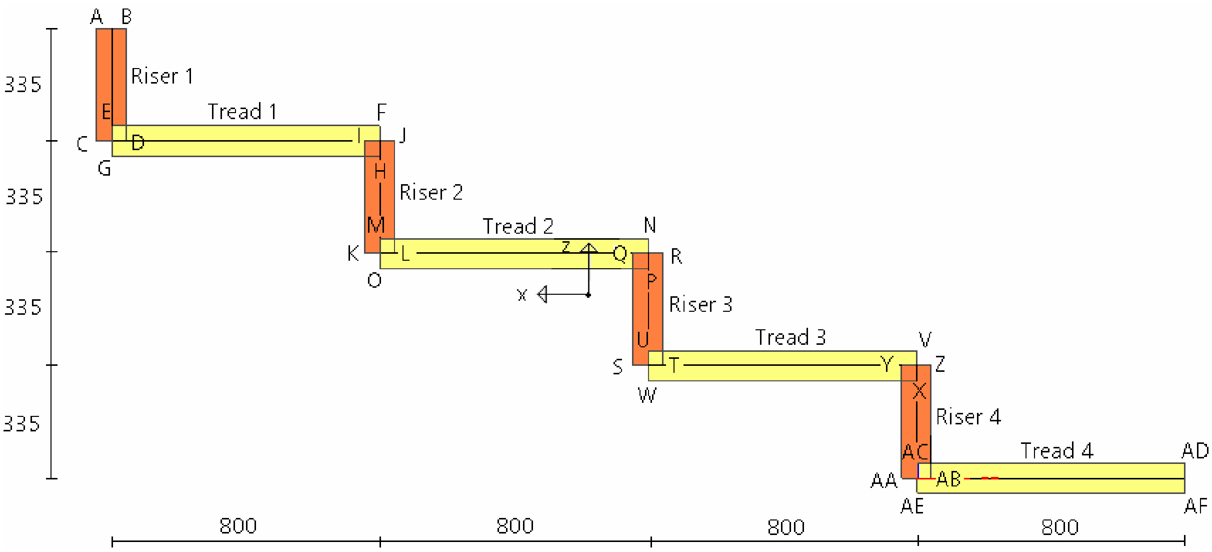



Figure 111: Locations of the observed stresses

The stresses that act on these elements are shown in table 9. The coordinates are determined from figure 110.

Table 9: Stresses acting at the grandstand element

| Element | Point | x [mm] | z [mm] | E [N/mm$^2$] | $\sigma_m$ [N/mm$^2$] |
|---|---|---|---|---|---|
| Riser 1 | A | 1478.8 | 767.5 | 13800 | -7.06 |
| | B | 1388.8 | 767.5 | 13800 | -8.57 |
| | C | 1478.8 | 432.5 | 13800 | 6.81 |
| | D | 1388.8 | 432.5 | 13800 | 5.30 |
| Tread 1 | E | 1433.8 | 467 | 10500 | 3.52 |
| | F | 633.8 | 467 | 10500 | -6.66 |
| | G | 1433.8 | 398 | 10500 | 5.69 |
| | H | 633.8 | 398 | 10500 | -4.48 |
| Riser 2 | I | 678.8 | 432.5 | 13800 | -6.57 |
| | J | 588.8 | 432.5 | 13800 | -8.08 |
| | K | 678.8 | 97.5 | 13800 | 7.30 |
| | L | 588.8 | 97.5 | 13800 | 5.79 |
| Tread 2 | M | 633.8 | 132 | 10500 | 3.89 |
| | N | -167.2 | 132 | 10500 | -6.28 |
| | O | 633.8 | 63 | 10500 | 6.07 |
| | P | -167.2 | 63 | 10500 | -4.11 |
| Riser 3 | Q | -122.2 | 97.5 | 13800 | -6.08 |
| | R | -212.2 | 97.5 | 13800 | -7.58 |
| | S | -122. | -237.5 | 13800 | 7.79 |
| | T | -212.2 | -237.5 | 13800 | 6.28 |
| Tread 3 | U | -167.2 | -203 | 10500 | 4.27 |
| | V | -967.2 | -203 | 10500 | -5.91 |
| | W | -167.2 | -272 | 10500 | 6.44 |
| | X | -967.2 | -272 | 10500 | -3.74 |
| Riser 4 | Y | -922.2 | -237.5 | 13800 | -5.59 |
| | Z | -1012.2 | -237.5 | 13800 | -7.09 |
| | AA | -922.2 | -572.5 | 13800 | 8.28 |
| | AB | -1012.2 | -572.5 | 13800 | 6.78 |
| Tread 4 | AC | -967.2 | -538 | 10500 | 4.64 |
| | AD | -1767.2 | -538 | 10500 | -5.54 |
| | AE | -967.2 | -607 | 10500 | 6.81 |
| | AF | -1767.2 | -607 | 10500 | -3.36 |

From the above, it appears that the maximum bending stress in the risers is 8.57 N/mm$^2$ For the treads, this maximum stress appears to be 6.81 N/mm$^2$.

Since LVL elements are at stake, there has to be accounted for a material factor $\gamma_M$ = 1.2. Since the grandstand elements are in an outside climate, under cover, service class 3 applies. The modification factor is therefore taken 0.65.

Considering the bending strength of both Kerto-Q and Kerto-S, the following values are found:

Kerto-S: $f_{m;edge;d} = f_{m;edge;k} * \frac{k_{mod}}{\gamma_M} = 44 * \frac{0.65}{1.2} = 23.83\ N/mm^2$

Kerto-Q: $f_{m;flat;d} = f_{m;flat;k} * \frac{k_{mod}}{\gamma_M} = 36 * \frac{0.65}{1.2} = 19.50\ N/mm^2$

It is therefore stated that the proposed grandstand element complies with the requirements largely (U.C. = 0.36).

§ Deflections

When deflections are concerned, the governing combination appears to be:

SLS 1a: $1.0 * (1.74 + 0.20 * 3.2)(\downarrow) \quad + 1.0 * 5.0 * 3.2(\downarrow) \quad + 1.0 * 0.25 * 0.5 * 3.2(\leftarrow)$

This implies that:

$q_{x;rep}$ = 0.40 kN/m and $M_{y;d} = 1/8 * 0.40 * 5.5^2 = 1.51$ kNm

$q_{z;rep}$ = 18.38 kN/m and $M_{z;d} = 1/8 * 18.38 * 5.5^2 = 69.50$ kNm

Implementing this values in the formula as presented earlier, the following values are found:

$\kappa_{x;rep}$ = 0.754 * $10^{-6}$ /mm

$\kappa_{z;rep}$ = 1.866 * $10^{-6}$ /mm

Making use of these values for the curvature, the deflections in both directions are determined. The grandstand is schematised as being a simple beam on two supports, being loaded by the earlier found load q. This situation is schematised in figure 112.

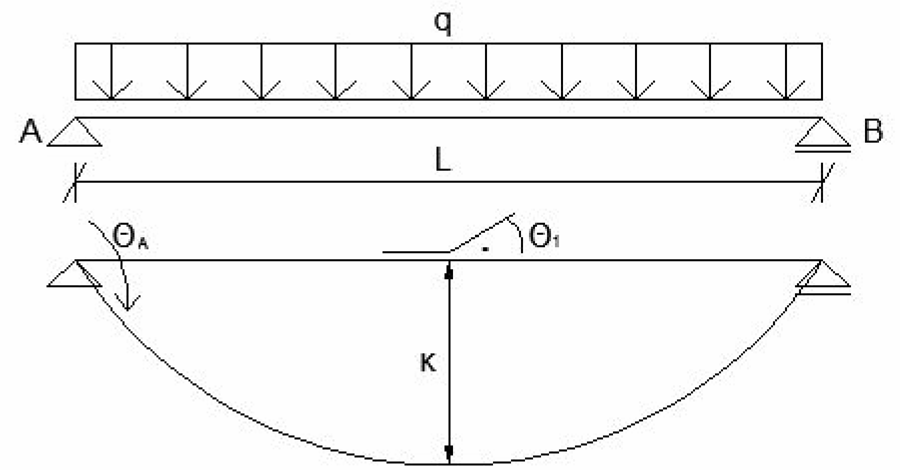



Figure 112: Schematisation and κ-diagram of the grandstand structure

From the above, it is found that:

$\theta_{1;z} = \frac{2}{3} * l * \kappa_z = \frac{2}{3} * 5500 * 1.866 \cdot 10^{-6} = 0.0068$ (area of the parabola)

Since the deflection at B should be equal to zero, it is stated that $\theta_{A;z} = 0.5 * \theta_{1;z} = 0.0034$.

Starting at point A, the deflections a mid-span are determined, while making use of figure 113.

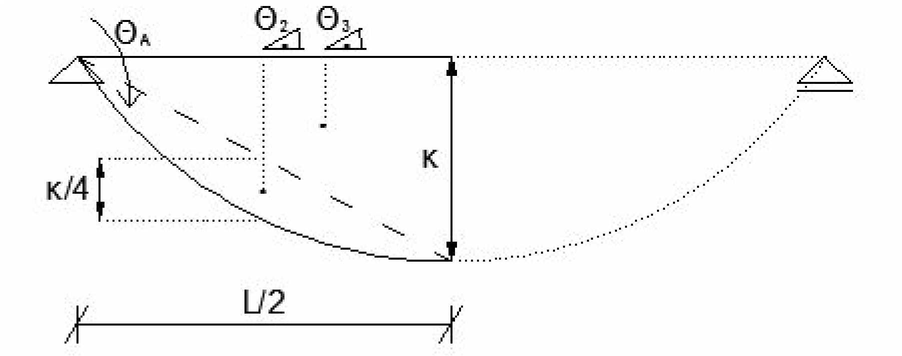



Figure 113: Part of the κ-diagram required to determine the deflection at mid-span

The κ-value varies over the length of the beam and is determined as follows:

$$\kappa(x) = \frac{\kappa}{l^2} * \left(l^2 - (l - 2x)^2\right)$$

At the point where the centre of the parabola is located ($x_c$ = 0.25*l = 1.375 m), it is found that the value for κ is about 1.400 * $10^{-6}$ /mm. At this location, the triangle has a value of 0,5 κ (= 0.933* $10^{-6}$ /mm), whereby the remaining value of the parabola is only 0.467 * $10^{-6}$ /mm.

This results in the following values for $\theta_{2;z}$ and $\theta_{3;z}$:

$$\theta_{2;z} = 2/3 * \frac{l}{2} * \kappa_{z;parabola} = 2/3 * \frac{5500}{2} * 0.467 \cdot 10^{-6} = 0.0009$$

$$\theta_{3;z} = \frac{l}{2} * \kappa_{z;triangle} = \frac{5500}{2} * 0.5 * 1.866 \cdot 10^{-6} = 0.0026$$

Making use of these values, the vertical deflection at mid-span is determined:

$$w_{z;mid} = \theta_{A;z} * \frac{l}{2} - \theta_{2;z} * \frac{l}{4} - \theta_{3;z} * \frac{l}{6} = 0.0034 * \frac{5500}{2} - 0.0009 * \frac{5500}{4} - 0.0026 * \frac{5500}{6} = 5.87 \text{ mm}$$

Making use of the same procedure, the deflection at mid-span is determined for the x-direction as well.

This results in $w_{x;mid}$ = 2.37 mm.

When deflections are concerned, it is required that $w_{max} \leq \frac{1}{250} l$. With a span-width of 5.5 m, this results in a maximum deflection of 22 mm. It is therefore concluded that the grandstand complies with this requirement largely (U.C. = 0.27)

## § Dynamic behaviour

When vibrations are concerned, the governing load combination appears to be:

SLS 2a: 1.0 * (1.74 + 0.20 * 3.2) $(\downarrow)$ + 1.0 * 0.7 * 5.0 * 3.2 $(\downarrow)$ + 1.0 * 0.6 * 0.5 * 3.2 $(\leftarrow)$

This implies that:

$q_{x;rep}$ = 0.96 kN/m and $M_{x;d} = 1/8 * 0.96 * 5.5^2 = 3.63$ kNm

$q_{z;rep}$ = 13.58 kN/m and $M_{z;d} = 1/8 * 13.58 * 5.5^2 = 51.35$ kNm

Implementing these values in the formula as presented earlier, the following values are found:

$\kappa_{x;rep}$ = 0.568 * $10^{-6}$ /mm

$\kappa_{z;rep}$ = 1.406 * $10^{-6}$ /mm

Making use of the same procedure as shown in the preceding section, the horizontal and vertical deflections are determined:

$w_{x;rep}$ = 1.79 mm

$w_{z;rep}$ = 4.43 mm

For vibrations, it was stated earlier that the natural frequency of the grandstand should exceed 5 Hz. Making use of a simplified method to determine the deflections at mid-span, while accounting for a vibration acceleration of 0.315 m/s², it is found that:

$$w_{max} = \frac{a}{f_e^2} = \frac{0.315}{25} = 12.6mm$$

The proposed grandstand element complies with this requirement largely (U.C. = 0.35)

§ Maximum span-width of the proposed elements

Accounting for the proposed grandstand element, it is determined what the maximum span-width of such an element is. Therefore, an iterative calculation is made whereby the span is increased until the element fails to comply with one of the requirements (strength, deflections, dynamic behaviour). The procedure that is applied is similar to the one as elaborated on in this section.

It appears that the maximum span-width of the proposed element is 7.1 m. Accounting for such a span, the following results are found:

| | | | |
|---|---|---|---|
| Strength: | $\sigma_{m;max}$ | = 14.28N/ mm² | < $f_{m;edge;d}$ = 28.4 N/mm² |
| Deflections: | $w_{z;def}$ | = 16.3 mm | < $w_{max;def}$ = 28.4 mm |
| Dynamic behaviour: | $w_{z;dyn}$ | = 12.3 mm | < $w_{max;dyn}$ = 12.6 mm |

§ Maximum span-width while accounting for the grandstand support beams

Up till now, the grandstand support beams have been neglected in the design. Since the requirements on vibrations apply to the grandstands as a whole, there has to be accounted for the vertical deflections of the support beams as well.

The grandstand elements are supported at both edges by support beams. These are in turn supported by columns.

The loads on the support beam follow directly from the support reactions of the grandstand element. Since the grandstand elements are supported over a width of 3.2 m and a height of 1.34 m, the vertical and horizontal loads are divided through these values to obtain a equally divided load:

$$q_{x,beam} = 2 * \frac{0.5q_{x,element} * l_{element}}{h_{element}} \text{ and } q_{z,beam} = 2 * \frac{0.5q_{z,element} * l_{element}}{b_{element}}$$

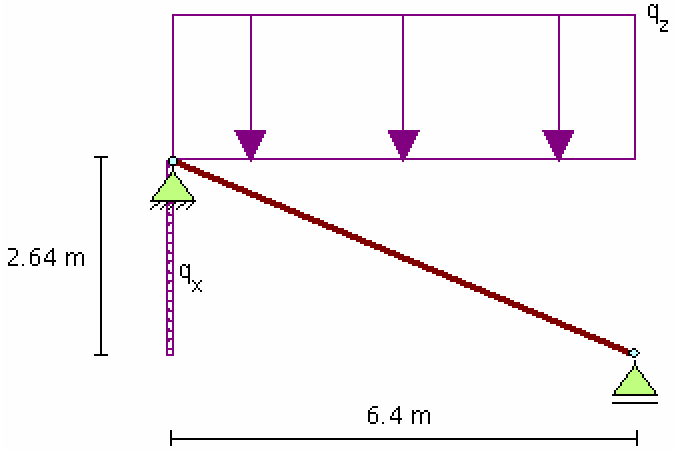



Figure 114: Load conditions on the grandstand support beams

Accounting for these values, the support beams are analysed. The span-width is again assumed to be 5.5 m.

o Strength

When strength is concerned, the following loads are found:

$q_{x;d}$ = 0.66 kN/m and $q_{z;d}$ = 29.54 kN/m

This results in:

$$q_{x,beam} = 2 * \frac{0.5 * 0.66 * 5.5}{1.32} = 2.75 kN/m$$

$$q_{z,beam} = 2 * \frac{0.5 * 29.54 * 5.5}{3.20} = 50.77 kN/m$$

Accounting for these values, the following loads act on the support beams:

M = 262 kNm; V = 151 kN; N = 63 kN

As a first assumption, the support beams are taken GL28h beams with dimensions 200 by 750 mm. This results in the following stresses:

$$\sigma_M = \frac{M}{W} = \frac{6 * 262 \cdot 10^6}{200 * 750^2} = 13.97 \frac{N}{mm^2} \quad < f_{m;0;d} = 28 * \frac{0.8}{1.25} = 17.92 \frac{N}{mm^2}$$

$$\sigma_N = \frac{N}{A} = \frac{63 \cdot 10^3}{200 * 750} = 0.42 \frac{N}{mm^2} \quad < f_{t;0;d} = 19.5 * \frac{0.8}{1.2} = 13.00 \frac{N}{mm^2}$$

$$\tau_d = \frac{3}{2}\frac{V}{A} = \frac{3}{2}\frac{151 \cdot 10^3}{200 * 750} = 1.51 \frac{N}{mm^2} \quad < f_{v;d} = 3.2 * \frac{0.8}{1.2} = 2.05 \frac{N}{mm^2}$$

The proposed beam complies to the requirements on strength, which are assumed to be governing.

o Dynamic behaviour

Accounting for the proposed support beam, the deflections concerning dynamic behaviour are determined. For this situation, the following loads were determined earlier this section:

$q_{x;d}$ = 0.96 kN/m and $q_{z;d}$ = 13.58 kN/m

And thereby:

$$q_{x,beam} = 2 * \frac{0.5 * 0.96 * 5.5}{1.32} = 4.00 kN/m$$

$$q_{z,beam} = 2 * \frac{0.5 * 13.58 * 5.5}{3.20} = 23.34 kN/m$$

The maximum deflections following from these loads amounts 7.0 mm. Together with the deflections that were found for the grandstand elements (under vibration conditions), the total deflection of the system amounts to 11.43 mm.

An iterative calculation showed that the maximum span-width that can be reached for the combined system is 5.70 m.

### 6.3.6 Conclusions on first design

§ The proposed grandstand element, composed of 90 mm thick Kerto-S risers and 69 mm thick Kerto-Q treads, just satisfies all requirements when the span-width is taken 7.1 m.

§ The combined system of grandstand support beams (GL28h) and the grandstand elements just satisfies all requirements when the span-width is limited to 5.7 m.

§ Governing in the design of the grandstand elements are the requirements on dynamic behaviour.

### 6.3.7 Recommendations

- Since it appeared that the grandstand support beams have huge influence on the maximum span-width of the grandstands, it is recommended to optimise these beams. One could, for instance, introduce beams having larger dimensions or stiffer beams (such as steel).

- It is recommended to perform research on the connections between the treads an the risers. Up till now, it was assumed that these connections were infinitesimally stiff. Since this connection is presumably made by means of fasteners, it is highly questionable whether such a situation can be reached.

## 6.4 Design adjustments

In the previous section, a first idea was obtained on the maximum span-width that is reached when making use of timber grandstand elements. The results followed from a first design only, wherefore it is expected that the obtained results (= maximum span-width) can even be somewhat increased.

To improve the structural behaviour of the element, one can opt for increasing the stiffness of the Kerto-plates. This is achieved by altering the dimensions of the treads and the risers.

This is actually easier said than done, since the shape of the grandstand is an inviolable given. The treads require a specific width, while the risers have a height that is directly related to the C-factor.

In this section, the influence of adapting the dimensions of the elements is considered. Goal is to provide insight in the possibilities rather than to deliver an optimised design.

The following design adjustments are considered:

- Increasing the height of the risers (section 6.4.1)
- Increasing the thickness of the treads (section 6.4.2)

The grandstand support beams are neglected here, since their influence can be reduced to a minimum extent easily, for example, by introducing beams with a considerably higher stiffness. They should therefore not be regarded limiting at this stage of the design.

### 6.4.1 Increasing the riser height

#### Analysis

Since the external shape of the grandstand is inviolable, the risers can only be enlarged by adding material at the bottom. Such a solution influences the free height that is available underneath the grandstands, as well as the free height available underneath the support beams. As mentioned in section 2.3.1, the required free height underneath the grandstand should be at least 2.1 m for the purpose of parking. Increasing the riser height, decreases the free height available.

Consult with the project's principal clarified that such a reduction is not really desirable since it limits parking space, but remains acceptable when additional measures are taken, such as proper signalling. When the support beams are concerned, it is mentioned that the height for parking underneath is already limited for the basic situation (without enlarged risers). A further limitation results in the parking spaces become completely unavailable. These consequences are not considered here.

From the above, one could argue that it is undesirable to consider enlarged risers. On the other hand, the 'problems' that arise are only at stake for the lower risers, since the space underneath is limited here. In addition, the behaviour of the grandstand element with enlarged risers might still be interesting for other situations, where the required free height is not a limiting factor.

#### Implementation

As mentioned above, the height of the risers of the grandstand element is doubled, i.e. 670 mm for the type-1 elements and 800 mm for the type-2 elements. Since the 'step' height should remain its original value, the treads are connected halfway the risers. For the lower grandstand element (type-1), this results in the following shape, see figure 115.

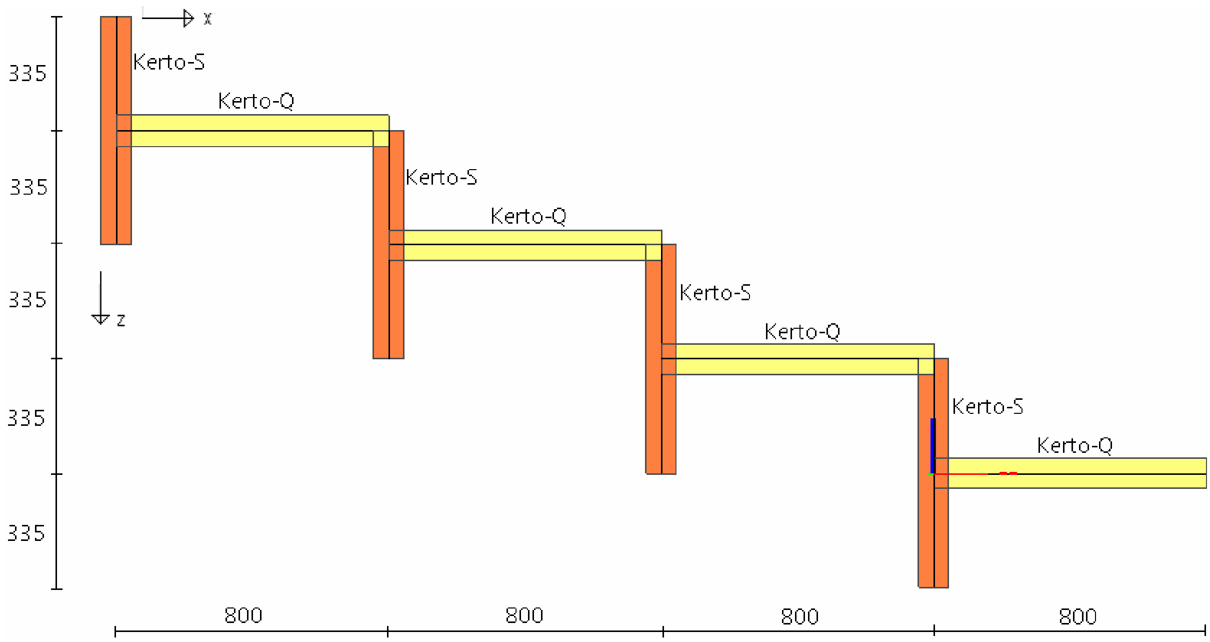



Figure 115: Cross-sectional shape of a grandstand element with increased riser height

Note: no attention is paid to the connection between the treads and the risers. It is therefore recommended to investigate the detailing of such a connection.

Sectional properties

Accounting for the adapted grandstand element, the following coordinates are found for the centre of gravity:

$x_c$ = 1409.2 mm
$z_c$ = 837.5 mm

Making use of these values, the stiffness is determined for the main directions. This results in the following values:

$$EI_{zz} = 9.176 \cdot 10^{14}\ \text{Nmm}^2$$

$$EI_{xx} = 48.621 \cdot 10^{14}\ \text{Nmm}^2$$

$$EI_{xz} = -18.917 \cdot 10^{14}\ \text{Nmm}^2$$

Structural behaviour

Accounting for the proposed grandstand element, it is determined what the maximum span-width of such an element is. Therefore, an iterative calculation is made whereby the span is increased until the element fails to comply with one of the requirements (strength, deflections, dynamic behaviour). The procedure that is applied is similar to the one as elaborated on in 6.3.5.

Due to the increased element dimensions, the self-weight of the element becomes:
$q_{self-weight} = (4 * 69 * 800 + 4 * 90 * 670) * 5.1 = 2.36 kN/m$

The maximum span-width under these conditions appears to be 10.4 m.

Accounting for this value, the aspects of strength, deflections and dynamic behaviour is considered:

§ Strength

At first, the governing load combination is considered:
ULS 2a: 1.32 * (2.36 + 0.20 * 3.2)$(\downarrow)$ + 1.65 * 5.0 * 3.2$(\downarrow)$ + 1.65 * 0.25 * 0.5 * 3.2$(\leftarrow)$

This implies that:

$q_{x;rep}$ = 0.66 kN/m and $M_{x;d} = 1/8 * 0.66 * 10.4^2 = 8.92$ kNm

$q_{z;rep}$ = 30.36 kN/m and $M_{z;d} = 1/8 * 30.36 * 10.4^2 = 410.47$ kNm

Accounting for the values for the accompanying curvature, the acting stresses are determined at any point of the structure by making use of the following formulae:

$$\sigma(x,z) = E\kappa_x * x - E\kappa_z * z = E * 0.888 \cdot 10^{-6} * x - E * 2.280 \cdot 10^{-6} * z$$

The maximum stress that is found for this situation amounts 14.20 N/mm$^2$, which satisfies the requirements:

$\sigma_{m;max}$ = 14.20 N/ mm$^2$ < $f_{m;edge;d}$ = 28.4 N/mm$^2$

§ Deflections

At first, the governing load combination is considered:

SLS 1a: 1.0 * (2.36 + 0.20 * 3.2)$(\downarrow)$ + 1.0 * 5.0 * 3.2$(\downarrow)$ + 1.0 * 0.25 * 0.5 * 3.2$(\leftarrow)$

This implies that:

$q_{x;rep}$ = 0.40 kN/m and $M_{x;d} = 1/8 * 0.40 * 10.4^2 = 5.41$ kNm

$q_{z;rep}$ = 19.00 kN/m and $M_{z;d} = 1/8 * 19.00 * 10.4^2 = 256.88$ kNm

Making use of the same procedure as shown in the preceding section, the vertical deflection is determined:

$w_{z;def}$ = 16.1 mm < $w_{max;def}$ = 28.4 mm

§ Dynamic behaviour

At first, the governing load combination is considered:

SLS 2a: 1.0 * (2.36 + 0.20 * 3.2) $(\downarrow)$ + 1.0 * 0.7 * 5.0 * 3.2$(\downarrow)$ + 1.0 * 0.6 * 0.5 * 3.2$(\leftarrow)$

This implies that:

$q_{x;rep}$ = 0.96 kN/m and $M_{x;d} = 1/8 * 0.96 * 10.4^2 = 12.98$ kNm

$q_{z;rep}$ = 14.20 kN/m and $M_{z;d} = 1/8 * 14.20 * 10.4^2 = 191.98$ kNm

Making use of the same procedure as shown in the preceding section, the vertical deflection is determined:

$w_{z;dyn}$ = 12.3 mm < $w_{max;dyn}$ = 12.6 mm

### Results

It appears that the maximum span-width becomes 10.4 m. Vibration are again governing in the design.

### Conclusions

- When the height of all risers is doubled, the maximum span-width that can be reached is 10.4 m .
- Due to the shape of these elements, the amount of elements that can be transported simultaneously is limited.

### Recommendations

- It is recommended to investigate the feasibility of the connection between the treads and the risers.

### 6.4.2 Increasing the thickness of the treads

#### Analysis

Up till now, the thickness of the members was taken a first assumption, being 69 mm for the treads and 90 mm for the risers. Since vertical deflections seem to be governing in the design, the best improvement is gained by increasing the stiffness in vertical direction. Increasing the thickness of the treads ($h^3$) is therefore beneficial over increasing the thickness of the risers (b).

#### Implementation

To acquire knowledge on the influence of such an increased thickness, the situation is considered where the thickness of all treads is increased to 138 mm. The risers are left untouched.

The schematisation that is used is similar to the one used for the preliminary design.

#### Results

It appears that the maximum span-width that can be reached becomes 8.1 m. Vibrations are again governing in the design.

#### Conclusions

§ Increasing the thickness of the treads to 138 mm results in a (limited) increase in span-width (8.1 m instead of 7.1 m).

## 6.5 Conclusions & Recommendations

As stated in the introduction of, the goal of this chapter was to provide an answer to the following research question:

§ *What spans can be obtained when timber grandstands are concerned?*

Summarising the most striking results, categorised per subject, an answer is given to this question. Unless stated otherwise, the conclusions apply to the situation where the risers are Kerto-S elements with a thickness of 90 mm and the treads are Kerto-Q elements with a thickness of 69 mm.

### 6.5.1 Conclusions

#### Boundary conditions

§ From both a practical and a structural point of view, it is beneficial to make use of elements which are as large as possible.

§ When transport with approval is concerned, the grandstand element may consist of four tiers (i.e. 4 risers and 4 treads) at most.

§ When transport without approval is concerned, it is better to make use of elements consisting of 3 tiers only, especially for the type 3 elements (riser height of 540 mm).

§ The most beneficial solution can be reached by supporting all risers.

#### Preliminary design

§ The maximum distance which the proposed grandstand elements can span is 7.1 m.
§ The maximum span-width for the combined system (grandstand elements and support beams) is limited to 5.7 m.

#### Design adjustments

§ When the riser height is doubled, a span-width of 10.4 m is reached for the elements.

§ Increasing the thickness of the treads to 138 mm results in a maximum span-width of 8.1 m for the grandstand elements.

### 6.5.2 Proposal

It is considered beneficial to implement grandstand elements which have a large span-width.

Accounting for the grid dimensions that followed from the proposed floor solution (9.85 m), it is stated that the span-width of the preliminary grandstand design is governing (5.7 m).

From this point of view, the elements with an increased riser height are the most beneficial solution. The maximum span-width that is reached for this situation is 10.4 m, which exceed the grid dimensions of the floor design.

One should bear in mind that the grandstand support beams have been neglected for the latter situation. Since these deflect to some extent as well, the maximum span-width of the grandstand elements decreases: the system as a whole should comply with the requirements on vibrations.

For the final design, the maximum span-width of the grandstand element should be set to a value that fits the grid dimensions of the other structural elements. To be able to obtain such a span-width for the whole grandstand system, the stiffness of the support beam should be increased since these element proved to be limiting.

### 6.5.3 Recommendations

For the final design stage, it is recommended to pay attention to the following aspects:

- Design of the grandstand support elements and its consequences on the maximum span-width.
- The connection between the treads and risers.
- The connections between the grandstand elements and the grandstand support beams.

# Evaluation Preliminary design

## Structural system

In chapter 4, two kind of structural systems were investigated and evaluated. From this chapter it is concluded that the timber central core, see figure 116, system fits the stadium structure best. For the Eastern building part of the Euroborg stadium a total of 3 cores are required.

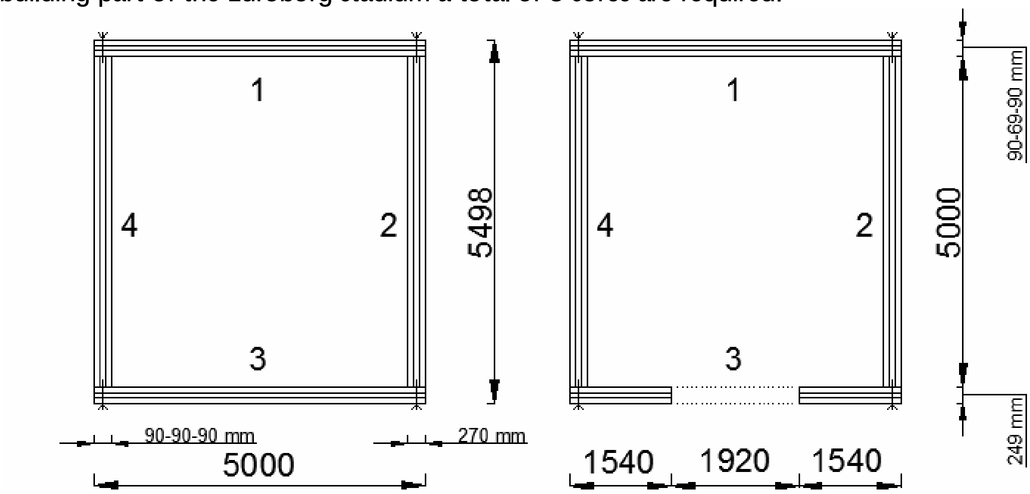



Figure 116: Sections of the fully enclosed core and the core with openings

The main benefits of this structural system are summarized here:

+ Structural behaviour
  Since the core walls are connected to each other, all walls are addressed when the transfer of horizontal loads is concerned. As a consequence, the individual walls are less heavily loaded than for the situation where individual walls are concerned and they can be provided with somewhat smaller dimensions.

+ Fits the functional plan
  Within the stadium structure there has to be accounted for stair cases and elevator shafts, which, according to the architect, disturb the aesthetical appearance of the structure. Due to the proposed dimensions of the cores, these can be extracted from sight by implementing them within the cores.

+ Ease of erection
  Since the proposed cores are directly stable by their selves, there is no need for temporary strutting. As a consequence, other structural elements can be connected to these core directly after erection, which increases the speed of erection to a great extent.

Apart from these benefits, the implementation of the timber core system also introduces some challenges, or points of attention. The most important ones are mentioned here:

* Division in segments
  Consult with an expert on construction technology clarified that the erection of a timber core with such dimensions introduces some problems during erection. It was therefore suggested to divide the core in smaller (storey-high) segments which can be erected one after another. As a consequence, attention is required for the transfer of the acting forces at the joints and the stiffness of the core as a whole. This subject is elaborated on later.

* Corner connections

The core walls perpendicular to the direction of the acting horizontal loads are only addressed when all core walls are connected to each other. Therefore, there has to be accounted for a connection between the edges of the distinct wall elements. This subject is elaborated on later.

∗ Choice of material

The proposed core walls are composed from LVL elements. Due to the available dimensions of these elements there has to be accounted for intermediate connections, to acquire the required width. Since there are other products on the market as well, it is interesting to perform research on these products. This subject is elaborated on later.

For a complete description of the central core system reference is made to section 4.4.

## Floors

In chapter 5, the structural consequences of various floor panel configurations were determined. It was concluded that the “stacked bond pattern in EW-direction”, see figure 117, was most beneficial for the timber stadium structure.

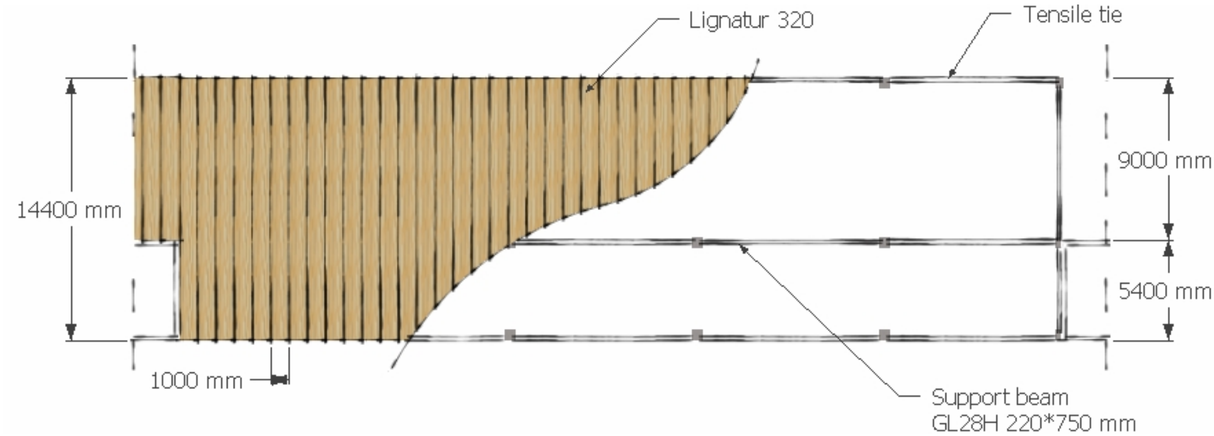



Figure 117: Typical plan of the office floors

The main benefits of such a configuration are summarized here:

+ Grid dimensions
  Since the floor elements span between the façades at the front and the back of the stadium building, the grid dimensions (in NS-direction) are limited by the span-width of the support beams, rather than the span-width of the floor elements. Since the support beams hold more stiffness than the floor elements, the grid size is larger than for the other configurations.

+ Shear connectors required in one direction only
  Since each floor element spans between both façades, there has to be accounted for joints in the longitudinal direction only. This limits the amount of fasteners while it increases the speed of erection.

+ Implementation of ducts
  When stadium structures are concerned, it is regarded beneficial when the ducts run parallel to the stadium’s perimeter. Since the floor support beams run in this direction as well, these ducts are not obstructed by these beams.

Although the proposed panel configuration holds the mos benefits, the implementation introduces some points of attention and negative consequences as well. The most important ones are mentioned here.

∗ Grid dimensions limited due to maximum beam height
  As elaborated on earlier, the maximum span-width of the floor support beams is limited by the required free height. Accounting for stiffer support beams (LVL or steel) or multiple beams placed adjacent to each other, the maximum span-width could even be further increased.

– Heavy loaded floors require intermediate support beams
  The maximum span-width of the Lignatur floor elements is smaller than the required 9.0 m when heavy loaded floors are concerned. As a consequence, an additional support beam is required (at axis A’). Due to the appearance of a parking underneath these floors, in this case, the implementation of

such a beam (and the accompanying columns) does not interfere with the functional requirements. When other functional areas, e.g. offices, are at stake this is generally not acceptable.
Considering the other configurations, the span-width in this direction is limited by the support beams. Implementing stiffer support beams, or multiple support beams adjacent to each other, the intermediate column is not required any more.

For a complete description of the stacked bond pattern reference is made to section 5.5.

## Grandstands

In chapter 6, a first design has been made for the grandstand structure. It was concluded that the maximum span-width of the grandstand elements is limited to 10.4 m. Since the deflections of the grandstand support beams are not accounted for here, this span-width will reduce to some extent.

From the preliminary design it appeared that the minimum span-width of the grandstand system, i.e. elements and support beams, is about 5.7 m.

It is therefore concluded that the span-width of the grandstand system will be somewhere between these boundaries. The exact span-width depends on the stiffness of the support beams and the proposed shape of the grandstand elements.

Accounting for the above, the main points of attention are elaborated on.

* Span-width

  Although the first design is not fully optimised, it provides knowledge on the range in which the maximum span-width can be found. It appeared that a span-width of 10.4 m is reached by increasing the height of the risers. Without such an increase, the span-width of the grandstand elements is limited to 7.1 m.

  Accoutning for the latter design, it appeared that the span-width of the grandstand system (i.e. elements and support beams) is only 5.7 m, due to the deflections of the support beams. Such span-width is not considered acceptable, for the following reasons:

  - For this span-width, a huge amount of grandstand support beams is required. This implies an increased amount of material to be used and an increased number of hosing operations.
  - Considering the requirements on parking, it is stated that only 2 parking lots (2*2.5 m + 2 *0.15 m = 5.3 m) can be implemented in between the grandstand support beams. The total amount of parking lots is thereby reduced to a great extent.

  Considering the span-width of 10.4 m that was found for the system with enlarged risers, it is mentioned that there has not yet been accounted for the support beams here. Such a span-width may only be reached by implementing infinitesimally stiff beams, which do not deflect at all. Additionally, implementation of these elements requires some additional attention:

  - Due to the inviolable shape of the grandstand, the risers are enlarged at the bottomside of the grandstand. Except for this solution reducing the free height underneath the grandstand, additional attention is required to support these elements on the support beams.

* The connections between the treads and the risers have to be investigated

  The design as discussed in chapter 6, did not account for the connections between the treads and the risers, i.e. these were assumed infinitesimally stiff. When the elements are connected to each other by means of mechanical fasteners, it is highly questionable whether such a stiffness can be reached.

  Before drawing conclusions on the maximum span-width of the grandstand elements, an analysis should be made on these connections. It is expected that the maximum span-width that can be reached reduces to a considerable extent.

For a complete description of the grandstand design, reference is made to chapter 6.

## Proposal for the Detailed design

Accounting for the evaluation in the previous sections, it is stated that the maximum grid dimensions in NS-direction range between 5.7 m (preliminary grandstand system) and 9.65 m (maximum span-width of the floor support beams)[(1)].

Considering the functional requirements as stated in section 2.3.1, it is found that the width of a parking place should be at least 2.5 m when parking under an angle of 60º. Since an additional column is required at axis B, an additional safety margin of 0.15 m should be applied for those lots adjacent to these columns. Accounting for 3 parking lots in between these columns, a centre-to-centre distance of 7.8 m is required. Four parking lots would already require a c.t.c.-distance of 10.3 m, which is outside the range.

It is therefore opted to design both the grandstand and the floor support beams to a span-width of 7.8 m. This implies that the dimensions of the floor support beams could be reduced.

The grandstand requires additional attention:

§ The maximum span-width of the preliminary grandstand elements is limited to 7.1 m. To obtain a span-width of 7.8 m, the riser height should therefore be increased.

§ Although the elements with enlarged risers are able to span 10.4 m, the span-width of the grandstand system reduces to some extent (due to deflections of the support beams). To obtain a span-width of 7.8 m, the optimal ratio has to be determined between the stiffness of the support beams and the height of the riser.

Since it promises to be feasible to obtain a span-width of 7.8 m, this proposal for the final design has a grid dimension that is equal to this value. An overview on the proposed design is shown in figure 118.

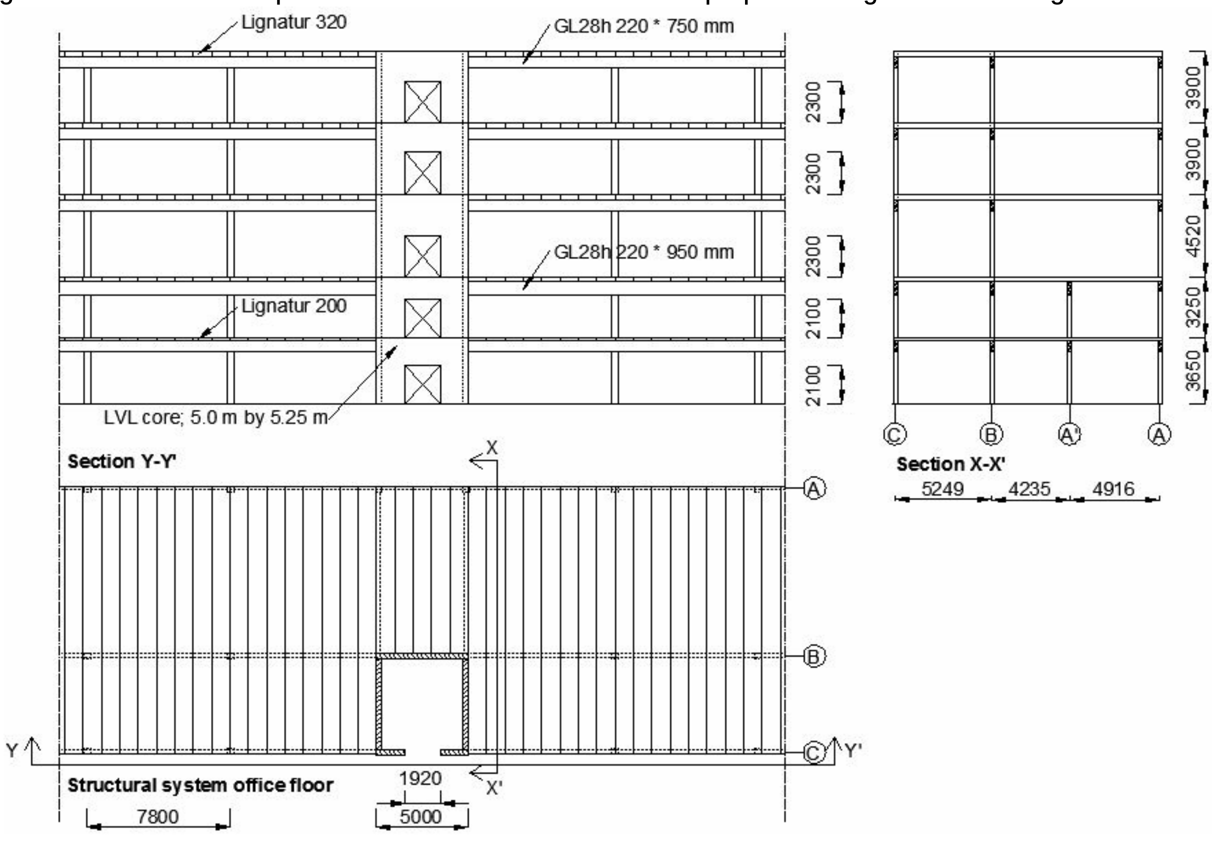



Figure 118: Overview of the proposed stadium structure

The grandstands are left out of this overview, since additional research is required on the dimensions of the support beams and the connections with the stadium building.

(1) The influence of the connections between the grandstand elements and the option to implement stiffer support beams is neglected here.

# Detailed design

In the preceding part of this thesis, the most beneficial structural solution has been determined on the subjects ‘structural system’, ‘floor system’ and ‘grandstands’. This resulted in a design proposal as elaborated on in the section Evaluation Preliminary design and as shown in figure 118.

The preliminary design phase was concluded by identifying the points of attention of the proposed timber stadium structure. Due to the preliminary character of the proposed design, there are still several aspects that require detailed attention.

In this part of the thesis, the most interesting points of attention are investigated in detail. Due to the limited time span, not all aspects are covered. The focus is on the timber core system, since the feasibility of this structural system depends on several aspects that have not been validated in the preliminary design.

## Chapter 7 Lintels

As mentioned in section 4.4.1, one of the core walls contains openings to provide a passage from and to the stair cases (or elevators) which are located within the cores. In the preceding part of this thesis, no attention has been paid to the lintels which span above these openings. Since the feasibility of the timber core is dependent on these lintel, it is investigated whether the lintels above these openings are able to resist the acting forces and whether the material at the ‘corners’ is able to transfer the acting forces.

## Chapter 8 Core walls divided in segments

Evaluating the preliminary design phase, it was recommended to divide the core in storey-high segments. In this chapter the structural consequences of such a division are investigated, whereby the feasibility of the solution is checked. At first, attention is paid to the anchorage that is required to transfer the tensile forces from one element to another (section 8.2). Thereafter, attention is paid to the transfer of the shear forces (section 8.3). The chapter is concluded by an analysis on the stiffness of the segmented core.

## Chapter 9 Core wall connections

In the preliminary design phase the core wall connections were more or less left out of consideration. Since these connections hold large consequences on the stiffness of the core, these are considered more in detail. In this chapter, two kinds of connections are distinguished for the proposed core system:

§ The intermediate wall connections (section 9.2), which are required to obtain the required wall width of 5.0 m. These were completely left out of consideration in the preliminary design.
§ The corner connections (section 9.3), which connect the distinct walls to each other. In the preliminary design, the lay-out of this connection was taken a first assumption only.

## Chapter 10 Core walls composed of CLT elements

In the preliminary design phase the walls of the timber core were designed being LVL elements, since these elements were considered beneficial over CLT elements. Since there has to be accounted for intermediate connections when the CLT core is concerned, the CLT elements might offer benefits over the LVL elements.

## Chapter 11 Additional checks

Accounting for the results obtained in the chapters concerning the detailed design, additional checks are performed on the feasibility of the timber core. At first, attention is paid to the behaviour under fire conditions, thereafter, there is elaborated on buckling of the individual core walls.

Note: starting point for all subjects discussed in this detailed design phase is the proposed preliminary design, unless explicitly stated otherwise.

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# 7 Lintels

## 7.1 Introduction

As mentioned in the chapter structural system, the proposed core holds openings in core wall 3 to provide entrances to staircases and elevator shafts. Since these openings should provide an unobstructed passageway, they have a height that is equal to the required free height, which is dependent on the specific use function at that floor.

Since the storey-height exceeds the free height at all levels, some sort of beams can be distinguished in between the openings, see figure 119. From now on, these beams are referred to as the 'lintels'. The timber adjacent to the openings is referred to as 'columns'.

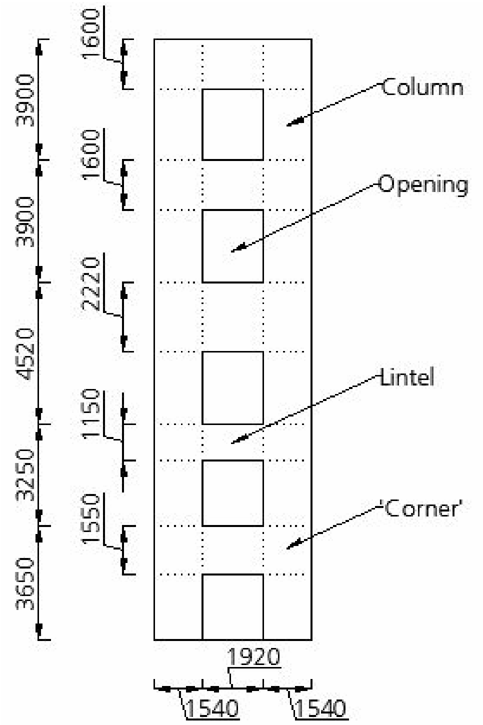



Figure 119: Core wall containing openings

In this chapter, it is investigated whether the lintels are able to resist the acting loads. Next to that, it is investigated whether the timber is able to transfer the loads from the lintel to the columns.

## 7.2 Analysis

The lintels are composed from one layer of Kerto-Q (69 mm) and two layers of Kerto-S (180 mm). The Kerto-Q layer has its main grain direction in transverse direction (from one column to another). When the columns are concerned, the build-up is similar, having the Kerto-S elements spanning in longitudinal direction.

As a consequence, the Kerto-Q layer is the main load-bearing layer for the columns, while the Kerto-S elements carry the loads at the columns.

## 7.3 Design assumptions

- Only the governing NS-direction is considered
- For the bending stiffness of the core, there is accounted for the stiffness including openings

## 7.4 Loads

The acting loads and the various load combinations were already determined in Annex C.1. Therefore, only the governing combination is presented here, see table 14.

Table 10: Governing loads NS-direction

| | ULS |
|---|---|
| $q_{wind}$ [kN/m] | 28 |
| N [kN] | 4632 |

Accounting for these loads, the load distribution over the height of the core is determined, see table 15.

Table 11: Load distribution over the height of the core (NS)

| Level | M [kNm] | V [kN] | N [kN] |
|---|---|---|---|
| 2 | 213 | 164 | 927 |
| 1 | 852 | 282 | 1853 |
| 0 | 2125 | 391 | 2780 |
| -1 | 3394 | 487 | 3706 |
| -2 | 5171 | 538 | 4632 |

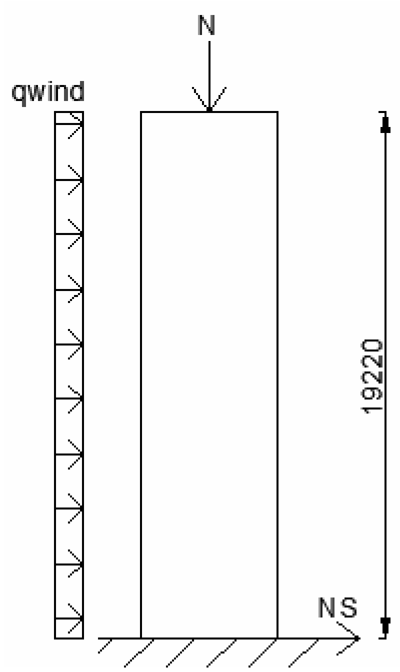



Figure 120: Governing load combination in NS-direction

## 7.5 Strength of the lintels

### 7.5.1 Analysis

The lintels should be able to resist the forces that act on it. To determine the governing load, use is made of the theory as presented by Sagel [ 22 ]. It appears that the governing shear forces follow from the difference in bending moment above and underneath the lintel, see figure 121.

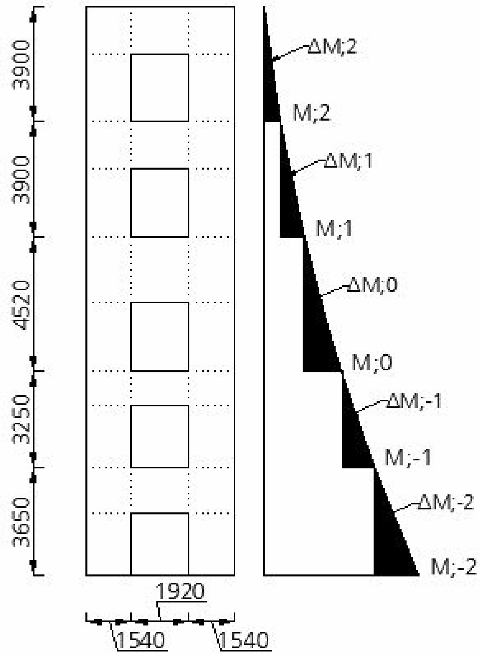



Figure 121: Bending moment distribution over the height of the core

Making use of the values for the bending moment as presented in table 15, the following values are obtained, see table 12.

Table 12: $\Delta M$ acting on the core (NS-direction)

| Level | $h_{lintel}$ [mm] | $\Delta M$ [kNm] |
|---|---|---|
| 2nd | 1600 | 213 |
| 1st | 1600 | 639 |
| 0 | 2220 | 1273 |
| -1st | 1150 | 1269 |
| -2nd | 1550 | 1777 |

From the above, it is concluded that the lintels at -2nd and -1st floor are governing for the design, having the largest $\Delta M$ and the smallest lintel height respectively. These lintels are therefore considered in the continuation of this chapter.

Considering Annex C.1.4, it is found that the bending stiffness of the core (including openings) in NS-direction amounts $1.501*10^{17}$ N/mm$^2$, while wall 3 holds a Young's modulus of 10641 N/mm$^2$.

## 7.5.2 Stresses

Making use of these values, the bending stresses that act at wall 3 are determined by making use of the following formula:

$$\sigma_{\Delta M-2} = \frac{0.5 * E_i * h_i * \Delta M_{-2}}{EI_{eff}} = \frac{0.5 * 10641 * 5000 * 1777 \cdot 10^6}{1.501 \cdot 10^{17}} = 0.32 N/mm^2$$

$$\sigma_{\Delta M-1} = \frac{0.5 * E_i * h_i * \Delta M_{-1}}{EI_{eff}} = \frac{0.5 * 10641 * 5000 * 1269 \cdot 10^6}{1.501 \cdot 10^{17}} = 0.23 N/mm^2$$

These stresses act near the edge of wall 3, indicated as $\sigma_2$ in figure 122. Near the openings, where the lintels are found, these stresses are reduced to a certain value $\sigma_1$.

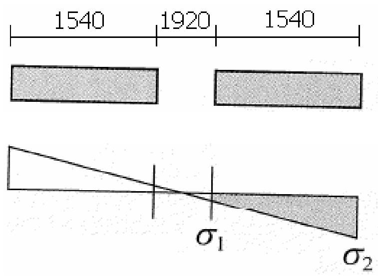



Figure 122: Section of the wall with openings and accompanying stresses

The value for $\sigma_1$, follows directly from the value of $\sigma_2$ and the layout of the wall:

$$\sigma_{1;-2} = \frac{\sigma_{2;-2}}{0.5 * h} * 0.5 * h_{opening} = \frac{0.32}{0.5 * 5000} * 0.5 * 1920 = 0.12 N/mm^2$$

$$\sigma_{1;-1} = \frac{\sigma_{2;-1}}{0.5 * h} * 0.5 * h_{opening} = \frac{0.23}{0.5 * 5000} * 0.5 * 1920 = 0.09 N/mm^2$$

The maximum shear force that has to be resisted by the lintel, follows from the capacity of the stress diagram:

$$V_{d;-2} = b * h_{column} * \frac{\sigma_{2;-2} + \sigma_{1;-2}}{2} \cdot 10^{-3} = 249 * 1540 * \frac{0.32 + 0.12}{2} \cdot 10^{-3} = 84.4 kN$$

$$V_{d;-1} = b * h_{column} * \frac{\sigma_{2;-1} + \sigma_{1;-1}}{2} \cdot 10^{-3} = 249 * 1540 * \frac{0.23 + 0.09}{2} \cdot 10^{-3} = 61.4 kN$$

This shear force acts in the middle of the lintel, which is schematised as being fixed to the columns at both ends, see figure 123. As a consequence, the shear force introduces a bending moment near the columns.

$$M_{d;-2} = V_{d;-2} * \frac{h_{opening}}{2} \cdot 10^{-3} = 84.4 * \frac{1920}{2} \cdot 10^{-3} = 81.0 kN$$

$$M_{d;-1} = V_{d;-1} * \frac{h_{opening}}{2} \cdot 10^{-3} = 61.4 * \frac{1920}{2} \cdot 10^{-3} = 58.9 kN$$

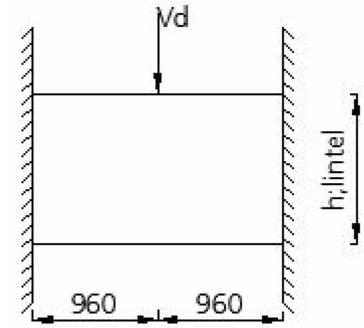



Figure 123: Schematisation for the lintel calculation

Accounting for these values and the sectional properties of the distinct lintels, the bending and shear stresses are determined.

§ -2nd lintel

$A_{eff;-2}$ = 249 * 1550 = 386.0 * $10^3$ $mm^2$

$W_{eff;-2}$ = 1/6 * 249 * $1550^2$ = 99.7 * $10^6$ $mm^3$

$$\sigma_{M;-2} = \frac{M_{d;-2}}{W_{eff;-2}} = \frac{81.0 \cdot 10^6}{99.7 \cdot 10^6} = 0.81 N/mm^2$$

$$\tau_{max;-2} = \frac{3}{2}\frac{V_{d;-2}}{A_{eff;-2}} = \frac{3}{2}\frac{84.4 \cdot 10^3}{386.0 \cdot 10^3} = 0.33 N/mm^2$$

§ -1st lintel

$A_{eff;-1}$ = 249 * 1150 = 286.3 * $10^3$ $mm^2$

$W_{eff;-1}$ = 1/6 * 249 * $1150^2$ = 54.9 * $10^6$ $mm^3$

$$\sigma_{M;-1} = \frac{M_{d;-1}}{W_{eff;-1}} = \frac{58.9 \cdot 10^6}{54.9 \cdot 10^6} = 1.07 N/mm^2$$

$$\tau_{max;-1} = \frac{3}{2}\frac{V_{d;-1}}{A_{eff;-1}} = \frac{3}{2}\frac{61.4 \cdot 10^3}{286.3 \cdot 10^3} = 0.33 N/mm^2$$

It appears that the lintel underneath the -1st floor is governing in the design.

### 7.5.3 Strength verification

As mentioned in section 4.4, the lintels are composed of three layers: one Kerto-Q (69 mm) and two Kerto-S (180 mm). When the bending strength of this lintel is concerned, one should note that only the Kerto-Q layer is activated:

$$f_{m;0;d} = \frac{69 * 10500 * 32}{69 * 10500 + 180 * 490} * \frac{0.9}{1.2} = 21.40 \ N/mm^2$$

Since this value exceeds the acting stress by far, no problems are to be expected concerning bending. The same is valid for the shear stresses, since the shear strength is quite large as well:

$$f_{v;0;edge;d} = 4.5 * \frac{0.9}{1.2} = 3.38 \ N/mm^2$$

## 7.6 Strength of the ‘corner connection’

### 7.6.1 Analysis

Another important aspect that has to be considered, concerns the connection between the ‘columns’ and the ‘lintel’. Since the grain direction of the columns is perpendicular to the grain direction of the lintels, it has to be checked whether the material is able to transfer the acting forces.

Before elaborating on the design calculations, a schematisation is provided on the connection in question, see figure 124.

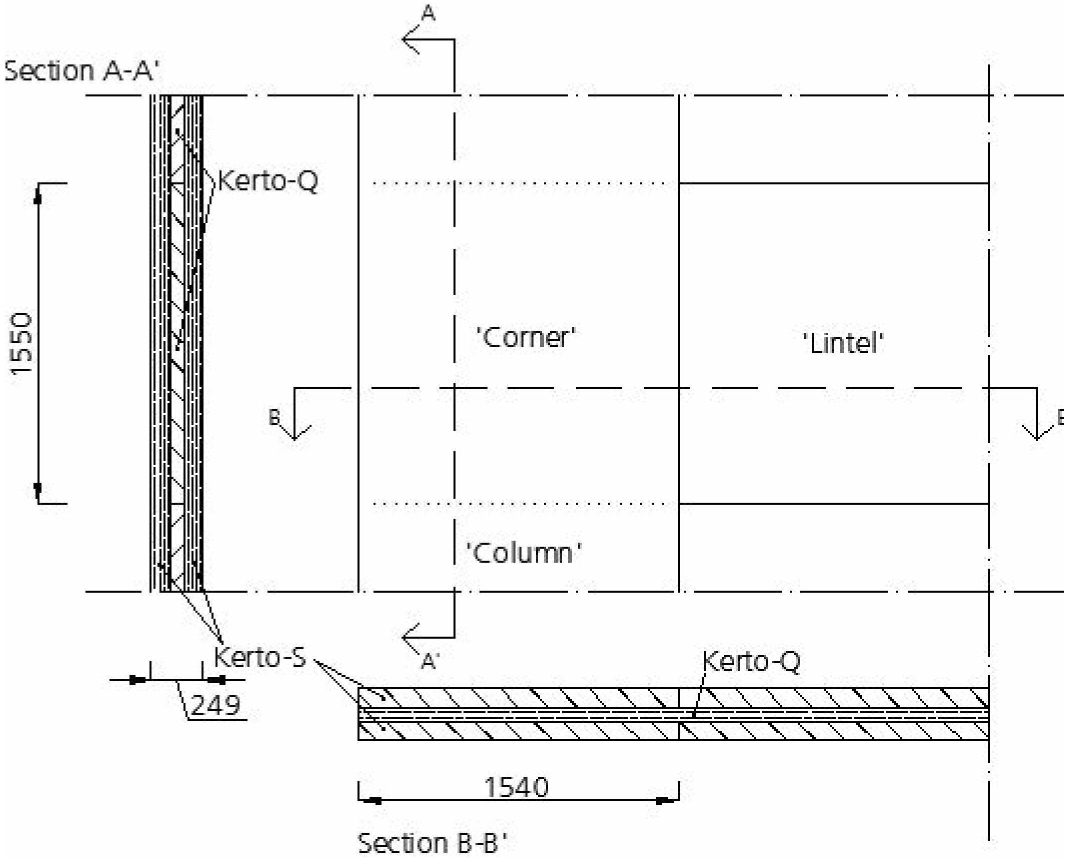



Figure 124: Connection of a ‘lintel’ to a adjacent ‘column’

It is assumed that the Kerto-Q layer of the lintel is glued in between the Kerto-S layers of the columns.

### 7.6.2 Global distribution of forces

Considering the loads as presented in section 7.4, it is mentioned that the horizontal loads are divided over both side walls, being wall 1 and 3. It is assumed that these loads are divided equally over both walls, whereby the acting shear forces at wall 3 are only half of the shown values.

Additionally, it is assumed that this shear force is in turn equally divided over both columns. This results in the following load distribution, see figure 125 and table 13.

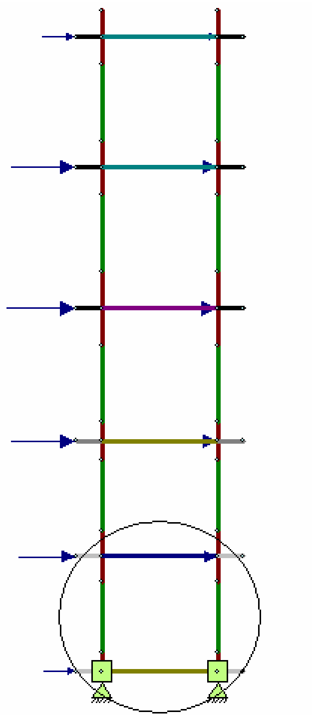


Figure 125: Load distribution at core wall 3

Table 13: Load distribution at core wall 3

| Level | $V_{col;1} = V_{col;2}$ [kN] |
|---|---|
| 3 | 13.6 |
| 2 | 27.3 |
| 1 | 29.5 |
| 0 | 27.2 |
| -1 | 24.2 |
| -2 | 12.8 |

Accounting for the load distribution as presented here, the flow of forces around the governing part (indicated in figure 125) of the structure is considered by making use of a Technosoft model, see figure 126.

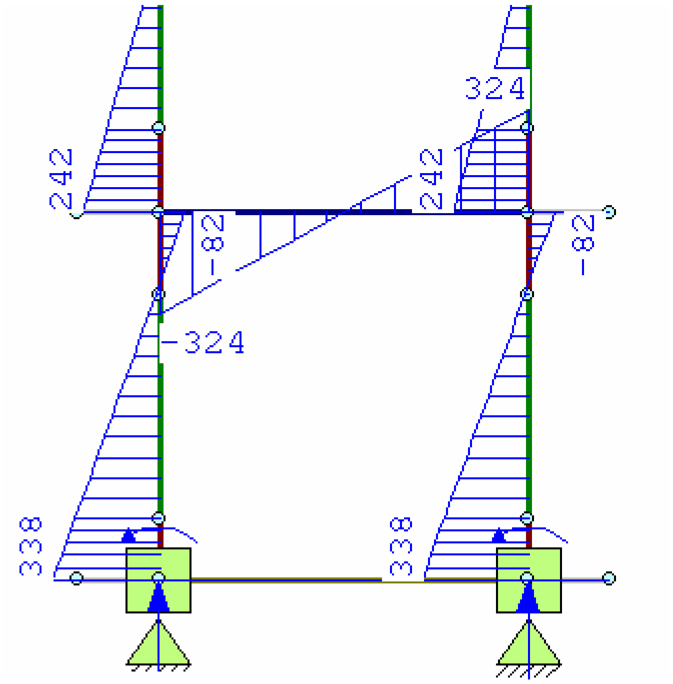



Figure 126: Flow of forces around the connection.

### 7.6.3 Local distribution of forces

The global distribution of forces being known, a closer look is taken at the flow of forces within the corner itself, where the columns and lintels meet. It is assumed that the leverage arm of the bending moment is about 2/3 of the height of the member. This results into the flow of forces as shown in figure 127.

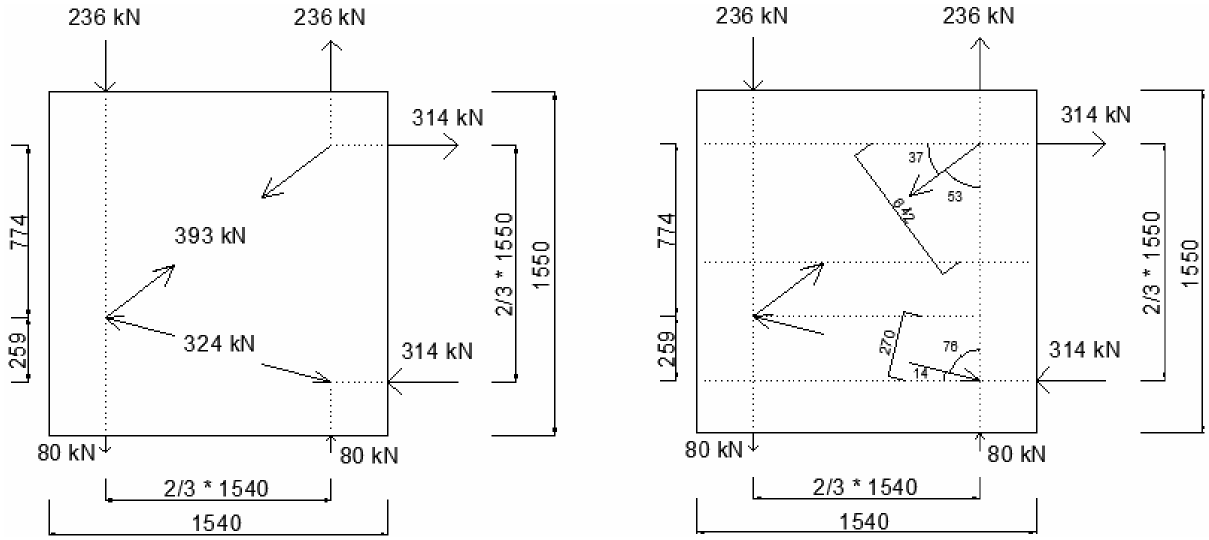



Figure 127: Flow of forces within the node

The maximum force in the node, due to the bending moments, appears to be a tensile force of 393 kN. This load acts under an angle of about 37 degrees to the grain of the lintel (and thus and angle of 53 degrees to the grain of the column). In addition, a compression force with a magnitude of 324 kN acts under an angle of about 14 degrees to the grain of the lintel.

## 7.6.4 Verification of the strength of the 'corner'

It is checked whether the cross-section is able to withstand these acting forces. Therefore, the strength of the column and the lintel are determined first, when loaded at an angle to their grain. This results in the following values, both for tension and compression:

§ Lintel (Kerto-Q)

$$f_{t;33;d} = \frac{f_{t;0;d}}{\frac{f_{t;0;d}}{f_{t;90;d}} * \sin^2 37 + \cos^2 37} = \frac{19.5}{\frac{19.5}{4.5} * \sin^2 37 + \cos^2 37} = 8.83\ N/mm^2$$

$$f_{c;14;d} = \frac{f_{c;0;d}}{\frac{f_{c;0;d}}{k_c f_{c;90;d}} * \sin^2 14 + \cos^2 14} = \frac{19.5}{\frac{19.5}{0.835 * 6.75} * \sin^2 14 + \cos^2 14} = 17.05\ N/mm^2$$

§ Column (Kerto-S)

$$f_{t;57;d} = \frac{f_{t;0;d}}{\frac{f_{t;0;d}}{f_{t;90;d}} * \sin^2 53 + \cos^2 53} = \frac{26.25}{\frac{26.25}{0.6} * \sin^2 53 + \cos^2 53} = 0.93\ N/mm^2$$

$$f_{c;76;d} = \frac{f_{c;0;d}}{\frac{f_{c;0;d}}{k_c f_{c;90;d}} * \sin^2 76 + \cos^2 76} = \frac{26.25}{\frac{26.25}{0.835 * 4.5} * \sin^2 76 + \cos^2 76} = 3.96\ N/mm^2$$

The effective width, over which the acting loads act, is determined from figure 127. It is assumed that the tensile load can only be taken by that part of the connection that is stressed in tension. The same yields for the compression force.

This results $b_{t;eff} = \frac{1/3 * h}{\cos(37)} = 642mm$ and $b_{c;eff} = \frac{h_{reduced}}{\cos(14)} = 270mm$.

$$F_{t;d} = f_{t;37;d} * b_{t;eff} * t_{lintel} + f_{t;53;d} * b_{t;eff} * t_{column} = (8.83 * 642 * 69 + 0.93 * 642 * 180) \cdot 10^{-3} = 499kN$$

$$F_{c;d} = f_{c;14;d} * b_{c;eff} * t_{lintel} + f_{c;76;d} * b_{c;eff} * t_{column} = (17.05 * 270 * 69 + 3.96 * 270 * 180) \cdot 10^{-3} = 510kN$$

Compared to the acting values for both the compression and tensile force, it is concluded that the material is able to transfer the loads from the lintel to the column.

Note: The capacity of the glueline is not checked here, since in most design calculations, it is automatically assumed that the glue is not governing for the connection. Therefore, it is considered acceptable to neglect the check on the glue.

More information on glued moment resisting connections can be extracted from Leijten [ 23 ].

## 7.7 Conclusions and recommendations

§ The proposed lintels are perfectly capable of resisting and transferring the acting loads

§ It is recommended to investigate the behaviour of the glued corner connection more into detail.

# 8 Core walls divided in segments

## 8.1 Introduction

In the preliminary design phase, the cores were designed as being composed of four individual walls, each wall having a height of 19.22 m and a width of 5.0 m.

When the structural systems were evaluated, see section 4.6, an expert posed questions on the feasibility of erecting a core with such dimensions. He recommended to divide the core in segments with a height of only one or two storeys.

In this chapter, it is investigated what the structural consequences are, when the core is divided in segments of only a single storey.

### 8.1.1 Analysis

As discussed in section 4.4, the cores (see figure 128) are loaded by a bending moment caused by the wind and a normal force caused by the loads on the distinct floors and the core's self-weight. It appeared that, at least near the foundation, the tensile stresses caused by the bending moment exceed the normal stresses due to the vertical load. As a result tensile stresses occur in the core walls.

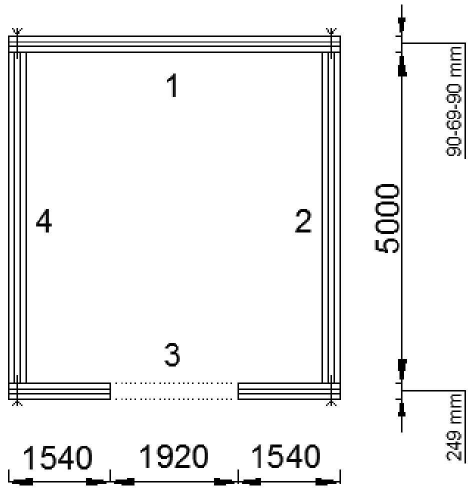



Figure 128: Timber central core

In case the core walls are composed from one single element, the material is able to transfer these tensile stresses by itself. But since the walls are divided in segments, tensile anchorage is required at the horizontal joints to provide an adequate solution.

Apart from the requirements on strength, also the stiffness of the core is at stake. Due to the division in segments, the tensile anchorage plays an important role. It is investigated whether the tensile anchorage is able to provide adequate stiffness, or whether an alternative solution is required.

### 8.1.2 Approach

At first, two solutions to anchor the tensile stresses are investigated (section 8.2). Accounting for the acting loads, the required amount of anchorage is being determined for each storey.

Thereafter, three possible solutions are investigated to provide adequate transfer of shear forces (section 8.3).
Finally, it is investigated whether the proposed solutions provide the core with adequate stiffness (section 8.4). If this is not the case, an alternative solution is presented.

### 8.1.3 Design assumptions

§ The design is based on the cores as proposed at the end of the preliminary design phase, i.e. the influence of the connections (chapter 9) and of CLT elements (chapter 10) are not considered.

§ The focus is on the situation where the core is loaded by the combination of the maximum acting wind force and the horizontal grandstand force (EW-direction).

§ The leverage arm (z) over which the bending moment is divided is assumed to be 2/3*h. It is checked later, whether this assumption holds.

## 8.2 Anchorage of tensile stresses

When the core is divided in segments, measures are required to transfer the tensile forces at the horizontal joints. In this section, two solutions are considered:

§ Anchorage by means of steel bars
§ Anchorage by means of glued-in rods

### 8.2.1 Analysis

Since the acting bending moment increases towards the foundation, the acting (tensile) stresses vary over the height of the core: near the foundation the stresses are the largest, while they are close to zero at the top of the structure. The tensile stress that has to be transferred at each horizontal joints follows from the bending moment at these locations.

As is elaborated on later, the steel bars are implemented in such a way that these only transfer the stresses due to the difference in bending moment at each horizontal joint (ΔM), see figure 129.

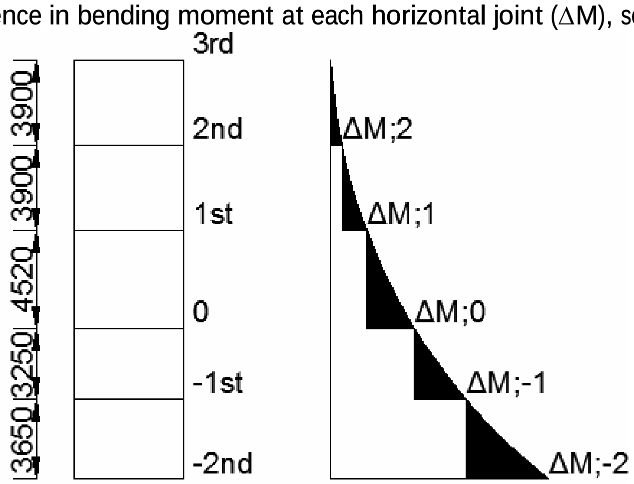



Figure 129: Distribution of the bending moment over the core height

The glued-in rods should be able to transfer the total acting tensile stress at each horizontal joints.

### 8.2.2 Loads

The acting loads and the various load combinations were already determined in Annex C.1. Therefore, only the governing combination is presented here, see table 14.

Table 14: Governing loads

| | ULS |
|---|---|
| $q_{wind}$ [kN/m] | 134 |
| $F_2$ [kN] | 64 |
| $F_0$ [kN] | 52 |
| $F_{-1}$ [kN] | 52 |
| N [kN] | 4632 |

Accounting for these loads, the load distribution over the height of the core can be determined, see table 15.

Table 15: Load distribution over the height of the core

| Level | M [kNm] | V [kN] | N [kN] |
|---|---|---|---|
| 2 | 1019 | 261 | 927 |
| 1 | 4326 | 848 | 1853 |
| 0 | 10708 | 1412 | 2780 |
| -1 | 17158 | 1984 | 3706 |
| -2 | 26280 | 2743 | 4632 |

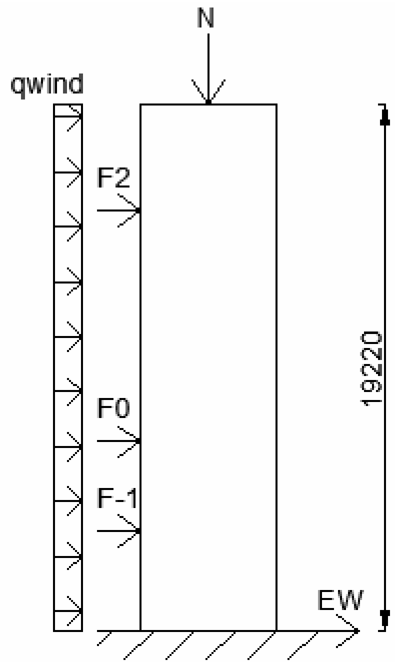



Figure 130: Governing load combination in EW-direction

### 8.2.3 Stresses

The stress distribution is determined from the loads as presented in table 15, by making use of the following formulae:

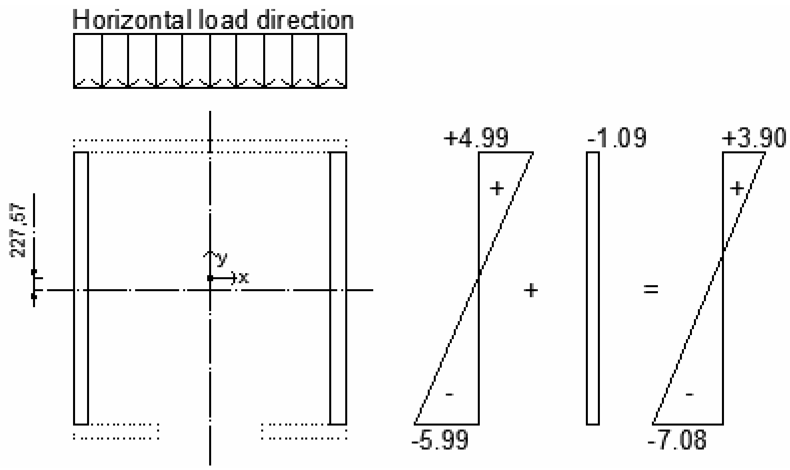
$$\sigma_{m;i} = \frac{0.5 * E_i * h_i * M}{EI_{ef}} \pm \frac{\gamma_i * E_i * a_i * M}{EI_{ef}} \; ; \; \sigma_N = \frac{N * E_i}{EA_{tot}}$$

The sectional properties of the core walls are provided in table 16 and follow directly from Annex C.1.4.

Table 16: Sectional properties of the individual core walls

| i | $h_i$ [mm] | $\gamma_{y;i}$ [-] | $E_{eff;i}$ [N/mm$^2$] | $a_{y;i}$ [mm] | $EI_y$ [Nmm$^2$] | $EA_i$ [N] |
|---|---|---|---|---|---|---|
| 1 | 249 | 0.547 | 10641 | 2396.93 | $16.53*10^{16}$ | $1.325*10^{10}$ |
| 2 | 5000 | 1.0 | 13800 | 227.57 | $16.53*10^{16}$ | $1.863*10^{10}$ |
| 3 | 249 | 0.662 | 10641 | 2852.07 | $16.53*10^{16}$ | $0.816*10^{10}$ |
| 4 | 5000 | 1.0 | 13800 | 227.57 | $16.53*10^{16}$ | $1.863*10^{10}$ |

Due to the a-symmetrical shape of the core, the maximum tensile and compression stresses differ when the direction of the horizontal load is changed 180 degrees. As an example, the maximum stresses at wall 2 of the -2nd core segment are shown for differing load direction, see figure 131.



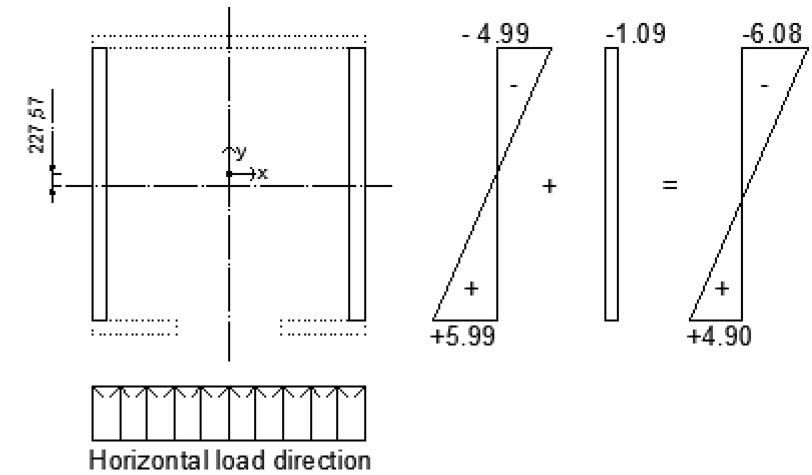



Figure 131: Stress distribution in walls 2/4, accounting for the governing load situation

Applying this approach to all walls and all core segments, the following overview is obtained, see table 17.

Table 17: Stresses on the distinct walls of the individual core segments

| Level | Wall number | $\sigma_M$ [N/mm$^2$] | $\sigma_N$ [N/mm$^2$] | $\sigma_{c;max}$ [N/mm$^2$] | $\sigma_{t;max}$ [N/mm$^2$] |
|---|---|---|---|---|---|
| 2 | 1 | 0.10 | 0.17 | 0.27 | 0.00 |
| | 2/4 | 0.23 | 0.22 | 0.45 | 0.01 |
| | 3 | 0.13 | 0.17 | 0.30 | 0.00 |
| 1 | 1 | 0.40 | 0.34 | 0.72 | 0.08 |
| | 2/4 | 0.98 | 0.44 | 1.42 | 0.54 |
| | 3 | 0.56 | 0.34 | 0.88 | 0.24 |
| 0 | 1 | 0.99 | 0.50 | 1.49 | 0.49 |
| | 2/4 | 2.44 | 0.65 | 3.09 | 1.79 |
| | 3 | 1.39 | 0.50 | 1.89 | 0.89 |
| -1 | 1 | 1.59 | 0.67 | 2.26 | 0.92 |
| | 2/4 | 3.91 | 0.87 | 4.78 | 3.04 |
| | 3 | 2.23 | 0.67 | 2.90 | 1.56 |
| *-2* | 1 | 2.43 | 0.84 | 3.27 | 1.59 |
| | *2/4* | *5.99* | *1.09* | *7.08* | *4.90* |
| | 3 | 3.41 | 0.84 | 4.25 | 2.57 |

## 8.2.4 Tensile and compression forces

Accounting for the stresses in table 17, the maximum acting tensile and compression forces are determined. To clarify the approach that has been used to determine these forces, the following example is provided. This example applies to wall 2 of the -2$^{nd}$ core segment and is directly related to figure 131.

The maximum compression force:

$$F_c = 0.5 * \sigma_{c;max} * b * \left( \frac{l}{\sigma_{c;max} + \sigma_{t;max}} * \sigma_{c;max} \right) = 0.5 * 7.08 * 270 * \left( \frac{5000}{7.08 + 3.90} * 7.08 \right) = 3082 kN$$

The maximum tensile force:

$$F_t = 0.5 * \sigma_{t;max} * b * \left( \frac{l}{\sigma_{c;max} + \sigma_{t;max}} * \sigma_{t;max} \right) = 0.5 * 4.90 * 270 * \left( \frac{5000}{6.08 + 4.90} * 4.90 \right) = 1476 kN$$

Applying the same procedure for the other walls and core segments, the following overview is obtained, see table 19. In this overview, also the difference in tensile force between adjacent core segments is shown. For example, $\Delta F_{t;-2}$ is the difference in tensile force between $F_{t;-2}$ and $F_{t;-1}$.

Table 18: Stresses on the distinct walls of the individual core segments

| Level | Wall number | $F_{c;max}$ [kN] | $F_{t;max}$ [kN] | $\Delta F_t$ [kN] |
|---|---|---|---|---|
| 2 | 1 | 230 | 0 | 0 |
| | 2/4 | 459 | 0.2 | 0.2 |
| | 3 | 230 | 0 | 0 |
| 1 | 1 | 437 | 5 | 5 |
| | 2/4 | 756 | 109 | 109 |
| | 3 | 467 | 35 | 35 |
| 0 | 1 | 757 | 82 | 77 |
| | 2/4 | 1439 | 483 | 374 |
| | 3 | 867 | 192 | 157 |
| -1 | 1 | 1084 | 180 | 98 |
| | 2/4 | 2154 | 871 | 388 |
| | 3 | 1273 | 368 | 176 |
| -2 | 1 | 1485 | 351 | 171 |
| | 2/4 | 3082 | 1476 | 605 |
| | 3 | 1788 | 654 | 286 |

For wall 2/4, the maximum acting stresses and the resulting forces on each segment are presented visually in figure 132.

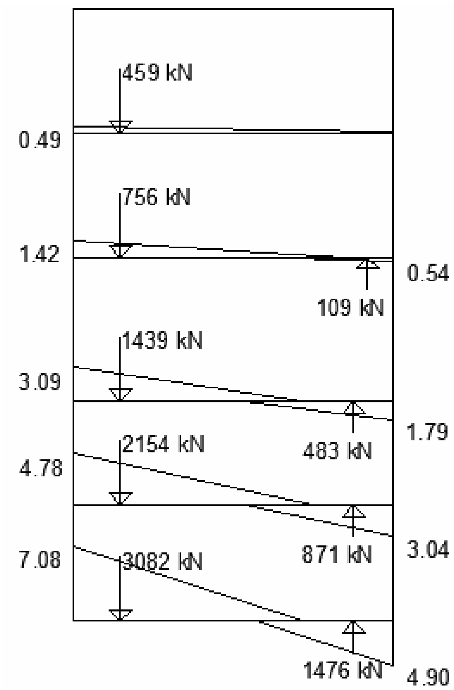



Figure 132: Maximum stresses and resulting forces on each segment (wall 2/4)

### 8.2.5 Anchorage by means of steel bars

When steel bars are at stake there a two options to anchor the segments:

§ Anchor all segments at the top of the structure
§ Anchor the segments individually

It is regarded beneficial to anchor each of the segments individually, since the total length of the anchorage bars can be reduced in this way. This solution is therefore accounted for in this thesis.

This implies, though, that recesses are to be incorporated in the core walls, to anchor the bars above each core segment. The bars their selves should protrude through ducts within the elements. To introduce the tensile forces into the walls, steel plates are applied which divide the tensile force over the required timber area. An example of the proposed solution is shown in figure 133.

Figure 133: Principle of the proposed solution to anchor the core walls (indicative)

### Required amount of steel bars

As elaborated on earlier, the anchorage bars have to transfer the difference in tensile force between two adjacent segments only.

As a first assumption, there is accounted for steel bars (Φ30, 6.8). The maximum tensile force these bars can transfer is:

$$F_{t;bar;max} = f_s * \frac{1}{4}\pi d^2 = 480 * \frac{1}{4}\pi * 30^2 \approx 340kN$$

Considering table 19, it is found that the maximum tensile force in wall 2 amounts 1476 kN, which implies that at least 5 bars are required. In the same way it is found that for both wall 1 and 3, at least 2 bars are required. It is now determined how these bars should be divided over the height of the walls.

§ Wall 2/4

- $\Delta F_{t;-2}$ = 605 kN à 2 bars required (capacity 680 kN)
  à remaining capacity 75 kN is not used to anchor above-lying elements

- $\Delta F_{t;-1}$ = 388 kN à 2 bars required (capacity 680 kN)
  à remaining capacity 292 kN à 1 protrudes through the above-lying element

- $\Delta F_{t;0}$ = 374 kN à 1 additional bar required next to the protruding bar (capacity 632 kN)
  à remaining capacity of 258 kN à 1 protrudes though the above-lying element

- $\Delta F_{t;1}$ = 109 kN à Protruding bar is sufficient (capacity 258 kN)
  à remaining capacity 149 kN à 1 protrudes through the above-lying element

- $\Delta F_{t;2}$ = 0.2 kN à Protruding bar is sufficient (capacity 149 kN)

Since the direction of the horizontal loads can differ, the tensile anchorage is applied at both ends of the wall element. The above is visually presented in figure 134.

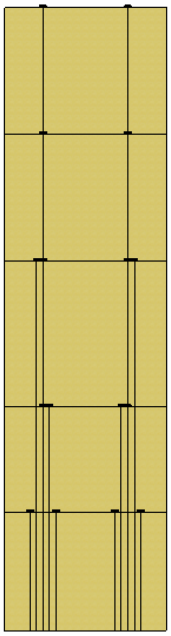


Figure 134: Anchorage of tensile bars over the height of the wall (wall 2/4)

§ Wall 1/3

As can be concluded from table 19, the forces in wall 3 become much larger than those in wall 1. This can be explained by the appearance of (local) openings in wall 3. To prevent indistinctness during erection, it is decided to implement an equal amount of anchors in wall 3 and wall 1. Wall number 3 is therefore governing.

- $\Delta F_{t;-2}$ = 286 kN à 1 bar required (capacity 340 kN).
  à remaining capacity 54 kN à 1 protrudes through the above-lying element.

- $\Delta F_{t;-1}$ = 176 kN à 1 bar required (capacity 340 kN), protruding bar available as well (cap. 54 kN).
  à remaining capacity 218 kN à 1 protrudes through the above-lying element.

- $\Delta F_{t;0}$ = 157 kN à Protruding bar is sufficient (capacity 218 kN).
  à remaining capacity of 61 kN à 1 protrudes though the above-lying element.

- $\Delta F_{t;1}$ = 35 kN à Protruding bar is sufficient (capacity 26 kN).

- $\Delta F_{t;2}$ = 0 kN à No bar required.

Since the direction of the horizontal loads can differ, the tensile anchorage is applied at both ends of the wall element. The above is visually presented in figure 135.

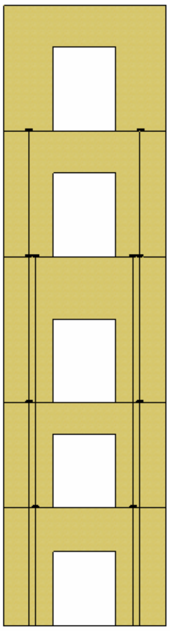


Figure 135: Anchorage of tensile bars over the height of the wall (wall 1/3)

### Dimensions of steel anchorage plate

As mentioned earlier this section, the bars are anchored in a recess within the wall element. To introduce the anchorage force in the timber element, a steel plate is incorporated. This plate transfers the tensile force that is present in the distinct bars directly to the wall element, where it acts as a compression force.

The minimal dimensions of this plate are directly related to the compression strength of the timber elements. Subsequently wall 2/4 and wall 1/3 are discussed.

### § Wall 2/4

Wall 2/4 is composed of Kerto-S elements only. This results in the following compressions strength:

$$f_{c;0;d} = f_{c;0;k} * \frac{k_{mod}^{(1)}}{\gamma_M} = 35 * \frac{0.9}{1.2} = 26.25 \ \text{N/mm}^2$$

$^{(1)}k_{mod}$ has been taken for short-term loading (wind), while the normal force follows directly from permanent loading (self-weight). To This has been neglected here.

This results in the following minimum dimensions for the steel plate:

$$A_{plate} = \frac{\Delta F_{t;max}}{f_{c;0;d}} = \frac{307 \cdot 10^3}{26.25} = 12000 \ \text{mm}^2$$

It is assumed that the acting force spreads equal in all directions, whereby the plates can be made square, resulting in dimensions of at least 110 by 110 mm$^2$. It is not investigated what thickness is required for these plates.

### § Wall 1/3

Wall 1/3 is composed of 2 layers Kerto-S and one layer Kerto-Q. As a result, the compression strength is somewhat reduced:

$$f_{c;0;d} = f_{c;0;k} * \frac{k_{mod}^{(1)}}{\gamma_M} = \frac{35 * 180 + 9 * 69}{249} * \frac{0.9}{1.2} = 20.85 \ \text{N/mm}^2$$

$^{(1)}k_{mod}$ has been taken for short-term loading (wind), while the normal force follows directly from permanent loading (self-weight). This has been neglected here.

This results in the following minimum dimensions for the steel plate:

$$A_{plate} = \frac{\Delta F_{t;max}}{f_{c;0;d}} = \frac{307 \cdot 10^3}{20.85} = 14750 \ \text{mm}^2$$

It is assumed that the acting force spreads equal in all directions, whereby the plates can be made square, resulting in dimensions of at least 125 by 125 $mm^2$. It is not investigated what thickness is required for these plates.

### Detailing of the anchorage system for wall 2

To provide the reader with an idea on the detailing of the anchorage system, additional information is provided on this subject.

As discussed in section 8.2.5, at the bottom of the wall 2/4, a total of (2 times) 5 tensile bars is implemented in the design. Two of these are anchored above the lowest floor element, while the other three continue through the above-lying element.

It is assumed here, that these bars should be located at a centre to centre distance of at least 4*d (=120 mm) to each other. As a result, the minimum distance in between the two most outer bars becomes 480 mm. When the end and edge distances are considered, it is assumed that these distances should be at least 1,5*d (= 45 mm). Incorporating plates of 120 by 120 $mm^2$, this requirement is satisfied.

The consequences of the above for the anchorage above the -2nd core segment and the -1st core segment is visually presented in figure 136 and figure 137 respectively.

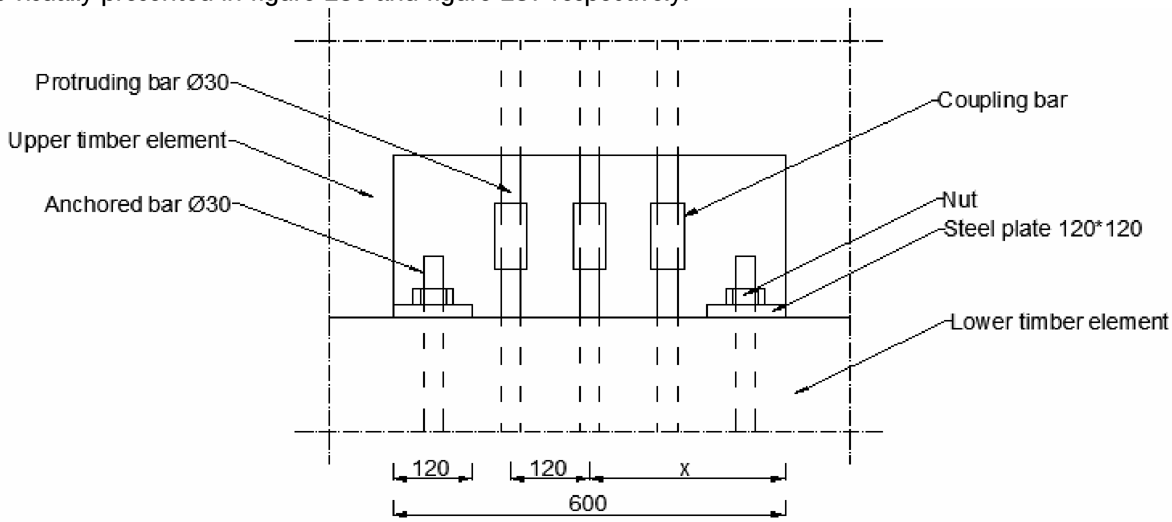



Figure 136: Proposed anchorage above the -2nd floor core segment

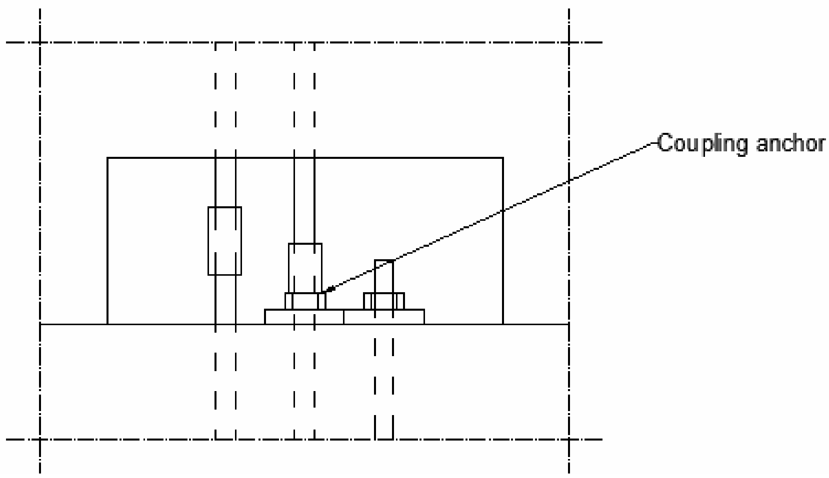



Figure 137: Proposed anchorage above the -1st floor core segment

### Check on the internal leverage arm

In section 8.1.3, it was stated that, as a general design assumption, the internal leverage arm over which the bending moment acts was taken 2/3*h. It is now checked whether this assumption holds.

At first, the minimum required area for the compression zone is determined. Making use of the maximum compression strength of the timber wall, the following can be found:

$$x = \frac{F_c}{0.5b * f_{c;0;d}} = \frac{3082 \cdot 10^3}{0.5 * 270 * 26.25} = 870\text{mm}$$

The point of action of the compression force, is assumed to be on a distance of 1/3 to the end of this area, being on a distance of 290 mm to the edge of the wall.

In the previous section, it was found that the centre to centre distance in between the outer tensile bars, should be at least 480 mm. For the steel plates, a width of 110 mm is required. This leads to a total length of about 600 mm, over which the tensile ties should be applied, see figure 136.

Due to the appearance of the compression zone next to the recess, the centre of the tensile ties can be found at a distance of about 1200 mm (870 + 600/2) to the end of the wall element.

As a result, the internal leverage arm becomes about 3510 mm (5000 – 1200 – 290), which is larger than the assumed leverage arm of 3333 mm (5000 * 2/3). The acting forces can therefore even be somewhat further reduced, whereby the analysis as performed above can be considered conservative.

### Conclusions and recommendations

§ Steel bars provide a feasible solution to transfer the tensile forces at the horizontal joints.

§ It is recommended to determine the vertical loading for the proposed design, and the division over the various floors. Due to the proposed grid, which differs from the reference stadium, the vertical load on the cores becomes somewhat smaller. As a result, the tensile stresses might become somewhat larger.

§ It has to be determined whether the thickness of the steel plate is acceptable.

§ The anchorage system should be protected against fire. This is achieved by enclosing the recesses after the bars have been anchored. The enclosure can be made from the material that is removed to create the recess.

## 8.2.6 Anchorage by means of glued-in rods

In the previous section, it has been investigated whether steel bars could be used to transfer the tensile from one element to another. It appeared that the proposed solution is feasible, but that the implementation requires additional attention.

In this section, an alternative solution is considered where use is made of glued-in rods. Since the goal is to provide insight in the possibilities, a simplified analysis is made.

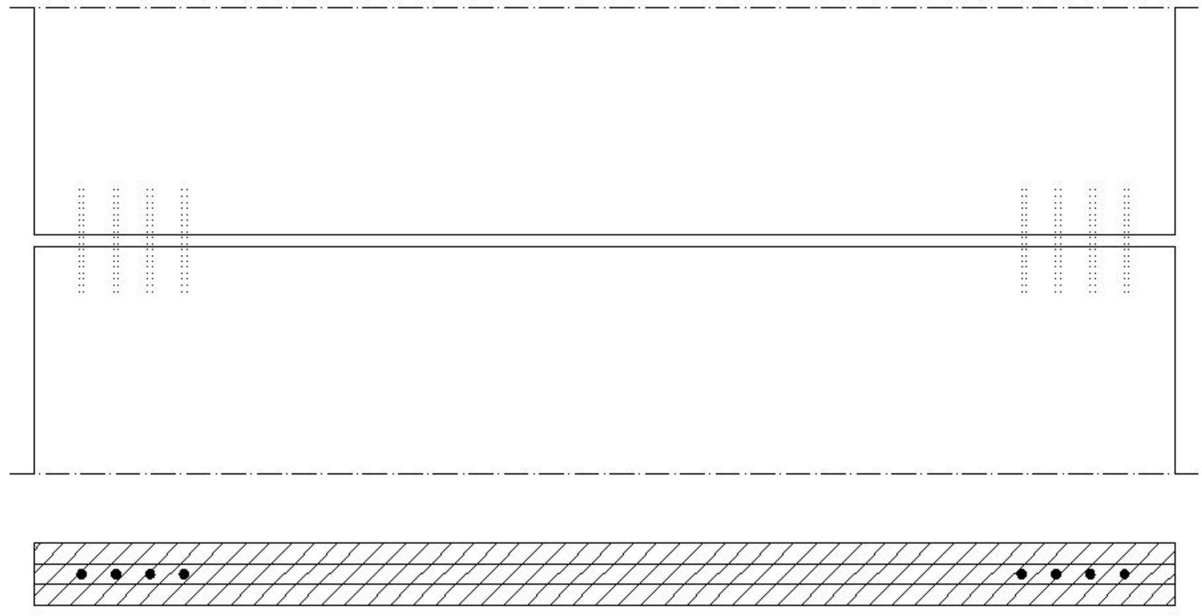


Figure 138: Principle of the proposed solution (indicative)

*Design assumptions*

§ Glued-in rods, M20, quality 8.8
§ Non-brittle glue is used (such as 2-component polyurethane)
§ $\rho_k$ = 510 kg/m$^3$ (Kerto-S)
§ $\gamma_M$ = 1.20 (LVL)
§ $k_{mod}$ = 0.90 (short-term loading)

In this section, the minimum required amount of glued-in rods is determined which is required to transfer the maximum tensile force. When glued-in rods are used, these rods should transfer the maximum tensile forces that act at these joints.

### Step 1: Loads

The rods are axially loaded in this situation, where they are aligned parallel to the grain. The maximum compression force is assumed to be taken by the material itself.

The governing loads follow directly from 8.2.4 and are shown in table 19.

Table 19: Tensile forces acting on the distinct walls of the individual core segments

| Level | Wall number | $F_{t;max}$ [kN] |
|---|---|---|
| 2 | 1 | 0 |
| | 2/4 | 0.2 |
| | 3 | 0 |
| 1 | 1 | 5 |
| | 2/4 | 109 |
| | 3 | 35 |
| 0 | 1 | 82 |
| | 2/4 | 483 |
| | 3 | 192 |

| | | |
|---|---|---|
| -1 | 1 | 180 |
| | 2/4 | 871 |
| | 3 | 368 |
| -2 | 1 | 351 |
| | 2/4 | 1476 |
| | 3 | 654 |

Step 2: Load-carrying capacity

Ribberholt (1988) suggests, that for axially loaded rods, the axial capacity is determined as:

$$R_{ax;k} = f_{ws} * \rho_k * d * \sqrt{l_g}$$

in which:

$f_{ws}$ = 0.65 (strength parameter for ductile glues)

$l_g$ = the glued in length [mm]

As a first assumption, $l_g$ is taken 200 mm.

Accounting for these values, it is found that:

$$R_{ax;k} = 0.65 * 510 * 20 * \sqrt{200} = 93.76 kN$$

and

$$R_{ax;d} = R_{ax;k} * \frac{k_{mod}}{\gamma_M} = 93.76 * \frac{0.90}{1.2} = 70.32 kN$$

Step 3: Required amount of glued-in rods

Accounting for the acting loads and the load-carrying capacity, the required amount of glued-in rods is determined. For the anchorage at the foundation, the following amount of glued-in rods is required:

$$n_{required} = \frac{F_{t;d}}{R_{ax;d}} = \frac{1476}{70.32} = 21 \text{ glued-in rods}$$

For the remaining walls and segments the same procedure is applied resulting in the following overview, see table 20.

Table 20: Required amount of glued-in rods to transfer the acting tensile forces

| Level | Wall number | $F_{t;max}$ [kN] | n |
|---|---|---|---|
| 2 | 1 | 0 | 0 |
| | 2/4 | 0.2 | 1 |
| | 3 | 0 | 0 |
| 1 | 1 | 5 | 1 |
| | 2/4 | 109 | 2 |
| | 3 | 35 | 1 |
| 0 | 1 | 82 | 2 |
| | 2/4 | 483 | 7 |
| | 3 | 192 | 3 |
| -1 | 1 | 180 | 3 |
| | 2/4 | 871 | 13 |
| | 3 | 368 | 6 |
| -2 | 1 | 351 | 6 |
| | 2/4 | 1476 | 21 |
| | 3 | 654 | 10 |

For walls 2 and 4 this amount of rods is required at both ends of the element, since the governing load can act from both sides.

The maximum amount of glued-in rods that has to be accounted for at a single wall element is therefore (2 times) 21. With a required spacing of only 2d (= 40 mm) it is concluded that it is no problem to implement such an amount in the core walls.

Conclusion

§ Glued-in rods provide an acceptable solution to transfer the acting tensile forces at the joints

### 8.2.7 Conclusions and recommendations

§ Both steel bars and glued-in rods provide an acceptable solution to transfer the acting tensile stresses at the horizontal joints

## 8.3 Transfer of shear forces

When the core is divided in segments, the shear forces are not automatically transferred at the horizontal joints. It is therefore investigated what solutions can provide this shear transfer.

In this section three solutions are discussed:

§ a solution with connector plates
§ a connection by means of glued-in rods
§ a so-called toothed-connection

The goal of this section is to provide insight in the possibilities of the proposed solutions, rather than to provide a perfectly working solution.

### 8.3.1 Connector plates

Analysis

This solution accounts for timber plates which are attached to the core walls at those locations where two segments meet each other. As a first assumption, these plates are composed from the material Kerto-Q. Hereby, these plates are made from the same material as the core walls. The reason to choose for Kerto-Q instead of Kerto-S is the slightly higher shear strength.

From both an aesthetical and a functional point of view, it is not desirable to attach a connector plate completely outside the cross-section of the core walls. It is therefore decided to implement the connector plate within the core walls, thereby reducing the cross-section of the core walls locally, see figure 139.

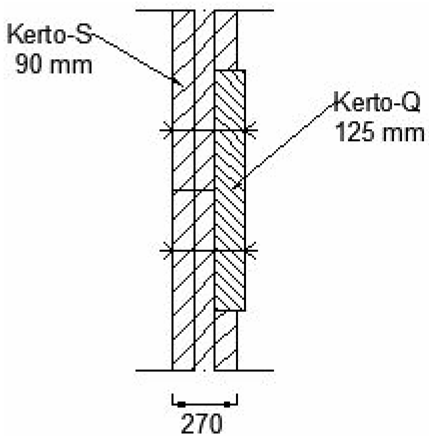



Figure 139: Location of connector plate within the core wall (wall 2/4)

At first, it is determined what the minimum width of this plate should be to be able to transfer the maximum acting shear force. Therefore, the shear strength of Kerto-Q is determined:

$$f_{v;d} = f_{v;d} * \frac{k_{mod}}{\gamma_M} = 4.5 * \frac{0.9}{1.2} = 3.375 N/mm^2$$

It appears that the required thickness of the connector plate is 125 mm $\left(\frac{3*1372\cdot 10^3}{2*5000*3.375}\right)$. Since it is not desirable to reduce the layer thickness locally, while having core walls composed of three layers of 90 mm, it is decided to implement the connector plate in the outer layer (90 mm) only. As a result, the connector plate comes out of the core for 35 mm, see figure 139.

With a total wall thickness of 270 mm, the remaining thickness of the core wall (Kerto-S) is 180 mm, which is sufficient to resist the acting shear force.

Fastener capacity

The connection is made by means of 250 mm long screws, having a diameter of 20 mm and an ultimate strength $f_{u;k}$ of 400 N/mm$^2$.

Since it is highly beneficial to stress the timber parallel to its grain, it is decided to implement a connector plate having its grain parallel to the direction of the shear force.

For the Kerto-S core walls, it is found that:

$$f_{h;90;k;1} = \frac{0.082*(1-0.01*d)*\rho_k}{k_{90}\sin^2\alpha + \cos^2\alpha} = \frac{0.082*(1-0,01*20)*510}{(1.30+0.015*20)\sin^2 90 + \cos^2 90} = 20.91 N/mm^2 \text{ and}$$

For the Kerto-Q connector plate it is found that:

$$f_{h;0;k;2} = \frac{0.082*(1-0.01*d)*\rho_k}{k_{90}\sin^2\alpha + \cos^2\alpha} = \frac{0.082*(1-0,01*20)*510}{(1.30+0.015*20)\sin^2 0 + \cos^2 0} = 33.46 N/mm^2 \text{ and}$$

Making use of Johnson's equations, the following is found:

$$F_{v;Rk} = \min[a = 83.64; b = 52.28; c = 27.92; d = 28.86; e = 22.21; f = 20.72] = 20.72 kN/nail$$

This results in a capacity of the fasteners of:

$$F_{v;d} = F_{v;Rk} * \frac{k_{mod}}{\gamma_M} = 20.72 * \frac{0.9}{1.2} = 15.54 N/mm^2$$

### Required amount of fasteners

Considering the loads as mentioned in table 15 (section 8.2.2), the governing shear force appears to be 2743 kN. Since two walls are present (2 and 4), the acting shear force at the governing joint is about 1372 kN.

This results in a minimum amount of 89 screws, located at a centre to centre distance of at least 100 mm (5d). This results in a required width of 8900 mm, where only 5000 mm is available. It is therefore decided to divide the amount of screws over 5 rows, spaced at a distance of 100 mm.

To obtain a effective number of at least 89 screws, five rows of 33 screws should be accounted for $\left(5*33^{0.9}*\sqrt[4]{\frac{100}{13*20}} = 91\right)$ just above and underneath the joint. This results in a total of 330 screws, see figure 140.

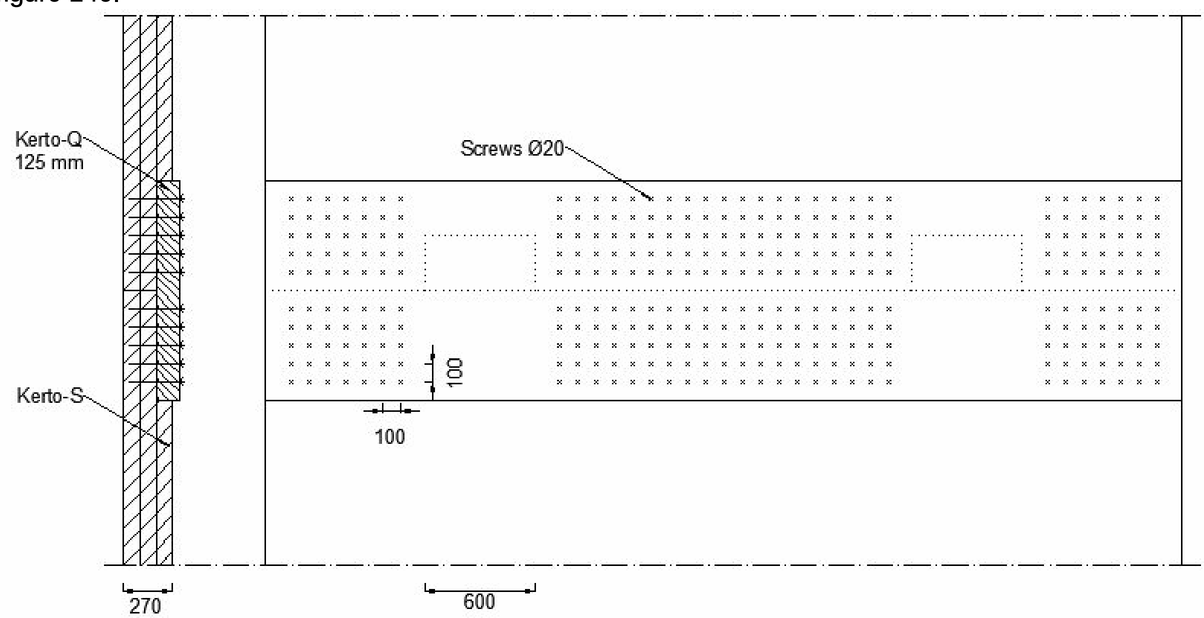



Figure 140: Proposed arrangement of screws to transfer the maximum shear force

Conclusion

§ 2 times 5 rows of 33 screws are sufficient to transfer the maximum acting shear force at wall 2, when a Kerto-Q plate is applied.

## 8.3.2 Glued-in rods

In this section, it is investigated how many glued-in rods are required to transfer these between the elements. The rods are laterally loaded in this situation, where they are aligned parallel to the grain.

*Design assumptions*

§ Glued-in rods, M20, quality 8.8

§ Non-brittle glue is used (such as 2-component polyurethane)

§ $\rho_k$ = 510 kg/m$^3$ (Kerto-S)

§ $\gamma_M$ = 1.20 (LVL)

§ $k_{mod}$ = 0.90 (short-term loading)

As a first step, the loads are determined.

Step 1: Loads

The governing loads follow directly from 8.2.2 and are shown in table 21.

Table 21: Shear forces acting at wall 2/4 of the individual core segments

| Level | $V_d$ [kN] |
|---|---|
| 2 | 132 |
| 1 | 424 |
| 0 | 706 |
| -1 | 992 |
| -2 | 1372 |

Step 2: Load-carrying capacity

Ribberholt (1988) suggests that for laterally loaded rods, glued in parallel to the grain, the capacity is determined as:

$$R_{V;k} = \left( \sqrt{e^2 + \frac{2M_{y;k}}{d * f_h}} - e \right) d * f_h$$

in which:

$f_{es}$ = $(0.0023 + 0.75d^{-1.5})\rho_k = (0.0023 + 0.75 * 20^{-1.5})510 = 5.45 N/mm^2$

$M_{y;k}$ = $0.3 f_{u;k} * d^{2.6} = 0.3 * 640 * 20^{2.6} = 0.463 kNm$

e = the distance from where the loads act to the material edge

As a first assumption, e is taken $0.5 * l_g$ = 100 mm.

Accounting for these values, it is found that:

$$R_{V;k} = \left( \sqrt{100^2 + \frac{2 * 0.463 \cdot 10^6}{20 * 5.45}} - 100 \right) 20 * 5.45 = 3.92 kN$$

and

$$R_{V;d} = R_{V;k} * \frac{k_{mod}}{\gamma_M} = 3.92 * \frac{0.90}{1.2} = 2.94 kN$$

Step 3: Required amount of glued-in rods

Accounting for the acting loads and the load-carrying capacity, the required amount of glued-in rods is determined. For the anchorage at the foundation, the following amount of glued-in rods is required:

$$n_{required} = \frac{V_d}{R_{V;d}} = \frac{1372}{2.94} = 467 \text{ glued-in rods}$$

For the remaining walls and segments, the same procedure is applied, resulting in the following overview:

Table 22: Required amount of glued-in rods in wall 2/4 to transfer the acting shear forces

| Level | $V_d$ [kN] | n |
|---|---|---|

| | | |
|---|---|---|
| 2 | 132 | 45 |
| 1 | 424 | 145 |
| 0 | 706 | 241 |
| -1 | 992 | 338 |
| -2 | 1372 | 467 |

Conclusion

§ This solution does not provide a feasible solution since a huge amount of glued-in rods is required.

### 8.3.3 Toothed connection

When a toothed connection is concerned, the individual walls of the core segments are provided with notches which in turn 'hook' into each other, see figure 141.

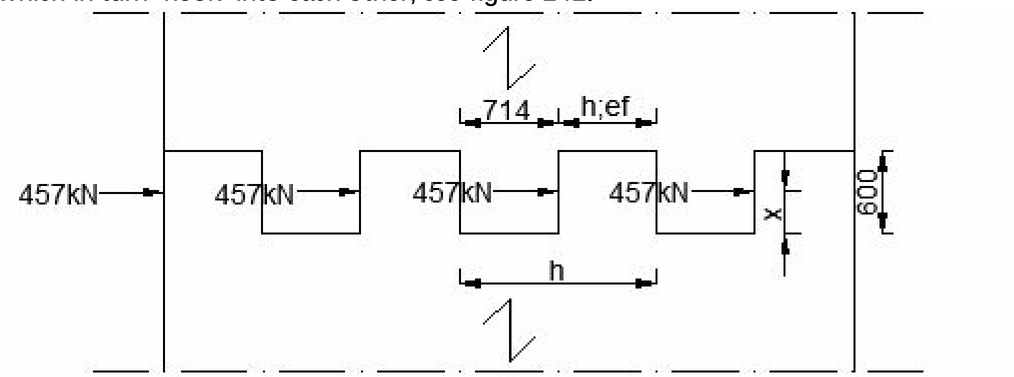



Figure 141: Schematization of a toothed connection (wall 2/4)

The focus is on walls 2 and 4, since at these walls the maximum shear force acts. Again, the maximum acting shear force at these walls is 1372 kN.

As a first assumption, the amount of teeth is taken three, whereby the maximum shear force that acts on each pair of teeth is about 457 kN.

Since there is no guidance to design such a connection, use is made of the theory of a more or less comparable system, being the theory on notched beams as presented in the Eurocode. [ 20 ]

For such a beam, it is required that $\tau_d \leq k_v * f_{v;d}$ , in which $k_v$ should be taken as:

$$k_v = \frac{k_n}{\sqrt{h}\left(\sqrt{\alpha(1-\alpha)} + 0.8\frac{x}{h}\sqrt{\frac{1}{\alpha} - \alpha^2}\right)} \text{ (LVL: } k_n = 4.5)$$

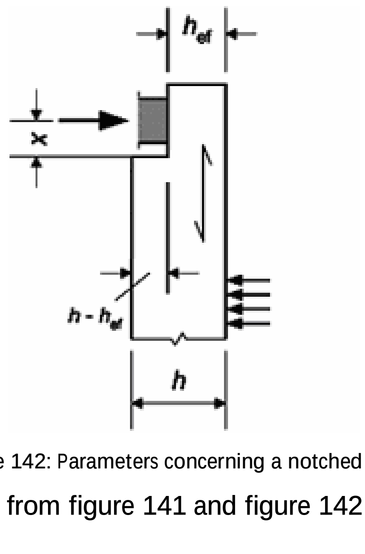



Figure 142: Parameters concerning a notched beam

The meaning of the parameters in this formula can be extracted from figure 141 and figure 142.

Verification of the acting shear stress

§ Shear stress

The length of the walls is 5000 mm, which leads to a tooth-length of 714 mm, see figure 141. Accounting for this length, the shear stress that should be transferred by the toothed-connection becomes:

$$\tau_{toothed} = \frac{3}{2}\frac{V_{tooth}}{b * l_{tooth}} = \frac{3}{2}\frac{457 \cdot 10^3}{270 * 714} = 3.055 \ N/mm^2$$

§ Shear strength

Making use of the theory on notched beams, see figure 142, the shear strength of the toothed connection is determined. Accounting for the proposed schematization, the following is stated:

- The shear force acts at the middle of the tooth, x thereby being $0.5h_{tooth}$ = 300 mm
- $h_{ef}$ is equal to $l_{teeth}$ = 714 mm
- All teeth have an equal length, whereby h is 2 * $l_{teeth}$ = 1428 mm (and thus α becomes 0.5 $\left(= h_{ef}/h\right)$)

Accounting for these values, it is found that $k_v$ = 0.16.

The maximum shear stress that the Kerto-S elements can resist is:

$$f_{v;d} = f_{v;d} * \frac{k_{mod}}{\gamma_M} = 4.1 * \frac{0.9}{1.2} = 3.075 N/mm^2$$

Combining these values, the maximum shear strength is determined:

$$\tau_d \leq k_v * f_{v;d} \rightarrow 3.02 N/mm^2 > 0.16 * 3.075 = 0.492 N/mm^2$$

Conclusion

§ Since the shear strength is smaller than the shear stress, it is concluded that the proposed solution is not feasible.

## 8.3.4 Conclusions and recommendations

§ The most beneficial solution to transfer the shear forces at the horizontal joints is obtained by applying a Kerto-Q connector plate over these joints.

§ A substantial percentage of the acting shear force is already transferred due to friction between the timber segments. The magnitude of this reduction can even be in the order of 50-60% of the acting compression force. It is recommended to investigate this subject more into detail.

## 8.4 Stiffness of the segmented core

Accounting for the various solutions that have been mentioned on the transfer of the loads that act at the horizontal joints, it is investigated what consequences the implementation of these solutions hold for the stiffness of the core as a whole.

The focus is on two different solutions here:
§ The combination of steel bars and connector plates.
§ The combination of glued-in rods and connector plates.

From the previous sections it appeared that these solutions holds the most potential.

### 8.4.1 Analysis

In the preceding part of this chapter it appeared that tensile anchorage is required when the core is divided in segments. As a result, for the part of the core that is in tension, the tensile anchorage provides this stiffness at the joints instead of the timber itself. In this section, it is investigated what consequences this holds for the stiffness of the core.

Since the serviceability limit state is concerned, the governing load combination in EW-direction appears to be SLS 1b, see table 23. These values follow directly from C.1.3.

Table 23: Governing load combination

| Situation: | N [kN] | $q_w$ [kN/m] | $F_{hor;2nd}$ [kN] | $F_{hor;0}$ [kN] | $F_{hor;-1st}$ [kN] |
|---|---|---|---|---|---|
| SLS 1b: | 3245 | 81 | 39 | 32 | 32 |

With reference to C.2.5, it is stated that the core as proposed in chapter 4 complies largely with the requirements on stiffness. Neglecting the horizontal grandstand loads, it was found that the maximum deflection becomes:

$$w_{max} = \frac{q * l^4}{8EI_{ef;y;ser}} = \frac{81 * 19220^4}{8 * 17.83 \cdot 10^{16}} = 7.75mm$$

The maximum acceptable deflection was limited to 25.6 mm, where has already been accounted for the rotational stiffness of the foundation. The minimally required bending stiffness EI thereby becomes 5.398 * $10^{16}$ $Nmm^2$.

It is now investigated whether such stiffness can be obtained while accounting for the tensile anchorage. To simplify the calculation, the influence of the connector plates is neglected. Since these hold stiffness in vertical direction as well, this analysis is considered conservative.

### 8.4.2 Stiffness while accounting for steel bars

When the solution with steel anchorage bars is concerned, these bars protrude from the foundation to the location where they are anchored to a segment. It is therefore stated that the stiffness of such a core follows from the combined system of the bars in the tensile zone and the timber that is in compression.

Before determining the stiffness of such a system, the design assumptions are mentioned.

*Design assumptions*
§ The connection efficiency factor at wall 1 is $\gamma_{1;ser}$ = 0.644 (see chapter 4)
§ The connection efficiency factor at wall 3 is $\gamma_{3;ser}$ = 0.746 (see chapter 4)
§ Wall 3 is considered in tension and wall 1 is in compression (this is the governing situation)
§ Only 1 linear meter of wall 2 and wall 4 is in compression

The schematization that is used is presented in figure 143.

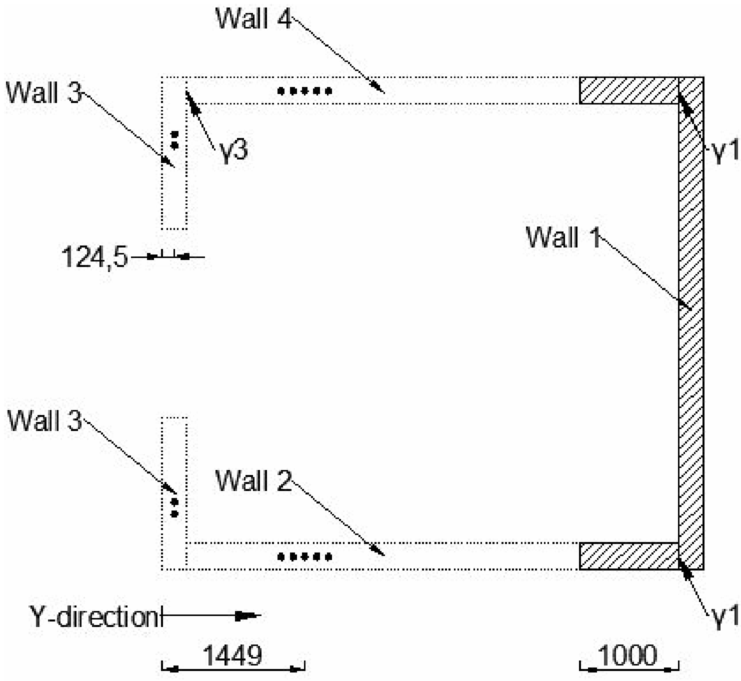



Figure 143: Schematization used to determine the stiffness of the core

### Sectional properties

Considering figure 143, the sectional properties of the compression zone are determined. For wall 2 and 4, the values of table 24 apply.

Table 24: Sectional properties of the timber in the compression zone

| Wall | $\gamma_i$ | $y_{i;c}$ [mm] | E [N/mm$^2$] | A [mm$^2$] | I [mm$^4$] |
|---|---|---|---|---|---|
| 2/4 | 1.0 | 4749 | 13800 | 270000 | 22.5 $*10^9$ |
| 1 | 0.644 | 5373.5 | 10641 | 622500 | 3.22 $*10^9$ |

When the tensile zone is at stake, the stiffness depends on the amount of bars available. These values can be extracted from section 8.2.5. Accounting for the design parameters underneath, the following sectional properties are found for the tensile zone, see table 25.

$E$ = 210000 N/mm$^2$
$\gamma_{2;ser}$ = 1.0
$\gamma_{3;ser}$ = 0.746
$y_{2;c}$ = 1499 mm
$y_{3;c}$ = 124.5 mm

Table 25: Sectional properties of the bars in the tensile zone

| Level | Wall | $n_{bars}$ | A [mm$^2$] | I [mm$^4$] |
|---|---|---|---|---|
| *-1$^{st}$ floor* | 2/4 | 5 | 3534 | 198804 |
| | 3 | 2 | 1414 | 79522 |
| *Ground floor* | 2/4 | 3 | 2121 | 119282 |
| | 3 | 2 | 1414 | 79522 |
| *1$^{st}$ floor* | 2/4 | 2 | 1414 | 79522 |
| | 3 | 2 | 707 | 39761 |
| *2$^{nd}$ floor* | 2/4 | 1 | 707 | 39761 |
| | 3 | 1 | 707 | 39761 |
| *3$^{rd}$ floor* | 2/4 | 1 | 707 | 39761 |
| | 3 | 0 | 0 | 0 |

### Centre of gravity of the combined system

Making use of the values presented in table 24 and table 25, the centre of gravity of the combined system is determined making use of the following formula:

$$y_{c;\text{floor } i} = \frac{\sum \gamma EA_i * y_{c;i}}{\sum \gamma EA_i}$$

This results in:

$y_{c;-1st\ floor}$ = 4659 mm
$y_{c;ground\ floor}$ = 4769 mm
$y_{c;1st\ floor}$ = 4827 mm
$y_{c;2nd\ floor}$ = 4951 mm
$y_{c;3rd\ floor}$ = 5016 mm

As a start assumption, it was stated that walls 2 and 4 were in compression for at least 1 linear meter. The compression zone was therefore assumed to start at y = 4249 mm (5000+249-1000).

Considering the centre of gravity of the system as a whole, it appears that the compression zone is even smaller than that (see figure 144). By performing an iterative calculation, the maximum height of the compression zone is determined. This leads to the following results, see table 26.

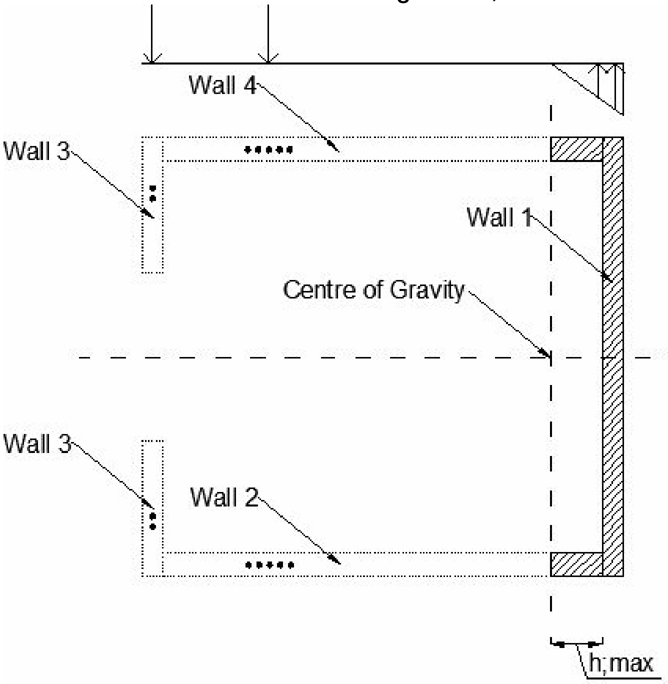



Figure 144: Maximum height of the compression zone in wall 2/4

Table 26: Maximum height of the compression zone and the accompanying sectional properties

| Wall | $h_{compression\ zone}$ | A [mm$^2$] | I [mm$^4$] |
|---|---|---|---|
| *-1$^{st}$ floor* | | | |
| 2/4 | 605 | 163350 | 4.98 *10$^9$ |
| *Ground floor* | | | |
| 2/4 | 470 | 126900 | 2.24 *10$^9$ |
| *1$^{st}$ floor* | | | |
| 2/4 | 395 | 106650 | 1.39 *10$^9$ |
| *2$^{nd}$ floor* | | | |
| 2/4 | 180 | 48600 | 0.13 *10$^9$ |
| *3$^{rd}$ floor* | | | |
| 2/4 | 10 | 2700 | 22500 |

Accounting for these values, the centre of gravity of the combined system is determined again. This resulted in the following values:

$y_{c;-1st\ floor}$ = 4640 mm
$y_{c;ground\ floor}$ = 4774 mm
$y_{c;1st\ floor}$ = 4855 mm
$y_{c;2nd\ floor}$ = 5069 mm
$y_{c;3rd\ floor}$ = 5237 mm

### Stiffness of the core

By applying Steiner's rule, while accounting for the stiffness of the compressed timber and the bars in tension, the stiffness of the core is determined at each of the floors. This results in:

| Level | $EI_{EW}$ [$Nmm^2$] |
|---|---|
| -1$^{st}$ floor | $1.45 * 10^{16}$ |
| ground floor | $1.13 * 10^{16}$ |
| 1$^{st}$ floor | $0.96 * 10^{16}$ |
| 2$^{nd}$ floor | $0.51 * 10^{16}$ |
| 3$^{rd}$ floor | $0.22 * 10^{16}$ |

To obtain an idea on the deflections of the core, these values are incorporated in a Technosoft model. This results in the following figure, see figure 145.

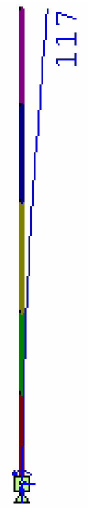



Figure 145: Deflections of the proposed core, accounting for steel bars

As can be seen in figure 145, the horizontal deflection exceeds the maximum acceptable value (being 25.6 mm, see section 8.4.1) by far.

### Conclusions and recommendations

§ The stiffness of the segmented core is insufficient when there is accounted for steel bars only.

§ Since the connector plates hold stiffness as well, the obtained results should be considered highly conservative. It is recommended to incorporate the effect of these plates in detailed design stages.

§ It is recommended to investigate whether the results can be improved by pre-stressing the bars. A first investigation is carried out in the upcoming section.

### 8.4.3 Pre-stressing the anchorage bars

From the previous section it appeared that the stiffness of the segmented core, while accounting for anchorage bars, was insufficient. Therefore, improvements are to be made on this design.

Probably the best solution could be obtained by reducing the acting tensile stresses. When the core is completely in compression, the stiffness is provided by the timber itself. As a result, the deflections are equal to those found in the chapter 4.

A solution that is generally applied for concrete structures in this case is the implementation of pre-stressing bars. Hereby, an additional compression force is introduced on the structure, whereby the resulting tensile stresses are reduced. In this section, a first investigation is carried out on this subject.

#### Analysis

To prevent tensile stresses to act on the core structure, a compression stress that equals the maximum acting tensile stress (4.90 N/mm$^2$) should be applied. Since the wind can act from both directions, the pre-stressing tendons should be applied on both sides of the core's cross-section. As a result, the side that is under compression by the acting wind load and the vertical loads, is compressed even further due to the applied pre-stressing force.

Having a maximum compression stress of 7.08 N/mm$^2$ caused by the external loads, and a pre-stressing stress of 4.90 N/mm$^2$, the resulting compression stress on the core will be 11.98 N/mm$^2$. From section 4.4 it appeared that the core walls are able to resist such a stress.

Due to the absence of bond between the timber and the steel bars, post-tensioned steel tendons are introduced. These tendons run through ducts, as for the anchorage bar system proposed in the previous section. The main benefits of such a solution are the rapid erection and the economical connections.

In recent years, research has been carried out in Australia on the feasibility of pre-stressed timber structures The research was mainly focused on experimental testing of structural elements within prototype buildings. Another interesting reference on the design of post-tensioned timber structures is a thesis by Sarti [ 21 ].

When pre-stressed structures are at stake, there should be accounted for a loss of pre-stress due to long-term effects. Due to the lack of design procedures for pre-stressed timber structures, this loss is taken equal to the creep percentage that applies to the core wall material. For LVL, in climate class 1, this value is 0.6.

This value is considered a maximum, and therefore conservative, value. It should be investigated whether this statement holds. It goes beyond the goal of this thesis to carry out such an investigation.

#### Design calculation

With reference to section 8.2.4, it is stated that the maximum acting tensile force in the Ultimate Limit State is 1476 kN for wall 2/4. Accounting for a 60% loss due to long-term effects, the required pre-stressing force is 2360 kN.

Applying unbonded strands, FeP 1860, d = 15.7 mm, with a cross-sectional area of 150 mm$^2$, it is found that the maximum pre-stressing force per tendon is:

$$F_{pu} = \frac{f_{pu}}{\gamma_M} * A_p = \frac{1860}{1.1} * 150 = 253.6\text{kN}$$

The maximum pre-stressing force just after pre-stressing is limited to: $F_{pi}$ = 0.8 * $F_{pu}$ = 202.9 kN.

Accounting for these values, it is found that at least 12 $\left( = \frac{F_{t;max}}{F_{pi}} = \frac{2360}{202.9} \right)$ of these wires are required at both sides of walls 2 and 4.

### Conclusions and recommendations

- Unbonded post-tensioned tendons are required to pre-stress timber structures
- Pre-stressed timber promises to be a feasible solution to provide the core with sufficient stiffness.

- Additional research is required on pre-stressing timber, for example, to determine the percentage of losses due to long-term effects.

## 8.4.4 Stiffness while accounting for glued-in rods

When glued-in rods are incorporated in the design, one should note that these do anchor the segments near the horizontal joints only. Accounting for the proposed solution as discussed in 8.2.6, the anchorage length at both sides of the horizontal joints is only 200 mm. As a consequence, the stiffness of the core is only reduced over a small height near these joints.
In this section it is investigated whether the results for this solution comply with the requirements. It is considered conservative to assume that the stiffness of the core is reduced over the whole length of the rods, i.e. over a length of 400 mm near the joints.

To simplify the design procedure, the influence of the connector plate is neglected again.

### Stiffness determination

To determine the stiffness of this solution, the same procedure as used in the preceding section is carried out. Without elaborating on all design calculations, the most important parameters are shown here.

Accounting for the required amount of glued-in rods, as determined in 8.2.6, the following sectional properties are found, see table 27.

Table 27: Sectional properties of the glued-in rods

| Location | Wall number | n | $A_{tot}$ [$mm^2$] | $I_{tot}$ [$mm^4$] |
|---|---|---|---|---|
| Joint -2nd floor | 3 | 10 | 1341 | 78539 |
| | 2/4 | 21 | 6597 | 164933 |
| Joint -1st floor | 3 | 6 | 1885 | 47123 |
| | 2/4 | 13 | 4084 | 102101 |
| Joint ground floor | 3 | 3 | 942 | 23561 |
| | 2/4 | 7 | 2199 | 54977 |
| Joint 1st floor | 3 | 1 | 314 | 7853 |
| | 2/4 | 2 | 628 | 15707 |
| Joint 2nd floor | 3 | 0 | 0 | 0 |
| | 2/4 | 1 | 314 | 7853 |

In an iterative way, the centre of gravity of the combined system is determined again. Making use of this value it is determined which part of walls 2 and 4 is in compression. Making use of these values, the second moment of area is determined for the wall in compression, see table 28.

Table 28: Maximum height of the compression zone and the accompanying sectional properties

| Wall | $h_{compression\ zone}$ | A [$mm^2$] | I [$mm^4$] |
|---|---|---|---|
| Joint -2nd floor | | | |
| 2/4 | 1025 | 276750 | $24.23 \cdot 10^9$ |
| 1 | 249 | 1245000 | $6.43 \cdot 10^9$ |
| Joint -1st floor | | | |
| 2/4 | 760 | 205200 | $9.87 \cdot 10^9$ |
| 1 | 249 | 1245000 | $6.43 \cdot 10^9$ |
| Joint ground floor | | | |
| 2/4 | 480 | 129600 | $2.49 \cdot 10^9$ |
| 1 | 249 | 1245000 | $6.43 \cdot 10^9$ |
| Joint 1st floor | | | |
| 2/4 | 240 | 64800 | $0.31 \cdot 10^9$ |
| 1 | 249 | 1245000 | $6.43 \cdot 10^9$ |
| Joint 2nd floor | | | |
| 2/4 | 0 | 0 | 0 |
| 1 | 249 | 1245000 | $6.43 \cdot 10^9$ |

Accounting for the stiffness moduli of the timber and the glued-in bolts, the stiffness of the combined system is determined at the distinct joints. The stiffness distribution over the height of the core is presented in table 29.

Table 29: Stiffness distribution over the height of the core

| Location | h [mm] | $EI_{EW}$ [$Nmm^2$] |
|---|---|---|
| Segment 2 | 3700 | $17.83 * 10^{16}$ |
| Joint 2nd floor | 400 | $3.27 * 10^{16}$ |
| Segment 1 | 3500 | $17.83 * 10^{16}$ |
| Joint 1st floor | 400 | $2.17 * 10^{16}$ |
| Segment 0 | 4120 | $17.83 * 10^{16}$ |
| Joint ground floor | 400 | $1.25 * 10^{16}$ |
| Segment -1 | 2850 | $17.83 * 10^{16}$ |
| Joint -1st floor | 400 | $0.42 * 10^{16}$ |
| Segment -2 | 3250 | $17.83 * 10^{16}$ |
| Joint -2nd floor | 200 | $0.14 * 10^{16}$ |

To obtain an idea on the deflections of the core, these values are incorporated in a Technosoft model. This results in the following figure, see figure 146.

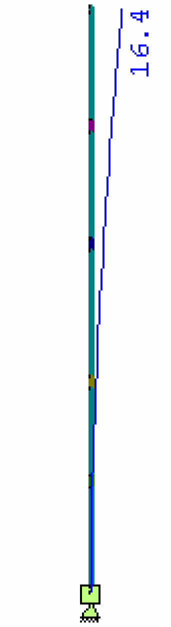



Figure 146: Deflections of the proposed core, accounting for glued-in bolts

With reference to section 8.4.1 it is stated that the stiffness of the proposed core complies with the requirements.

Conclusions and recommendations

§ The segmented core complies with the requirements on stiffness, when glued-in bolts are implemented at the horizontal joints.

§ Since the connector plates hold stiffness as well, the obtained results should be considered highly conservative. It is recommended to incorporate the effect of these plates in detailed design stages.

### 8.4.5 Conclusions

§ To provide the segmented core with sufficient stiffness, the implementation of either glued-in rods or pre-stressing tendons offers a feasible solution.

## 8.5 Conclusions and recommendations

### 8.5.1 Conclusions

§ Dividing the core in storey-high segments requires a lot of additional attention, but does not seem to limit the feasibility of the solution.

§ Both steel bars and glued-in rods provide an acceptable solution to transfer the acting tensile stresses at the horizontal joints

§ The most beneficial solution to transfer the shear forces at the horizontal joints is obtained by applying a connector plate over these joints. Despite that, a considerably large amount of fasteners is required.

§ To provide the segmented core with sufficient stiffness, the implementation of either glued-in rods or pre-stressing tendons offers a feasible solution.

### 8.5.2 Recommendations

§ Up till now, the solutions concerning shear transfer and tensile anchorage are considered independently. Since the proposed solutions interfere with each other, it is recommended to perform additional research on this subject.

§ It is recommended to investigate the shear transfer due to friction between the elements in detail.

§ The influence of the connector plates on the stiffness of the core as a whole should be taken into consideration.

§ It is recommended to perform additional research on pre-stressing timber structures.

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# 9 Core wall connections

## 9.1 Introduction

In this chapter is elaborated on the connections that should be accounted for when designing a timber core with LVL elements. Since these connections are not infinitesimally stiff, they influence the overall stiffness and thereby the load distribution of the core.

In the preliminary design phase of this thesis, a highly simplified design has been made in which the distinct connections are not considered in detail. To determine whether the proposed timber core system is feasible, though, such a detailed analysis should be carried out.

The focus is on two kinds of connections:

§ The intermediate connections which connect the Kerto elements to form a single wall (section 9.2)
§ The corner connections which connect the distinct walls to become mechanically jointed (section 9.3)

To provide the reader with understanding of the actual situation, the preliminary design of the core system is recalled first. Thereafter, the governing design criteria are being mentioned.

### 9.1.1 Preliminary design

Considering the design proposal as elaborated on in the section Proposal for the Detailed design, each core is composed of 4 walls which are interconnected at their edges. As a first assumption, the fasteners of these corner connections were taken dowels (Φ24 mm), at a centre-to-centre distance of 150 mm.

Furthermore, it appeared that due to the limited width of the Kerto-LVL elements, the core walls should be composed of multiple elements which are interconnected, see figure 147. In preceding design calculations, such an intermediate connection has not been accounted for (whereby $\gamma$=1.0)

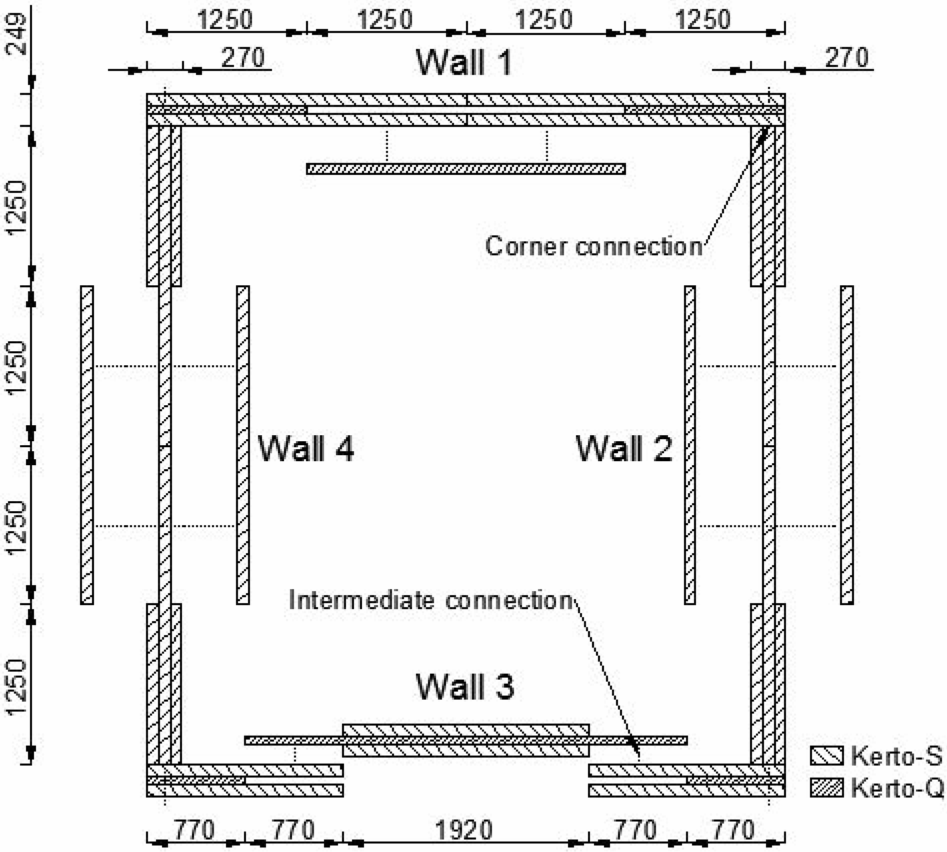


Figure 147: Proposed build-up of the distinct core walls

## 9.1.2 Design criteria

Governing in the design of the core are the requirements on strength and stiffness. From the design calculations, see Annex C.1, it appeared that the fastener lay-out influences the connection efficiency factor (γ) and thereby the cooperation between the walls.

When the connection efficiency decreases, the stiffness of the core decreases as well. From the requirements on stiffness the minimally required bending stiffness EI can be determined.

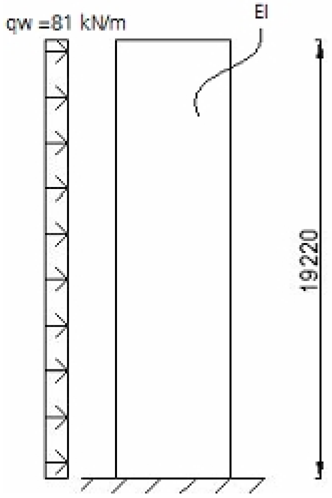


Figure 148: Schematisation

$$u_{\max} = 1/500 * l = 1/500 * 19220 = 38.4mm$$

$$j = \frac{M_{rep}}{C} = \frac{\left(0.5 * 81 * 19.22^2\right)}{19975147} = 0.00075rad$$

$$u_{foundation} = j * l = 0.00075 * 19220 = 14.4mm$$

$$u_{\max;bending} = u_{\max} - u_{foundation} = 38.4 - 14.4 = 24mm$$

$$EI_{required} = \frac{ql^4}{8u_{\max;total}} = \frac{81 * 19220^4}{8 * 24} = 5.757 \cdot 10^{16}\ Nmm^2$$

This value is being used as a basic assumption for the final design of the core wall connections.

## 9.1.3 Design assumptions

§ The core walls as proposed in the preliminary design phase are composed from Kerto elements. Although it might be beneficial to make use of other kinds of materials (see chapter 10), there is accounted for Kerto elements in this chapter.

§ Due to the maximum available thickness for Kerto, several elements should be glued on top of each other to obtain the required wall thickness. Consultation with the manufacturer, Finnforest, clarified that such a connection is common-practice. The glued connection can be assumed infinitesimally stiff.

## 9.2 Intermediate wall connections

### 9.2.1 Introduction

As mentioned earlier, the desired wall width of 5.0 m can only be reached by connecting several Kerto elements to each other. Due to transport restrictions these elements should be connected at the building site, whereby mechanical fasteners are to be used. This implies a reduction in stiffness due to slip of the fasteners.

Goal of this section is to determine what consequences the implementation of such a connection holds for the overall stiffness of the cores. Up till now, the intermediate wall connection was neglected, resulting in an overestimated stiffness.

To be able to do so, at first, the minimum required amount of fasteners is determined, i.e. the amount required to transfer the acting forces from one element to another.
Consecutively, it is investigated to what extent the stiffness of the walls is reduced when accounting for such an amount of fasteners.
Finally, the influence of the reduced stiffness on the stress distribution is taken into consideration.

### 9.2.2 Minimum required amount of fasteners

The required amount of fasteners depends on the acting loads, the fastener properties and the material properties. Since the walls are composed of three elements, the fasteners are regarded as being in double shear, see figure 149.

For now, the following design assumptions are taken in consideration:

§ Glued-in bolts, M16, quality 8.8
§ $\rho_k$ = 510 kg/m$^3$ (Kerto-S)
§ $\gamma_M$ = 1.20 (LVL)
§ $k_{mod}$ = 0.90 (short-term loading)

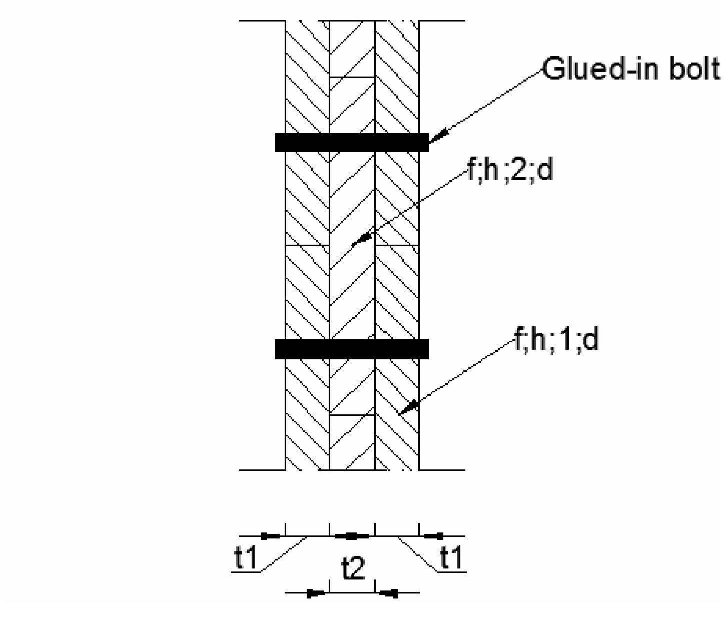


Figure 149: Glued-in bolts acting in double shear

As a first step in the design procedure, the acting loads are determined.

Step 1: Loads
Due to the physical division of each wall in parts, the connections should be able to transfer the acting loads from one to another part. For this particular situation the maximum acting shear stress, which acts near the foundation, appeared to be governing.

With reference to Annex C.1, the following shear stress distribution(1) is found for walls 2 and 4.

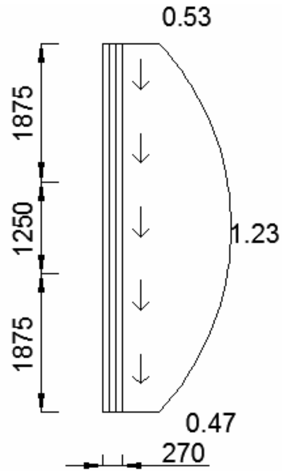


Figure 150: Shear stress acting at wall 2/4

[(1)]One should note that the presented stress distribution was determined by assuming that the walls were made of a single element (or at least by assuming γ = 1.0). Due to the introduction of the fasteners, the stress will increase somewhat. It is expected that this does not lead to any problems.

Since the fasteners are not located in the middle of the wall, the shear stress that acts at the fasteners is somewhat smaller than that at the centre of the wall. For the sake of simplicity, this is neglected and use is made of the maximum acting shear force of 1.23 N/mm$^2$. This leads to an acting shear force of:

$$V_{fastener} = 1.23 * 270 * 1000 = 332 kN/m$$

### Step 2: Embedment strength

The design load-carrying capacity of glued-in bolts (arranged perpendicular to the grain) is calculated according to article 8.5.1 of the Eurocode 5. The effect of the glue is incorporated by increasing the embedment strength with a factor 1.2 [ 24 ]:

$$f_{h;0;k;i} = 1.2 * 0.082 * (1 - 0.01 * d) * \rho_k = 1.2 * 0.082 * (1 - 0{,}01 * 16) * 510 = 42.15 N/mm^2$$

For walls 2 and 4, all elements are Kerto-S having their grain in the same (vertical) direction. As a result,

$$f_{h;0;k;1} = f_{h;0;k;2} = 42.15 N/mm^2 \text{ and } \beta = \frac{f_{h;0;k;2}}{f_{h;0;k;1}} = 1.0$$

### Step 3: Load carrying capacity

The load carrying capacity of the connection is determined by making use of Johansen's equations for fasteners in double shear, see figure 149.

$$F_{v;k} = \min[g; h; j; k] \text{ where:}$$

$$g = f_{h;1;k} * t_1 * d$$
$$h = 0.5 * f_{h;2;k} * t_2 * d$$
$$j = 1.05 \frac{f_{h;1;k} * t_1 * d}{2 + \beta}\left[\sqrt{2\beta(1+\beta) + \left(\frac{4\beta(2+\beta) * M_{y;k}}{f_{h;1;k} * d * t_1^2}\right)} - \beta\right]$$
$$k = 1.15 * \sqrt{\frac{2\beta}{1+\beta}}\sqrt{2M_{y;k} * f_{h;1;k} * d}$$

Making use of the earlier mentioned bolt characteristics, the following can be found:

$$M_{y;k} = 0.3 * f_{u;k} * d^{2{,}6} = 0.3 * 640 * 16^{2.6} = 0.259 \cdot 10^6 kNm$$

Implementing this value and the earlier mentioned material parameters in the formulae above, the following is found:

$F_{v;k} = \min[60.70;30.35;55.70;21.51] = 21.51\text{kN}$ for each shear plane

$F_{v;d} = 21.51 * \frac{0.9}{1.2} = 16.13\text{kN}$ for each shear plane

Since two shear planes are present (double shear), the capacity of the glued-in bolts becomes 32.26 kN.

### Step 4: Minimum required amount of fasteners

The acting loads and the load-carrying capacity of the proposed bolts being known, the minimum amount of fasteners which are required to comply with the requirements on strength are determined:

$$n_{ef;required} = \frac{V_{fastener}}{F_{v;d}} = \frac{332}{32.26} = 10.3 \rightarrow 11 \text{ fasteners/m}$$

Since 11 fasteners are required per linear meter, the spacing in between will be in the order of 90 mm when the bolts are aligned parallel to the grain direction (i.e. in vertical direction). Since the spacing influences the effectiveness of each fastener, the effective number of fasteners should be determined:

$$n_{ef} = \min\left[\begin{array}{l} n \\ n^{0.9} * \sqrt[4]{\frac{a_1}{13*d}} \end{array} = \begin{array}{l} 11 \\ 11^{0.9} * \sqrt[4]{\frac{90}{13*16}} = 7.02 \end{array}\right] = 7.02$$

Since this amount is smaller than the minimum required amount, additional fasteners should be implemented. Making use of the same formula as shown above, it can be found that at least 20 fasteners should be implemented per linear meter, to obtain an effective number of 10.3 fasteners.

Since an increased number of bolts arranged perpendicular to the grain (i.e. aligned horizontally) increases the vertical spacing, it may be beneficial to incorporate 5 rows of 4 bolts ($a_1$ = 4*50 mm), instead of 20 bolts in one row ($a_1$ = 50 mm). This will be elaborated on later.

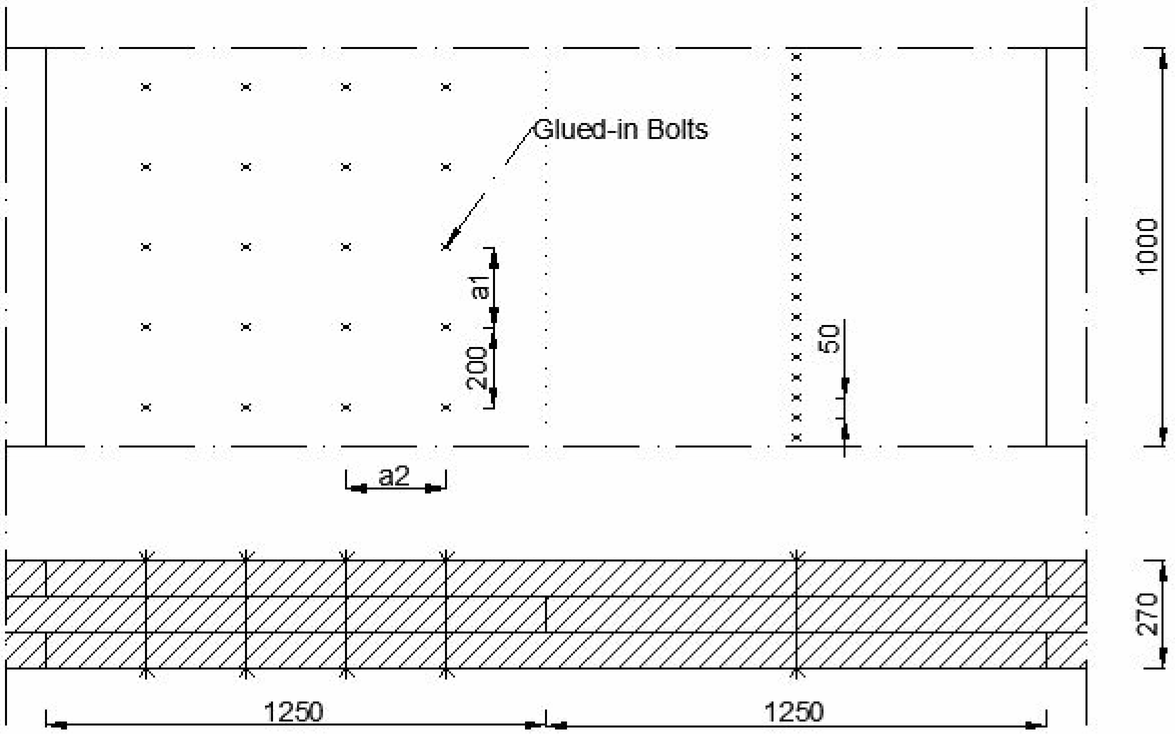


Figure 151: Different fastener arrangements (left: 5 rows of 4 bolts, right: 20 bolts in one row)

### 9.2.3 Stiffness reduction due to intermediate connections

In contradistinction to the analysis as performed in the preliminary design phase, it is now assumed that the core walls are composed of several elements which are interconnected by glued-in bolts. Due to the initial slip of these fasteners, the stiffness of the individual walls decreases to some extent.

In this section, the stiffness reduction of the core as a whole due to these intermediate connections is determined. Thereafter, it is determined whether the core is still able to comply with the requirements on horizontal deflections.

The design procedure is performed in 4 steps:
Step 1: the slip modulus of the proposed connection is determined
Step 2: the connection efficiency factor is determined
Step 3: the stiffness reduction of each individual wall is determined
Step 4: the stiffness reduction of the core as a whole is determined

But first, an overview is provided on the shape and the dimensions of an individual core wall, see figure 152. The proposed walls are composed of three elements, referred to as element 1, element 2 and element 3.

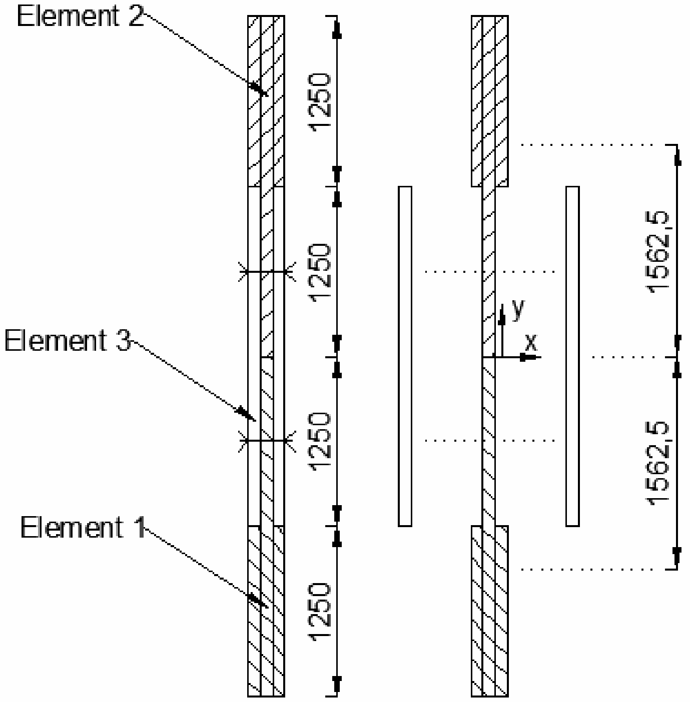


Figure 152: Build-up of core wall 2

#### Step 1: Slip modulus of the connection

The stiffness of the connection depends on the properties of the fasteners and the timber, which were presented in section 9.2.2. Making use of these values, the slip modulus of a single fastener is defined:

$$K_{ser;i} = \frac{1}{23}\rho^{1.5} * d = \frac{1}{23}510^{1.5} * 16 = 8012 N/mm$$

Since the fasteners are in double shear, this value is multiplied by 2:

$$K_{ser;double;i} = 2 * K_{ser;i} = 2 * 8012 = 16024 N/mm$$

#### Step 2: Efficiency of the connection

The efficiency of the connection depends on the connection efficiency factor, which is defined as

$$\gamma_{1/3;ser} = \left[1 + \pi^2 * \frac{E_1 * A_1 * s}{K_{ser;total} * l_{ef}^2}\right]^{-1}$$ in which s is the spacing in the direction of the acting load (vertical).

Generally, it is stated that the higher the connection efficiency factor, the higher the cooperation of the flanges (wall 1 and 3). Analysing the formula above, it is found that this factor becomes higher when the ratio $\frac{s}{K_{ser}}$ becomes smaller.

But at the same time, it should be noticed that s and $K_{ser}$ are linearly dependent on each other: the slip of all 20 fasteners is divided over a height of 1 m. When the vertical spacing is increased (less fasteners in vertical direction), the combined slip modulus of each row of fasteners increases as well due to the presence of additional fasteners in this direction. This is indicated in figure 153, where the ratio between the amount of fasteners and the available spacing remains the same.

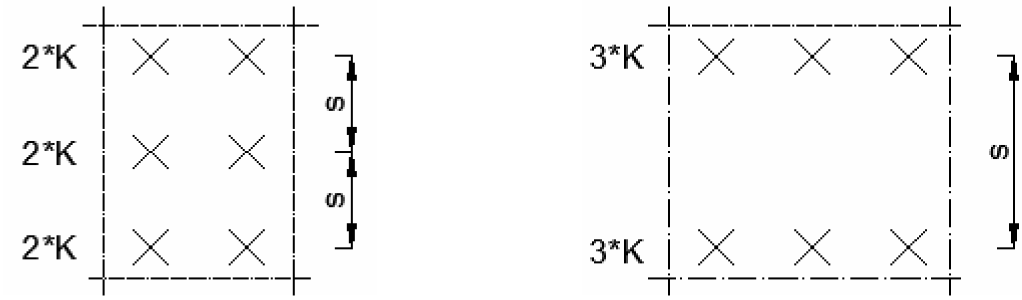


Figure 153: Influence of fasteners arrangement on ratio between s and K

The minimum values for the vertical and horizontal spacing amount 5d (= 80 mm) and 4d (= 64 mm) respectively. It is decided to account for a vertical spacing of 200 mm from this point on. Since 20 fasteners are required (see section 9.2.2), this results in 5 rows of 4 bolts every linear meter.

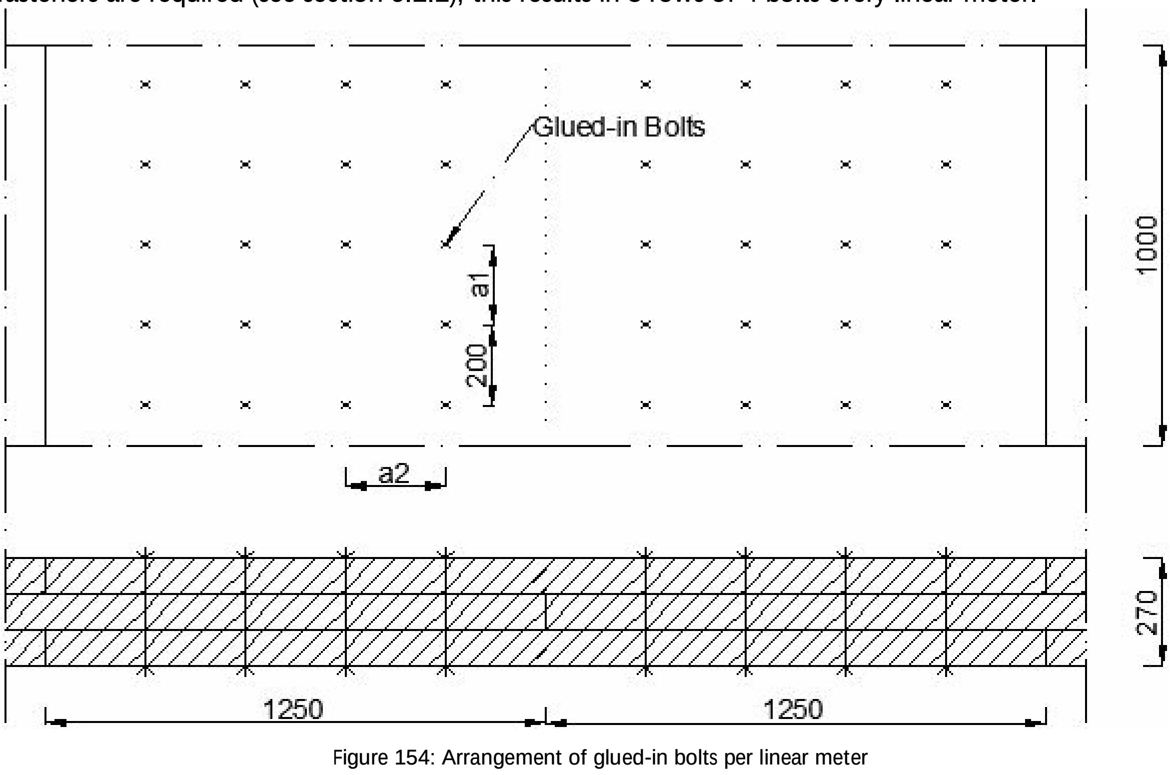


Figure 154: Arrangement of glued-in bolts per linear meter

Incorporating these values in the formula above, the following can be found:

$$\gamma_{element\ 1/3;ser} = \left[1+\pi^2 * \frac{6.21\cdot 10^9 * 200}{4 * 16024 * 38440^2}\right]^{-1} = 0.885\,; \qquad \gamma_{element\ 2;ser} \text{ is taken 1.0.}$$

Step 3: Stiffness reduction of the individual wall

As stated above, 20 glued-in bolts are incorporated per linear meter, equally divided over 5 rows. This resulted in a connection efficiency factor $\gamma_{ser}$=0.885.

Making use of this factor, the bending stiffness of the distinct Kerto elements can be defined as follows:

$$EI_{y;element;ser} = \sum_{i=3}\left(E_{ef} * \frac{1}{12} b_{element;i} * h_{element;i}^{3} + \gamma_i * EA_{element;i} * y^2_{c;local;element;i}\right)$$

Making use of this formula the following overview can be obtained, see table 30.

Table 30: Sectional properties for the distinct elements of wall 2/4

| Element | EA [Nmm$^2$] | $y_{c;local}$ | $EI_y$ [Nmm$^2$] |
|---|---|---|---|
| 1 | 6.21 * $10^9$ | 1562.5 | 15439 * $10^{12}$ |
| 2 | 6.21 * $10^9$ | 0 | 3234 * $10^{12}$ |
| 3 | 6.21 * $10^9$ | 1562.5 | 15439 * $10^{12}$ |
| Wall | 18.63 * $10^9$ | - | 34112 * $10^{12}$ |

Recalling the stiffness of this wall as determined in Annex C.1.4 (being $EI_{y;wall2/4;k}$=38810*$10^{12}$ Nmm$^2$), it is concluded that the stiffness of the wall reduces to about 87% of its original value due to the presence of the intermediate connections.

Making use of this value for the walls 2 and 4, it is determined whether the stiffness of the core as a whole is still sufficient to comply with the requirements.

### Step 4: Stiffness reduction of the complete core

When the walls 1 and 3 are taken into consideration, it is stated that the influence of their intermediate connection is negligible when the bending stiffness in y-direction is concerned, see figure 147.

Accounting for the reduced stiffness of walls 2 and 4, as stated in table 30, the following results can be obtained for the core as a whole:

Table 31: Reduced stiffness of the core as a whole

| Wall | $EI_y$ [Nmm$^2$] |
|---|---|
| 1 (no reduction) | 87 *$10^{12}$ |
| 2 | 34112 * $10^{12}$ |
| 3 (no reduction) | 54 * $10^{12}$ |
| 4 | 34112 * $10^{12}$ |
| Core as a whole | 16.89 * $10^{16}$ |

From the chapter structural system, it is extracted that the bending stiffness is 17.83 * $10^{16}$ Nmm$^2$ when the intermediate connection is assumed infinitesimally stiff. The stiffness reduction due to the intermediate connections is therefore only about 5 %.

### Conclusion

The minimally required bending stiffness of the core in y-direction appeared to be 5.757 *$10^{16}$ Nmm$^2$, see section 9.1.2. With the reduced stiffness still being 16.89 *$10^{16}$ Nmm$^2$, it is concluded that no problems concerning horizontal deflections are to be expected.

## 9.2.4 Renewed stress distribution

As a result of the stiffness reduction, the stresses that act on the section of the core increase somewhat. In this section, it is investigated whether the core is still able to resist these stresses.

Since the strength of the structure is at stake, the ultimate limit state applies. As a result, there has to be accounted for the slip modulus $K_u$ instead of $K_{ser}$ and the connection efficiency factor $\gamma_u$ instead of $\gamma_{ser}$:

$$K_{u;double;i} = \frac{2}{3} K_{ser;double;i} = \frac{2}{3} * 16024 = 10682 N/mm$$

$$\gamma_{element\ 1/3;ser} = \left[1 + \pi^2 * \frac{6.21 \cdot 10^9 * 200}{4 * 10682 * 38440^2}\right]^{-1} = 0.837$$

Applying the same procedure as used in the preceding chapter, it is found that the stiffness of the core (in y-direction) becomes $15.30*10^{16}$ $Nmm^2$, which is about 92% of its 'original' value in the ultimate limit state ($16.53*10^{16}$ $Nmm^2$).

Implementing this value in the following formulae, the maximum stresses can be determined, see table 32.

$$\sigma_{m;i} = \frac{0.5 * E_i * h_i * M}{EI_{ef}}\ ;\ \sigma_i = \frac{\gamma_i * E_i * a_i * M}{EI_{ef}}\ ;\ \sigma_N = \frac{N * E_i}{EA_{tot}}$$

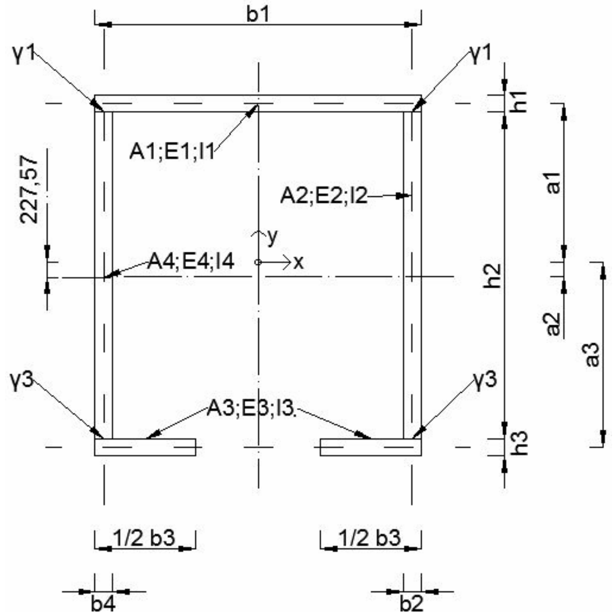


Figure 155: Parameters concerning the stress distribution

Table 32: Stress distribution in the y-direction on the distinct core walls (accounting for intermediate connections)

| i | $\sigma_{m;y;i}$ [N/mm$^2$] | $\sigma_{y;i}$ [N/mm$^2$] | $\sigma_{N;y}$ [N/mm$^2$] |
|---|---|---|---|
| 1 | +/- 0.23 | +/- 2.40 | 0.84 |
| 2 | +/- 5.93 | +/- 0.54 | 1.09 |
| 3 | +/- 0.23 | +/- 3.45 | 0.84 |
| 4 | +/- 5.93 | +/- 0.54 | 1.09 |

Comparing these results with those found in the Annex C.1, see table 33, it is concluded that the bending stresses increase to a minimal extent only. This could already be expected beforehand, since the stiffness reduction is only minimal.

Table 33: Stress distribution in the y-direction on the distinct core walls (neglecting intermediate connections)

| i | $\sigma_{m;y;i}$ [N/mm$^2$] | $\sigma_{y;i}$ [N/mm$^2$] | $\sigma_{N;y}$ [N/mm$^2$] |
|---|---|---|---|
| 1 | +/- 0.21 | +/- 2.22 | 0.84 |
| 2 | +/- 5.49 | +/- 0.50 | 1.09 |
| 3 | +/- 0.21 | +/- 3.20 | 0.84 |
| 4 | +/- 5.49 | +/- 0.50 | 1.09 |

## 9.2.5 Conclusions

- A total of 20 fasteners is required in longitudinal direction, for each linear meter.
- The required amount of fasteners follows from strength rather than stiffness requirements
- The intermediate core wall connections influence the stiffness of the cores to a minor extent.
- The bending stresses on the core walls increase minimally due to the reduced stiffness.

## 9.3 Corner connection

### 9.3.1 Introduction

The proposed core is composed of 4 individual walls, which are assumed to be connected at their edges. In the preliminary design phase, this connection was assumed to be made with dowel-type fasteners, having a diameter of 24 mm, see Annex C.1.6. These fasteners were assumed to be spaced at a c.t.c. distance of 150 mm.

Considering the governing y-direction (EW-direction), it was found that such a fastener lay-out resulted in a reduced bending stiffness, as can be seen in table 34.

Table 34: Stiffness reduction due to corner connections (SLS)

| $\gamma_{1;y;ser}$ | $\gamma_{3;y;ser}$ | $EI_{eff;y;k}$ [$Nmm^2$] |
|---|---|---|
| 1.0 | 1.0 | $22.22 * 10^{16}$ |
| 0.664 | 0.746 | $17.83 * 10^{16}$ |

The reduction in stiffness due to the slip of the corner connections appeared to be over 20%. Although these values are still sufficient to comply with the requirements on stiffness, one can imagine that the material is used quite inefficiently.

Goal of this section is therefore to investigate whether the efficiency of the corner connection can be improved, thereby increasing the overall stiffness of the core.

To be able to do so, in this section, the influence of the amount of fasteners on the connection efficiency is considered.

### 9.3.2 Influence of the fasteners on the connection efficiency

As mentioned above, due to the slip of the mechanical fasteners the stiffness of the timber core is reduced. In this section, it is investigated whether the stiffness reduction can be decreased by accounting for a different fastener lay-out.

Before starting the analysis, the most important design assumptions are mentioned:

- § The influence of the intermediate connections, see section 9.2, is neglected
- § Screws are considered the most suitable kind of fasteners here
- § $\rho_k = 510$ kg/$m^3$ (Kerto-S)

Step 1: Analysis

To be able to improve the connection efficiency, the parameters which influence the γ-factor are analysed:

$$\gamma_{ser;i} = \left[1 + \pi^2 \frac{EA_i * s}{2K_{ser} * l_{ef}^2}\right]$$

$$K_{ser} = \frac{1}{23}\rho_{mean}^{1.5} * d_{fastener}$$

Considering the parameters that concern the fasteners, it is found that a higher γ-factor is obtained by decreasing the ratio $\frac{s}{K}$ or even more specific $\frac{s}{d_{fastener}}$, where s is the vertical spacing.

It is therefore concluded that it is beneficial to implement the fasteners as close to each other as possible (small s), while the diameter of the fasteners should be taken as large as possible.

Since the minimum acceptable spacing is directly related to the used diameter, the ratio as shown above is a constant when accounting for the minimum vertical spacing: $\frac{s_{min}}{d_{fastener}} = \frac{n * d}{d} = n$.

To be able to decrease this ratio additional fasteners should be implemented in horizontal direction, whereby the stiffness K is increased: implementing n fasteners horizontally, the ratio decreases by a factor $n_{hor}$: $\frac{s_{min}}{n_{hor} * K_{ser}}$, see figure 156.

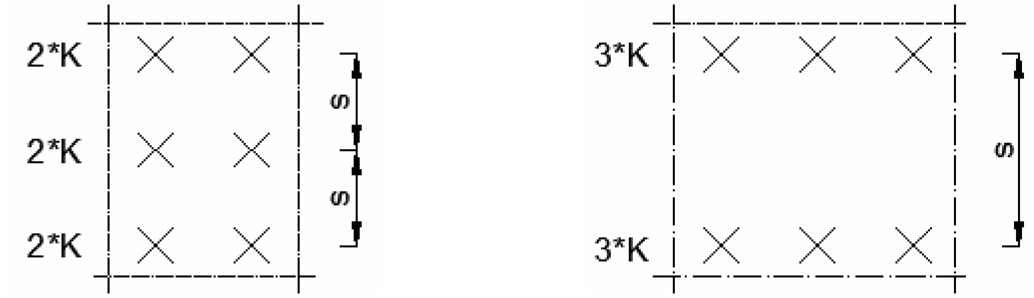


Figure 156: Influence of fasteners arrangement on ratio between s and K

Since there are also requirements on horizontal spacing, the amount of fasteners that can be used is limited.

### Step 2: Influence of the amount of fasteners

The building codes state the following on the minimum required distances (accounting for an angle of 0° to the grain):

Table 35: Required spacing for several fastener types

| | $a_1$ [mm] | $a_2$ [mm] | $a_3$ [mm] | $a_4$ [mm] |
|---|---|---|---|---|
| Large screws(1) (d> 6mm) | 4*d | 4*d | 4*d | 3*d |
| Nails / small screws(2) (d< 6 mm) | 5*d | 3*d | 7*d | 3*d |

(1)the effective number of fasteners parallel to the grain should be taken $n_{ef} = n^{0.9} * \sqrt[4]{\frac{a_1}{13d}}$ with a maximum of n.

(2)the effective number of fasteners parallel to the grain should be taken $n_{ef} = n^{k_{ef}}$

The thickness of the core walls in which the fasteners are to be anchored is 270 mm. Accounting for the values as shown in table 35, the following can be found when accounting for large screws, see table 36.

Table 36: Maximum diameter for a given amount of fasteners parallel to each other (large screws)

| $n_{horizontal}$ | $s_{min;horizontal}$ | $d_{max}$ [mm] (large screws) |
|---|---|---|
| 2 | 10*d | 20 |
| 4 | 18*d | 15 |
| 5 | 22*d | 12 |
| 6 | 26*d | 10 |

When small screws of 6 mm are considered, it can be found that the maximum number of screws that can be implemented parallel to each other is about 13$\left(\frac{270 - 2*(3*d)}{3*d}\right)$.

Making use of the formulae for K and γ, in combination with the sectional properties of the cores, the following results can be obtained while varying the number of fasteners, see table 37.

Table 37: Influence of the fastener pattern on the connection factor $\gamma_{1;y;ser}$

| d [mm] | $n_{horizontal}$ | $s_{vertical}$ [mm] | $n_{total}$ [/m] | $n_{horizontal} * K_{ser}$ [N/mm] | $\gamma_{1;y;ser}$ | $\gamma_{3;y;ser}$ | $EI_{eff;y;k}$ [$Nmm^2$] |
|---|---|---|---|---|---|---|---|
| *24* | *1* | *150* | *6.7* | *12020* | *0.664* | *0.746* | *17.83 * $10^{16}$* |
| 20 | 2 | 80 | 25 | 20030 | 0.850 | 0.902 | 20.42 * $10^{16}$ |
| 15 | 4 | 60 | 67 | 30045 | 0.919 | 0.948 | 21.26 * $10^{16}$ |
| 12 | 5 | 48 | 105 | 30045 | 0.934 | 0.958 | 21.44 * $10^{16}$ |
| 10 | 6 | 40 | 225 | 30045 | 0.944 | 0.965 | 21.56 * $10^{16}$ |
| 6 | 13 | 30 | 434 | 39059 | 0.967 | 0.979 | 21.83 * $10^{16}$ |

Conclusion
From table 37 it can be concluded that the connection efficiency factor, and thus the stiffness of the core as a whole, can be improved by implementing additional fasteners. In the ultimate situation, an efficiency factor of even 0.967 can be achieved. Despite that, it is highly questionable whether it is desirable to implement, for example, 433 fasteners per linear meter.

On the other hand, compared with the first assumption (d = 24 mm, s = 150 mm), significantly better results can already be obtained by implementing only few additional fasteners. It goes beyond the goal of this thesis to determine the most beneficial amount of fasteners and spacing.

## 9.4 Conclusions

### 9.4.1 Intermediate wall connections

§ From strength requirements, 20 fasteners have to be implemented in the design per linear meter
§ The stiffness reduction due to these intermediate fasteners is limited (only about 5%)
§ Bending stresses on the core walls increase minimally due to such a stiffness reduction.

### 9.4.2 Corner connections

§ The more fasteners there are implemented, the smaller the stiffness reduction
§ Accounting for a large amount (434 per linear meter) of small diameter (d=6 mm) fasteners, the stiffness reduction can be limited to only 2%.

### 9.4.3 Recommendations

§ It should be investigated whether the combination of the intermediate and the corner connections provide adequate stiffness.
§ It should be determined whether it is more beneficial to implement a large amount of fasteners, or to accept a stiffness reduction of about 20%.
§ Accounting for these connections, the dimensions of the core could be further optimised
§ Instead of a corner connection by means of fasteners, a connection with staggered elements(1) could be investigated.

(1) A highly simplified analysis, which is not included in this report, was carried out on a staggered connection with storey high elements. It appeared that the material holds sufficient strength to resist the acting shear stress. When the stiffness of such a connection is concerned, additional research is required. The theory available on this subject is still experimental and accounts for prefabricated concrete elements rather than timber elements.

# 10 Core walls composed of CLT elements

## 10.1 Introduction

In the preliminary design phase, it was considered beneficial when the core walls were made from elements with a large length, i.e. with a length equal to the height of the core. The choice for such elements implies that no intermediate connections are required in longitudinal direction.

Due to transport restrictions, see Annex F, elements with a length of 19.22 m may only have a width of about 4 m. Since the required wall width is 5 m, intermediate connections in transverse direction can not be avoided.

As a consequence, it is found that both the most interesting plate-like timber products, being LVL and CLT, can be used from the viewpoint of available dimensions, see table 38. Since LVL holds a greater stiffness and a higher shear strength than CLT, the choice was made to account for Kerto LVL elements.

Table 38: Overview on properties of both LVL and CLT

| Property | LVL: Kerto | CLT: LenoTec |
|---|---|---|
| b [mm] | up to 2500 mm | up to 4800 mm |
| h [mm] | up to 90 mm | up to 500 mm |
| l [mm] | up to 25 m | up to 20 m |
| $E_{0;mean}$ [N/mm²] | 13800 (S-type)<br>10500 (Q-type) | 11000 |
| $E_{90;mean}$ [N/mm²] | 430 (S-type)<br>2400 (Q-type) | 370 |
| $f_{m;0;k}$ [N/mm²] | 44 (S-type)<br>32 (Q-type) | 24 |
| $f_{v;k}$ [N/mm²] | 4.1 (S-type)<br>4.5 (Q-type) | 2.7 |

When the proposed structural systems were evaluated, see section 4.6, an expert posed questions on the feasibility of erecting a core with a height of 19.22 m. Therefore, it has been investigated whether it is possible to divide the core in segments of only one storey, see chapter 8. It appeared that such a solution requires some additional attention, but that it is feasible.

Accounting for such a solution, dividing the core walls in 5 segments with equal height, the length of the elements becomes about 4 m. Considering the transport restrictions again, see Annex F, it is found that elements having a width of 5 m can still be transported.

As a result, the limited maximum width of LVL becomes a negative characteristic: still, several elements have to be connected to obtain the required width. From the chapter 9, it appeared that quite a lot of fasteners were required to provide such connections, not even to mention the reduction in stiffness of the system.

Considering the maximum width of CLT, it is found that the segments can be made from single elements when the width is limited to 4.8 m. Since this is considered highly beneficial, the structural behaviour of the CLT elements under the acting loads is investigated in this chapter.

## 10.2 Analysis

Goal of this chapter is to investigate the structural behaviour of a timber core made from CLT elements. Thereafter, the results are compared to those of a Kerto-core were has been accounted for intermediate connections.

To perform a 'honest' comparison, use is made of CLT elements having dimensions that are close to those of their Kerto equivalents. From the suppliers' specification [ 16 ] it is found that the most suitable elements are the LenoTec 264 element and the LenoTec 231 element. These elements have the following structural build-up, where the bold layers run in longitudinal direction:

Walls 1 and 3: LenoTec 231: 33 – 33 – 33 – 33 – 33 – 33 – 33
Walls 2 and 4: LenoTec 264: 33 – 33 – 33 – 33 – 33 – 33 – 33 – 33

The maximum width of the elements is 4.8 m, whereby the dimensions of the core become somewhat smaller, see figure 157.

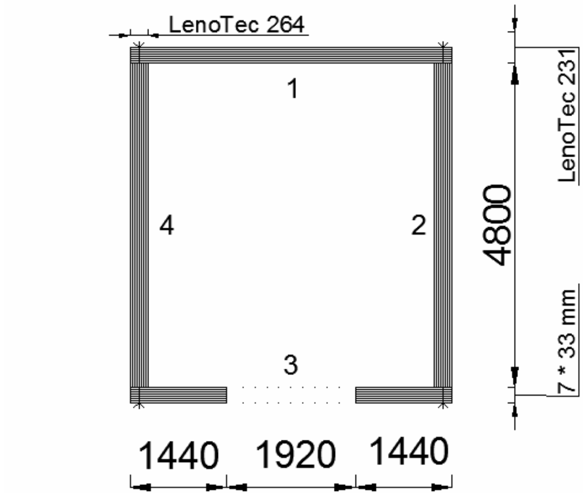


Figure 157: Timber central core

As discussed in section 4.4, the cores are loaded by a bending moment caused by the wind and a normal force caused by the loads on the distinct floors and its own self-weight.

## 10.3 Design assumptions

- § There is accounted for the same core lay-out as for the proposed Kerto core (including openings)
- § Only the governing EW-direction is taken in consideration
- § The vertical loads follow from the weight calculation rather than the proposed preliminary design
- § The elements are connected at their corners by means of dowels (d=24 mm), equal to the Kerto core
- § The stiffness reduction due to the division in segments is neglected

## 10.4 Sectional properties

Accounting for the lay-out as shown in figure 157 and the properties as mentioned in table 38, the following can be found:

$EA_1$ = 11000 N/mm$^2$ * 4800*165 mm$^2$ + 370 N/mm$^2$ * 4800 * 66 mm$^2$= 0.883 * $10^{10}$ Nmm$^2$
$EA_2 = EA_4$ = 11000 N/mm$^2$ * 4800*198 mm$^2$ + 370 N/mm$^2$ * 4800 * 66 mm$^2$= 1.057 * $10^{10}$ Nmm$^2$
$EA_3$ = 11000 N/mm$^2$ * 2880*165 mm$^2$ + 370 N/mm$^2$ * 2880 * 66 mm$^2$= 0.530 * $10^{10}$ Nmm$^2$

with $a_{y;1}$ = 2263.6 mm
$a_{y;2} = a_{y;4}$ = 251.9 mm
$a_{y;3}$ = = 2767.4 mm

The slip moduli accounting for the connection of the distinct walls, $K_{ser}$ and $K_u$, are determined as follows:

$$K_{ser} = \frac{1}{23}\rho_m^{1.5} * d_{bolt} = \frac{1}{23} * 500^{1.5} * 24 = 11666 \ \text{N/mm}$$

$$K_u = \frac{2}{3}K_{ser} = 7778 \ \text{N/mm}$$

### Connection efficiency factors

With help of the slip moduli, the connection factors γ are determined.

- § Serviceability limit state

$$\gamma_{1;y;ser} = \left[1+\pi^2 \frac{E_1A_1 * s}{2K_{ser} * l_{ef}^2}\right]^{-1} = \left[1+\pi^2 \frac{0.883\cdot 10^{10} * 150}{2 * 11666 * 38440^2}\right]^{-1} = 0.725$$

$$\gamma_{3;y;ser} = \left[1+\pi^2 \frac{E_3A_3 * s}{2K_{ser} * l_{ef}^2}\right]^{-1} = \left[1+\pi^2 \frac{0.530\cdot 10^{10} * 150}{2 * 11666 * 38440^2}\right]^{-1} = 0.815$$

§ Ultimate limit state

$$\gamma_{1;y;u} = \left[1+\pi^2 \frac{E_1A_1 * s}{2K_{u} * l_{ef}^2}\right]^{-1} = \left[1+\pi^2 \frac{0.883\cdot 10^{10} * 150}{2 * 7778 * 38440^2}\right]^{-1} = 0.637$$

$$\gamma_{3;y;u} = \left[1+\pi^2 \frac{E_3A_3 * s}{2K_{u} * l_{ef}^2}\right]^{-1} = \left[1+\pi^2 \frac{0.530\cdot 10^{10} * 150}{2 * 7778 * 38440^2}\right]^{-1} = 0.746$$

### Bending stiffness

One should note that both wall 1 and wall 3 are loaded in their plane. As a result, there has to be accounted for the flexible bond between the individual longitudinal layers, which is referred to as the shear deformation.

Therefore, the same procedure has to be applied as used in Annex D.1.2, to determine the stiffness of these walls, i.e. use has to be made of the following formulae and figure 158.

$$I_{eff} = \sum^{n} n_i * I_i + \gamma_i * n_i * A_i * a_i^2$$

The flexibility factor γ can be determined by making use of the following formula:

$$\gamma_i = \left(1+\frac{\pi^2 * EA_i * \overline{h_i}}{G_R * b * l^2}\right)^{-1}$$ and $\gamma_2 = 1$.

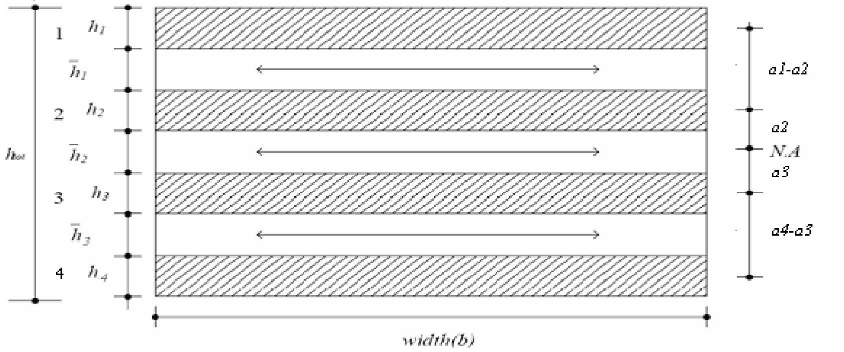


Figure 158: Built-up of a CLT panel

Incorporating the found values for these walls in the following formulae, the overall bending stiffness of the core can be determined:

§ Serviceability limit state

$$(EI)_{ef;y;ser} = \sum (EI)_{y;i} + \gamma_{i;y;ser} * (EA)_i * a_{i;y}^2 = 10.74\cdot 10^{16} Nmm^2$$

§ Ultimate limit state

$$(EI)_{ef;y;u} = \sum (EI)_{y;i} + \gamma_{i;y;u} * (EA)_i * a_{i;y}^2 = 10.07\cdot 10^{16} Nmm^2$$

## 10.5 Serviceability

The governing load combination in EW-direction appears to be SLS 1b, see table 23. These values follow directly from Annex C.1.3.

Table 39: Governing load combination

| Situation: | N [kN] | $q_w$ [kN/m] | $F_{hor;2nd}$ [kN] | $F_{hor;0}$ [kN] | $F_{hor;-1st}$ [kN] |
|---|---|---|---|---|---|
| SLS 1b: | 3245 | 81 | 39 | 32 | 32 |

The maximum acceptable deflection was limited to 25.6 mm, where has already been accounted for the rotational stiffness of the foundation. This results in a minimally required bending stiffness EI of:

$$w_{max} = \frac{q * l^4}{8EI} = \frac{81 * 19220^4}{8 * 25.6} = 5.398 \cdot 10^{16} Nmm^2$$

From the previous section, it is found that the bending stiffness of the CLT core is 10.74 * $10^{16}$ $Nmm^2$, which largely fulfils the requirements.

## 10.6 Strength

To determine the stresses at the most heavily loaded cross-section, use is made of the procedure as applied in Annex C.1.6. Accounting for the sectional properties as shown in table 40 and figure 240, the presented loads and the applying formulae, the stresses as shown in table 41 are found.

### Sectional properties

Table 40: Properties for the individual walls

| i | $h_i$ [mm] | $\gamma_{y;i}$ [-] | $E_{eff;i}$ [N/mm$^2$] | $a_{y;i}$ [mm] | $EI_y$ [Nmm$^2$] | $EA_i$ [N] |
|---|---|---|---|---|---|---|
| 1 | 231 | 0.637 | 7693 | 2263.6 | $10.07*10^{16}$ | $0.883*10^{10}$ |
| 2 | 4800 | 1.0 | 8343 | 251.9 | $10.07*10^{16}$ | $1.057*10^{10}$ |
| 3 | 231 | 0.746 | 7693 | 2767.4 | $10.07*10^{16}$ | $0.530*10^{10}$ |
| 4 | 4800 | 1.0 | 8343 | 251.9 | $10.07*10^{16}$ | $1.057*10^{10}$ |

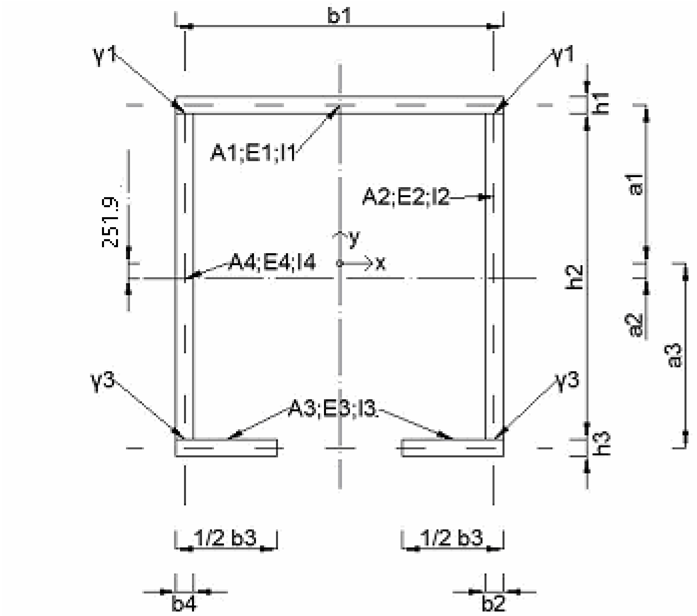


Figure 159: Parameters concerning the stress distribution

### Loads

The acting loads follow directly from the governing load combination, see Annex C.1.4.

$N_d$ = 4633 kN ;$M_d$ = 26280 kNm; $V_d$ = 2743 kN

Stress distribution

$$\sigma_{m;i} = \frac{0.5 * E_i * h_i * M}{EI_{ef}} ; \sigma_i = \frac{\gamma_i * E_i * a_i * M}{EI_{ef}} ; \sigma_N = \frac{N * E_i}{EA_{tot}}$$

$$\tau_{1/3/5;max} = \frac{\gamma_1 * E_i * A_i * a_i}{b_i^a * EI_{ef}} * V ; \tau_{2/4;max} = \frac{\gamma_1 * E_1 * A_1 * a_1 + 0.5 * E_2 * b_2 * (0.5h_2 + a_2)^2}{b_2{}^a * EI_{ef}} * V$$

Table 41: Stresses due to bending moment, vertical loading and shear

| i | $\sigma_{m;y;l}$ [N/mm$^2$] | $\sigma;_{y;i}$ [N/mm$^2$] | $\sigma_{N;y}$ [N/mm$^2$] | $\tau_{y;i;min;1}$ [N/mm$^2$] | $\tau_{y;i;max}$ [N/mm$^2$] | $\tau_{y;i;min;3}$ [N/mm$^2$] |
|---|---|---|---|---|---|---|
| 1 | +/- 0.24 | +/- 3.00 | 1.05 | 0 | 0.75 | 0 |
| 2 | +/- 5.23 | +/- 0.55 | 1.10 | 0.66 | 1.27 | 0.56 |
| 3 | +/- 0.24 | +/- 4.29 | 1.05 | 0 | 0.64 | 0 |
| 4 | +/- 5.23 | +/- 0.55 | 1.10 | 0.66 | 1.27 | 0.56 |

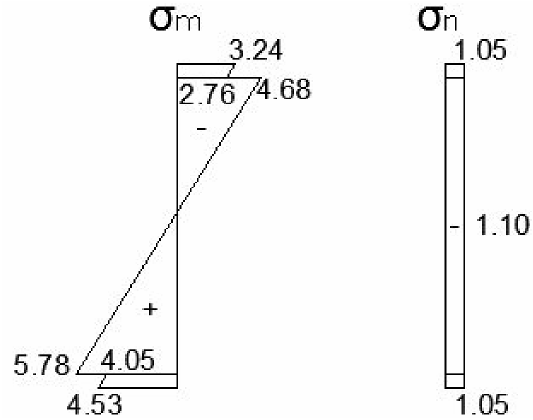


Figure 160: Distribution of normal and bending stresses over the distinct core walls

Since these stresses are rather small, it is stated that these comply with the requirements.

## 10.7 CLT core versus Kerto core

In the preceding sections, the structural behaviour of a core composed of CLT elements has been considered. This solution is now compared to the earlier found Kerto core. Since the main advantage of the CLT core over the Kerto core is the absence of intermediate connections, the proposed solution is compared to the Kerto core as discussed in section 9.2.

### 10.7.1 Stiffness

Accounting for the values found in section 10.4 and section 9.2, see table 42, it is concluded that the stiffness of the CLT core is significantly smaller than that of a Kerto core with intermediate connections.

Table 42: Reduced stiffness of the core as a whole

| Core material | $EI_y$ [Nmm$^2$] |
|---|---|
| CLT | 10.07 $*10^{16}$ |
| Kerto | 16.89 $* 10^{16}$ |

Despite that, both kind of cores comply with the requirements on serviceability (EI > 5.398 $* 10^{16}$ Nmm$^2$).

### 10.7.2 Strength

When the stresses on the CLT core (table 41) are compared with those of a Kerto core with intermediate connections (table 32), it is concluded that the difference can be found in the distribution over the core distinct walls.

Table 43: Stress distribution in the y-direction on the distinct core walls (accounting for intermediate connections)

| i | $\sigma_{m;y;i}$ [N/mm$^2$] | $\sigma_{y;i}$ [N/mm$^2$] | $\sigma_{N;y}$ [N/mm$^2$] |
|---|---|---|---|
| 1 | +/- 0.23 | +/- 2.40 | 0.84 |
| 2 | +/- 5.93 | +/- 0.54 | 1.09 |
| 3 | +/- 0.23 | +/- 3.45 | 0.84 |
| 4 | +/- 5.93 | +/- 0.54 | 1.09 |

When the Kerto core is considered, wall number 2 and 4 are far more heavily stressed than wall number 1 and 3. Main reason can be found in the difference in stiffness of these walls ($E_{2/4}$ = 13800 N/mm$^2$ vs $E_{1/3}$ = 10641 N/mm$^2$). When the CLT core is considered, the stresses are distributed more even over the four walls ($E_{2/4}$ = 8343 N/mm$^2$ vs $E_{1/3}$ = 7693 N/mm$^2$).

The magnitude of the stresses is for both solutions in the same range, and complies with all requirements.

### 10.7.3 Practical considerations

When the Kerto core is concerned, three Kerto elements are to be connected to obtain the required wall width of 5.0 m. This implies that, in longitudinal direction, at least (2 times) 20 fasteners are required to provide an adequate connection.

When the CLT core is considered, a single element suffices to cover the required dimensions. As a result, the core can be erected much faster.

## 10.8 Conclusions and recommendations

§ The proposed CLT core complies with the requirements on both strength and stiffness.

§ The structural behaviour of a CLT core is less beneficial than a Kerto core

§ The CLT core offers benefits over the Kerto core during erection

§ Since the CLT core offers benefits during the erection phase, while it complies with all requirements, it is recommended to account for CLT elements for the final design

§ It might be interesting to investigate a combined system of CLT and Kerto walls

§ It is recommended to optimise the dimensions of both the Kerto core and the CLT core, and to compare the systems again afterwards.

# 11 Additional checks

From chapter 10 it appeared that the CLT core is most beneficial to be used in a timber stadium structure. Although all design calculations before chapter 10 concerned core walls composed from LVL elements, it is stated that the CLT cores presumably comply with the requirements as well.

In this chapter some additional checks are performed concerning the CLT cores. At first, buckling of the individual core walls is considered. Thereafter, the behaviour under fire conditions is at stake.

## 11.1 Buckling of the 'columns'

From C.1.6, it appears that the proposed LVL core as a whole satisfies the requirements on flexural buckling largely. It is therefore assumed that the CLT core satisfies the requirements as well. Despite that, it remains unsure whether the individual core walls of the CLT core satisfy these requirements as well. This is therefore investigated in this section.

### 11.1.1 Analysis

Next to the vertical loading following from the floor loads, the cores are under influence of a horizontal load caused by wind and grandstand loading. As a consequence, there has to be accounted for flexural buckling under the load combination of vertical force and bending moment.

Since the horizontal wind load that acts on a core has only one main direction at any given time, the resulting bending moments in EW- and NS-direction can not occur at the same time. It is therefore sufficient to account for the combination of bending in one direction and vertical loading.

Governing in the design are the walls that are most heavily loaded or the walls that have the largest height in between two floors. It goes without saying that the first condition applies to the walls at the -2$^{nd}$ floor. The second condition applies to the walls at ground floor level, having a height of 4.52 m.

### 11.1.2 Loads

The acting loads and the various load combinations were already determined in Annex C.1. Therefore, only the governing combination is presented here, see table 14.

Table 44: Governing loads

| | $ULS_{EW}$ | $ULS_{NS}$ |
|---|---|---|
| $q_{wind}$ [kN/m] | 134 | 28 |
| $F_2$ [kN] | 64 | - |
| $F_0$ [kN] | 52 | - |
| $F_{-1}$ [kN] | 52 | - |
| N [kN] | 4632 | 4632 |

Accounting for these loads, the load distribution is determined for the governing floors, see table 15.

Table 45: Load distribution for the governing floors

| Level | $M_{EW;d}$ [kNm] | $M_{NS;d}$ [kNm] | $N_d$ [kN] |
|---|---|---|---|
| 0 | 10708 | 2125 | 2780 |
| -2 | 26280 | 5172 | 4633 |

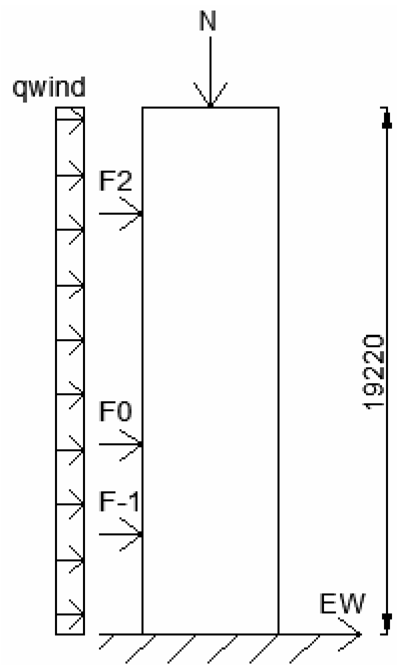


Figure 161: Governing load combination

## 11.1.3 Stresses

The stress distribution is determined from the loads as presented in table 15, by making use of the following formulae:

$$\sigma_{m;i} = \frac{0.5 * E_i * h_i * M}{EI_{ef}} \pm \frac{\gamma_i * E_i * a_i * M}{EI_{ef}} \; ; \; \sigma_N = \frac{N * E_i}{EA_{tot}}$$

The required sectional properties of the core can be traced back from section 10.4.

Applying this approach to all walls and all core segments, the following values are obtained for the governing ground and -2nd level, see table 17.

Table 46: Stresses on the distinct walls at the governing levels

| Level | Wall number | $\sigma_{M;EW}$ [N/mm²] | $\sigma_{M;NS}$ [N/mm²] | $\sigma_N$ [N/mm²] |
|---|---|---|---|---|
| 0 | 1 | 1.32 | 0.42 | 0.63 |
| | 2/4 | 2.35 | 0.43 | 0.66 |
| | 3 | 1.85 | 0.25 | 0.63 |
| -2 | 1 | 3.24 | 1.02 | 1.05 |
| | 2/4 | 5.78 | 1.07 | 1.10 |
| | 3 | 4.53 | 0.61 | 1.05 |

## 11.1.4 Sectional properties of the walls

Since the focus is on flexural buckling of the individual walls, the sectional properties of these walls have to be accounted for. Making use of the formulae as presented in section 10.4 and the cross-sectional shape as shown in figure 162, the overview as presented in table 47 is found.

Table 47: Sectional properties of the individual walls

| Wall number | A [mm²] | $EI_{EW}$ [mm⁴] | $EI_{NS}$ [mm⁴] |
|---|---|---|---|
| 1 | $0.883*10^{10}$ | $50*10^{12}$ | $16730*10^{12}$ |
| 2/4 | $1.057*10^{10}$ | $20070*10^{12}$ | $72*10^{12}$ |
| 3 | $0.530*10^{10}$ | $30*10^{12}$ | $15660*10^{12}$ |

The walls at the -2nd floor have a height of 3.65 m and are assumed to be hinged at both their bottom and top. The walls at the ground floor have a height of 4.52 m and are given the same support conditions.

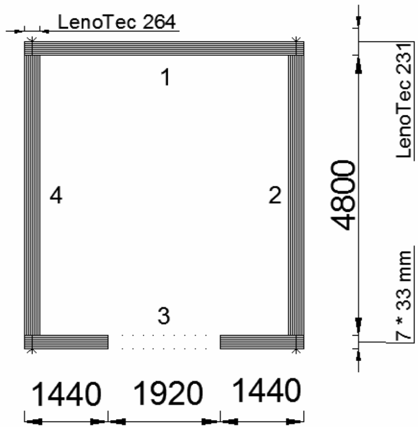


Figure 162: Timber central core

## 11.1.5 Material properties

Accounting for the structural build-up of the core walls as presented in figure 162, the following material properties are found:

LenoTec 231: walls 1 and 3

$$f_{m;0;k} = \frac{24*5*33*11000}{5*33*11000+2*33*370} = 23.68 \text{N/mm}^2$$

$$f_{c;0;k} = \frac{24*5*33*11000+5.5*2*33*370}{5*33*11000+2*33*370} = 23.75 \text{ N/mm}^2$$

$$E_{0.05} = \frac{8800*5*33*11000}{5*33*11000+2*33*370} = 8683 \text{ N/mm}^2$$

LenoTec 264: walls 2 and 4

$$f_{m;0;k} = \frac{24*6*33*11000}{6*33*11000+2*33*370} = 23.73 \text{N/mm}^2$$

$$f_{c;0;k} = \frac{24*6*33*11000+5.5*2*33*370}{6*33*11000+2*33*370} = 23.79 \text{ N/mm}^2$$

$$E_{0.05} = \frac{8800*6*33*11000}{6*33*11000+2*33*370} = 8702 \text{ N/mm}^2$$

Accounting for a material factor $\gamma_M$=1.25 and a modification factor $k_{mod}$ = 0.8 (service class 1, medium-term), the following design values are found:

LenoTec 231: walls 1 and 3

$$f_{m,0,d} = f_{m,0,k} * \frac{0.8}{1.25} = 15.15 \frac{N}{mm^2}$$

$$f_{c,0,d} = f_{c,0,k} * \frac{0.8}{1.2} = 15.20 \frac{N}{mm^2}$$

LenoTec 264: walls 2 and 4

$$f_{m,0,d} = f_{m,0,k} * \frac{0.8}{1.25} = 15.18 \frac{N}{mm^2}$$

$$f_{c,0,d} = f_{c,0,k} * \frac{0.8}{1.25} = 15.23 \frac{N}{mm^2}$$

### 11.1.6 Design values of the member buckling resistance

The design values that have to be accounted for are determined making use of the following formulae:

$$\lambda_{ef;i} = l_{ef} * \sqrt{\frac{EA}{EI_i}}$$

$$\lambda_{rel;i} = \frac{\lambda_{ef;i}}{\pi} * \sqrt{\frac{f_{c;o;k}}{E_{0.05}}}$$

$$k_i = 0.5(1 + 0.1(\lambda_{rel;i} - 0.3) + \lambda_{rel;i}^2)$$

$$k_{c;i} = \frac{1}{k_i + \sqrt{k_i^2 - \lambda_{rel;i}^2}}$$

Accounting for these formulae and the sectional properties as shown in table 47, the following overview is obtained, see table 48 and table 49.

Table 48: Design values of member buckling resistance in EW-direction

| Level | Wall number | $\lambda_{ef;EW}$ | $\lambda_{rel;EW}$ | $k_{c;EW}$ | $k_{EW}$ |
|---|---|---|---|---|---|
| 0 | 1 | 58.34 | 0.971 | 1.002 | 0.80 |
| | 2/4 | 3.28 | 0.055 | **à** | 1.0 |
| | 3 | 60.08 | 1.000 | 1.032 | 0.78 |
| -2 | 1 | 47.11 | 0.784 | 0.829 | 0.91 |
| | 2/4 | 2.65 | 0.044 | **à** | 1.0 |
| | 3 | 48.51 | 0.808 | 0.849 | 0.90 |

Table 49: Design values of member buckling resistance in NS-direction

| Level | Wall number | $\lambda_{ef;NS}$ | $\lambda_{rel;NS}$ | $k_{c;NS}$ | $k_{NS}$ |
|---|---|---|---|---|---|
| 0 | 1 | 3.19 | 0.053 | **à** | 1.0 |
| | 2/4 | 54.76 | 0.911 | 0.943 | 0.84 |
| | 3 | 2.63 | 0.044 | **à** | 1.0 |
| -2 | 1 | 2.57 | 0.042 | **à** | 1.0 |
| | 2/4 | 44.22 | 0.736 | 0.789 | 0.93 |
| | 3 | 2.12 | 0.035 | **à** | 1.0 |

## 11.1.7 Check on flexural buckling of the individual walls

As mentioned in 11.1.1, it is sufficient to account for the combination of bending in one direction and vertical loading only:

$$\frac{\sigma_{c;0;d}}{k_{c;EW} * f_{c;0;d}} + \frac{\sigma_{m;EW;d}}{f_{m;EW;d}} \leq 1 \quad \text{and} \quad \frac{\sigma_{c;0;d}}{k_{c;NS} * f_{c;0;d}} + \frac{\sigma_{m;NS;d}}{f_{m;NS;d}} \leq 1$$

Accounting for these formulae and the values as shown earlier this chapter, the following is found, see table 50 and table 51.

Table 50: Flexural buckling in EW-direction

| Level | Wall number | $\frac{\sigma_{c;0;d}}{k_{c;EW} * f_{c;0;d}}$ | | $\frac{\sigma_{m;EW;d}}{f_{m;EW;d}}$ | | U.C. |
|---|---|---|---|---|---|---|
| 0 | 1 | 0.052 | + | 0.087 | = | 0.14 |
| | 2/4 | 0.043 | + | 0.155 | = | 0.20 |
| | 3 | 0.053 | + | 0.122 | = | 0.18 |
| -2 | 1 | 0.076 | + | 0.214 | = | 0.29 |
| | 2/4 | 0.072 | + | 0.381 | = | 0.45 |
| | 3 | 0.077 | + | 0.300 | = | 0.38 |

Table 51: Flexural buckling in NS-direction

| Level | Wall number | $\frac{\sigma_{c;0;d}}{k_{c;NS} * f_{c;0;d}}$ | | $\frac{\sigma_{m;NS;d}}{f_{m;NS;d}}$ | | U.C. |
|---|---|---|---|---|---|---|
| 0 | 1 | 0.041 | + | 0.028 | = | 0.07 |
| | 2/4 | 0.035 | + | 0.028 | = | 0.07 |
| | 3 | 0.041 | + | 0.017 | = | 0.06 |
| -2 | 1 | 0.069 | + | 0.067 | = | 0.13 |
| | 2/4 | 0.047 | + | 0.071 | = | 0.12 |
| | 3 | 0.069 | + | 0.040 | = | 0.11 |

## 11.1.8 Conclusion

§ With reference to table 50 and table 51, it is concluded that no problems are to be expected concerning flexural buckling of the individual core walls.

## 11.2 Structural behaviour in case of fire

In this section it is investigated how the structural behaviour of the core is influenced when it is under fire conditions.

### 11.2.1 Analysis

In case of fire, the core should still be able to resist the acting loads. Since the cores are the main load-bearing elements of the stadium structure, a fire resistance of at least 90 minutes is required (see B.1.2.2).

To determine the behaviour under fire conditions, use is made of the effective cross-section method. This method accounts for the capacity of the uncharred cross-section. When the cores are concerned, it is assumed that only the outer perimeter of the core walls are subjected to fire. The opening within the core is assumed to be enclosed by a fire resistant door.

The core walls are considered not being protected. All fasteners should be protected against fire at all times. This is not elaborated on in detail.

The design procedure that is accounted for in this chapter, follows from NEN-EN 1995-1-2. At first, the loads are determined.

### 11.2.2 Loads

According to NEN-EN 1995-1-1, the accidental combination applies for fire design. Accounting for the normal design values ($X_d$), the fire design values can be determined as $X_{d;fi} = \eta * X_d$.

Generally, it is conservative to reduce the acting loads with a value of $\eta = 0.6$ (NEN-EN 1995-1-2). With reference to the acting loads on the proposed core, see Annex C.1.6.2, the following results are found:

$N_{d;fi} = 0.6 * N_d = 0.6 * 4633 = 2780kN$

$M_{d;fi} = 0.6 * M_d = 0.6 * 26280 = 15768kNm$

$V_{d;fi} = 0.6 * V_d = 0.6 * 2743 = 1646kN$

### 11.2.3 Sectional properties reduced due to fire

In the case of fire, the cross-sectional area reduces due to charring. When the effective cross-section method is applied, the effective cross-section is determined from the effective charring depth:

$d_{ef} = d_{char;n} + k_0 d_0$ where $d_0$ = 7 mm

Since the core walls are unprotected, and require a fire resistance over 20 minutes, $k_0$ is taken 1.0

$d_{char;n}$ follows from the charring rate of the material (CLT: 0.7 mm/min) and the required fire resistance (90 minutes). This result in a charring depth of 63 mm, and an effective depth of 70 mm.

Considering the cross-section with openings as proposed in section 10.4, the wall thickness is reduced by this value at the outer perimeter, see figure 163.

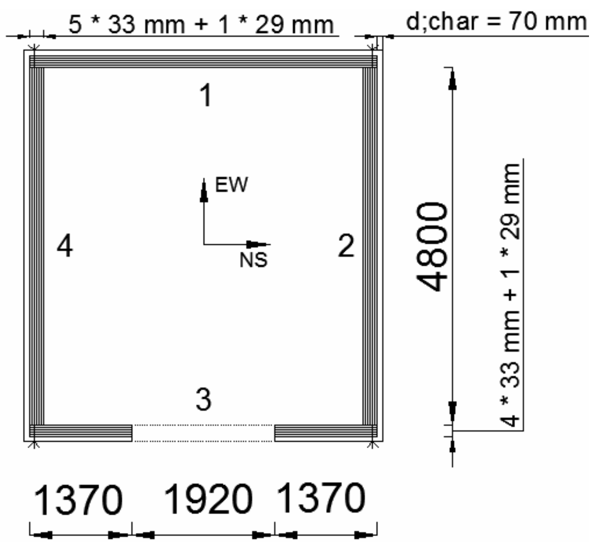

Figure 163: Reduced cross-section after fire

### § Strain stiffness

Accounting for the residual cross-section, the following values are found:

$EA_1$ = 11000 N/mm² * 4660 * 3 * 33 mm² + 370 N/mm² * 4660 * (29+33) mm²
= 0.518 * $10^{10}$ Nmm²

$EA_2 = EA_4$ = 11000 N/mm² * 4800 * 4 * 33 mm² + 370 N/mm² * 4800 * (29+33) mm²
= 0.708 * $10^{10}$ Nmm²

$EA_3$ = 11000 N/mm² * 2740 * 3 * 33 mm² + 370 N/mm² * 2740 * (29+33) mm²
= 0.305 * $10^{10}$ Nmm²

with $a_{y;1}$ = 2244mm
$a_{y;2} = a_{y;4}$ = 236.5 mm
$a_{y;3}$ = = 2717 mm
and $a_{x;2} = a_{x;4}$ = 2233 mm
$a_{x;1} = a_{x;3}$ = 0 mm

### § Slip modulus

The slip modulus in case of fire is determined as follows:

$K_{fi;u} = 0.67 K_u = 0.67 * 7778 = 5211$ N/mm (bolts, dowels)

### § Connection factors

Accounting for the slip modulus and the sectional properties under fire conditions, the connection efficiency factors are determined. The connection is equal to the one proposed earlier in chapter 10.

$$\gamma_{1;EW;u} = \left[1 + \pi^2 * \frac{EA_{fi;1} * s}{2 * K_{u;fi} * l_{ef}^2}\right]^{-1} = \left[1 + \pi^2 * \frac{0.518 \cdot 10^{10} * 150}{2 * 5211 * 38440^2}\right]^{-1} = 0.668$$

$$\gamma_{3;EW;u} = \left[1 + \pi^2 * \frac{EA_{fi;3} * s}{2 * K_{u;fi} * l_{ef}^2}\right]^{-1} = \left[1 + \pi^2 * \frac{0.305 \cdot 10^{10} * 150}{2 * 5211 * 38440^2}\right]^{-1} = 0.773$$

$$\gamma_{1;NS;u} = \gamma_{3;x;u} = \left[1 + \pi^2 * \frac{EAfi;_2 * s}{2 * K_{u;fi} * l_{ef}^2}\right]^{-1} = \left[1 + \pi^2 * \frac{0.708 \cdot 10^{10} * 150}{2 * 5211 * 38440^2}\right]^{-1} = 0.595$$

$$\gamma_{2;EW;u} = \gamma_{4;EW;u} = \gamma_{2;NS;u} = \gamma_{4;NS;u} = 1.0$$

§ Bending stiffness

Applying the same procedure as elaborated on in section 10.4, while accounting for the reduced cross-section, the following values are found:

$$(EI)_{ef;EW;fi;u} = \sum (EI)_{EW;i} + \gamma_{EW;i;u} * (EA)_i * a_{EW;i}{}^2 = 6.24 \cdot 10^{16} N/mm^2$$

$$(EI)_{ef;NS;fi;u} = \sum (EI)_{NS;i} + \gamma_{NS;i;u} * (EA)_i * a_{NS;i}{}^2 = 5.98 \cdot 10^{16}\ N/mm^2$$

## 11.2.4 Stresses

Making use of the formulae as presented here, the stress distribution as shown in table 105 is found. Since the EW-direction is governing, only this direction is taken into consideration.

$$\sigma_{m;i} = \frac{0.5 * E_i * h_i * M}{EI_{ef}}; \sigma_i = \frac{\gamma_i * E_i * a_i * M}{EI_{ef}}; \ \sigma_N = \frac{N * E_i}{EA_{tot}}$$

$$\tau_{1/3/5;max} = \frac{\gamma_1 * E_i * A_i * a_i}{b_i^a * EI_{ef}} * V; \ \tau_{2/4;max} = \frac{\gamma_1 * E_1 * A_1 * a_1 + 0.5 * E_2 * b_2 * (0.5h_2 + a_2)^2}{b_2{}^a * EI_{ef}} * V$$

Table 52: Stresses due to bending moment and normal forces at the residual cross-section (EW-direction)

| i | $\sigma_{m;EW;i}$ [N/mm²] | $\sigma_{N;EW}$ [N/mm²] | $\tau_{y;EW;min}$ [N/mm²] | $\tau_{y;EW;max}$ [N/mm²] | $\tau_{y;EW;min;3}$ [N/mm²] |
|---|---|---|---|---|---|
| 1 | +/- 2.75 | 0.86 | 0 | 0.64 | 0 |
| 2 | +/- 5.06 | 0.94 | 0.53 | 1.06 | 0.44 |
| 3 | +/- 3.81 | 0.86 | 0 | 0.52 | 0 |
| 4 | +/- 5.06 | 0.94 | 0.53 | 1.06 | 0.44 |

## 11.2.5 Verification of stresses

To verify the strength of the material under fire conditions, use has to be made of the following strength values:

$X_{d;fi} = k_{mod;fi} * k_f * \frac{X_d}{\gamma_{M;fi}}$ in which $k_f$=1.15 (CLT) and $\gamma_{M;fi}$=1.0

$k_{mod;fi}$ is 1.0 when the reduced cross-section method is applied.

With reference to section 10.6, it is stated that the acting stresses decrease while the strength values increase. Therefore, it is concluded that the cores comply with all requirements under fire conditions.

## 11.2.6 Conclusion

§ The cores are able to resist the acting loads under fire conditions for at least 90 minutes

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# Evaluation Detailed design

In the detailed design phase attention has been paid to the points of attention that followed from the preliminary design phase. Main reason to investigate these points of attention was to determine whether the timber core sysem is actually feasible.

In this section, the results obtained in the detailed design phase are being evaluated. Investigating the main points of attention, several possible solution have been found which, from a structural viewpoint, fulfil the requirements. Evaluating these solutions, the most beneficial solution is determined for the timber stadium structure.

From the detailed design phase it appeared that the most interesting aspects of the timber core are the division in segments and the choice of material for the core walls. Therefore, these subjects is elaborated on here. The focus here is on the consequences that the implementation of a certain solution holds for the dimensions of the elements and the method of construction.

## Division in segments of the core

From the preliminary design phase, it appeared that the feasibility of erecting a timber core with the proposed dimensions is questionable. It has therefore been decided to investigate what structural consequences it holds to divide the cores in storey-high segments. It appeared that such a division introduces several ‘problems’ which have to be accounted for in the design.

These problems apply to the transfer of forces at these horizontal joints and the overall stiffness of the core. Due to the division in segments, these forces are not automatically transferred from one segment to another.

In chapter 8, research has been carried out on several solutions to counteract these ‘problems’. From this chapter it appeared that two combined systems proved to be feasible:

§ A system that combines post-tensioned tendons and a shear plate
§ A system that combines glued-in rods and a shear plate

In this section, at first, some possibilities are mentioned to reduce the effects of the earlier mentioned problems. Thereafter, the solutions as mentioned above are summarized and evaluated.

### Reducing the effect of the problems

#### § Increasing the amount of vertical loading

Preferably, adaptations to the design are prevented to the greatest extent. This can be achieved by decreasing the acting tensile stresses, i.e. by increasing the vertical loading on the core.

Considering the structural build-up as presented in the Preliminary design phase, it is mentioned that there is accounted for columns close to the cores. These columns support the beams which in turn support the floor elements.

By removing these columns and by connecting the support beams directly to the cores, the vertical loads of the floors are transferred to the core walls instead of to the columns. The vertical load thereby increases, resulting in smaller values for the tensile stresses.

Despite that, it is highly questionable whether the vertical loads can be increased to such an extent that the effect of the tensile stresses disappears. This solution should therefore be considered in combination with other design adjustments. In this section, there is not accounted for this possible increase in vertical loading.

Note 1: As a consequence of this solution an additional bending moment is introduced at the core, due to the eccentric point of action of this vertical load. Compared to the bending moment due to the horizontal loading this additional moment is considered negligible.

Note 2: the vertical load that is accounted for in the design calculations was determined from a preliminary weight calculation and the functional plan of the Euroborg stadium. Accounting for the structural build-up as proposed at the end of the preliminary design phase, it appears that the vertical loading becomes even smaller (see Annex G).

§ Accounting for friction between the segments

In preceding design calculations, it was assumed that additional measures are required to transfer the acting horizontal force that acts on a core from one segment to another. In reality, part of this load is automatically transferred by means of friction.

At those parts of the core that are in compression, the segments are 'pushed' against each other. Due to the course edges of the material, at these parts, resistance is offered against horizontal movement. It is assumed that this friction force amounts 50 to 60% of the total compression force. As a consequence, the shear force reduces to a great extent.

It is recommended to investigate the effect of friction in greater detail, since a lot of benefits could be reaped, i.e. less fasteners required, other solutions become feasible, etc.

## Combination of post-tensioned tendons and a shear plate

§ Principle behind the solution

From chapter 7 it appeared that the implementation of steel bars was sufficient to transfer the acting tensile stresses, but that problems were expected concerning the stiffness of the proposed (section 7.4.2.). Therefore, the solution has been altered to some extent, by introducing post-tensioned tendons instead of anchorage bars (section 7.4.4).

Accounting for such a solution, the acting tensile stresses are counteracted by applying a compressive stress with a magnitude that is at least equal to these tensile stresses. As a consequence, the complete core is in compression, whereby the timber cross-section provides the required stiffness again.

To transfer the acting shear stresses, a shear plate is applied over the horizontal joints. This plate is incorporated partly within the core wall and is connected t the wall both above and underneath the joint. This solution has been discussed in section 7.3.1 and proved to be the most beneficial solution.

Implementing such a solution the tensile and shear stresses are dealt with, while the core holds sufficient stiffness to comply with the requirements on deflections.

§ Implementation

Accounting for the maximum acting tensile stress at the governing walls (2/4), a first analysis showed that at least 12 tendons (FeP 1860, d = 15.7 mm) are required at each side of these walls.

To connect the shear plate to the core walls, a total of 168 screws (d = 20 mm) is required both underneath and above the governing horizontal joint at wall 2/4.

§ Consequences on the element dimensions

Since there is accounted for pre-stressing tendons within the segments, all walls should be provided with the required amount of ducts. These ducts should already be made at the manufacturer.

For the proposed solution each segment is anchored individually, which implies that there has to be accounted for the possibility the tension the tendons after erection of the core. This is accounted for by incorporating recesses at the bottom of each wall. These recesses are again assumed to be made at the manufacturer.

To be able to incorporate the shear plate within the core walls, there has to be accounted for a reduction in thickness at both the top and bottom of each segment. This reduction amounts 90 mm, which is equal to the thickness of an individual LVL layer.

§ Consequences on the construction method

Since the core is divided in segments, the whole structure has to be erected segment after segment. This implies a lot of additional work, especially since the pre-stressing tendons have to be applied.

After placement of each individual segment, the pre-stressing tendons have to be placed within the ducts. Consecutively, these have to be screwed in the coupling nut at the bottom side. At the topside they have to be anchored to the segment or provided with a coupling nut. Finally, the tendons are pre-stressed after which the next segment can be hoisted in place.

When two segments are placed on top of each other, the horizontal joints are sealed by the shear plates. These plates are connected to the wall segments by making use of a large amount of screws (at least for the governing horizontal joint), while accounting for the presence of the recesses behind these plates.

§ Points of attention

The main draw-back for this solution is the state of knowledge on pre-stressed timber structures. Only few companies/institutes perform research on the subject and the obtained knowledge is not yet translated in design standards.

## Combination of glued-in rods and a shear plate

§ Principle behind the solution

From section 7.2.6 it appeared that glued-in rods are perfectly capable of transferring the acting tensile stresses. Since the rods are only of influence in the direct environment of the horizontal joints, it appeared that no problems concerning the stiffness are to be expected (section 7.4.4). The stiffness reduces only locally, near the joints, whereby the average stiffness is still sufficient.

To transfer the acting shear stresses, again, a shear plate is applied over the horizontal joints. This plate is incorporated partly within the core wall and is connected t the wall both above and underneath the joint. This solution has been discussed in section 7.3.1 and proved to be the most beneficial solution.

§ Implementation

Accounting for the maximum acting tensile stress on the governing walls (2/4) at the governing segment (-2$^{nd}$), a first analysis showed that at least 21 rods (M20, 8.8, 400 mm long) are required.

To connect the shear plate to the core walls, a total of 168 screws (d = 20 mm) is required both underneath and above the governing horizontal joint at wall 2/4.

§ Consequences on the element dimensions

The rods are glued-in at both sides of the horizontal joints. Therefore, all segment walls should be provided with the required amount of oversized holes, having with a length of at least 200 mm each (such value was accounted for in the design).

To provide sufficient bond between the rods and the glue, use is made of treaded rods. The glue is injected over the whole length of the rods through so-called injection holes which are accounted for in the walls.

To be able to incorporate the shear plate within the core walls, there has to be accounted for a reduction in thickness at both the top and bottom of each segment. This reduction amounts 90 mm, which is equal to the thickness of an individual LVL layer.

§ Consequences on the construction method

When placing one segment on top of another, there has to be accounted for the rods. It is assumed that the rods are already glued in the lower segment at the manufacturer, whereby these rods can be guided in the holes of the upper elements. Afterwards, the holes of the upper segment have to be injected to provide sufficient bond.

When two segments are placed on top of each other, the horizontal joints are sealed by the shear plates. These plates are connected to the wall segments by making use of a large amount of screws (at least for the governing horizontal joint), while accounting for the presence of the rods within the walls.

## Conclusion on division in segments

Accounting for the above, it is concluded that the combined system of glued-in rods and a shear plate is most beneficial in this situation.

Main benefits of this system are the speed of erection and the lack of major adaptations to the wall elements.

## Material choice

The core walls as proposed in the preliminary design phase were composed of LVL elements. Due to the limited available dimensions of this product, additional measures are required to provide the core walls with the required width. It has therefore been investigated in chapter 10 whether the core walls could be composed of CLT elements as well.

In this section, it is being evaluated what consequences the implementation of each material holds and what material is considered most beneficial.

## Core walls made from LVL elements

### § Implementation

With reference to figure 164 it is stated that all core walls have a width of 5.0 m. Due to the maximum width of the LVL elements of 2.5 m, the required width is only reached by interconnecting the elements.

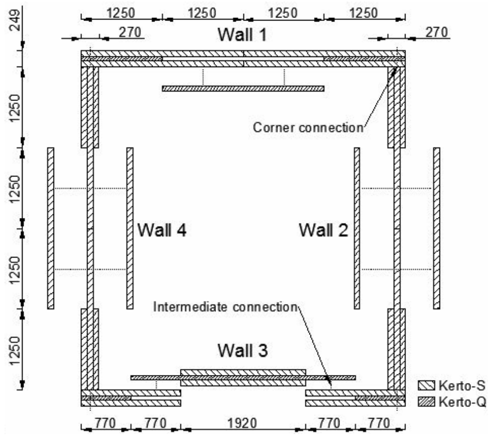


Figure 164: Proposed LVL core

In chapter 9, a proposal has been made for such an intermediate connection. Considering wall 2/4, it appeared that at least (2 times) 20 glued-in bolts (M16, 8.8) were required each linear meter, to be able to transfer the maximum acting shear stress.

To be able to transfer the loads acting at the lintels above the openings, a Kerto-Q layer is accounted for in walls 1 and 3.

### § Consequences on the structural behaviour

Since the elements are interconnected by means of fasteners, there has to be accounted for a certain amount of slip. This influence is accounted for by means of the connection efficiency factor. Accounting for (2 times) 20 glued in bolts each linear meter, it appears that the stiffness of these walls reduces to about 87% of its original value. Considering the core as a whole, the reduction in stiffness is about 5%.

### § Consequences on the construction method

To provide a wall with the required width of 5 m, several elements are to be interconnected. This implies the implementation of a lot of fasteners, which results in an increased time to fabricate these walls.

In addition to that, the implementation of these fasteners might interfere with other adaptations to the design, such as the earlier mentioned implementation of a shear plate.

## Core walls made from CLT elements

### § Implementation

With reference to figure 165 it is stated that all core walls have a width of 4.8 m, which is due to the maximum width of the CLT elements.

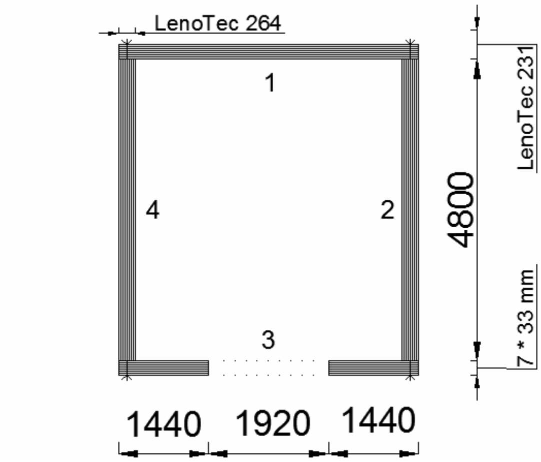


Figure 165: Proposed CLT core

### § Consequences on the structural behaviour

Due to the maximum element width, the dimensions of the core are limited to 4.80 by 5.26 m. As a consequence the second moment of area reduces to some extent, compared with the LVL core as proposed in chapter 4.

In addition, the CLT elements are composed from laminates that span both in longitudinal and transverse direction. Since the transverse laminates hold little stiffness in longitudinal direction, the stiffness of the elements is considerably smaller than for CLT elements.

As a consequence the bending stiffness of the core is smaller than that of the proposed LVL core. Despite that, the CLT core complies with the requirements largely. The same is valid for the strength of the core.

### § Consequences on the construction method

Since the CLT elements are available in the required dimensions, no additional measures are required. As a consequence, the cores can be composed of four single elements. This increases the speed of building and limits the costs.

## Conclusion

In the preceding section, the consequences of implementing LVL elements and CLT elements have been mentioned. Accounting for these consequences, it is concluded that the CLT elements offer the most beneficial solution.

Main reason is found on the field of construction: it is regarded highly beneficial to make use of standardized elements, without the need to make a lot of adaptations.

Regarding the reduced strength and stiffness properties of CLT, it is concluded that the proposed cores comply with all requirements. The material properties are therefore not considered a draw-back for the use of these elements.

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# Discussion

In this final part of the thesis, an answer is provided to the main research question and the obtained results are summarized.

At first, the proposed design is presented: consecutively the structural system, the floor system and the design of the grandstands are discussed.

Thereafter, an answer is provided to the research question while accounting from the design as proposed earlier.

Subsequently, the timber stadium structure is compared with a ‘traditional’ concrete stadium. Although it is no purpose of the thesis to make such a comparison in a qualitative way, it is still regarded interesting to qualify the most important differences.

This part is concluded by making some recommendations to improve the proposed design.

# Proposed design

## Structural system

Three timber cores are implemented to stabilize the Eastern building part of the stadium structure, see figure 166. These cores are composed from 4 walls which are interconnected at their edges. These walls are made from Cross Laminated Timber (CLT) elements, see figure 167.

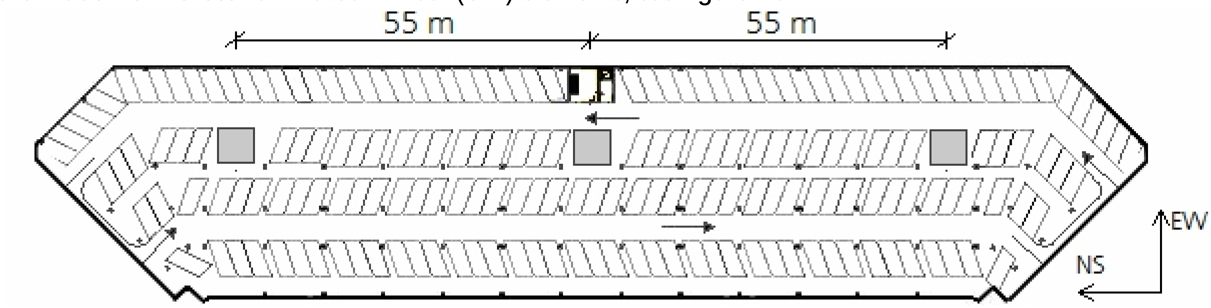


Figure 166: Arrangement of cores at the Eastener building part

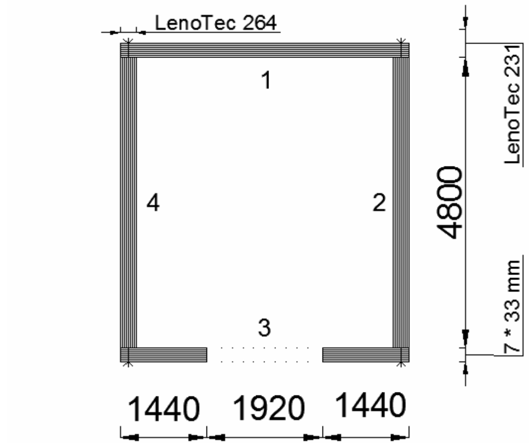


Figure 167: Build-up of CLT cores

When the core is divided in storey-high segments, additional measures are required to transfer the acting stresses at the horizontal joints and to provide the core with adequate stiffness.

To transfer the acting shear stresses there is accounted for a Kerto-Q element which is laid over the horizontal joints, see figure 140. These plate are connected to the core by means of screws. The total amount of screws required might be reduced to a considerable extent by accounting for friction between the timber segments.

The acting tensile stresses are transferred by means of glued-in bolts, which are implemented at the joints, see figure 138.

The stiffness of the core is provided by the combined system of the glued-in bolts (at the horizontal joints) and the timber segments.

## Floors

The floors within the stadium structure are made from Lignatur elements, which are arranged in a so-called stacked bond pattern. As a consequence, these elements span between both façades at the upper floors of the stadium structure, see figure 168.

The floors are designed to act as diaphragms, wherefore these are interconnected by means of dowels. The diaphragm is enclosed by chords which are able to act as tensile ties, see section 5.5.4.

The Lignatur elements are supported by support beams made from Glued Laminated Timber. At the lower floors, these elements span over 4 support beams. At the upper floors, only three supports beams are required, figure 168.

The optimal span-width of the floor support beams is 7.8 m, which fits the functional plan of the stadium at best.

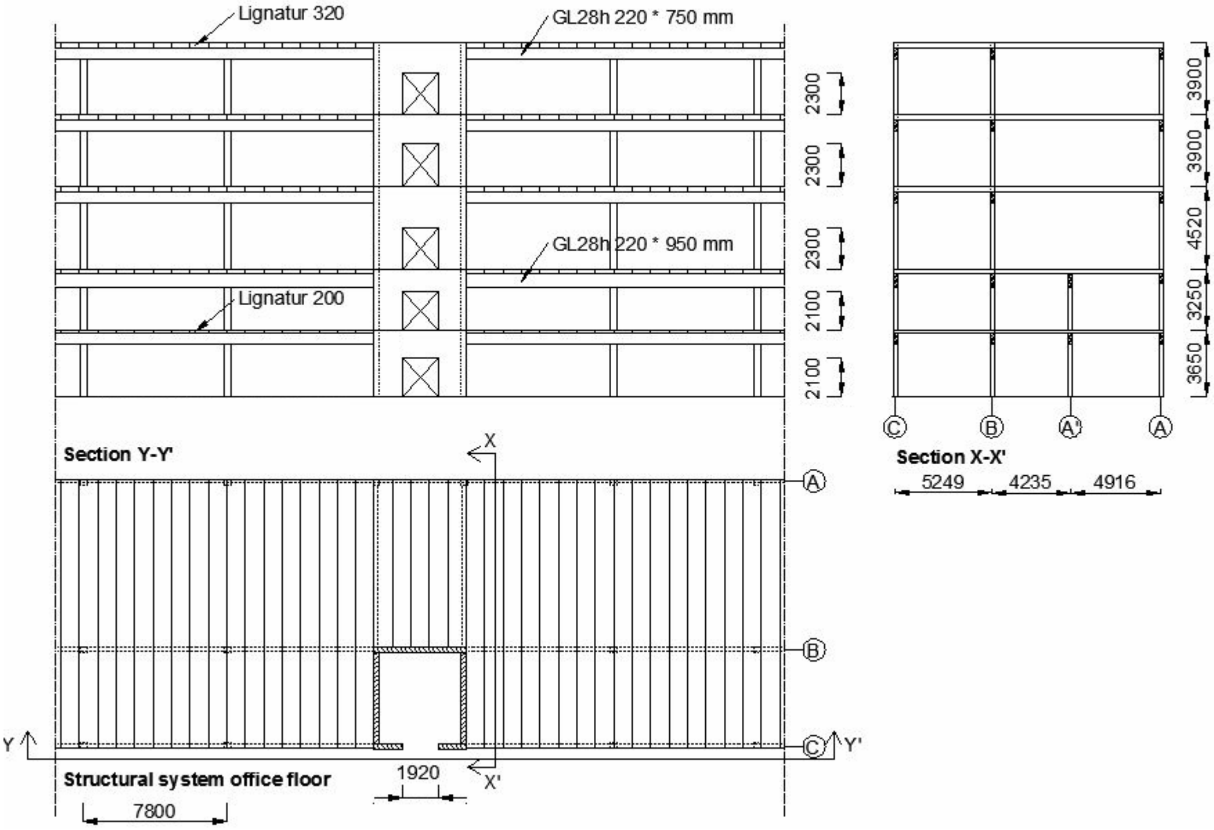


Figure 168: Structural build-up of the proposed floor solution

## Grandstand

The grandstand consists of two structural elements, being the grandstand elements and the grandstand support beams. The grandstand elements span in between the support beams and provide the terraces which accommodate the seating.

The grandstand elements are composed of 69 mm thick Kerto-Q treads and 90 mm thick Kerto-S risers. The external shape of these elements follows from the requirements on visibility (the C-factor). To reach the optimal span-width of 7.8 m the risers are enlarged. Since the reachable span-width follows from the combined system of support beams and grandstand elements, the support beams should be designed with sufficient stiffness.

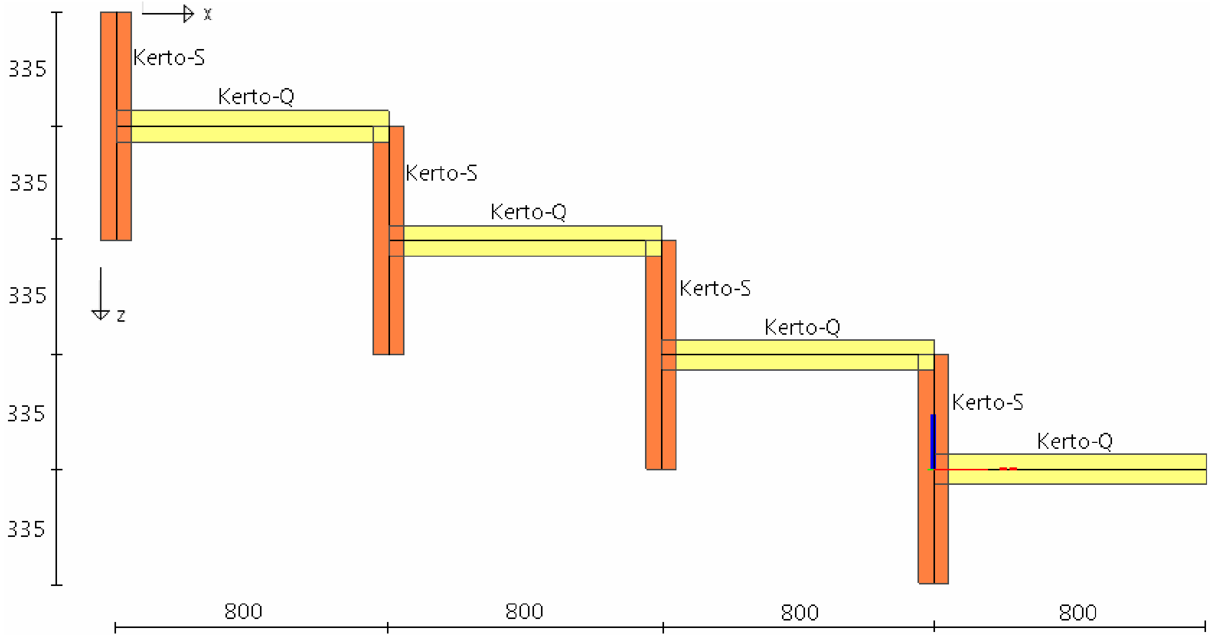


Figure 169: Grandstand element with enlared risers (indicative)

# Research question

The research as presented in the preceding parts of this thesis was carried out to provide an answer to the research sub questions and the main research question. The main research question, as presented in the introduction, is formulated as:

*To what extent is the utilisation of structural timber in football stadium engineering feasible?*

Accounting for the knowledge gained by answering the subquestions, it is stated that there are no significant structural drawbacks to abandon the utilisation of timber for any of the structural elements that have been investigated.

For the structural system, the floors and the grandstand a feasible solution has been found which complies with the requirements set. The optimal span-width of the timber stadium structure is 7.8 m.

Although there a no structural drawbacks, one should be aware of the fact that the implementation of the proposed solutions does influence the architectural design, i.e. the grid dimensions differ from those of the ‘traditional’ stadium.

# Evaluation stadium structure in timber

In this thesis it has been investigated whether it is feasible to engineer a stadium structure that is composed of timber elements. Although it is no goal of this thesis, it is interesting to compare the proposed timber structure with the 'traditional' stadium structure, as discussed in chapter 2. In this section an overview is provided on the main benefits and draw-backs of the timber stadium structure.

### + Lightweight

One of the main benefits of timber is found in its considerably small self-weight (about 1/5$^{th}$ of concrete). As a consequence, the loads on the structure follow from the imposed loads rather than the dead load. Another advantage is found in the handling at the building site, which becomes easier. On the other hand, as discussed earlier, the low self-weight of the cores can also be regarded a disadvantage.

### + Prefabrication

Since large timber elements are introduced at the core walls, building time increases significantly compared with the situation where the cores are cast in-situ.

### + Reusable and adaptable

When designed properly, the timber stadium structure holds potential to be adapted and reused. By adaptation a change in the functional plan as well as the possibility to reassemble and rebuild the structure is meant. When dismantling the structure, the elements are not destructed whereby these can be reused at other structures.

### + Aesthetically appealing

Although this is a subjective criteria, it is stated that timber is often regarded as being aesthetically appealing. This provides the structure with an attractive and warm appearance.

### + Sustainability

As elaborated on earlier, timber is a naturally renewable material which is regarded highly sustainable, on the condition of being harvested both legally and with a sustainable approach. The production of the timber elements requires only a small amount of energy compared with concrete or steel elements. In addition, the contribution to the greenhouse effect is only marginal.

### – Grid dimensions

Compared to the 'traditional' stadium structure it appears that the grid dimensions decrease to some extent: the floor elements are not able to span the required 14.4 m at once, while in the other direction the columns have a smaller centre-to-centre distance. As a consequence, more support elements should be implemented in the functional plan. From a principal's point of from view, this is regarded a negative consequence since the net floor area reduces. In addition, the speed of erection reduces due to the implementation of an increased amount of structural elements.

### – Connections

When the structure is composed of timber elements a lot of attention is required for the connections. Generally, these connections are mechanically jointed by means of fasteners, which implies additional work on site.

# Recommendations

## Structural design

### Loads

§ Accounting for updated loads
The (vertical) loads that were accounted for in the design calculations have been determined from the functional plan of the Euroborg stadium. To provide a complete design, use should be made of the 'updated loads' which follow from proposed design, e.g. the preliminary design, see Annex G.

### Structural system

§ The dimensions of the cores
The cores are designed with dimensions that are taken as a first assumption. These cores comply with the requirements largely, wherefore it is recommended to optimise the dimensions.

§ Research on the pre-stressing of timber elements
From the detailed design phase it appeared that the timber core should be pre-stressed to provide sufficient stiffness. Since there is a lack of design standards on this subject, it is recommended to perform a thorough research on the knowledge available and to perform additional research when required.

§ Alternative solutions to transfer the shear forces at the horizontal joints
Although a feasible solution was reached by using a connector plate, a large amount of fasteners was required. It is therefore recommended to investigate alternative solutions, or at least, to strive for an optimised design.

### Floors

§ The span-width of the support beams can be increased
The maximum span-width of the support beams follows from the required free height underneath these beams. Implementing beams having a larger stiffness (e.g. steel beams) or multiple beams adjacent to each other, the maximum span-width can be increased. As a consequence, fewer columns are to be implemented in the design.

§ Diaphragm-action
Diaphragm-action of the floor has been considered by making use of a simplified analysis. This analysis accounts for the maximum acting loads, rather than the actual loads. Improvements can be gained by performing a more in detail analysis.

### Grandstands

§ The connection between the treads and the risers
In this thesis it was assumed that this connection was infinitesimally stiff, without elaborating on the connection itself. It has to be investigated how such a connection can be executed and what stiffness can be obtained.

§ Detailing of the connection between the grandstand element and the support beams
The grandstand elements are supported by the grandstand support beams. In this thesis no attention was given to the detailing of such a connection. Presumably some kind of notch should be added to the support beams. It is recommended to pay attention to this subject.

§ Choice of material
The grandstand elements were designed while being composed from LVL elements. The dimensions of the elements (treads, risers) were based on the functional requirements and the available element dimensions. It is recommended to investigate the behaviour of other plate-like timber materials.

The same holds for the support beams: these are designed being glued laminated timber elements. The dimensions of these support beams become rather large, while these also deflect somewhat. An improved behaviour can be obtained by making use of stiffer materials, such as steel.

§ Behaviour on vibrations

It appeared that vibrations are governing for the design of the timber grandstand. In this thesis use has been made of a highly simplified analysis. It is therefore recommended to evaluate the obtained results with experts on dynamic behaviour. The natural frequency was taken to exceed 5Hz. Experimental research should be carried out to determine whether such value is acceptable for the timber grandstands.

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Figure 3: Stadio delle Alpi, 1990, Turin

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Figure 4: Gelredome, 1996, Arnhem

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Figure 6: Pitch lay-out according to FIFA regulations

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Figure 10: 'True' space frame

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Figure 11: 'Simulated' space frame

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Figure 17: Explanation of the C-value

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Figure 35: Cantilevered roof at the Autzen stadium (Eugene, USA)

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Figure 36: Atlantico Pavilion (Lisbon, Portugal)

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Figure 37: Typical section of the Atlantico Pavilion

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Figure 39: Savill Building (Windsor, United Kingdom)

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Figure 43: Expotheater CineMec, Ede (The Netherlands)

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Figure 44: Section of the grandstand at the Expotheater of CineMec

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Figure 46: Axonometric view of the Stadthaus

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Figure 47: Central core

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Figure 48: Stability by means of bracings

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Figure 68: Lignatur Kasten Element (LK-element)

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Figure 69: Lignatur Flachen Element (LF-element)

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Figure 70: LF-element including a fire protective layer and a sound absorbing layer

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Figure 71: BSP Crossplan element

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Figure 73: Decke element

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