The Use of Timber in a Temporary Multi-storey Car Park

Proof of Concept for a Structural System for a Temporary Multi-storey Car Park

P. Brink

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Proof of Concept for a Structural System for a Temporary Multi-storey Car Park

by

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Preface

This report presents my research on the applicability of timber in a temporary multi-storey car park. It is a master thesis report, which for me concludes my time as a student at the TU Delft. During my bachelor in civil engineering and master building engineering, I have become more observant for the role of the building industry in global warming. Furthermore, timber structures have always intrigued me with their beautiful and warm appearance and in recent years, building in timber has really taken off. This has triggered me to search for a topic for my thesis, related to building with timber, which has become the use of timber in a temporary multi-storey car park.

Apart for my interest in timber, in my master, I have developed an interest in using digital tools in the field of structural engineering. I'm motivated to automate repeating tasks and search for new insights using for example parametric engineering in Grasshopper. Although this is not the topic for my master thesis, it was an important aspect for me to use such digital tools during my thesis and develop new skills in the digital field of structural engineering. This has worked out and as a result, I have gained new insights in the applicability of digital tools in structural engineering as well as its current limitations, motivating me to help further develop this field in my future career.

Conducting this research would not have been possible without the members of my committee and various colleagues from Sweco. First of all, I would like to thank Erwin Jacobs for his close commitment to the project and his personal guidance. Our weakly Fika's have not only helped me with my thesis, but were also always a source of inspiration for what is possible in the field of digital engineering. Second, I'd like to thank Steven Janssens for his guidance and expertise in structural engineering, helping me further during our Fika's. Also a big thanks to Mariana Popescu for her help in narrowing down the research questions and sharing her view on the application of digital tools in engineering. I would like to thank Sander Pasterkamp for his critical eye on the structural calculations, keeping me sharp. Finally, also thank you to Sander van Nederveen for his help on the different views of the design and research process of a proof of concept.

At this point I would also like to say a big thank you to all my friends. Writing a master thesis is a long process during which you come across many challenges. It is great to always feel the support of the people close around you and all the great times together are extremely motivating to continue the work. Finally, I would also like to thank my parents, brother and sister for their support during not just one, but two master thesis projects and of course also the rest of my study time.

Concluding this preface, this report marks the end of my time as a student in Delft. Studying has been an amazing journey, which started with my bachelor at the TU Delft. Joining DSB has given me opportunities to develop me further on personal skills and hobbies with as a highlight a full-time year without studying as the board of the fraternity. It has always felt as a second home to me, where I have made friends for the rest of my life. Studying during the COVID-19 pandemic has been weird, but also has a positive outcome as with this thesis I'm finishing my second master. For now the time has come to move on to the next phase and say goodbye to Delft. Enjoy reading this thesis!

P. Brink

Delft, March 2023

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Abstract

Problem definition: In the car park industry, a new trend has emerged, shifting the demand for parking from permanent one-of-a-kind parking structures towards temporary modular structures (Drenth, 2020). Meanwhile, the building industry faces a transition into a more sustainable industry. This requires rethinking of the design process, taking into account possible reuse of a building or its elements as well as potential use of different materials. Timber is a renewable material and as a result of developments in timber engineering, now allows the construction of taller and larger structures. Given these trends and challenges, the objective of this thesis is to provide a structural system proof of concept for a temporary multi-storey car park, using timber as primary structural material.

Approach: A broad literature study provides background information on the various relevant topics and is followed by a refinement of the objective, substantiated by two sub-studies. This has resulted in two sub-goals: the minimization of total deck height (< 700 mm) and minimization of the structural systems weight (column load < 1,000 kN). A proof of concept is developed through a parametric study with structural validation into four main design variants (consisting of a deck and framing system), for which the effect of altering four geometric parameters (parking deck span, use of struts, distance between columns in transverse direction and minimum distance between joists) on these sub-goals is studied. Furthermore, a sensitivity analysis is performed on the effect of altering the requirements concerning fire-safety, vibrations and deflections on the deck height and weight. Finally, the proof of concept's performance is compared to five alternative modular and/or timber car park concepts in a case study.

Evaluation: It is concluded, that design variants using long-span deck systems result in a relatively small deck height, but high self-weight. A CLT rib deck should be applied under "standard" assessment criteria, but significant reductions in deck height (31%) and weight (49%) can be achieved by using LVL rib decks when vibrations are not considered and deformation limits are increased. The total deck height of design variants with short-span decks, which use solid LVL panels, highly depends on the supporting framing system, but its self-weight is roughly halve of the long-span design variants. The effect of alternative assessment criteria on design variants with short-span decks is rather limited. It is concluded that the use of struts significantly reduces the total deck height (17-30%) and structures weight (5-11%) and a transverse column distance of a single parking bay width is preferred. The distance between joists is of less relevance, but a bigger parking deck span increases both deck height and total weight.

The resulting proof of concept consists out of a short-span solid LVL deck system, which spans two times 2.5 m (single parking bay width). It is supported by glulam main girders, spanning the entire parking deck, which are simply supported by a set of columns at a transverse distance of 2.5 m and are additionally supported by slanted struts between the columns and main girders.

The results from the case study show, that the total deck height of the proof of concept (666 mm) is smaller that the sub-goal of 700 mm and comparable to alternatives with decks made of steel, concrete and GFRP. Furthermore it is 40% thinner than the alternative in timber as a result of the use of LVL and struts. In terms of weight, the use of timber is much lighter than using concrete decks (<25%) and only 17% heavier than the extremely lightweight GFRP decks. Furthermore, it results in the smallest loads on the foundation (678 kN which is smaller than the sub-goal of 1,000 kN). A first extremely simplified durability analysis on amounts of embodied carbon in the main structural elements indicates timber has significantly lower amounts of embodied carbon in comparison to the alternative concepts.

Implications: Based on the results presented, it is concluded that timber is applicable for almost all main structural elements of a temporary and modular multi-storey car park. The case study has shown that the proof of concept can compete with alternative modular car park concepts in terms of deck height and outperforms most alternatives in terms of the structures weight. It is also indicated, that timber has the potential to significantly improve the performance of a structure in terms of sustainability.

For future research, it is advised to further develop the proof of concept, focussing on the connections and durability risks and study the incorporation of circular design strategies into the design and construction process of the proof of concept. Furthermore, alternative assessment criteria should be validated. Finally, it is advised to study the potential use of other innovative sustainable building materials like BauBuche and make a detailed LCA study assessing the performance on sustainability.

PART I:
Introduction

1
Introduction

1.1 Background and Relevance

Nowadays, a car park may no longer be seen as a permanent structure, but rather as a construction to facilitate in a temporary demand for parking. This trend has various origins. For example, when large building complexes (like hospitals) are expanded, resulting in less remaining area for parking or temporary demand for alternative parking, a temporary multi-storey car park is required during the construction process (Bekkering, 2017). Similarly, large redevelopment projects in cities can result in a decrease in parking space for multiple years, which can be solved through a temporary car park (E. Jansen, 2013; Mobiliteits Platform, 2013). Also uncertainty in the future need for parking area may result in in a demand for car parks, which are fast and easy to construct and can easily be expanded, scaled down or removed (Drenth, 2020). According to McKinsey (Möller, Padhy, Pinner, & Tschiesner, 2019), four main trends will influence the car industry and this uncertainty: autonomous driving, connectivity, electrification and shared mobility. These trends will have an influence on the role and location of car parks. For example mobility hubs and P+R facilities at the edge of cities are expected, shifting demand for parking area from city centres to its edges (Dekfordt, 2019). All of these trends showcase an increase in future demand of temporary car parks.

A temporary construction has multiple advantages, like shorter processing times for building permits (Drenth, 2020) and less strict building requirements. As a result, a simpler and cheaper construction can be built. However, it can be a waste of resources to demolish a construction already after a short period of time, especially if these materials can no longer be reused or recycling requires a lot of extra steps (Kralj, 2008). Therefore, in the design process of a temporary car park, it is desirable to design for the long term and think about the demountability and possible reuse of the structure.

Thinking about the end of life phase of a building in the design process becomes an important theme as the building industry has to make a transition towards a more sustainable sector. The building sector was responsible for 35% of the global final energy consumption in 2019 and 38% of CO_2 emissions (United Nations Environment Programme, 2020) and therefore has to take action. This transition towards sustainability requires rethinking on how and which building materials are used, how buildings are designed, the construction and fabrication process and the end of life of a building.

Recently, the building industry has shown a lot of interest towards the use of timber as a structural material (Roberts, 2020). There are several reasons to use timber in constructions. First of all, timber has been around in the building industry for ages (Woods, 2016). In 2004, Eurocode 1995: Design of timber structures was released in Europe, which has been updated multiple times since its release, broadening the potential to use timber in larger and higher structures (Nederlands Normalisatie Instituut, 2011). Timber is a renewable material and can be carbon positive. Furthermore it performs well in fires and earthquakes, is aesthetically appealing and allows faster construction with lower labour costs, producing less waste (Wood for Good, 2011). This makes it a material with a high potential.

The fabrication and construction process of the industry is changing. The focus is increasingly turning to aspects such as building speed, the use of prefabrication and implementation of digital tools (Box, 2020). Prefabrication can lead to better working conditions, higher quality products and faster construction on the building site, while producing less waste and using less energy, but resulting in higher productivity (Bertram et al., 2019). Digital tools can help optimize and automate this process.

1.2 Research Definition

1.2.1 Problem Statement

To provide in the quest for a temporary car park designed for reuse, different mostly steel, modular and demountable structural systems with concrete decks have been developed (Ballast Nedam Parking, 2011; Park4all, 2019a). These concepts are designed using a new fabrication and construction process. The result is a car park, which can be dismantled and reused and therefore make full use of the maximum lifespan of its construction elements, which make the structure a more sustainable option. The modularity of the concepts results in larger amounts of prefabrication and shorter building times. Concepts combining steel and concrete in their structural systems have therefore proved to be able to provide a solution to the temporary character of a car park, while also embracing the changes in the fabrication and construction process of the building industry. However, concerning the challenges related to sustainability faced by the industry, it might be possible to further reduce the use of energy and production of emissions. Question is whether timber, considering all of its potential advantages, can fulfil the role of structural material in a car park with a temporary character and help further reduce the footprint of the structure. This results in the following problem statement:

The temporary character of car parks results in only a short required lifespan of its building elements, resulting in a waste of resources. Meanwhile, the building industry faces a sustainability transition, challenging it to rethink which and how building materials are used and how the production and construction process is organized. Question is if and how timber, considering its high potential, can fulfil a roll in both challenges as a structural material.

1.2.2 Objective

The aim of this thesis is to make a preliminary design of a structural system for a temporary multi-storey car park, incorporating the challenges faced by the building industry. Therefore, the technical feasibility of the use of timber as a primary structural material in the design is checked as well as how changing production processes influence the design of building elements. This results in the following objective:

Provide a structural system proof of concept for a temporary multi-storey car park, using timber as primary structural material.

1.2.3 Research Questions

Following the objective, the main research question is defined as:

“To what extent can timber be applied as a structural material in the structural system of a temporary multi-storey car park and how does it compare to alternative concepts?”

The main research question is supported by the following four sub-questions:

1) How to design a structural system for a temporary car park using timber as primary structural material?
    a. *What are the design requirements for a multi-storey car park?*
    b. *What are the possible typologies for a multi-storey car park?*
    c. *How to construct using timber as a structural material?*

2) How do the challenges faced by the construction industry towards a sustainable industry influence the design of a temporary multi-storey car park?
    a. *Which sustainability building strategies can be applied on the design of a temporary multi-storey car park?*
    b. *What is the impact of sustainability building strategies on the design of building elements?*

3) How do different typologies for a structural system for a temporary multi-storey car park using timber as primary structural material structurally perform?
    a. *What characteristics of a temporary multi-storey car park using timber as primary structural material are relevant to assess its structural performance?*
    b. *What level of structural performance is requested from the design for a temporary multi-storey car park?*
    c. *What is the impact of altering geometric parameters and limit state requirements on the performance of the different designs for a temporary multi-storey car park using timber as primary structural material?*

4) How does the proof of concept for a temporary multi-storey car park, using timber as primary structural material compare to alternative modular and/or timber multi-storey car park concepts?

1.2.4 Scope

This research is performed as a proof of concept and therefore, only the preliminary design phase is considered. To be able to answer the research questions within the timeframe of a master thesis, the following scope is defined:

- **Materials:** Timber is considered as the primary building material, but other materials are not excluded.
- **Superstructure:** The main elements of the superstructure of the structural system of the multi-storey car park are considered. These include a deck system and a framing system including stability bracings.
- **Foundation:** The substructure of the structural system is briefly considered in terms of a range of desired maximum foundation loads.
- **Connections:** Connections are only considered in terms of the types of connections. They are not modelled into detail.
- **Weather conditions:** Effects of temperature, moisture and protection are only considered in terms of their effect on the structure and required protective measures. The effects are not modelled into detail.
- **Sustainability:** Possible sustainability strategies are considered. A brief sustainability performance assessment is made, with the goal to show the potential of the proof of concept. No further detailed LCA analysis is conducted.
- **Façades:** Only the loads coming from the façades and acting on the structural system are considered. No façade design is made.
- **Finishing:** Items like installations, plumbing and electrical equipment are not considered.
- **Regulations:** The European norms and regulations (Eurocodes) are applied.
- **Fire safety:** Fire safety of the car park is only considered briefly. General effects of fire on the deck system are taken into account, however detailed modelling of the effects of fire on the structure and required fire suppression systems are not considered.
- **Vibrations:** General requirements for vibrations on structures are taken into account for the deck system of the car park. However a detailed analysis of what specific requirements would be optimal concerning a car park is not made. Vibrational behaviour as a result of earthquakes is not considered.

1.2.5 Methodology

1.2.5.1 Research Design and Process

The methodology used for this research is based on Hall's (1968) methodology for systems engineering. To help describe the methodology, the different phases of Hall's design cycle are projected over the different parts of the report. A visual overview including the methods used can be seen in Figure 1.

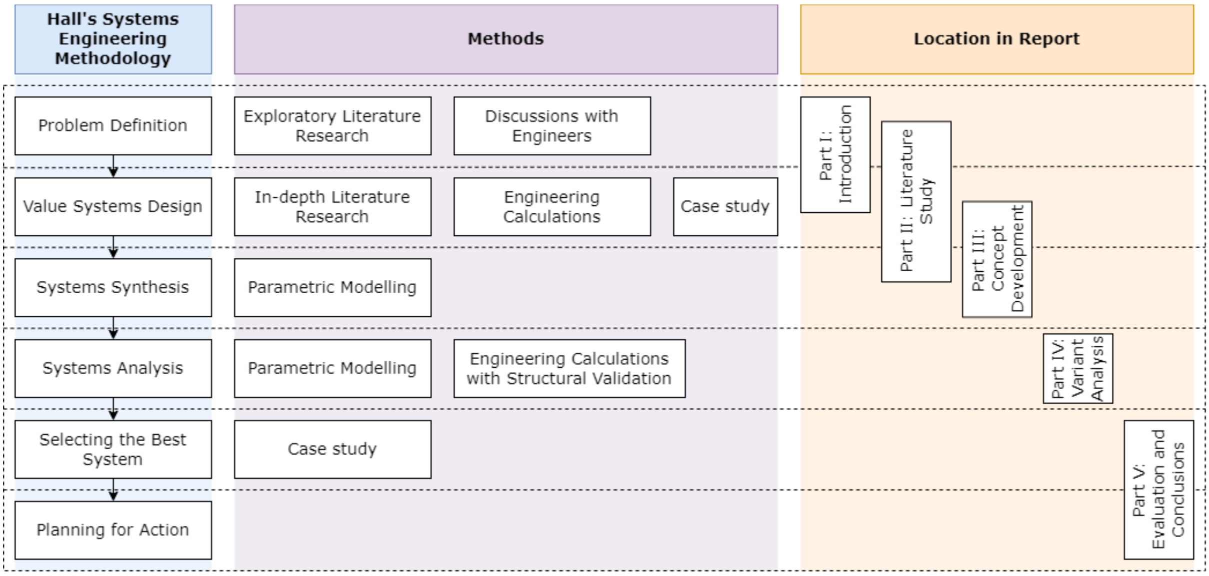


Figure 1. Design Methodology

1. **Problem definition**
   This master thesis started with the problem definition. Through exploratory literature research and discussions with engineers, the problem context of temporary car park structures and building in timber was studied. This helped to understand the challenges the parking industry has to deal with and the challenges the entire building industry faces. As a result, a first problem statement was developed, which acted as the basis for this master thesis. Furthermore, a main research question and first set of sub-questions was developed.

2. **Value systems design**
   In the value systems design phase, the question at heart was what the objective is and what the criteria are that a solution has to fulfil. This objective followed from the problem statement defined in the first design phase and was to provide a structural system proof of concept for a temporary multi-storey car park, using timber as primary structural material. In order to come up with the criteria the solution has to fulfil, a rather broad but in-depth literature study was performed into the requirements for such a proof of concept. Based on the exploratory research and objective, three topics were pointed out as being highly relevant to develop a successful proof of concept for the objective. The first relevant topic was related to the general design of car parks, considering aspects like regulations, comfort and economics. A second topic was related to the challenges faced by the building industry to become a sustainable industry, which included aspects like what sustainability strategies exist and how such strategies should be incorporated in the design of a construction. The third topic was related to the field of timber engineering. This includes aspects like how to build a timber structure in terms of different components and connections, but also how to protect it from the durability risks that a timber structure comes with.

   The in-depth literature study resulted in a broad overview of relevant aspects for the development of a temporary multi-storey car park primarily in timber. Next, focus needed to be applied to this broad set of aspects, considering what aspects would be further investigated and what not, since it is not possible to consider all aspects in a master thesis project. This was done in the concept development part, which includes two small sub-studies into existing modular and/or timber car parks and possible foundation methods, to further specify what is expected from the proof of concept to deem it successful. As a result, the research questions were further refined and a more specific set of requirements for the proof of concept was presented.

3. **Systems synthesis**
   The systems synthesis is all about generating alternatives. In the literature study, an overview of different typologies for car parks was presented as well as an overview of potential application of different building materials. In the concept development part, a selection of these typologies and application of building materials was made, resulting in an overview of design variants that were further studied. Through the use of parametric modelling, these design variants were further divided into subvariants with slight alterations in geometry, to see if different geometries can result in a more optimal solution for the proof of concept.

4. **Systems analysis**
   In the systems analysis phase, the different design (sub)variants, developed through parametric modelling, were first structurally analysed and optimized to result in a set of structurally validated designs. Next, the performance of these designs on the objectives and criteria stated in the value system design phase was assessed. This allowed for comparison between the different designs on these aspects, to be able to select a best performing design subvariant for the proof of concept.

5. **Selecting the best system**
   Through a case-study, the developed structural system was compared to a selection of five alternative concepts for a modular and/or timber multi-storey car park. The concepts were assessed on their total deck height, weight of the structural system (including load on the foundation) and a first impression was given for their performance in terms of sustainability. Through this case study, one can conclude how the performance of the proof of concept relates to alternative concepts and therefore if it is feasible. This phase therefore ends in a conclusion and discussion on the performance of the developed structural system using timber as primary structural material, answering the main research question.

6. **Planning for action**
   The last phase consists out of the recommendations for future use of timber as a structural material in temporary multi-storey car parks.

1.2.5.2 Data Collection

Exploratory research

Data for the exploratory research was collected through literature research and discussions with engineers. For the exploratory literature research, journals, reviews, news articles and websites were used. This allowed for the author to quickly create an understanding of the relevant topics. Furthermore, discussions with engineers in the field of general structural engineering, timber engineering, digitalization and car park design helped to further understand where challenges are currently found in the various fields of car park engineering. For this exploratory research, a broad scope was applied, looking for information on car park construction and building in timber. On most aspects, a wide variety of literature was available, however the number of studies on car parks build in timber was very limited. Nevertheless, the data found presented a broad overview of relevant information and allowed to create a clear objective.

In-depth literature research

For the in-depth literature study, a lot of information found in the exploratory research could be further investigated. In this phase, the focus was more on scientific journals, handbooks and industry norms. This allowed to create a deeper understanding of the various relevant aspects and find research gaps. As already found in the exploratory research, studies into timber car parks were very limited, but it was found that even less research was conducted into modular and demountable timber car parks.

Concept development

In the concept development part, five different car parks were studied. These car parks were chosen based on the concept of their structural system. These were either modular (and demountable) or (partially) made from timber. These concepts were compared to each other to see how existing timber car parks perform relative to alternative modular car park concepts in different materials. Data on these structures was found from different sources: scientific journals, municipal archives, direct contact with architects and in-field measurements. Not for every potential interesting car park (the same amount of) data was available. The final selection of car park systems considered, all had sufficient data available for comparison.

Also possible foundation methods were studied and what loads can be carried by different soil build-ups in The Netherlands. Information on the various possible methods was found from literature. For the analysis of load carrying capacities of different soils, DINOloket was used to find different soil patterns accompanied by cone penetration tests. Load carrying capacities were calculated based on engineering calculation using Technosoft foundations software.

Case study

The case study included the same car park structures that were studied in the concept development. For all these car parks, the general build-up of the structure and dimensions of main structural elements were available through either architectural drawings or in-field measurements.

1.2.5.3 Data Analysis Methods

Literature

The literature found in the exploratory and in-depth literature studies was grouped into different (sub)categories. The main categories were: car park design, sustainable construction and timber engineering. Furthermore, each item was given a rating and short description. This clustering of literature helped to quickly find relevant information during the different phases of the thesis. The ordering of sources was done using EndNote.

Parametric modelling

Creation of geometry for the design (sub)variants was done through parametric modelling in Rhinoceros in combination with Grasshopper. These software packages allow for fast creation of a large design space for geometry. Furthermore, such a parametric model can easily be expanded in case new insights are requested for alterations on the developed geometry. A parametric model in Grasshopper also allows for coupling with various structural analysis plugins or software packages for further analysis of the developed structure.

Structural validation

Structural analysis and validation was done using the software package SCIA. This package was chosen as it is widely used in the engineering industry and as primary structural analysis package at Sweco. Furthermore, a plugin to connect the software package to Grasshopper is available. Although it was expected this would allow for a fully automated process from generation of geometry to structural analysis, validation and optimization, this turned out not to be the case. Such a fully automated design process requires further development of the software package and plugin for SCIA.

Comparison

Comparison of the performance of the various car par design alternatives as well as different car park concepts in the case study was done through Excel. Python was used to automate creation of the Excel file for comparison based on inputs from the structural analysis and study of structural elements of alternative car park concepts.

1.3 Reading Guide

Following the structure of the methodology, this paragraph describes the structure of the thesis report, see Figure 2. It globally shows what is discussed in the various parts of the report and what research questions are answered in what chapter.

Reading Guide

Report Part	Chapter	Research question answered	Description
Part I: Introduction	Ch. 1		Introduction including background of topic and research definition
Part II: Literature Study	Ch. 2 Ch. 3 Ch. 4	1a 1b 2a 2b 1c	Broad literature study including background information on car park design, the challenges faced by the building industry towards a more sustainable industry and building in timber.
Part III: Concept Development	Ch. 5 Ch. 6 Ch. 7 Ch. 8	3a 3b	Development of specification of what is expected from the proof-of-concept in terms of perfomance and what design variants are considered.
Part IV: Variant Analysis	Ch. 9 Ch. 10	3c	Detailed analysis of structural performance of deck and framing system of alternative design variants.
Part V: Evaluation and Conclusions	Ch. 11 Ch. 12 Ch. 13	4 Main research question	Case-study with comparison of proof-of-concept and alternative modular and/or timber car park concepts followed by conclusion, discussion and recommendations.

Figure 2. Overview of the structure of the thesis

Part I: Introduction
Chapter 1 forms the introduction of this thesis, describing the context of the problem and presenting the problem statement. Furthermore it presents the research definition, including objective, research questions, scope and methodology.

Part II: Literature Study
The second part of the thesis is the literature research. This literature study is rather broad and describes many different aspects related to the design of a temporary and modular multi-storey car park. It functions as background information for the development of the multi-storey car park. Chapter 2 focusses on the design of car parks and answers sub questions 1a (design requirements) and 1b (typologies). Chapter 3 is all about the challenges in the building industry, looking into sustainable building strategies and its impact on building elements, answering sub questions 2a and 2b. Literature about timber structures is researched in chapter 4, looking into different timber products, connections and durability and answers sub question 1c.

Part III: Concept Development
In part three of the thesis, focus is applied on what specific aspects found in the literature study are considered in the development of the proof of concept and what is expected from the resulting proof of concept. Chapter 5 specifies the main goal and specific sub-goals for the proof of concept, resulting in an answer to sub question 3a. Furthermore, hypotheses are developed for the expected results. Finally, the different design variants are presented, which will be studied in the variant analysis part. In chapter 6, it is defined what are the starting points for the development of the proof of concept concerning structural rules and norms. A first case-study is done in chapter 7. Five alternative concepts for a modular and/or timber multi-storey car park are studied, based on which performance criteria for the proof of concept are developed. This partially answers sub question 3b. Chapter 8 focusses on possible foundation methods and further answers sub question 3b considering performance criteria related to loads on the foundation.

Part IV: Variant Analysis
In part four, the different design (sub)variants are studied and compared to each other. Chapter 9 focusses on the possible deck systems for the structural systems and chapter 10 focusses on the

framing system. These chapters include a parametric study into the use of different geometric parameters and assessment criteria and what effect this has on the performance criteria stated in part three. These chapters result in an answer to sub question 3c.

Part V: Evaluation and Conclusions
Part 4 forms the last part of the thesis. First, in chapter 11, a case-study is conducted, comparing the earlier mentioned five alternative car park concepts with the developed proof of concept, which answers sub question 4. Next, in chapter 12 the final conclusions of this thesis are drawn, answering the main research question. Finally, chapter 13 forms the discussion of this thesis and includes recommendations for future research.

PART II:
Literature Study

2
Car Park Design

The design of a multi-storey car park involves integration of many different aspects. First of all, the design has to fulfil the requirements of the owner and users of the car park. These are largely reflected by specific car park design norms. Furthermore, the design has to fulfil the general rules and norms for the structures of buildings. Bearing these prerequisites in mind, different layouts for car parks and different typologies for its construction can be distinguished. These influence each other and goal is to find the most optimal design, which fulfils the clients requirements. This chapter aims to provide an answer through in-depth literature research to what the design requirements are for a multi-storey car park and what the possible typologies are.

2.1 User Requirements

Users of car parks are most concerned with functional requirements. The ease of finding a parking space and doing so without risk at damage to their vehicle is important to them (Van Drooge, 2020). For users it doesn't matter if a car park is temporary. They expect the same level of performance of a temporary car park as from a long term car park. Several characteristics, influencing the structural design of car parks, can be pointed out as being essential for a good user experience: car park category, dimensions of lanes and ramps, dimensions of bays and aisles, circulation through the car park and other considerations like drainage and security. Many of these user requirements are represented by the norm NEN 2443 - design standards and recommendations on parking facilities for passenger cars. This paragraph presents background information on what aspects concerned with user requirements should be considered. Specific dimensions and other recommendations from the parking facilities norm are discussed in chapter 2.4.

2.1.1 Car Park Category

Car parks may suit different types of users, which mostly depends on the location of the car park. One can understand, that supermarket customers have different expectations for a car park compared to business people, who park their car for an entire day. According to Hill (2005), four different types of car park categories can be distinguished: short stay, medium stay, long stay and tidal.

The first category is short stay and is described by intensive usage with high turnaround rates. The average duration of stay per car for short stay is less than 3 hours. This type of car park is usually associated with busy supermarket-type shopping activities.

The second category is medium stay and has less intensive usage and a lower turnaround rate compared to short stay car parks, but still is used quite intensive. The average duration of stay per car is more than 3 hours. This type of car park is mostly used in urban areas, where the car park is used by both business related users as well as visitors who come for shopping.

The third category is long stay, described by an average stay per car of over 12 hours and a light but continuous in- and outflow of vehicles. This category car park is often found at major transport terminals, which can occasionally result in short periods of intensive vehicle movement.

The fourth and last category is tidal and is characterized by its peak moments of in- and outflow. This type of car park serves mostly staff, resulting in a high inflow in the morning and outflow in the evening.

2.1.2 Dimensions of Lanes and Ramps

The dimensions of lanes and ramps are concerned with users of a car park being able to drive through the car park comfortably without the risk of easily incurring damage. Based on the standard design vehicle, minimum widths and radii for corners are known (see chapter 2.4.1), however these minimum values don't give users a comfortable feeling. In turns, users don't like to have full lock when turning. Therefore, it is recommended to provide wider turns at around 150% of the turning circle of the standard design vehicle for corners up to 90º and 200% for corners where 360º turns are anticipated. In general, it is preferred by users to have no turns bigger than 90º, so no spiral ramps and all turns in the same direction to prevent wild steering movements. (Hill et al., 2005)

When turning onto a ramp or coming off a ramp, wide turns are even more important. Also, the slope of the ramp has a big impact on comfort for users. Having to stop on a ramp and drive off again, is difficult if the slope of the ramp is too large. Slopes of ramps are therefore limited for comfort. For steeper ramps, transitional slopes are introduced to prevent grounding of vehicles or hitting the ceiling. When driving in a car park and especially on ramps, clear sight lines are important to prevent accidents between vehicles crossing paths (Van Drooge, 2020). Practical guidelines for the maximum slope percentages of ramps and clearance heights can be found in chapter 2.4.3.

Ramps influence the dynamic capacity of car park, which differs per type of ramp. It is therefore important to consider the required capacity of a car park, which type and how many ramps to use. For short and medium stay car parks, two-way-flow on ramps are not preferred as they result in conflicts between traffic flows (Hill et al., 2005). In other types of car parks, they can be very efficient in boosting static efficiency of a car park.

2.1.3 Dimensions of Bays and Aisles

Dimensions of bays and aisles are concerned with the ease drivers experience in turning into a parking bay and how much room is left for entering and exiting the vehicle by opening the doors. Considering the dimensions of bays, the length should be sufficient for at least the standard design vehicle to fit in the bay (see Figure 3), but the width is more important. Varying the width of the bay can have a significant impact on both the dynamic capacity of the car park as well as the ease of stepping in and out of a vehicle. A wider bay means it is easier to turn into. Considering the gap between adjacent vehicles, the room to get in and out of the car, a small increase in width can have a significant impact on how far a door can open and so how easy it is to get in and out of the vehicle (Hill et al., 2005). Chapter 2.4.2 presents clear guidelines for the dimensions of bays and aisles.

When two vehicles are parked next to each other, the space between both vehicles can be used for getting in and out of the vehicle, while if you are parked next to a wall or column, only the space remaining in the bay of one vehicle is available (see Figure 3). Therefore, it is preferred not to have walls or columns located next to the doors of vehicles. Furthermore, it is preferred not to have columns located at the front of bays as they hinder vehicles from turning into the bay. The best option is to have long column free spans over the parking decks (Van Drooge, 2020).

Figure 3. Examples of problematic dimensions of parking bays and location of columns (Van Drooge, 2020)

Aisles provide space for vehicles to manoeuvre into and out of bays and often for pedestrians to walk through the car park. When two-way-flow aisles are used, they have to be wide enough for two vehicles to pass each other, resulting in enough space to manoeuvre in and out of bays and provide space for pedestrians. If one-way-aisles are used, the aisles can be narrower, which can make the span over the parking deck smaller. Using angled bays can reduce the width even more, at the same time making parking easier and simpler (The Institution of Structural Engineers, 2011). This however leaves less room for pedestrians which should be kept in mind.

Last of all, parking decks can be designed as a slope, which can be an efficient design. This slope however has some disadvantages concerning comfort for users. Shopping cars and wheelchairs can drift off and if the slope becomes too steep, it becomes more difficult to open and close doors as a result of gravity (The Institution of Structural Engineers, 2011).

2.1.4 Circulation

The routing in a car park is very important for users. A clear and logical routing with good sight lines and a minimum number of conflict points, supported with signs, helps users in finding their way in the car park (Van Drooge, 2020). The circulation pattern can also have a significant impact on the dynamic capacity of a car park. Therefore, depending on the type of car park, a suitable circulation pattern should be chosen (Hill et al., 2005).

2.1.5 Miscellaneous User Experiences

As stated earlier, clear sight lines in car parks are important for safety while driving. Users like to have a feeling of security in general (Troup & Cross, 2003). The structure can have a significant impact on this feeling as an open and light structure often gives the feeling of an area being safe, while a closed and dark structure with large walls blocking sight lines gives a feeling of unsafety.

Another general aspect for users of car parks is drainage. Large puddles of water may not only present a risk to the structure, but also hinder stepping in and out of vehicles and when located in the aisles, people can get splashed by vehicles driving by. Therefore, it is required to minimize the possibilities of forming of puddles by proper drainage (Hill et al., 2005).

2.2 Owner Requirements

Owners or developers of car parks, the clients of car park designers, have to make trade-offs between many decisions. They want to provide their customers with the best high level car park, but at the same time keep the costs within budget. Also aspects like sustainability play a role in choices for a structural design. Apart from cost-efficiency and sustainability, design flexibility, the construction process and durability and maintenance have been identified as important aspects for the owner of a car park.

2.2.1 General

Car parks can be developed and owned by different types of clients. It can be a public owner like a municipality or hospital, a public company, a private client or a project developer. Each client has its own goals and needs. A public owner for example might have less flexibility and budget, while a project developer might want to achieve the highest quality (Troup & Cross, 2003).

Regardless of the type of client, for the planning of a car park, several general requirements have to be stated. It should be determined what the capacity of the car park should be and what type of category car park it should be or what level of service should be provided (Hill et al., 2005; Troup & Cross, 2003). These aspects will influence the choice for a certain layout of the car park. Other aspects are the intended location of the car park and its maximum dimensions and number of levels.

2.2.2 Cost-efficiency

For clients, several characteristics of a car park can give a general idea about its performance. These characteristics are concerned with how efficiently the structure is laid out in terms of average area per parking place in m^2 (Pollak, 2003) and the average costs per parking space (Cross, 2002). These numbers are important as the costs of the construction itself are normally 60 to 70% of the total construction costs (Pollak, 2003). The static efficiency, average area per parking space, generally varies between 20 and 30 m^2 per car (Hill et al., 2005). For planning purposes, often 28 to 30 m^2 is used (Simon, 2001). The construction costs for a car park above ground can vary significantly, depending on their construction and design. On average in the US in 2019, according to Cudney & Smith (2019), the construction costs for a new parking structure were $21,500 per space. Walker (2016) did a similar study in the US in 2015 and found construction costs varying between $15,000 and $25,000 per space with an average of $19,000.

2.2.3 Design Flexibility

In the design, one should take into account several factors. In the future, a client might want to react to a changing market. As a result, one might want to expand or reduce the parking facility (Simon, 2001). Another option would be to converse (part of) the building to another function. To provide in such flexibility requirements, the adaptation possibilities should be incorporated into the design of the original structure.

Many car parks have a similar structural system, but differ significantly in appearance. This is the result of different types of façades. To provide in the possibility for different appearances as well as possible future changes in appearance, the structural system of a car park should provide opportunity for aesthetic expression (Troup & Cross, 2003).

2.2.4 Sustainability

As climate change is more evident than ever before, sustainability in construction projects becomes more important (White, Hardisty, & Habib, 2019). Users expect higher levels of sustainability in all types of services, of which car parks are one. As a result, also developers start to consider sustainability in their projects more often as an important aspect. Sustainability also effects the structural design. Important aspects to keep in mind are the choice for certain building materials; where do they come from, how are they produced and what can be done with them after the building is demolished (Troup & Cross, 2003). Also the construction process is important to keep in mind considering sustainability; transportation and production location of materials for example.

2.2.5 Construction Process

The construction process of a car park is an important step for a developer or owner. During construction, a lot of expenses are made to construct the building. When the car park becomes operational, the owner can start to make a return on its investment, so the sooner it is finished, the better. The speed of construction is therefore an important aspect for a car park and can be influenced by the chosen structural system (Pollak, 2003). A modular steel structure or prefab concrete structure for example is much faster to erect compared to a cast-in-situ construction. Also detailing of connections and the size of structural elements have a significant impact on erection times (Precast/Prestressed Concrete Institute, 1997). In the design, one should also take into account maximum dimensions and weights for transportation and make a trade-off between bigger structural elements for faster erection versus the higher costs as a result of the requirement for heavier equipment like cranes.

2.2.6 Durability and Maintenance

Durability and maintenance of a construction go hand in hand and are massively influenced by the choice for a certain structural system, details and building materials (Troup & Cross, 2003). It influences the life expectancy of the structure and can be a huge cost item (Precast/Prestressed Concrete Institute, 1997). An owner often prefers minimal required maintenance, to minimize downtime of the car park (Simon, 2010). As car parks are open to weather influences and de-icing salts are brought in by cars, protection of the structure by design is essential during development of the car park. After completion, regular inspection and maintenance is required (see Figure 4 for examples of damages to car park structures). It is important to give attention to these aspects during the design process as a good design can minimize maintenance and possible damage. For example a sufficient topping can provide a watertight membrane and by minimizing restraints to movements, tensile stresses which can cause cracking of floors and walls can be minimized (Cross, 2002). A cheaper variant in construction can often proof to be more expensive in the long term.

Figure 4. Damage to car park structures (British Parking Association, 2012; Ducorr, 2018)

2.3 Structural Rules and Norms

A multi-storey car park is a special type of building. It is no regular building like an office or residential complex with a thermal shell nor is it a civil structure for infrastructure like a bridge or tunnel. As a result, in The Netherlands, the Dutch building degree (Ministerie van Binnenlandse Zaken en Koninkrijkrelaties, 2021) and the European Eurocodes for the building sector (NEN, 2021) have to be applied. The Dutch parking norm is not required as it is not part of the building degree, but it consists of elaborate guidelines, compiled from practical examples, and is therefore used in this thesis.

- Dutch Building Degree 2012
  (Ministerie van Binnenlandse Zaken en Koninkrijkrelaties, 2021)
- NEN-EN 1990+NA: Eurocode 0, Basis of structural design
- NEN-EN 1991+NA: Eurocode 1, Actions on structures
- NEN-EN 1993+NA: Eurocode 3, Design of steel structures
- NEN-EN 1995+NA: Eurocode 5, Design of timber structures
- NEN-EN 1997+NA: Eurocode 7, Geotechnical design
  (NEN, 2021)
- NEN 2443: Design standards and recommendations on parking facilities for passenger cars.
  (Nederlands Normalisatie Instituut, 2013)

The focus of this thesis is on the structural design of a car park and the use of timber as a structural material. Therefore, the different rules and norms are discussed only on aspects, which influence the structural design of the main load bearing structure. Other requirements fall out of the scope and are not discussed in this thesis. Relevant norms on the design and calculation for different building materials are also considered. Norms on the design and calculation for earthquakes are not considered.

2.3.1 Preconditions and Assumptions

For the design and calculation of the temporary multi-storey car park, several assumptions are made. Although the aim is to make a modular and demountable design, the structure is not classified as such. The design life of the structure will be similar to a normal fixed building. The only difference is that this structure during its design life will be erected at several places, but it has to withstand the same loads as it would when constructed at a single location.

Type of building:	Public multi-storey car park	
Consequence class:	CC2	
Reliability class:	RC2	$k_F = 1.0$
Design service life category:	Class 3	50 years

2.3.2 Loads and Limit States

The structure of a multi-storey car park should withstand several permanent and variable loads. These should be combined in the various limit states. An overview of the relevant equations for combination of loads in ultimate limit state (ULS) and serviceability limit state (SLS), including the combination factors can be found in appendix A. Also values for the maximum deflections and displacements are presented in this appendix.

2.3.2.1 Permanent Loads

Permanent loads acting on the structure are self-weight of both structural and non-structural elements. These loads are dependent on the design and building materials used and have to be determined during the design process. For a car park, among others one can think permanent loads following from self-weight of the following elements (Nederlands Normalisatie Instituut, 2019a):

Structural elements
- Floor elements
- Beams
- Columns
- Bracings

Non-structural elements
- Surfacing and coverings
- Partition walls
- Hand rails, safety barriers, parapets and kerbs
- Façades
- Fixed services

2.3.2.2 Variable Loads

Various variable loads can be present on the structure of a multi-storey car park. The imposed vehicle load, specifically concerned with a car park is discussed below. Accidental loads are also relevant for car parks, but for the preliminary design of this thesis are left out of scope. An overview of the other more general variable loads, acting on any building can be found in appendix A. These include snow and wind loads.

Imposed Loads on Garages

Considering the variable loads acting on the structure, a multi-storey car park is classified as category F “Traffic and parking areas for light vehicles (≤ 25 kN gross vehicle weight and ≤ 8 seats not including driver) (Nederlands Normalisatie Instituut, 2019a). A distributed and concentrated load are imposed, see Table 1. The concentrated load has to be applied as an axle load, following the dimensions in Figure 5, in which the square surfaces have sides of 100 mm.

Table 1. Imposed loads on garages and vehicle traffic areas. (Nederlands Normalisatie Instituut, 2019a)

Category of traffic area	q_k [kN/m^2]	Q_k [kN]
F (light vehicles ≤ 25 kN)	2	10

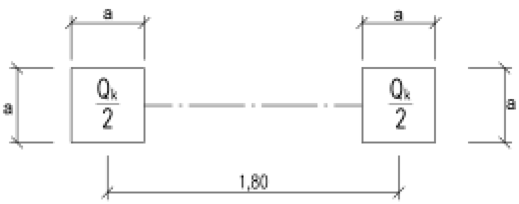


Figure 5. Dimensions of axle load. (Nederlands Normalisatie Instituut, 2019a)

2.3.3 Vibrations

According to the Eurocode (2015), structures of buildings should provide satisfactory vibration behaviour. This behaviour relates to the comfort of the user and the functioning of the construction or its structural elements (no cracking etc.). To assess whether a structure meets the requirements for satisfactory vibration behaviour, the natural frequency of vibrations of the structure or structural members are used under serviceability limit state combinations. For timber floors in residential buildings, Eurocode 5 (Nederlands Normalisatie Instituut, 2011) prescribes additional checks in case the natural frequency of the floor lies beneath 8 Hz, to prevent resonance from persons walking on the floor.

In car parks, vibrations are induced by both people walking, but also by moving vehicles. However, for car parks, no advice is given in the Eurocode concerned with allowed vibrations. Since visitors only walk in a car park for a very short time, they are often not disturbed by floor vibrations (Zanon, 2019). When driving in their car, sufficient damping of vibrations is provided by the suspension of the car. Therefore, there are no formal requirements for car parks for vibrational behaviour. Nevertheless, vibrations in the structure can be annoying. Therefore, the vibrational behaviour for the floors is checked on a less strict design class (Class II, (Schirén & Swahn, 2019)), which requires a frequency criterion of 4.5 Hz and stiffness criterion of 0.500 mm.

2.3.4 Fire Safety

2.3.4.1 Predicting the Structural Behaviour

When assessing the resistance to fire of a load-bearing structure, in general the five steps in the chain of events in case of fire are followed. These are (Breunese & Maljaars, 2015):

1) Ignition — This is the first step in the development of a fire. This step requires no explicit task from the engineer, since the assumption is that somewhere in the lifetime of a structure, it is exposed to a fire.

2)	Fire development	As the fire develops, the convection and radiation temperature rise. The speed at which these rise depends on the thermal load and layout of the structure.
3)	Thermal response	Based on the temperatures from the previous step and the thermal properties of the structural elements, the temperature of these structural elements can be determined; the thermal response.
4)	Mechanical properties	Mechanical properties of materials like the material strength and Young's modulus are dependent on the temperature. As the temperature of the structural elements is determined, the mechanical properties of the materials under these conditions can be determined.
5)	Mechanical response	Based on the layout of a structure, the loads under fire conditions and the found mechanical properties under fire, the mechanical response (strength and stability) of the structure can be determined.

To determine the behaviour of a structure under fire conditions, two types of methods can be used: the conventional approach and the fire safety engineering approach (FSE).

The conventional approach is the simplest and is in practice used for relatively standard structures. For such a structure, the mechanical response of each individual component is determined based on the use of a generalized fire curve for the fire load, see Figure 6. With increasing temperature over time, it is determined when a component fails. The time it takes for the first component to fail determines the failure of the entire structure in terms of fire resistance of the structure in minutes. This doesn't mean that in reality the entire structure will have failed, but from a regulatory point of view, this is the case. This method gives a gross approximation of reality in terms of both the approximation of the fire load and of the structural response. However, the method is simple to apply and used in most regulations (Breunese & Maljaars, 2015).

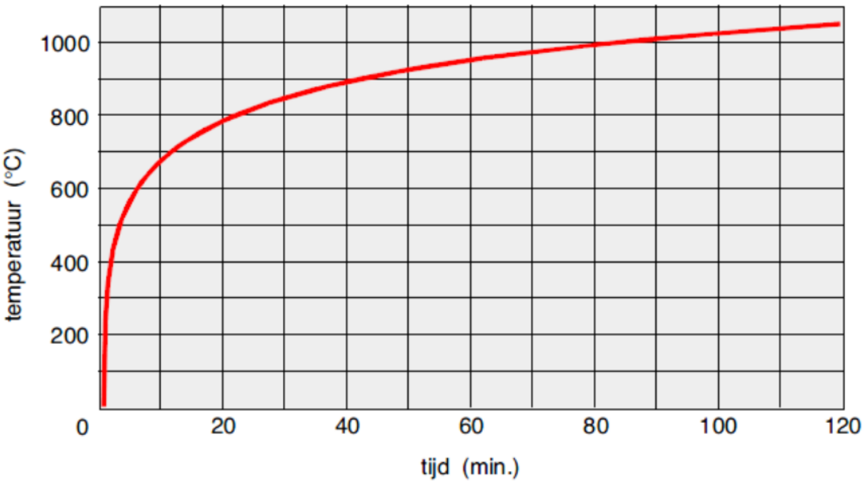


Figure 6. Standard fire curve (Baaij, n.d.)

With the more complex FSE approach, a much more accurate temperature of the fire is determined using the natural fire safety concept (NFSC). Instead of using a generalized fire curve to determine the temperatures, characteristics of the fire compartment are considered like: type and amount of combustible materials, floor area and height of compartment, openings in compartment, thermal capacity of the boundary enclosure and active measures such as roof venting, automatic fire alarms and automatic fire extinguishing devices. With this approach, a temperature-time curve is determined for all compartments of a structure. This is done using either relatively simple zone models or for more accurate results with CFD models (Breunese & Maljaars, 2015).

2.3.4.2 Fire Development in a Car Park

The standard fire is based on the development of a fire in a general building (office/residential). Such a building contains a fire load which consists of all kinds of furniture made of different materials. After ignition of the fire, the growth phase starts, in which temperatures in the fire compartment start to rise. At this point, the fire is a local fire. After a certain time, a flashover takes place, resulting in a very fast increase of temperature. This introduces the next phase of a fully developed fire, in which case all combustible materials in the fire compartment contribute to the fire as a result of the very high temperatures. The entire fire compartment is now on fire. After some time, when most of the combustible materials have been burned, the temperatures start to drop again as the fire comes to the decay phase (Breunese & Maljaars, 2015). The standard fire curve starts at the point of flashover and assumes a continuing growing fire. The decay phase is not represented by the standard fire curve.

A fire in a car park differs from the standard fire as the main fire load in a car park are the cars. A fire starts in a car and after some time, it will spread to an adjacent car. However, a car fire has the characteristic to be a fire with a relatively high rate of heat release, but for a relatively short amount of time (D. Jansen, 2010). As a result, at the moment of fire spread to the adjacent vehicle, the original car is already almost burned out (Van Herpen, 2014). Therefore, it is highly unlikely for a fire in a car park to grow into a fully developed compartment fire. The fire scenario is more that of a local moving fire (Jansze & Van Acker, 2014).

As a result of the different fire scenario, the fire curve of a fire in a car park differs from the standard fire curve. Compared to a standard fire, a car park fire is more intense in the beginning, but much shorter in time as it remains a local moving fire. Figure 7 shows the combined normative fire curve for a car park fire against the standard fire curve. As can be seen in the graph, until 35 minutes, the temperature of a fire is sometimes higher than the standard fire curve. However, after 35 minutes, the temperature is significantly lower compared to the standard fire curve. As a car park in general has to withstand a fire for 90 minutes, the standard fire curve can be seen as a very conservative approach (Goesten, 2022).

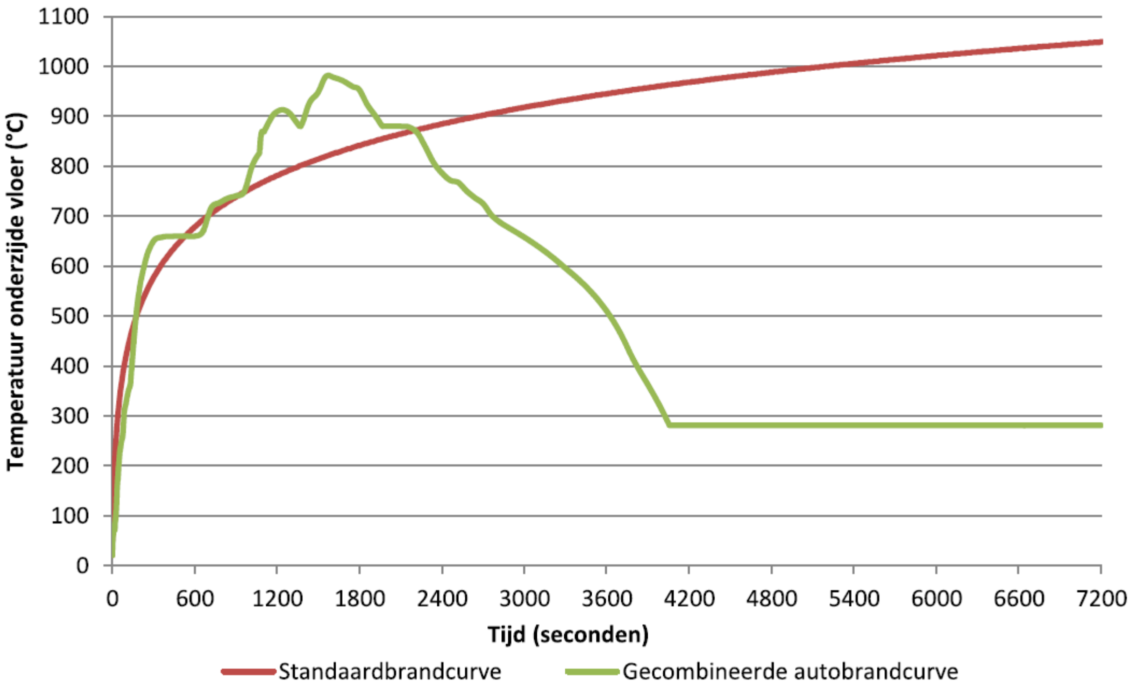


Figure 7. Local and global thermal load combined to a single normative car park fire curve (Dijkstra, 2014)

2.3.4.3 Electric Vehicles

In recent years, the number of electric vehicles on the roads has been on a fast rise. Such vehicles with their large batteries present new fire risks. Several studies have been conducted on the fire development of electric vehicles. It has been found that the maximum heat release rate (HRR) of an electric vehicle is comparable with a standard car with a combustion engine as can be seen in Figure 8. Also the evolution of HRR over time is comparable (Sun, Bisschop, Niu, & Huang, 2020). A difference between electric vehicles and cars with a combustion engine is that electric vehicle fires are more difficult to suppress. Their battery packs are difficult to access and they can re-ignite without continuous suppression. The best method of suppression is still the use of water, although continuous large amounts of water are required (Brzezinska & Bryant, 2022). As the HRR of electric vehicles is comparable with cars with an combustion engine, no different measures have to be taken in terms of fire safety for the main load bearing structure. The use of the standard fire curve for example is still a conservative method. However, the development of new suppression systems aimed at fires in electric vehicles would be very useful.

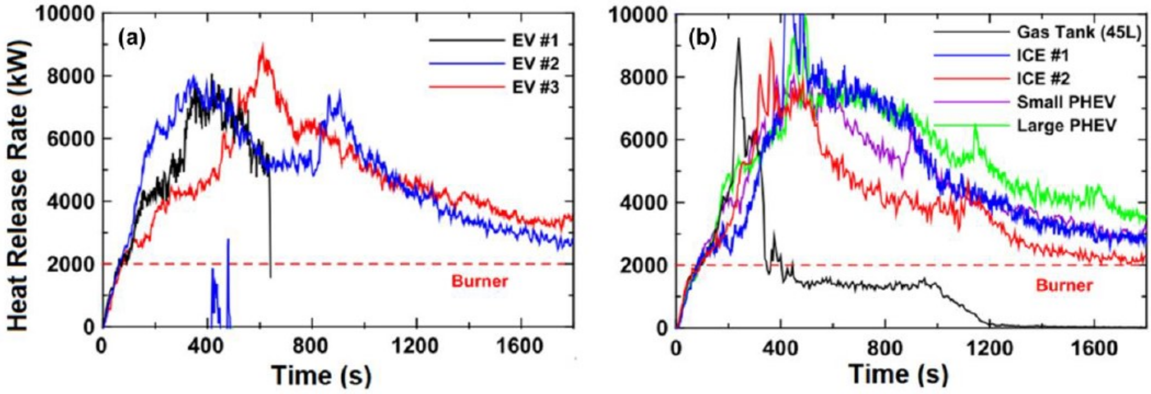


Figure 8. Evolution of HRR versus time for test vehicles which were suspended over a propane burner of 2 MW: (a) three different pure battery EVs, and (b) a small PHEV and a large PHEV compared with the gas tank and internal combustion engine (ICE) vehicles (Sun et al., 2020)

2.3.4.4 Impact of Fire Safety on Structural Design

According to the Dutch building degree, a fire compartment is 1,000 m^2 at maximum. Car parks however are often larger. Smaller fire compartments can be created by installing fire screens, which drop down from the ceiling in case of fire (Stöbich, 2022). However, it is allowed for a fire compartment to be bigger than 1,000 m^2, based on the principle of equivalence, if correct motivation is given why a fire in the larger compartment is as containable as a fire in a compartment smaller than 1,000 m^2. Such motivation can include installations such as a sprinkler system or smoke and heat disposal system (Kersten, 2011). It is also possible to make detailed zone model calculations, which can provide proof that for the principle of equivalence. As detailed fire safety measures are outside of the scope of this thesis, it is assumed the required motivation if required is available and therefore, fire compartments bigger than 1,000 m^2 are allowed. Starting point for this thesis is that every floor level is a fire compartment as a whole. However, it is also studied what the effect is if the entire car park is considered to be a single fire compartment, which is also allowed based on the principle of equivalence.

For the main load bearing structure of a multi-storey car park higher than 5 m, in general a fire resistance requirement against collapse of 90 minutes applies. This time can be reduced to 60 minutes in case the permanent fire load is smaller than 500 MJ/m^2. In case of a timber structure, this reduction is not allowed as timber is a combustible material and adds to the fire load. Therefore, a fire resistance against collapse of 90 minutes is assumed for this thesis as a starting point. If the entire car park is seen as a single fire compartment, a fire resistance against collapse of only 30 minutes is required (Hamerlinck, Breunese, Noordijk, Jansen, & Van Oerle, 2011; Ministerie van Binnenlandse Zaken en Koninkrijkrelaties, 2021).

2.4 Car Park Design Rules and Norms

A car park is a special type of building, for which specific rules and norms have been written, which in The Netherlands is the NEN 2443. This norm is not required, but is extremely useful as many of the user and owner requirements have been translated into this norm. A car park, which fulfils all the requirements of this norm can therefore be seen as a comfortable car park for its users. The norm is therefore used as a basis in this thesis, from which is only deviated if this results in a significant more optimal design. All of the information in this paragraph is based on NEN 2443: Design standards and recommendations on parking facilities for passenger cars (Nederlands Normalisatie Instituut, 2013).

2.4.1 Dimensions of Lanes and Ramps

2.4.1.1 Straight Lanes

The minimum width of lanes in car parks can be found in Table 2. Lanes require a redress lane on both sides of 0.25 m wide. For lanes longer than 30 m, the redress lane should be 0.50 m wide. To provide extra space for pedestrians, extra space parallel to the lane can be reserved with a minimum width of 0.90 m.

Table 2. Lane widths for straight lanes. (Nederlands Normalisatie Instituut, 2013)

Type of lane	Width [m]	Width redress lane [m]	Total width lane [m]
One-way-flow	> 2.75 m	0.25 m	> 3.25 m
Two-way-flow	> 5.50 m	0.25 m	> 6.00 m
Space for pedestrians	> 0.90 m		> 0.90 m

2.4.1.2 Curved Lanes

For curved lanes, the lanes have to be wider to accommodate the turning of vehicles. A horizontal slack of 0.50 m between cars and objects has to be incorporated and a horizontal slack of 100 m in case of two-way-flow for upcoming cars. The width of the lanes is dependant on the radius of the lanes, see Figure 9 and Table 3.

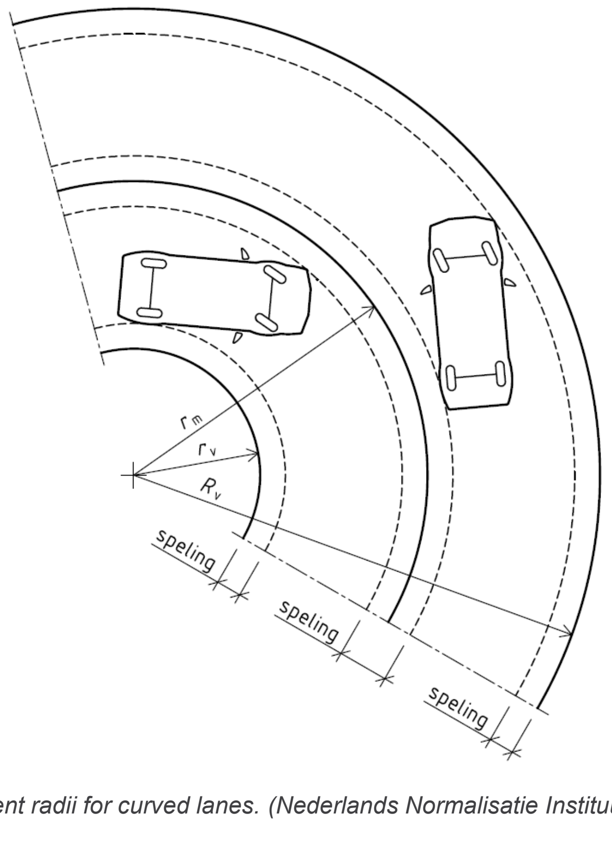


Figure 9. Overview of different radii for curved lanes. (Nederlands Normalisatie Instituut, 2013)

Table 3. Relation between different radii for one-way and two-way-flow, including redress lanes and slack (Nederlands Normalisatie Instituut, 2013).

One-way-flow		Two-way-flow		
Obstacle free inner radius r_v [m]	**Obstacle free outer radius** R_v [m]	**Obstacle free inner radius** r_v [m]	**Centre radius** r_m [m]	**Obstacle free outer radius** R_v [m]
3.00	7.80	3.00	7.50	11.60
4.00	8.75	4.00	8.45	12.50
5.00	9.70	5.00	9.35	13.40
6.00	10.65	6.00	10.30	14.30
7.00	11.60	7.00	11.25	15.20
8.00	12.55	8.00	12.20	16.10
9.00	13.50	9.00	13.10	17.00
10.00	14.45	10.00	14.05	17.90

2.4.1.3 Spiral Ramps

Spiral ramps are not popular with motorists, since they give the feeling of being very tight, which could result in damage. Based on the norm vehicle, minimum radii for sprial ramps are given in Table 4. However for comfort, recommended larger radii are also given. For comfort, a minimum cant of 3% is required in spiral ramps.

Table 4. Radii for spiral ramps including redress lanes and obstacle free zone (Nederlands Normalisatie Instituut, 2013).

	Obstacle free inner radius r_v [m]	**Centre radius** r_m [m]	**Obstacle free outer radius** R_v [m]
One-way-flow minimal	4.50	NA	9.00
One-way-flow recommended	6.00	NA	10.50
Two-way-flow minimal	4.50	9.00	13.30
Two-way-flow recommended	6.00	10.50	14.80

2.4.1.4 S-corner

On locations with a transition between two parallel lanes (or < 20º difference), so called S-corners apply. An overview of applicable dimensions can be seen in Figure 10 together with the dimensions from Table 3 for one-way-flow.

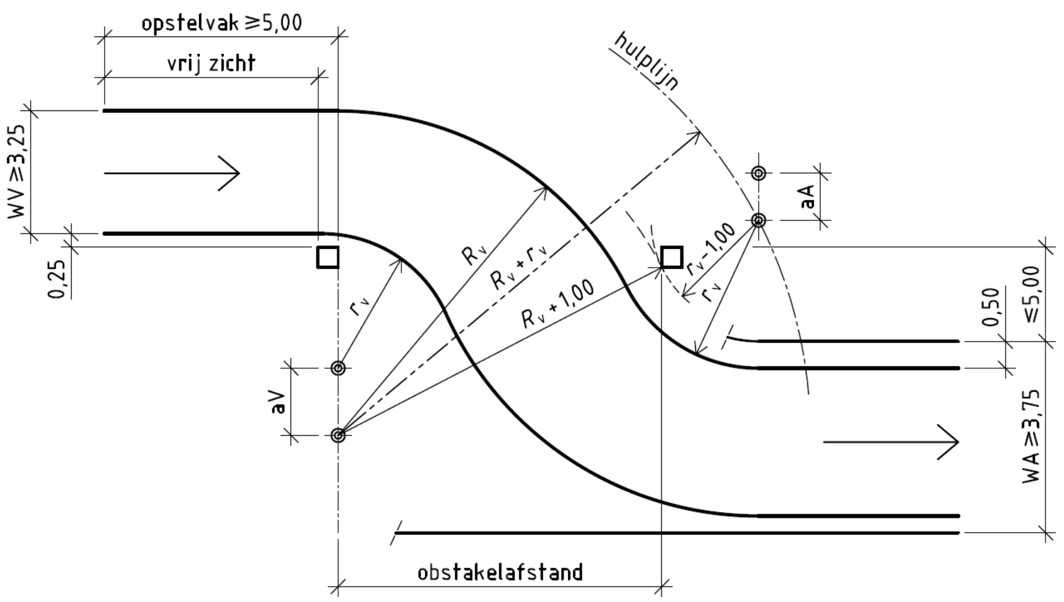


Figure 10. Dimensions for S-corners (Nederlands Normalisatie Instituut, 2013).

2.4.2 Dimensions of Parking Bays and Aisles

A standard car park bay for an intensive used public car park has a width of 2.50 m and a length of 5.13 m. To accommodate turning into the bay from the aisle, for 90º parking, an aisle width of 6.00 m is required. Using smaller parking angles, the width of aisles can be reduced, however the length of the parking bay will increase, see Figure 11. Table 5 gives an overview of the different dimensions of parking bays and aisles for different parking angles.

Table 5. Dimensions for different parts of public car parks for different parking angles (Nederlands Normalisatie Instituut, 2013).

Parking angle (α) [º]	**Depth of single parking strip (P_1) [m]**	**Depth of double parking strip (P_2) [m]**	**Depth of double parking strip using herringbone setup (P_2) [m]**	**Width of aisle (W) [m]**	**Width of parking bay (b) [m]**	**Total width of parking unit for one-side parking (B_1)**	**Total width of parking unit for two-side parking (B_2)**
See Figure 11	Top-left	Top-right	Bottom	All	Top	Top-left	Top-middle
30	4.19	6.80	7.30	3.80	2.50	7.99	12.18
45	4.95	8.61	8.37	3.80	2.50	8.75	13.70
60	5.38	9.85	11.39	3.80	2.50	9.18	14.56
65	5.44	10.11	11.41	3.80	2.50	9.24	14.68
70	5.46	10.29	11.35	3.82	2.50	9.28	14.74
80	5.37	10.44	NA	4.87	2.50	10.24	15.61
90	5.13	10.26	NA	6.00	2.50	11.13	16.26

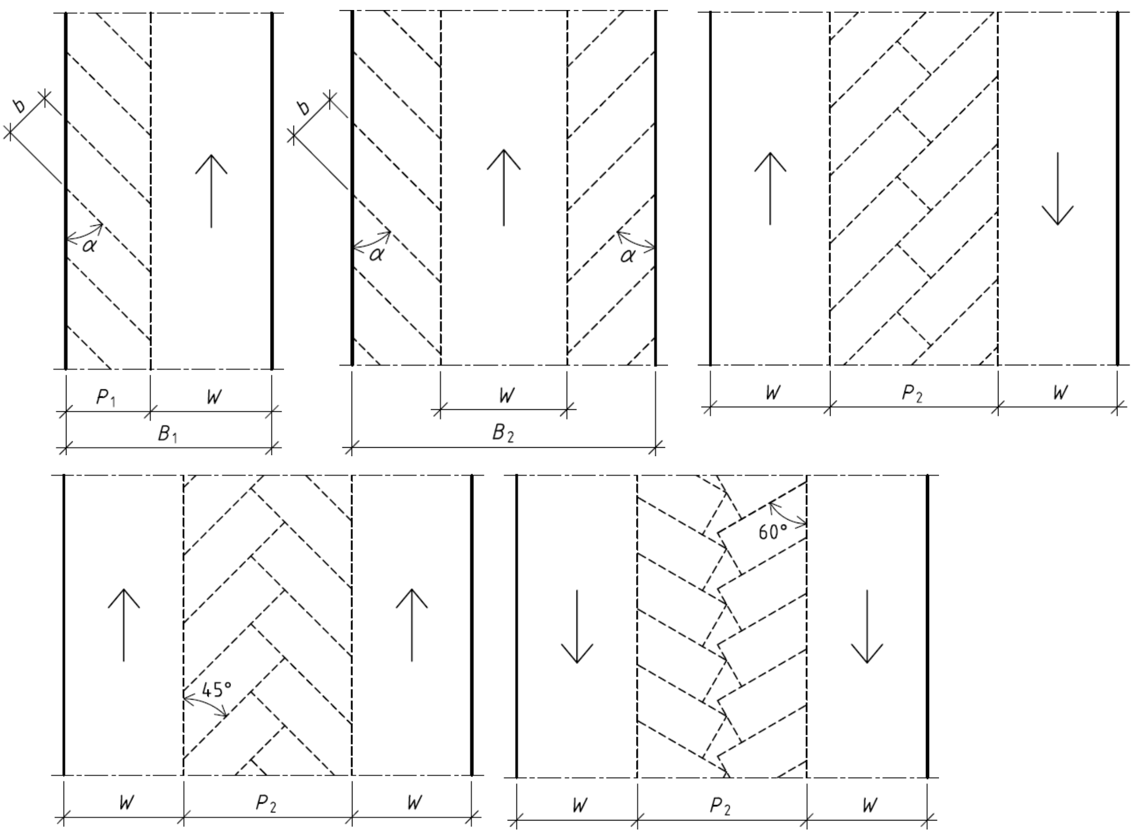


Figure 11. Top-left: One-sided parking with varying parking angles. Top-middle: Two-sided parking with varying parking angles. Top-right: Depth double parking strip. Bottom-left: Herringbone setup (parking angle 45º). Bottom-right: Herringbone setup (parking angle 60º). (Nederlands Normalisatie Instituut, 2013)

2.4.2.1 Dead Ends

When dead end lanes are longer than 8 m, two-way-flow aisles are required. In this case, it is also recommended to provide a turning possibility. Furthermore, for dead end lanes longer than 8 m, an extra space at the end of the dead end of 1 m is required.

2.4.2.2 Columns, Side Walls and Other Obstacles

Preferably, car parks have long column free parking decks. However, it is possible to locate columns in between parking bays. The location of columns should not hinder vehicles from turning into the parking bay and columns should not block doors from opening. Therefore, columns at the beginning of a parking bay should not be closer than 0.50 m to the aisle and be at maximum 1.50 m away from the aisle, see Figure 12. Columns at the back of the parking bay should not extend further than 0.60 m from the back of the parking bay. If the position of columns does not fulfil these requirements, extra space next to the parking bay should be added. In case a column is positioned on one side of the bay an extra width of 0.15 m should be added to the width of the parking bay. In case on both sides of the bay a column is located, an extra width of 0.35 m should be added. For parking angles < 45º, wider parking bays are also required.

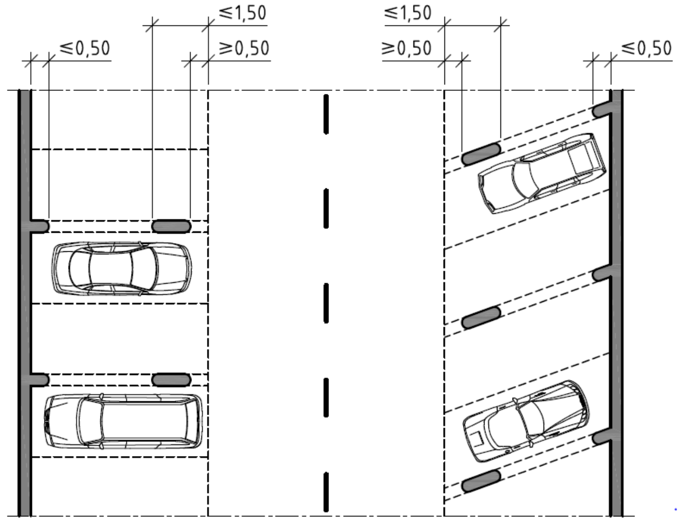


Figure 12. Locations where columns can be placed without requiring wider parking bays (Nederlands Normalisatie Instituut, 2013).

If angled parking is used, some of the dead space in front of the bays may be used to position columns. For public car parks, a space of 0.50 m in length should be left in front of a line perpendicular to the parking bay side line, see Figure 13 left.

Another location where columns can be located without the requirement for wider bays is in the middle of four bays. In this case, columns can extend 0.20 m into the bays, see Figure 13 right.

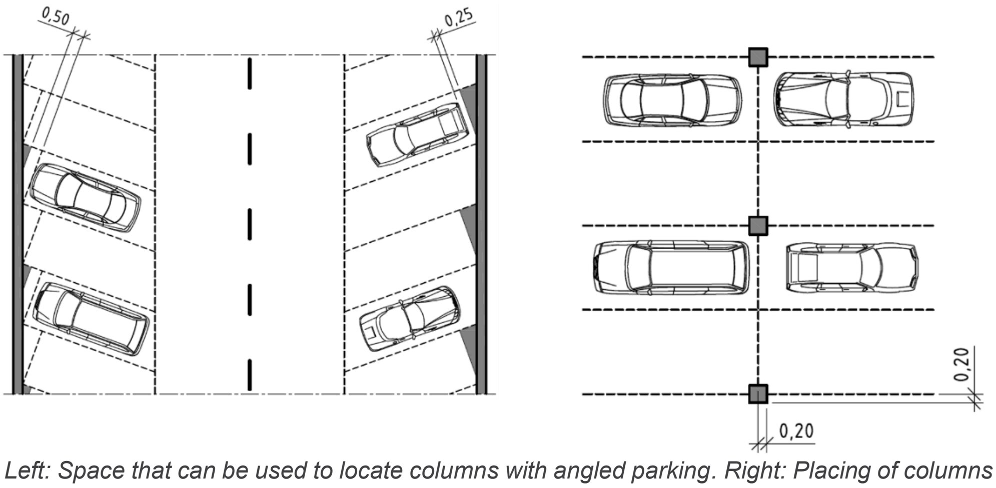


Figure 13. Left: Space that can be used to locate columns with angled parking. Right: Placing of columns in middle of four parking bays. (Nederlands Normalisatie Instituut, 2013).

2.4.3 Slope Percentages and Clearance Heights

In car parks, different slope percentages are applied, depending on several factors:

- The height difference to be bridged;
- The available horizontal length;
- Whether it is a carriageway or a sloped parking deck;
- The car park category of the facility.

The slope percentage is measured in the centre of the lane for straight ramps. For curved ramps, also the middle of the lane is used to measure the slope, however for two-way-flow ramps, it is measured in the centre of the inner lane.

2.4.3.1 Slopes in Lanes

The maximum slope in a public car park is 14%. An exemption to this requirement is in case of a d'Humy ramp system, which is used in split-deck car parks. In this case a maximum slope of 15% is allowed. The maximum slope also depends on the length of the ramp. These maximum slope percentages can be found in Figure 14.

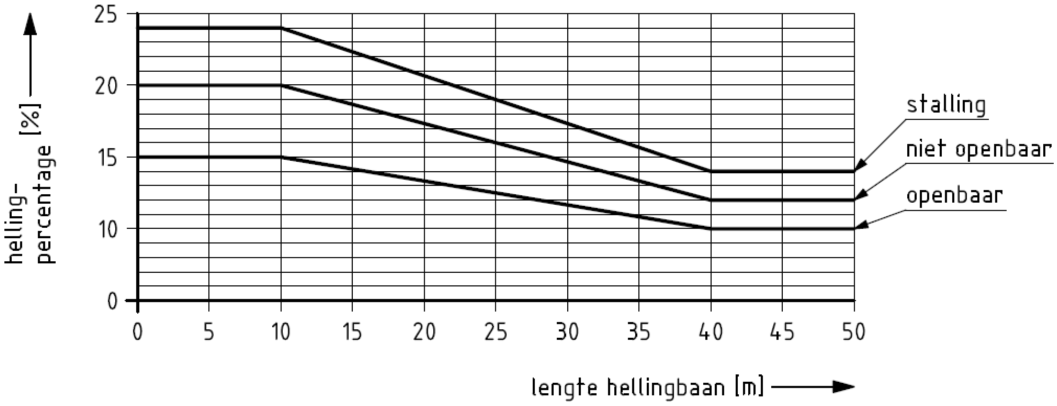


Figure 14. Maximum slope percentages depending on length of ramp (Nederlands Normalisatie Instituut, 2013)

2.4.3.2 Sloped Parking Decks

For sloped parking decks in public car parks that are used intensively, a maximum slope of 3% is allowed.

2.4.3.3 Compound Ramps

If two or more ramps are used in succession, these ramps have to be separated by a horizontal plane with a minimum length of 10 m.

2.4.3.4 Stopping on Ramps

For ramps, which are prone to vehicles coming to a stop during normal traffic operations, a maximum slope percentage is applied. For ramps with traffic-flow in upwards direction, the maximum slope is 6%. For downward ramps, it is 10%.

2.4.3.5 Clearance Heights

Public car parks should have a clear height of at least 2.30 m in areas intended for moving vehicle traffic. Underneath beams or pipes, it is occasionally allowed to reduce the clear height to 2.20 m. These heights also apply for areas, accessible for pedestrians.

2.4.3.6 Transition Ramps

Steep slopes may result in contact between the bottom or top of vehicles and the structure, see Figure 15. Therefore, in case of ramps with a slope larger than 14%, a transition ramp has to be incorporated with a maximum slope half of the slope of the main ramp. The length of a transition ramp at the bottom of a ramp is 2.77 m and for the top it is 1.385 m.

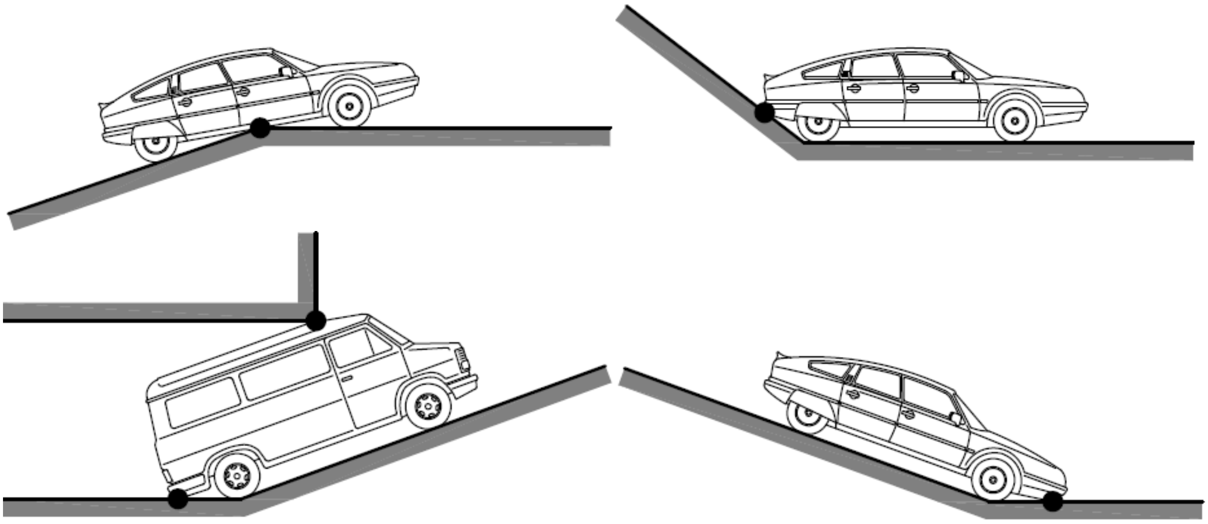


Figure 15. Possible locations for damage to vehicles (Nederlands Normalisatie Instituut, 2013).

2.4.3.7 Clearance Height at Top and Bottom of Ramp

The clearance height at the top and bottom of a ramp should meet the requirements of Figure 16.

a) At the bottom of a ramp without a transition ramp: draw a line at a height of 2.20 m starting from both sides of the nod at a distance of 2.77 m from the nod.
b) At the bottom of a ramp with a transition ramp: draw a line parallel to the transition ramp at a height of 2.30 m until it intersects with the line at a height of 2.20 m parallel to the main ramp and horizontal deck.
c) At the top of a ramp without transition ramps: draw two lines over a length of 2.77 m at a height of 2.30 m parallel to the ramp and horizontal deck.
d) At the top of a ramp with transition ramps: the clear height should be 2.20 m for the entire top of the ramp.

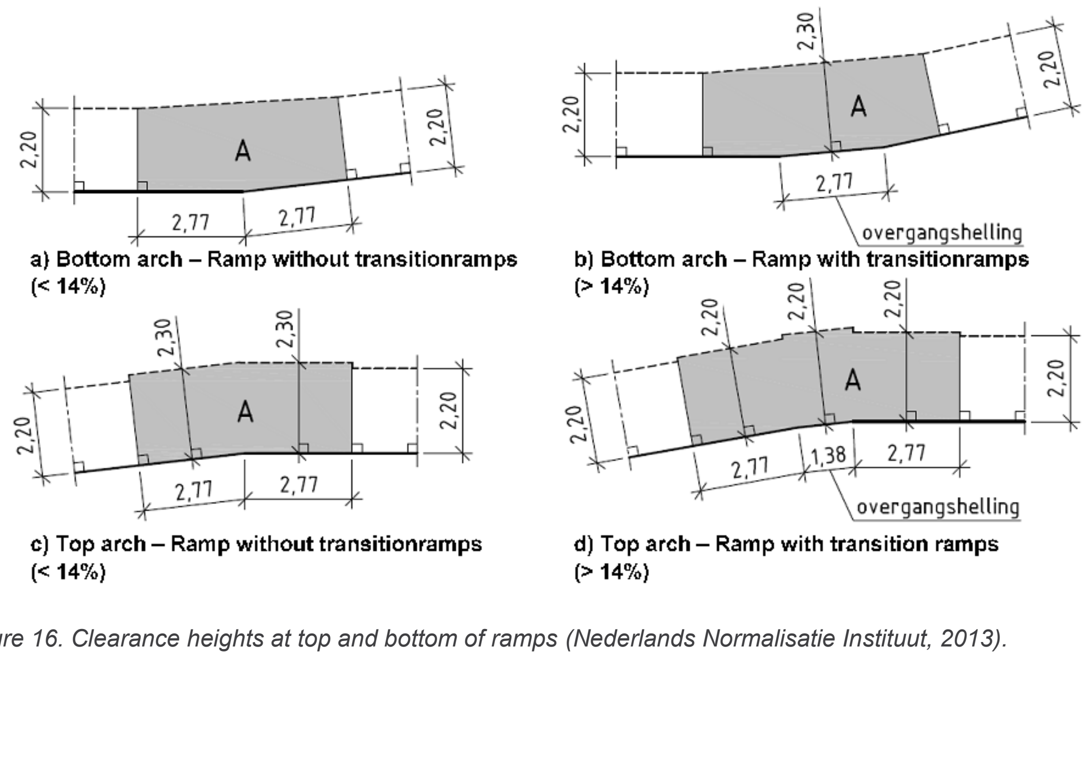


Figure 16. Clearance heights at top and bottom of ramps (Nederlands Normalisatie Instituut, 2013).

2.4.4 Other Considerations

2.4.4.1 Watertightness

Watertightness of the structure of a multi-storey car park is very important. As it is often an open construction, weather conditions can have a significant impact on the structure. Furthermore, de-icing salts can be introduced on the parking decks, dripping of cars. De-icing salts can affect (rebar) steel, resulting in damage of the structural system. Therefore, the assembly of all structural components of the car park should be impermeable.

Water dripping from the upper parking decks onto cars can result in damage of the paint on cars. Therefore, again measures must be taken to prevent water flowing through and under the decks and elements of the structure.

To collect water coming from cars and rain, sufficient drainage should be applied. A drainage system consists out of both drains and gutters, but also the gradient of the floor elements.

2.4.4.2 Open Structure

A car park that is classified as an open structure has several advantages concerning requirements for example on ventilation and fire safety. To classify as an open structure, at least the following requirements should be met:

- At least two walls opposite one another should be exterior walls and have to be provided with openings, which cannot be closed.
- These two walls are in one direction of the car park at a maximum distance of 54 m from each other.
- The lowest deck of the car park can be at maximum 1.30 m beneath ground level.
- Interior walls should not limit ventilation.
- At least one of the following conditions should be met:
    - The total of all openings in exterior walls, not able to be closed, must together comprise 1/3 of the total area of the outer and interior walls of a compartment.
    - All openings that cannot be closed in two exterior walls, opposite of each other, must per compartment and for each exterior wall be at least 2.5% of the gross area of the parking deck in the compartment.

2.5 Car Park Layout

To achieve the objective of this thesis, which is to provide a structural system proof of concept for a temporary car park, a basic layout of the car park has to be determined. This layout is concerned with aspects like parking bay configurations and traffic flow within the car park. This will influence for example the location of ramps and required dimensions for aisles, which in term will influence the layout of the structural system. Since this thesis focuses on a temporary car park without a specific location, the layout should be applicable in a wide variety of potential building locations.

2.5.1 Circulation Design Aspects

2.5.1.1 Circulation Efficiency

The route drivers must take in their search for a parking bay is determined by the circulation design. It is most efficient, if all of the bays can be searched with just one circuit of the aisles and access-ways. However, this is not always the case, as often drivers must drive through the same aisle more than once. As a result, the circulation design has an impact on the circulation efficiency. This efficiency can be further improved through the use of signage and ability to bypass aisles (Hill et al., 2005). When exiting the car park, it is also desirable to be able to bypass the aisles with bays and drive directly to the exit using only the ramps, improving the circulation efficiency for exiting the car park.

Circulation efficiency can be expressed as a percentage of the shortest travel distance, which is to pass through an aisle with parking bays on both sides, entering from one side and exiting on the other. When ramps and access-ways are used, the circulation efficiency will drop, since extra distance has to be travelled (Hill et al., 2005). Circulation efficiency can be expressed as equation 2.1, in which the travel distance is measured along the centre of the aisles and access-ways.

$$\text{Circulation Efficiency} = \frac{\text{Width of parking bays} \;/\; 2}{\text{Travel distance} \;/\; \text{Number of bays}} \tag{2.1}$$

2.5.1.2 Crossovers

For user-friendliness in general, the layout of a car park should not hold any surprises and therefore, drivers should be able to observe other motorists movements. Crossover conditions may result in conflicts and should therefore be avoided (Hill et al., 2005).

2.5.1.3 Dead Ends

In search for a parking bay, it is difficult to see into a dead end (cul-du-sac). Therefore, dead ends should be avoided. If they are used anyhow, the number of bays in length should be limited to six (The Institution of Structural Engineers, 2011).

2.5.1.4 Parking Angle and One-way vs Two-way Aisles

In the circulation design, one-way or two-way aisles can be used. Both have their advantages and disadvantages. When two-way aisles are applied, parking bays will have a 90º angle. In case of one-way aisles, 90º parking bays as well as bays with a smaller angle may be applied.

An advantage of using one-way aisles is the prevention of conflicts between two cars, coming from opposite directions, looking for a vacant bay (Stuart, 2007). Furthermore, it can reduce confusion and congestion. A possible safety hazard appears, when drivers disregard the one-way traffic flow when they see a vacant bay (The Institution of Structural Engineers, 2011).

The advantages for one-way aisles become bigger when smaller parking angles are used. It makes it easier for drivers to enter and exit parking bays, with better visibility when backing out (Stuart, 2007). As a result, parking bays with a smaller angle have a higher dynamic and turnover capacity (The Institution of Structural Engineers, 2011). According to Stuart (2007), using 75º parking results in a more efficient layout, providing more space as a result of smaller aisles. However, according to The Institution of Structural Engineers (2011) reductions in the order of 3% are to be expected for parking angles between 90º and 70º up to 20% for 45º. Hill et al. (2005) and Troup & Cross (2003) also state smaller angle parking to be less efficient. In general, the efficiency of the parking layout is a combination of parking angle, aisle width, location of turns, ramps and barriers. An example of different parking layouts for varying parking bay angles can be seen in Figure 17.

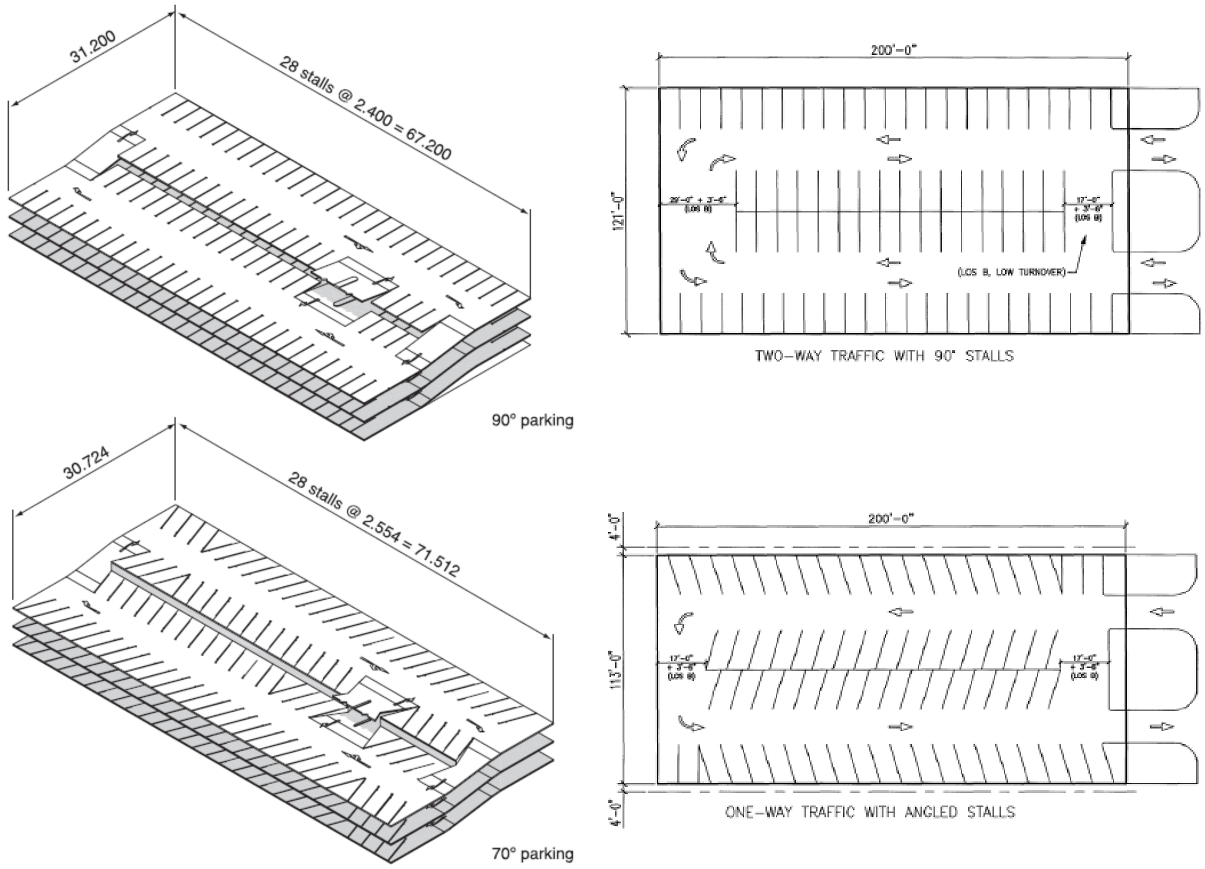


Figure 17. Comparison between different parking layouts for varying parking bay angles (Hill et al., 2005; Stuart, 2007; Troup & Cross, 2003)

2.5.2 Circulation Layouts

Many different circulation layouts for multi-storey car parks exist, each with its own advantages and disadvantages. Choosing the right circulation layout is key in satisfying motorists and clients. Basically it should be simple (Hill et al., 2005). When choosing a circulation system, a critical factor is the volume of vehicles expected to arrive and depart in the peak hour, which has to do with the intended purpose of the car park (short stay or long stay) and its capacity (Stuart, 2007). Another factor influencing the choice for a certain layout is the layout of the site. In general, three different type of layouts can be distinguished: flat decks, split-level decks and sloped parking decks. Within these categories, still many options exist, which mostly differ in the type and location of ramps used and the circulation routing. The general advantages and disadvantages of the three categories are discussed below. A detailed description of the different types within these categories can be found in appendix B. A summary of this appendix in tabular format can be found in paragraph 2.5.2.4, Table 6.

2.5.2.1 Flat Decks

Flat deck car parks are normally built in multiples of a bin width and have an adaptable layout (The Institution of Structural Engineers, 2011). Overall, three different types of flat deck layouts can be distinguished, based on the ramp system used: internal ramps (see Figure 18 left), half external ramps and external ramps (see Figure 18 right). Generally speaking, internal ramps result in low dynamic and static efficiency. Half external ramps cost extra, but reduce the amount of bays used to complete the circulation route, boosting dynamic efficiency. In small car parks, half external ramps may be uneconomic, but with larger car parks, its static efficiency increases. External ramps function independently form the car park and are often used in large car park facilities, providing high dynamic efficiency and for large car parks reasonable static efficiency. (Hill et al., 2005)

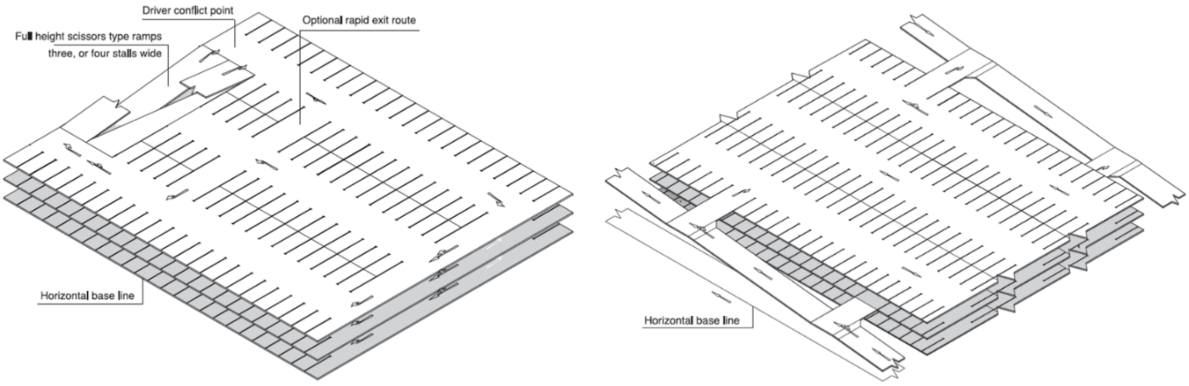


Figure 18. Two examples of flat deck circulation layouts. Left: one-way flow with side-by-side ramps (scissors type). Right: storey height, straight ramps. (Hill et al., 2005)

2.5.2.2 Split-level Decks

Split-level deck car parks generally have a good static and dynamic efficiency and are easy to drive around. They can accommodate large capacities with rapid in- and outflow routes, with most variants using one-way aisles, allowing any angle of parking. These layouts with their internal ramp systems (see Figure 19) are very compact and have a high static efficiency, most varying around 21 m^2 per car. The shortest variant can be constructed with a length of just 24 metres, resulting in a static efficiency of 26.75 m^2 per car. Except for the variants which are three or more bins wide, for drivers all turns are in the same direction and no single turn is greater than 90º. (Hill et al., 2005)

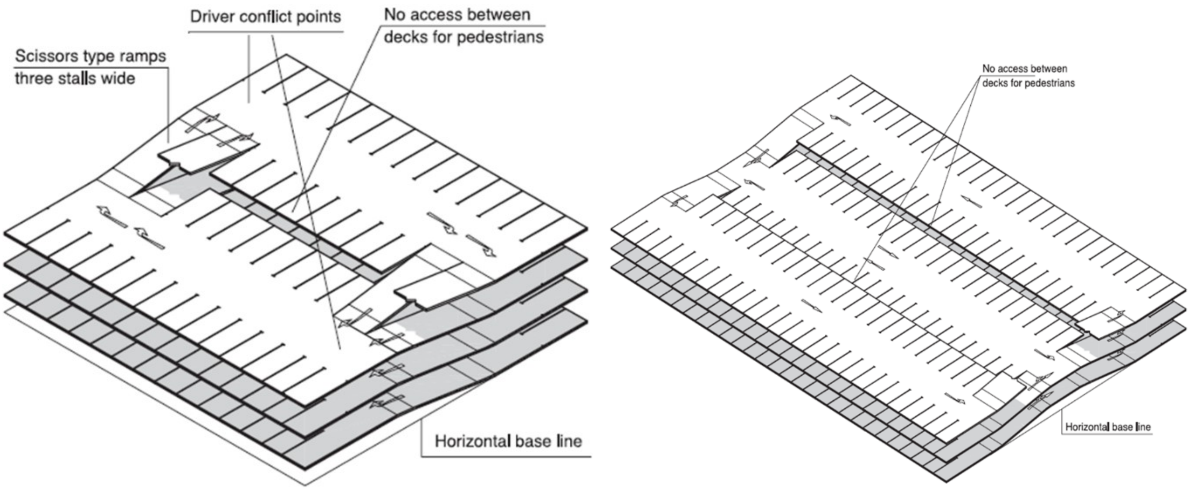


Figure 19. Two examples of split-level deck circulation layouts. Left: one-way flow with side-by-side ramps (scissors type). Right: combined one-way-flows, three bins or more wide. (Hill et al., 2005)

2.5.2.3 Sloped Parking Decks

Sloped parking decks switch the ways parking decks and ramps are constructed. They use sloped parking decks with a limited slope and flat ramps, see Figure 20. The slope of the parking deck is limited (maximum 5%) by the effect of gravity on opening and closing of doors. Also for disabled drivers in wheelchairs and the use of shopping cars, sloped parking decks are a disadvantage. These layouts generally have good static efficiency between 20 and 22 m^2 per car. The dynamic efficiency largely depends on the variant and whether it has a rapid in- and/or outflow route. Sloped parking decks can also be combined with one of the normally sloped decks replaced by a flat deck to compensate some of the disadvantages of a sloped parking deck. This however results in longer parking decks, greater than 72 metres. (Hill et al., 2005)

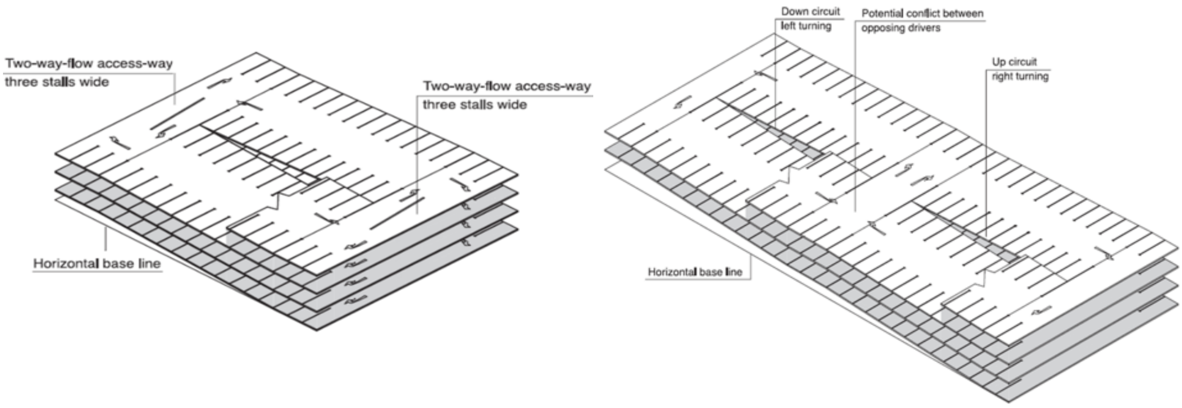


Figure 20. Two examples of sloped parking deck circulation layouts. Left: two-way-flow single helix. Right: double helix, end connected with one-way-flow on the central access-way. (Hill et al., 2005)

2.5.2.4 Overview of Layouts

The various possible layouts of the three different categories of circulation layouts are presented in detail in appendix B. Aspects such as its efficiency, circulation routing and minimum dimensions are discussed here. An overview of these aspects is presented on the next page in Table 6.

Table 6. Combined overview of different characteristics of car park layouts

Circulation layout	Average static efficiency (most compact or extra large decks)	Dynamic efficiency	Circulation efficiency	Recir- culation	Outflow route	All turns in same direction	No turn greater than 90°	Conflict points	Dead ends (depth)	Minimum width	One- or two-way aisles	One- or two-way ramps	Maximum capacity
Flat decks													
Internal ramps running across the bins	23.5 m^2/car	Low	Low	Simple	Reasonably	Yes	Yes	Many	Yes	3 bins	One	One or two	
Internal ramps running parallel with the traffic aisles	28 m^2/car	Medium	Low	Simple	Reasonably	No	Yes	Some	No	3 bins	One	One or two	
Half external ramps	25 m^2/car	Good	Medium	Simple	Rapid	No	Yes	Some	No	2 bins	One	One or two	
Circular external ramps	28 m^2/car (24 m^2/car)	High	High	Simple	Rapid	No	Variant dependent	Variant dependant	Variant dependant	1 bin	One or two	One or two	
Storey height, straight external ramps	21 m^2/car	Good	High	Simple	Rapid	No	Yes	Some	No	2 bins	One or two	One or two	
Split-level decks													
One-way-flow with side-by-side ramps (scissors type)	21 m^2/car (27 m^2/car)	Low	Very high	Simple	Slow	Yes	Yes	Many	No	2 bins	One	One	
One-way traffic flow with an excluded rapid outflow route	22 m^2/car	Medium	High	Simple	Rapid	Yes	Yes	Little	No	2 bins	One	One or two	1100 bays
One-way traffic flow with an included rapid outflow route	22 m^2/car	Medium	Very high	Simple	Rapid	Yes	Yes	Many	No	2 bins	One	One	400 bays
Two-way-flow with 'combined' ramps	21 m^2/car (27 m^2/car)	Low	Very high	Difficult	Slow	No	No	Many	No	2 bins	Two	Two	
Three bins or more wide	21 m^2/car	Low	Low	Simple	Slow	Yes	Yes	Many	No	3 bins	One or two	One	
Sloped parking decks													
Single helix	22 m^2/car (24 m^2/car)	Low	Very high	Difficult	Slow	No	No	Many	No	2 bins	Two	Two	300 bays
Single helix with half external ramps	22 m^2/car	Medium	Very high	Simple	Rapid	No	Yes	Little	No	2 bins	One	One	600 bays
Double helix	21 m^2/car (20 m^2/car)	Medium	Very high	Simple	Slow	No	Yes	Many	No	2 bins	One	One or two	
Combined helix	23m^2/car (21 m^2/car)	Low	Low	Simple	Slow	Depends c	Yes	Many	No	3 bins	One or two	One	
Combined flat and sloping deck layouts with internal cross-ramps	22 m^2/car	Good	Medium	Simple	Reasonably	Yes	Yes	Some	No	2 bins	One	One or two	

Legend

Highest static efficiency	Very preferential characteristic	One-way
	Preferential characteristic	One or two-wa
	Weak characteristic	Two-way
Lowest static efficiency	Problematic characteristic	
	Variant dependant	

2.6 Structural Typologies

The structural system of a multi-storey car park consist out of many different elements. In general, one can split up the system into a deck system, a framing system, a stability system and a foundation. Except for the foundation, different typologies for these systems are discussed below.

2.6.1 Deck System

The deck system is one of the most important parts of a car park. According to Cross (2002), the framing system is not the part of a car park that will fail, the deck is. The cause of deck failure is often moisture and chlorides attacking the rebar in concrete or steel in the decks. To protect the deck, a membrane has to be applied or other sufficient measures should be taken to prevent water and chlorides from entering the deck system. Several different types of deck systems exist of which the most common are described below. The different deck systems all have their favourable accompanying framing system, described in the next paragraph for which different building materials can be chosen. Other types of deck systems can be thought of, but the types of deck systems presented below and its way of transferring forces to the rest of the structure can be used as reference material.

2.6.1.1 Long-span Prefab Elements

Long-span prefab elements like double-tees are fabricated in a factory in a controlled environment, resulting in high quality products and minimizing on-site work. They are transported to the building site and hoisted into place (see Figure 21), ready for use, resulting in a very fast erection process. Double-tees are a very economical and durable option (Simon, 2002).

Another long-span prefab element used in car parks that is an economic solution are hollow core slabs, see Figure 21 (Lemieux & Van Kampen, 2018). Opposite to double-tees, hollow core slabs often require an in-situ applied structural screed to be able to drive on the floor. This extra step slows down erection a bit and results in a solid floor slab, which is more difficult to dismantle. Nevertheless, hollow core slabs are still a fast construction technique and relatively light weight. A big advantage compared to double-tees is the ability of hollow core slabs to accommodate free formed floor plans (Soons, Van Raaij, Wagemans, Pasterkamp, & Van Es, 2014). Special care should also be taken to prevent water and de-icing salts from entering the concrete. A membrane and seep holes are possible solutions.

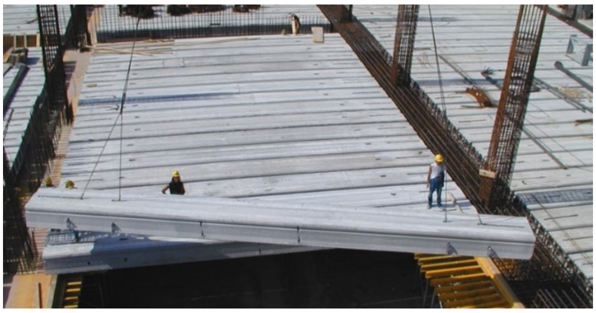


Figure 21. Left: Double T floor (Haitsma Beton, 2018). Right: Hollow core slab floor (Croom Concrete, 2018).

2.6.1.2 Short-Span Prefab Elements

For shorter spans (< 10 m), precast concrete planks can be used as a deck system. It is possible to use these planks as formwork, to create a solid floor slab. However, it is also possible to simply use completely prefabricated concrete deck elements and place them directly on the supporting girders, making construction very fast (Max Boegl, 2020). Through the use of shear connectors on the supporting girders, composite action can also be realized

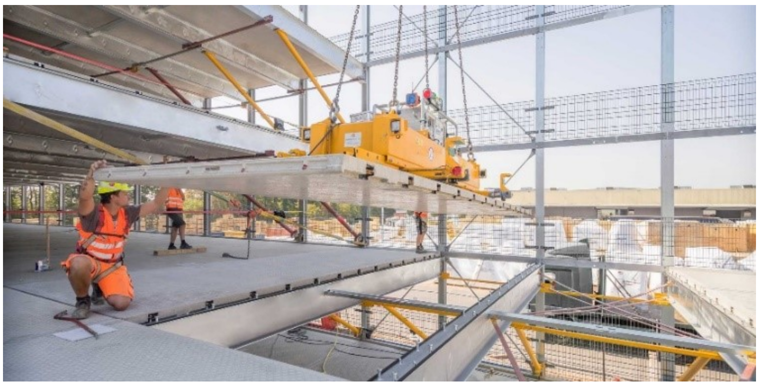


Figure 22. Prefab concrete floor elements (Max Boegl, 2020).

with this type of floor elements, see Figure 22. Since these elements are created in the factory, a very high quality product can be delivered, able to withstand de-icing salts.

A new development of building materials in the construction industry are composite materials. Short-span floor elements can also be made using composite glass fibre. A big advantage of this material is that it is very lightweight and durable. It is not affected by chlorides, making it a very interesting product to use in a car park.

2.6.1.3 Composite Floors

Several types of deck systems make use of composite action. Possibly the most known and often used in car parks are corrugated sheet metal sheets combined with in-situ concrete, see Figure 23. The metal sheets act as formwork for the concrete, which speeds up construction as no separate formwork is required. The metal sheets also act as main reinforcement with additional reinforcement to minimize cracking, which could lead to ingress of de-icing salts in a car park deck. This deck system has a low self-weight and can span up to 9.60 m. The depth of the floor can be reduced as the corrugated sheets can be placed on the widened bottom flange of a profile. Trough applying metal studs as shear connectors on the girders supporting the floor, the steel girder will take up tensile stresses and the concrete compression, resulting in a composite beam. (Sarka, 2005; Soons et al., 2014)

Figure 23. Corrugated sheet metal floor under construction in car park (SMD Ltd, 2019).

2.6.1.4 Cast-in-place

A cast-in-place deck has the advantage of giving a high degree of crack control, which is important in car parks. Though post-tensioning, the thickness of the floor can be reduced and cracking can again be controlled. Also free floor plans can be constructed (Kaminker, 2004). Disadvantages of the system are the slower construction and high self-weight. An example of an cast-in-place car park can be found in Figure 24.

Figure 24. Cast-in-place car park (City of Stonnington, 2018).

2.6.2 Framing System

The framing system of a multi-storey car park is build-up of columns and beams supporting the floor elements and a certain stabilizing system, keeping the system upright. The most used framing systems are shown below. More systems and many variations can be thought of, but these systems cover most general principles of the available systems. Different materials can be used for the systems, although not every system might be useful or efficient for each building material. Similarly, not every framing system works with every deck system. Certain combinations of deck system, framing system and chosen building material are used often. Below, different structural typologies for the framing system are presented as potential options for the structural design of the car park.

2.6.2.1 Two-way Carrying Slab

The most basic looking framing system is shown in Figure 25. It is composed of a set of columns, which carry a two-way carrying floor slab. Considering building materials, this type of system is only constructed using concrete. Many variations exist, for example using drop panels around columns to limit floor depth or a waffle slab, minimizing self-weight and resulting in a more efficient structure (ACI Committee 362, 1997). Columns can be located at the end of the parking bays, resulting in a column free bin or can be placed between bays, minimizing floor depth.

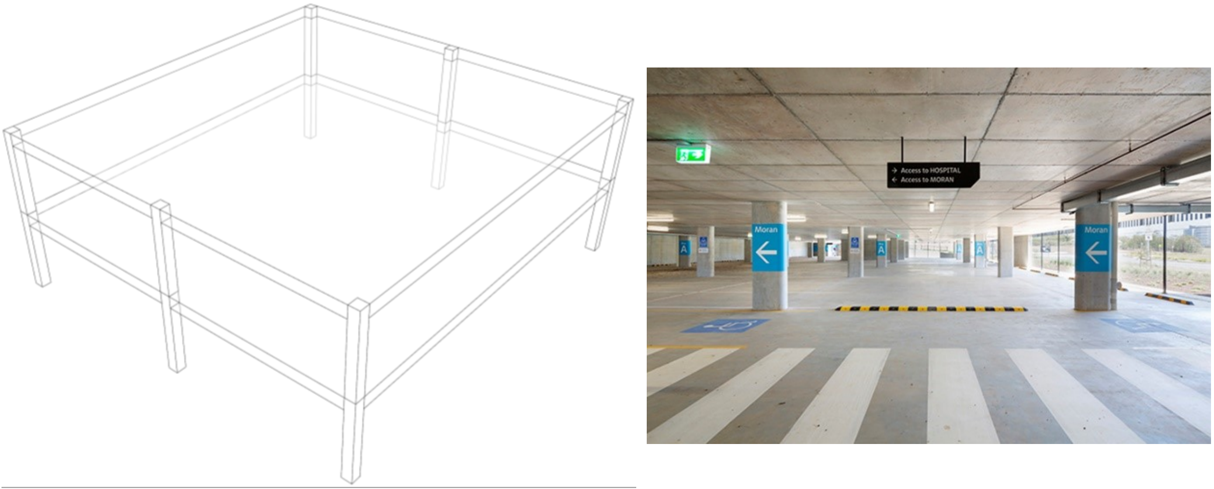


Figure 25. Two-way carrying slab framing system (AMC, 2021).

2.6.2.2 Bin Wide Floor Elements

Many (modular) car parks are constructed using double-t floor elements (Englot & Davidson, 2001). This type of floor requires a very simple framing system, comprised out of columns, connected by a girder on top of which the floor elements are placed, which span the entire parking deck, see Figure 26. Other floor elements like hollow core slabs can be thought of, but it is a challenge to find those able to span the parking deck up to more than 16 m.

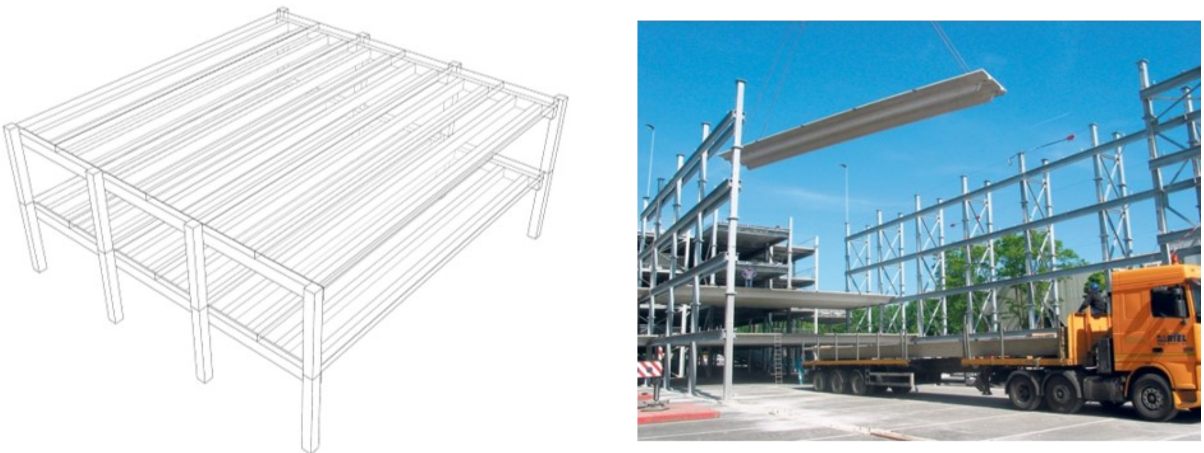


Figure 26. Bin wide floor elements framing system (Cobouw, 2017)

2.6.2.3 Bin Wide Girders Supporting Perpendicular Floor Elements

This framing system consists out of columns, which carry girders spanning the entire parking deck, see Figure 27. Floor elements are placed perpendicular to the girders, reducing their span. A variant of this system requires less columns, which are replaced by an extra girder spanning in between the columns in the other direction and carry one girder supporting the deck, see Figure 28. This type of framing system can be constructed using many different building materials and types of girders.

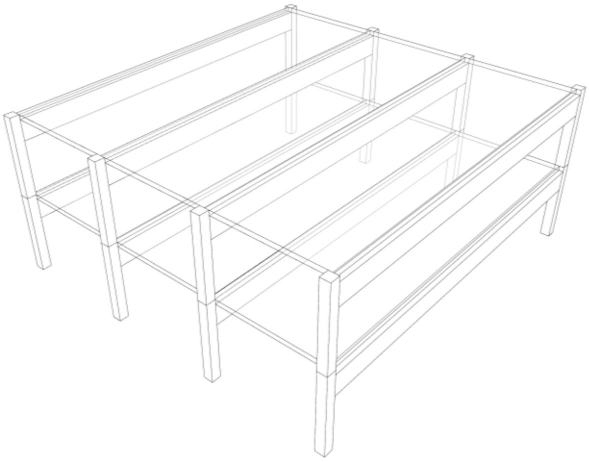
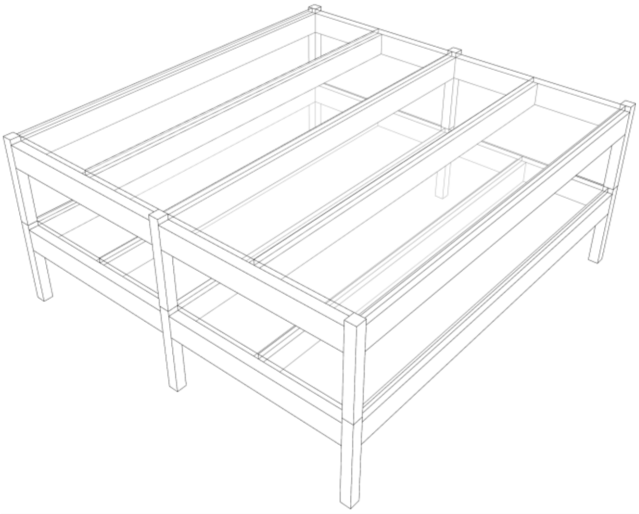


Figure 27. Bin wide girders supporting perpendicular floor elements framing system (Zaugg, 2018).



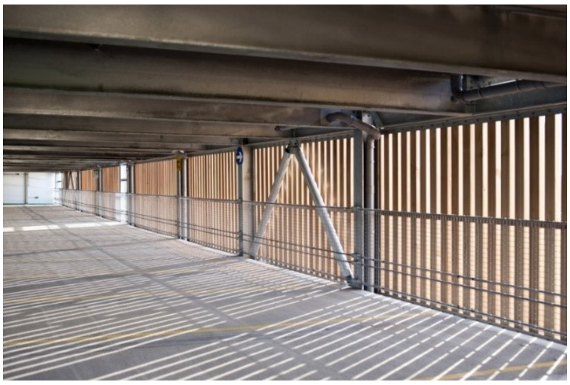


Figure 28. Bin wide girders supporting perpendicular floor elements with extra girder framing system (Bourne Parking Ltd., 2012).

2.6.2.4 Bin Wide Girders with Perpendicular Joists Supporting Floor Elements

Reducing the span of floor elements, this framing system is build-up of columns, girders spanning the entire parking deck and joists in between the girders, see Figure 29. The shorter floor spans presents opportunities to use floor elements with a significantly smaller depth. Considering the different types of spans of beams and floor, different building materials can be used.

Figure 29. Bin wide girders with perpendicular joists supporting floor elements framing system (Park4all, 2019b)

2.6.2.5 Girders Across Parking Bays with Primary and Secondary Joists Supporting Floor Elements

This framing system looks rather complicated. The system is normally used featuring columns in between the parking bays, see Figure 30. The columns are connected by girders, which carry a grid of joists on top of which a floor can be placed. This system is useful when very thin floor elements are used, which require a small span. As a lot of connections have to be made, construction can take longer for this type of system.

Figure 30. Girders across parking bays with primary and secondary joists supporting floor elements framing system (Osborne, 2019).

2.6.3 Stability System

2.6.3.1 *Vertical Stability*

To resist lateral loads, several types of vertical stabilizing systems can be applied: clamped columns, diagonal braces, moment frames, shear walls and cores.

Columns clamped in the foundation can provide stability for relatively low car parks (up to three levels) (Van den Broek, 2019). For higher car parks, the forces on the connection will become too large and another type of stability system has to be incorporated.

Diagonal bracing systems are generally the most economic option (Brinksma, 2004). A double diagonal cross using steel elements is often applied (X-bracing), in which case the members are only loaded in tension. Other types of bracing exist, like K, X or V braces, see Figure 31 (Ülker, Işık, & Ülker, 2017).

Moment frames are a more expensive option and may result in complex details (Troup & Cross, 2003). An additional benefit however is the possible reduction in depth of beams, which may result in a less thick floor.

Shear walls are another solution to provide stability to a building. Such walls can be found in car parks, made out of concrete. A disadvantage of shear walls is that they can block sight lines. Parking garages are preferably open with clear sight lines to provide a feeling of safety (The Institution of Structural Engineers, 2011).

A stiff core like an elevator shaft can also be used to provide stability. Such a single stiff core is more often used in higher buildings and not in relatively low and open structures like car parks. It is however possible to use the stairwells as stabilizing elements.

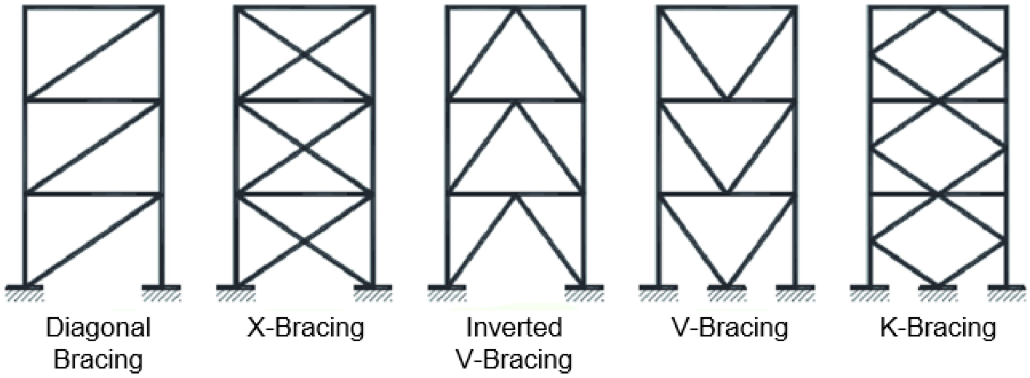


Figure 31. Different types of diagonal braces (Ülker et al., 2017)

2.6.3.2 *Horizontal Stability*

Horizontal stability of car parks is often provided by its floors, which can transfer horizontal loads to the stabilizing elements in the building, which can transfer the loads to the foundation. Depending on the type of floor system used, floor elements might have to be coupled together to be able to transfer these loads. Other types of horizontal stabilizing elements often used are diagonal braces. Both types of horizontal stabilizing systems can are shown in Figure 32. (Van den Broek, 2019)

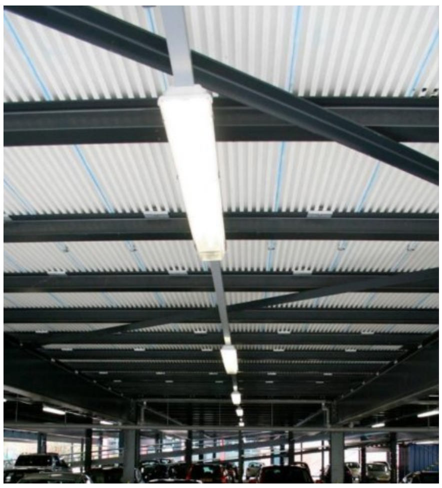


Figure 32. Left & middle: Couplers between deck panels to create horizontal stabilizing element. Right: Diagonal bracing as horizontal stabilizing system (Park4all, 2019a).

2.7 Concluding Remarks

To conclude this chapter, sub-question 1a, “What are the design requirements for a multi-storey car park?”, and 1b, “What are the possible typologies for a multi-storey car park?”, can be answered.

Considering the design requirement for a multi-storey car park, we can conclude that users of car parks are mostly concerned with functional requirements like the (dynamic) capacity of a car park and how easy the circulation through them is. Such requirements are translated by the parking norm NEN 2443 into a large set of minimum and standard dimensions, rules for layouts of ramps of placement of columns and other general requirements. If these requirements are incorporated in the design of a multi-storey car park, one can say with reasonable certainty, that the design will be appreciated by its users.

The owner’s requirements of course also reflect the requirements of the users, since they aim to provide their clients with a pleasurable parking experience. However, apart from such requirements, the owners are also concerned with aspects of the construction that affect their ability to get a return on their investment. This means they require a construction, which is durable, fast to construct and requires minimal maintenance. Furthermore, a high static efficiency and low average cost per parking space should result in a cost-efficient design. Last of all, more recently, owners are also concerned with the sustainability performance of the design as well as its flexibility for future use in other functions.

Different possible typologies for the design of a multi-storey car park have also been presented. These typologies vary between general layouts of car parks, structural typologies for its construction and different types of elements that can be used within them. The different requirements of the users and owners of the car park have to be reflected by the chosen typology. This chapter has presented characteristics of the different typologies, which can be used to assess which car park layouts and structural designs are feasible. As designing is a very creative process with endless possibilities, this chapter functions as reference material for the further design process.

3

Challenges Towards a Sustainable Building Industry

As the building industry is in the middle of a transition towards a more sustainable industry, it has to change the way buildings are designed and constructed. This requires understanding what building sustainable is and how its principles can be translated into the design of a building. The transition can be supported by several developments and innovations that have been introduced into the building industry in recent years. Understanding how such innovations can help to bridge the gap between the traditional building industry and the future sustainable building industry will play an essential role in the speed with which the transition takes place. This chapter aims to provide an answer to what building sustainable is and with what challenges it comes. Furthermore, it aims to provide an answer to how these challenges influence the design of a building.

3.1 Building Sustainable

The building industry is not the only sector which is in a transition towards a more sustainable industry. It is part of a much broader topic, which concerns all people and industries; sustainable development. In 1987, the Brundtland Report (WCED, 1987) published today's most used definition of sustainable development: "Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs". The report highlights the inequality between poverty in one part of the world and the non-sustainable consumption and production in the other part. As a result, within sustainability, three key areas can be distinguished (Pitt, Tucker, Riley, & Longden, 2009):

1) Environmental responsibility
2) Social awareness
3) Economic profitability

3.1.1 Impact of the Building Industry

The building industry has a significant impact on sustainability as it affects all three key areas. In terms of environmental aspects, the construction and demolition of a building results in a large use of building materials and waste creation. The production of many building materials also results in carbon emissions and other pollutions (air, noise), which among others affect the bio-diversity. Last of all, during construction, but especially in the use phase, large amounts of energy and water are used (Czarnecki & Kapron, 2010; Pitt et al., 2009).

Buildings also affect social aspects of sustainability. The quality, design and performance of a building directly affects the quality of life in and around the building. Sustainable buildings are healthier to live and work in and can result in greater productivity. Also the location of a building with its access to services and recreation adds value to a society (Pitt et al., 2009). Social aspects also play an important role in the construction phase, as construction work is a physically heavy job in which health and safety is very important (Heller, Hawgood, & Leo, 2007).

Sustainable buildings also have to be funded, so economic aspects always play an important role. In the development of sustainable buildings, the client is a key driver, who is primarily focussed on the aspects of time and costs (Bowen, Cattel, Hall, Edwards, & Pearl, 2012). Sustainable construction therefore has to be made attractive. Although the initial cost of a sustainable building is higher, they attract higher rents and prices, attract tenants and reduce tenant turnover. Also, in operation and maintenance, sustainable buildings cost less. As the public is more concerned with sustainability, a sustainable building and Value Management (VM), minimizing environmental and social damage, can be attractive in attracting investments (Pitt et al., 2009). By making a cost-effective design, a sustainable building can also become feasible from a financial point of view (Czarnecki & Kapron, 2010).

3.1.2 Principles for Sustainable Construction

Pitt et al. (2009) point out the importance of the design and the design team of a building in sustainable development. Although the client is the key driver and final person to make decisions, the designers have the opportunity to influence and inform the client. They can build on their own reputation and image concerning sustainability. Decisions like the selection of building materials have a significant impact on energy use and greenhouse emissions. Choices made by structural engineers can reduce costs and use of materials. The design team can help demonstrate ways to a client for a faster or better return on investment, to increase flexibility, to reduce costs, to increase market appeal and to improve the clients image (Pitt et al., 2009).

Nine basic principles for sustainable construction can be distinguished (Babalola, 2017; Czarnecki & Kapron, 2010):

- Minimization of consumption of resources;
- Maximization of reuse of resources, components and structures (renovation);
- Use of renewable and recyclable resources;
- Waste management;
- Minimization of emission of pollutants;
- Protection and preservation of natural environment;
- Development of a healthy and non-toxic environment;
- Comfort of use (quality) of built environment;
- Quality control, construction process management and building management in developing the built environment.

3.1.3 Barriers of Sustainable Construction

The building industry is in the middle of a transition towards a more sustainable industry. There are many examples, in which new buildings are designed according to the principles of sustainable construction. However, sometimes projects start off in the feasibility stage of the design aiming for maximum sustainability levels, but scale down this ambition when costs start to rise or when contractors for example do not feel comfortable with new construction methods like timber construction (Fryer, 2012).

Several barriers have been identified as holding back sustainability in the construction industry. Pitt et al. (2009) did research into these barriers and found affordability to be the main barrier. Affordability is associated with the higher initial cost of sustainable buildings and a slow rate of return on the investment of this higher initial cost (Matar, Georgy, & Abou-Zeid, 2010). Another significant barrier is a lack of client demand, which was also found by Matar et al. (2010), who described a lack of interest from major industry stakeholders towards sustainable construction. This might be connected with another barrier, which is lack of client awareness. This stresses the importance of the design team to inform clients on sustainable construction. Other barriers are a lack of business case understanding, lack of proven alternative technology, building regulations, planning policy and lack of labelling or measurement standard. Matar et al. (2010) add to these barriers the fact that the construction industry is a very fragmentated industry, resulting in misperception and misunderstandings, which make implementation of sustainability practices amongst all partners in the value chain a challenge. Furthermore, they point towards a lack of sufficient education and training among industry practitioners, the weak and slow adoption of digital technology and the clatter, confusion and inefficiencies of current tools and approaches to sustainable construction.

3.1.4 Drivers of Sustainable Construction

Sustainable construction is also pushed by several drivers. Pitt et al. (2009) also researched these drivers and found out financial incentives and building regulations to be the most important. Many governments have a sustainability agenda and present for example fiscal incentives for sustainable construction. Through legislation, certain requirements concerning sustainability also end up in standards and building requirements. Other drivers are client awareness and demand, which are becoming more important as corporate and social responsibility practices towards the encouragement of sustainable practices are recognized by the public. Companies want their image and reputation to be connected to sustainability. Less important drivers were planning policy, taxes or levies and investment.

3.2 Circular Construction

Many different approaches, methods, tools and strategies have been developed to contribute to the transition of the building industry towards a sustainable industry. They function as a means to a goal of achieving sustainable development (Anastasiades, Blom, Buyle, & Audenaert, 2020). One of these approaches, which is seen as very promising and which incorporates many of the basic principles for sustainable construction is circular construction or the circular economy (Bocken, de Pauw, Bakker, & van der Grinten, 2016). Guerra & Leite (2021) describe the ultimate goal of the circular economy model as retaining resources circulating at their highest value within the boundaries of the planet, in such a way that no additional resources are required to produce materials, and the discarded materials are not perceived as waste. Products and materials are continuously reused and renewable resources are used where possible, see Figure 33 (Bocken et al., 2016). The focus of the circular economy is wider than just use of materials. It's about better management of the resources by refusing, rethinking and reducing unnecessary consumption (Guerra & Leite, 2021). The European commission also associates the move to a circular economy with opportunities to create jobs and boost economies, contributing to it being a means to true sustainable development (European commission, 2015).

Figure 33. Linear vs circular economy (Petovarga, 2019)

3.2.1 Resource Loops

It's clear that the flow of resources is one of the most important aspects in circular construction. Traditionally, the building industry has always adopted a linear flow of resources, starting with the mining of raw materials, followed by production of building materials and elements, the use phase and finally demolition, in which the materials become waste. The lifecycle of these resources is an open loop and can be referred to as cradle-to-grave. In a circular economy, the start and end-of-life phases of the building cycle are coupled, creating a closed loop, which is referred to as cradle-to-cradle (Guerra & Leite, 2021). Circular construction is concerned with the alteration of these loops, closing them. Bocken et al. (2016) have distinguished three different aspects on how resource loops can be altered: slowing resource flows, closing resource flows and narrowing resource flows. These aspects can be visualized on three axes, resulting in different types of loops, see Figure 34. Apart from linear and circular flows, different combinations with also extended and/or narrowed flows are formed.

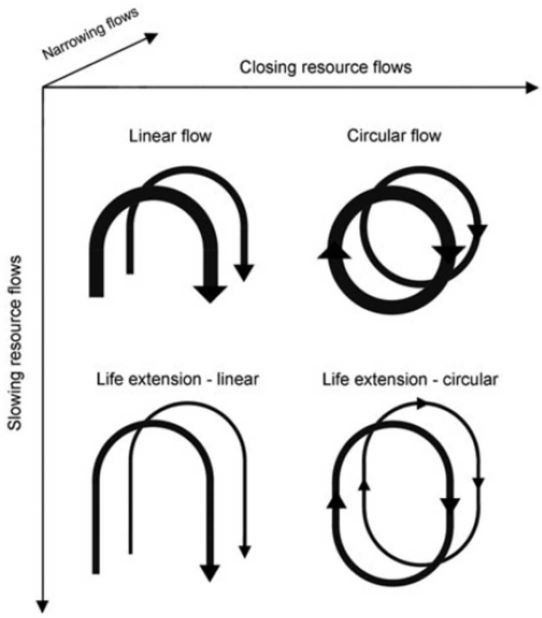


Figure 34. Different types of resource flows (Bocken et al., 2016)

3.2.2 Circular Design Strategies

Different circular design strategies can be defined and can be based on alteration of the resource loops as described above. Bocken et al. (2016) conducted research into these strategies and categorized them according to whether they aim at slowing, closing or narrowing resource loops. Guerra & Leite (2021) also did a literature study on circular design strategies, which can also be classified according to the categories of Bocken et al. An overview of the most cited circular design strategies can be found in Table 7. These strategies are discussed in the following paragraphs.

Table 7. Circular design strategies (Bocken et al., 2016; Guerra & Leite, 2021)

Design strategies to slow resource loops	**Design strategies to close resource loops**
Designing long-life products	Design for a technological cycle
Design for attachment and trust	Design for a biological cycle
Design for reliability and durability	Design for dis- and reassembly
Design for product-life extension	
Design for ease of maintenance and repair	**Design strategies to narrow resource loops**
Design for upgradability and adaptability	Design out waste
Design for standardization and compatibility	
Design in layers	
Design for dis- and reassembly	

One of the most often cited circular design strategies is to design for dis- and reassembly (Bocken et al., 2016; Dams et al., 2021). It's a strategy, which fits both strategies for closing and slowing loops. Through this strategy, it is possible to replace or repair components of a building, extending the buildings life. When a building is demolished, its components can be disassembled to be reused or recycled in other projects (Anastasiades et al., 2020). Dis- and reassembly require planning during the initial design stages. For example a well-documented disassembly plan in which recyclable and reusable materials are specified improves the possibilities for reuse of elements (Dams et al., 2021; Guerra & Leite, 2021). Also the way connections are designed is very important (Dams et al., 2021).

3.2.2.1 Slowing Loops

Design strategies concerned with slowing loops can be aimed at designing long-life products or at design for product-life extension. Strategies for designing long-life products are to design for attachment and trust and design for reliability and durability (Bocken et al., 2016; Dams et al., 2021). Goal of these strategies is to design building components, which are of high quality, ensuring a long utilization period. Important aspects for these strategies during design are the selection of materials and the appearance of the components. Durmisevic (2019) emphasizes the importance of the way materials are put together as this is essential for ultimate durability and circularity of buildings apart from the durability of its materials.

With design strategies for product-life extension, service loops are introduced, which extend the products life-span. Strategies are to design for ease of maintenance and repair, for upgradability and adaptability, for standardization and compatibility and to design in layers (Bocken et al., 2016; Dams et al., 2021; Guerra & Leite, 2021). Especially designing for adaptability is a very popular strategy as it allows for reconfiguration or conversion of a building, to reflect changes in its purpose or use during its lifespan, minimizing the risk of demolition as a result of changing economic, societal or functional demands (Anastasiades et al., 2020; Dams et al., 2021). Ease of maintenance and repair helps in keeping a building in optimal condition and standardization and compatibility results in parts that fit multiple products, making them interchangeable (Bocken et al., 2016). By designing buildings in layers, different components with different life-spans can be separated from each other, creating opportunities to replace layers or reuse them (Guerra & Leite, 2021).

3.2.2.2 Closing Loops

Design strategies concerned with closing loops, apart from designing for dis- and reassembly, are design for a technological cycle or for a biological cycle. In closing loops, the selection of materials is very important (Guerra & Leite, 2021). The technological cycle is concerned with designing products in such a way, that its materials can continuously and safely be recycled into new materials or products. The properties of the new materials or products should be a least of the same level as the previous materials or products. Downcycling therefore is not permitted as this will only slow down the resource flow, not closing it (Bocken et al., 2016).

The biological cycle is concerned with designing products using safe and healthy materials. These materials are biodegradable, so at the end of their lifespan, a new loop is started (Bocken et al., 2016). One can therefore say this is also a form of recycling.

3.2.2.3 Narrowing Loops

Design strategies concerned with narrowing loops are all about optimizing material use and minimizing the generation of waste (Guerra & Leite, 2021). The use of reused or recycled materials and products also reduces the generation of new waste.

3.3 Incorporating Circular Design Strategies

Several different circular design strategies have been presented. These strategies are often closely related to each other and for optimal and efficient results, all of them should be put into action. The following three key themes combining several circular design strategies are suggested:

- Modularization, prefabrication and standardization;
- Reuse, refurbish and recycle;
- Building materials.

Digitalization and digital tools are another development, which can support the three key themes above.

3.3.1 Modularization, Prefabrication and Standardization

A large group of construction methods, which adopt several circular design strategies is the group of modularization, prefabrication and standardization. Although these are three different concepts, they often go hand-in-hand (Guerra & Leite, 2021).

Architects often like to create unique, complex and aesthetically pleasing buildings, however considering circularity, the resulting buildings are often difficult to adapt and reuse. If more standard materials and components would be used, it would be easier to reuse these materials and components in other buildings and repairs would also be easier, extending the lifespan (Geldermans, 2016). Furthermore, the use of standard regular structural grids with standard dimensions, makes construction and adaptation much simpler (Dams et al., 2021). Standardization therefore facilitates the reuse and repurpose of building components.

Modularization and prefabrication are methods, which can achieve circular construction through standardization. As standard components are used, they can be combined into larger modules, which can be prefabricated in a factory. Through modularization and prefabrication, on-site material waste is reduced, working circumstances and safety are improved, higher quality products are produced and productivity is boosted (Guerra & Leite, 2021; Mischke, 2017). Furthermore, these methods can promote economic viability as project schedules can be reduced, efficiency is improved and costs are reduced (Dams et al., 2021).

3.3.2 Reuse, Refurbish and Recycle

Circular design strategies like designing for dis- and reassembly and designing for upgradability and adaptability are concerned with the theme of reuse, refurbish and recycle. To close resource loops, it is important to first reduce materials used, than reuse them and finally recycle them. To what extend materials and components can easily be reused or recycled depends on whether a building has been designed and composed in a demountable way (Kayaçetin, Verdoodt, Leferve, & Versele, 2022). A building is composed of several layers, which each have a different life expectancy. Often, the "six S'" system is used, which distinguishes the site, structure, skin (façade), services (mechanical and electrical), space plan (internal layout) and stuff (occupant possessions) (Brand, 1995), see Figure 35 left. To be able to reuse layers or replace them, extending the lifespan of the entire building, it is important that these layers are accessible and have reversible connections. Important aspects of these layers are material impact, functional independence, technical detachability, physical characteristics and recyclability (Kayaçetin et al., 2022).

The different layers of a building or systems themselves are also composed of different components and elements, see Figure 35 right. The connections between these elements and components also have to be demountable to be able to reuse components and materials to full potential. Dry mechanical connections like bolts, screws, dowels and clamps are preferred. Nails and toothed plates are less suitable for disassembly (E. Durmisevic, 2006; Morgan & Stevenson, 2005). Dry connections should be accessible in its broadest sense as Anastasiades et al. (2020) point out paint can hide screws, hindering disassembly. Durable and reusable connections can be made using stainless steel (Guy & Ciarimboli, 2008) and highly suitable connections for circular construction are interlocking or gravity-connected connections in shear (A. Durmisevic, 2019).

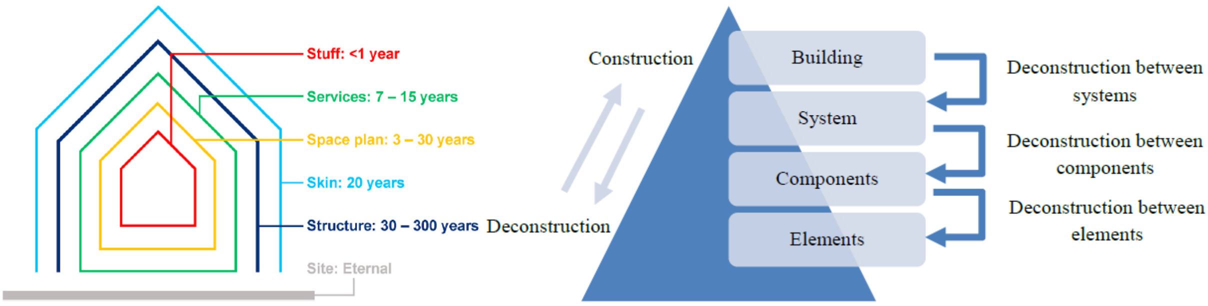


Figure 35. Left: The "six S'" diagram (Brand, 1995). Right: Levels of decomposition (Durmisevic, 2006).

Adaptability of a building starts with the design of the original building. Key principles for adaptive reuse are functional performance, structural performance, a buildings ecosystem and its visual appearance (Patil, Patil, & Patil, 2021). Functional flexibility can be created through designing large spans, maximising use of mobile partitions and simple open-plan space (Guy & Ciarimboli, 2008). Structural flexibility can be created through clever over dimensioning certain elements to suit future adaptations like larger foundation capacity or stronger columns, providing the possibility for additional floor levels in the future (Anastasiades et al., 2020). As Strumillo (2016) points out; reusing existing buildings presents the greatest opportunity for the reduction of energy consumption and carbon emissions.

3.3.3 Building Materials

The selection and specification of materials for building construction is a widely discussed topic in literature. It is a very important aspect in closing resource loops through the strategies of designing for a technological and biological cycle. In terms of the technological cycle, to be able to reuse materials, they have to be durable and of high quality as they have to be able to withstand destructive factors such as changes in temperature, humidity, static and dynamic loads and chemical or biological aggressions (Sagan & Sobotka, 2021). Materials can also be recycled, but it is important that the properties of the recycled materials are at least of the same quality as the original material. Furthermore, one should also take into account the process of recycling and how sustainable this process is.

Taking in mind the aspects concerned with demountability in the previous paragraph, one can evaluate main stream building materials like steel and concrete. A steel structure can easily be constructed using dry mechanical connections and its elements are easy to inspect and dismantle, making it easy to reuse them (Densley Tingley & Allwood, 2014). As a final option, steel can be recycled, creating new steel of the same quality. Although this requires 67% less energy compared to using virgin materials, it still does require the consumption of a large amount of energy (Johnson, Reck, Wang, & Graedel, 2008). Concrete is more difficult to reuse or recycle. Its elements are often connected through plastic joints, making disassembly very hard. Furthermore, concrete elements contain rebar, designed for a specific purpose. As the rebar is surrounded by concrete, it is difficult to inspect elements for reuse and if technical drawings are missing, it becomes impossible to see from the outside how an element is reinforced (Anastasiades et al., 2020). In terms of recycling of concrete, over the years, a lot of research has been conducted in this field of research. It is possible to recycle large amounts of concrete, however producing new concrete solely using recycled concrete is very difficult (Le & Bui, 2020). As a result, concrete is still often downcycled and not recycled.

The use of materials in the biological cycle should also be done in a clever way. The use of these materials is only environmentally beneficial if these bio-based resources are managed in a sustainable way. Timber can be such a material, however during its production and transportation, emissions are also produced. Sustainable production and a design life span matching at least the timber rotation period are therefore crucial (Anastasiades et al., 2020).

3.3.4 Digital Tools

The digital era has presented many new ICT possibilities to the world. Many industries have embraced digitalization and as a result have seen a boost in productivity and efficiency (McKinsey & Company, 2017). Digitalization can also provide a supportive role in circular construction. A software database can store all kinds of data of a building and can be coupled to all kinds of design and planning applications. Building materials and elements from a donor site can in this way be registered, creating an inventory of available construction elements for reuse (Dams et al., 2021). Using for example parametric engineering software, new designs can be optimized or designed using standard elements. It is clear that digital tools provide many possibilities to support circular construction.

3.4 Sustainability Assessment

As presented, many different circular design strategies can be applied to the design of a building. Furthermore, other sustainability strategies aimed at for example minimizing the use of energy in the use phase of a building or generation of green energy can be applied. To be able to say something about the sustainability performance of a building, somehow an assessment should be made.

Many different types of sustainability assessment methods exist. One of the most used type of methods is a Life Cycle Assessment (LCA). Goal of a LCA is to measure the environmental impact of a product over its life cycle from cradle-to-grave (Muralikrishna & Manickam, 2017). LCA's however have several shortcomings if one wants to assess the circularity of a building. LCA's tend to be not practically applicable enough during the design and development phase of a building. To make them useful, they should be simplified. Also, LCA's consider the complete life cycle of a product, but it has been found that in many studies, the end-of-life phase is not or not consistently taken into account (Anastasiades et al., 2020). With circular design strategies, aiming to close resource loops and reuse materials and elements, the end-of-life phase or with it the start of a new cycle is very important. LCA's do not assess the reuse performance of a product enough for a good circularity assessment (Anastasiades et al., 2020).

Today, buildings are often rated for their sustainability performance using a certain certification. Widely used certification systems are LEED and BREEAM (Guerra & Leite, 2021). Again, these frameworks assess the sustainability performance of a building and apply some LCA principles, but they are no good ratings for circularity as their focus is not on the circular design strategies (Anastasiades et al., 2020).

To assess the circularity performance of a building, Anastasiades et al. (2020) suggest to perform a Multi-Cycle Assessment (MCA). Such a method is especially focussed on circular design strategies like designing for disassembly and adaptability. Dams et al. (2021) have developed such a framework, which they call the Circular Construction Evaluation Framework (CCEF). It has specifically been developed to complement LCA assessments on circularity topics and has been developed to be used in the early design and planning stages of a project. The framework evaluates a building as a whole as well as its elements on component level. On whole building level, the main themes are: recorded information, adaptability in design, simplicity in design and health and safety. On component level, the main themes are: durability, material inventory, finishes/treatments, reversibility of connections, reusability and recyclability.

3.5 Concluding remarks

In conclusion of this chapter, an answer can be provided to sub-question 2: "How do the challenges faced by the construction industry towards a sustainable industry influence the design of a temporary multi-storey car park?". Also both its sub-questions can be answered: 2a "Which sustainability building strategies can be applied on the design of a temporary multi-storey car park?" and 2b "What is the impact of sustainability building strategies on the design of building elements?".

It is clear that the building industry can make a significant impact on sustainable development. Construction projects are linked to all three key areas of sustainable development; environmental responsibility, social awareness and economic profitability. Contributing to a sustainable building can be done using the presented nine principles for sustainable construction. Circular construction has been presented as a very promising approach, which incorporates many of these basic principles and gives more guidance on how a sustainable design can be realised. Circular construction aims to make the transition from the linear economy model, which is traditionally used by the building industry, towards a circular economy, in which products and materials and are continuously reused and where renewable materials are used where possible. Three circular design strategies have been identified, which are aimed at altering the resource flows in the building industry, to make the transition from a linear to a circular economy. These are: slowing resource loops, closing resource loops and narrowing resource loops. For each of these strategies, several measures have been presented.

Incorporation of these circular design strategies has a significant impact on the design of a building. Three key themes have been identified, which incorporates the different circular design strategies and affects the design (process) of a building. Modularization, prefabrication and standardization will fundamentally change the way buildings are designed and produced, as different buildings are designed using similar components. The second theme of reuse, refurbish and recycle, makes use of the first theme as it means standardized building elements can be reused in different building, maximizing its lifespan. The third theme is concerned with the use of different (renewable) building materials, which requires designers and contractors to rethink how they can build with such materials. Digital tools can fulfil a supportive role in this renewed design and construction process.

4

Timber Structures

Timber as a building material has a very long history. Where it started with logs being stacked on top of each other in order to form a structure, over many centuries highly-developed woodworking techniques have been developed. Together with other developments in construction techniques and machinery, this has resulted in an continuing growth of possibilities in the field of timber construction. More recent developments in wood products have paved the way for timber to be used in an even broader range of structures with larger spans, increased loads and higher elevations. This chapter aims to answer the question on how to construct using timber as a constructive material. It gives an overview of the different wood products and components used in construction today and types of connections used. Furthermore, an overview of durability aspects is given and what measures should be taken, as this is inextricably linked to building with timber. Last of all, some reference projects concerning the use of timber in car parks are presented.

4.1 Wood Products

Many types of wood products can be distinguished. Naturally, a division can be made between soft- and hardwoods and the different species amongst them. However, more recent developments in the timber industry have resulted in so-called engineered wood products. These products have improved characteristics, increasing the potential use of timber in constructions. Each type of product has different characteristics, determining its potential use. Apart from soft- and hardwoods, wood products can be categorized as solid wood and solid wood products or wood-based products. Differences between these products are based on the components they are made of as can be seen in Table 8.

Table 8. Wood construction products and their components (Blaß & Sandhaas, 2017)

	Wood product	Components	
Solid wood products	Logs	Stems	Variance declines ↓
	Sawn timber	Squared timber, planks, boards and battens	
	Glued laminated timber (glulam) & Cross-laminated timber (CLT)	Boards	
Wood-based products	Laminated veneer lumber (LVL)	Veneers	
	Plywood	Veneers or sawn timber	
	Parallel strand lumber (PSL)	Veneer strands	
	Particleboards	Particles (chips)	
	Fibreboards	Fibres	

4.1.1 Soft- and Hardwood

Wood can be classified in two different types: softwood and hardwood. Most timber products are made from softwood, about 80% (Arets et al., 2011). Also in structural timber, both soft- and hardwoods are used. Compared to softwood, hardwood is much stronger. Although its strength is much higher, other characteristics like the modulus of elasticity and the stiffness of these structural elements do not increase to the same extent as their strength (Kaufmann, Krötsch, & Winter, 2018). Softwoods are less dense, in general easier to work with and much cheaper than hardwood as it grows much faster (Diffen, n.d.). Softwoods are most popular in construction, but the increasing availability of hardwoods and new technological innovations open up new possibilities (Kaufmann et al., 2018).

4.1.2 Solid Wood and Solid Wood Products

Solid wood and solid wood products are made using sawn timber elements. These can be combined and rearranged to create improved wood products. These solid wood products however remain build-up of wood elements in their original form.

4.1.2.1 Solid Timber

Solid timber are members completely made of wood in its natural structural form. These members are produced through sawing logs longitudinally. The resulting products depending on their cross-sectional dimensions are boards, planks, battens and square timber (Blaß & Sandhaas, 2017), see Figure 36 left. Sawn sections can also be glued together to form products like four-piece beams and duo/trio beams (Herzog, Söffker, & Thrift, 2004).

4.1.2.2 Glued Laminated Timber (Glulam)

Glulam members are produced through laminating boards arranged parallel to the grain together with an adhesive, see Figure 36 middle. As growth-related defects in the wood are partly eliminated, the result is a rigid more homogeneous and therefore stronger beam. Glulam is an engineered wood product and allows for larger cross-sections than sawn timber, but besides simple, straight components, also allows forms with a variable cross-section and/or single or double curvature or twist about the longitudinal axis. It is also possible to mix different strength class timber boards, resulting in a non-homogeneous member. These glulam members are particularly suited to accommodate bending stresses. They are often used as beams and columns. The maximum length of the elements is limited by transportation. (Blaß & Sandhaas, 2017; Herzog et al., 2004)

4.1.2.3 Cross-laminated Timber (CLT)

Cross-laminated timber (CLT) is another engineered wood product. Unlike glulam, which result in linear members, CLT products are plate-shaped, see Figure 36 right. It is made up of multiple cross-wise arranged board layers, which are glued together and can be produced in a wide range of panel thicknesses. The cross-sections can take many shapes as it is possible to make CLT panels partially filled with gaps or with an asymmetrical cross-section. It is also possible to bent a panel into a plane curve. Like with glulam, CLT panels take advantage of being more homogeneous elements, but this time the panels are approximately isotropic in plane. The panels have high strength and stiffness properties and have a high load-bearing capacity relative to their self-weight. They can be loaded perpendicular to the plane (bending parallel or perpendicular to the grain of the external layers) or in plane (bending, compression and tension in plane and parallel or perpendicular to the grain of the external layers). CLT is often used as wall and floor elements, but can also be used for beams or shell structures. (Blaß & Sandhaas, 2017; Föreningen Sveriges Skogsindustrier, 2019)

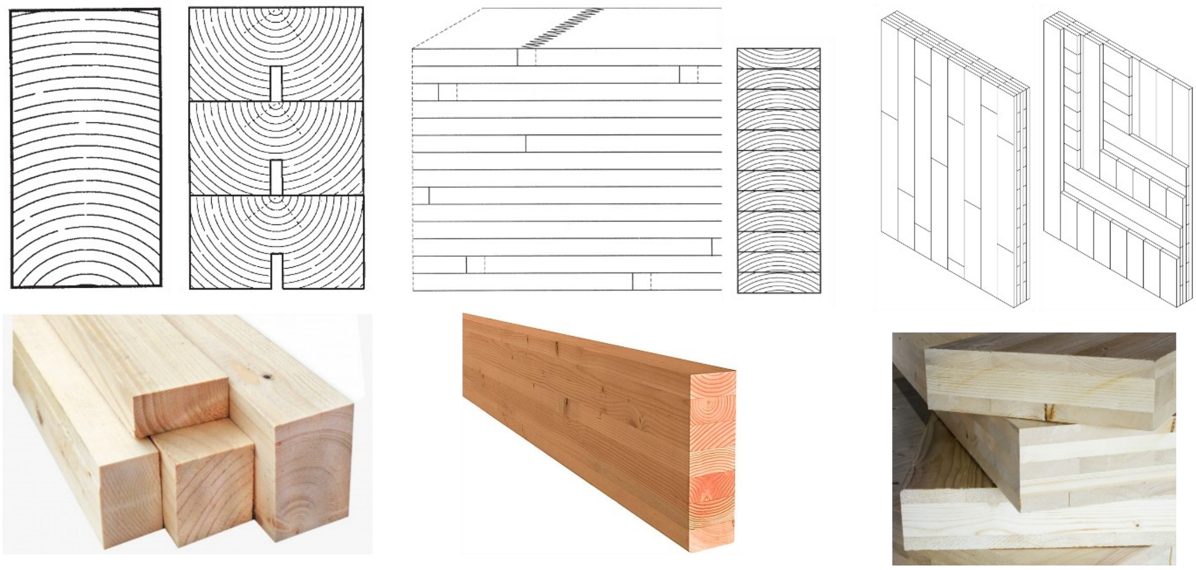


Figure 36. Left: solid timber products (Herzog et al., 2004; Ilarch, 2017), middle: glulam (Adis, nd; Kaufmann et al., 2018), right: CLT (Föreningen Sveriges Skogsindustrier, 2019; Kaufmann et al., 2018).

4.1.3 Wood-based Products

Instead of solid wood products, wood-based panels are produced by disassembling and subsequently reassembling wood, using adhesives. For different products, different starting materials are used like sawn timber, veneers, particles, wood shavings or fibres. The resulting panels are characterised by their approximated isotropy in plane and low level of variance in properties (Blaß & Sandhaas, 2017).

4.1.3.1 Veneer-based Panels (Plywood, LVL & PSL)

Veneer-based panels consist of multiple layers of veneer bonded to each other using resin. Veneer layers are produced by rotary peeling logs, resulting in 0.5 to 6.0 mm thick strips. The defective portions of the veneer are than removed, so the resulting panels have homogeneous properties. Depending on the configuration in which different layers are placed on top of each other, different products are formed (Blaß & Sandhaas, 2017).

For plywood, the veneer layers are bonded to each other in alternative crosswise layers. By varying the used wood species or thickness of the veneer layer, panels with different properties can be produced. The resulting plywood panels have approximately isotropic properties and are used mainly as sheathing materials in horizontal or vertical diaphragms (floors, roofs or shear walls) (Blaß & Sandhaas, 2017).

In Laminated Veneer Lumber (LVL), either the fibre direction of all the veneers is parallel to the longitudinal direction of the LVL or predominantly parallel and slightly (up to 25%) perpendicular to the longitudinal direction, see Figure 37 left. As a result, LVL is applied mostly for more or less linear construction elements. It can be used as load-bearing sheathing or as a bar-shaped member (Blaß & Sandhaas, 2017; Herzog et al., 2004).

Parallel Strand Lumber (PSL) is a newer invention. It is produced by waxing and pressing strips of veneer on top of each other, resulting in unlimited length beams. There is no general technical approval yet for PSL, but it has high strength values, increased stiffness and dimensional tolerance, making it applicable for beams, structural members, purlins, columns and trusses (Blaß & Sandhaas, 2017).

4.1.3.2 Wood-based Panels made of Chips (LSL, OSB & Particleboard)

These type of panels are produced by pressing multiple layers of wood chips or strands together. Laminated Strand Lumber (LSL) is very similar to LVL and PSL, except for that in the production of LSL, small veneer trim strands are used instead of sheets or long strips of veneer. Depending on the width of the panels, a larger proportion of the trim strands are in a perpendicular board direction for wider panels. The resulting panels are of high strength (Blaß & Sandhaas, 2017).

Oriented Strand Board (OSB) is made using longitudinal strands, which are primarily placed parallel to the board surface, see Figure 37 middle. Especially in the outer layers, the strands are in parallel direction and in the layers in between, it can vary, resulting in different properties. As a result, OSB panels have a high bending strength in the longitudinal direction, but a significantly lower strength in perpendicular direction (Blaß & Sandhaas, 2017; Herzog et al., 2004).

In particleboards, relatively small wood chips are used, which are sprayed with adhesive on a level underlay and then pressed together. The particles are preferably oriented parallel to the panel plane, resulting in advantageous in-plane tensile and compressive strengths, but low tensile strengths for stresses perpendicular to the plane. The boards are often used as structural floor, roof decking or structural wall sheathing (Blaß & Sandhaas, 2017; Herzog et al., 2004).

4.1.3.3 Wood-based Panels, Mineral Bonded

Mineral bonded boards are used as sheathing material in timber frame constructions for dry lining and fire protection. Different types of these panels exist, see Figure 37 right for example, but they all use a certain type of wood particle combined with cement or gypsum as binder (Blaß & Sandhaas, 2017).

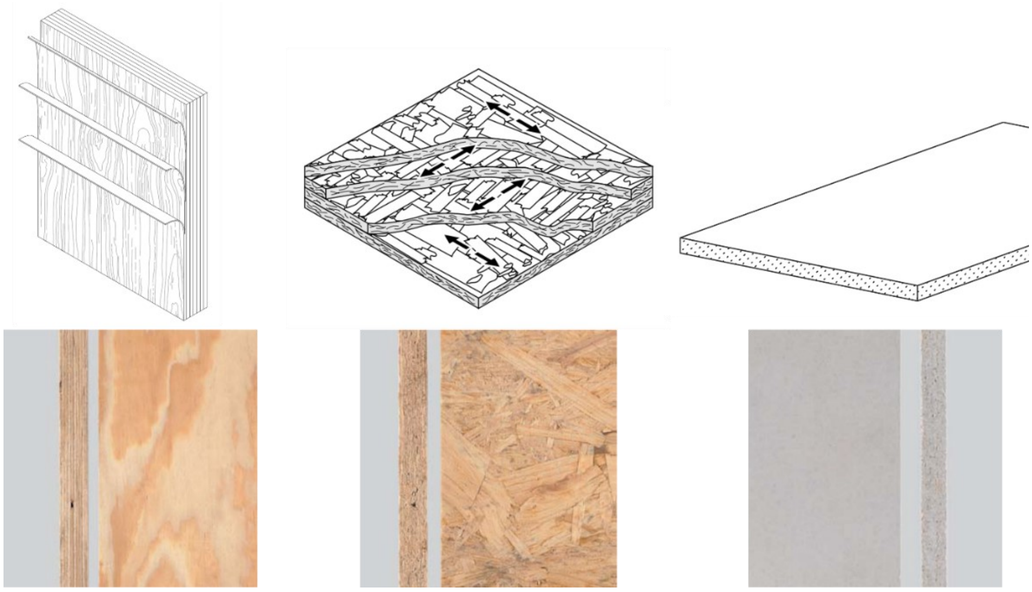


Figure 37. Left: LVL (Herzog et al., 2004), Middle: OSB (Herzog et al., 2004; Structural Board Association, 2004), right: mineral board (Herzog et al., 2004)

4.2 Structural Components

A timber structure is build-up of different structural components. In a multi-storey car park, the main structural components are beams, floors and columns. Each variant has its own characteristics, making it optimal for use in a certain span or under certain support conditions. Therefore, an overview of the relevant different types is given.

4.2.1 Beams

Timber beams are linear elements, supporting slabs and roof elements, transferring their loads to the supports. As a result of their function, beams are subjected to bending loads. Depending on parameters like the span of and load on the element, several beam types can be used (Kaufmann et al., 2018).

4.2.1.1 Solid Timber Beams

For limited spans up to around 6 metres, solid timber beams can be used (Soons et al., 2014). Their cross-section and length are limited by the logs that have been felled. Through the use of finger joints, longer beams can be made. Cross-sections can be enlarged by gluing two or three beams on top of each other (Kaufmann et al., 2018).

4.2.1.2 I-beams & Box Beams

I-beams and box beams are light weight, making them easy to handle on the construction site. The stronger flanges take up to tension and compression forces in the element. Box beams as a result of their geometry provide better resistance against lateral buckling compared to I-beams (Anctil, 2021; Kaufmann et al., 2018). Their maximum spans are limited to around 8 metres (Elliott Brothers Ltd., n.d.; Forest & Wood Products Australia Limited, 2008).

4.2.1.3 Glulam, LVL & PSL Beams

Glulam, LVL and PSL beams can be used for larger spans and loads. These wood engineered elements create the possibility for larger and more homogeneous cross-sections, which are stronger and much less likely to warp, twist, bow or shrink than solid wood beams (Anctil, 2021; Kaufmann et al., 2018). They can span up to 30 metres for a beam with a continuous cross-section and by varying the size of the cross-section like with saddle roofs, spans up to 40 metres can be realised (Soons et al., 2014).

4.2.1.4 Trusses

For larger spans, truss beams can be applied. The members of a truss are subjected mainly to normal forces and as a result, it is possible to subject the lower flange by a steel cable, resulting in an under stressed beam (Kaufmann et al., 2018). Trusses made of solid wood can span up to 25 metres. When engineered wood elements are used, they can span up to 80 metres (Soons et al., 2014).

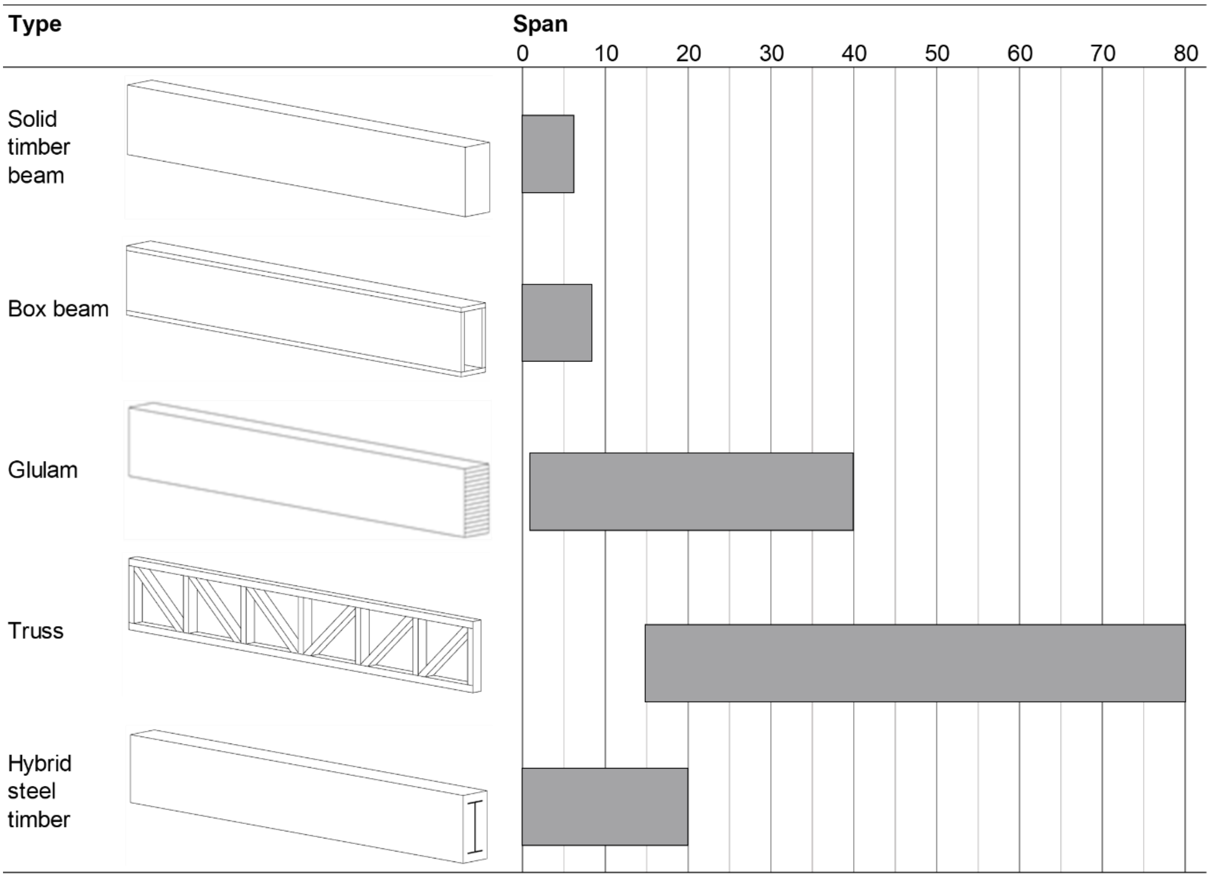
4.2.1.5 Hybrid Steel Timber Beams

A lot of research is conducted on the use of hybrid steel timber beams. A big advantage of such hybrid elements are their reduced depths as a result of increased load bearing capacity. Such elements still have a relatively low self-weight and as the steel elements can be encased in timber, they are protected against fire loads and stabilized for buckling (Tavoussi, Winter, Pixner, & Kist, 2010; Winter, Tavoussi, Riola-Parada, & Bradley, 2016). Timber beams can also be prestressed using different techniques. The prestressing results in an increased strength, giving the beams a higher load bearing capacity and reducing deformation. Test results show an improvement of up to 885% in SLS and 286% in ULS (de Lima, Costa, & Rodrigues, 2018). Hybrid steel timber beams therefore show significant potential for use in long span elements for which reduced depth is important.

4.2.1.6 Overview

An overview of the approximate possible spans for the different types of timber beams can be found in Table 9

Table 9. Spans for different types of timber beams (Kaufmann et al., 2018)



4.2.2 Floors

Timber floors are panel shaped elements, which directly support the imposed live loads by users on them. Floor elements can carry loads in one or two directions, depending on the type of floor system used. The elements are generally supported by beams to which they transfer their loads. Depending on parameters like the span of and load on the element, different types of floor systems can be used.

4.2.2.1 Dowel Laminated Timber Floor

A dowel laminated timber floor consists out of a number of solid wood planks joined together with the wood fibres running parallel with its span direction, resulting in a load-bearing solid wood structural component. It is a one-way load-bearing component in which each board carries the proportion of load that is placed on it between its supports. For this reason, it requires linear supports. To provide some cohesion between the planks and spread loads, adjacent boards are joined to each other by nailing, gluing or use of dowels. When dowels are used, the resulting component is not rigid enough to function as a plate, bracing the building. As a solution, the dowel laminated component can be nailed or screwed to a suitable wood-based material panel like OSB. The maximum thickness of this component is limited to around 240 mm as a result of the maximum board width, also limiting its maximum span distance. (Kaufmann et al., 2018)

4.2.2.2 Beam Floor

A beam floor is build-up of a series of beams covering the primary span on which panels or boards span the distance between the beams. They are often a very cost-effective solution for spans of 4 to 5 meters. For both the beams and panels or boards, different types of products can be used. As a result, many different spacings of the beams and spans can be achieved. OSB panels for example can span a smaller distance than CLT or LVL panels of different thicknesses. The materials used, their dimensions and the loads the floor has to carry make for a puzzle for which an optimal configuration has to be determined. The beams and plates can be glued or screwed to each other, but for long spans and large load areas, gluing is required. A rigid plate component can be formed by screwing suitable wood-based material sheeting to the beam floor component. (Kaufmann et al., 2018)

4.2.2.3 Box Floor

Box floors are somewhat the hollow core slabs of timber. These prefabricated lightweight floor slabs consist out of ribs combined with top and bottom planking and edge beams to result in a very high-performance planar support structure spanning in one direction. The planking functions both as load-bearing in composite action while providing dimensional stability at the same time. The planks can be connected to the ribs using glue, screws or carpenters joints. The ribs are braced by the planks and can therefore be very slender. Often a relatively small spacing of 40-70 cm is used. The use of box floors minimizes the thickness of a floor and are used for medium and long spans between 5 and 20 meters, although longer spans are possible. They can be supported by either linear supports or supports at various points and also cantilevering is possible. Downside of these elements is that they are complex and costly to produce. (Kaufmann et al., 2018)

4.2.2.4 CLT Floor

CLT panels can be used as floor elements. Because of their homogenous properties, they can carry load in two directions, can be supported by both linear supports and supports at various points and provide possibilities for cantilevers. CLT floors are limited by transport dimensions and generally, panels of 4 metres wide and up to 22 meters long are used with varying thickness (60 – 400 mm) depending on the span and load. CLT elements are also very rigid plates and therefore suitable to brace buildings. Multiple elements can be joined to each other by butt joints, overlapping, top joints or tongue-and-groove joints. (Kaufmann et al., 2018)

4.2.2.5 LVL Floor

LVL floor elements are homogeneous, although as a result of the direction of the layers fibres, there is a clear main span direction. It is possible to produce LVL with some cross layers, in which case the elements can also accommodate some loads in the other direction. In general, these panels require linear supports, but if cross layers are used, it is also possible to use supports at various points. Depending on the thickness, loads and support conditions, spans of up 4 metres can be achieved with a single LVL panel (Hakkarainen et al., 2019). It is also possible to stack multiple LVL panels on top of each other, resulting in possible spans of up to 19 m (Stora Enso, 2022d). As LVL elements are rigid plates, they are very effective at bracing buildings. (Kaufmann et al., 2018)

4.2.2.6 Composite Timber-concrete Slab

Composite timber-concrete slabs are often used for medium to long spans. Advantages of these elements are their improved structural performance, sound proofing, fire safety characteristics and reducing effect on vibrations as a result of their higher self-weight. The elements are made of a layer of concrete (6 – 12 cm) in compression and a layer of timber in tension which are connected to each other by various means to realise composite action. The type of timber material used can vary from a beam floor to CLT. These slabs are used in single spans up to 10 metres. (Kaufmann et al., 2018)

4.2.2.7 Overview

An overview of the different characteristics of the different types of timber floors can be found in Table 10.

Table 10. Characteristics of different types of timber floors

Type	One-/two-way load bearing	Support type	Plate action	Span (0 – 10 – 20)
Dowel laminated timber floor			No (suitable wood-based material sheating required)	
Beam floor			No (suitable wood-based material sheating required)	
Box floor		or	Yes	
CLT floor		or	Yes	
LVL floor	mainly	mainly	Yes	
Composite timber-concrete slab		or	Yes	

4.2.3 Columns

Columns are linear members, mostly in compression, transferring load in vertical direction. Timber columns can be made of various materials like sawn timber, glulam, CLT and LVL. Three different types of columns in timber can be distinguished: solid wood column, spaced column and box column, see Figure 38 (Fridley, 1999). Advantage of the latter types is their reduced weight, while placing their material around the periphery of the cross-section, making it a more effective cross-section against buckling loads (Harries, Petrou, & Brooks, 2000).

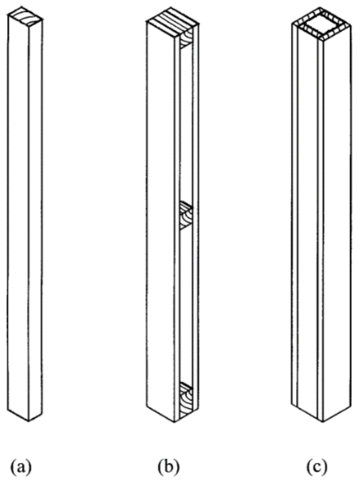


Figure 38. Types of timber columns: a) solid wood, b) spaced column, c) box column (Fridley, 1999)

4.3 Connection Types

Connections in a timber structure are, although they make up only a small part of the structure, one of the most important parts of it. The primary function of a connection is to transfer loads from one element to another. Many other aspects however also play an important role in the choice for a certain method and type of connection. The type of connection used influences the appearance of a structure, how elements are fabricated, connected and erected and with which ease. Furthermore, it determines how cost-efficient a design is and it can have an impact on fire-safety performance (Augustin, Bell, Kuklik, Lokaj, & Premrov, 2008; Herzog et al., 2004). Joints in timber constructions are generally weaker compared to the members which are being connected to each other and often have a semi-rigid character. They can be divided into three different categories: glued joints, carpentry joints and mechanical joints (Blaß & Sandhaas, 2017).

4.3.1 Glued Joints

Like engineered wood products are produced using adhesives, it is also possible to make glued connections between different construction elements. Finger joints are often used in glulam frame construction to form a rigid connection, see Figure 39 left (Herzog et al., 2004). Another glued type of connection used in mostly glulam or LVL constructions are glued-in rods. The rods are placed in or around connections in curved and notched beams to prevent cracks due to tensile stresses perpendicular to the grain or as a way to transmit forces in a construction or part of it, see Figure 39 right. These glued-in rod joints can transmit significant concentrated loads, result in a very rigid joint when exposed to axial load and have effective fire resistance properties (Blaß & Sandhaas, 2017). Glued joints result in rigid connections that are not suitable to dismantle. They are also labour-intensive and as a result a costly option (Herzog et al., 2004).

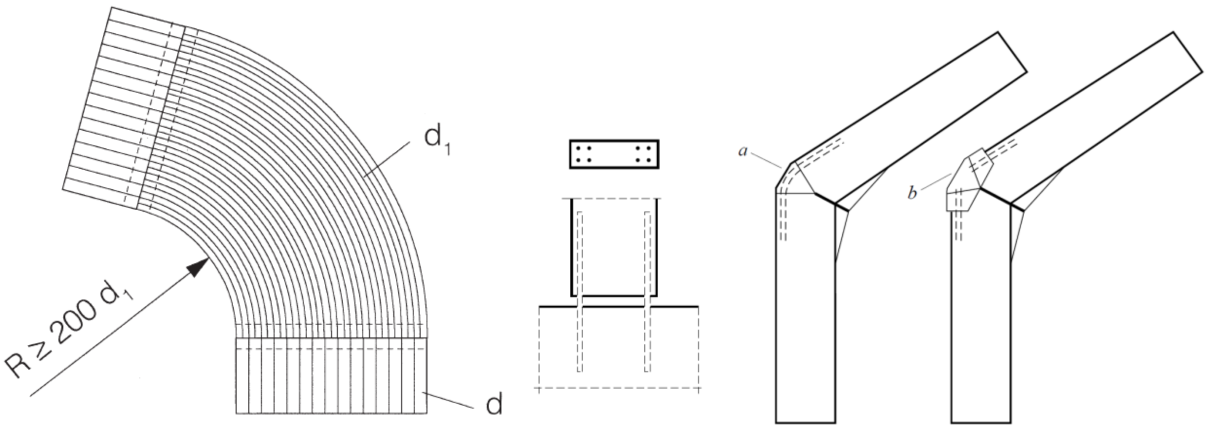
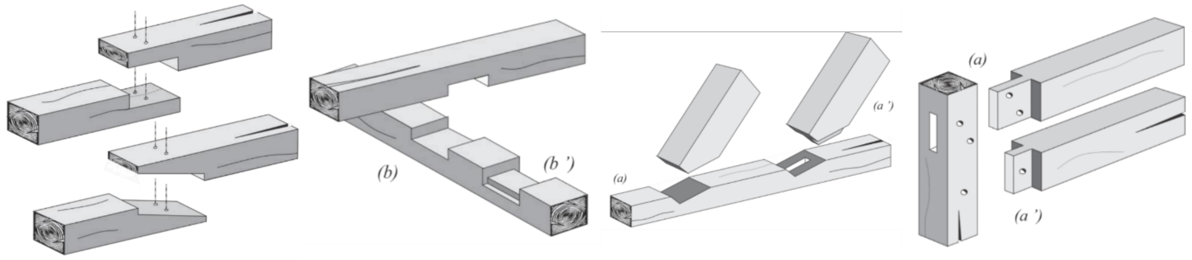


Figure 39. Left: Finger-jointed frame corner (Herzog et al., 2004). Right: Glued-in rods in a clamped column end joint and in a moment-resisting joint (Blaß & Sandhaas, 2017).

4.3.2 Carpentry Joints

Carpentry joints, also called contact joints, are joints in which forces are transmitted solely by contact and, where applicable, friction. They are semi-rigid, but often have significant moment-resisting capacity (Harte & Dietsch, 2015). Sometimes, wooden or metal fasteners are used to secure the members into position, but these may also contribute in the transmission of force. These joints are used to extend timber elements in a direction parallel to the grain or as transverse joints (Blaß & Sandhaas, 2017). Disadvantage of these type of joints is the weakening of the members. Due to severe shearing and eccentricity effects, only relatively low loads can be carried (Herzog et al., 2004). Advantages of these joints is that no glue or steel is required. The connections are easy to assemble and can also easily be dismantled. Four types of carpentry joints are distinguished (see Figure 40): half-lap joints, step joints, mortise and tenon joints and cogged joints. A detailed description of every type of carpentry joint can be found in appendix C.



1) Scarf joint 2) Half-lap joint 3) Step joint 4) Through pinned mortise and tenon joint

Figure 40. Different types of carpentry joints (Harte & Dietsch, 2015).

4.3.3 Mechanical Joints

Mechanical joints are engineered connections that in general consist of steel or metal elements, joining two members together. They can be clustered into two groups: dowel-type fasteners and surface-type fasteners. A characteristic of the used mechanical fasteners and dowel-type fasteners in particular, is they are ductile and the mechanical joints undergo deformations. Therefore, mechanical joints are always semi-rigid joints, although correct configurations of the fasteners can improve the rigidity of the joint. Advantages of these semi-rigid joints is their ability to deform before failure, as plastic deformation or creep may result in load redistribution, relieving a highly stressed area. To prevent a brittle failure of the joint, the timber might require strengthening by a metal plate. (Blaß & Sandhaas, 2017)

Mechanical joints require special attention. The metal elements in mechanical joints can be prone to fire and corrosion and should be protected. To prevent cracks from swelling and shrinkage, fasteners should be clustered on one side of the connected member and if this is not possible, reinforcements should be applied (Blaß & Sandhaas, 2017). Eccentricities which may result from the type of fasteners used, layout of the joint or layout of the structural system, should be prevented as much as possible as this results in bending moments in the members (Herzog et al., 2004). Cracking and splitting of the wood has to be prevented by proper spacing between the fasteners and distance to the edges of the member. If required, predrilling for the fasteners should be applied. Last of all, to prevent overloading due to excessive differences in stiffness, gluing and mechanical fasteners must not be combined (Blaß & Sandhaas, 2017; Herzog et al., 2004).

4.3.3.1 Dowel-type Fasteners

The load transfer using dowel-type fasteners involves both bending and tensile stresses in the fasteners and embedment and shear stresses in the timber along the shank of the dowel (Augustin et al., 2008). Three groups of dowel-type fasteners can be identified: nails and staples, bolts, dowels and threaded rods and screws, see Figure 41. These type of fasteners are often used both in joints directly between two timber members and between two timber members with additional steel plates, see Figure 41 right. Finally, they can also be used to create a connection with cold-formed steel connectors. A detailed overview of the different dowel-type fasteners can be found in appendix C.

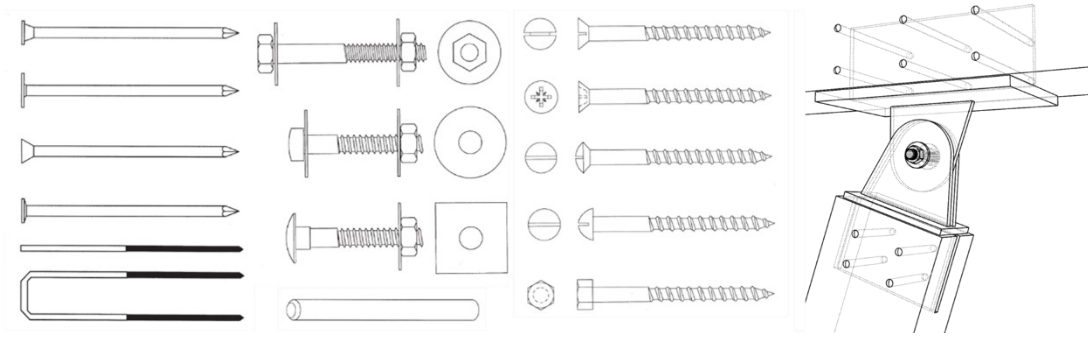


Figure 41. Left: Nails & staples. Middle-left: Bolts & dowel. Middle-right: Screws. (Herzog et al., 2004) Right: Dowels and steel plates including dowel hinged connection.

4.3.3.2 Surface-type Fasteners

With surface-type fasteners, forces are spread over the surface area of the timber. Transfer of loads is achieved primarily by a large bearing area at the surface of the members (Augustin et al., 2008). Two categories of surface-type fasteners can be distinguished: connectors and punched metal plate fasteners, see Figure 42. Within the group of connectors, multiple options exist, but all make use of a fastener like a bolt in combination with a connector, which takes up the task of transferring the loads. With punched metal plates, the plate has both the function of transferring loads and fastening the connecting element. A detailed description of the various connectors and punched metal plates can be found in appendix C.

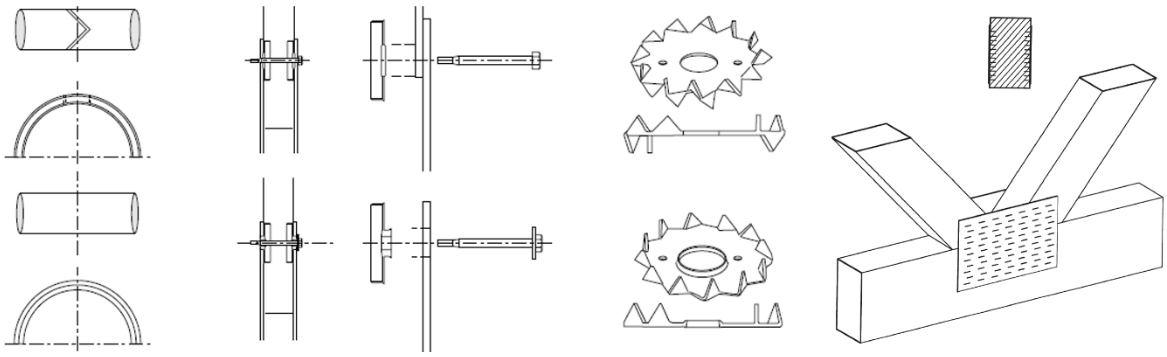


Figure 42. Left: double-sided split ring connector. Middle-left: single-sided shear plate connector. Middle-right: double- and single-sided toothed-plate connector. Right: punched metal plate joint. (Blaß & Sandhaas, 2017; Herzog et al., 2004).

4.3.4 Comparison

All of the different types of connections have their advantages and disadvantages. In Table 11, a comparison between the different connections is made on seven characteristics. A further detailed explanation of the comparison can be found in appendix C. For a temporary car park, the connections have to be reversible. As a result, glued joints cannot be used. Also nails and staples are more difficult to disassemble. Carpentry joints are a good option for joints, which do not require a very stiff connection and have rather small loads. In case higher load bearing capacities or stiffer joints are required, mechanical joints using either dowel-type fasteners or connectors are a better option.

Table 11. Comparison of different types of connections.

Connection type			Stiffness	Loadbearing capacity	Types of forces in plane	Ease of assembly	Dismantability for reuse	Fire resistance	Costs
Glued joints			High	High	M, N & V	Difficult	Not possible	Good	High
Carpentry joints		**Scarf joint**	Low	Low	V	Easy	Easy	Good	Medium
		Lap joint	Low	Low	N & V	Easy	Easy	Good	Medium
		Step joint	Low	High	N	Easy	Easy	Good	Medium
		Mortise and tenon joint	Medium	Low	M, N & V	Easy	Easy	Good	Medium
Mechanical joints	**Dowel-type fasteners**	**Nails & staples**	Low	Medium	N & V	Easy	Medium	Medium	Low
		Bolts, dowels & threaded rods	Medium to high	High	M, N & V	Easy	Easy	Medium	Low
		Screws	Low	Low	N & V	Easy	Easy	Medium	Low
		Combination with steel plates or connectors	High	High	M, N & V	Easy	Easy	Low to medium	Medium
	Surface-type fasteners	**Connectors**	Medium to high	High	M, N & V	Easy	Easy	Medium	Low
		Punched metal plate fasteners	Medium	Medium	M, N & V	Easy	Easy	Low	Low

4.4 Durability

Timber is a natural building material, which can be part of the transition towards a sustainable building industry. However, to be able to fulfil a role in a sustainable and circular economy, the material should also be durable. As timber is a natural material, it can deteriorate very quickly if not protected from the elements (Anastasiades et al., 2020). Three challenges concerning the durability of timber can be distinguished: biological degradation (fungal attack), environmental degradation (weathering) and safety concern (fire hazard), see Figure 43 (Ayanleye, Udele, Nasir, Zhang, & Militz, 2022). Sufficient protective measure should be taken to prevent the degradation of the timber, while also environmental factors are of concern.

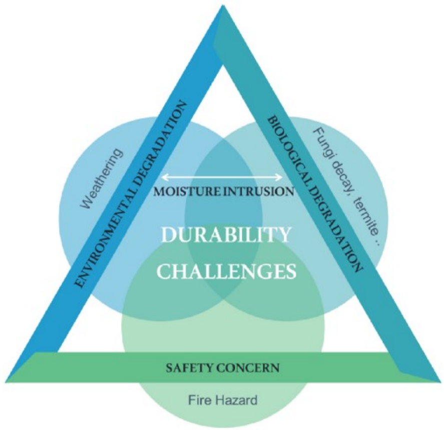


Figure 43. Durability challenges for timber (Ayanleye et al., 2022)

4.4.1 Risks

4.4.1.1 Biological Degradation

Fungal attack

Micro-organisms like bacteria and fungi break down large wood cellulose molecules into smaller and simpler molecules, effectively breaking down the timber to be redirected into the materials cycle, see Figure 44 left. This natural process is part of the life cycle of wood, but should be prevented from occurring in a construction (Blaß & Sandhaas, 2017). For the microbial breakdown to occur, four basic requirements for growth should be fulfilled: wood as a food source, oxygen, water and appropriate temperatures (Kuklik et al., 2008). The optimal range of moisture for most types of fungi is between 30 and 60% and the temperature can vary between -2.5 and +40ºC with an optimum between 19 and 31ºC. It is not possible for wood-destroying fungi to grow when the wood moisture drops below 20% and they die at temperatures exceeding 60ºC (Blaß & Sandhaas, 2017; Kuklik et al., 2008). In general, these conditions are all satisfied in forests, but usually not in buildings. Therefore, in buildings, usually protective measures are taken if required to keep the moisture content of the timber below 20%.

Insect attack

Dry-wood insects can infest semi-dry and dry timber and construction wood. Damage to the timber is mainly the result of the burrowing of larvae by the insects (see Figure 44 right), which can result in a dramatic reduction of the cross-section and load-bearing capacity of the wood. It is possible for larvae to develop with moisture contents of just 8-12%, meaning just keeping the timber dry by constructional measures is not enough. Therefore, it is important to prevent larvae from infesting the timber in the harvesting process by sufficient measures. Furthermore, growth of insects is encouraged by warm conditions, stimulating their development and reproduction (Blaß & Sandhaas, 2017).

Figure 44. Left: fungi rot (a. white partial, b. white full, c. brown), right: common furniture beetle (a & b. adult beetle, c. larvae) (Kuklik et al., 2008)

4.4.1.2 Environmental Degradation

Environmental degradation of timber is the result of weathering effects. Timber ages naturally by exterior factors like UV radiation, air, temperature and moisture changes, together with stress conditions. This results in irreversible changes in the appearance and properties of the timber, see Figure 45 (Kuklik et al., 2008).

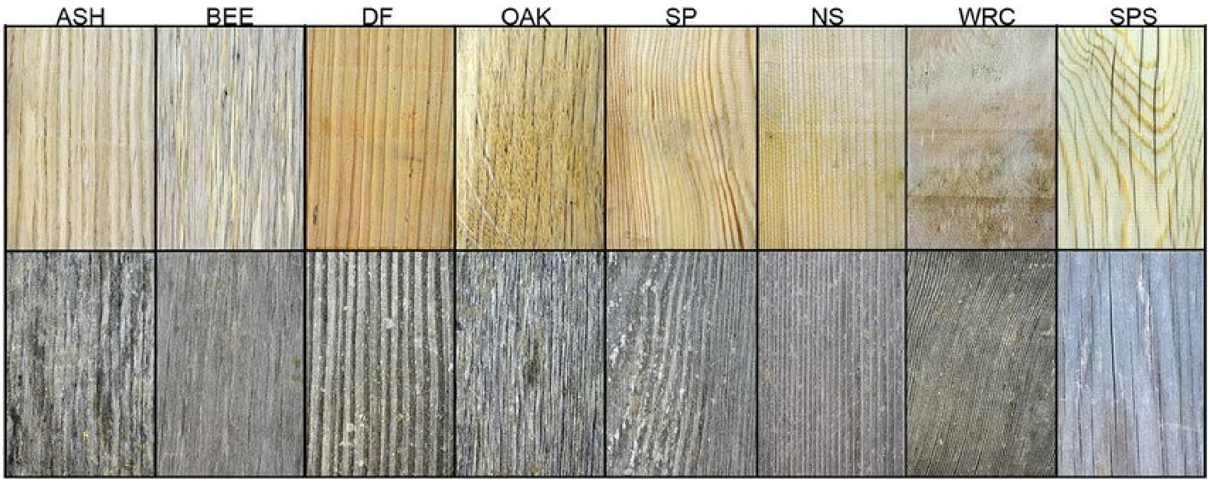


Figure 45. Weathering effects on timber specimens (Niklewski, Brischke, Frühwald Hansson, & Meyer-Veltrup, 2018)

Weathering effect scan also result in shrinkage and swelling of timber elements, which can lead to cracks in the timber. Insects can lay eggs in such cracks, resulting in larvae and following degradation by insects attacks. Furthermore, weathering effects can also affect connections between timber elements made using metal fasteners. These fasteners can corrode, which can ultimately result in failure of the connection (Blaß & Sandhaas, 2017).

4.4.1.3 Safety Concerns

Unlike steel and concrete, wood is a combustible material, meaning it can contribute to the fire load of a building. As a result, timber buildings are often considered by the public to be a bigger risk in terms of fire safety. However, the risk of starting a fire in a building is not related to the material it is made of. It is related to aspects like improper electrical installations or human error (Kaufmann et al., 2018).

Wood as a building material is generally seen as a material with an acceptable ignition risk. For solid timber to ignite, surface temperatures exceeding 400 ºC or contact with an ignition source of more than 300 ºC is required. When wood is ignited, the speed of flame spreading is acceptably low for a combustible material. The characteristics of wood-based materials exposed to a fully developed fire are seen as favourable. The surface of the material, once ignited, will burn vigorously resulting in the formation of a charcoal layer. This layer, even more than wood itself, is a very poor thermal conductor. The result is that the charcoal layer forms an insulating layer around the unburnt wood, protecting it from the heat of the fire. Therefore, wood naturally actually protects itself from fire and thus has favourable characteristics under fire conditions (Blaß & Sandhaas, 2017; Kuklik et al., 2008)

4.4.2 Protective Measures

4.4.2.1 Natural Durability

Wood constitutes of its main molecules cellulose, hemicellulose and lignin, but also incorporates other constituents, which contribute to a natural durability of the wood (Kuklik et al., 2008). These constituents can make the wood more resistant against fungi and insect attacks. They vary significantly between different wood species. Therefore, to describe the natural durability of wood species, durability classes are assigned to the different species. For the natural durability against fungi attack, a number is given to the wood species and for natural durability against insects and wood pests in sea water, a letter is given, see Table 12. Generally speaking, tropical woods are more durable species and also European hardwoods have good durability characteristics. Sapwood is considered to be non-durable against fungi attacks (Blaß & Sandhaas, 2017).

Table 12. Natural durability classes

Against fungi		**Against insects and wood pests in sea water**	
1	Very durable	D	Durable
2	Durable	M	Moderately durable
3	Moderately durable	S	Vulnerable
4	Slightly durable	SH	Where heartwood is also classed as vulnerable
5	Non-durable		

4.4.2.2 Service Classes

To determine how vulnerable elements of a timber structure are to durability risks, service classes are used, which describe the environmental conditions to which a timber element is exposed, see Table 13. Based on this service class together with the durability classes, one can determine whether the wood used has sufficient natural durability or if extra protective measures should be taken (Blaß & Sandhaas, 2017; Herzog et al., 2004).

Table 13. Service classes for timber elements (Blaß & Sandhaas, 2017)

Service Class		**Moisture content / exposure**	**General condition of use**	**Threat from**				**Leaching**
				Insects	**Fungi**	**Soft rot**	**Wood pests in the sea water**	
0		Dry (permanently ≤ 20%), average relative humidity up to 85%	Wood or wood product under roof, not exposed to weathering or moisture, the risk of structural damage caused by insects can be excluded	No	No	No	No	No
1		Dry (permanently ≤ 20%), average relative humidity up to 85%	Wood or wood product under roof, not exposed to weathering or moisture	Yes	No	No	No	No
2		Occasionally humid (> 20%), average relative humidity over 85% or occasional humidification due to condensation	Wood or wood product under roof, not exposed to weathering, high relative humidity can lead to occasional but not permanent humidification	Yes	Yes	No	No	No
3	**3.1**	Occasionally humid (> 20%), accumulation of water in wood, even on a limited basis, not expected	Wood or wood product not under roof, exposed to weathering, but without permanent contact with soil or water, accumulation of water in wood, even on a limited basis, is not expected due to swift drying	Yes	Yes	No	No	Yes
	3.2	Frequently humid (> 20%), accumulation of water in wood, even on a limited basis, expected	Wood or wood product not under roof, exposed to weathering, but without permanent contact with soil or water, accumulation of water in wood, even on a limited basis, expected	Yes	Yes	No	No	Yes
4		Mainly to permanently humid (> 20%)	Wood or wood product in contact with earth or freshwater and hence mainly to permanently exposed to moisture leading to moderate to severe stress	Yes	Yes	Yes	No	Yes
5		Permanently humid (> 20%)	Wood or wood product permanently exposed to sea water	Yes	Yes	Yes	Yes	Yes

4.4.2.3 Constructional Measures

Constructional measures are the most important protective measures to prevent degradation of the wood of a building. By clever design, problematic changes in the moisture content of the timber components can be prevented. Such changes in moisture content can result in conditions suitable for fungi to destroy the timber of can result in swelling and shrinkage, leading to damaging deformations (Herzog et al., 2004).

Furthermore, central heating systems inside buildings result in optimal conditions for insects to grow. Good design (make structural barriers against termites or covering all sides of timber elements for example) can prevent insects from gaining access to concealed components, preventing attacks (Herzog et al., 2004; Kaufmann et al., 2018). Natural durability of the wood and using heartwood combined with regular inspection tends to further improve the protection against insects attacks. Again limiting changes in moisture content prevents crack formation, which can be a location for egg-laying (Blaß & Sandhaas, 2017).

The constructional measures taken result in a certain service class classification. Aim is to have a service class as low as possible (Herzog et al., 2004). The following protective constructional measures should be taken into account (Blaß & Sandhaas, 2017):

- Prevent humidification of wood as far as possible
- Provide swift water drainage and ventilation, if occasional humidification cannot be prevented
- Prevent direct water influx
- Protect end grain surfaces from water penetration
- Take condensation problems into consideration
- Prevent wood from exposure to moisture from contact with ground or areas, where snow can accumulate or water can splash
- Use woods from a sufficient natural durability class, if permanent humidification cannot be prevented
- Correct storage of timber elements during construction

4.4.2.4 Wood Preservatives

Like some wood species naturally have better durability, it is also possible to improve the durability of wood using chemical wood preservatives containing pesticides (Herzog et al., 2004). Timber elements that are completely exposed to continuous changing high moisture contents cannot be protected by chemicals. In such case, the chemicals can only delay the degradation at best. However, chemicals can be used to improve the characteristics of the wood for situations where occasional humidification occurs (Kaufmann et al., 2018). Furthermore, paints and coatings can for example be a means of weathering protection, mitigating precipitation, high humidity, UV rays and mechanical stresses (Blaß & Sandhaas, 2017). Several methods like brushing, spraying, dip-treatment and pressure treatment are used, which should result in a uniform distribution and adequate retention of the preservatives by the timber. Although preservatives in the past have proven to be successful, in recent years, newer preservatives have been developed, which are more environmentally friendly. Also concerns about the health of residents and users of the buildings resulted in development of new preservatives (Ayanleye et al., 2022).

4.4.2.5 Wood Modification

The aim of wood modification is to reduce the wood's affinity for moisture and block the cell walls by penetrating the nanopores in the cell wall by chemical and/or thermal modification. As a result, the wood can absorb less water and thus becomes less vulnerable to biodegrading, which requires moisture (Ayanleye et al., 2022; Blaß & Sandhaas, 2017). Different thermal treatments have been developed like the exposure of wood to high temperatures (160-230ºC) under reduced oxygen conditions or using different pressure regimes (vacuum, non-pressurized, pressurized) or shield gasses. As a result of these thermal treatments, the physical and chemical structure of the timber is altered. Such treatments have proven to increase the wood's resistance against weathering influences. The downside however is the reduction of certain strength properties of the timber. Thermally modified wood is also more vulnerable to insect attacks. Chemical treatment has proven to be able to improve the wood's resistance against both biological and environmental degradation, without reducing its strength properties. Downside of chemical treatment is similar with using wood preservatives. Their effects on the environment and health of residents is an important concern (Ayanleye et al., 2022).

4.4.2.6 Corrosion

Wood is normally not attacked by acids and bases. If metallic fasteners are used, these however are vulnerable to corrosion. Different protective measures can be taken like painting or coating or the use of special stainless steel fasteners (Blaß & Sandhaas, 2017).

4.4.2.7 Fire Safety

A timber building should meet the same fire safety regulations as buildings in steel and concrete. This means its structural elements should remain stable (R criterion) and the structural elements enclosing a space should prevent the passage of smoke and gasses (E criterion) and heat (I criterion) (Kaufmann et al., 2018). As wood and especially the charcoal layer in case of fire are poor heat conductors, this favours criterion I. Considering the R and E criterion, timber structural elements should not lose their load-bearing capacity or undergo too large deformations under fire conditions (Kuklik et al., 2008).

To verify the load-bearing capacity of structural elements under fire conditions, two methods can be used: reduced cross-section method (RCSM) or reduced properties method (RPM) (Blaß & Sandhaas, 2017). The RCSM method is a simplified method, in which a remaining unburnt cross-section is considered to depict the fire resistance duration of a structural member. Depending on the applied protective measures and required fire resistance duration, an effective charring depth is calculated. The cross-section of the structural member is reduced with this depth and the remaining effective cross-section should be able to carry the load on the element under fire conditions, see Figure 46. The RPM method is a more accurate method, which works similar like the RCSM method, however this time, the mechanical properties of the remaining cross-section are reduced.

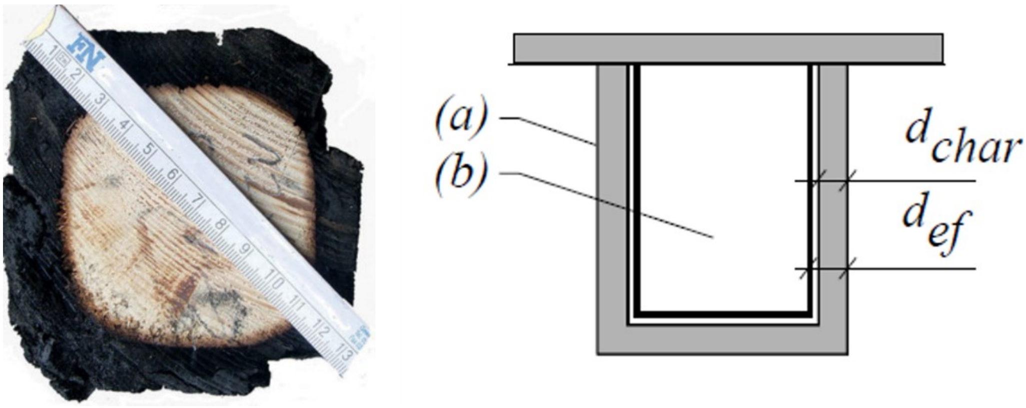


Figure 46. Left: burned wooden element with charcoal layer (Kaufmann et al., 2018), right: reduced cross-section (Blaß & Sandhaas, 2017)

It is also possible to further improve the resistance of timber elements to fire. A coating can be applied to the wood or it can be impregnated with a fire retardant. This does not change the wood into a non-combustible material, however it can make it more difficult to ignite. A downside of such chemicals are often its bad environmental properties (Blaß & Sandhaas, 2017).

Joints with metal fasteners are of concern when exposed to fire. The metal has a much higher thermal conductivity than wood, meaning it can effectively transmit heat to the inside of the members, causing both embedment strength and withdrawal capacity to decline. Furthermore, the yield strength of the metal declines very rapidly with increasing temperatures. Metal fasteners in joints therefore should be protected. Unprotected joints, but with side members made of timber, have a fire resistance duration of 15 minutes. This time can be increased by increasing the member thickness and the end and edge distances to a maximum of 30 minutes. Using the reduced stresses method, it is possible to realise fire resistance durations of up to 40 minutes for unprotected joints and in combination with other methods, this can be increased to 60 minutes. Steel plates with unprotected edges can withstand fire up to 60 minutes, depending on their thickness. It is also possible to protect joints by cladding them with members made of wood (boards or plugs) (Blaß & Sandhaas, 2017).

Like joints, structural elements can also be protected by cladding them with wood or wood-based materials offering resistance to heat, which is also applied with steel structures. Similarly, other general fire safety measures like sprinkler systems, smoke detection and ventilation mechanisms can be applied in timber buildings. These measures are more general fire safety measures and are not further discussed here.

4.5 Pavement Surfacing

Using timber in buildings often has the advantage that finishing of the interior is no longer required, as timber itself already has the required aesthetics. However, in a car park, timber decks are not directly suitable to be driven on and require an appropriate surfacing system. As timber car parks are very new, only a very limited number of car parks are built in timber and research on timber decks in car parks is very limited as well. However, timber bridges have been built for many years and research on timber decks for bridges is much more extensive. Differences can be found between timber bridges and parking decks, but it can be a good starting point as a reference for the timber decks of a car park.

4.5.1 Function

In case timber material is used for the decks of a car park, a suitable surfacing system should be applied to the timber, fulfilling the following functions: 1) provide a smooth and safe surface for vehicle traffic and furthermore improve the poor skid resistance of timber decks (Ritter, 1990; Schiere, Müller, Bueche, & Angst, 2022), 2) protect the deck from vehicle loads (Haynes, Coleri, & Estaji, 2019; Ritter, 1990), and 3) protect the deck from the elements (rain, sun, snow etc.), to prevent biological degradation (Haynes et al., 2019; Ritter, 1990; Schiere et al., 2022).

Timber decks that are made of planks are prone to varying deflections when vehicles drive over them. In such case, the surfacing system should help to minimize such varying deflections and provide a smooth surface (Ritter, 1990). For timber panels such as CLT, these problems are limited to the locations of connection between the various panels. Furthermore, timber can be very slippery, especially when wet. To provide a safe surface, the surfacing system should increase the skid resistance of the decks (Ritter, 1990; Schiere et al., 2022).

The weight of vehicles is transferred to the construction only through the small patches of rubber from the wheels, resting on the parking decks. As a result, relatively large loads are placed on only a small area of the deck. Timber can be vulnerable to such large localized loads due to the anatomy of wood (Blaß & Sandhaas, 2017). The surfacing system can help to spread the loads over a larger area, protecting the timber. Furthermore, at the locations of joints between different panels, again timber is vulnerable for loads from vehicles passing over these joints.

As discussed in paragraph 4.4, building with timber comes with several durability challenges. Timber is very vulnerable for moisture and therefore requires protection from direct exposure to the elements (Haynes et al., 2019). The surfacing system provides this protection. It should seal off the timber surface, preventing water from entering the timber (Scharmacher, Müller, & Brunner, 2014).

4.5.2 Surfacing Systems With or Without Shear Connection

Many different types of pavement surfaces have been developed. In their layup however, one can choose for a layup in which the surfacing system is fully connected to the timber deck (full shear connection) or for a layup in which the surfacing system floats and is bolted to the timber deck (without shear connection) (Scharmacher et al., 2014). Difference between these types of layups is how shear forces are transferred from the surfacing layer to the timber deck. Most pavement surfaces can be used with both types of layups.

In general, starting on top of the timber deck, the surfacing system is build-up out of a surface coating (for systems with shear connection) or separation layer (for systems without shear connection), sealant, protective layer and top layer, see Figure 47. The surface coating acts as the “glue” between the bridge deck and sealant (Scharmacher et al., 2014). In case of a system without shear connection, a separation layer is used (glass-fibre mat or oil-paper), to which the sealant can attach (Schiere et al., 2022). The separation layer is connected to the deck through circular brackets (Haynes et al., 2019). The sealant layer seals of the timber element and provides a watertight barrier, protecting the timber. Furthermore, in case asphalt is used, the sealant layer partially protects the timber from the heat of the pouring process of the asphalt (Haynes et al., 2019). The protective layer acts as an intermediate layer, protecting the sealant from vehicle impacts and can help to spread vehicle loads. Finally, the top layer acts as the driveway with proper skid resistance (Scharmacher et al., 2014).

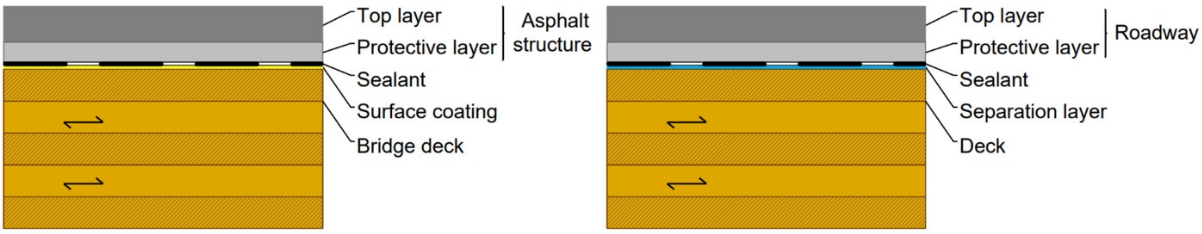


Figure 47. Left: Sketch of a system with a bonded shear connection between sealant and deck. Right: Sketch of a system without a shear connection between sealant and deck (Scharmacher et al., 2014).

4.5.2.1 With Shear Connection

Advantageous of systems with shear connection, see Figure 48, is that traffic loads are transmitted directly to the deck, including large breaking and acceleration loads. As a result, they can be loaded with heavier traffic and allow for larger slopes (> 6%) (Schiere et al., 2022). If asphalt is used as top layer, its fatigue life is improved with a shear connection system.

The shear capacity of the joint between deck and sealant layer depends on the combination of bonding agent and waterproofing membrane (sealant layer). Research has shown the shear capacity is not influenced by the type of deck and timber performs comparable to concrete and steel decks. The type of failure mechanism (brittle/ductile) is also solely dependent on the type of adhesive used and is not influenced by the material of the deck (Scharmacher et al., 2014; Schiere et al., 2022).

Figure 48. Test specimen of system with shear connection (Haynes et al., 2019)

Blistering

A disadvantage of the use of systems with shear connection is that blistering can occur in case asphalt is used as top layers. Blistering is the result of water vapour, rising from the timber and accumulating directly at the bottom face of the sealant layer, when hot asphalt is poured on the deck. As a result, the accumulated water vapour can cause partial separation of the sealant from the timber deck, see Figure 49, and in some cases it can penetrate the sealant layer, which causes “blisters” in the asphalt (Scharmacher et al., 2014). If the sealant is penetrated, the watertight barrier is no longer intact and water can come in contact with the timber. Furthermore, as a result of the partial separation, the shear capacity of the plane between the sealant and deck is reduced by approximately 10 – 15% (Scharmacher et al., 2014). Since timber contains a relatively large amount of water, timber decks are more prone to blistering compared to steel or concrete decks.

Figure 49. Test specimen of system with shear connection experiencing blistering (Scharmarcher, Müller, & Brunner, 2014)

To prevent or minimize the risk of blistering, several measures should be taken (Scharmacher et al., 2014; Schiere et al., 2022):

- The pouring temperature of asphalt should be below 200 °C;
- The thickness of the protective asphalt layer should not exceed 25 mm;
- The hot asphalt should be placed by hand instead of by a road finishing machine;
- When choosing the type of sealant layer, one should keep its effect on blistering in mind;
- The timber panels should have good surface quality.

4.5.2.2 Without Shear Connection

Advantageous of systems without a shear connection, see Figure 50, is the increased moisture resistance, making blistering no longer a risk. It also eliminates cracking of the pavement at joints between different (glulam) joints (Ritter, 1990). Since no full connection exists between the timber deck and sealant layer, the timber can breathe during the use phase (Haynes et al., 2019).

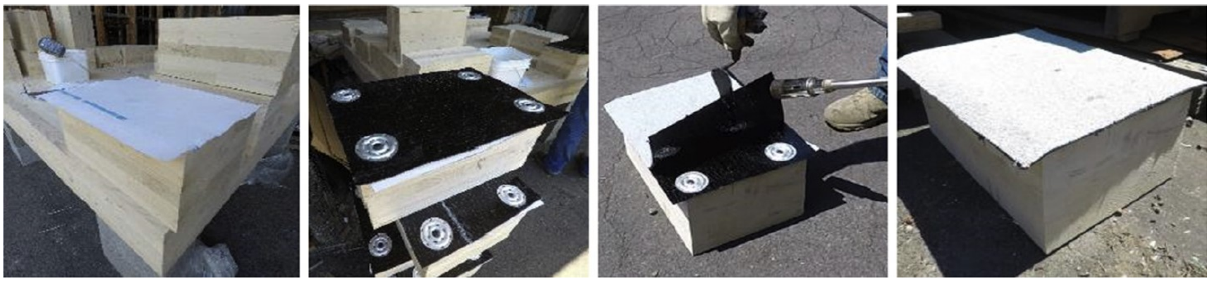


Figure 50. Test specimen of system without shear connection (Haynes et al., 2019)

Disadvantageous of systems without a shear connection is its lower shear capacity compared to a system with shear connection, although this is still more than sufficient (Haynes et al., 2019). Another disadvantage is the risk of development of surface waves as a result of breaking and acceleration (Scharmacher et al., 2014). Such a system is therefore not applicable in locations were frequent and/or heavy acceleration or breaking occurs. As a result, in Germany, all road bridges with timber decks are required to use a system with shear connection (Scharmacher et al., 2014).

4.5.3 Types of pavement surfaces

4.5.3.1 Asphalt

Asphalt is one of the most used pavement surfaces in the world. It is often used on (timber) bridge decks and current existing timber car parks also use asphalt as pavement surface (HK Architecten, 2021; Zaugg, 2018). Two types of asphalt pavement surfaces exist: mastic asphalt and rolled asphalt. Whereas mastic asphalt is virtually waterproof, rolled asphalt is fairly permeable and as a result not fully waterproof. Furthermore, rolled asphalt requires large compaction energy, which can damage timber decks (Haynes et al., 2019). Based on these two aspects, rolled asphalt is considered not applicable to timber parking decks.

Mastic asphalt has a high heat resistance, flexibility and fatigue strength. As a result of its waterproof properties, it protects the timber decks and its surface is skid resistant. As a result of the thickness of the asphalt layers, it spreads vehicle loads over a larger area, significantly reducing deformations (Haynes et al., 2019). Disadvantageous is its larger self-weight as a result of the thickness of the layer. Its placement requires high temperatures, which can result in blistering (Haynes et al., 2019). With good maintenance, it can service over 20 years (Schiere et al., 2022).

Figure 51. Timber deck with mastic asphalt pavement surface (Haynes et al., 2019)

The asphalt pavement is build-up of a surface coating or separation layer, followed by a bitumen membrane, a first asphalt protection layer and finally an asphalt top layer, see Figure 51.

4.5.3.2 Coating with Aggregate

Many different types of bridges (steel/concrete/timber) are fitted with a wearing layer, which is based on a coating layer into which aggregate is pressed, see Figure 52. First, a primer is applied to the timber deck for proper bonding with the membrane layer, which ensures watertightness. Next, a basecoat layer (epoxy or polyurethane often used) is poured. Aggregate is broadcasted over this layer to add friction to the driving surface. Finally, a topcoat is applied (Haynes et al., 2019).

This type of coating provides a watertight, flexible surface, which is highly resistant against fatigue cracking (Haynes et al., 2019). The system furthermore resists heat, chemicals and de-icing salts. It has a relatively small thickness, resulting in a low self-weight compared to asphalt (approximately 15x lighter) (HIM, 2021). Disadvantageous is the very limited ability to spread loads.

Figure 52. Polyurethane wearing course (HIM, 2021)

4.5.4 Modular and Temporary Structure

The objective of this thesis is concerned with a temporary and modular multi-storey car park. As a result, one of the large challenges for the design of the construction is that all elements should be easy to install and disassemble. Creating a monolithic parking deck by pouring for example an asphalt layer over all different floor panels hinders the disassembly process and is therefore undesirable. Starting point is that the pavement surface should be applied only to a single floor element. The different floor elements are placed against each other, resulting in a joint between them. This joint comes with new challenges for the detailing of the floor elements and the surfacing system.

4.5.4.1 Expansion Joints in Timber Bridge Decks as Reference

Schiere et al. (2022) studied details of timber bridges with asphalt wearing surfaces. Amongst the details were expansion joints, side edges along curbs and details around drains. Expansion joints are considered to be most similar to the joints between the different floor elements in a timber car park. They are highly loaded by traffic, causing impact loads on the edges of the joints. Translating the requirements of expansion joints in timber bridges to the floor joint in a timber car park, they have to fulfil requirements considering sufficient load bearing strength, fatigue resistance, deformation and elasticity, low noise emission and good premises for inspection and repair. Furthermore, the joints in the floors of car parks should be closed, to prevent water from dripping to lower decks.

Schiere et al. (2022) presented a set of recommendations for expansion joints of timber bridges. Relevant recommendations for the joints between the floor elements of a timber car park are as followed:

- The structure on both sides of the joint should have a similar vertical stiffness, to prevent large relative deformations between the two elements when a vehicle crosses the joint.
- The maximum vertical deformation should be limited.

- If timber decks do not have sufficient stiffness, they can be reinforced with steel beams.
- Proper sealing of the timber around the joints should be ensured.
- Ventilation in the vicinity of the joint should be encouraged to improve the drying capacity of the timber elements.

4.5.4.2 Effect of Joints on Type of Surface Pavement

It is clear that the joints between the different floor panels result in important requirements for the detailing of the timber deck panels around this area. However, this also affects the surface pavement which is placed on top of the timber panels. Similar to the timber, the pavement surface should also be able to withstand the impact loads of vehicles, have sufficient fatigue resistance, deformation and elastic capacity. An asphalt layer has a larger thickness and might be vulnerable to impact loads. A system with a coating with aggregate is thinner and might be less vulnerable, but may cause the timber panel to take more impact. Additional research is advised for proper detailing of the joints between floor elements and what the impact of different pavement surfaces is.

4.5.5 Choice of Pavement System

In a car park, vehicles are continuously braking, accelerating and turning to navigate the car park. Although the speeds are low, this results in continuous shear loading of the pavement surface and might lead to development of surface waves. Furthermore, ramps in a car park can exceed a slope of 6%. Considering these aspects, a pavement system with shear connection is chosen.

For the type of surface pavement, both systems of asphalt and a coating with aggregate can be applied, although more research is required considering the joints between the floor elements. Based on the literature research, both systems fulfil the requirements, but the system of a coating with aggregate is much lighter. Therefore, this system is chosen for the development of the temporary multi-storey car park of this thesis.

4.6 Timber Car Parks

Although timber has been used in the construction industry for ages, only recently the material has again become more popular for multi-storey construction and new height records are quickly following each other. The use of timber in car parks however has been quite limited. The world’s first significant timber car park has only been constructed very recently in 2018. Recently, several designs for multi-storey car parks in timber have been developed, but many of them are yet to be realised. It seems like the industry is still somewhat sceptical about the realisation of a multi-storey car park in timber. Following are a realised and proposed multi-storey car park project in timber, which function as reference projects.

4.6.1 Timber-built Car Park Studen, Switzerland

The world’s first timber-built car park has been realised in Studen, Switzerland. The car park provides 1800 parking stalls for a specialist in automotive transport and logistics, COTRA Autotransport. One of the main reasons to choose for timber as the main structural material were weight issues with the foundation. A solid concrete or hybrid steel-concrete structure proved to be too heavy for the site-conditions. A lightweight three-storey high construction was required and an all-timber construction proved to be feasible and even economical (Zaugg, 2018).

4.6.1.1 Structural Concept

The car park is a modular construction, which has no minimal or maximum size. It has a grid of 5,1 x 15 m and a level height of 2,15 m with bays of 5 x 2,55 m, see Figure 53. The parking deck is column free, allowing for problem-free parking and exit.

Floor system

The inter-storey floor slabs are made of five-ply cross-laminated timber, covered with two layers of mastic asphalt with a separation layer and sealing membrane. These form the carriageway and also act as a fire compartment-forming element. The panels have a thickness of 140 mm and are 2,5 m wide. The floor slab is a multi-span panel with individual spans of 5,1 m.

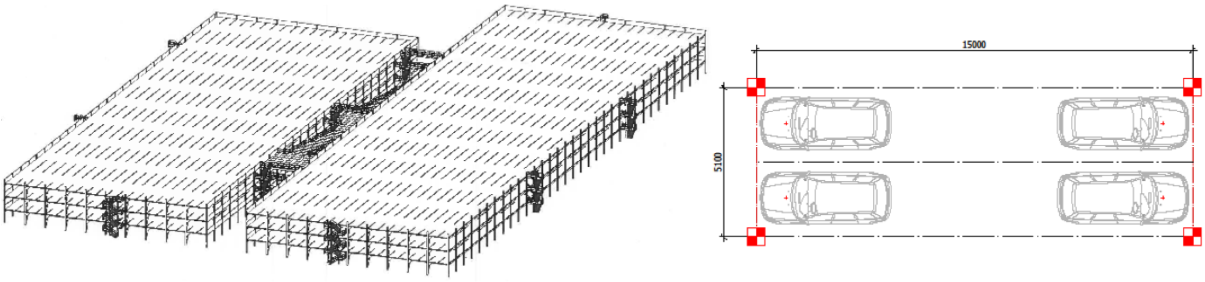


Figure 53. Left: overview of structure of timber-built car park Studen, right: dimension of grid (Zaugg, 2018)

Supporting beams

The CLT floor slabs are supported by glulam slab joists of 200 x 960 mm at every 5,1 m, see Figure 54. The slab joists are coupled multi-span elements with individual spans of 15 m and coupled by articulated joints. Various strength classes are used for different elements, varying between GL24h, GL28k and GL32k.

Figure 54. Structure of all-timber car park (Zaugg, 2018)

Columns

The slab joists are supported by V-shaped columns, allowing for a smaller span width and enabling optimum preconditions to be created for horizontal force dispersion, see Figure 55. The horizontal member in the columns, supporting the slab joists, is made of BauBuche GL70, to be able to transfer the large loads perpendicular to the grain. The struts supporting this member are made of glulam GL24h and varying cross-sections of up to 200 x 360 mm. In the middle of the column is a vertical glulam GL24h member, finally transferring the loads to the strip foundation.

Figure 55. V-shaped columns (Zaugg, 2018)

Stability

The CLT aisle slabs function as shear panels, transferring horizontal loads. In longitudinal direction, the horizontal loads are transferred to the V-shaped pillars. Horizontal stability in the transverse direction of the structure is realised by steel tie bars forming x-bracings. To take-up the additional vertical forces from the wind bracing, the pillars for the wind loads are made of BauBuche GL70. An extra horizontal coupling between the slab joists is made using small glulam beams.

Joints

Most of the joints used in the design are carpentry joints. The struts in the columns transfer the loads via end-grain contact to the vertical pillars. Step joints are used (see appendix C for a detailed description), in which the members are kept in place by an additional bolt at the bottom, see Figure 55. The horizontal member in the V-column is joined by a half-lap joint with again a bolt securing the members.

The large slab joists are placed on top of the horizontal member of the V-shaped column. They are however also connected to the vertical pillar by a grid of dowels and two bolts, securing the connection, see Figure 55. The slab joists are connected to each other several meters away from the columns at the point of minimal bending moments, see Figure 56. This joint is made up of a single large bolt with steel plates attached to both members.

The joints between the different columns and attachment of the bracing members to the columns is done via slotted in steel plates, see Figure 56. The CLT floor slabs are placed on top of the slab joists, but it remains unclear what specific connection between the floor slab and the joist is made.

Figure 56. Different joints in structure (Zaugg, 2018)

Vibrations

Vibrational behaviour of a multi-storey car park in timber was unknown for a timber structure of this size. Therefore, no reference information or standard practice was available. An existing hybrid construction for a car park with steel support structure and CLT floor panels was used as a reference. Both this construction and the new timber design were modelled and compared to each other to come to an estimation for comfortable behaviour or the all-timber structure.

4.6.1.2 Durability

To protect the structure from weathering and other durability concerns, several measures have been taken. The first are the separation layer and sealing membrane between the mastic asphalt layers and the CLT. This should prevent water coming of vehicles from coming in touch with the wood. The floors have also been placed with a 2% pitch to provide sufficient run-off of the water.

The design choice has been made to use hot-dip galvanised steel façade pillars and stair towers instead of timber variants, see Figure 57. As these elements are located around the façade of the building, they are significantly more exposed to weathering conditions and therefore of more risk to degradation.

Figure 57.Steel elements in façade of car park (Zaugg, 2018)

4.6.1.3 Production and Assembly

The design of the car park has been made modular. The V-shaped column elements were assembled in the factory and transported to the site. At the construction site, assembly of the columns, slab joists and bracings was fast and easy by simple dowels and bolts. The assembly process was vertical, which means every time a segment of three storeys was constructed and afterwards sealed simultaneously. As a result, the entire assembly process was very fast.

The car park has been realised as a permanent structure. Although many connections are made using bolts and dowels, the parking deck is sealed, which makes it very difficult to disassemble without damage. In terms of reuse, the car park is therefore difficult to disassemble.

4.6.2 Construction System for Multi-storey Car Parks in BauBuche

At the Technical University of Munich, a new construction system for a multi-storey car park has been developed, which is made of beech laminated veneer lumber (BauBuche). As beech is much stronger and more stable than softwood and it is highly available in Germany, the use of BauBuche was very interesting to study. The design is a modular system, which can also be disassembled, making it a very interesting reference project. However, the project has not been realised and remains a design on paper, so we cannot draw conclusions on how it performs in practice (Technical University of Munich & Pollmeier Massivholz GmbH & Co.KG, 2015).

4.6.2.1 Structural Concept

The car park is a modular system made up of multiple hybrid parking modules. These parking modules have a column-free span of 16.5 m and are 2.5 m wide, see Figure 58. The bays are 5 by 2.5 m. Car parks of many sizes can be created by combining the parking, ramp and stairwell modules in different configurations, making it a very flexible system. It allows for different types of car park layouts like full storey or split-level and different types of ramps. The maximum height of the construction is in theory only limited by building regulations.

Floor system

The floors are made of 130 mm thick prefabricated reinforced concrete elements, spanning 2.5 m. These elements require no extra layers to drive on, resulting in a floor system that is fast in erection and easy to disassemble. The floors are constructed on an angle of 2% for proper drainage and a gutter is incorporated into the concrete plates. In vertical direction, the floors also act as a fire barrier.

Supporting beams

The concrete floor elements are supported by timber BauBuche GL75 beams of 240 x 600 mm, which are cambered. They span the entire parking deck and have a length of 16.5 m. As the design of the car park is a system of multiple parking modules, the beams are single-span elements.

Columns

The beams are supported by square columns made of BauBuche. They are 240 x 240 mm and again of quality GL75. The columns extend over the entire height of the parking level which is 2.93 m with a clearance height of 2.2 m.

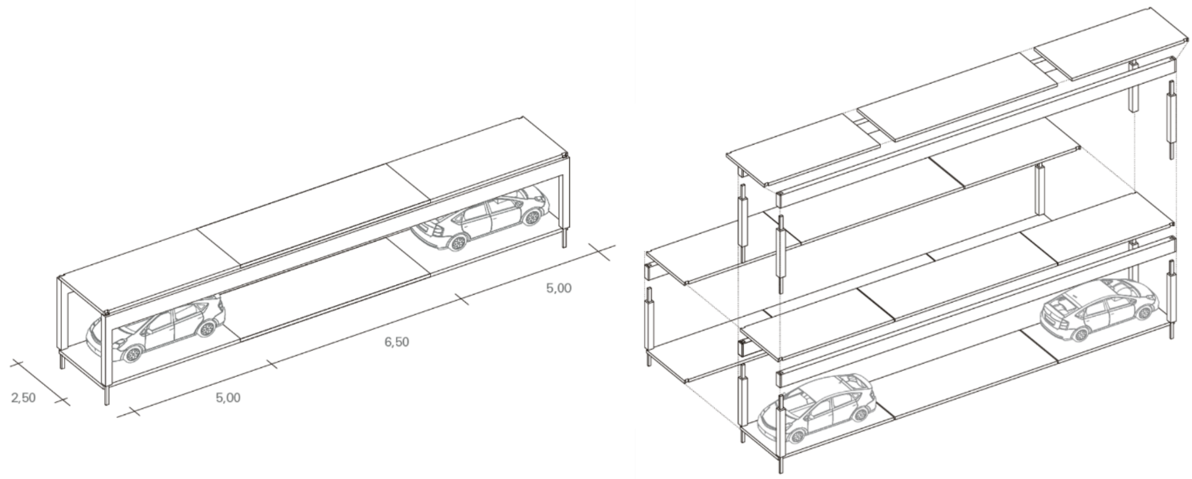


Figure 58. Parking modules and combination of modules (Technical University of Munich & Pollmeier Massivholz GmbH & Co.KG, 2015)

Stability

Stability in longitudinal direction is realised through steel elements in the post planes. In transverse direction, the building is stiffened by reinforced concrete walls of the stairwells.

Joints

The pre-cast concrete deck elements are connected to the supporting timber beams using birdsmouth joints, which are a type of carpentry joint (see appendix C for a detailed description). The result is that the concrete element and timber beam form a rigid structure.

The columns are connected to each other using hollow steel profiles (90 x 50 x 8 mm), to which a steel anchoring plate (200 x 200 x 8 mm) is welded as well as a slotted plate (150 x 200 x 10 mm), see Figure 59. The timber columns are in this way joined by the steel slotted plates using 12 mm steel dowels. The joint between the columns and the timber beams is filled with low-shrinkage expansive mortar.

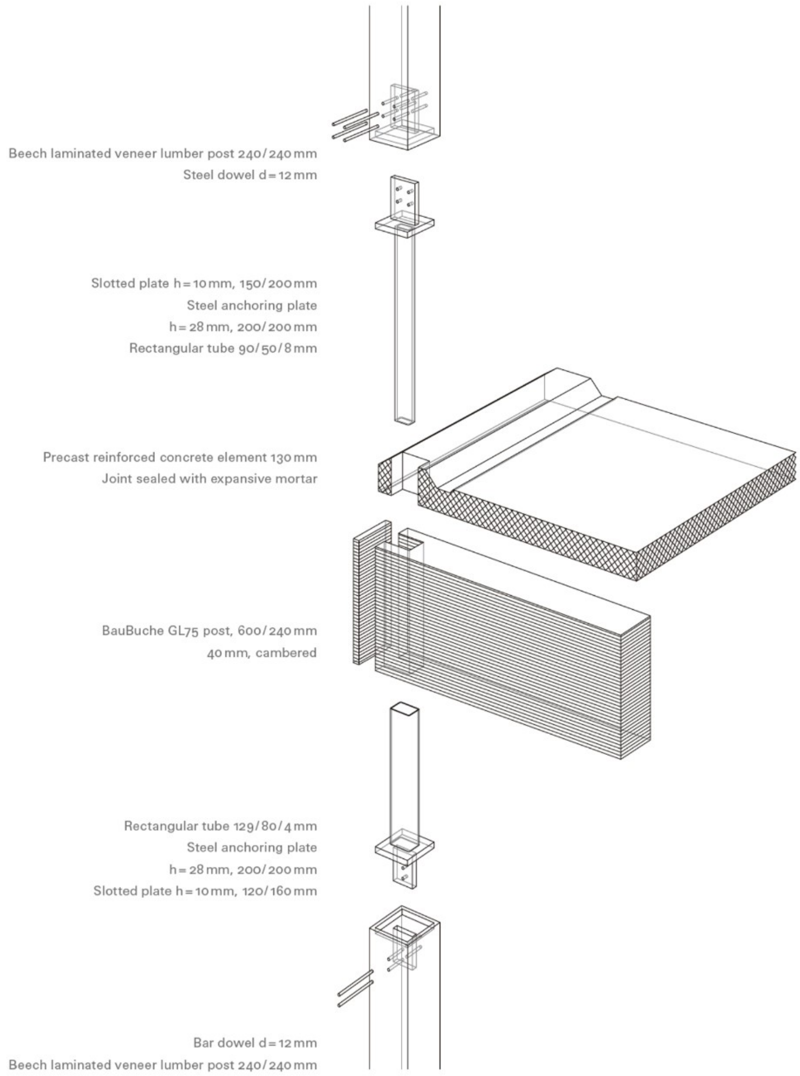


Figure 59. Detailed overview of construction system (Technical University of Munich & Pollmeier Massivholz GmbH & Co.KG, 2015)

4.6.2.2 Durability

Beech LVL elements are expected to last for more than 50 years, if proper installation, effective protection from the elements and regular inspections are ensured. Therefore, several measures are taken in the design to protect the timber elements. All the timber beams are coated for extra protection against moisture intake. Furthermore, the timber columns are separated from the concrete deck elements by expansive mortar. This prevents water collecting on the concrete surface to cause damage to the columns. Finally, on the top level, a roof is constructed which protects the rest of the structure against the elements.

4.6.2.3 Production and Assembly

The design incoporates a high degree of prefabrication. This ensures the highest possible quality, while keeping waste to a minimum. Furthermore, it allows for easy maintenance and makes it easy to replace a component of the structure if required. The system is completely standardised, resulting in very fast construction on-site. During construction, the concrete parking decks protect the rest of the timber elements and building, which is under construction, from the weather.

The modular system is also easy to disassemble. It uses reversible steel connections like push-in tubes, slotted plates and steel dowels. The large individual elements are therefore easy to separate and can be re-used or recycled.

A general impression of the design can be seen in Figure 60.

Figure 60. Artist impression of multi-storey car park in BauBuche (Technical University of Munich & Pollmeier Massivholz GmbH & Co.KG, 2015)

4.7 Concluding remarks

Concluding this chapter, sub-question 1a, “How to construct using timber as a structural material?”, can be answered.

In recent years, many developments in the timber engineering industry have paved the way for new possibilities for timber construction. Engineered wood products like glulam and CLT make it possible to span larger distances with timber and the improved characteristics (more homogeneous properties for example) of the materials make it possible to use timber in ways that earlier were not possible. Using these engineered wood products, different types of structural components can be made, with varying cross-sections and even composite beams. The results of the developments are visible in newly constructed multi-storey structures build with timber and also in the designs that are made for car parks in timber.

Building with timber requires special attention. Detailing of a timber construction is extremely important. Proper transfer of forces considering the inhomogeneous properties of timber and allowing for fast and cheap erection requires ingenious design. Also the type of connection chosen impacts the load bearing capacity of the joint, but also the reusability of the structural components. Therefore, one should consider what characteristics are required from a connection when choosing between glued, carpentry or mechanical joints. Carpentry joints are a good option for joints with rather small loads, which do not require a very stiff joint. In other cases, mechanical joints using dowel-type fasteners or connectors are a better option.

Also durability of a timber structure requires ingenious design as timber is prone to biological and environmental degradation as well as to fire safety risks. In general, the best protection of the timber is realised through constructional measures. Selecting the most appropriate wood species or type of wood engineered material is another important factor in designing for a durable construction. Also considering fire-safety of timber structures, ingenious structural design is required as not the timber elements themselves are of concern, but the joints between them are. Especially for a multi-storey car park, protection of the timber decks is of concern and requires a proper pavement surfacing.

From a technical point of view, the possibilities to construct a multi-storey car park using timber are endless. Ingenious design incorporating all aspects like transfer of forces, joining of different elements, protection against biological and environmental degradation and fire, allowing fast on-site construction and all of this while keeping the costs competitive is the challenge for the designer.

PART III: Concept Development

5

Definition Concept Development

In the literature study, a large amount of information considering the design of a temporary multi-storey car park in timber has already been discussed. The next step is to use this knowledge to create a proof of concept for the structural design of the car park. This requires filtering of the information found in the literature study and making choices about aspects like the car park layout to be considered, which structural typologies are considered for the design variants and which specific materials are considered. Furthermore, it should be clear what is to be expected from the design for the proof of concept and on what aspects its performance is to be compared to existing variants. This chapter aims to address these aspects and define the concept development by diving into the goals and expected results, the general assumptions and considered design variants.

5.1 Goals for Concept Development

Through the in-depth literature study and interviews with various experts in the AEC industry, a clear overview has been obtained concerning the challenges that arise when building a temporary multi-storey car park in timber. Based on this study and the challenges identified, we can more specifically formulate the main goal for the concept development, which incorporates solving various of the main challenges found in the literature study. Apart from this main goal, which is a rather broad description, two specific sub-goals are defined, which help to measure the performance of the design variants that are to be developed and which have an effect on many of the challenges found in the literature study. It is also discussed, what results are expected from the concept development, including a set of fourteen hypotheses. Next, the assumptions for the concept development are described and the different variants for the concept development are presented.

5.1.1 Main Goal

The objective of this thesis has been defined as to "provide a structural system proof of concept for a temporary multi-storey car park, using timber as primary structural material". The literature study has resulted in an overview of aspects which such a structure has to fulfil and what challenges have to be solved. Based on these results, when elaborating on the original objective, we can depict that the main goal for the concept development is threefold.

First of all, the concept for the temporary multi-storey car park should help the AEC industry in its path towards a sustainable industry. CO_2 emissions and energy consumption should be reduced, for which using timber as a primary structural material is a first step. However, in the concept development, more aspects related to making the sustainability transition should be taken into account. Incorporating the circular design strategies, as described in paragraph 3.3, in the concept is important to make the structure truly sustainable. This includes apart from closing the loop through the use of timber, strategies like designing for dis- and reassembly, reliability and durability, ease of maintenance and repair and upgradability and adaptability.

Second, building with timber comes with its challenges as discussed in chapter 4. Creating a durable construction through ingenious design is very important with this material as it comes with several durability risks, see paragraph 4.4. Building with timber comes with challenges concerning moisture, fire, acoustics and vibrations. Furthermore, one has to keep a close eye on the material characteristics to make optimal use of the material in the right places.

Third, the objective is concerned with a ‘temporary multi-storey car park’. Considering the tendency for timber structures to be more expensive compared to structures in traditional steel and concrete, it is important to win back costs where possible. Together with the temporary nature of the construction, a fast construction process is an important aspect as this can reduce the building costs. As the structure has to be able to function at various locations and with various user goals in mind, it should be modular, useful in different configurations and function at a high level of quality.

5.1.2 Specific Sub-goals

To make the main goal more concrete, two specific measurable sub-goals are defined. These sub-goals are based on some of the larger challenges found in the literature study, through interviews with various experts as well as analysis of several existing multi-storey car parks with a temporary character or designs in timber. These sub-goals help to later measure the performance of the different developed design variants, but in the process of the design of these variants, the main goal as presented before should always be kept in mind.

5.1.2.1 Minimizing Total Deck Height

In the literature study, it could already be seen from existing car parks in timber (see paragraph 4.6.1), that using glulam timber to span the large spans over the decks of a car park results in a large total deck height. In comparison, a concrete TT-slab can span the same distance with a total deck height halve that of the timber variant discussed in 4.6.1. With such spans, especially the deflection and vibrations of timber beams presents a challenge as the limitations on these deflections result in large required beam heights. Larger floor depths mean the total height of a floor level increases and longer ramps are required to reach the next level. All of this occupies space, which preferably would be used for parking cars instead of being required for the structure. Considering the lower weight of timber, it might also be possible to stack more elements on top of each other for transportation when the elements height is limited. Therefore, a first specific and easy to measure sub-goal is to minimize the total deck height of the concept for the temporary multi-storey car park.

In chapter 7, five different modular and/or timber car park structural concepts are compared to each other. This comparison includes the total deck height of the system and results in a concrete measurable number as a goal for the minimization of the total deck height of the timber variants to be designed.

5.1.2.2 Weight Minimization

The second specific sub-goal is to minimize the weight of the multi-storey car park. Minimizing the weight of the car park has many advantages considering both its temporary nature as well as the sustainability ambitions. A lighter structure requires less transportation as more elements can be transported per truck and requires less heavy machinery on-site for hoisting the elements into place. It requires smaller foundation works and possibly no foundation piles. Also the supporting structure of the floors for example has to carry less heavy loads, which can save on material. All of these aspects speed up the assembly and disassembly process, save costs and make the structure more sustainable, making it a very relevant and measurable goal. No exact numbers on costs will be gathered due to a lack of data and highly fluctuating material prices at the moment of publication of this thesis, due to the aftermath of the COVID-19 pandemic and war in Ukraine. However, minimizing weight in general has a positive influence on many relevant aspects.

Although a lower weight of the structure has many advantages, it also brings some challenges. Floors with a higher weight are for example less prone to vibrations in the structure and better prevent the transmission of sound compared to lighter floor types. Such aspects are important to keep in mind.

In chapter 8, it is studied what foundation types are possible for the car park and what range of loads coming from the columns of the car park could reduce foundation costs and improve the foundation’s level of sustainability. This results in another specific measurable sub-goal concerned with minimizing the weight of the structure.

5.1.3 Expected Results

Various alternative designs will be developed and analysed for the above mentioned goals. These can then be compared to each other for the specific sub-goals. The best performing variant for the structural design of the temporary multi-storey car park should provide the objective's proof of concept that fulfils the various requirements.

During the development process of the design variants, several input parameters will be altered to gain insight into what impact changing these parameters has on the structure and the (sub-)goals. Through this analysis, new knowledge is created on what type of design is best for which set of variables, what impact for example changing regulations (such as deformation limits) have on designs and if there are certain limits to designs for car parks in timber. These parameters can be split into two groups: geometric parameters and assessment criteria parameters.

The first set of parameters are related to the geometric design of the structural system of the car park. In paragraph 2.4, already a large set of parameters concerned with the geometry of the car park was given based on the design standard for parking facilities – NEN 2443 (Nederlands Normalisatie Instituut, 2013). However, this still leaves a lot of room for the designer to create the exact geometry of the structure. Furthermore, this standard is not required and can be deviated from. Such design choices can have a significant impact on for example the cross-sections of elements and therefore the specific sub-goals presented above.

The second set of parameters are concerned with the assessment criteria that the construction has to fulfil. As discussed before, due to the lightweight properties of timber, timber elements can be prone to vibrational behaviour. Also deformation limits can determine the minimum cross-section dimensions. The limits of such assessment criteria can be set by the designer and may have a significant effect on the structural system. Therefore, alteration of such assessment criteria can be highly interesting to study.

For the first set of parameters, it is studied what their effect is on both the choice of deck system and frame system (if parameter is applicable at design variant), presented in paragraph 5.3. For the second set, only their effect on the deck system is studied in detail, due to limitations of the used software package. However, the effect of these parameters on the framing system is shortly discussed in paragraph 10.4.4.

Geometric parameters:
- Parking deck span
- Use of struts in main span (yes / no)
- Distance between columns in transverse direction
- Distance between joists

Assessment criteria parameters (deck system only):
- Deflections
- Fire resistance class
- Vibrations

5.1.3.1 Hypotheses

Altering the parameters mentioned above will have an effect on the structural system of the multi-storey car park. Therefore it will also affect the outcomes of the specific sub-goals. For each parameter a hypothesis is drawn concerned with what its expected effect is on each specific sub-goal.

Parking deck span

A larger parking deck span is expected to require thicker floor elements or larger girders, increasing both the total weight of the structure as well as the total deck height. This results in hypotheses H1a and H1b.

H1a: Increasing the parking deck span will result in an increase of the total deck height.

H1b: Increasing the parking deck span will result in an increase of the total construction weight.

Use of struts in main span

Using struts to support the main girder of the structural system shortens the unsupported span of the main girder. Therefore, it is expected that this will reduce the cross-section of this girder, resulting in a decrease of the total deck height. Furthermore, it is expected that the reduction of the cross-section of the main girder will result in a larger decrease of material use than the additional material required for the struts. This results in hypotheses H2a and H2b.

H2a: The use of struts in the main span of the structural system of the car park will result in a decrease of the total deck height.

H2b: The use of struts in the main span of the structural system of the car park will result in a decrease of the total construction weight.

Distance between columns in transverse direction

Increasing the distance between the columns in transverse direction requires the floors or girders supporting the parking deck to span a larger distance. Therefore it is expected that this results in larger floor thicknesses and cross-sections, resulting in a larger total deck height and higher construction weight. The reduction of material use as the result of a smaller amount of columns is not expected to cover the additional required material for the larger cross-sections of floors and girders. This results in hypotheses H3a and H3b.

H3a: Increasing the distance between columns in transverse direction will result in an increase of the total deck height.

H3b: Increasing the distance between columns in transverse direction will result in an increase of the total construction weight.

Distance between joists

Increasing the distance between the joists results in a longer deck span. Therefore, it is expected this requires a thicker deck and results in bigger loads on the supporting joists, requiring larger cross-sections. This is would result in a larger total deck height and higher construction weight. The lower number of required joists is not expected to compensate for the larger weight of each joist. This results in hypotheses H4a and H4b.

H4a: Increasing the distance between joists will result in an increase of the total deck height.

H4b: Increasing the distance between joists will result in an increase of the total construction weight.

Fire resistance class (decks only)

The fire resistance class applied to a timber structure significantly influences the dimensions of the cross-sections of its structural elements, since for timber every minute of required fire resistance adds a certain thickness to the element (see paragraph 4.4.2.7). A lower fire resistance class can therefore result in smaller cross-sections. This results in hypotheses H5a and H5b.

H5a: Applying a lower fire resistance class to the structural system of the multi-storey car park results in a decrease of the total deck height.

H5b: Applying a lower fire resistance class to the structural system of the multi-storey car park results in a decrease of the total construction weight.

Vibrations (decks only)

Whether vibrational behaviour of a construction is checked can affect the dimensions of its structural elements. Heavier elements generally improve the vibrational behaviour of the structure. This of course impacts the total weight of the structure and the total deck height, resulting in hypotheses H6a and H6b.

H6a: Setting lower or no criteria for the vibrational behaviour of the structural system of the multi-storey car park will result in a decrease of the total deck height.

H6b: Setting lower or no criteria for the vibrational behaviour of the structural system of the multi-storey car park will result in a decrease of the total construction weight.

Allowed deflections (decks only)

Allowing larger deflections for the floors and girders of the structural system is expected to result in smaller floor heights and cross-sections, reducing total weight of the construction and total deck height. However, this is only valid if the serviceability limit state is governing. If the ultimate limit state is governing, no reduction of the floor height and cross-sections is possible. In general, hypotheses H7a and H7b are stated.

H7a: Allowing larger deflections for the structural elements of the car park will result in a decrease of the total deck height.

H7b: Allowing larger deflections for the structural elements of the car park will result in a decrease of the total construction weight.

5.1.3.2 Deliverables

As final result of this thesis, the goal is to present a proof of concept with design considerations and prerequisites for the design of a temporary multi-storey car park using primarily timber as building material as an addition to existing knowledge on building in timber. These results are related to the following aspects:

- The applicability of timber in a temporary multi-storey car park.
- The number of levels possible in timber.
- Expected dimensions for various elements.
- Expected floor heights for a timber car park.
- Effect of angled parking on the structure of a multi-storey car park.

5.2 General Assumptions Concept Development

For the development of the concept for the structural system of the temporary multi-storey car park, several assumptions have to be made. Already in the literature study, several assumptions have been described concerning aspects like user and owner requirements, structural rules and norms and car park design rules and norms. This paragraph focusses on some of the most important assumptions that influence the design of the structural system.

5.2.1 Car Park Design

A concept for the structural system of a temporary multi-storey car park will be developed, which has to be flexible considering the possible different locations and user functions it has to fulfil. Although it has a temporary character, it has to fulfil the high quality standards of a normal fixed car park both in terms of service to its users as well as structural requirements as described in chapter 2. To fulfil all possible requirements, the concept is developed for the following category: a public multi-storey car park, which is intensively used, see paragraph 2.1.1.

5.2.1.1 Main Span

For the dimensions of bays and aisles, the minimum dimensions from NEN 2443 are used as a starting point, see paragraph 2.1.3 and 2.4.2. The main span is formed by the combination of the dimensions of the length of a parking bay, the width of the aisle, the additional width of the redress lanes and optionally room for a pedestrians walkway. Depending on the parking angle, the length of the aisle and the optional extra room for pedestrians, the main span can vary from a minimum of 12.18 m to a maximum of 17.66 m for the standard dimensions of NEN 2443.

5.2.1.2 Other Dimensions

Starting point for the width of the parking bays is 2.50 m as stated in NEN 2443. A standard clearance height of 2.30 m is used. Furthermore, a column free parking deck is considered required for the concept design. The various dimensions are discussed in detail in paragraph 2.4.

5.2.2 Car Park Circulation Layout

In the literature study in paragraph 2.5 and in appendix B, various car park circulation layouts have been discussed. For the development of the concept design, a choice has to be made which layout(s) is/are considered. For the circulation layout of the car park, the sloped parking deck layouts are ruled out as they contain multiple undesirable characteristics. Considering the user-friendliness of the car park, their sloped parking decks hinder users in getting in and out of their car as doors slam shut or are difficult to close and shopping cars and wheel chairs roll away on the decks. Considering the required modularity of the concept, sloped parking deck layouts result in fixed lengths for the decks as they connect the different floor levels. This hinders the modularity and scalability of the car park concept.

Flat deck circulation layouts are considered as an option. Although their static efficiency is not the best, their other characteristics are generally good. A large advantage of flat deck layouts is that they can be highly modular. With separate ramp systems, car parks of all kinds of sized and layouts can be realised, which is very relevant for the development of a temporary multi-storey car park.

Split-level deck circulation layouts are the preferred option considering their average static efficiency. Compared to the flat deck layouts, their dynamic efficiency is not as good, but at the same time, their circulation efficiency is very high and other characteristics are also good. Downside of the split-level decks is that although they can vary in size, they generally consist out of two bins and require ramps between the split-level decks, making them less flexible for different layouts.

As the structure of both flat decks and split-level decks uses a comparable structure to span the parking decks, it might be possible to develop a structural system, which can function both as a flat deck layout as well as a split-level deck layout.

5.2.3 Building Materials

The primary building material to be used for the design of the multi-storey car park is timber. However, also other building materials may be used if their advantages over the use of timber are very significant and their impact on sustainability is acceptable. As the production of concrete results in high amounts of CO_2 emissions and energy use, is very difficult to fully recycle and adds a large amount of weight to a structure, it is not used for the concept development of this thesis. Steel, although requiring large amounts of energy for its production is allowed as it is fully recyclable, durable, has a high reuse potential and is a very good material to be used in for example connections.

5.3 Design Variants

In the literature study, different structural typologies (frame and deck systems) have been studied, see paragraph 2.6. For the concept development, a choice has to be made concerned with which structural typologies are used to create design variants. Although the framing system and deck system influence each other, they are discussed separately. The deck systems are divided in short and long span systems and for the framing systems, it is stated what type of deck system is required.

5.3.1 Deck System

Possible deck systems have been discussed in paragraph 2.6.1 (deck typologies in general) and 4.2.2 (possible types of timber decks). Considering the modular and temporary aspects of the multi-storey car park to be developed, only two possible type of systems are deemed possible. These are either long-span or short-span prefab elements. Within these type of deck systems, various alternatives exist.

5.3.1.1 Long-span Prefab Deck System

A total of seven different long-span prefab deck systems are considered, which are presented below. These systems use either CLT elements (see Figure 61) or LVL elements (see Figure 62):

- CLT panel;
- CLT open rib panel;
- CLT closed rib panel (box floor);
- LVL panel;
- LVL open rib panel;
- LVL semi-open rib panel;
- LVL closed rib panel.

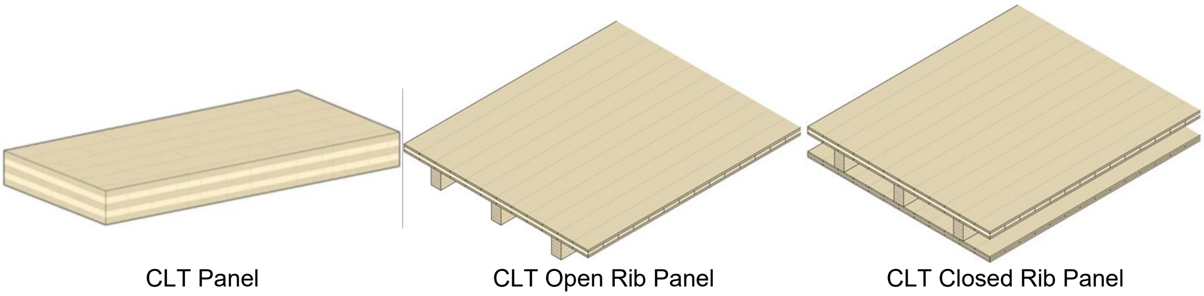


Figure 61. Long-span prefab deck systems in CLT (Stora Enso, 2022f)

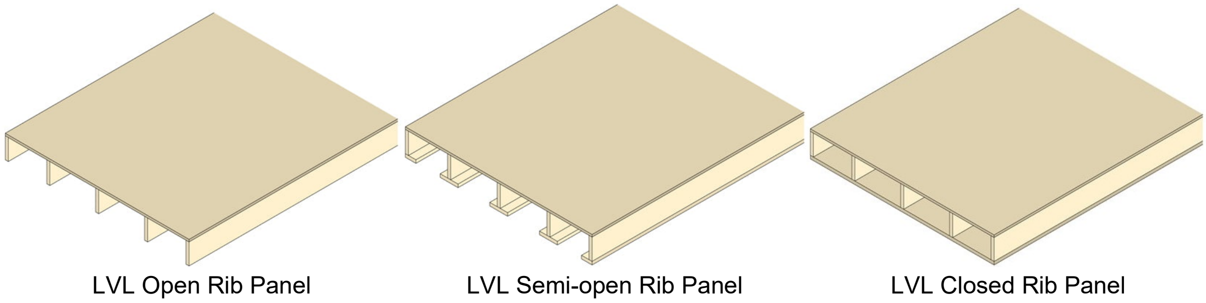


Figure 62. Long-span prefab deck systems in LVL (Stora Enso, 2022f)

5.3.1.2 Short-span Prefab Deck System

For the short-spans, only two types of prefab elements are considered (see Figure 63), which may be supported by multiple girders, further shortening the (sub)spans:

- CLT panel;
- LVL panel.

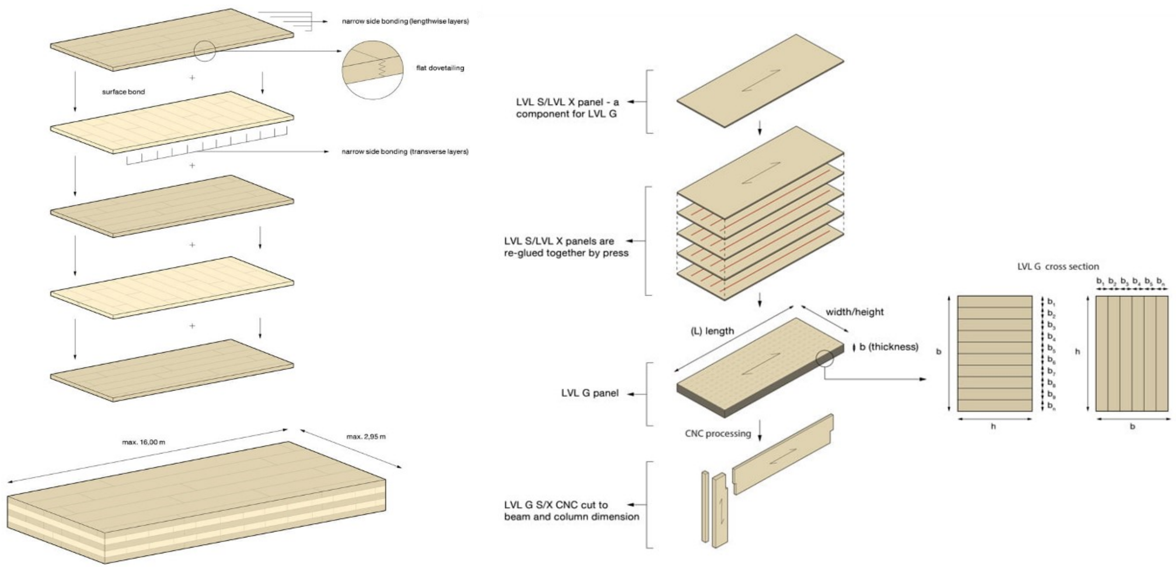


Figure 63. CLT panel (left) and LVL panel (right) used for short-span prefab deck systems (Stora Enso, 2022c, 2022d)

5.3.2 Framing System

Following from the literature study (see paragraph 2.6.2), four different main variants for the framing system of the car park are seen as potential solutions for the concept design. These have a column free parking deck and lent themselves to be build using primarily timber materials. The exact design of each variant, like the way the different elements are connected to each other and their dimensions may be altered. The general configuration with the span directions of each element however will remain unchanged.

5.3.2.1 Variant 1

The first variant to be studied consists out of a structural frame from columns and transverse girders, see Figure 64. These girders support bin wide floor elements, for which one of the long-span prefab deck systems is to be chosen. The columns can be positioned every 2.5 m (one parking bay width) or 5.0 m (two parking bay widths). Also the parking deck span can be varied as well as the number of levels. Variant 1 does not contain struts in its structural system.

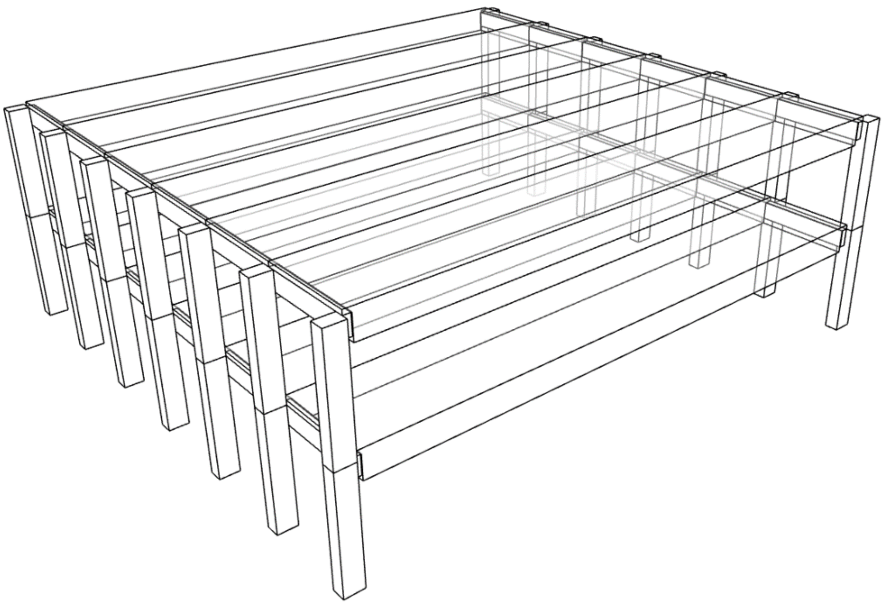


Figure 64. Variant 1 - 3D overview of design variant

5.3.2.2 Variant 2a

The second variant to be studied consists out of a structural frame using columns, main girders and optional struts, see Figure 65. The main girders support a thin parking deck, for which a short-span prefab deck system has to be chosen. The columns can be positioned every 2.5 m (one parking bay width) or 5.0 m (two parking bay widths). Also the parking deck span can be varied as well as the number of levels.

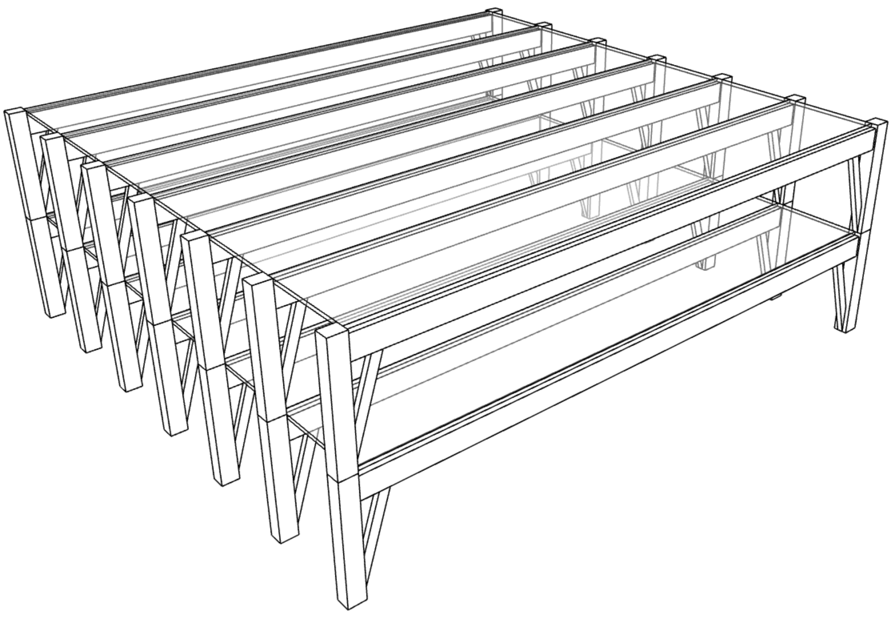


Figure 65. Variant 2a - 3D overview of design variant

5.3.2.3 Variant 2b

Design variant 2b is an alteration of previous design variant. It contains only halve the number of columns as every subsequent main girder is supported by additional transverse girders instead of columns, see Figure 66. These transverse girders transfer the loads to the remaining columns. The main girders support a short-span prefab deck system that has to be chosen. The columns are positioned at 5.0 m (two parking bay widths) and struts are not used. The parking deck span can be varied as well as the number of levels.

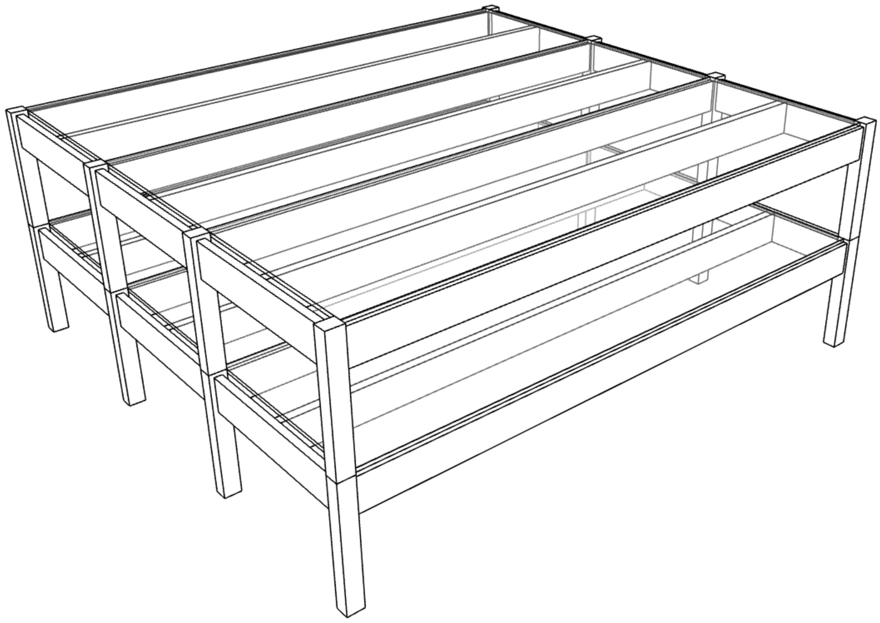


Figure 66. Variant 2b - 3D overview of design variant

5.3.2.4 Variant 3

The last variant to be studied looks similar to variant 2a and uses a framing system consisting out of columns, main girders and optional struts, but additionally contains secondary girders (joists), see Figure 67. These secondary girders span the distance between the main girders and support a short-span prefab deck system over a shorter span. The columns are positioned at 5.0 m (two parking bay widths) and the parking deck span can be varied as well as the number of levels.

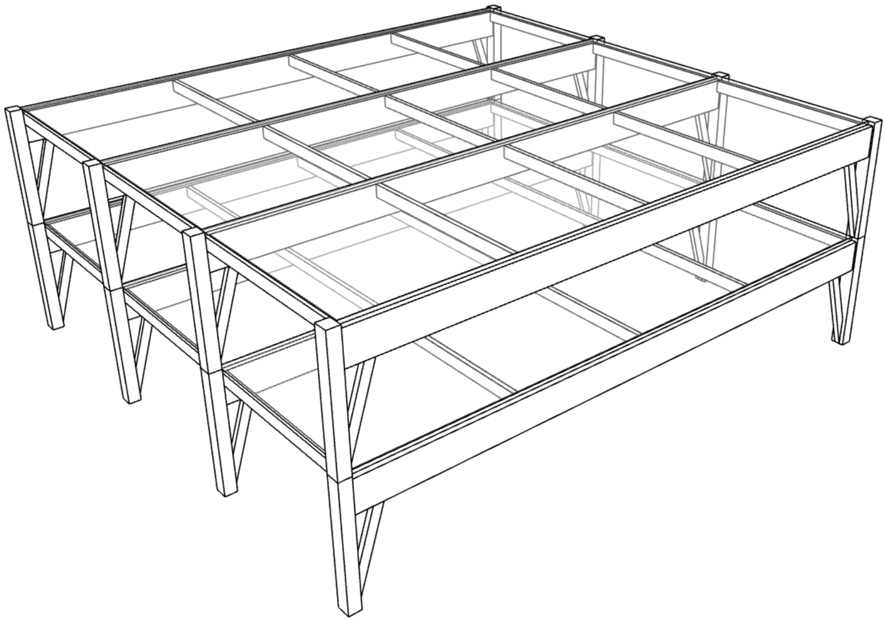


Figure 67. Variant 3b - 3D overview of design variant

5.4 Concluding Remarks

In conclusion of this chapter, sub-question 3a, "What characteristics of a temporary multi-storey car park using timber as primary structural material are relevant to assess its structural performance?" can be answered.

Following from the literature study, this chapter has resulted in a narrowed down definition of the concept development. The main goal for the concept development has been stated and is threefold: focus on the lifecycle of the structural system, solve the material challenges related to building with timber and take into account the temporary nature of the concept to be developed, linked to costs and modularity. Two specific concrete and measurable sub-goals have also been stated to be able to assess the performance of the various designs and compare them to alternatives: minimization of total deck height and minimization of the weight of the structural system.

Furthermore, four geometric parameters and three assessment criteria parameters are chosen to study their effect on the performance of the different variants. These are the parking deck span, use of struts in the main span, distance between columns in transverse direction, minimum distance between joists, allowed deflections, fire resistance class and vibrational criteria. For each of these parameters, a hypothesis is stated for its relation to the two specified sub-goals.

Multiple design variants will be studied for their performance. These consist out of a deck and framing system. For the deck system, a total of seven different long-span prefab deck systems are considered and two short-span prefab deck systems. For the framing system, four different typologies are chosen to be studied.

6

Structural Starting Points Conceptual Design

In the design process of the proof of concept for the modular and temporary multi-storey car park, choices have to be made, steering the design process towards the final proof of concept. In chapter 5, already a selection was made from the information found in the literature study concerned with the general design of the car park and its structural typology, resulting in the selection of four different framing systems, seven different long-span deck systems and two different short-span deck systems. This chapter focusses on the structural starting points, which are required for the later calculations of the design variants. Much of the information in this chapter was already discussed in the literature study in paragraph 2.3, but this chapter acts as a summary and clear overview of the structural starting points for the design and calculation of the design variant.

6.1 General

The car park is classified as a public car park according to NEN 2443 (Nederlands Normalisatie Instituut, 2013). This norm is regarded as leading and any deviation from its contents will be explicitly mentioned.

The structural system of the car park consists of a timber frame. The deck is made from CLT or LVL and the beams and columns are made of glulam or glued solid timber. Vertical stability is provided by steel bracing and horizontal stability is provided by in-plane action of the decks.

6.2 Applied Structural Rules and Norms

For the design, in principle the following rules and norms have been applied. Any deviation from this will be explicitly mentioned.

- Dutch Building Degree 2012
  (Ministerie van Binnenlandse Zaken en Koninkrijkrelaties, 2021)
- NEN-EN 1990+NA: Eurocode 0, Basis of structural design
- NEN-EN 1991+NA: Eurocode 1, Actions on structures
- NEN-EN 1995+NA: Eurocode 5, Design of timber structures
- NEN-EN 1997+NA: Eurocode 7, Geotechnical design
  (NEN, 2021)

6.2.1 Consequence Class, Service Class, Design Life & Load Factors

The structure of the multi-storey car park is designed according to NEN-EN 1990 (2011) + NA (2019) (NEN, 2021) and NEN-EN 1995-1-1 (2005), assuming the following conditions:

Type of building:	Public multi-storey car park	
Consequence class:	CC2	
Reliability class:	RC2	$k_F = 1.0$
Design service life category:	Class 3	50 years
Service class (timber):	Class 2	

Table 14. Load factors ULS and SLS (Nederlands Normalisatie Instituut, 2015)

Description	Type of load	Factor	6.10a	6.10b
Ultimate Limit State (ULS)	Permanent	$\gamma_{G;unfavourable}$	1.35	1.2
	Permanent	$\gamma_{G;favourable}$	0.90	0.9
	Variable	γ_Q	1.5 $\psi_{0,1}$	1.5
Serviceability Limit State (SLS)	Permanent	$\gamma_{G;unfavourable}$	1.0	1.0
	Permanent	$\gamma_{G;favourable}$	1.0	1.0
	Variable	γ_Q	1.0	1.0

6.2.2 Load Combinations

Load combinations for the Ultimate Limit States (ULS) are assumed according to art. 6.4.3 of NEN-EN 1990 (Nederlands Normalisatie Instituut, 2015).

Load combinations for the Serviceability Limit States (SLS) are assumed according to art. 6.5.3 of NEN-EN 1990 (Nederlands Normalisatie Instituut, 2015).

Appendix A gives a more detailed overview of the formulas for load combinations.

6.2.3 Function of Building & Combination Factors

All levels of the building are designated as category F: Traffic area, vehicle weight ≤ 25 kN, according to NEN-EN 1990 + NA (Nederlands Normalisatie Instituut, 2015). Table 15 gives an overview of the relevant combination factors for the building.

Table 15. Combination factors for buildings. (Nederlands Normalisatie Instituut, 2015)

Action	ψ_0	ψ_1	ψ_2
Imposed loads on buildings:			
Category F: Traffic area, vehicle weight ≤ 25 kN	0.7	0.7	0.6
Snow loads	0	0.2	0
Wind loads	0	0.2	0
Standing water	0	0	0

6.3 Loads

This paragraph describes the characteristic load values for permanent and variable loads on the structure.

6.3.1 Permanent Loads

The following permanent loads act on the structure, see Table 16. As the car park is naturally ventilated and only a simple and lightweight lighting scheme is assumed (a single luminaire per relatively large area), no permanent load for installations is considered.

Table 16. Permanent loads car park

Description	Weight	
Self-weight	(Differs per element)	
Aggregate coating parking deck	0.1 kN/m^2	(HIM, 2020)
Façade & safety barrier	2.0 kN/m^2	(Kaa, 2020)

6.3.2 Variable Loads

6.3.2.1 Imposed Loads

For category F, traffic and parking area for light vehicles, a distributed and concentrated load are imposed, see Table 17. The concentrated load has to be applied as an axle load, following the dimensions in Figure 68, in which the square surfaces have sides of 100 mm.

Table 17. Imposed loads on garages and vehicle traffic areas. (Nederlands Normalisatie Instituut, 2019a)

Category of traffic area	**q_k [kN/m^2]**	**Q_k [kN]**
F (light vehicles ≤ 25 kN)	2	10

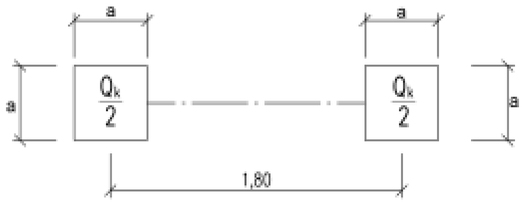


Figure 68. Dimensions of axle load. (Nederlands Normalisatie Instituut, 2019a)

6.3.2.2 Wind Loads

For the wind load, the following assumptions are made (see Appendix A for detailed formulas):

Wind region: I
Terrain category: 0 (coastal area)
Height: 21.0 m
c_sc_d: 1.0

From these assumptions follows: $q_p(z_e)$ = 1.82 kN/m^2

6.3.2.3 Snow Loads

The top deck of the car park also acts as a roof. Therefore, the roof shape of the car park is flat: μ_1 = 0.8. It is assumed snow cannot accumulate against barriers on the deck. This results in the following snow load (see Appendix A for detailed formulas):

s = 0.56 kN/m^2

6.4 Deformations

According to the Dutch Building Degree, deformations of a structure do not have to be tested. However, at first, the advised maximum allowed deformations according to the Eurocode NEN-EN 1990 NA (Nederlands Normalisatie Instituut, 2015) are applied. Later, it is studied what the effect is when larger deformations are allowed.

6.4.1 Vertical Deformations

The maximal allowable vertical deflections can be found in Table 18. The different parameters used are displayed in Figure 69.

Table 18. Allowed vertical deflections. (Nederlands Normalisatie Instituut, 2015)

Type	**Allowed additional deflection ($w_2 + w_3$) [$*\ell_{rep}$]**	**Allowed final deflection (w_{max}) [$*\ell_{rep}$]**
Floors	0.003	0.004

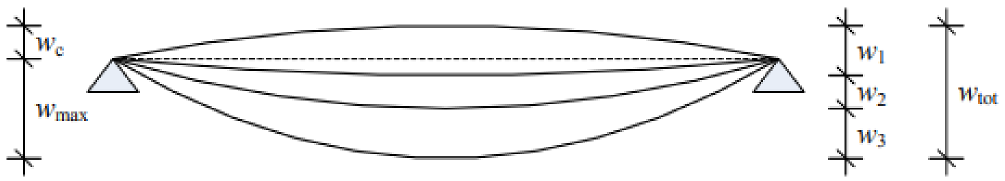


Figure 69. Vertical deflection parameters. (Nederlands Normalisatie Instituut, 2015)

6.4.2 Horizontal Displacements

The total maximum horizontal displacement for buildings with more than one storey under the characteristic combination of actions is:

- h/300 per storey (u_i Figure 70)
- h/500 for the entire building (u Figure 70)

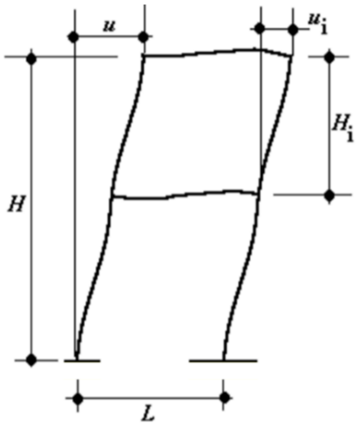


Figure 70. Definition of horizontal displacements. (Nederlands Normalisatie Instituut, 2015)

6.5 Vibrations

Vibrations of the decks of the structure are checked for a frequency criterion of 4.5 Hz and a stiffness criterion of 0.500 mm, see paragraph 2.3.3. Later on, it is also studied what the effect on the performance of the structure is when these criteria are not considered.

6.6 Fire Safety

For the main load bearing structure of the multi-storey car park, as a starting point, a fire resistance requirement against collapse of 90 minutes is applied. Later on it is studied what effect reducing the fire resistance requirement has on the dimensions of the structural system. See paragraph 2.3.4 for more information.

6.7 Design Parameters

To limit the range of possible designs to be studied, several design parameters have to be set. This paragraph presents a set of fixed design parameters as well as several variable parameters (as discussed in paragraph 5.1.3), of which their effect on the structural design of the multi-storey car park is studied.

6.7.1 Fixed Parameters

Several parameters for the design of the multi-storey car park are fixed or are related to other parameters of the car park. An overview of these parameters can be found in Table 19.

Table 19. Fixed design parameters

Parameter	**Value**
Clearance height	2.30 m
Parking bay width	2.50 m
Vertical height strut	Level height
Horizontal length strut	0.67 m
Dimensions for wind calculation	
Total car park length	60 m
Total car park width	40 m
Nr. of levels	5

Material classes	
Glulam timber	GL 28h
Glued solid timber	C24
Steel	S235

6.7.2 Variable Parameters

Some parameters are varied, to study what effect this has on the rest of the structure. These variable parameters were already mentioned in paragraph 5.1.3. Inn Table 20 an overview is given of these parameters, their range of values and for which design variants they are relevant.

Table 20. Variable design parameters

Parameter	**Values**
Number of floor levels	1 to 5
Parking deck span	12.18 m
	13.70 m
	14.74 m
	15.61 m
	16.26 m
	17.16 m
	17.66 m
Allowed deformations	Allowed additional deflection 1/333 * l_{rep}
	& allowed final deflection 1/250 * l_{rep}
	Allowed additional deflection 1/183 * l_{rep}
	& allowed final deflection 1/150 * l_{rep}
Distance between columns (variants 1 & 2a)	2.5 m
	5.0 m
Distance between joists (variant 3)	Max 2.5 m
	Max 3.75 m
Use of struts (variants 2a & 3)	Yes
	No
Fire resistance class	R 90
	R 30
Vibrations	Criteria: frequency 4.5 Hz & stiffness 0.500 mm
	No check on vibrations

7

Modular or Timber Car Park Concepts

Over the years, many different types of multi-storey car parks have been constructed using various structural concepts. The aim of this thesis is to provide a structural system proof of concept for a temporary multi-storey car park, using timber as primary structural material. Given this objective, one of the sub research questions is to study how such a structural system compares to existing alternative concepts. In this chapter, the basis for the later comparison of the developed structural system is laid by comparing five different modular or timber structural systems. The performance of these alternative concepts is assessed for the specific sub-goals presented in chapter 5. This helps to create a feeling for what level of performance is to be expected from the proof of concept to be able to compete with these alternative concepts.

7.1 Description of Concepts

Since the goal is to design a structural system which uses primarily timber and also is modular, it would be best to compare it to an existing multi-storey car park with such characteristics. However, to the knowledge of the author, such a car park has not been realised yet. Therefore, the focus has been placed on finding either modular car park structures of car parks constructed primarily in timber, to compare the structural system to. This has resulted in five different multi-storey car parks, which have five different types of structural systems. The various concepts are discussed in the following paragraphs.

7.1.1 ModuPark

ModuPark is a modular multi-storey car park concept developed by Ballast Nedam Parking, which is comprised of prefab steel and concrete elements, see Figure 71 (BNParking, 2016). Advantageous of the system is the speed of construction. The car park can be built in just three months. When no longer needed, the entire structure can quickly be disassembled and reused at a new location. Downside of the system is the required heavy foundation as a result of the concrete elements, which often requires concrete foundation piles, which are difficult to remove and reuse.

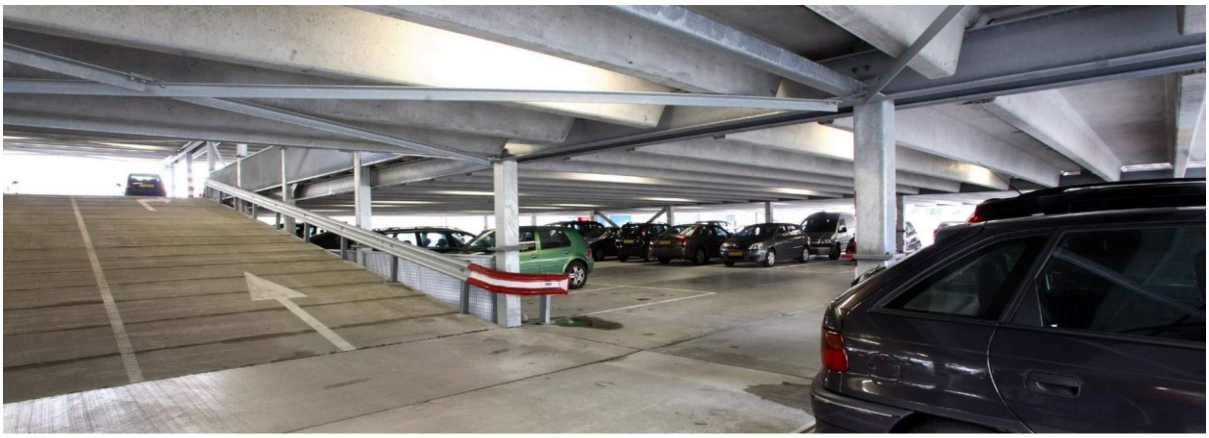


Figure 71. ModuPark Maarssen (ASK Romein, 2018b)

The concept uses steel columns, transverse girders and bracing, creating a frame, on which concrete TT-plates are placed, spanning the entire parking deck, see Figure 71. These TT-plates do not require any finishing layer and can be directly driven on. A standard TT-plate has a width of 2400 mm (one parking bay) and a length of generally 16.5 m (ASK Romein, 2018a). Columns are placed at a distance of 4.8 m from each other, creating a grid of 16.5 x 4.8 m. Depending on the number of levels, a different thickness of the columns is used to support the larger loads from the upper decks. A total of 6 levels is perfectly possible. Using the standard elements, car parks of many different shapes can be created and by adapting some of the elements, even more shapes are possible. This makes the concept useful in many locations and stimulates reuse.

7.1.2 Car Park Koopman int.

The multi-storey car park for Koopman int., located at the Distelweg in Amsterdam is comprised of prefab steel and fibre-reinforced plastic elements (Park4all, 2020). The system is relatively fast to construct and can easily be disassembled and reused. Since only steel and fibre-reinforced plastics are used, the total structure is extremely lightweight, allowing for a less heavy foundation.

The structural system of the car park uses steel columns, main girders and secondary girders on top of which a fibre-reinforced plastic floor is placed, see Figure 72. This floor uses Fiberline heavy duty planks with a thickness of only 40 mm and a weight of only 8.43 kg/m for a standard width of 500 mm (Fiberline, 2021). A column free parking deck with a span of 15.65 m is realised and columns are placed at every 5.0 m. The car park is realised for two levels, but is designed for addition of two more levels (Park4all, 2020). The car park is designed for a specific client, but can easily be disassembled and reused. As the concept uses all prefabricated elements which are bolted together, it also can be used as the basis for a modular and flexible car park system.

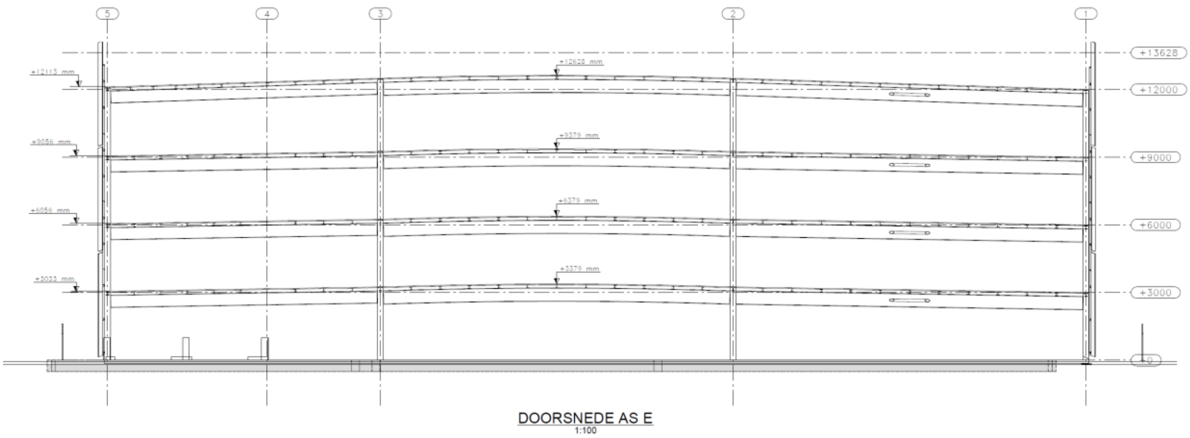


Figure 72. Section of car park Koopman int. Amsterdam (EdJ, 2019)

7.1.3 Car Park Morspoort

The multi-storey car park Morspoort in Leiden is a temporary and demountable structure comprised of prefab steel and steel-concrete composite elements. As it is designed as a temporary structure, ease of disassembly has been taken into account during the design (JVZ Ingenieurs, 2012). The result is a structural concept, which is fast to construct and disassemble and can be reused. The use of steel-concrete composite elements reduces the total weight of the structure and allows lighter foundations.

The car park's structural system is build-up using steel columns at every 6.85 m, supporting the main girders spanning the parking deck with a span of 14.5 m, see Figure 73 (JVZ Ingenieurs, 2012). The main girders support prefabricated composite steel-concrete floor panels, which is the Corus Quantum Deck system. This floor system is a prefabricated steelframeconcretefloor, build-up of cold formed C-profiles, poured into a 51 mm thick concrete plate. This results in a reduced weight of 180 kg/m^2 (Tata Steel, 2021). The structural system can be adapted for many different shapes and sizes of car parks, including rounded edges.

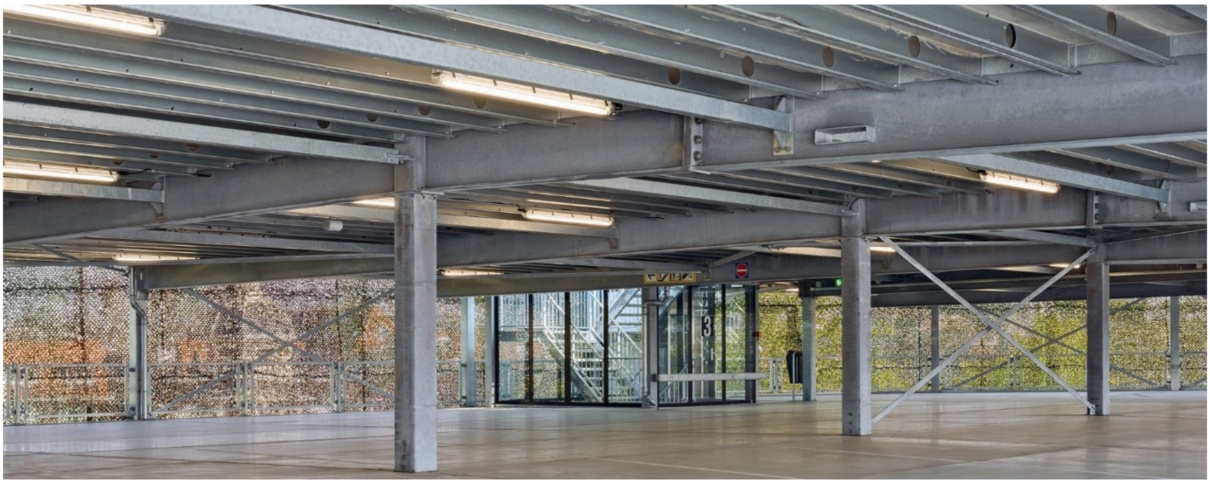


Figure 73. Car Park Morspoort, Leiden (De Ruiter, 2011)

7.1.4 BauBuche Concept

At the Technical University of Munich, a concept for a modular and demountable multi-storey car park in timber and concrete has been developed (Technical University of Munich & Pollmeier Massivholz GmbH & Co.KG, 2015). This concept was already discussed in paragraph 4.6.2, but is shortly discussed here as well. The structural system is comprised of timber elements in BauBuche (a new development in glulam beams with higher strength properties) and prefabricated concrete floor elements. The entire system is designed to be highly modular, demountable, fast in construction and easy to reuse. The timber elements made from BauBuche allow for smaller dimensions of its cross-sections, but the concrete floor elements significantly add weight to the total system.

As can be seen in Figure 74, the concept uses a standardized modular element made of timber columns at every 2.5 m, supporting timber main girders, spanning the parking deck of 16.5 m. These main girders support 130 mm thick prefab concrete floor panels. The columns and main girders are connected to each other using a ingenious system of steel tubes, slotted plates and dowels, which allow large loads to be transferred from upper decks to the foundation (Technical University of Munich & Pollmeier Massivholz GmbH & Co.KG, 2015). The concept is developed to be used in many different types of car park layouts, such as flat decks, split level and sloped decks.

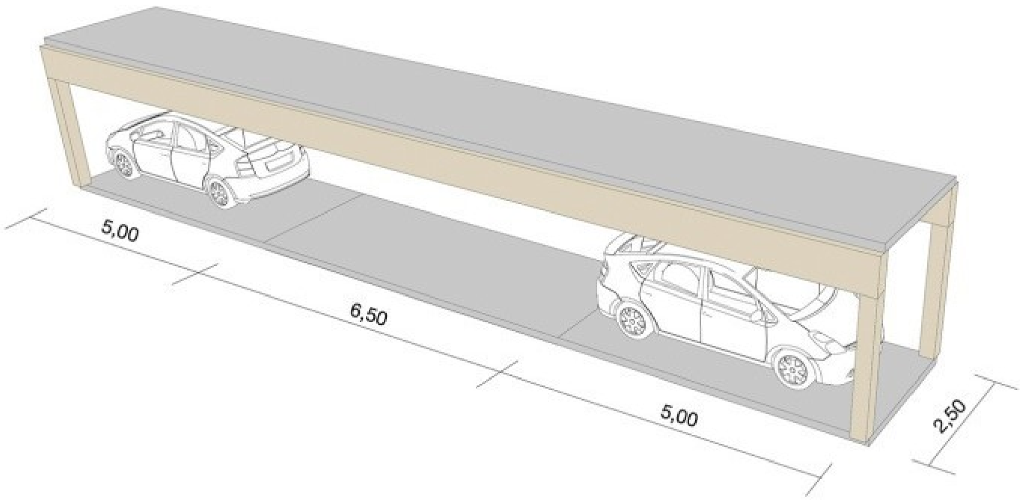


Figure 74. Modular element of modular car park concept in BauBuche (Technical University of Munich & Pollmeier Massivholz GmbH & Co.KG, 2015)

7.1.5 B&O-Holzparkhaus

The B&O-Holzparkhaus is the only multi-storey car park in this chapter which has a superstructure almost completely made from timber, see Figure 75. It is not designed as a modular, demountable or reusable structure, but is very useful as a concept fully build in timber to compare to the other concepts. The car park is made from different types of wood engineered products, including BauBuche, glulam and CLT. As timber is a relatively lightweight material, the entire structure also has a relative low self-weight (HK Architecten, 2021).

The car park has only one span and two levels. Glulam columns are placed at every 2.60 m, supporting the main girders made from BauBuch GL75, spanning the parking deck of 16.26 m. A CLT parking deck is placed on top of the main girders and is finished with a 60 mm thick layer of asphalt (HK Architecten, 2021). This unfortunately makes disassemble and reuse of the deck very difficult. Between the main girders, additional CLT panels are placed for stability as well as steel wind bracings in the façades.

Figure 75. Timber multi-storey car park Bad Aibling (HK Architecten, 2022)

7.2 Comparison

A comparison is made between the five different modular or timber multi-storey car park concepts presented in previous paragraph. This comparison helps to gain insight into how the use of different concepts and different building materials results in different performance on various aspects: total deck height, weight of the structure and load from a “standard” column on the foundation. It helps to get a feel for what range of values for the various aspects are to be expected and can act as a basis for what performance is requested from the timber car park designs. Later, the timber car park designs can be compared to these values and there performance can be assessed.

As all five car park concepts have different dimensions, they first have to be altered to be able to compare them to each other. The dimensions found in Table 21 are used for all concepts. If necessary, profile dimensions are also in- or decreased to be able to support the loads given the dimensions for the comparison. An overview of the details of the car park concepts like used profile dimensions and their properties can be found in appendix E, Table 55 and Table 56.

The comparison of the three aspects are discussed in the following paragraphs. Table 22 gives an overview of the performance of the five car park concepts on the three aspects. It also shows a comparison of four car park concepts relative to the ModuPark concept, which has the smallest total deck height.

Table 21. Standardized car park characteristics

Car park characteristics			
Parking bay width	2.5 m	Span parking deck	16.26 m
Parking bay length	5.13 m	Clearance height	2.30 m
Aisle width	6.00 m	Column every 1 or 2 parking bays	
Parking angle	90°		

Table 22. Comparison of various modular or timber car park concepts on floor depth, weight and standard column load

Car park	**Floor depth [mm]**	**Rel. to ModuPark**	**Weight /level [kg]**	**Rel. to ModuPark**	**Load standard column on foundation - 4 levels [kN]**	**Rel. to ModuPark**
ModuPark	550	-	61,261	-	2,472	-
Koopman int.	640	116%	10,477	17%	1,258	51%
Morspoort	651	118%	35,110	57%	1,845	75%
BauBuche concept	730	133%	59,341	97%	1,193	48%
B&O Holzparkhaus	935	170%	17,547	29%	717	29%

7.2.1 Total Deck Height

The first aspect on which the five car park concepts are compared is the total deck height. This height is defined as the total height difference between two car park levels minus the clearance height. It is the height required for the construction of the parking deck to span the large span of the parking deck. Depending on the structural design of the concept the total deck height might only include the thickness of the floor, but it can also include the height of primary or secondary girders.

In Figure 76, the total deck height for the five car park concepts are presented next to each other. The graph clearly shows that the ModuPark concept outperforms the other concepts. This is due to the characteristic of the ModuPark concept that it uses TT-plates which span the entire parking deck and are supported at the sides. The monolithic connection between the ribs of the plate and the deck result in a very efficient element in terms of its length to height ratio.

The other four concepts all use structural systems in which the parking deck panels are supported by primary (and sometimes secondary) girders. These systems result in larger total deck heights as their elements are mainly stacked on top of each other and the elements often do not act together in a way similar to the ribs and plate of the TT-plates. The total deck height of the Koopman int. car park concept is primarily the result of the height of the primary girders (600 mm), while the thickness of the floor is very limited (40 mm). This results in a total deck height of 640 mm, which is 16% larger than the ModuPark concept. The Quantum Deck floor of the Morspoort car park has a total height comparable to that of the Koopman int. car park of 651 mm, which is 18% thicker than the ModuPark concept. It also uses a primary girder of 600 mm which supports only the concrete part of the floor of 51 mm.

The last two concepts are the timber-concrete BauBuche concept and the fully timber build B&O-Holzparkhaus. The BauBuche concept has a total deck height of 730 mm, which is build-up of a concrete plate with a thickness of 130 mm and a primary girder with a height of 600 mm. The height of the primary girder and as a result the total deck height is limited by the use of the stronger BauBuche material. The total deck height is 33% larger compared to the ModuPark concept. The B&O-Holzparkhaus uses a CLT panel for the floor with a thickness of 100 mm and a primary girder with a height of 760 mm. It is the only deck, which cannot directly be driven on, adding another 75 mm of thickness to the total deck height for asphalt and finishing layers. This results in a much larger total deck height of 70% compared to the ModuPark concept.

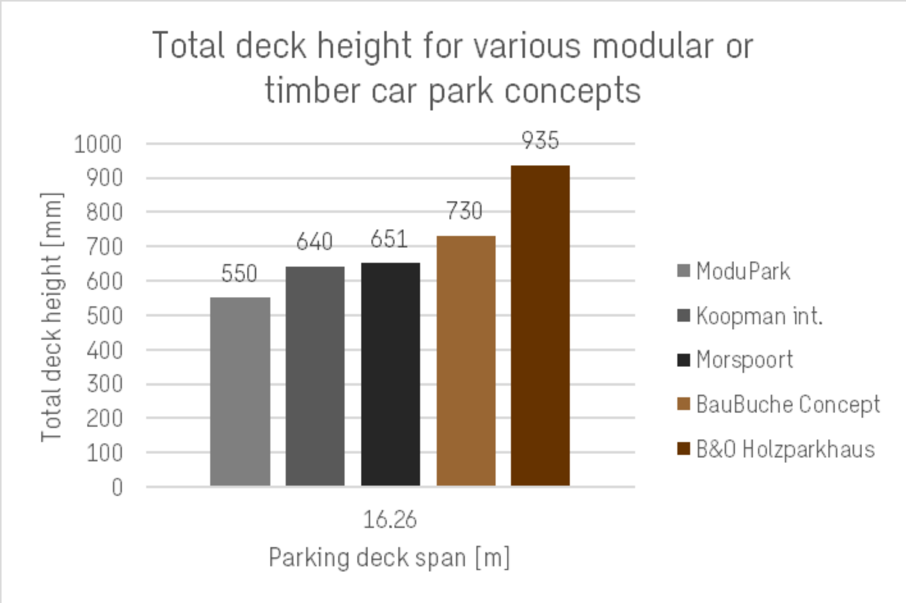


Figure 76. Comparison of total deck height (floor element + supporting girder (if above parking deck)) for various modular or timber car park concepts

7.2.2 Weight per Level

The second aspect on which the five car park concepts are compared is the weight per level. The weight is calculated for one level with a width of 4 parking bays (4 x 2.5 = 10 m) and a length of a parking deck span of 16.26 m (90° parking, 6.00 m aisle). The weight calculation only includes all main load bearing structural elements, present at every grid position of the car park. Local wind bracing, is therefore excluded.

In Figure 77, the weight per level for the five car park concepts is presented next to each other. The graph immediately shows the large differences in weight between the five concepts. The concepts which use concrete floor elements are the heaviest, followed by the concept using a steel-concrete composite floor. The use of timber (CLT) or fibre-reinforced plastics in the floor results in the lightest construction.

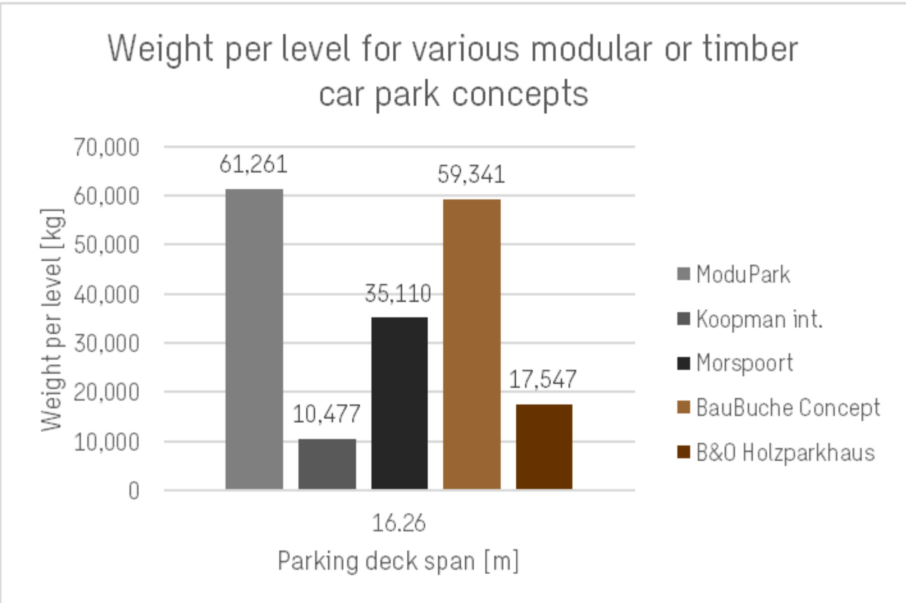


Figure 77. Comparison of weight per level (4x2.50 m bay wide, 16.26 m span) for various modular or timber car park concepts

The ModuPark concept with its heavy concrete TT-plates results in the smallest total deck height, but as a downside also the highest weight of the structure at 61,261 kg. The BauBuche concept also uses concrete floor panels, which in combination with the BauBuche elements (heavier than standard glulam elements) result in a structure which is only 3% lighter (59,341 kg) compared to the ModuPark concept. The use of steel-concrete composite floor elements in the Morspoort car park significantly reduces the weight of the structure to 57% (35,110 kg) of the weight of the ModuPark concept. The fact that timber is a very efficient strength to weight material is shown by the B&O-Holzparkhaus. It has a weight of only 29% of that of the ModuPark concept at 17,547 kg. Finally, steel in combination with the extremely lightweight fibre-reinforced plastic floor elements, the concept of the Koopman int. car park, is the lightest at 10,477 kg which is only 17% of the weight of the ModuPark concept.

7.2.3 Load from Standard Column on Foundation

The third aspect on which the five car park concepts are compared is the load on a “standard” column on the foundation for 4 parking levels. This “standard” column is a column in the centre of the car park, which means it carries the load from two halve parking decks over a width of the distance between the columns, see Figure 78. Essentially, it carries the loads of a standard grid from 4 parking levels. The total load includes the self-weight of the structure, the finishing of the parking decks and the imposed loads on the parking decks.

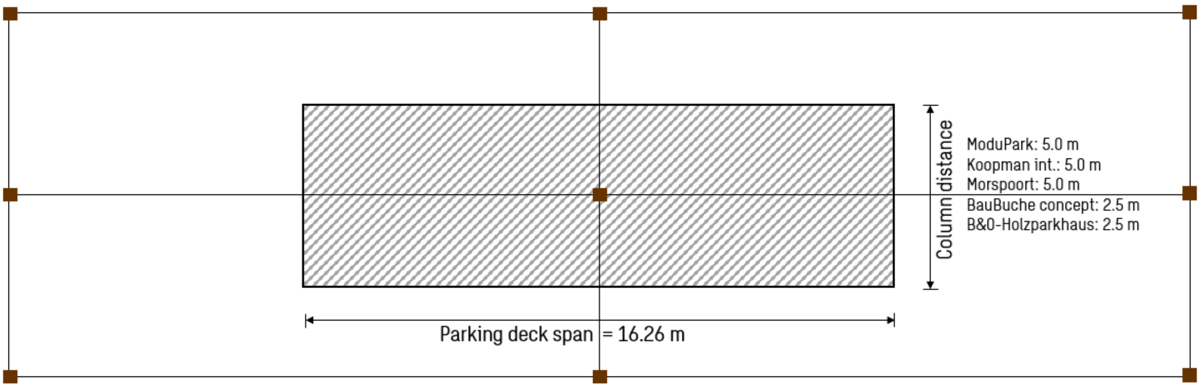


Figure 78. Load area on "standard" column

The loads from a “standard” column on the foundation for 4 levels for the various car park concepts are presented in Figure 79 next to each other. This results in a different view on the comparison between the various variants compared to the weight of these variants. This is the result of different distances between the columns (parking deck span is the same for all variants) for the variants. A different distance between the columns affects many aspects of the construction, such as the dimensions of the cross-sections of girders and thickness of the floors. Therefore, it also affects the total deck height as well as the weight of the structure. The reason that it is important to also consider the load from a “standard” column on the foundation is that these loads in the end have to be supported by the foundation.

The heavy ModuPark concept similar to the total weight, also results in the highest column loads on the foundation at 2,472 kN with 5.0 m centres for 4 levels. Due to the smaller centres distances of 2.5 m for the BauBuche concept, its column load is at 48% of that from the ModuPark concept at 1,193 kN. For the more lightweight concepts, as a result of their lower self-weight, the imposed load becomes a bigger part of the total load on the foundation. The concept of the Morspoort car park, with centres of 5.0 m, results in a load of 1,845 kN (75% of ModuPark concept) of which 53% is due to the imposed load on the parking decks. The concept from the Koopman int. car park has a similar load to the BauBuche concept at 1,258 kN (51% of ModuPark concept), but it centres are at twice the distance (5.0 m), compared to the BauBuche concept. In this case, 78% of the total load comes from the imposed loads. Finally, the B&O-Holzparkhaus concept has the smallest loads on the foundation as a result of its materials and small centres distance of 2.5 m. Its load is 717 kN (29% of ModuPark concept) from which 68% is imposed load.

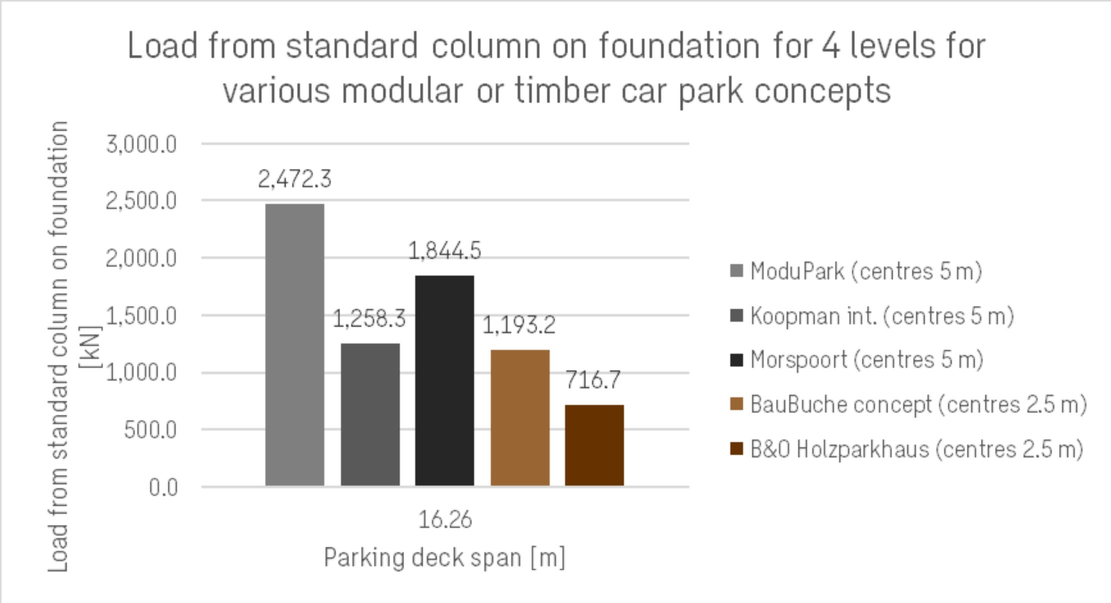


Figure 79. Comparison of load on a standard column on foundation for 4 levels for various modular or timber car park concepts

7.3 Concluding Remarks

In conclusion of this chapter, sub-question 3b can be partially answered: “What level of structural performance is requested from the design for a temporary multi-storey car park?”

It can be stated that the ModuPark concept has the smallest total deck height, but at the same time is the heaviest concept with the largest load of a column on the foundation. The total deck heights of other modular systems lie around 650 mm. Using primarily timber as building material results in a significant increase in total deck height. For structural systems in timber to compete in terms of total deck height with other modular systems, they should approach a total deck height around 650 mm. Therefore, a goal is set to limit the total deck height for the structural systems to be developed in this thesis to **700 mm** for a span of 16.26 m.

Considering the weight of the structure, the concepts discussed in this chapter have shown that the use of concrete in floors of the structure drastically increase the total weight. In order to reduce the weight of the structure and loads on the foundation, concrete should not be used in the floors. It is clear that a timber structure is much lighter compared to concepts in steel and concrete. Only the concept in steel and fibre-reinforced plastics performs better. In chapter 8, possible foundation methods for the car park structure are studied and a range of preferred maximum loads from the columns on the foundation can be presented.

8

Foundation

Like any structure, a car park requires a proper foundation. Since the objective is to design a multi-storey car park with a temporary and modular character, this comes with several challenges. It is preferred to create a foundation that can be disassembled and reused. Furthermore, the car park can be located anywhere in The Netherlands, which creates a challenge for the design of the foundation, since it is typically designed for a specific location. In this chapter, first the different soil compositions in The Netherlands are discussed. Next, two types of foundations are discussed: a shallow foundation and a pile foundation. As a result, the advantages and disadvantages of both types of foundations are presented as well as an estimate of what range of loads can be carried by a certain type of foundation. This helps to create feeling for what the performance of the proof of concept should be, concerned with weight minimization as discussed in paragraph 5.1.2.2.

8.1 Soil Composition

The car park is not designed for a specific location, but should be applicable to a wide range of different locations and shapes. For this reason, the structure is designed in a modular and dismountable way. As the type of soil differs for each location, a set of boundary conditions should be established, for which the car park and its foundation can be applied in most locations. To establish these boundary conditions, the soil composition of twelve different locations in The Netherlands are analysed, see Figure 80. These locations are based on the general types of soil structures, found in The Netherlands.

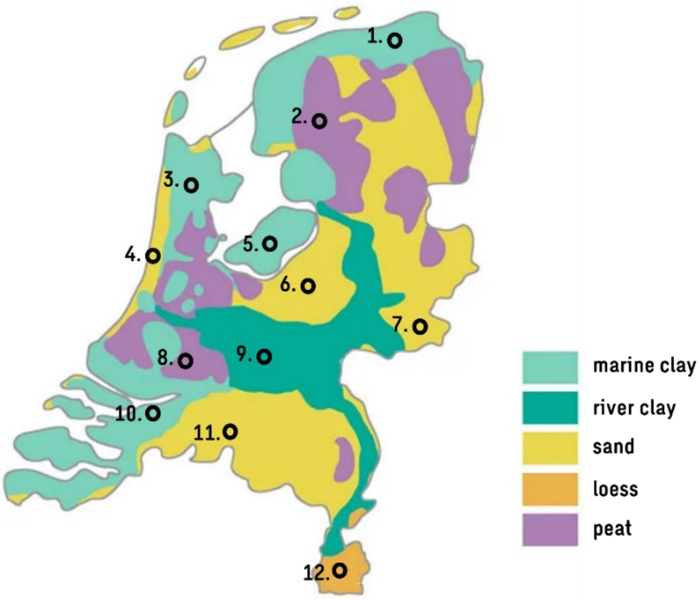


Figure 80. Main soil types in The Netherlands (Florum, 2020)

The main types of soil found near the surface are marine clay, river clay, sand, loess and peat (Florum, 2020). Of these different types of soil, sand is the only one strong enough to build a foundation on. Whether it is possible to use a shallow foundation depends on the depth of the sand layer relative to the ground level. Through soil investigation, the build-up of the different soil layers can be found. Based on the soil data found in DINOloket (2022), the soil composition for the different selected locations from

Figure 80 can be established, see Figure 81. From this figure it can be deduced, that in the areas with sand soils, a shallow foundation can be applied. For the other areas, sometimes a sand layer can be found relatively close to ground level, in which case it is also possible to apply a shallow foundation after excavating the top layer. In locations, where the sand layer lies at larger depths, a pile foundation is required.

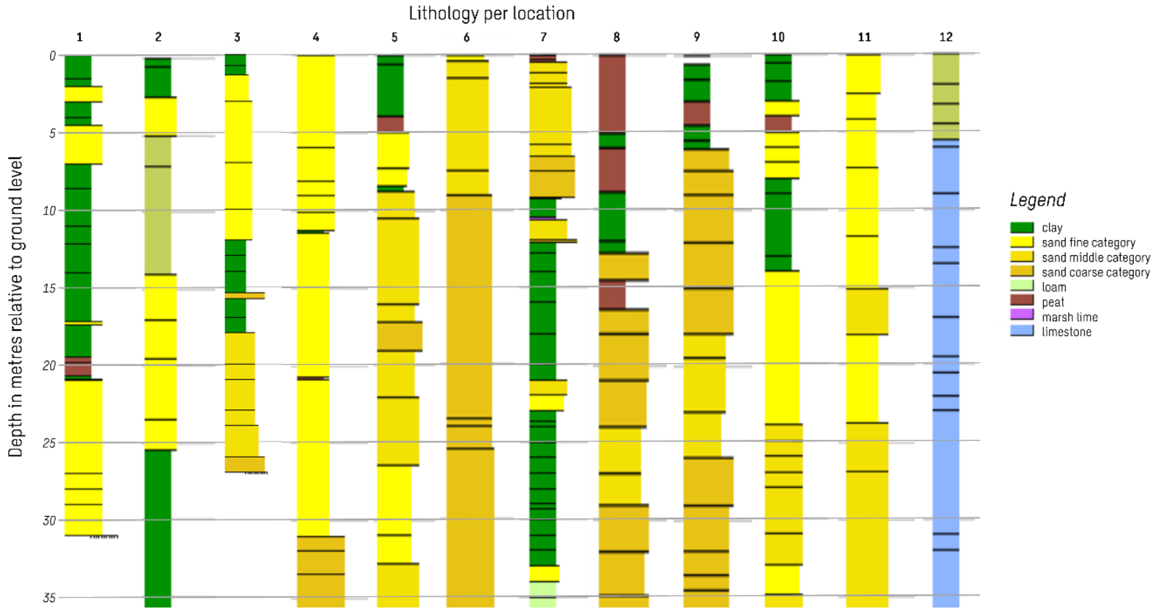


Figure 81. Soil composition per location (DINOloket, 2022)

The groundwater level in The Netherlands is several decimetres below ground level in most places. However, in some locations like the Veluwe, it can lie up to several meters below ground level (Ons Water, 2020). The depth of the groundwater level affects the load carrying capacity of a foundation. The higher the groundwater level, the lower the load carrying capacity of the soil, as result of a reduction of the effective stress (J.L. Bijnagte, A.F. Van Tol, & R. Elprama, 2006). Therefore, when establishing the boundary conditions for the load carrying capacity of the foundation, the level of the groundwater should be taken into account. Figure 82 shows the average smallest depth of the groundwater table for The Netherlands.

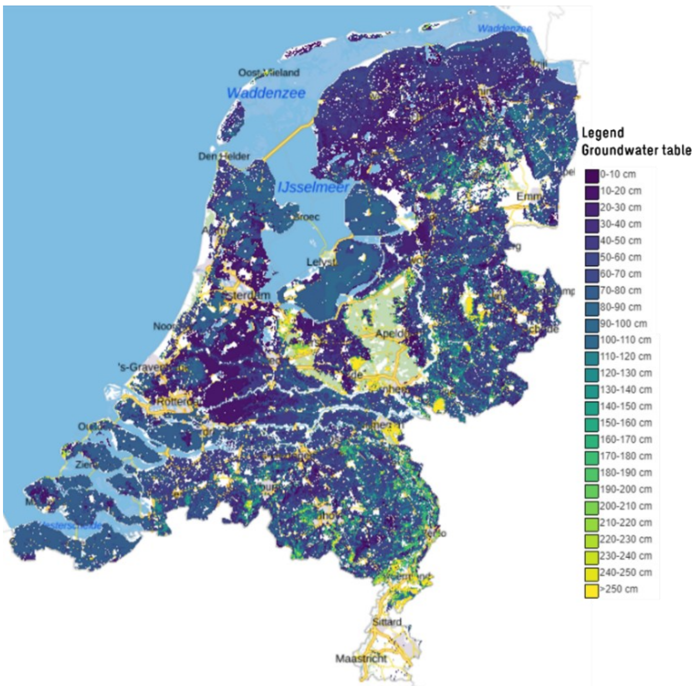


Figure 82. Groundwater table, average smallest depth 2021(BROloket, 2021)

8.2 Shallow Foundation

A shallow foundation can be applied in case a sand layer is found close to ground level (up to 2.0 m). It normally consist out of a concrete strip footing or isolated footing, which spreads the loads coming from the structure over the sand layer. The top layer of the soil is excavated to place the foundation directly on the sand layer and below the frostline in the soil. A shallow foundation is relatively cheap and causes little nuisance for the surrounding area (Bouwkosten Online, 2022). Furthermore, it can easily be removed when disassembling the construction. Below, a range concerning the load that can be carried by a shallow foundation for a single column of the car park is established.

8.2.1 Assumptions

To determine boundary conditions for the load carrying capacity of a shallow foundation, the following assumptions are made:

Footing

- The loads from a single column are supported by an isolated footing as type of shallow foundation;
- The footing is made from reinforced concrete;
- The footing is prefabricated;
- Dimensions footing: 2000 x 2000 x 200 mm (length x width x height);
- Dimensions concrete column on footing: 300 x 300 x 900 mm (length x width x height); (Column 300 mm above ground level to prevent moisture from deteriorating the timber columns)
- Weight concrete: 25.0 kN/m^3;
- Two construction depths for the bottom face of the footing are studied: 800 mm and 2,000 mm below ground level.

Soil Profile

- Load carrying capacity of the shallow foundation is determined for the 12 different locations of Figure 80;
- No soil improvement techniques are considered;
- Groundwater level is at 200 mm below ground level for all locations;
- The footings are covered with loose sand till ground level.

Other

- Only vertical loads are considered.

8.2.2 Load Carrying Capacity

To determine the load carrying capacity of a shallow foundation on the different selected locations, cone penetration tests (CPT's) from the selected location are used, see appendix D.1. Based on the soil properties from the CPT's and given the assumptions discussed above, the load carrying capacity of the soil is calculated using Technosoft, see Table 23 and Table 24 (detailed output calculations in appendix D.2). The characteristic allowable load on a single column of the car park is found after subtracting the weight of the concrete footing, column and soil cover and taking into account the partial load factors. The load carrying capacity of the soil is calculated for two construction depths: 800 mm and 2,000 mm below ground level. Since the soil has different layers, this can result in significantly different load carrying capacities. Deformations are calculated for two different characteristic loads from a single column of the car park: 350 and 1,000 kN.

Table 23. Load carrying capacity soil for different locations at construction depth -0.80 m

Location	**Undrained**	**Drained**	**Punch**	**Minimum**	**Weight footing and cover**	**Allowed load car park**	**Deformation (for F_k = 350 kN)**	**Deformation (for F_k = 1,000 kN)**
-0.80 m	R_d [kN]	R_d [kN]	R_d [kN]	R_d [kN]	F_d [kN]	F_k [kN]	[mm]	[mm]
1	*3848*	*499*	*668*	499	74.3	**353.9**	230.4	342.1
2	*3686*	*306*	*339*	306	74.3	**193.1**	614.7	931.4
3	*944*	*412*	*444*	412	74.3	**281.4**	302.8	426.4
4		*1426*	*13419*	1426	74.3	**1126.4**	5.4	**8.9**
5	*3750*	*520*	*586*	520	74.3	**371.4**	438.7	647.9
6		*2066*		2066	74.3	**1659.8**	3.3	**6.6**
7		*1812*	*6969*	1812	74.3	**1448.1**	4.1	**7.9**
8	*1492*	*319*	*518*	319	74.3	**203.9**	627.0	1073.1
9	*3740*	*496*	*587*	496	74.3	**351.4**	569.0	881.6
10	*3686*	*592*	*660*	592	74.3	**431.4**	397.3	585.3
11		*1521*	*4262*	1521	74.3	**1205.6**	4.7	**9.2**
12	*3686*	*820*	*1759*	820	74.3	**621.4**	92.4	**138.1**
	= large sand layer near ground level					*= large loam layer near ground level*		

Table 24. Load carrying capacity soil for different locations at construction depth -2.00 m

Location	**Undrained**	**Drained**	**Punch**	**Minimum**	**Weight footing and cover**	**Allowed load car park**	**Deformation (for F_k = 350 kN)**	**Deformation (for F_k = 1,000 kN)**
-2.00 m	R_d [kN]	R_d [kN]	R_d [kN]	R_d [kN]	F_d [kN]	F_k [kN]	[mm]	[mm]
1	*979*	*969*	*1116*	969	144.4	**687.2**	**123.3**	178.6
2	*430*	*385*	*462*	385	144.4	**200.5**	490.3	696.2
3	*430*	*870*	*1017*	430	144.4	**238.0**	**142.0**	196.7
4		*2064*	*9566*	2064	144.4	**1599.7**	5.1	**8.7**
5	*979*	*386*	*475*	386	144.4	**201.4**	490.4	689.4
6		*3002*		3002	144.4	**2381.4**	2.7	**6.2**
7		*2429*	*5010*	2429	144.4	**1903.9**	4.7	**8.8**
8	*979*	*467*	*616*	467	144.4	**268.9**	549.1	985.8
9	*796*	*457*	*591*	457	144.4	**260.5**	648.0	899.3
10	*979*	*498*	*776*	498	144.4	**294.7**	444.3	630.2
11		*1794*	*2995*	1794	144.4	**1374.7**	6.3	**11.5**
12	*1893*	*1315*	*1488*	1315	144.4	**975.5**	67.0	**117.0**
	= large sand layer near ground level					*= large loam layer near ground level*		

From the results it becomes clear that at locations with sand layers near the surface (locations 4, 6, 7 & 11), the soil has significant load carrying capacity and loads result in only very small deformations (< 10 mm). For shallow foundations constructed at a depth of 0.80 m, the soil can easily support a load of 1,000 kN from a column of a car park and even over 1,600 kN, depending on the location. At a construction depth of 2.00 m, even larger loads can be supported in the range of 1,300 kN upwards to 2,300 kN. In the far south, the soil structure consists of a layer of loam on top of lime stone. These soils can also carry a significant load from a shallow foundation in the range of 600 kN for a construction depth of 0.80 m and close to 1,000 kN for a construction depth of 2.00 m. Deformations in this case are much larger compared to the sand layers, however for the allowable loads they are still smaller than 150 mm and therefore allowable. In terms of horizontal loading, for locations with sand layers near the surface, horizontal loads of up to more than 100 kN can be resisted (see Appendix D3).

The other types of soil structures have much smaller load carrying capacities compared to the sand layers. Furthermore, even smaller loads of 350 kN will result in large deformations (> 150 mm). as a result, these soil structures are unsuitable for a shallow foundation for a car park.

8.3 Pile Foundation

In case a shallow foundation does not generate enough load bearing capacity or results in too large deformations, a pile foundation is used. In this case, one or multiple piles are driven into the ground to reach into a deeper sand layer, which is strong enough to support the load on the pile. The load carrying capacity of a pile foundation comes from a combination of end bearing support and support by skin friction (Vroom Funderingstechnieken, 2019). Pile foundations are in general more expensive compared to a shallow foundation, can give a lot of nuisance for the surroundings and are often difficult to remove and reuse (Bouwkosten Online, 2022).

8.3.1 Types of Pile Foundations

Many different types of pile foundations exist. For this thesis, only timber driven piles and hammer driven prefab concrete piles are considered. The differences between both types in terms of advantages and disadvantages are discussed below.

Timber Foundation Piles

Advantages (Abebe & Smith, 2016; Civil Today, 2019)

- Sustainable material;
- Relatively cheap;
- Installation is easy;
- Low possibility of damage;
- Easy to pull out;
- Easy to trim to required length.

Disadvantages (Abebe & Smith, 2016; Civil Today, 2019)

- Length limited to ~21 m;
- Diameter limited to standard of 160 mm, although bigger diameters are possible;
- Not suitable to be used as end-bearing piles;
- Limited load bearing capacity (up to several hundreds of kN);
- Prone to biological attacks (rot).

Prefab Concrete Foundation Piles

Advantages (Abebe & Smith, 2016; Civil Today, 2019)

- High resistance against chemical and biological cracks;
- High load bearing capacity (up to several thousands of kN);
- Large lengths available (up to 39 m);
- Standard dimensions varying up to 500 x 500 mm.

Disadvantages (Abebe & Smith, 2016; Civil Today, 2019)

- Requires heavy machinery for installation;
- Installation by hammering causes a lot of nuisance for the surroundings;
- Possibility of breakage or damage during handling and driving of piles;
- Difficult to pull out;
- Relatively difficult to cut.

Considering the aim of this thesis to develop a structural system for a temporary and modular multi-storey car park, like for the structural material for the superstructure, timber as a sustainable material is also the preferred option for the substructure. Furthermore, the characteristic that they are easy to pull out, makes them even more suited, as after the car park is disassembled, the building site should be returned to its original state. Because of the temporary character of the car park, rot of the timber piles is less of an issue, as long as they are thick enough to withstand their loads after some rot has occurred.

The limitations of timber piles concerning available lengths, diameters and load bearing capacity mean they cannot always be applied or require installation of more piles compared to prefab concrete piles. Whether timber piles are suitable largely depends on the build-up of the soil layers of a building site. Timber piles are most suitable as friction piles and therefore are most suitable to be used in soils which can generate sufficient cohesion between soil and pile (Abebe & Smith, 2016).

To determine a range of loads that can be supported by a timber pile foundation for a column of the car park, the maximum load bearing capacities of a timber foundation pile for the various locations of Figure 80 are calculated. Two different dimensions are used and as reference, also maximum loads for a prefab concrete pile are calculated.

8.3.2 Assumptions

To determine boundary conditions for the load carrying capacity of a pile foundation, the following assumptions are made:

Timber Pile

- Single pile load bearing capacity;
- Circular pile head;
- Diameter 160 mm or 200 mm;
- Maximum length 21 m;
- No concrete extension applied;
- Structural class C16.

Prefab Concrete Pile

- Single pile load bearing capacity;
- Square pile head;
- 400 x 400 mm;
- Maximum length 30 m;
- Structural class C35/45.

Soil Profile

- Load carrying capacities of the different pile foundations are determined based on CPT's for the 12 different locations of Figure 80;
- Pile tip at depth generating highest load bearing capacity (within maximum length of pile);
- Negative skin friction trajectory over compressible layers until sand layer in which pile head reaches;
- Positive skin friction trajectory over sand layer in which pile head reaches;
- Groundwater level is at 200 mm below ground level for all locations;
- A distributed load of 10.0 kN/m^2 is added to the surface to take into account the load from the lower parking level (8 cm paving stones, 10 cm sand, 20 cm rubble, variable load from vehicles).

Other

- Only vertical loads are considered.

8.3.3 Load Carrying Capacity

To determine the load carrying capacities for different pile foundations on the different selected locations, cone penetration tests (CPT's) from the selected location are used (presented in appendix D.1). Based on the soil properties from the CPT's and given the assumptions discussed above, including the skin friction trajectories, the load carrying capacity of the soil is calculated using Technosoft, see Table 25, Table 26, and Table 27 (detailed output calculations in appendix D.3). The geotechnical load bearing capacity as well as the structural load bearing capacity, based on the material assumptions given above, are calculated. The minimum value of both is prescriptive and after taking into account the partial load factors, the allowable load from a column of the car park given an single foundation pile are found.

Table 25. Load carrying capacity timber foundation pile (Ø160 mm) for different locations

Timber Ø160 mm								
Location	Depth pile tip	Share in total bearing capacity: End bearing	Share in total bearing capacity: Pos. skin friction	Reduction bearing capacity by neg. skin friction	Geotechnical bearing capacity	Structural bearing capacity	Minimum bearing capacity	Allowed load car park
	[m]	[%]	[%]	[%]	$R_{c;netto;d}$ [kN]	$R_{s;d}$ [kN]	R_d [kN]	F_k [kN]
1	-4.0	100%	0%	78%	5	263	5	**4**
2	-20.7	65%	35%	67%	91	263	91	**76**
3	-5.9	52%	48%	10%	85	263	85	**71**
4	-8.0	66%	34%	0%	257	263	257	**214**
5	-14.3	37%	63%	5%	457	263	263	**219**
6	-19.3	16%	84%	0%	659	263	263	**219**
7	-14.5	31%	69%	0%	488	263	263	**219**
8	-2.0	100%	0%	262%	-3	263	-3	**-3**
9	-15.7	39%	61%	11%	288	263	263	**219**
10	-20.1	53%	47%	49%	160	263	160	**134**
11	-14.2	26%	74%	0%	326	263	263	**219**
12	-9.3	42%	58%	14%	92	263	92	**76**

= soils with large compressible layers in first 20 m

= soils with compressible layers, but strong enough load carrying sand layer in first 20 m

Table 26. Load carrying capacity timber foundation pile (Ø200 mm) for different locations

Timber Ø200 mm								
Location	Depth pile tip	Share in total bearing capacity: End bearing	Share in total bearing capacity: Pos. skin friction	Reduction bearing capacity by neg. skin friction	Geotechnical bearing capacity	Structural bearing capacity	Minimum bearing capacity	Allowed load car park
	[m]	[%]	[%]	[%]	$R_{c;netto;d}$ [kN]	$R_{s;d}$ [kN]	R_d [kN]	F_k [kN]
1	-4.0	100%	0%	66%	11	411	11	**9**
2	-20.6	70%	30%	58%	171	411	171	**142**
3	-5.7	57%	43%	10%	111	411	111	**93**
4	-7.8	70%	30%	0%	374	411	374	**312**
5	-14.1	44%	56%	5%	616	411	411	**342**
6	-19.1	20%	80%	0%	850	411	411	**342**
7	-14.3	33%	67%	0%	611	411	411	**342**
8	-2.0	100%	0%	214%	-4	411	-4	**-3**
9	-15.5	45%	55%	10%	385	411	385	**321**
10	-19.9	60%	40%	45%	240	411	240	**200**
11	-14.0	29%	71%	0%	419	411	411	**342**
12	-9.1	48%	52%	13%	122	411	122	**102**

= soils with large compressible layers in first 20 m

= soils with compressible layers, but strong enough load carrying sand layer in first 20 m

Table 27. Load carrying capacity concrete foundation pile (#400 mm) for different locations

Concrete #400								
Location	Depth pile tip	Share in total bearing capacity		Reduction bearing capacity by neg. skin friction	Geotechnical bearing capacity	Structural bearing capacity	Minimum bearing capacity	Allowed load car park
		End bearing	Pos. skin friction					
	[m]	[%]	[%]	[%]	$R_{c;netto;d}$ [kN]	$R_{s;d}$ [kN]	R_d [kN]	F_k [kN]
1	-28.2	61%	39%	58%	341	3728	341	**284**
2	-27.1	58%	42%	24%	1884	3728	1884	**1570**
3	-6.1	56%	44%	8%	327	3728	327	**273**
4	-7.1	83%	17%	0%	1703	3728	1703	**1419**
5	-13.2	56%	44%	5%	1742	3728	1742	**1451**
6	-16.0	44%	56%	0%	2597	3728	2597	**2164**
7	-13.4	37%	63%	0%	1543	3728	1543	**1286**
8	-26.8	57%	43%	40%	866	3728	866	**721**
9	-14.6	60%	40%	9%	1182	3728	1182	**985**
10	-23.7	42%	58%	37%	865	3728	865	**721**
11	-13.1	43%	57%	0%	1178	3728	1178	**981**
12	-9.3	51%	49%	12%	355	3728	355	**296**

= soils with large compressible layers in first 20 m (light shading)

= soils with compressible layers, but strong enough load carrying sand layer in first 20 m (dark shading)

For the bearing capacity of pile foundations, the locations with compressible soil layers are of relevance (highlighted in tables). From the results it becomes clear that the load bearing capacity of a single timber pile is rather limited. Using prefab concrete piles, which can have a larger cross-section and can be installed at larger depths, can result in much higher load bearing capacities.

Since the range of load bearing capacities of timber piles for the different locations with compressible soil layers is rather broad (0 to 411 kN), it is difficult to give a number for the load coming from a column of the car park that can be supported by a timber pile foundation. To gain insight into the build-up of the bearing capacity of a pile, the percentages of bearing capacity realised by end bearing and positive friction are given. Also, a percentage is given which expresses how much the total bearing capacity of a pile based on end bearing and positive skin bearing capacity is reduced by the negative skin friction. The results show the build-up of the bearing capacity is highly dependent on the structure of the soil layers and based on these limited number of locations, it is difficult to give a well-founded conclusion on how the bearing capacity in general is build-up. What can be concluded is that timber piles are not suitable for soil structures with mainly compressible soil layers within the first 20 metres (light shading in tables).

For soil structures with bigger sand layers within the first 20 metres (dark shading in tables), a minimum load carrying capacity of 140 kN for a timber pile is possible. When a pile group is used, the load carrying capacity might increase as a result of soil compaction. To carry a load from a column of the car park of 1,000 kN would require roughly 7 timber piles. Depending on the soil structure of the location it might be less. For locations, in which timber piles are not suitable, concrete piles should be used.

8.4 Conclusion

In conclusion of this chapter, the remaining part of sub-question 3b, related to the performance on weight minimization can be answered: “What level of structural performance is requested from the design for a temporary multi-storey car park?”.

Based on the results presented above, it is concluded that a load of a column from the car park of **1,000 kN** can be supported by the following foundations:

- A shallow foundation in case large sand layers are found near the surface and no large compressible soil layers are found close to this sand layer;
- A timber pile foundation with up to 7 piles in case within the first 20 m, a sand layer with large thickness is found;
- A prefab concrete pile foundation with up to 4 piles in case within the first 20 m, only sand layers of insufficient thickness can be found to support timber piles.

Depending on the exact location of the car park, a smaller or simpler foundation might also be possible. For a shallow foundation, the range of loads that can be carried by the soil varies between roughly 600 up to over 2,300 kN, depending on location and depth of the foundation. At most locations, a load of 1,000 kN can easily be carried by the soil. For pile foundations, at halve of the location studied, a load of 200 kN can be carried by a single timber foundation pile, which would result in a total of 5 piles to carry a load of 1,000 kN. At other locations, more piles or concrete piles are required.

PART IV: Variant Analysis

9 Deck System

In the literature study, various types of timber deck systems have been studied, see paragraph 4.2.2. In this chapter, these deck systems are further studied in detail, to assess their performance on height and weight if to be used to construct the parking decks of the multi-storey car park. Furthermore, the systems are studied on how they perform when different requirements are used considering the ultimate and serviceability limit states. The deck systems are divided into long-span and short-span systems, studied after each other. Finally, this chapter results in a choice for a certain deck system for each design variant as presented in 5.3.2.

9.1 Long-span Prefab Deck System

The only design variant considered to have a long-span deck system is variant 1 (see paragraph 5.3.2.1). Depending on the chosen span of the parking deck, it requires a deck system to span between 12.18 and 17.66 m. To bridge such a long span, several possible prefab timber deck systems can be used, as presented in paragraph 5.3.1.1. These are CLT or LVL decks with sometimes glulam or LVL ribs: CLT panel, CLT open rib panel, CLT closed rib panel, LVL panel, LVL open rib panel, LVL semi-open rib panel and LVL closed rib panel. These seven long-span deck systems are discussed in the following paragraphs, in which the required height of the deck is determined for a base case, in which all standard rules and norms apply and various other cases, for which one or multiple requirements are altered. Finally, the results will provide insight into the performance of the different long-span deck systems considering their weight and height as well as what effect changing parameters and/or requirements have on those aspects.

9.1.1 General

For design variant 1, floor slabs of 2500 mm wide are considered, spanning the entire parking deck and simply supported at both ends, see Figure 83. Concerning the loads on the floors, the top level parking deck is considered as it is normative, which results in the following loads acting on the deck: self-weight, permanent load from parking deck coating, imposed load, snow load and wind load. The values of the loads are mentioned in the starting points in chapter 6 and in Table 28. For the wind load, the downward action is governing in the load combinations, for which the coefficient for downwards wind loading is used for the entire parking deck.

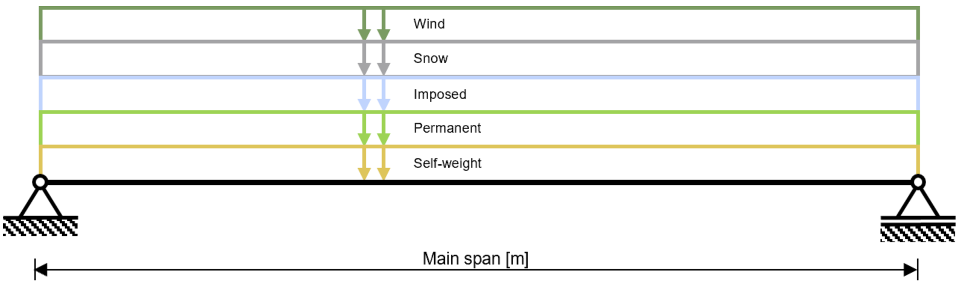


Figure 83. Schematisation long-span deck system

Table 28. Load values long-span deck system

Load	Value
Self-weight	Differs per type and span
Permanent	0.1 kN/m^2
Imposed	2.0 kN/m^2
Snow	0.56 kN/m^2
Wind	0.36 kN/m^2

All of the considered long-span deck systems are studied for their required thickness for a different number of span lengths. From the required deck thickness also follows the weight of the deck. First, the decks are studied for the basic requirements for ULS, SLS, fire and vibrations as stated in chapter 6. Next, the requirements concerning SLS, fire and vibrations are altered in accordance with the overview of parameters in paragraph 0. Finally, the different deck systems are compared to each other.

For the long-span deck systems, the standard CLT and LVL products of manufacturer Stora Enso are considered (Stora Enso, 2022b, 2022e). The calculations are made using the Calculatis software package by Stora Enso (Stora Enso, 2022a). Examples of calculations can be found in Appendix F.

9.1.2 CLT Panel

The first studied system makes use of a single CLT panel, which forms the deck, see Figure 84. Based on calculations on the standard CLT panels of Stora Enso (Stora Enso, 2022e), it is concluded that for the base case, even for the smallest span of 12.18 m, the largest standard CLT panel (CLT 320 L8s-2) does not fulfil the ULS requirements (see appendix F.1). Only with reduced SLS requirements and no check on vibrations, would the largest panel be sufficient, but only for a span of 12.18 m. It is therefore concluded, that a single solid CLT panel is not an option for the long-span prefab deck system.

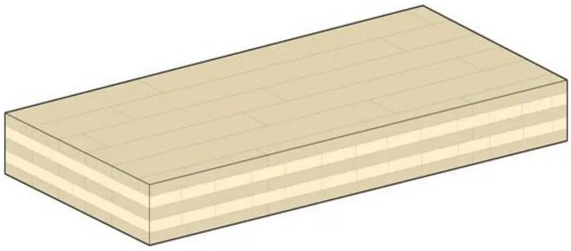


Figure 84. CLT panel (Stora Enso, 2022f).

9.1.3 CLT Open Rib Panel

The CLT open rib panel consist out of a CLT panel with several glulam ribs glued to this panel, resulting in a ribbed deck with a full shear connection between rib and panel, see Figure 85. This system is somewhat similar to the TT-slabs in concrete, but in a timber variant.

A CLT open rib panel can be realised in many different configurations. Different CLT panels (thickness and direction of layers) can be used, different cross-sections for the ribs as well as its material quality can be chosen and the distance between the ribs can be varied. Since this results in an extremely large number of possible configurations, this design space has to be narrowed down. Based on the product advice of Stora Enso (Stora Enso, 2022f), given a fire resistance class R90, a panel thickness of 120 mm is chosen, a rib width of 200 mm and a rib spacing of 625 mm. The height of the rib follows from a load calculation for the different parking deck spans. The parameters advised by Stora Enso are a trade-off between minimizing the total deck height and minimizing the weight while fulfilling the fire-safety requirements. This has resulted in the advised parameters.

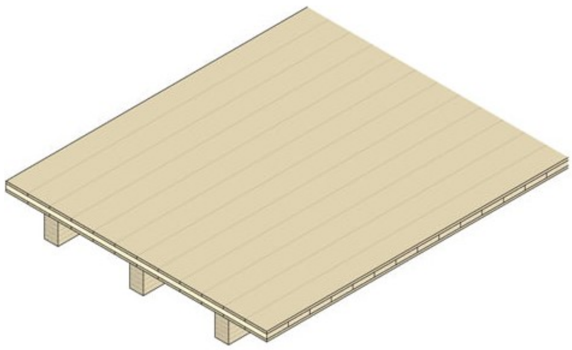


Figure 85. CLT open rib panel (Stora Enso, 2022f).

9.1.3.1 Base Case

Depending on the span distance of the parking deck, the required rib height of the CLT open rib panel varies between 360 and 800 mm and the total deck height between 480 and 920 mm, see Table 29. In this table, for each specific parking deck span, the required thickness of the ribs of the deck for each requirement (ULS, ULS fire, SLS & vibrations, see chapter 6) is presented. Also the self-weight of the panels is shown for the governing requirement. Both the total deck height and self-weight for a 2500 mm wide deck element are plotted against the parking deck span in Figure 86. The graph shows a slightly increasing exponential increase in deck height and weight for increasing spans. Depending on the specific parking deck span, a certain rib height might be just sufficient (UC almost 1) or might have some margin (0.8 < UC < 1.0). As a result, the parking deck spans of 16.26 m and 17.16 m require the same CLT open rib panel.

From Table 29 it can be clearly seen that the vibrational criteria are governing for all spans. Therefore this requirement results in an increase of the deck height and weight compared to the other requirements.

Table 29. CLT open rib panels for various span distances and the required rib height for each requirement.

CLT OPEN RIB PANEL				BASE CASE							
L	Spacing	$CLT_{Top\ panel}$	w_{Rib}	h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height			
[m]	[mm]	[mm]	[mm]	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	625	120	200	360	480	1.18	2.94	<200	<200	300	360
13.70	625	120	200	450	570	1.32	3.30	<200	<200	360	450
14.56	625	120	200	500	620	1.40	3.50	<200	<200	400	500
15.61	625	120	200	600	720	1.56	3.90	200	200	450	600
16.26	625	120	200	700	820	1.72	4.30	240	240	500	700
17.16	625	120	200	700	820	1.72	4.30	240	240	500	700
17.66	625	120	200	800	920	1.88	4.70	300	300	600	800

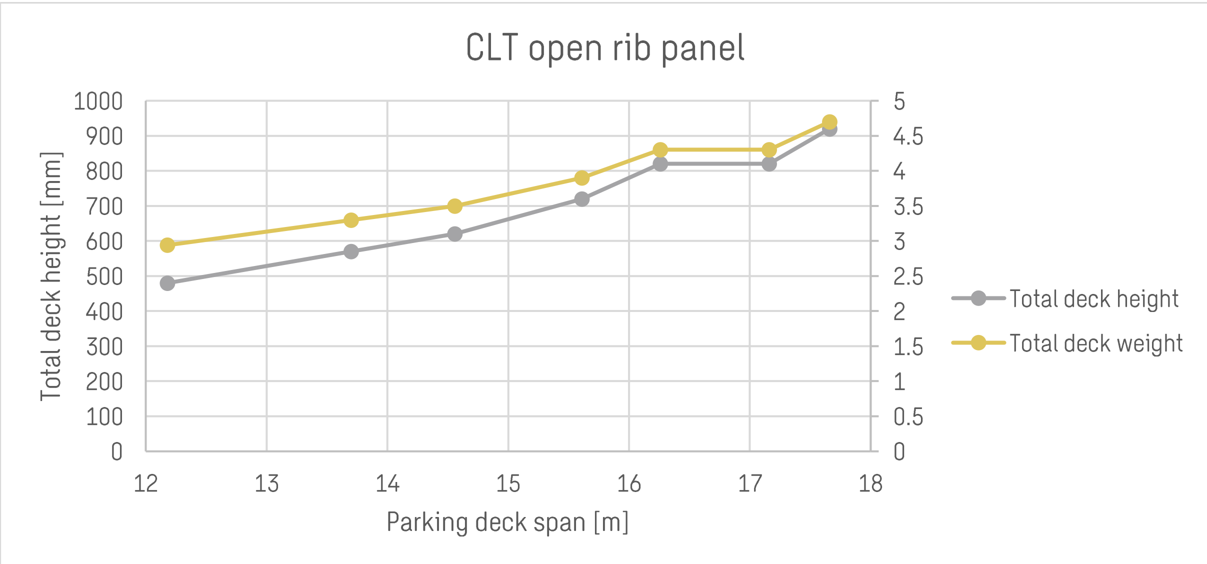


Figure 86. Total deck height for CLT open rib panels for various spans.

9.1.3.2 Alternative Requirements

In previous paragraph, it was already stated that the vibrational requirement is governing for all span distances and therefore altering of this requirement can result in significant reduction of the total deck height and weight. Table 29 already showed the different values, from which can be seen that the differences in rib height for the various requirements differ significantly.

Since serviceability limits are not required by the Dutch Building Degree, they can be altered by the engineer (Ministerie van Binnenlandse Zaken en Koninkrijkrelaties, 2021). The fire resistance class can be reduced to R 30 if the entire car park is considered as a single fire-compartment. This is possible based on the equivalence principle, which can be substantiated by special fire-safety calculations like zone-models as was discussed in paragraph 2.3.4. Also deformation limits can be altered, allowing bigger deformation than advised by the Eurocode. This is allowed by the Dutch Building Degree as discussed in paragraph 6.4. Last of all, also setting limitations related to vibrational behaviour of the structure of a car park is up to the designer as discussed in paragraph 2.3.3. In this paragraph, it is studied what are the effects of reduced requirements for fire-safety, SLS and vibrations on the required thickness of the CLT decks. One or more of these requirements are altered, while the other requirements remain the same (equivalent to the base case requirements).

In Figure 87, the effects of different alternative requirements on the total deck height of the CLT open rib panels can be seen for the various parking deck spans. The results that are at the basis of these figures in tabular format as well as an example calculation can be found in Appendix F.

Reduced Fire-safety Requirements

A reduction of the fire-safety requirements from class R90 to R30 does not result in any reduction of total deck height or weight, since the normal ULS strength requirements require the same rib heights as the R90 ULS fire strength requirements.

No check on vibrations

Since the requirements for vibrations are governing, removing the check on vibrations significantly reduces the total deck height and weight. In this case, the SLS deformation limits become governing. This results in a reduction of total deck height of 60 mm (13%) for the smallest span up to 200 mm (24%) for the spans of 16.26 and 17.16 m. The percentual decrease in weight is slightly smaller; 0.24 kN/m (8%) for the span of 12.18 m and 0.80 kN/m for the three largest spans (19% & 17%).

Alternative Serviceability Limit Requirements

As the SLS deformation criteria are governing after the vibrational criteria, altering these requirements can result in a further decrease of total deck height and weight. A further reduction of 60 mm (14%) in height for a span of 12.18 m and 200 mm (28%) for a span of 17.66 m is possible. In terms of weight reduction these numbers are 0.24 kN/m (9%) for a 12.18 m span and 0.80 kN/m (21%) for a 17.66 m span.

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

As already stated before, reduced fire-safety requirements do not result in a decrease in total deck height and weight.

Conclusion

The results presented in this paragraph clearly show the significant effects of alteration of various requirements concerned with serviceability limit states. For all spans, removing the vibrations check already significantly improves the performance of the decks in terms of height and weight. Since rib decks are used, the percentual effects are bigger for the total deck height compared to the weight. Further reducing the SLS requirements can result in a total reduction of height of 120 mm (25%) up to 400 mm (43%) for the largest span and 0.48 kN/m (16%) up to 1.60 kN/m (34%) respectively.

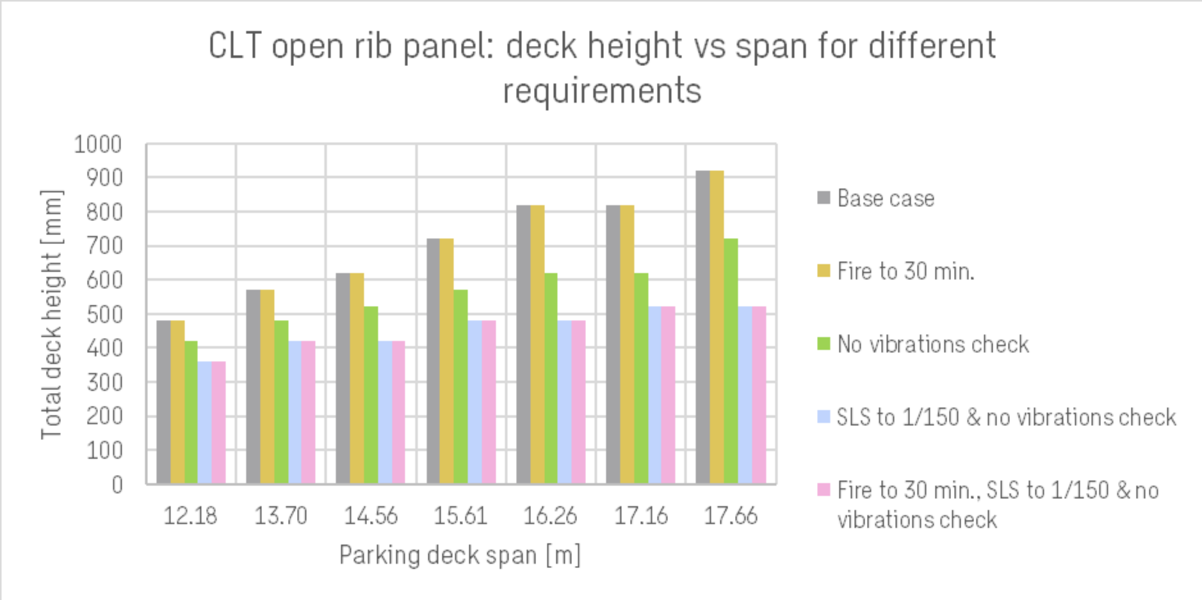


Figure 87. Differences in total deck height for various spans given alternative requirements for CLT open rib panel.

9.1.4 CLT Closed Rib Panel

The CLT closed rib panel consist out of a top CLT panel, bottom CLT panel and several glulam ribs glued to these panels, resulting in a closed rib deck with a full shear connection between ribs and panels, see Figure 88. This system is also often called a box floor.

Closed CLT rib panels can be realised in many different configurations, similar to the open CLT rib panels. Stora Enso does not advice use of specific configurations of the closed rib panels. Therefore, several parameters for the configuration of the closed CLT rib panels are based on the configuration of the open CLT rib panel. For both the top and bottom CLT panel, a thickness of 120 mm is chosen given the R90 fire resistance class. For the ribs, again a spacing of 625 mm is used, but the width of the ribs is reduced to 140 mm, since they are covered by CLT panels on both sides and therefore are protected from fire. The required rib height again follows from calculation for the different parking deck spans.

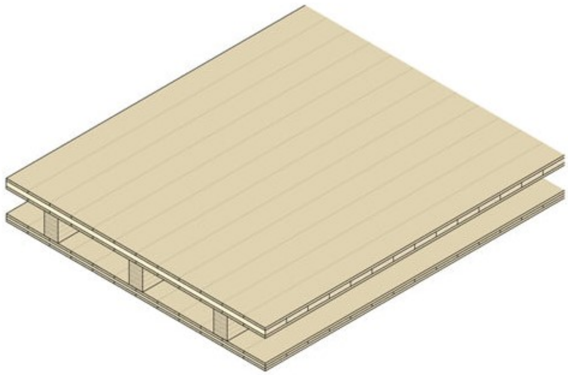


Figure 88. CLT closed rib panel (Stora Enso, 2022f).

9.1.4.1 Base Case

The required rib heights for the various spans as well as resulting total deck height for the closed CLT rib panels can be found in Table 30. The rib heights vary between 200 mm and 600 mm, or a total deck height of 440 to 840 mm. The table also shows what the required rib heights are for the different requirements for the base case. In Figure 89, the total deck height and self-weight of the deck are plotted against the parking deck span. The graph shows a roughly linear increase in height and weight for larger spans. Both the decks for a span of 17.16 and 17.66 can use the same deck.

Similar to the CLT open rib panels, for all spans, the criterion for vibrations is governing. Table 30 shows the significant potential for reduction of total deck height and weight if the vibrational requirement is removed.

Table 30. CLT closed rib panels for various span distances and the required rib height for each requirement.

CLT CLOSED RIB PANEL					BASE CASE							
L	Spacing	CLT Top panel	CLT Bottom panel	w_{Rib}	h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height			
[m]	[mm]	[mm]		[mm]	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	625	120	120	140	200	440	1.42	3.56	<200	<200	<200	<200
13.70	625	120	120	140	300	540	1.54	3.84	<200	<200	240	300
14.56	625	120	120	140	360	600	1.60	4.01	<200	<200	300	360
15.61	625	120	120	140	450	690	1.70	4.26	<200	<200	300	450
16.26	625	120	120	140	500	740	1.76	4.40	<200	<200	360	500
17.16	625	120	120	140	600	840	1.87	4.68	<200	<200	360	600
17.66	625	120	120	140	600	840	1.87	4.68	<200	<200	400	600

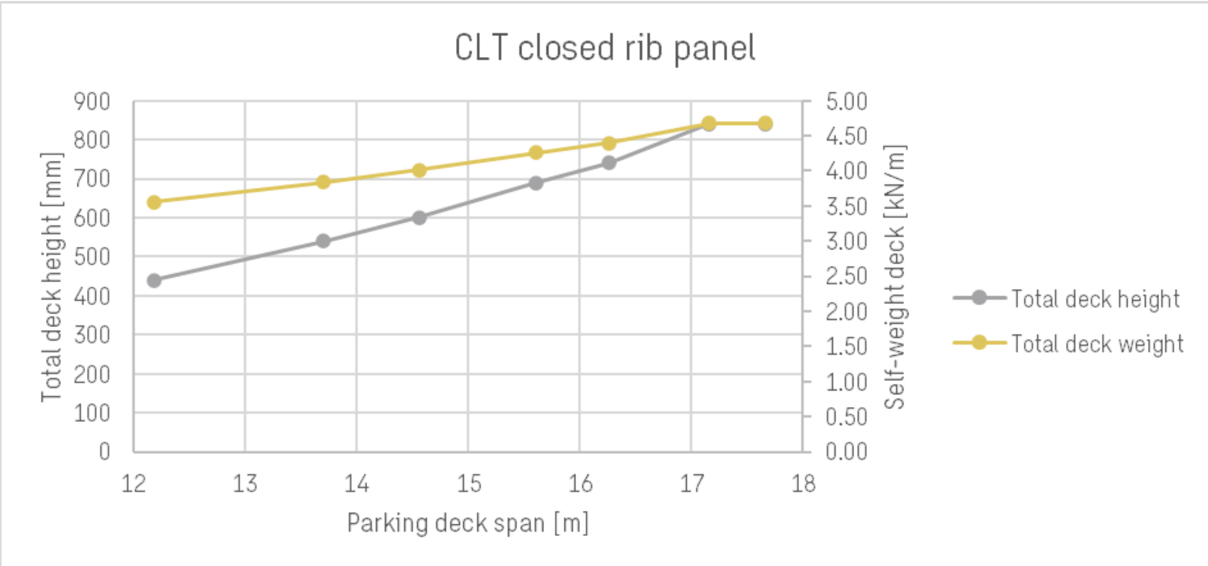


Figure 89. Total deck height for CLT closed rib panels for various spans.

9.1.4.2 Alternative Requirements

The impact of altering the requirements on the total deck height and weight can be seen in Figure 90 and are discussed below. The results that are at the basis of Figure 90 in tabular format can be found in Appendix F.

Reduced Fire-safety Requirements

A reduction of the fire-safety requirements from class R90 to R30 does not result in any reduction of total deck height or weight, since the normal ULS strength requirements require the same rib heights as the R90 ULS fire strength requirements.

No check on vibrations

As the vibration requirement is governing for all spans, removal of this requirement results in a decrease of total deck height and weight for all but one span. Only for the shortest span of 12.18 m, no reduction can be seen in the graph. This is due to the minimal applied rib height of 200 mm. For smaller rib heights, the CLT closed rib panel is not seen as an efficient panel. For a span of 13.70 m, a reduction in total deck height of 60 mm (11%) and weight of 0.17 kN/m (4%) is achieved. The largest reduction in height and weight is found for a span of 17.16 m: a reduction in height of 240 mm (29%) and a reduction in weight of 0.67 kN/m (14%).

Alternative Serviceability Limit Requirements

Altering the deformation limits on top of the removal of the vibration requirement can further reduce the total deck height and weight. A further reduction for a span of 13.70 m of 40 mm (8%) in height and 0.11 kN/m (3%) in weight is possible. The largest reduction is found for a span of 17.16 m: a further reduction of 120 mm (20%) in height and 0.34 kN/m (8%) in weight.

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

As already stated before, reduced fire-safety requirements do not result in a decrease in total deck height and weight.

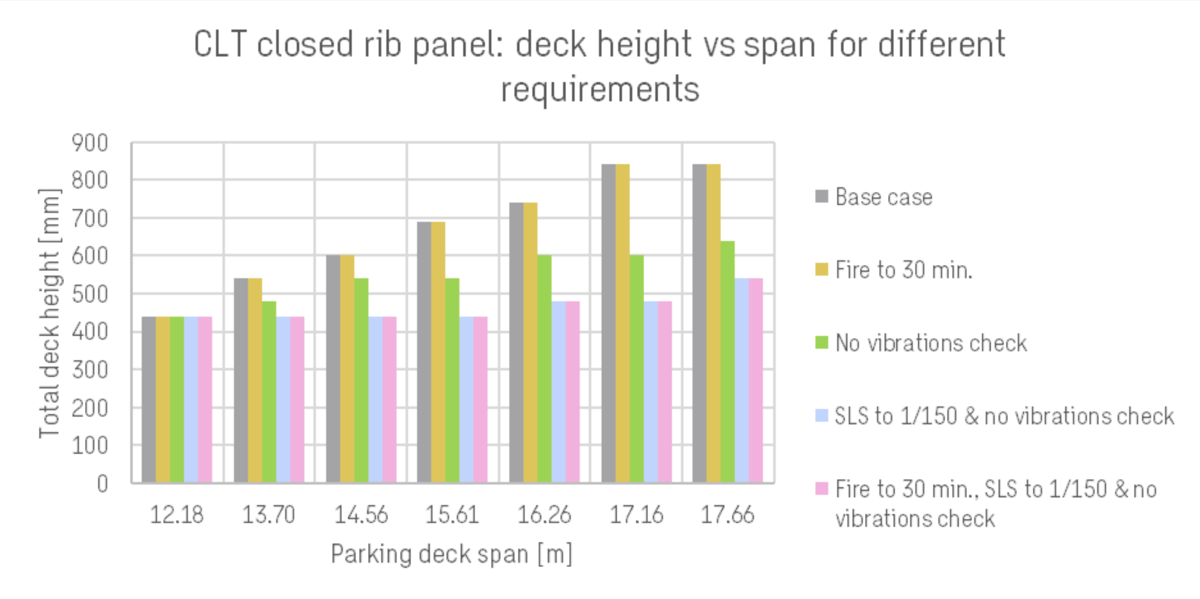


Figure 90. Differences in total deck height for various spans given alternative requirements for CLT closed rib panel.

Conclusion

From the results in this paragraph it has become clear, that altering the SLS requirements significantly reduces the total deck height and weight. The biggest reduction in height and weight is the result of removing the check for vibrations. Also allowing larger deformations results in a reduction of deck height and weight. A lower fire-safety class has no effect on the deck height and weight. The shortest span excluded, all spans see a significant reduction of height, varying from 100 mm (19%) for the 13.70 m span to 360 mm (43%) for the 17.16 m span. In terms of weight, a reduction of 0.28 kN/m (7%) is found for a span of 13.70 m up to a reduction of 1.01 kN/m (22%) for a span of 17.16 m.

9.1.5 LVL Panel

Thick LVL panels can also act as floor elements. For this, multiple LVL panels are glued on top of each other to create one thick strong LVL panel. A second layer of LVL typically runs across the grain at a 90° angle, to reinforce the veneer (Stora Enso, 2022d), see Figure 91. The result is a panel, which can support perpendicular loading and be used as a floor element.

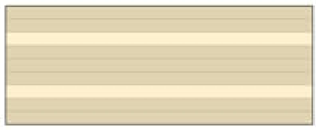


Figure 91. LVL panel (Stora Enso, 2022f).

9.1.5.1 Base Case

Applying a solid LVL panel to span the large spans of the parking deck results in rather heavy elements. Given the requirements for the base case, it is not possible to span all parking deck spans using the standard LVL panels of Stora Enso (Stora Enso, 2022e), which have a maximum thickness of 600 mm. Up to a span of 15.61 m, it is possible to use solid LVL panels with a thickness varying from 480 mm for the smallest span of 12.18 m, up to 600 mm for a span of 15.61 m, see Table 31 and Figure 92.

Similar to the CLT deck elements, in all cases for the solid LVL panel, the criteria for vibrations are governing. Alteration of these requirements can therefore result in a decrease of the height and weight of the LVL panel.

Table 31. LVL panels for various span distances and the required height for each requirement.

	BASE CASE							
L	$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	Normative			
[m]	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations
12.18	LVL G 480 8s	8	480	2.45	168	168	360	480
13.70	LVL G 540 9s	9	540	2.75	192	180	420	540
14.56	LVL G 540 9s	9	540	2.75	210	180	480	540
15.61	LVL G 600 10s	10	600	3.06	216	180	540	600
16.26	n.a.*	-	-	-	240	210	540	>600
17.16	n.a.*	-	-	-	252	210	600	>600
17.66	n.a.*	-	-	-	252	210	600	>600

** none of the standard panels by Stora Enso fulfils the minimum requirements of the base case*

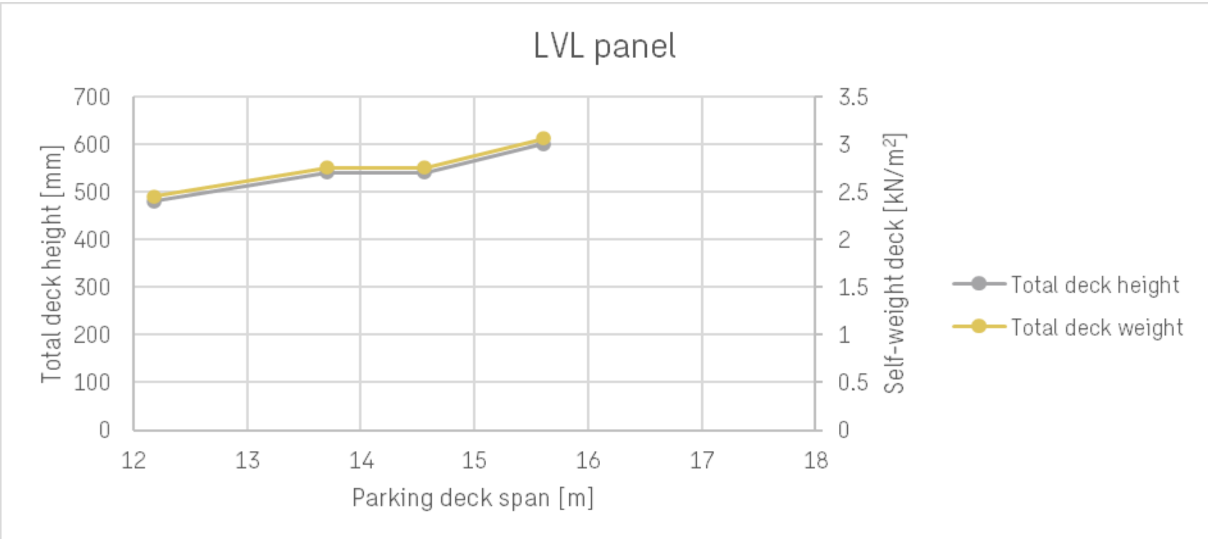


Figure 92. Height of LVL panel for various spans.

9.1.5.2 Alternative Requirements

The impact of altering the requirements on the total deck height and weight can be seen in Figure 93 and is discussed below. The results that are at the basis of Figure 93 in tabular format can be found in Appendix F.

Reduced Fire-safety Requirements

Altering the fire-safety class from R90 to R30 does not result in any reduction of the height of the LVL panels, since the normal ULS strength requirements require the same or thicker panels as can be seen in Table 31.

No check on vibrations

Removal of the check on vibrations results in a reduction of the deck height especially for the smaller spans. Furthermore, it allows the larger spans also to be within the range of the standard LVL panels. The biggest decrease in height is for the smallest span of 12.18 m, with a decrease in height and weight of 25% (120 mm and 0.61 kN/m^2). For a span of 15.61 m, it is a reduction of 10% (60 mm and 0.31 kN/m^2).

Alternative Serviceability Limit Requirements

Altering the serviceability limit requirements, including both vibrations and deformations, results in a further decrease in height and weight of the LVL panels. This reduction can be seen for all spans and lies around 20% for both height and weight. For the smallest span of 12.18 m, this is a reduction 66 mm in height and 0.34 kN/m^2 in weight, while for all spans larger than 13.70 m, a height reduction of 120 mm and weight reduction of 0.61 kN/m^2 is realised.

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

As already stated before, reduced fire-safety requirements do not result in a decrease in total deck height and weight.

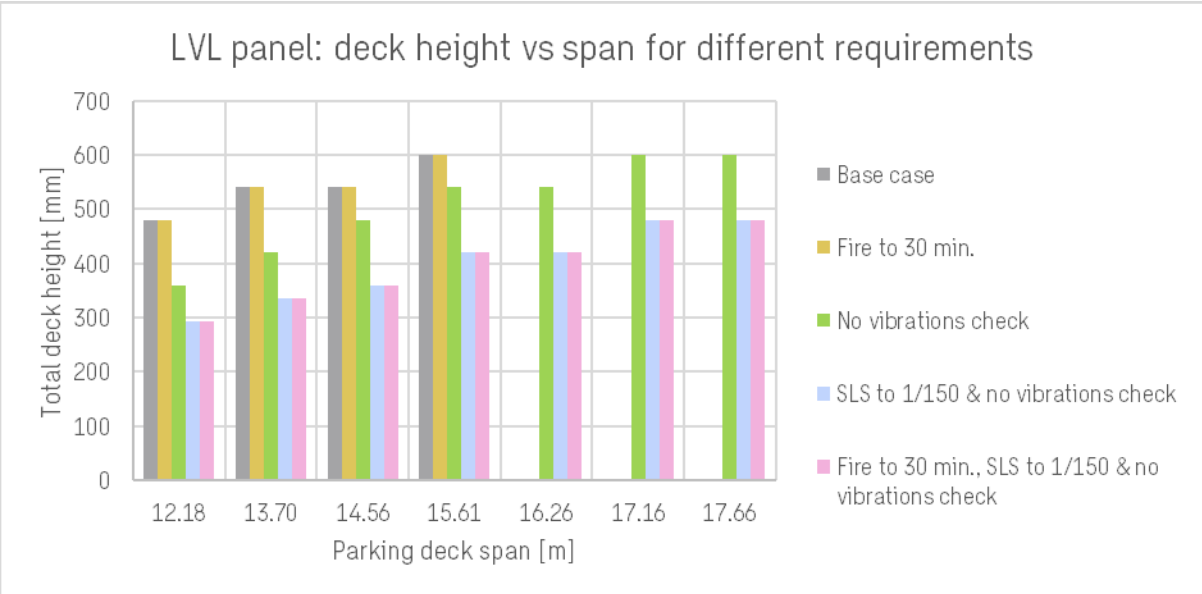


Figure 93. Differences in total deck height for various spans given alternative requirements for solid LVL panels.

Conclusion

From this paragraph it can be concluded that altering the serviceability limit requirements again can result in a significant reduction of height and weight of the LVL panels and can bring all parking deck spans into the range of the standard LVL panel dimensions. Similar to the earlier presented deck systems, removing the check on vibrations results in a significant decrease in deck height and weight. Also allowing larger deformations results in a reduction. A lower fire-safety class does not affect the total deck height and weight. The biggest reductions are found for the smaller spans at 39% in total (204 mm in height and 1.04 kN/m^2 in weight). For the larger spans, these values are slightly smaller.

9.1.6 LVL Open Rib Panel

The LVL open rib panel is similar to the CLT open rib panel, but instead of a CLT panel as deck, a LVL panel is used. For the ribs, again LVL is used instead of glulam ribs. Again, a full shear connection is realised between deck and ribs, resulting in a strong rib panel, able to carry high loads, see Figure 94.

Also for the LVL open rib panel, many different types of configurations can be created. In comparison with the CLT variant, the thickness of the deck and width of the ribs are much smaller as the result of the characteristics of the LVL material. However, this does pose a problem concerning fire-safety of the elements. Whereas with the CLT variant, the fire-safety requirements are met through the larger thickness of the elements, this is not possible for the thin LVL elements. Therefore, it is required to add 57 mm of gypsum board, covering the ribs of the panel, to provide a resistance against fire for the class of R90 (Stora Enso, 2023).

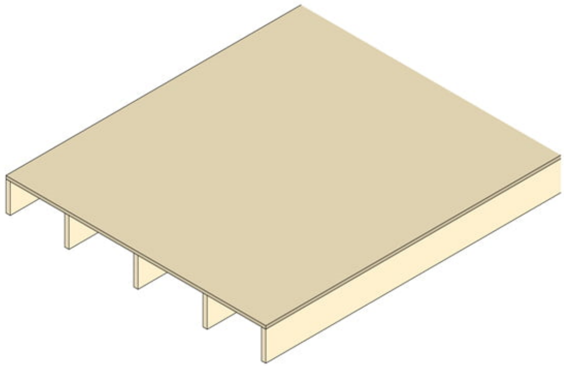


Figure 94. LVL open rib panel (Stora Enso, 2022f).

The parameters for the configuration of the LVL open rib panel are chosen based on the advice of Stora Enso (Stora Enso, 2022f). This has resulted in the use of a top panel with a thickness of 37 mm and a rib width of 51 mm. Considering the spacing between the ribs, two options are studied; a spacing of 625 mm and a spacing of 500 mm. The required rib height follows from calculation for the different parking deck spans.

9.1.6.1 Base Case

The results for the base case for both a spacing of 625 mm as well as for a spacing of 500 mm are presented in Table 32. From the results it becomes clear that the LVL open rib panel is not able to span the various parking deck spans, given the thickness of the elements and spacing stated above for the standard maximum height of the ribs of 600 mm. The table however also shows that in all cases, the criteria for vibrations are governing and in some cases also the deformation limits. However, when both criteria are governing, the vibrational criteria are limiting more than the deformation limits. No differences in required rib heights can be seen for the different spacings.

Table 32. LVL open rib panels for various span distances and the required rib height for each requirement.

LVL OPEN RIB PANEL				BASE CASE						
L	Spacing	$LVL_{Top\ panel}$	w_{Rib}	h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height		
[m]	[mm]	[mm]	[mm]	[mm]	[mm]	[kN/m]	[kN/m]	ULS	SLS	Vibrations
12.18	625	37	51	n.a.*	-	-	-	300	500	>600
13.70	625	37	51	n.a.*	-	-	-	360	600	>600
14.56	625	37	51	n.a.*	-	-	-	400	600	>600
15.61	625	37	51	n.a.*	-	-	-	450	>600	>600
16.26	625	37	51	n.a.*	-	-	-	450	>600	>600
17.16	625	37	51	n.a.*	-	-	-	450	>600	>600
17.66	625	37	51	n.a.*	-	-	-	500	>600	>600
12.18	500	37	51	n.a.*	-	-	-	300	500	>600
13.70	500	37	51	n.a.*	-	-	-	360	600	>600
14.56	500	37	51	n.a.*	-	-	-	360	600	>600
15.61	500	37	51	n.a.*	-	-	-	400	>600	>600
16.26	500	37	51	n.a.*	-	-	-	400	>600	>600
17.16	500	37	51	n.a.*	-	-	-	450	>600	>600
17.66	500	37	51	n.a.*	-	-	-	450	>600	>600

** none of the standard panels by Stora Enso fulfils the minimum requirements of the base case*

9.1.6.2 Alternative Requirements

As mentioned above, altering the requirements can bring the LVL open rib panel into the range of possible deck systems, for its standard dimensions. For the configurations chosen for this deck system, for most spans, LVL open rib panels are an option if alternative requirements are used. A comparison for the height for different spans and situations is presented in Figure 95. Below, it is studied what are the effects of reduced requirements for fire-safety, SLS and vibrations on the required thickness of the LVL decks. One or more of these requirements are altered, while the other requirements remain the same (equivalent to the base case requirements). The results that are at the basis of Figure 95 in tabular format can be found in Appendix F.

Reduced Fire-safety Requirements

Since the fire-safety requirements are met by addition of gypsum board to the panel, reducing the fire-safety requirements results in a different required thickness of the gypsum board. For a fire resistance class R30, gypsum boards of 42 mm thick are required (Stora Enso, 2023). This is a reduction of 15 mm in thickness. Logically, this does not have significant results for the total height and weight of the panel. Only reducing the fire-safety requirements therefore does not bring the parking decks into the range of possible standard configurations for the LVL open rib panel.

No check on vibrations

By removal of the check on vibrations, parking deck spans of 12.18 m up to 14.56 m become possible with LVL open rib panels. No difference is found for the various spacings applied.

Alternative Serviceability Limit Requirements

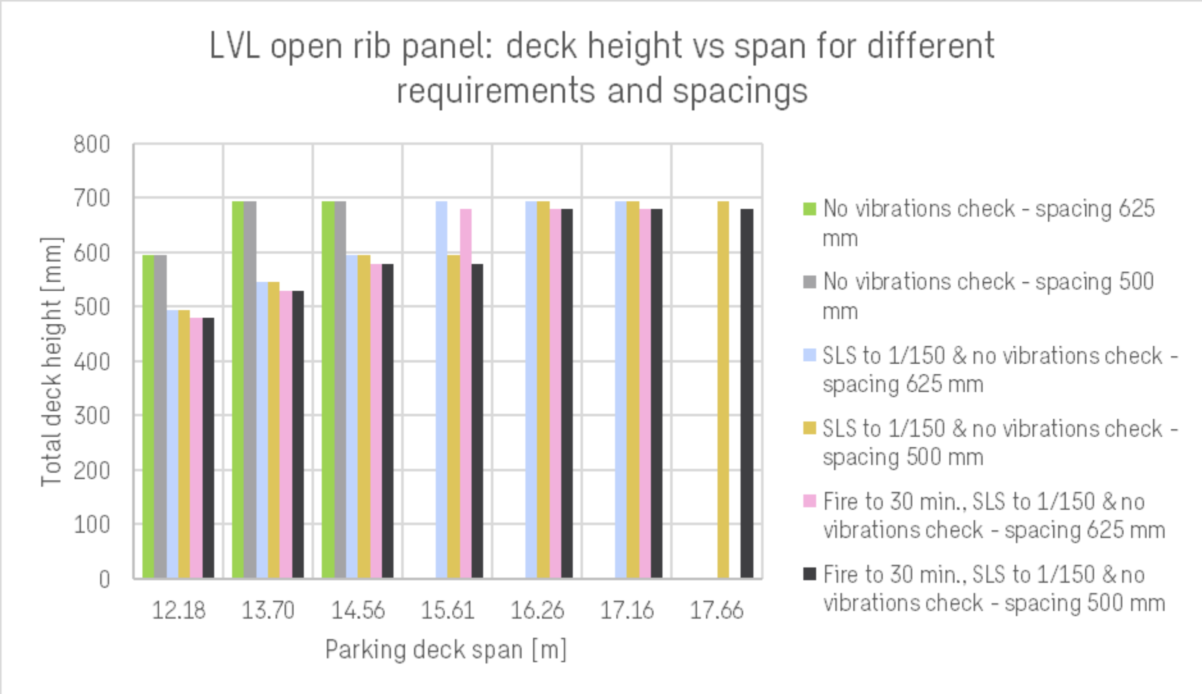
When applying lower deformation limits on top of the removal of the check on vibrations, this brings almost all parking deck spans into the range of the LVL open rib panels for the chosen configurations. This time, a difference is found between applying a spacing of 625 or 500 mm. The spacing of 500 mm performs better, reducing the total height of the panel, while only adding an insignificant amount of weight. Also with a spacing of 500 mm, it is possible to span also the longest span, which is not possible for a spacing of 625 mm. For the spans, which could already be covered by only removal of the vibration requirements, a height reduction between 100 and 150 mm (14-22%) is achieved and a weight reduction of 0.11 to 0.16 kN/m (6-9%).

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

Additionally reducing the fire-safety requirements does not result in a significant decrease in height and weight of the panel as this only results in a small reduction of the height and weight of the gypsum board.

Conclusion

In conclusion of this paragraph, it can be stated that alteration of the requirements is necessary to bring the different parking deck spans into the range of the LVL open rib panels. Only by altering all of the serviceability limit requirements it becomes possible to span also the longest parking deck span. A lower fire-safety class has no significant effect at the total deck height and weight. Removal of the check on vibrations is required to bring the different spans even into the range of available standard panels by Stora Enso. Reduction of the deformation limits results in a reduction of total deck height and weight.



** no standard panel possible for base case scenario and therefore not in graph | no standard panel possible for case without check on vibrations for spans 15.61 m to 17.66 m and therefore not in graph | no standard panel possible with spacing of 625 mm for case with R30 fire-safety class, reduced deformation limits and no check on vibrations for 17.66 m span and therefore not in graph*

Figure 95. Differences in total deck height for various spans given alternative requirements for LVL open rib panels.

9.1.7 LVL Semi-open Rib Panel

In addition to the LVL open rib panel, the LVL semi-open rib panel features an extra bottom tension flange to the ribs. The result is an I-shaped section, comprised of a LVL top panel, rib and bottom flange, fully structurally bonded together, see Figure 96. The bottom flange improves the structural performance of the panel, decreasing the height of the panel and making it possible to reach longer spans (Stora Enso, 2022f).

The LVL semi-open rib panels can of course also be created in many different configurations. Similar to the LVL open rib panels, the same parameters are used with the addition of the dimensions for the bottom flange. For this flange, a LVL panel with a thickness of 43 mm is used, which has a width of 300 mm, which is advised by Stora Enso (Stora Enso, 2022f). Also for the semi-open rib panel, addition of 57 mm of gypsum board is required to fulfil the requirements of the R90 fire-safety class. The required rib height is calculated for the different parking deck spans.

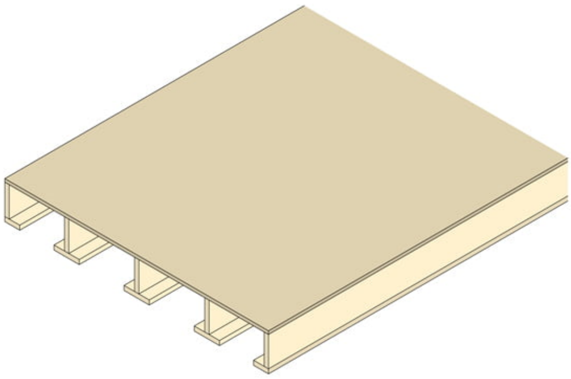


Figure 96. LVL semi-open rib panel (Stora Enso, 2022f).

9.1.7.1 Base Case

The results for the base case for both a spacing of 625 mm as well as for a spacing of 500 mm are presented in Table 33. From the results it becomes clear that the LVL semi-open rib panel is not able to span the various parking deck spans, given the thickness of the elements and spacing stated above for the standard maximum height of the ribs of 600 mm. The table however also shows that in all cases, the criteria for vibrations are governing and in one case also the deformation limits. However, when both criteria are governing, the vibrational criteria are limiting more than the deformation limits. Some differences in required rib heights can be seen for the different spacings.

Table 33. LVL semi-open rib panels for various span distances and the required rib height for each requirement.

LVL SEMI-OPEN RIB PANEL					BASE CASE						
L	Spacing	LVL_{Top} panel	LVL_{Bottom} panel	w_{Rib}	h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height		
[m]	[mm]	[mm]	[mm]	[mm]	[mm]	[mm]	[kN/m]	[kN/m]	ULS	SLS	Vibrations
12.18	625	37	43	51	n.a.*	-	-	-	240	360	>600
13.70	625	37	43	51	n.a.*	-	-	-	240	450	>600
14.56	625	37	43	51	n.a.*	-	-	-	300	500	>600
15.61	625	37	43	51	n.a.*	-	-	-	300	600	>600
16.26	625	37	43	51	n.a.*	-	-	-	300	600	>600
17.16	625	37	43	51	n.a.*	-	-	-	300	600	>600
17.66	625	37	43	51	n.a.*	-	-	-	300	>600	>600
12.18	500	37	43	51	n.a.*	-	-	-	200	360	>600
13.70	500	37	43	51	n.a.*	-	-	-	200	450	>600
14.56	500	37	43	51	n.a.*	-	-	-	240	450	>600
15.61	500	37	43	51	n.a.*	-	-	-	240	500	>600
16.26	500	37	43	51	n.a.*	-	-	-	240	600	>600
17.16	500	37	43	51	n.a.*	-	-	-	240	600	>600
17.66	500	37	43	51	n.a.*	-	-	-	300	600	>600

** none of the standard panels by Stora Enso fulfils the minimum requirements of the base case*

9.1.7.2 Alternative Requirements

In Table 33 it was already shown, that for alternative requirements, the LVL semi-open rib panel becomes a potential deck system for all spans (depending on the alterations). A comparison for the height for different spans and situations is presented in Figure 97. The results that are at the basis of Figure 97 in tabular format can be found in Appendix F.

Reduced Fire-safety Requirements

Since the fire-safety requirements are met by addition of gypsum board to the panel, reducing the fire-safety requirements results in a different required thickness of the gypsum board. For a fire resistance class R30, gypsum boards of 42 mm thick are required (Stora Enso, 2023). This is a reduction of 15 mm in thickness. Logically, this does not have significant results for the total height and weight of the panel. Only reducing the fire-safety requirements therefore does not bring the parking decks into the range of possible standard configurations for the LVL semi-open rib panel.

No check on vibrations

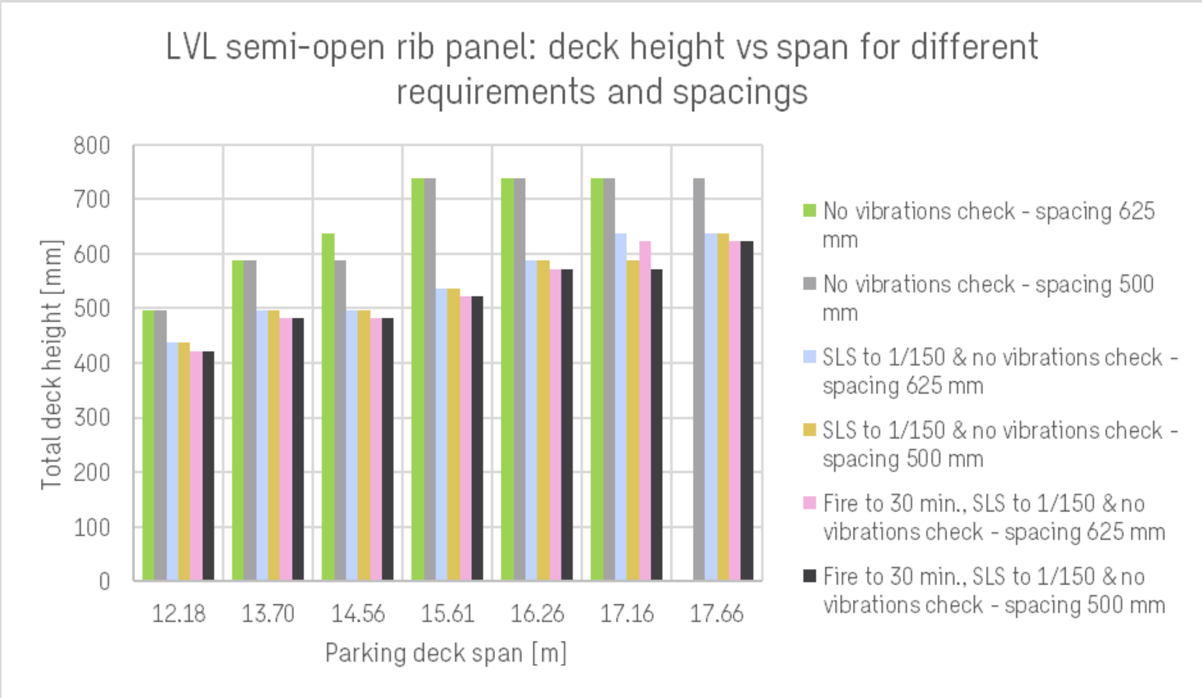
Removal of the check on vibrations brings almost all parking deck spans in the range of the LVL semi-open rib panels given the configuration of these panels. Only the longest span of 17.66 m is not possible when a spacing of 625 mm is used, but is possible for a spacing of 500 mm. Furthermore, a difference between both spacings is found for a span of 14.56 mm, in which case the spacing of 500 mm outperforms the larger spacing in a height difference of 50 mm.

Alternative Serviceability Limit Requirements

Altering the serviceability limit requirements, including both vibrations and deformations, results in a further decrease in height and weight of the LVL semi-open rib panels. A difference between both spacings is only found for a span of 17.16 m, in which case the smaller spacing outperforms the larger one in terms of height by 50 mm. For all spans, a significant reduction in height is found, which is the biggest for the spans in the middle of the set of spans considered. For the smallest span, the reduction in height is 60 mm (12%), while the weight reduction is rather small at 0.06 kN/m (3%). For a span of 15.61 m, the reduction in height is the biggest at 200 mm (27%), which gives a weight reduction of 0.21 kN/m (10%).

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

Additionally reducing the fire-safety requirements does not result in a significant decrease in height and weight of the panel as this only results in a small reduction of the height and weight of the gypsum board.



** no standard panel possible for base case scenario and therefore not in graph | no standard panel with spacing of 625 mm possible for case without check on vibrations for span 17.66 m and therefore not in graph*

Figure 97. Differences in total deck height for various spans given alternative requirements for LVL semi-open rib panels.

Conclusion

The results presented in this paragraph have shown that alternative requirements make it possible to span all parking deck spans with an LVL semi-open rib panel. Altering the fire-safety requirements has no significant effect, while altering the vibrational and deformation limits gives very significant reduction, especially in height.

9.1.8 LVL Closed Rib Panel

The LVL closed rib panel consists out of two LVL panels connected to each other by LVL ribs, see Figure 98. The result is a lightweight box floor system, of which the elements are fully structurally bonded together. Like with the other LVL systems, it requires additional gypsum boards to make the system pass the fire-safety requirements.

For the configuration of the LVL closed rib panel, the same parameters as for the semi-open rib panel are used, except the bottom flange is now a full panel. The top panel has a thickness of 37 mm, the bottom panel 43 mm and the ribs 51 mm. For the spacing, two situations are considered; a spacing of 625 mm and of 500 mm. The required rib height is calculated for the different parking deck spans.

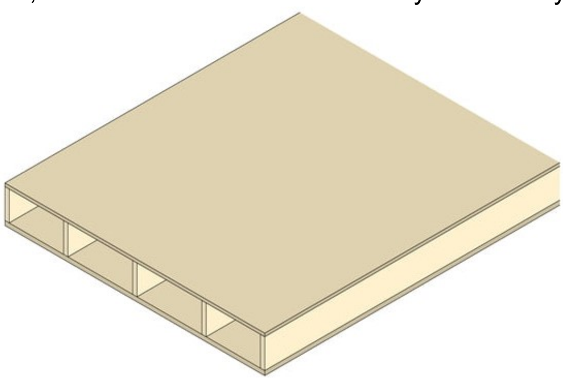


Figure 98. LVL closed rib panel (Stora Enso, 2022f).

9.1.8.1 Base Case

The results for the rib height and total panel height for the base case for the LVL closed rib panel are shown in Table 34. Given the requirements of the base case, it is possible to span a parking deck span of 12.18 or 13.70 m with the chosen configuration of the LVL closed rib panel for both spacings. Larger spans are not possible given the maximum rib height of 600 mm. For all spans, the criteria for vibrations are governing. Furthermore, some variation can be seen between both spacings for the governing rib heights for ULS requirements.

Table 34. LVL closed rib panels for various span distances and the required rib height for each requirement.

LVL CLOSED RIB PANEL					BASE CASE						
L	Spacing	LVL_{Top} panel	LVL_{Bottom} panel	w_{Rib}	h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height		
[m]	[mm]	[mm]		[mm]	[mm]	[mm]	[kN/m]	[kN/m]	ULS	SLS	Vibrations
12.18	625	37	43	51	600	737	0.98	2.44	300	360	600
13.70	625	37	43	51	600	737	0.98	2.44	360	400	600
14.56	625	37	43	51	n.a.*	-	-	-	360	450	>600
15.61	625	37	43	51	n.a.*	-	-	-	360	500	>600
16.26	625	37	43	51	n.a.*	-	-	-	400	500	>600
17.16	625	37	43	51	n.a.*	-	-	-	400	600	>600
17.66	625	37	43	51	n.a.*	-	-	-	400	600	>600
12.18	500	37	43	51	600	737	1.04	2.60	240	360	600
13.70	500	37	43	51	600	737	1.04	2.60	240	400	600
14.56	500	37	43	51	n.a.*	-	-	-	300	450	>600
15.61	500	37	43	51	n.a.*	-	-	-	300	500	>600
16.26	500	37	43	51	n.a.*	-	-	-	300	500	>600
17.16	500	37	43	51	n.a.*	-	-	-	360	600	>600
17.66	500	37	43	51	n.a.*	-	-	-	360	600	>600

** none of the standard panels by Stora Enso fulfils the minimum requirements of the base case*

9.1.8.2 Alternative Requirements

With alternative requirements, it is possible to span the entire range of considered parking deck spans with the LVL closed rib panel. A comparison for the height for different spans and situations is presented in Figure 99. The results that are at the basis of Figure 99 in tabular format can be found in Appendix F.

Reduced Fire-safety Requirements

Since the fire-safety requirements are met by addition of gypsum board to the panel, reducing the fire-safety requirements results in a different required thickness of the gypsum board. For a fire resistance class R30, gypsum boards of 42 mm thick are required. This is a reduction of 15 mm in thickness. Logically, this does not have significant results for the total height and weight of the panel. Only reducing the fire-safety requirements therefore does not bring the parking decks into the range of possible standard configurations for the LVL semi-open rib panel.

No check on vibrations

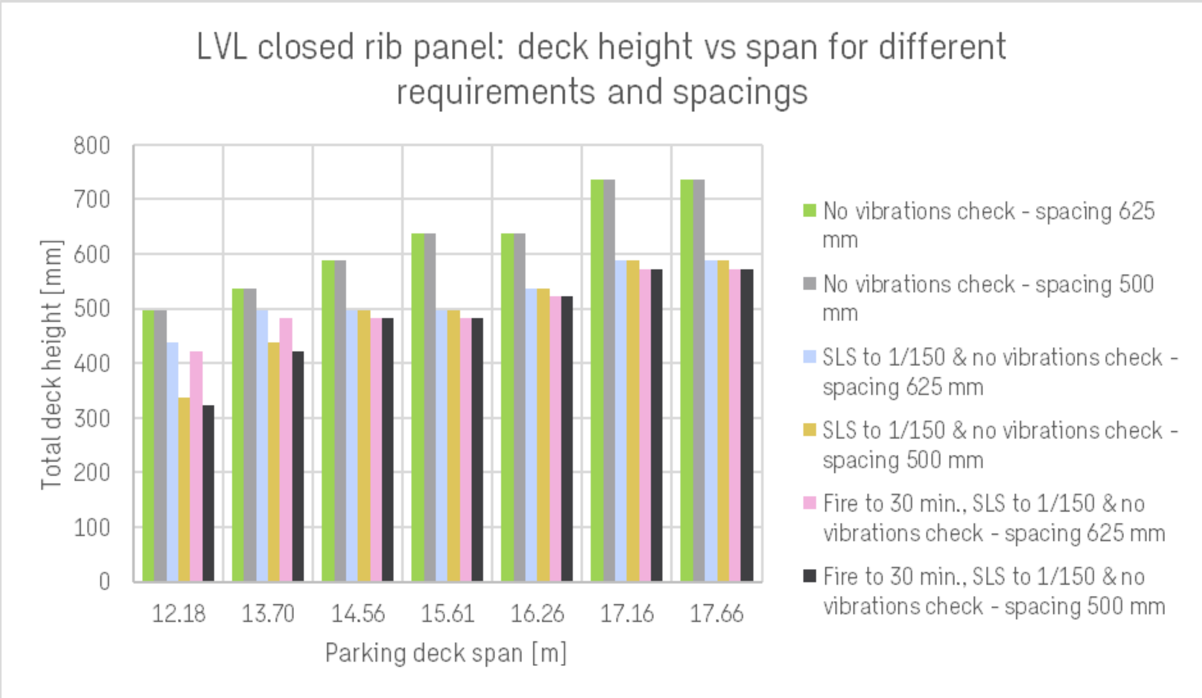
When the check on vibrations is removed, it is possible to cover the entire considered range of parking deck spans with the LVL closed rib panels. A significant reduction in height of the panel can also be seen for the two smallest spans, which could already be covered including the check on vibrations. For the span of 12.18 m, the height of the panel is reduced by 240 mm (33%), which lowers the weight by 0.26 kN/m (11%). No difference can be found between the spacings.

Alternative Serviceability Limit Requirements

Additionally lowering the deformation limits results in a further decrease in height and weight of the panels. For the smallest spans of 12.18 m and 13.70 m, a difference is found between both spacings. For a spacing of 500 mm, the highest reduction in height and weight is found: 160 mm (32%) and 0.21 kN/m (9%). For other spans, most reductions lie around 20% for height (100 - 150 mm) and 8% for weight (0.13-0.20 kN/m).

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

Additionally reducing the fire-safety requirements does not result in a significant decrease in height and weight of the panel as this only results in a small reduction of the height and weight of the gypsum board.



** no standard panel possible for base case scenario and therefore not in graph*

Figure 99. Differences in total deck height for various spans given alternative requirements for LVL closed rib panels.

Conclusion

This paragraph has shown that altering the requirements significantly impacts mostly the height of the panels. Without alteration, it is not possible to cover all parking deck spans, which is possible for a different set of requirements for the serviceability limits. Altering the fire-safety class of the structure has no significant effect on the total deck height and weight. By removal of the check on vibrations, it becomes possible to cover the various spans with the LVL closed rib panels. Allowing larger deformations results in a further decrease in total deck height and weight.

9.1.9 Comparison between Decks

A comparison is made between the seven different deck types, first for the base case and second for the alternative requirements. The decks are compared for their total deck height and weight. Following from the comparison, a choice can be made for a preferential deck system given a certain parking deck span and set of requirements.

9.1.9.1 Base Case

Given the requirements of the base case, a comparison on total deck height and weight is made for the seven deck systems. In Figure 100, the total deck heights of the different deck systems are presented next to each other. A similar graph, but for the deck weight is presented in Figure 101. Immediately it can be seen that for not a single span, a bar is found for all seven deck systems. Given the standard wood engineered products by Stora Enso, not a single one of the considered parking deck spans can be covered by a solid CLT panel, LVL open rib panel or LVL semi-open rib panel. These deck systems can therefore be ruled out as an option for the long-span deck systems considered for the structural system of the multi-storey car park. From the remaining four deck systems, the solid LVL panel and LVL closed rib panel are sufficient to span some of the considered parking deck spans. The CLT open rib panel and CLT closed rib panel can span all considered parking deck spans.

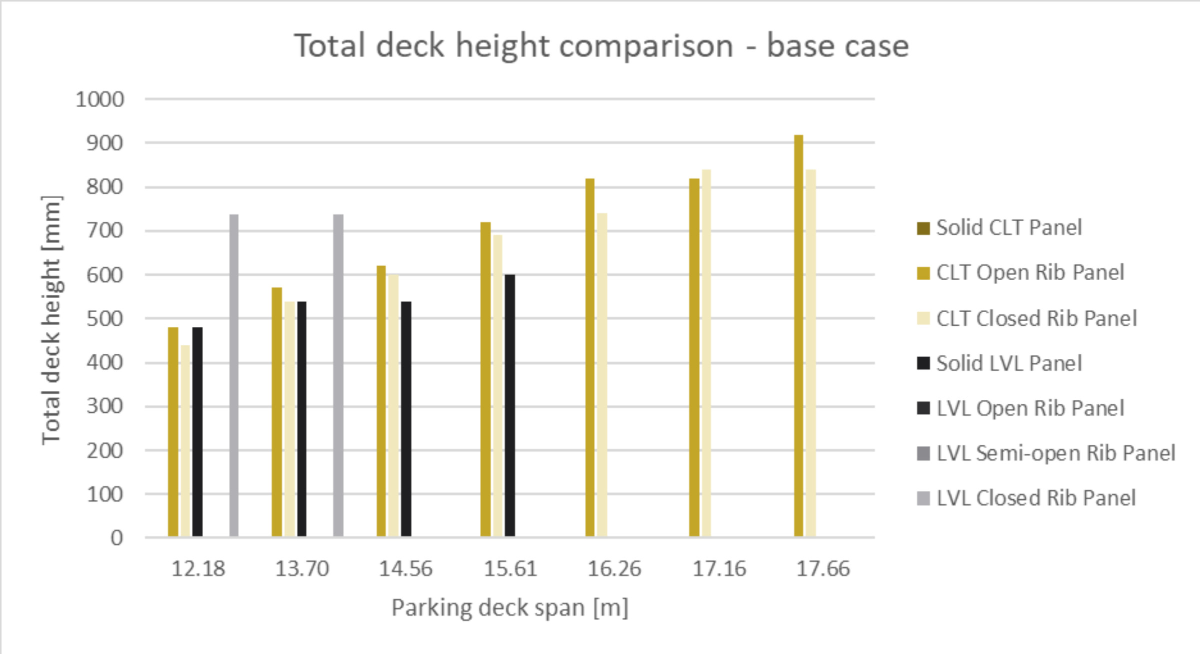
When looking at the weight of the deck systems in Figure 101, it stands out that the self-weight of the solid LVL deck for all possible spans with this system is around twice the weight of the different CLT deck systems and even more than twice the weight of the LVL closed rib panel. Despite its relatively small total deck height, the solid LVL panel is ruled out as a potential prefab long-span deck system as a result of its high self-weight.

The LVL closed rib panel, which is possible for the two shortest spans considered (12.18 and 13.70 m), has the smallest self-weight of the remaining potential deck systems. In comparison with the CLT open rib panel, its weight is 11% and 21% smaller for each span respectively. Compared to the CLT closed rib panel, its weight is 27% and 32% smaller. In terms of total deck height however, the systems has the largest total deck height of the remaining deck systems. Compared to the CLT open rib panel, it is 54% and 29% higher for each span respectively. In comparison with the CLT closed rib panel, these percentages are 68% and 36%. Since the percentual advantage in weight compared to the percentual disadvantage in height for the LVL closed rib panel against the other two systems is in all cases is smaller (for almost all cases by large amount), the LVL closed rib panel is ruled out as a potential prefab long-span deck system.

When comparing the two remaining potential long-span deck systems (CLT open rib panel and CLT closed rib panel), it can be seen that most often, the CLT closed rib panel performs better in terms of total deck height. However, its self-weight is always larger or equal to the weight of the CLT open rib panel. The difference in height between the systems varies between 0% and 10% and the weight difference between 0% and 17%. In the comparison of the two deck systems, if the percentual advantage in terms of height for example of one system is bigger than its disadvantage in terms of weight to the other system, this deck system is given the preferred choice. This results in the deck systems presented per parking deck span in Table 35.

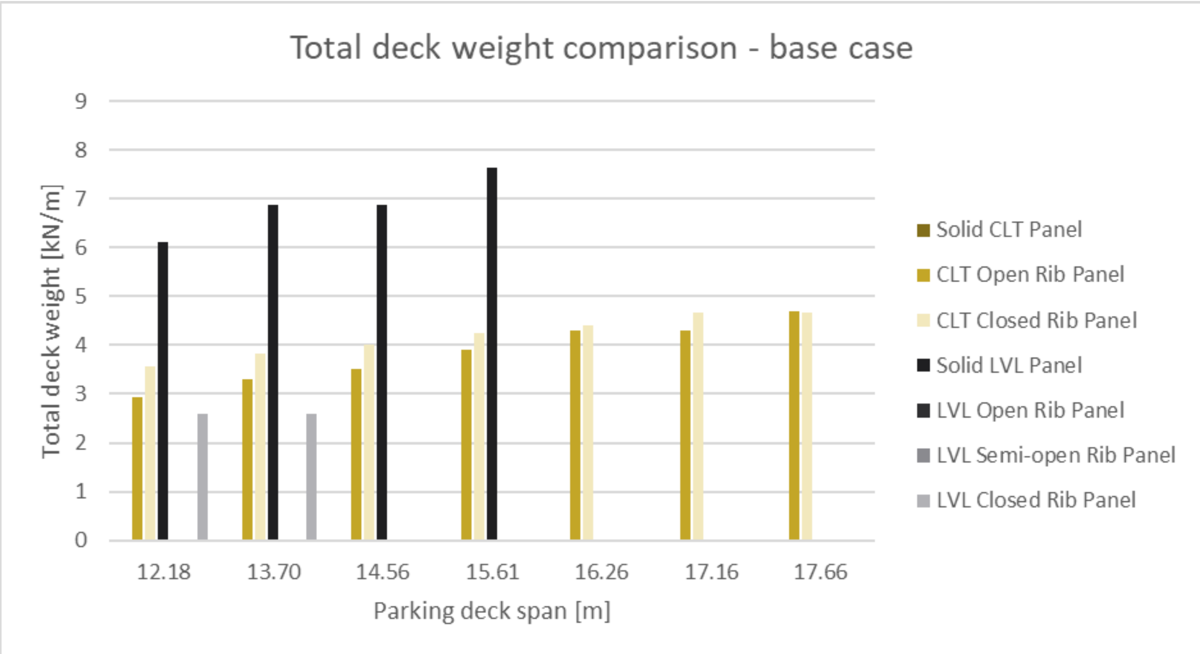
Table 35. Choice of prefab long-span deck system for various parking deck spans for base case.

	BASE CASE			
Span [m]	**Deck type** [-]	h_{tot} [mm]	$g_{0,k}$ [kN/m]	$g_{0,k,tot(2500mm)}$ [kN/m]
12.18	Open CLT deck: 120mm CLT - 4 ribs 360x200mm	480	1.18	2.95
13.70	Open CLT deck: 120mm CLT - 4 ribs 450x200mm	570	1.32	3.30
14.56	Open CLT deck: 120mm CLT - 4 ribs 500x200mm	620	1.40	3.50
15.61	Open CLT deck: 120mm CLT - 4 ribs 600x200mm	720	1.56	3.90
16.26	Closed CLT deck: 120mm CLT - 4 ribs 500x140mm - 120mm CLT	740	1.76	4.40
17.16	Open CLT deck: 120mm CLT - 4 ribs 700x200mm	820	1.72	4.30
17.66	Closed CLT deck: 120mm CLT - 4 ribs 600x140mm - 120mm CLT	840	1.87	4.68



** not all deck systems can span all spans for the base case scenario and therefore can be missing from the graph*

Figure 100. Total deck height comparison prefab long-span deck systems - base case.



** not all deck systems can span all spans for the base case scenario and therefore can be missing from the graph*

Figure 101. Total deck weight comparison prefab long-span deck systems - base case.

9.1.9.2 Alternative Requirements

Altering the requirements can result in a change of preferred long-span deck system. As discussed previously in this chapter, altering the requirements can bring other deck systems in range of the considered parking deck spans, which were not possible with the standard requirements of the base case. The changes in preferred deck systems for alternative requirements are shortly discussed.

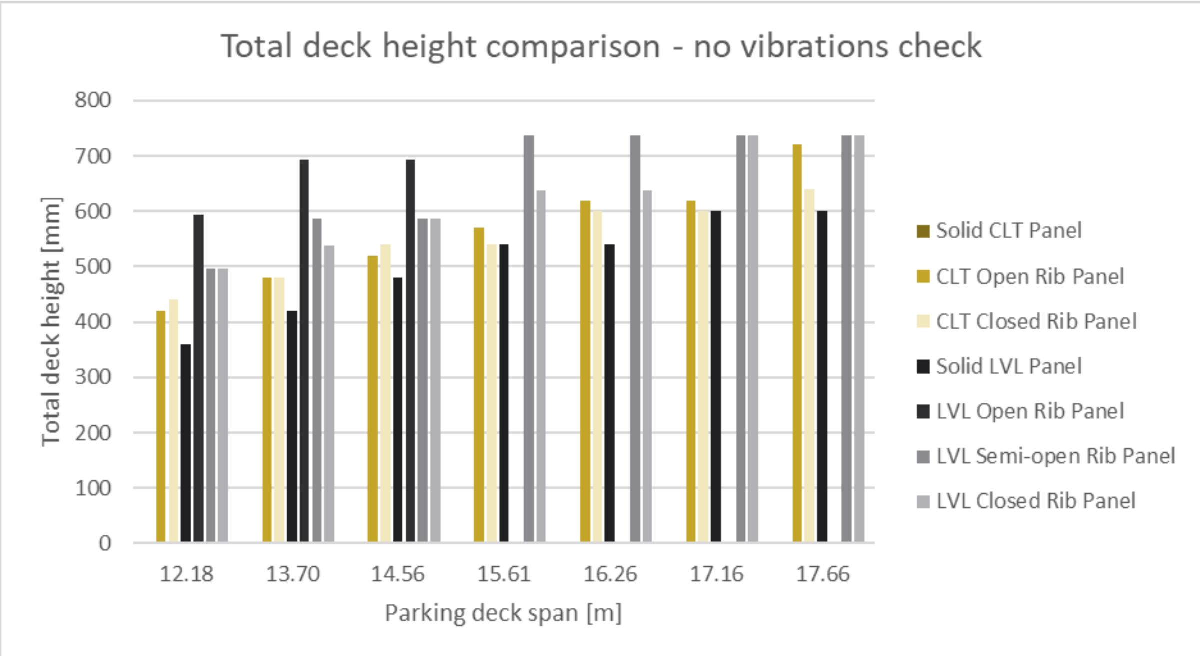
Reduced Fire-safety Requirements

As was already discussed in the various paragraphs for each deck system, altering the fire-safety requirements does not significantly influence the total deck height and weight of each long-span deck system. Therefore, this also does not result in a change of preferred long-span deck system under these altered requirements.

No check on vibrations

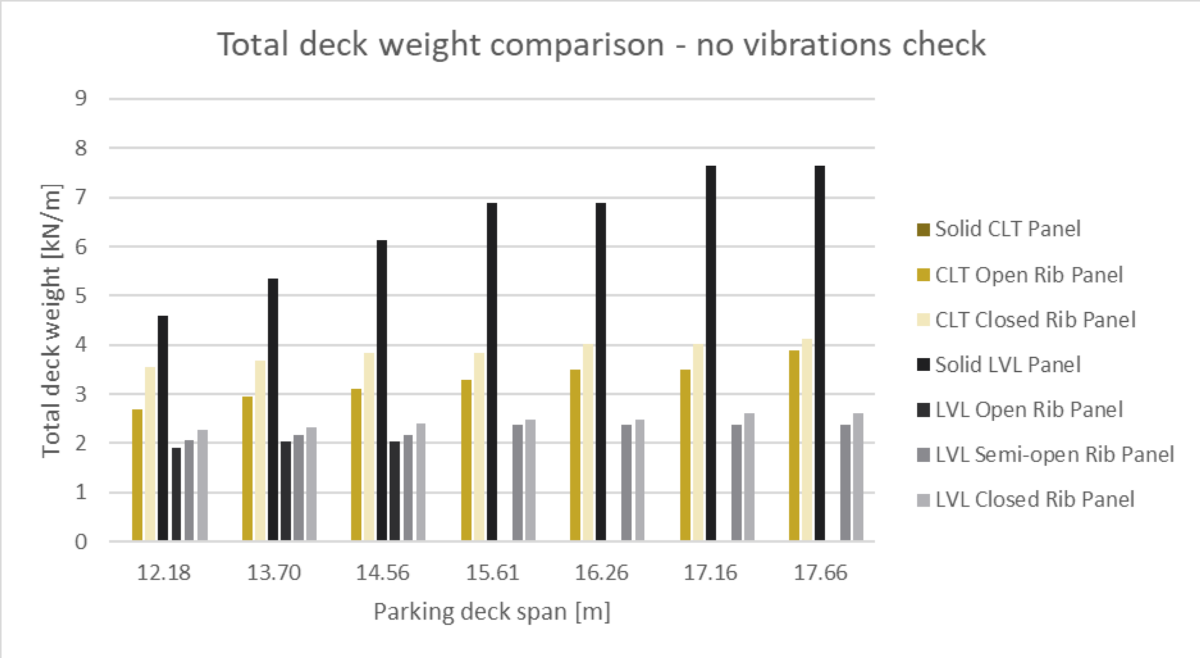
When vibrational behaviour of the long-span deck systems is not considered, this results in a different picture of the performance of the various long-span deck systems. As already discussed in previous paragraphs, this allows the use of different deck systems. What does not change is the much larger self-weight of the solid LVL panels in comparison with all other deck systems, as can be seen in Figure 103. Furthermore, in this graph can be seen that the other LVL deck systems outperform the CLT deck systems in terms of total deck weight. The total deck heights are presented in Figure 102 and present a slightly different picture, in which the LVL decks have a larger total deck height compared to the CLT decks.

Similar to the base case, a trade-off is made between the percentual advantage of a reduction in height and the percentual disadvantage of an increase in self-weight. Based on this trade-off, a choice is made for the preferred long-span deck system, when no check on vibrations is performed. The resulting deck systems are presented in Table 36. What stands out, is that these are all LVL systems, whereas for the base case these were CLT systems. Therefore it can be concluded, that the LVL systems are sensitive to vibrational criteria, but otherwise outperform the CLT systems.



** not all deck systems can span all spans for the scenario without check on vibrations and therefore can be missing from the graph*

Figure 102. Total deck height comparison prefab long-span deck systems - no vibrations check.



** not all deck systems can span all spans for the scenario without check on vibrations and therefore can be missing from the graph*

Figure 103. Total deck weight comparison prefab long-span deck systems - no vibrations check.

Table 36. Choice of prefab long-span deck system for various parking deck spans for alternative requirements, no vibrations check.

	No vibrations check			
Span [m]	**Deck type [-]**	**h_{tot} [mm]**	**$g_{0,k}$ [kN/m]**	**$g_{0,k,tot(2500mm)}$ [kN/m]**
12.18	LVL semi-open rib panel: h_{rib} = 360mm	497	0.82	2.06
13.70	LVL closed rib panel: h_{rib} = 400mm	537	0.93	2.34
14.56	LVL semi-open rib panel: h_{rib} = 450mm	587	0.87	2.18
15.61	LVL closed rib panel: h_{rib} = 500mm	637	0.99	2.47
16.26	LVL closed rib panel: h_{rib} = 500mm	637	0.99	2.47
17.16	LVL semi-open rib panel: h_{rib} = 600mm	737	0.95	2.38
17.66	LVL semi-open rib panel: h_{rib} = 600mm	737	0.95	2.38

Alternative Serviceability Limit Requirements

When on top of the removal of the check on vibrations, also the deformation limits are altered, some changes can be seen in the required deck heights and weights that follow. The results are presented in Figure 104 and Figure 105. In terms of the best performing deck system, still the LVL closed rib panel and LVL open rib panel perform best. However, as a result of the different deformation limits, depending on the parking deck span, sometimes the other deck system now performs better. An overview of the preferred deck system per parking deck span can be found in Table 37.

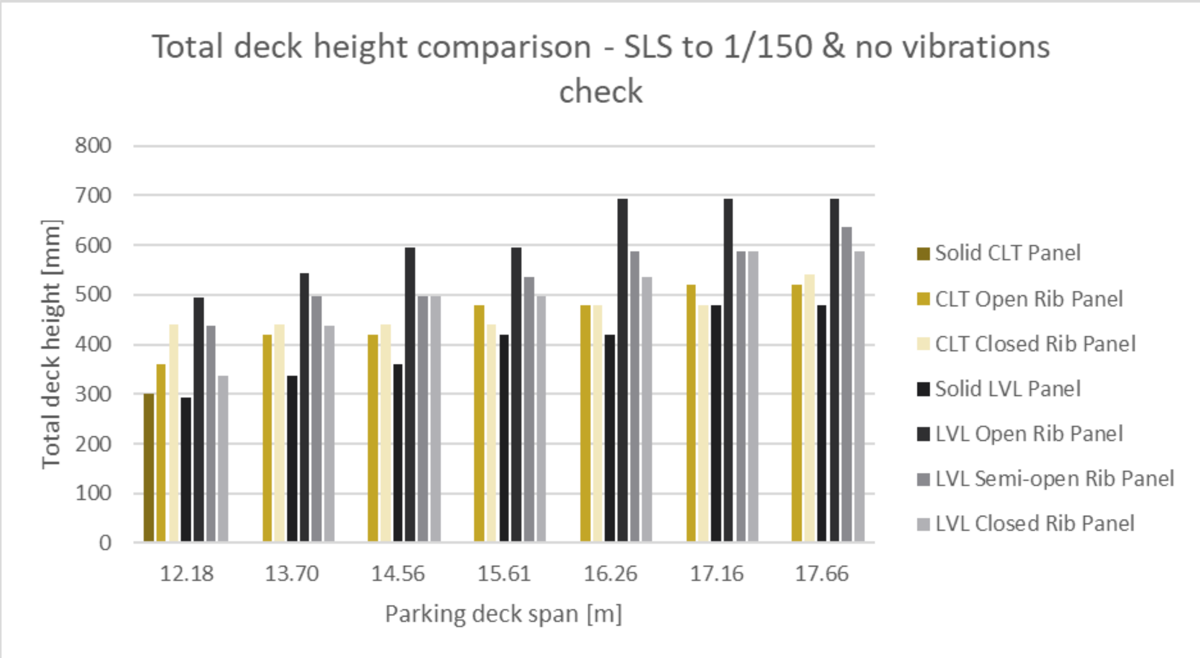


Figure 104. Total deck height comparison prefab long-span deck systems - SLS to 1/150 & no vibrations check.

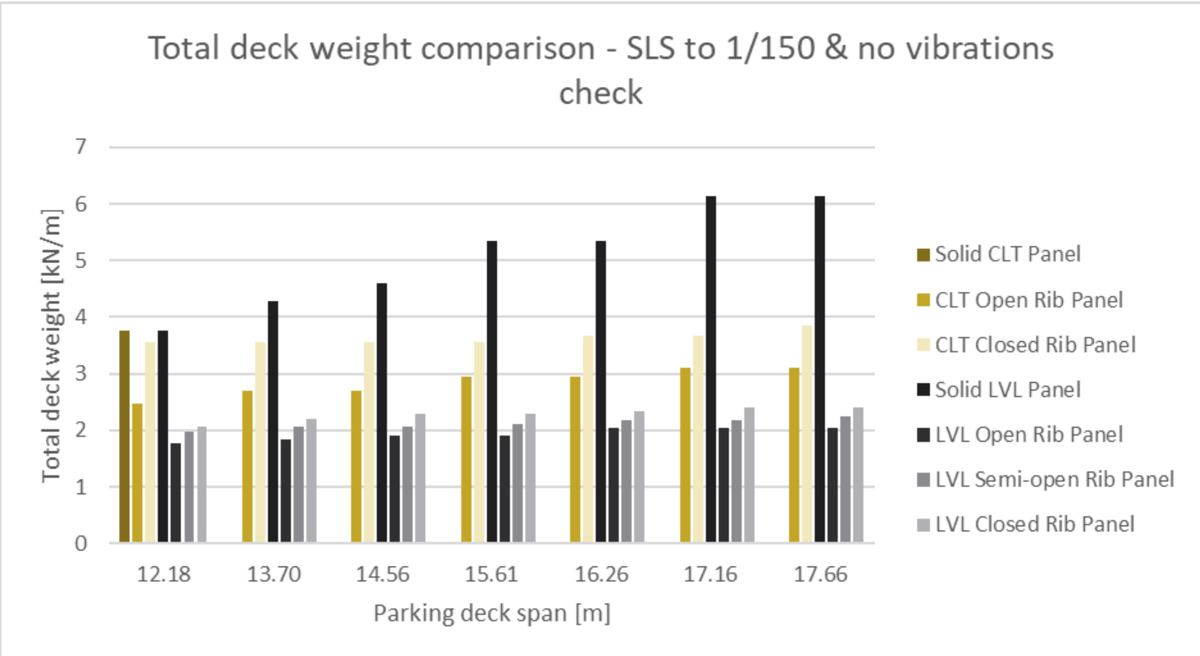


Figure 105. Total deck weight comparison prefab long-span deck systems - SLS to 1/150 & no vibrations check.

Table 37. Choice of prefab long-span deck system for various parking deck spans for alternative requirements, SLS to 1/150 & no vibrations check.

	SLS to 1/150 & no vibrations check			
Span [m]	**Deck type [-]**	**h_{tot} [mm]**	**$g_{0,k}$ [kN/m]**	**$g_{0,k,tot(2500mm)}$ [kN/m]**
12.18	LVL closed rib panel: h_{rib} = 200mm	337	0.83	2.07
13.70	LVL closed rib panel: h_{rib} = 300mm	437	0.88	2.20
14.56	LVL semi-open rib panel: h_{rib} = 360mm	497	0.82	2.06
15.61	LVL closed rib panel: h_{rib} = 360mm	497	0.91	2.28
16.26	LVL closed rib panel: h_{rib} = 400mm	537	0.93	2.34
17.16	LVL semi-open rib panel: h_{rib} = 450mm	587	0.87	2.18
17.66	LVL closed rib panel: h_{rib} = 450mm	587	0.96	2.40

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

Additionally reducing the fire-safety requirements does not result in a significant change in total deck height and weight. Therefore, the deck system types presented in Table 37 also stand for these alternative requirements.

9.2 Short-span Prefab Deck System

In this thesis, (sub)spans between 2.5 and 10 m are considered to be short-spans. Such spans are found in design variants 2 (a/b) and 3 (see paragraphs 5.3.2.2, 5.3.2.3 and 5.3.2.4), in which the deck is supported by different configurations of beam elements, spanning the parking deck. In these design variants, different spans and both single- as well as multi-span configurations can be found. For the short-span decks, two types of prefab deck systems are studied: a CLT deck and a LVL deck. Both types are discussed in the following paragraphs, in which the required height of the deck is determined for a base case, in which all standard rules and norms apply and various other cases, for which one or multiple requirements are altered. Finally, the results will provide insight into the impact of different configurations of short-span deck systems on the height and weight of the CLT or LVL floor as well as what effect changing parameters and/or requirements has on those aspects.

9.2.1 General

In Figure 106, an example of a schematisation of a 2-span short-span deck system is presented with its various loads. Similar to the long-span deck system, the top level parking deck is considered and the same load values as in Table 28 (see also starting points in chapter 6) are used. For the short-span deck system, a section width of 1 m is considered. Considering the vibration analysis, a total deck width of 12.18 m is used as a starting point.

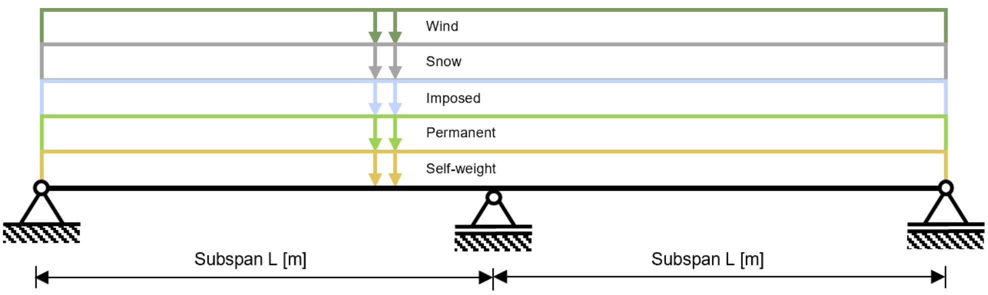


Figure 106. Schematisation short-span deck system (2-span)

Both the CLT deck and LVL deck system are studied for their required deck thickness for a different number of spans and span lengths. From the required deck thickness also follows the weight of the deck. First, the decks are studied for the basic requirements for ULS, SLS, fire and vibrations as stated in chapter 6. Next, the requirements concerning SLS, fire and vibrations are altered in accordance with the overview of parameters in paragraph 0.. Finally, the deck systems of CLT and LVL are compared to each other.

For the CLT and LVL decks, the standard products of manufacturer Stora Enso are considered (Stora Enso, 2022b, 2022e). The calculations are made using the Calculatis software package by Stora Enso (Stora Enso, 2022a). Examples of calculations can be found in Appendix F.2.

9.2.2 CLT Deck

9.2.2.1 Base Case

Depending on the span distance and number of spans, a CLT deck with a thickness varying between 100 mm and 300 mm is required, see Table 38. This table shows for different configurations of (sub)spans and number of spans the required minimum thickness of the CLT deck for each base requirement (ULS, ULS fire, SLS & vibrations, see Chapter 6). Based on Figure 107, it can be concluded that a 2-span system performs better compared to a single-span system for height or weight vs span ratio. A 3- or more multiple-span system however does not result in a further decrease of the height or weight of the CLT deck.

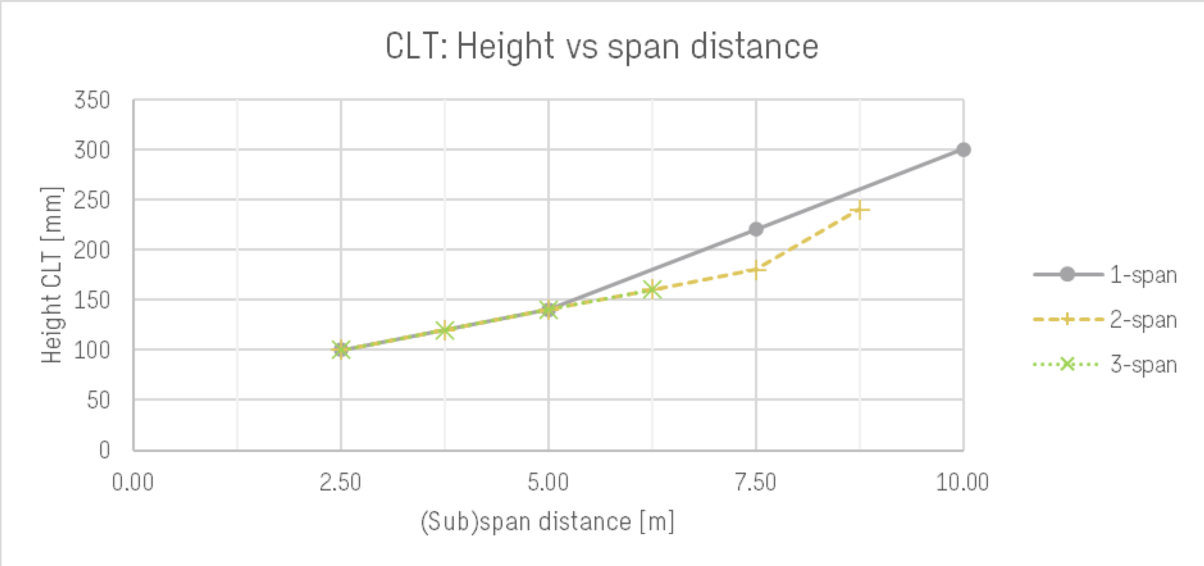


Figure 107. Comparison between single- and multi-span CLT systems for height vs span distance

Fire-safety requirements are a governing factor for almost all shorter spans up to 6.25 m. The minimum thickness of 100 mm is the result of fire-safety requirements, since a deck thinner than 100 mm will completely burn up within 90 minutes. Spans smaller than 2.5 m will therefore not result in a less thick deck. For decks thicker than 100 mm, bending under fire conditions is in most cases governing for spans up to 6.25 m.

Serviceability limits are governing in almost all configurations. Deformation limits are generally governing for spans longer than 6.25 m and in case of a single span system, also a 5 m span is already problematic. Strict requirements concerning vibrations are problematic for almost all configurations of CLT decks. For short and medium spans (up to ~6.25 m), especially the stiffness criterion is governing, while the frequency criterion is generally easily met. For longer spans (> 7.5 m), the frequency criterion becomes governing.

Table 38. CLT decks for various span distances and number of spans and required minimum height per requirement.

Name	Nr. sub-spans	L	Name$_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	Minimum height per requirement			
			BASE CASE							
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations
1x2.5m span	1	2.50	CLT 100 L3s	3	100	0.5	60	100	80	90
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.7	80	140	140	140
1x7.5m span	1	7.50	CLT 220 L7s - 2	7	220	1.1	110	160	220	220
1x10.0m span	1	10.00	CLT 300 L8s	8	300	1.5	140	180	300	300
2x2.5m span	2	2.50	CLT 100 L3s	3	100	0.5	60	100	60	90
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.6	60	120	90	120
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.7	80	140	110	140
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.8	90	160	160	160
2x7.5m span	2	7.50	CLT 180 L5s	5	180	0.9	110	160	180	180
2x8.75m span	2	8.75	CLT 240 L7s - 2	7	240	1.2	140	180	220	240
3x2.5m span	3	2.50	CLT 100 L3s	3	100	0.5	60	100	60	90
3x3.75m span	3	3.75	CLT 120 L3s	3	120	0.6	60	110	90	120
3x5.0m span	3	5.00	CLT 140 L5s	5	140	0.7	80	120	120	140
3x6.25m span	3	6.25	CLT 160 L5s	5	160	0.8	90	160	160	160
4x2.5m span	4	2.50	CLT 100 L3s	3	100	0.5	60	100	60	90
4x3.75m span	4	3.75	CLT 120 L3s	3	120	0.6	60	110	90	120
4x5.0m span	4	5.00	CLT 140 L5s	5	140	0.7	80	120	120	140
5x2.5m span	5	2.50	CLT 100 L3s	3	100	0.5	60	100	60	90
5x3.75m span	5	3.75	CLT 120 L5s	5	120	0.6	60	110	90	120
6x2.5m span	6	2.50	CLT 100 L3s	3	100	0.5	60	100	60	90

= *Normative height*

9.2.2.2 Alternative Requirements

From previous paragraph it has become clear, that the ULS requirements are not governing for the CLT decks, except in the case of fire (90 minutes). Table 38 already showed that large differences exist between the required minimum height of the CLT deck for the different requirements. Therefore, altering some of these requirements could result in a significant reduction of height and weight of the deck system.

Since serviceability limits are not required by the Dutch Building Degree, they can be altered by the engineer (Ministerie van Binnenlandse Zaken en Koninkrijkrelaties, 2021). The fire resistance class can be reduced to R 30 if the entire car park is considered as a single fire-compartment. This is possible based on the equivalence principle, which can be substantiated by special fire-safety calculations like zone-models as was discussed in paragraph 2.3.4. Also deformation limits can be altered, allowing bigger deformation than advised by the Eurocode. This is allowed by the Dutch Building Degree as discussed in paragraph 6.4. Last of all, also setting limitations related to vibrational behaviour of the structure of a car park is up to the designer as discussed in paragraph 2.3.3. In this paragraph, it is studied what are the effects of reduced requirements for fire-safety, SLS and vibrations on the required thickness of the CLT decks. One or more of these requirements are altered, while the other requirements remain the same (equivalent to the base case requirements).

In Figure 108, the effects of different alternative requirements on the height of CLT decks for a single-span system can be seen for various span distances. The same, but for multi-span systems can be seen in Figure 109. The results that are at the basis of these figures in tabular format can be found in Appendix F.

Reduced Fire-safety Requirements

Reducing the fire-safety requirements from R 90 to R 30 only results in a slight reduction of the deck height for the smallest span of 2.5 m. The result is a 10 mm (10%) reduction of the deck thickness and 0.05 kN/m^2 (10%) reduction in self-weight of the deck.

No check on vibrations

Removing the check on vibrations of CLT decks generally speaking has no effect on reducing the height and weight of the deck, since deformation limits or fire-safety requirements are still governing and don't allow reduction of the thickness of the deck.

Alternative Serviceability Limit Requirements

Removing the check on vibrations and allowing larger deflections of the decks results in a significant reduction in height and weight of the CLT decks for larger spans (≥ 7.5 m). The larger the span, the bigger the reduction of height and weight. For a span of 7.5 m, in case of a single-span, a reduction of 40 mm (18%) in height and 0.2 kN/m^2 (18%) in weight is achieved. In case of a multi-span system, 20 mm (11%) reduction in height and 0.1 kN/m^2 (11%) in weight are achieved. For longer spans, these number increase to 60 mm (20%) reduction in height for a single 10 m span and 0.3 kN/m^2 (20%) in weight.

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

When reduced fire-safety requirements are used as well as alternative serviceability limit requirements, a significant reduction of deck thickness and weight can be seen for all span distances. The biggest reduction in height and weight, 40 mm and 0.2 kN/m^2 (40%), is for the smallest span of 2.5 m. For other spans, the reduction in height and weight is in most cases around 20%.

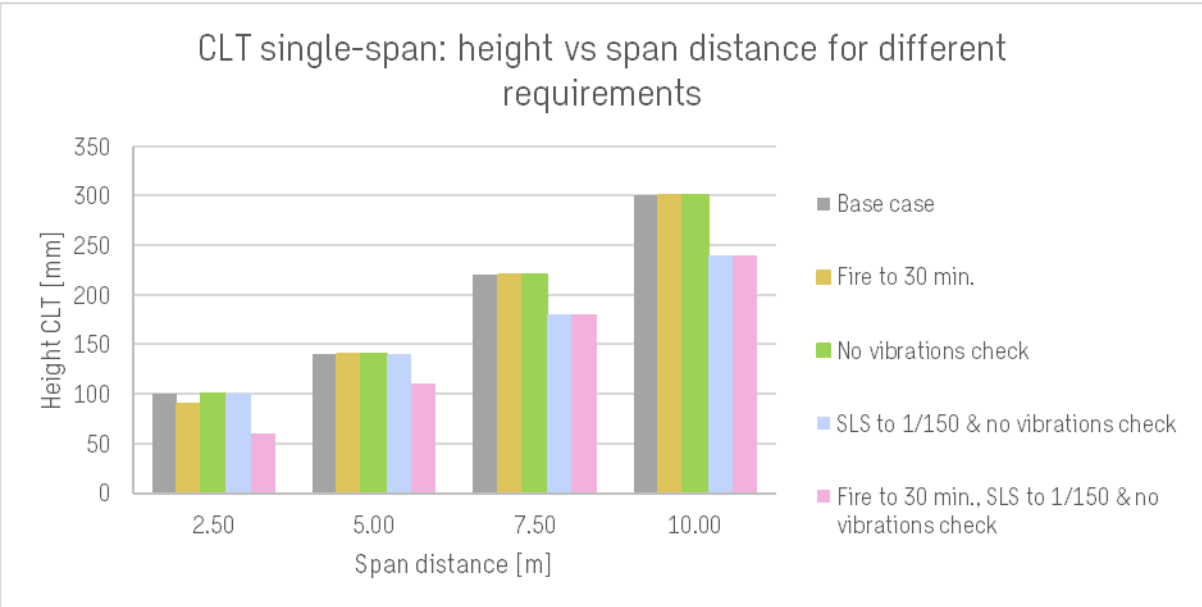


Figure 108. Differences in CLT deck thickness for various spans given alternative requirements (single-span)

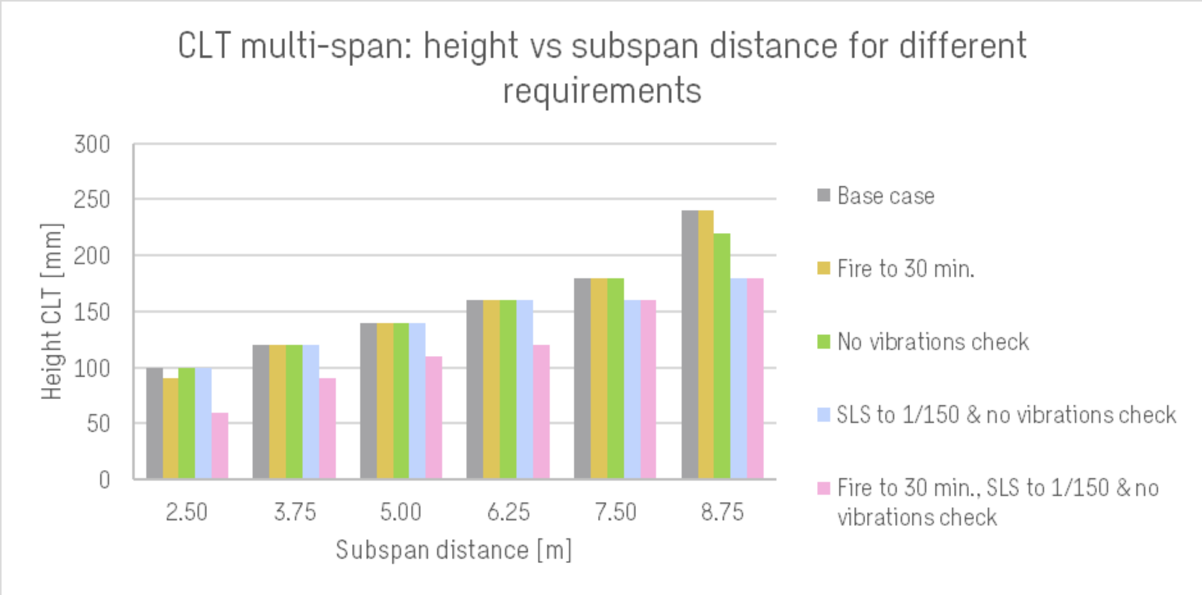


Figure 109. Differences in CLT deck thickness for various spans given alternative requirements (multi-span)

Conclusion

From the results of this paragraph, it follows that reducing the fire-safety requirements and using alternative serviceability limit requirements results in significant height and weight reductions for all spans. For longer spans (≥7.5 m), reduction of fire-safety requirements is not required to reach the same reduction. For smaller spans, significant reductions in height and weight of the CLT deck can only be achieved when both serviceability limits are altered as well as fire safety requirements.

The reduction in height and weight of the short-span deck system can be highly significant. The exact percentage of reduction depends on the exact configuration, but for very short spans (2.5 m), 40% reduction is possible and for other spans it varies between 11 and 25 %, with most configurations around 20%. This highlights the potential of weight and volume savings for deck systems depending on what requirements are stated.

9.2.3 LVL Deck

9.2.3.1 Base Case

The LVL deck is made up of several LVL panels glued together to create a thicker plate, which can span longer distances. For spans between 2.5 and 10 m, the total thickness of the panel varies between 96 and 294 mm, see Table 39. This table shows for different configurations of (sub)spans and number of spans the required minimum thickness of the LVL deck for each base requirement (ULS, ULS fire, SLS & vibrations, see Chapter 6). Based on Figure 110, it can be concluded that a 2-span system performs better compared to a single-span system for height or weight vs span ratio. A 3-span system performs similar to a two-span system, except for a 5 m sub-span, in which case it performs slightly worse than a two-span system, but better compared to a single-span system. This small difference is due to the deflection of the deck in case two fields are loaded by an imposed load and one field is not. In general, it is assumed that a multi-span system with two-spans performs best, since in most cases more sub-spans will not result in a further decrease of height and weight of the LVL deck.

In general, it is concluded that for LVL, often a single requirement is governing. Fire-safety requirements are the governing factor for all spans shorter than 5 m. The minimum thickness of 96 mm is the result of fire-safety requirements, since a deck thinner than 96 mm will completely burn up within 90 minutes. Spans smaller than 2.5m will therefore not result in a less thick deck.

Serviceability limits are governing for all configurations with spans of 5 m or more. Deformation limits are governing for almost all spans of 5 m or longer. Only for a span of 10 m, vibrations become the governing requirement, in which case the acceleration criterion is not met.

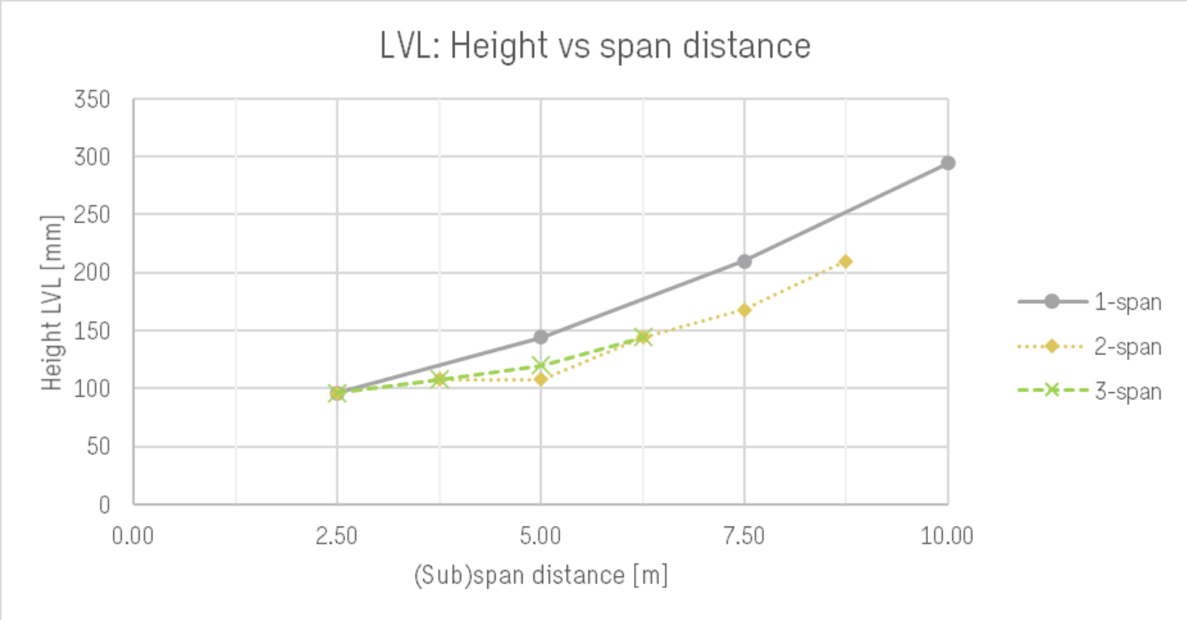


Figure 110. Comparison between single- and multi-span LVL systems for height vs span distance

Table 39. LVL decks for various span distances and number of spans and required minimum height per requirement.

Name	Nr. sub-spans	L	BASE CASE Name$_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	Minimum height per requirement ULS	ULS Fire	SLS	Vibrations
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m²]				
1x2.5m span	1	2.5	LVL G 96 2s	2	96	0.49	72	96	72	72
1x5.0m span	1	5.0	LVL G 144 3 s	3	144	0.73	72	108	144	72
1x7.5m span	1	7.5	LVL G 210 5s	5	210	1.07	90	120	210	72
1x10.0m span	1	10.0	LVL G 294 7s	7	294	1.50	126	168	288	294
2x2.5m span	2	2.5	LVL G 96 2s	2	96	0.49	72	96	72	72
2x3.75m span	2	3.8	LVL G 108 3s	3	108	0.55	72	108	84	72
2x5.0m span	2	5.0	LVL G 108 3s	3	108	0.55	72	108	108	72
2x6.25m span	2	6.3	LVL G 144 3s	3	144	0.73	72	120	144	72
2x7.5m span	2	7.5	LVL G 168 4s	4	168	0.86	90	126	168	72
2x8.75m span	2	8.8	LVL G 210 5s	5	210	1.07	108	144	210	120
3x2.5m span	3	2.5	LVL G 96 2s	2	96	0.49	72	96	72	72
3x3.75m span	3	3.8	LVL G 108 3s	3	108	0.55	72	108	84	72
3x5.0m span	3	5.0	LVL G 120 2s	2	120	0.61	72	108	120	72
3x6.25m span	3	6.3	LVL G 144 3s	3	144	0.73	72	120	144	72
4x2.5m span	4	2.5	LVL G 96 2s	2	96	0.49	72	96	72	72
4x3.75m span	4	3.8	LVL G 108 3s	3	108	0.55	72	108	84	72
4x5.0m span	4	5.0	LVL G 120 2s	2	120	0.61	72	108	120	72
5x2.5m span	5	2.5	LVL G 96 2s	2	96	0.49	72	96	72	72
5x3.75m span	5	3.8	LVL G 108 3s	3	108	0.55	72	108	84	72
6x2.5m span	6	2.5	LVL G 96 2s	2	96	0.49	72	96	72	72

= *Normative height*

9.2.3.2 Alternative Requirements

In previous paragraph concerned with CLT floors, it has already been stated that altering the requirements for serviceability limits and fire-safety is allowed and can result in significant reductions of floor thickness and weight. This is also the case for LVL deck systems as one could already see in Table 39. Therefore, in this paragraph, it is studied what are the effects of reduced requirements for fire-safety, SLS and vibrations on the required thickness of the LVL decks. One or more of these requirements are altered, while the other requirements remain the same (equivalent to the base case requirements).

In Figure 111, the effects of different alternative requirements on the height of LVL decks for a single-span system can be seen for various span distances. The same, but for multi-span systems can be seen in Figure 112. The results that are at the basis of these figures in tabular format can be found in Appendix F.

Reduced Fire-safety Requirements

Reducing the fire-safety requirements from R 90 to R 30 only results in a reduction of the deck height and weight for spans smaller than 5 m. This reduction however is significant. For a span of 2.5 m, a reduction of 24 mm (25%) and 0.12 kN/m^2 (25%) is achieved and for a 3.75 m span, a reduction of 24 mm (22%) and 0.12 kN/m^2 (22%) is achieved.

No check on vibrations

For short-span LVL decks, vibrations are much less of a problem, as can be seen in Table 39. Removing the check on vibrations of LVL decks therefore has generally speaking no effect on reducing the height and weight of the deck. Only for a 10 m single-span system, results removing the check on vibrations in a small reduction in height and weight of the deck.

Alternative Serviceability Limit Requirements

Removing the check on vibrations and allowing larger deflections of the decks results in a significant reduction in height and weight of the LVL decks for larger spans (≥ 6.25 m). For a span of 7.5 m, in case of a single-span, a reduction of 42 mm (20%) in height and 0.21 kN/m^2 (20%) in weight is achieved. In case of a multi-span system, 24 mm (14%) reduction in height and 0.13 kN/m^2 (15%) in weight are achieved. For longer spans, these numbers are similar with 54 mm (18%) reduction in height for a single 10 m span and 0.28 kN/m^2 (19%) in weight.

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

When reduced fire-safety requirements are used as well as alternative serviceability limit requirements, a significant reduction of deck thickness and weight can be seen for all span distances. The biggest reduction in height and weight 36 mm and 0.18 kN/m^2 (33%) is for a multi-span of 3.75 m. For other spans, the reduction in height and weight varies between 14 and 33%.

Conclusion

From the results of this paragraph, it follows that reducing the fire-safety requirements in combination with using alternative serviceability limit requirements results in significant height and weight reductions for all spans. For longer spans (≥6.25 m), reduction of fire-safety requirements is not required to reach the same reduction. For smaller spans, significant reductions in height and weight of the LVL deck can only be achieved when both serviceability limits are altered as well as fire safety requirements.

The reduction in height and weight of the short-span deck system can be highly significant. The exact percentage of reduction depends on the exact configuration, but varies between 14 and 33%. This highlights the potential of weight and volume savings for deck systems depending on what requirements are stated.

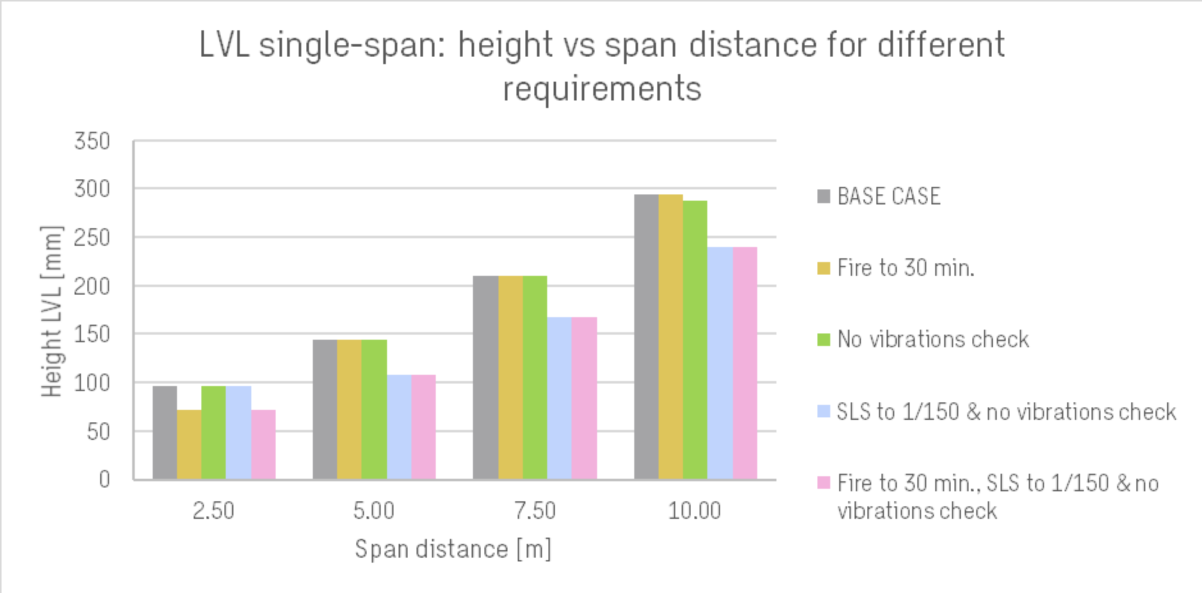


Figure 111. Differences in LVL deck thickness for various spans given alternative requirements (single-span)

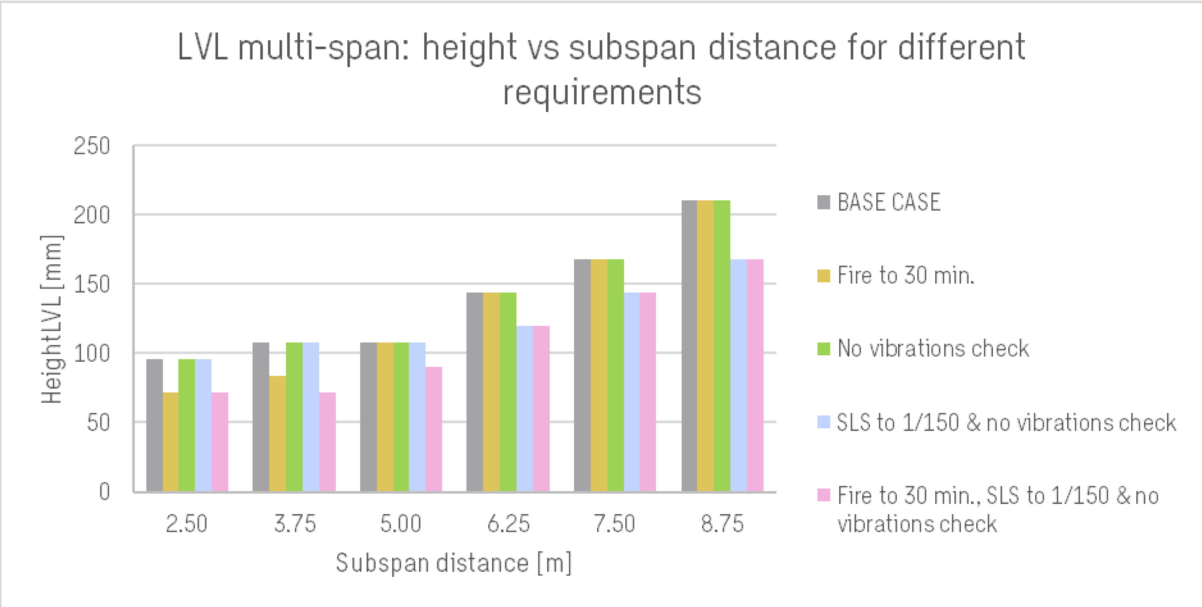


Figure 112. Differences in LVL deck thickness for various spans given alternative requirements (multi-span)

9.2.4 CLT vs LVL Deck Comparison

9.2.4.1 General

The CLT and LVL deck are compared to each other to see which deck system performs best considering the deck height and weight. From the analysis in previous paragraphs, one can already draw several conclusions.

- A 2-span system performs better than a single-span system for both types of decks;
- Generally speaking is the performance of a 3- or more multi-span system no better than a 2-span system for both types of decks;
- For CLT decks, often multiple requirements are governing for the minimum thickness of the deck, while for LVL decks, in most cases, a single requirement is governing;
- For both types of decks, fire-safety requirements are governing for spans shorter than ~6.25 m and the minimum thickness as a result of fire-safety requirements lies around 100 mm;
- Serviceability requirements are governing for spans longer than roughly 6.25 m for both types of decks, however in the case of CLT decks, both deformation limits and vibration criteria are governing, while for LVL decks, vibration criteria in general are not governing.

9.2.4.2 Base Case

A further comparison between the CLT and LVL deck is made for the differences in height between both deck types in Figure 113 and Figure 114 as well as for weight in Figure 115 and Figure 116. In these figures, the performance of the decks is compared for different configurations of spans as well as for different situations of alternative requirements. In Appendix F, tables can be found with the exact values and percentages in differences of performance of the CLT and LVL decks.

Focussing on the base case, Figure 113 shows that LVL performs slightly better for most single-spans. For a span of 2.5, 7.5 and 10 m, the LVL deck is between 4 and 11 mm less thick. For a span of 5 m however, the LVL deck is 4 mm thicker. These differences in height vary between -3 and 5%. For multi-span systems, the difference in height between CLT and LVL decks is more significant and always in the advantage of the LVL deck as can be seen in Figure 114. The biggest difference is found for a sub-span of 5 m, in which case the LVL deck is 32 mm thinner, which is a reduction of 23% compared to the CLT deck. For the other spans, the LVL deck is between 4 and 30 mm less thick, which is between 4 and 13% less thick than a CLT deck.

Looking at the weight of the decks, broadly the same picture emerges, see Figure 115 and Figure 116. The LVL deck performs slightly better for a single-span of 2.5 and 7.5 m and slightly worse for 5 m. For a multi-span system, the LVL deck always performs better compared to a CLT deck. In percentage terms, the weight reduction is between 1 and 3 percentage points behind the height reduction. In absolute terms, the maximum weight reduction is 0.15 kN/m^2 for a 5 m sub-span.

Given the better performance of the LVL panels for a multi-span system, it is the preferred choice for the short-span deck system under the base case scenario. Table 40 presents the choice of LVL panel for the various short-spans.

Table 40. Choice of prefab short-span deck system for various parking deck spans for base case.

			VARIANT 2A, 2B & 3 - BASE CASE			
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h$_{LVL}$**	**g$_{0,k}$**
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**
2x2.5m span	2	2.50	LVL G 96 2s	2	96	0.49
2x3.75m span	2	3.75	LVL G 108 3s	3	108	0.55
2x5.0m span	2	5.00	LVL G 108 3s	3	108	0.55
2x6.25m span	2	6.25	LVL G 144 3s	3	144	0.73
2x7.5m span	2	7.50	LVL G 168 4s	4	168	0.86

9.2.4.3 Alternative Requirements

Figure 113 through Figure 116 also show the difference in performance between the CLT and LVL deck for situations with alternative requirements. The instances for which a alternative set of requirements results in a significant difference in result from the base case are discussed.

Reduced Fire-safety Requirements

When the fire-safety requirements are reduced from 90 to 30 min., the performance of the LVL deck increases significantly compared to the CLT deck for shorter spans (≤ 3.75 m). For a span of 2.5 m, the height of the deck is reduced by 14 mm more compared to the base case and for a span of 3.75 m even 24 mm. In terms of weight reduction, the reduced fire-safety requirements result in a decrease in weight of 0.1 kN/m^2 for a span of 2.5 or 3.75 m. For other span distances, the reduced requirements do not result in changes compared to the base case as can be seen in Table 41.

Table 41. Choice of prefab short-span deck system for various parking deck spans for alternative requirements, fire to 30 min.

			Fire to 30 min.			
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h$_{LVL}$**	**g$_{0,k}$**
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**
2x2.5m span	2	2.5	LVL G 72 1s	1	72	0.37
2x3.75m span	2	3.75	LVL G 84 2s	2	84	0.43
2x5.0m span	2	5	LVL G 108 3s	3	108	0.55
2x6.25m span	2	6.25	LVL G 144 3s	3	144	0.73
2x7.5m span	2	7.5	LVL G 168 4s	4	168	0.86

No check on vibrations

Removing the check on vibrations only results in changes compared to the base case for spans of 8.75 m or longer. For a multi-span system of 8.75 m, the difference in height and weight between the CLT and LVL deck significantly reduces. In terms of height, the advantage of LVL over CLT is reduced from an advantage of 30 mm to only 10 mm. For a single-span of 10 m however, the advantage of LVL increases slightly from 6 to 12 mm.

Alternative Serviceability Limit Requirements

Removing the check on vibrations and allowing larger deflections of the decks results in two interesting points. For a single-span of 5 m and a multi-span of 6.25 m, the performance of the LVL deck increases by a very large amount compared to the CLT deck. For the 5 m span, a reduction in height of 36 mm is realised compared to the base case. The cause of this large difference is that for the LVL deck, the unity checks for the base case are relatively low (~ 70%), but a smaller type LVL deck is not possible. For the situation with less strict deformation limits and no vibration check however, the unity checks are very high (> 95%). Meanwhile, for the CLT deck, no change in thickness between both situations is realised. Together this results in the very large difference in performance between the CLT and LVL deck for this situation. One other case, in which the performance significantly differs from the base case is a multi-span of 8.75 m. In this case, the difference between CLT and LVL deck is much smaller compared to the base case. Given these alternative requirements, Table 42 presents the preferred choice of deck system for various multi-span distances.

Table 42. Choice of prefab short-span deck system for various parking deck spans for alternative requirements, SLS to 1/150 & no vibrations check.

			SLS to 1/150 & no vibrations check			
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h$_{LVL}$**	**g$_{0,k}$**
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**
2x2.5m span	2	2.5	LVL G 96 2s	2	96	0.49
2x3.75m span	2	3.75	LVL G 108 3s	3	108	0.55
2x5.0m span	2	5	LVL G 108 3s	3	108	0.55
2x6.25m span	2	6.25	LVL G 120 2s	2	120	0.61
2x7.5m span	2	7.5	LVL G 144 3s	3	144	0.73

Reduced Fire-safety Requirements & Alternative Serviceability Limit Requirements

Reducing the fire-safety requirements as well as the serviceability limit requirements results in multiple changes in the difference in performance of the CLT and LVL decks. Sometimes, the difference between CLT and LVL deck performance becomes bigger and sometimes smaller. This is highly dependent on the exact configuration of spans. In general, it can be concluded that the LVL deck performs better for all spans larger than 2.5 m, resulting in the choice of panels for the various spans as presented in Table 43.

Table 43. Choice of prefab short-span deck system for various parking deck spans for alternative requirements, fire to 30 min., SLS to 1/150 & no vibrations check.

			Fire to 30 min., SLS to 1/150 & no vibrations check			
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h_{LVL}**	**$g_{0,k}$**
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m²]**
2x2.5m span	2	2.5	CLT 60 L3s	3	60	0.30
2x3.75m span	2	3.75	LVL G 72 1s	1	72	0.37
2x5.0m span	2	5	LVL G 90 3s	3	90	0.46
2x6.25m span	2	6.25	LVL G 120 2s	2	120	0.61
2x7.5m span	2	7.5	LVL G 144 3s	3	144	0.73

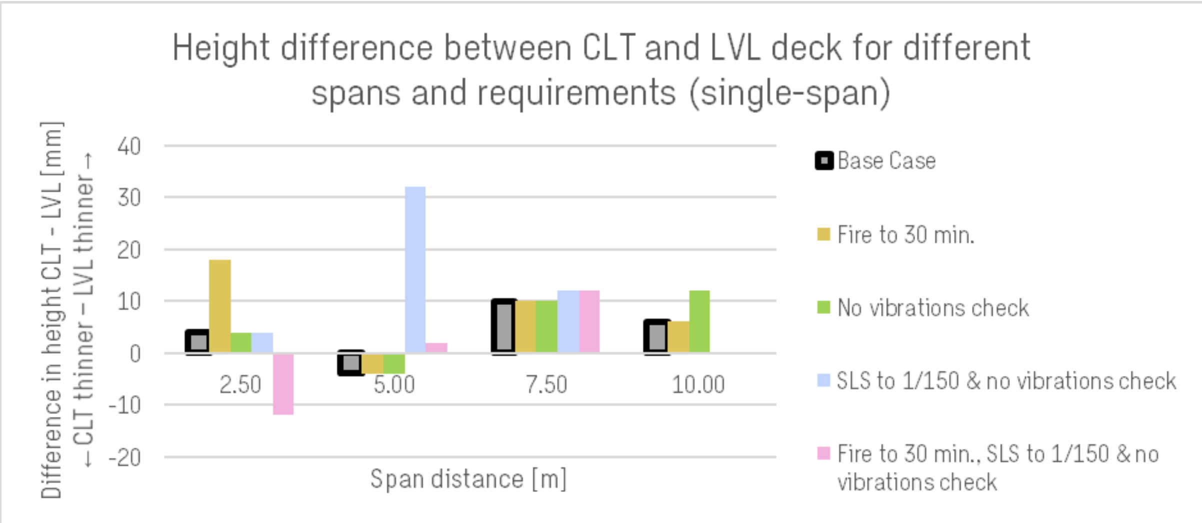


Figure 113. Comparison in height between CLT and LVL deck for different spans and requirements (single-span).

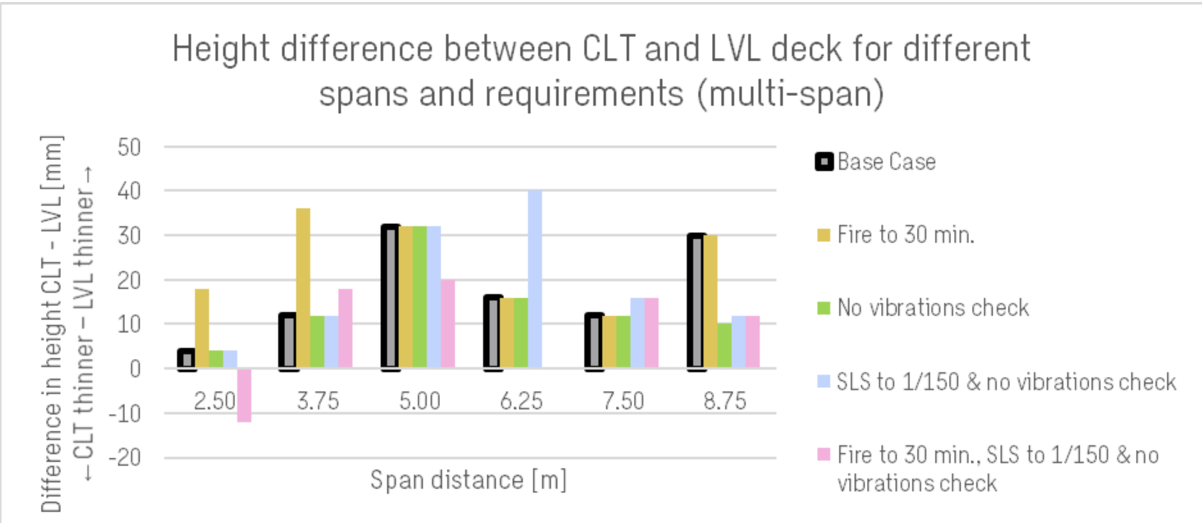


Figure 114. Comparison in height between CLT and LVL deck for different spans and requirements (multi-span).

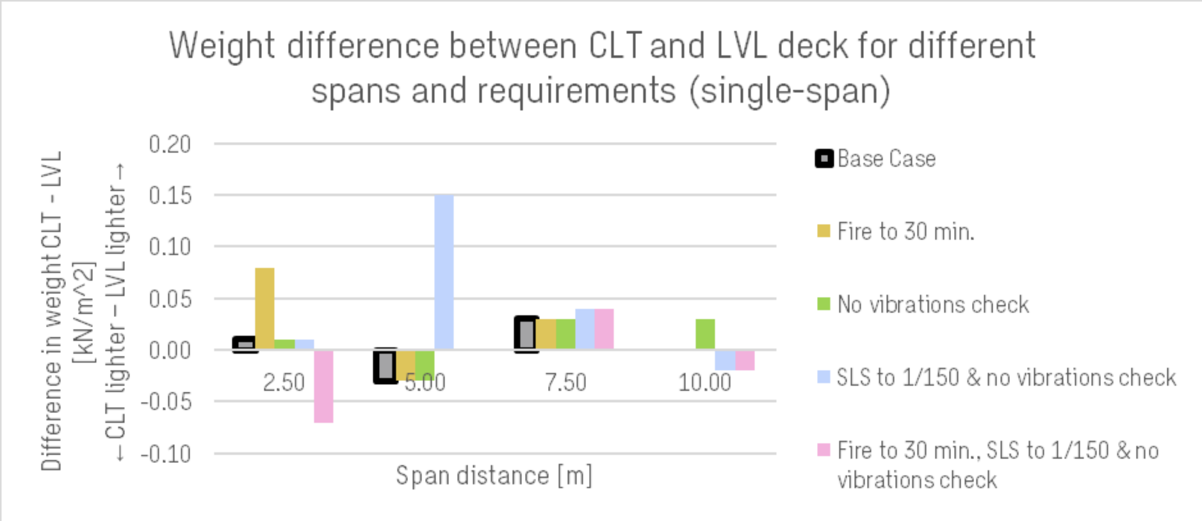


Figure 115. Comparison in weight between CLT and LVL deck for different spans and requirements (single-span).

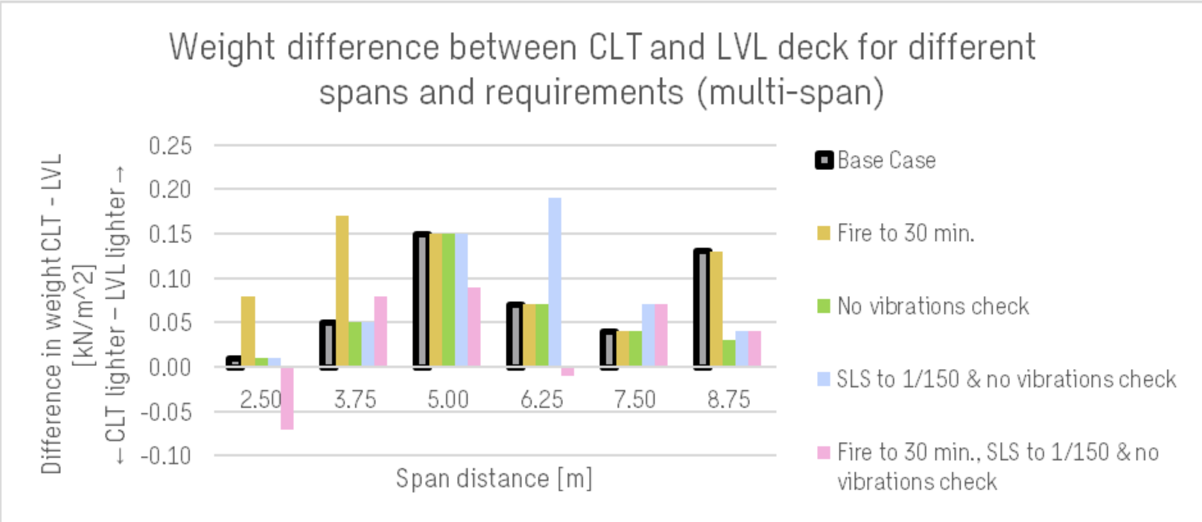


Figure 116. Comparison in weight between CLT and LVL deck for different spans and requirements (multi-span).

9.2.4.4 Conclusion

In conclusion, it can be stated that the LVL deck performs better in almost all situations, both for the base case scenario as well as for different sets of requirements. Especially for multi-span systems, the LVL deck always performs better (except 2.5 m span with different set of requirements). The amount by which the LVL deck outperforms the CLT deck depends highly on the exact configuration of spans and requirements, but it can amount to a difference in height of 36 mm (30%) and weight of 0.17 kN/m^2 (28%). Therefore, in most cases, the better choice in performance of height and weight of the deck for short-spans is an LVL deck.

9.3 Concluding Remarks

In this chapter, various long- and short-span prefab deck systems in CLT and LVL have been considered. These have been studied for different span lengths and for the short-span systems for different numbers of spans. Furthermore, they have been studied for a base case situation, in which case the standard requirements for ULS and SLS are considered (as mentioned in chapter 6), as well as for alternative situation in which these requirements have been altered. Based on the results, a choice has been made for the different design variants, which deck system to use in which situation. In conclusion of this chapter, sub-question 3c can be partially answered: “What is the impact of altering geometric parameters and limit state requirements on the performance of the different designs for a temporary multi-storey car park using timber as primary structural material?”. Furthermore, some of the hypotheses stated in paragraph 5.1.3.1 can be tested.

9.3.1 Long-span deck system

The long-span deck system is used for design variant 1. For the long-span deck systems, in the base case scenario, the CLT open rib panel and CLT closed rib panel systems outperform the other systems. The solid CLT panel and most LVL deck systems cannot even cover most spans considering their standard production dimensions for the base case scenario. Depending on the exact parking deck span, one of these systems is chosen. The chosen type, its total deck height and weight can be found in Table 44.

Considering the alternative requirements, several conclusions can be drawn. For the various decks using either CLT or LVL, vibrational criteria are always governing for long-span deck systems. If these are not considered, a significant reduction in height and weight is found for all spans considered as can be seen in Table 45. A reduction in height of up to 14% (103 mm for 17.66 m span) is possible and up to 49% (0.92 kN/m for 17.66 m span) in weight. After the vibrational criteria, the deformation limits are governing. Again allowing larger deformation results in a significant decrease of total deck height and weight for the various long-span deck systems. Relatively to the base case, a height reduction of up to 31% (253 mm for 17.66 m span) is possible and reduction in weight of up to 49% (0.91 kN/m for 17.66 m span). Reduction of the fire-safety class to R30 has no (significant) effect on the total deck height and weight of the long-span deck systems. Under alternative requirements, the various spans considered can also be covered by the solid CLT panel and various LVL deck systems, which was not possible under the base case scenario. In this case, the CLT systems are no longer the best performing ones. The LVL semi-open rib panel and LVL closed rib panel are now the better performing systems. Again, depending on the parking deck span, one of these systems is chosen, see appendix F.3.

Table 44. Choice of deck system for design variant 1.

	VARIANT 1 - BASE CASE			
Span [m]	**Deck type** [-]	h_{tot} [mm]	$g_{0,k}$ [kN/m]	$g_{0,k,tot(2500mm)}$ [kN/m]
12.18	Open CLT deck: 120mm CLT - 4 ribs 360x200mm	480	1.18	2.95
13.70	Open CLT deck: 120mm CLT - 4 ribs 450x200mm	570	1.32	3.30
14.56	Open CLT deck: 120mm CLT - 4 ribs 500x200mm	620	1.40	3.50
15.61	Open CLT deck: 120mm CLT - 4 ribs 600x200mm	720	1.56	3.90
16.26	Closed CLT deck: 120mm CLT - 4 ribs 500x140mm - 120mm CLT	740	1.76	4.40
17.16	Open CLT deck: 120mm CLT - 4 ribs 700x200mm	820	1.72	4.30
17.66	Closed CLT deck: 120mm CLT - 4 ribs 600x140mm - 120mm CLT	840	1.87	4.68

Table 45. Performance of choice of long-span deck systems under alternative requirements relative to base case.

	No vibrations check				SLS to 1/150 & no vibrations check			
Span	Δh_{tot}		$\Delta g_{0,k}$		Δh_{tot}		$\Delta g_{0,k}$	
[m]	[mm]	[%]	[kN/m]	[%]	[mm]	[%]	[kN/m]	[%]
12.18	-17	-4%	0.36	31%	143	30%	0.35	30%
13.70	33	6%	0.39	30%	133	23%	0.44	33%
14.56	33	5%	0.53	38%	123	20%	0.58	41%
15.61	83	12%	0.57	37%	223	31%	0.65	42%
16.26	103	14%	0.77	44%	203	27%	0.83	47%
17.16	83	10%	0.77	45%	233	28%	0.85	49%
17.66	103	12%	0.92	49%	253	30%	0.91	49%

9.3.2 Short-span deck system

For the design variants 2a, 2b and 3, short-span deck systems are used. For the short-span deck system, only two types of decks were considered: solid CLT panel or solid LVL panel. These can either be single-span or multi-span systems. It was concluded that a multi-span system performs better and that two-spans are sufficient. Therefore, a two-span short-span deck system is chosen for these design variants.

In the base case scenario, the LVL deck outperforms the CLT deck for all span distances (short-spans). The chosen short-span deck systems for the various span configurations can be found in Table 46. Also for most other scenario's does the LVL deck outperform the CLT deck and therefore is chosen as short-span deck system.

Considering the alternative requirements, also several conclusions can be drawn. The results have shown that for the short-span deck systems, for various spans, different criteria are governing. For both CLT and LVL short-span deck systems, in most cases, all criteria have to be altered to result in a reduction of total deck height and weight. For the smallest span, the criteria for fire-safety can be governing and for the longest span often vibrational criteria are governing. In between, multiple criteria including deformations are often governing. Nevertheless, the LVL deck also outperforms the CLT deck under alternative criteria. In Table 47, the preferred choice of short-span deck system is presented for various combinations of alternative requirements. When all requirements are altered, a reduction in deck height and weight of up to 39% is possible. However, in absolute terms, the maximum height reduction of the deck is only 36 mm, which is rather small in comparison to the possible height reductions for the long-span deck systems. Also in terms of reduction in weight, the possible reductions are much smaller compared to the savings for the long-span deck systems.

Table 46. Choice of deck system for variants 2a, 2b and 3.

			VARIANT 2A, 2B & 3 - BASE CASE			
Name	Nr. sub-spans	L	$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m^2]
2x2.5m span	2	2.50	LVL G 96 2s	2	96	0.49
2x3.75m span	2	3.75	LVL G 108 3s	3	108	0.55
2x5.0m span	2	5.00	LVL G 108 3s	3	108	0.55
2x6.25m span	2	6.25	LVL G 144 3s	3	144	0.73
2x7.5m span	2	7.50	LVL G 168 4s	4	168	0.86

Table 47. Performance of choice of short-span deck systems under alternative requirements relative to base case.

	Reduced fire-safety requirements				No vibrations check				SLS to 1/150 & no vibrations check				Fire to 30 min., SLS to 1/150 & no vibrations check			
Span	Δh_{tot}		$\Delta g_{0,k}$		Δh_{tot}		$\Delta g_{0,k}$		Δh_{tot}		$\Delta g_{0,k}$		Δh_{tot}		$\Delta g_{0,k}$	
[m]	[mm]	[%]	[kN/m]	[%]	[mm]	[%]	[kN/m]	[%]	[mm]	[%]	[kN/m]	[%]	[mm]	[%]	[kN/m]	[%]
2.50	24	25%	0.12	24%	0	0%	0.00	0%	0	0%	0.00	0%	36	38%	0.19	39%
3.75	24	22%	0.12	22%	0	0%	0.00	0%	0	0%	0.00	0%	36	33%	0.18	33%
5.00	0	0%	0.00	0%	0	0%	0.00	0%	0	0%	0.00	0%	18	17%	0.09	16%
6.25	0	0%	0.00	0%	0	0%	0.00	0%	24	17%	0.12	16%	24	17%	0.12	16%
7.50	0	0%	0.00	0%	0	0%	0.00	0%	24	14%	0.13	15%	24	14%	0.13	15%

9.3.3 Hypotheses Evaluation

The various deck systems have also been studied for what the effect of changing the requirements for ULS and SLS are on their total deck height and weight. Given the results of the long- and short-span deck systems, the hypotheses concerned with these aspects can now be tested.

H5a: Applying a lower fire resistance class to the structural system of the multi-storey car park results in a decrease of the total deck height.

H5b: Applying a lower fire resistance class to the structural system of the multi-storey car park results in a decrease of the total construction weight.

A reduction in fire resistance class has shown no or very little effect on the total deck height and weight of long-span deck systems. Therefore, hypotheses H5a and H5b cannot be confirmed for long-span deck systems. For short-span systems however, fire-safety requirements are often a governing requirement, although often in combination with other criteria. Especially for the smallest spans, can a reduction in required fire resistance class result in a decrease of total deck height and weight, especially in combination with reduction of SLS criteria. Therefore, for short-span deck systems, hypotheses H5a and H5b can be confirmed, but only for certain spans and combinations of requirements.

H6a: Setting lower or no criteria for the vibrational behaviour of the structural system of the multi-storey car park will result in a decrease of the total deck height.

H6b: Setting lower or no criteria for the vibrational behaviour of the structural system of the multi-storey car park will result in a decrease of the total construction weight.

For long-span deck systems, the criteria for vibrations are almost always governing. Reducing or removal of these requirements therefore can result in a significant reduction of total deck height and weight. This confirms hypotheses H6a and H6b for long-span deck systems. For short-span deck systems however, the criteria for vibrations are not always governing. For CLT decks, multiple criteria are governing and therefore, to reduce total deck height and weight, multiple criteria should be altered. For LVL decks, vibrational criteria are never governing for short-span decks. Therefore, hypotheses H6a and H6b cannot be confirmed for short-span decks.

H7a: Allowing larger deflections for the structural elements of the car park will result in a decrease of the total deck height.

H7b: Allowing larger deflections for the structural elements of the car park will result in a decrease of the total construction weight.

After the vibrational criteria, the requirements for deflections are governing for all long-span deck systems. Reducing them results in a significant reduction of total deck height and weight, confirming hypotheses H7a and H7b. Also for short-span deck systems, can reduction of the deflection limits result in a significant decrease in total deck height and weight, both when vibrational criteria are governing and when not. This confirms hypotheses H7a and H7b for short-span deck systems.

10
Framing System

The framing system is one of the main parts of the structure of the multi-storey car park. Together with the decks, it makes up the structural system of the car park. In paragraph 5.3.2, four different framing systems (design variants) have been presented for the development of the proof of concept for the modular, temporary multi-storey car park in timber. For each of these variants, also several parameters were stated, which could be altered, resulting in multiple subvariants. Examples of such parameters are the parking deck span and column distance. In this chapter, these (sub)variants are first structurally analysed and afterwards compared to each other for their performance on the total deck height, total deck weight and column load on the foundation, as discussed in paragraph 5.1.3.

10.1 Design subvariants

The four different main variants for the framing system were already presented in paragraph 5.3.2. Furthermore, in paragraph 5.1.3, different parameters were presented, which could be altered, resulting in a different structural system that might perform differently. Four of these parameters are used to alter the four main variants for the framing system (if applicable to the considered design variant), creating various subvariants. These parameters are: parking deck span, column distance (perpendicular to parking deck span), use of struts and distance between joists. In combination with the results from the deck systems in chapter 9, it is chosen which parameters are used for each (sub)variant. In total, 77 subvariants are analysed (detailed results in appendix G). The different subvariants are explained below for each main variant.

10.1.1 Variant 1

The first design variant, variant 1, is studied for seven different parking deck spans (12.18, 13.70, 14.56, 15.61, 17.16 and 17.66 m as discussed in paragraph 0). These include various long-span prefab deck systems. Furthermore, different column distances are considered. The columns are either placed at a width of one parking bay (2.5 m) or two parking bays (5.0 m). Larger distances are possible, but will probably result in a significant increase of the cross-section of the transverse main girder, which is not required. Therefore, only these two distances are considered. Using this set of parameters, a total of 14 subvariants are created for design variant 1, see Figure 117.

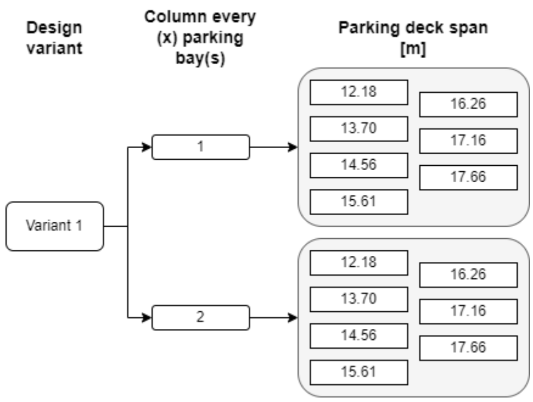


Figure 117. Overview of subvariants design variant 1.

10.1.2 Variant 2a

For design variant 2a, again seven different parking deck spans are studied. Furthermore, different column distances are considered. The columns are either placed at a width of one parking bay (2.5 m) or two parking bays (5.0 m). For larger distances, the height of the short-span parking deck will significantly increase as was found in paragraph 9.2 and also the height of the main girder is expected to increase. Therefore, only these two distances are considered. Finally, it is also studied what the effect is of the use of struts. Using this set of parameters, a total of 28 subvariants are created for design variant 2a, see Figure 118.

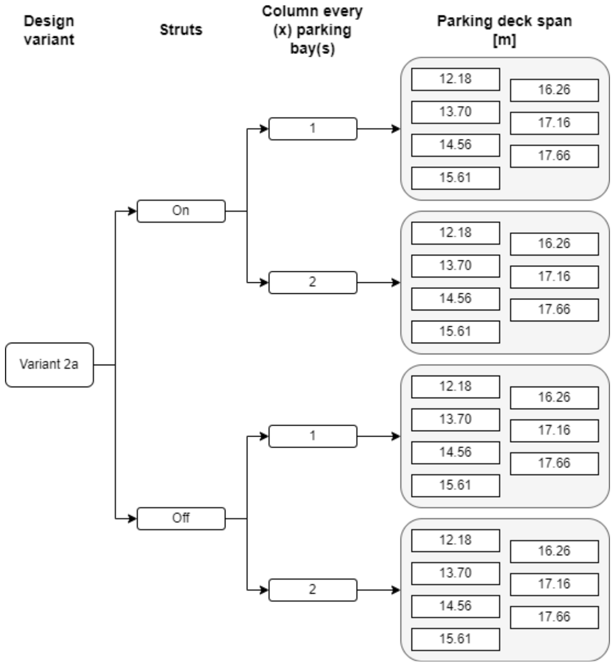


Figure 118. Overview of subvariants design variant 2a.

10.1.3 Variant 2b

In the case of design variant 2b, the only parameter that is varied is the parking deck span. The same seven distances, used for the other variants are also studied for this variant. The columns are placed at a distance of 5 m. A smaller distance would not result in a thinner parking deck as a result of the minimum thickness for fire-safety (see paragraph 9.2) and therefore is not beneficial. For larger distances, the transverse main girders would significantly increase in cross-section. The variation of the parking deck span results in a total of 7 different subvariants, see Figure 119.

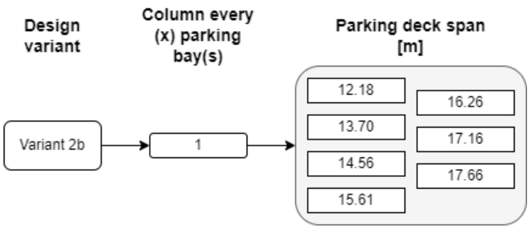


Figure 119. Overview of subvariants design variant 2b.

10.1.4 Variant 3

For the final main variant, design variant 3, three parameters are varied. First of all, again seven different parking deck spans are considered. Second, the effect of using struts is studied. Third, the number of joists is altered. This number depends on the parking deck span, but the distance between them is either 2.50 m at minimum or 3.75 m at minimum. A shorter span would not result in a decrease of deck thickness and weight and therefore is not beneficial. For a longer span, the deck could also directly span the distance between the main girders, since they are located at a distance of 5.0 m from each other. Using this set of parameters, a total of 28 subvariants are created for design variant 3, see Figure 120.

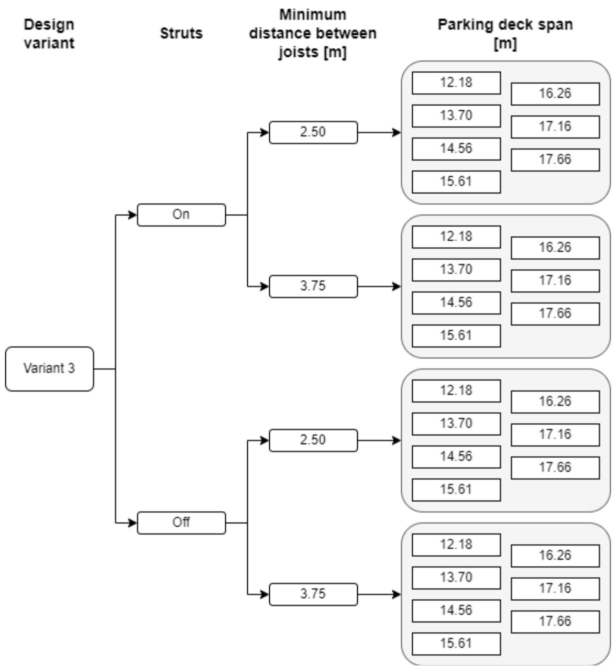


Figure 120. Overview of subvariants design variant 3.

10.2 Schematisation Structural System

To be able to structurally analyse the different subvariants, their structural systems have to be schematised. It is chosen to schematise the structural systems in (if required, multiple) 2D frames. The choice has been made to create the model in 2D, since a 3D-model would add a lot of complexity to both the creation of the parametric model as well as during the structural analysis, which is now avoided. The various 2D schematizations of the different main design variants are presented below, each containing all different structural elements present in the variant considered.

10.2.1 Variant 1

10.2.1.1 Structural elements

In Figure 121, at the bottom, the 2D schematisation with the different structural elements of design variant 1 is presented. At the top of the figure, a 3D drawing of the variant has been presented as well. View A contains the most important primary structural elements, used to transfer the loads from the floors to the foundation. In variant 1, the long-span prefab deck elements are used as concluded in paragraph 9.3. The loads coming from the deck elements are transferred to the transverse main girders, which again transfer their loads to the columns. The 2D-model of view A contains an additional girder as the top level railing, which is used as an extra element, to which the façade can be connected. Furthermore, a wind bracing in a single direction is modelled to act as a stability element against wind loading perpendicular to the parking deck span. It is modelled as a tension only element and in reality another set of wind bracings are mounted in the other direction as well.

Most structural elements of view B cannot be seen in the 3D overview. These elements act as the stabilizing structure in the other direction and are located in the façades at every 40 m width of the car park. They contain a set of columns, side girders, top level railings and wind bracings.

All columns are modelled as continuous elements. The transverse main girders and railing top level – x are modelled as simply supported beams in case of a span of 2.5 m and as continuous in case of a span of 5.0 m, to limit the deformations. The side girders and railing top level – y are all modelled as simply supported beams. All these elements are made of rectangular glulam timber sections. The wind bracings are made of steel L-sections, which are connected by hinges to the nodes of the structure. Finally, all columns are pinned supported to the foundations.

10.2.1.2 Loading

In terms of loading, the permanent loads (self-weight and finishing layer) coming from the floors are modelled as line loads on the transverse main girders. On the top transverse main girder, an additional line load for snow is modelled as well as a line load for wind pressure on the top parking deck. The self-weight of the other structural elements is later modelled by SCIA. Wind loading from both directions on the structure is modelled as point loads on the nodes between columns and transverse main girders/top level railings/side girders. These point loads include wind forces coming from pressure and suction on the façades as well as friction on the roof and façades. Finally the self-weight of the façade is modelled as a line load on the transverse main girders, side girders and top level railings, but in this case, these elements carry the loads of only halve a parking deck.

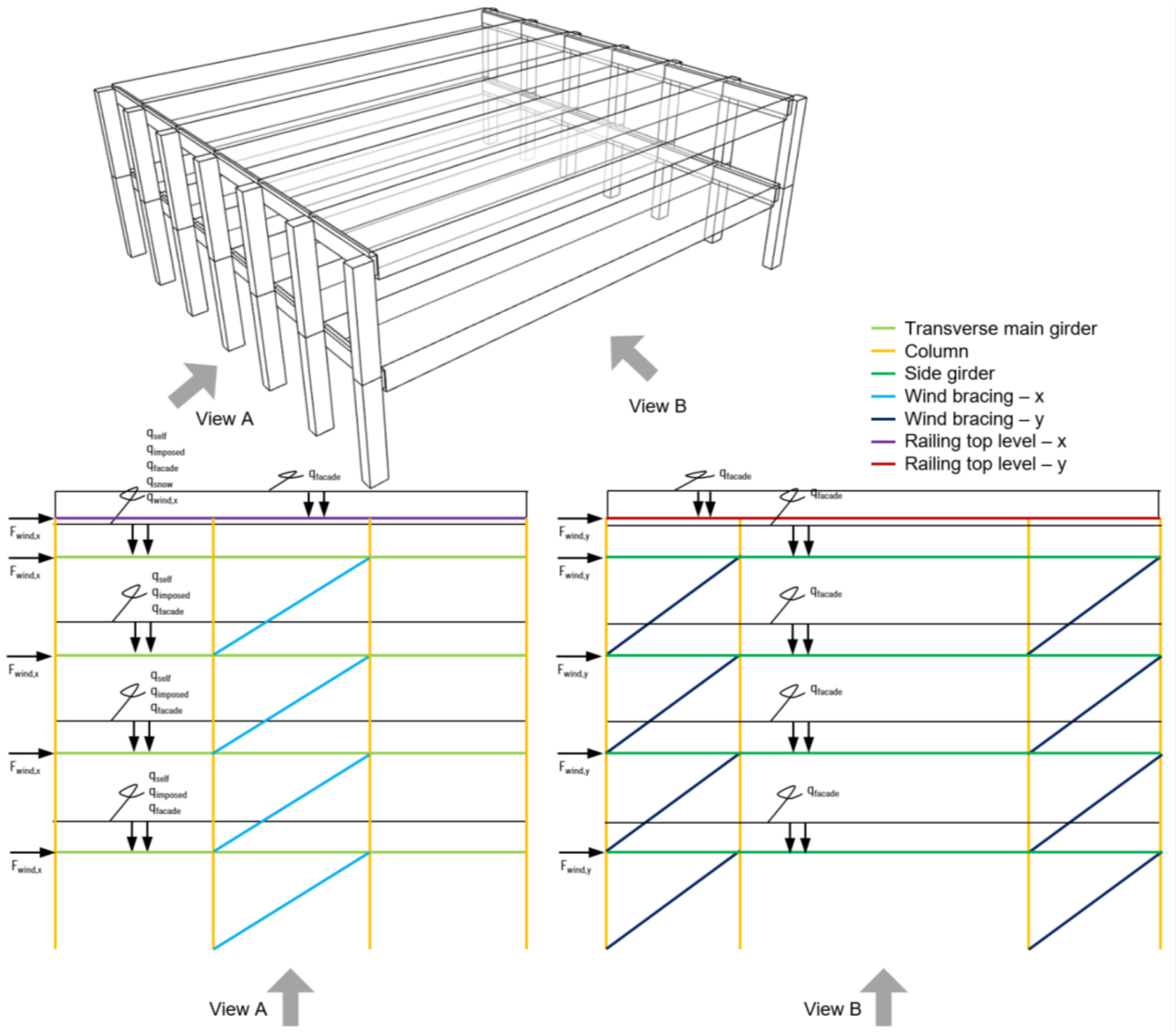


Figure 121. 2D schematisation variant 1.

10.2.2 Variant 2a

10.2.2.1 Structural elements

The schematisation of the structural system of design variant 2a is presented in

Figure 122. For this variant, the schematisation at the bottom of the figure presents the structural elements of the main load bearing structure, transferring the loads from the parking decks to the foundation. The parking decks are made of the short-span prefab deck panels as presented in paragraph 9.3. They transfer their loads to the main girders, spanning the parking deck. They then transfer the loads further to the columns and optionally (dependant on the subvariant) also to the struts, which shorten the span of the main girders.

The two top 2D schematisations are modelled to analyse the stabilizing elements of the structure. At the left, a wind bracing cross at every 40 m stabilizes the structure against wind loading, which acts perpendicular to the parking deck span. Only one wind bracing is modelled, which is a tension only member. Transverse main girders are modelled to support the façade elements, but depending on its weight, it can also be possible to connect the façade elements to the deck system. To stabilize the structure in the direction parallel to the parking deck span, another set of wind bracings are modelled, which are required for every three parking deck spans. The side girders than replace the main girder, supporting the parking deck.

All columns are modelled as continuous elements. The main girders are simply supported in the connection to the columns and are continuous over the struts. The transverse main girders and railing top level – x are modelled as simply supported beams in case of a span of 2.5 m and as continuous in case of a span of 5.0 m, to limit the deformations. The side girders and railing top level – y are all modelled as simply supported beams. The struts are connected by hinges to the main girder and column. All these elements are made of rectangular glulam timber sections. The wind bracings are made of steel L-sections, which are connected by hinges to the nodes of the structure. Finally, all columns are pinned supported to the foundations.

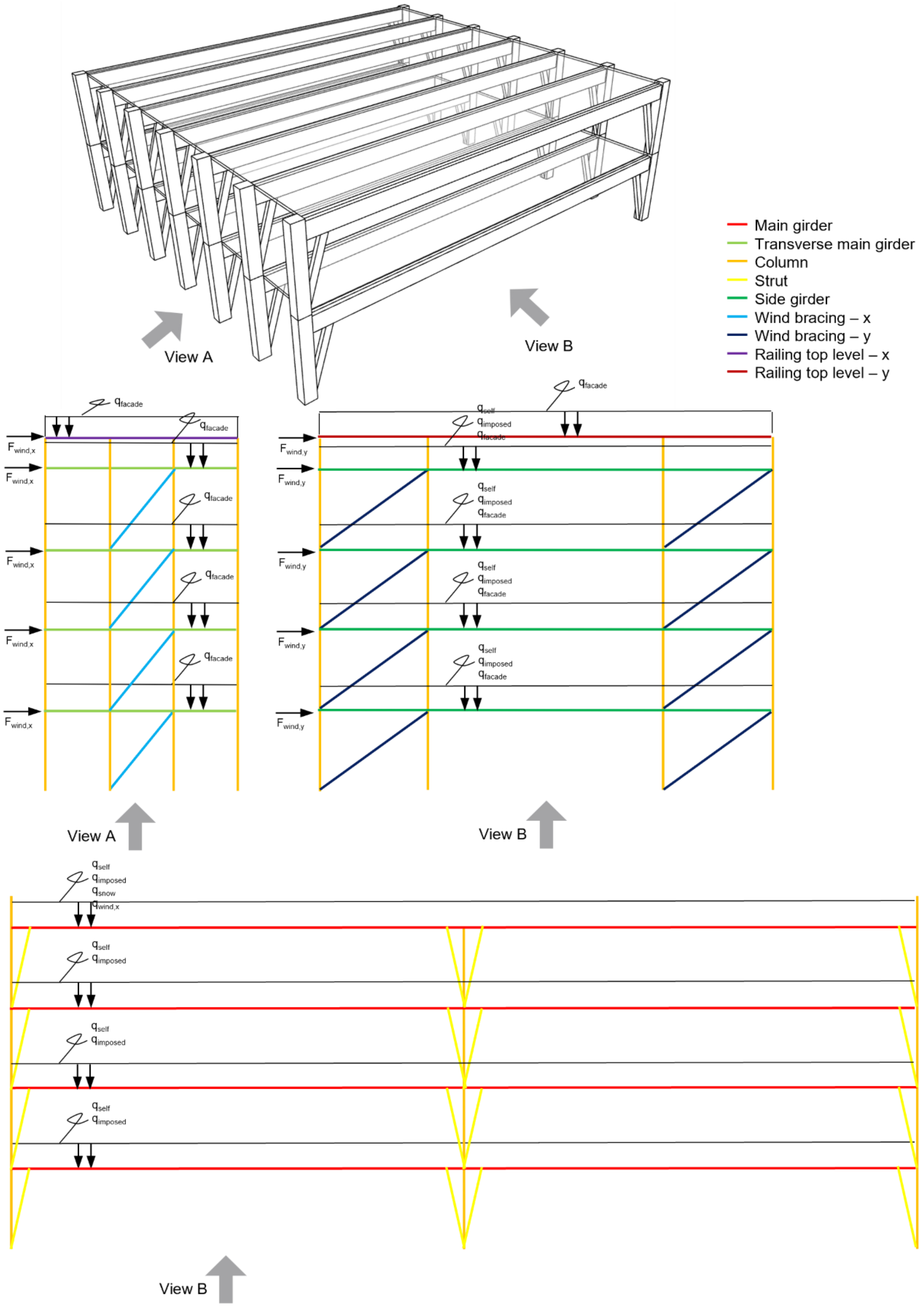


Figure 122. 2D schematisation variant 2a.

10.2.2.2 Loading

Considering the loads, the main girders are loaded by line loads for the self-weight and permanent loads of the parking deck as well as for the imposed loads. The top level main girder is also loaded by a line load for the snow loading and wind loading (pressure on the top level deck). All other self-weights of the structural elements are created by SCIA.

On the stabilizing structures, wind loading from both directions on the structure is modelled as point loads on the nodes between columns and transverse main girders/top level railings/side girders. These point loads include wind forces coming from pressure and suction on the façades as well as friction on the roof and façades. The stabilizing structures may also support the façade elements, which creates line loads on the transverse main girders, side girders and top level railings. In this case, only halve the loads coming from the parking deck are supported by these elements. Since the side girders also support the deck, here also line loads apply for self-weight, permanent loading and imposed loading of the parking deck. For the top level, also wind and snow loading applies.

10.2.3 Variant 2b

10.2.3.1 Structural elements

The schematisation for the structural system of design variant 2b is presented in Figure 123. In this variant, the short-span decks are supported by main girders, just like in variant 2a. However, not all main girders transfer their loads directly to the columns. Halve the number of girders transfer their loads to a transverse main girder, which then transfers the loads to the columns, trough which they are transferred to the foundation. To stabilize the structure, wind bracings are placed in the façade in a similar way to variant 2a, which are modelled as tension only members.

All columns are modelled as continuous elements. The main girders are simply supported in the connection to the columns. The transverse main girders and railing top level – x are modelled as continuous beams, to limit the deformations. The side girders and railing top level – y are all modelled as simply supported beams. All these elements are made of rectangular glulam timber sections. The wind bracings are made of steel L-sections, which are connected by hinges to the nodes of the structure. Finally, all columns are pinned supported to the foundations.

10.2.3.2 Loading

The loading scheme of variant 2b is somewhat different to that of variant 2a. The parking deck is supported by the main girders, which therefore are subjected to line loads for the self-weight, permanent loads, imposed loads, wind loads and snow loads. The reaction forces of the supports of the main girders are than placed as point loads at midspan of the transverse main girders and the nodes of the main girders with the columns. The point loads for the top level include wind and snow loading, while the others do not. Furthermore, the 2D frame in direction perpendicular to the parking deck span also acts as a stabilizing element with its wind bracings. Point loads on the nodes connecting the transverse main girders and top level railing to the columns include wind forces coming from pressure and suction on the façades as well as friction on the roof and façades. Finally, the transverse main girders (and top level railing) can also support the façade elements, which results in an extra line load, but in which case the girders support only halve a parking deck. For the stabilizing structure in the other direction, also point loads for the wind loads act on the nodes between side girders/top level railing and columns. The side girders support either the full load from a parking deck or halve its load and additionally the self-weight of the façade elements.

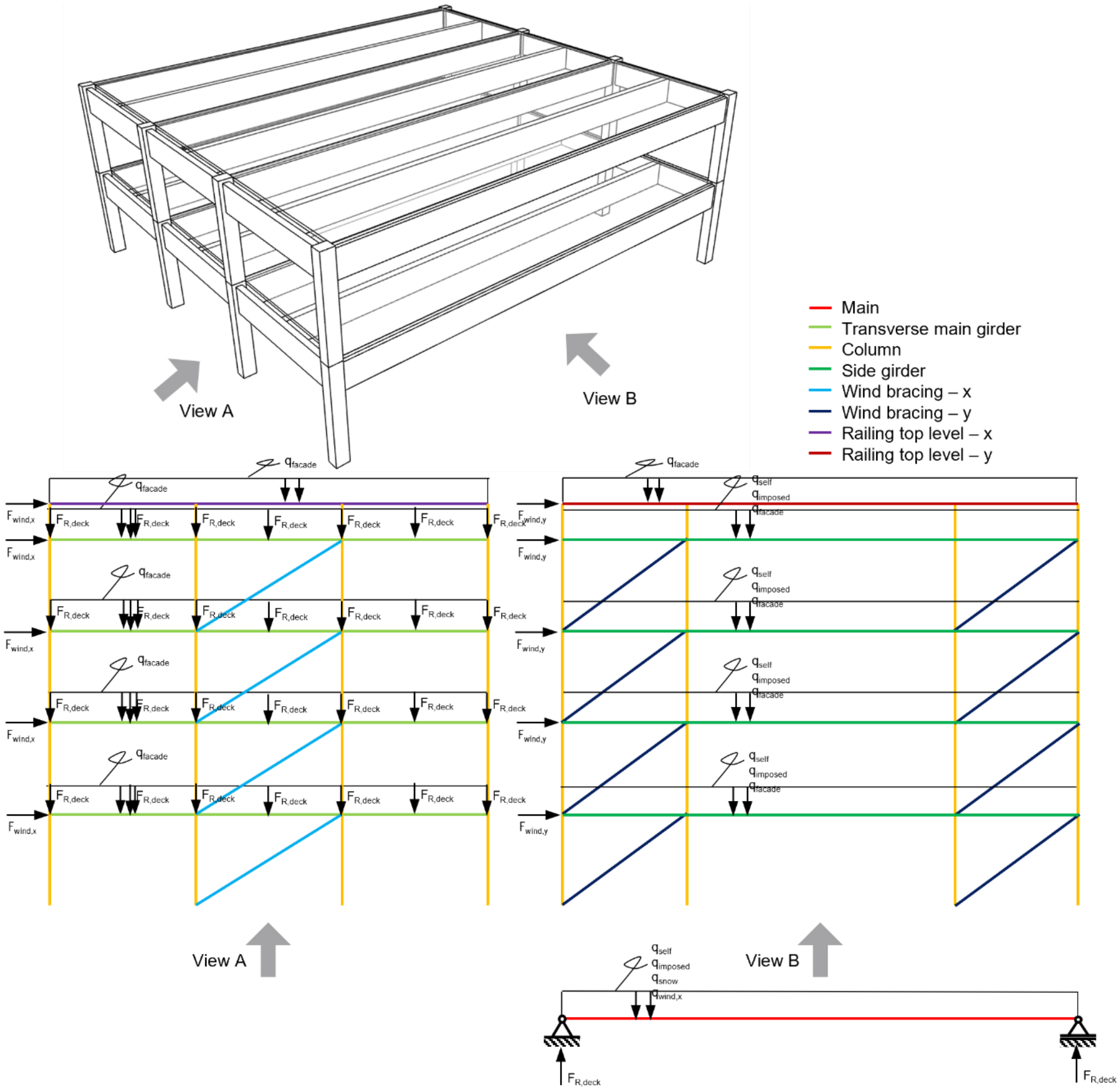


Figure 123. 2D schematisation variant 2b.

10.2.4 Variant 3

10.2.4.1 Structural elements

The schematisation for the structural system of design variant 3 looks similar to that of variant 1, but with one large difference. Variant 3 uses short-span deck panels, which are supported by joists. These joists then transfer the loads to the main girders, which are supported by the columns and optionally by struts, see Figure 124. The columns finally transfer the loads to the foundation.

To stabilize the structure, wind bracings are placed in the frame perpendicular to the parking deck span at every 40 m. Also in the other direction, wind bracings are placed in the façades and at every 40 m width of the car park for every three parking deck spans. For the wind bracings, only one element is modelled for simplifying the model, but in reality two bracings are placed in a cross. They are tension only elements.

All columns are modelled as continuous elements. The joists are simply supported beams. The main girders are simply supported in the connection to the columns and are continuous over the struts. The transverse main girders and railing top level – x are modelled as continuous elements, to limit the deformations. The side girders and railing top level – y are all modelled as simply supported beams. The struts are connected by hinges to the main girder and column. All these elements are made of rectangular glulam timber sections. The wind bracings are made of steel L-sections, which are connected by hinges to the nodes of the structure. Finally, all columns are pinned supported to the foundations.

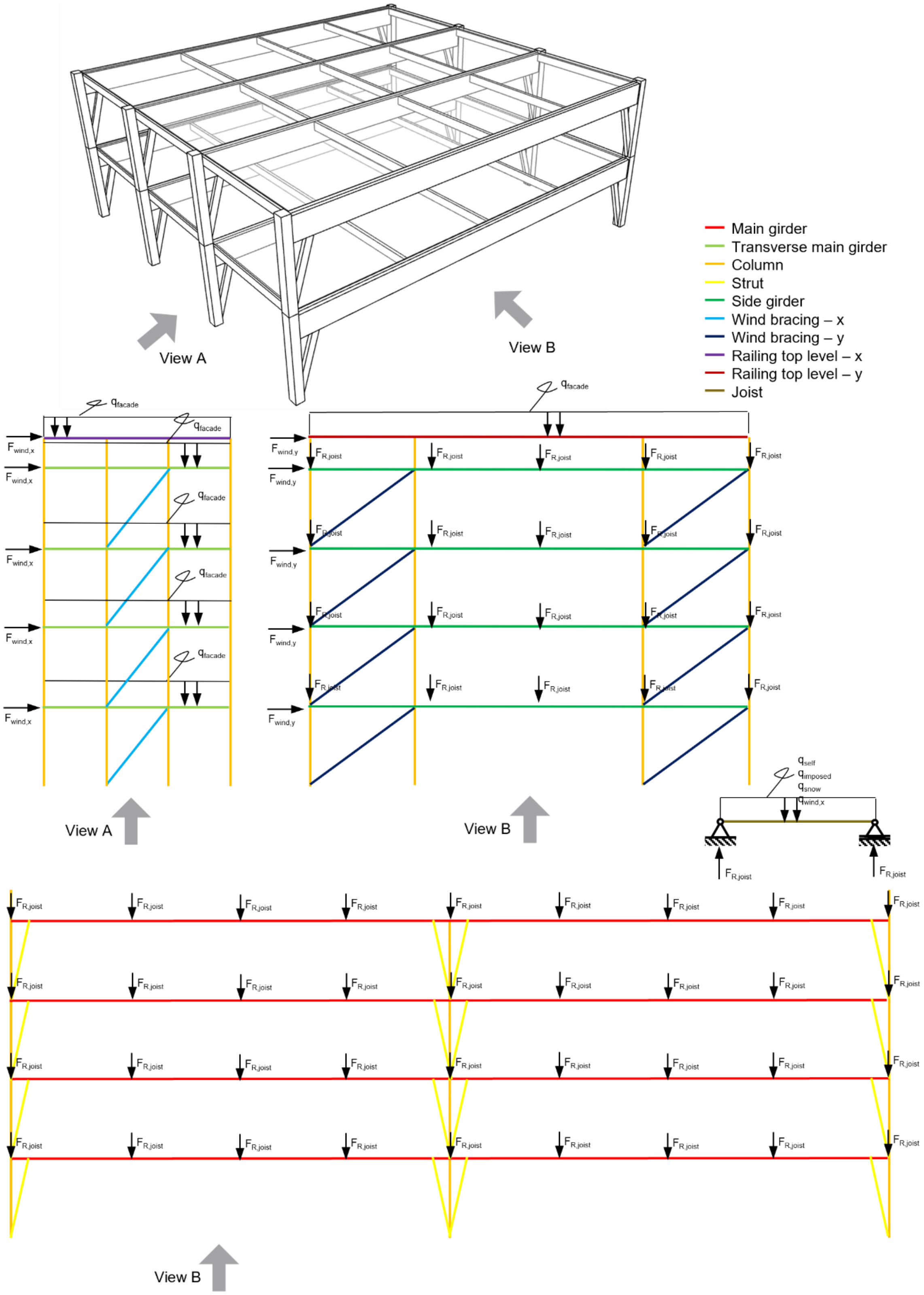


Figure 124. 2D schematisation variant 3.

10.2.4.2 Loading
The first elements of the loading scheme are the joists, which are loaded by the self-weight, permanent load, imposed load and for the top deck also wind and snow load. These are all placed on the joist as a line load. The reaction forces coming from the joists are than placed at the interval of the joists on the main girders and so include the loads coming from the parking decks. On the stabilizing frames, point loads are placed on the nodes connecting the transverse main girders, top level railings and side girders to the columns and include wind forces coming from pressure and suction on the façades as well as friction on the roof and façades. Furthermore, line loads for the self-weight of the façade are placed on the transverse main girders, side girders and top level railings. The side girders can also support the point loads of the joists, but are halve in case a façade load is applied.

10.3 Workflow
The workflow to analyse and compare the various subvariants consists out of several steps. The first step is the setup of a parametric model, based on the schematisation of the subvariants. This model has been created using Rhinoceros with Grasshopper. The use of the parametric model is twofold: it is first used to create the geometry for the subvariants and second it is used to prepare a structural model for structural analysis. The use of this parametric model speeds up the generation and analysis of the various subvariants. In the next step, a structural analysis is performed on each subvariant using SCIA. The input for this analysis is generated by the parametric model using the plugin Koala, allowing automatic generation of a structural analysis model for SCIA for each subvariant. In SCIA, the models of the subvariants are analysed and all its structural elements are optimized to fulfil the requirements for ULS and SLS as defined in chapter 6. After the structural analysis and optimization is completed, the resulting data considering the required cross-sections for all structural elements of the various subvariants is further processed using Excel. This finally results in an overview of data, concerned with total deck heights, average weight of the structure and loads on the foundation, which can be used to compare the various (sub)variant with each other and later on with alternative existing car park concepts.

10.3.1 Parametric Model
As stated before, a parametric model is created in Grasshopper. In this paragraph, the structure and general workflow of the parametric model is described. A more detailed description of the entire script can be found in appendix H.

An overview of the workflow within the script can be found in Figure 125. The script can be separated into two parts. The first part is concerned with creating the geometry of the structural elements to later be used for the second part, preparing a structural analysis model of the subvariant for SCIA, using the plugin Koala.

The script starts with the inputs. These are either parameterised inputs, like the parking deck span and column distance, but also include the specific parking deck system chosen in chapter 9. A second set of inputs are the so-called standardised inputs, which include car park characteristics like the parking bay width and clearance height, inputs for the wind load calculation and general load input values. These input values are further processed to determine often used input settings for the modelling of the geometry like the total story height.

Next, the geometry for the 2D structural model is created. These lines and nodes are later used to create the model for the structural analysis. For each main design variant, the geometry is created by a separate set of objects.

The preparation for the structural analysis model for SCIA starts with setting up the general settings of the analysis. Next, the different items required for the analysis model are created like the layers for the structural elements, load cases & groups and load combinations. For the cross-sections, self-made libraries for the standard available cross-section dimensions are inserted as well as material types and classes for later selection. The created lines and nodes from the geometry section are then used to create beam elements and support points. Cross-sections are linked to the elements and loads are applied. Finally, hinges are added where needed.

With all the inputs for the generation of the xml file by Koala ready, they are inserted into the xml generation component, which combines all elements into an xml file, which can be opened and analysed in SCIA. However, some settings required for the structural analysis of especially timber structures are not yet available in Koala components. Therefore, the created xml file is further processed by text editing in the Grasshopper model, to add certain settings to the file. This finally results in the xml file, which is used for structural analysis in SCIA.

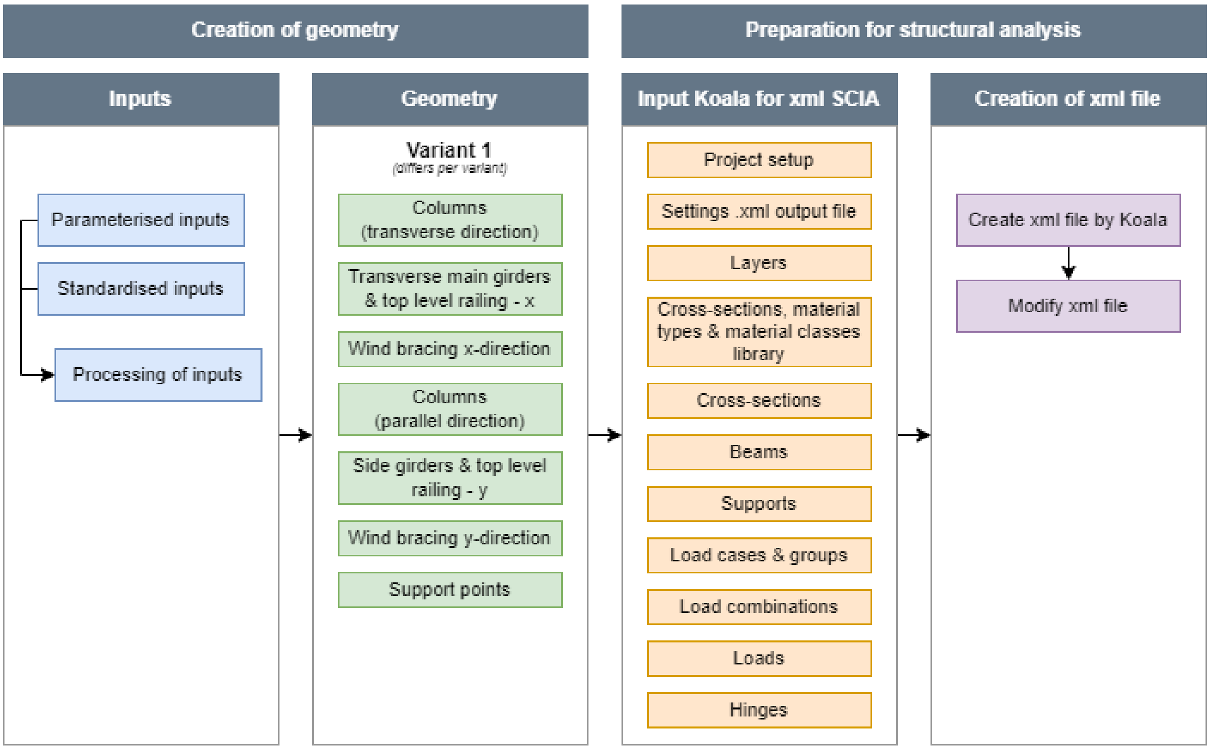


Figure 125. Workflow parametric model.

10.3.2 Structural Analysis

As stated before, the structural analysis of the structural systems of the various subvariants is conducted in SCIA Engineer. The workflow for the structural analysis can be found in Figure 126.

Before the analysis can be conducted, a template is created in SCIA. This template includes certain settings for the calculation, which could not be added to the xml file. Calculation settings for timber like the service class used and the national annex applied are included in this template.

In the next step, the xml file created by the parametric model can be loaded into the template file. SCIA analyses the structure after which its cross-sections can be optimized to fulfil the unity checks for ultimate and serviceability limit states. This optimization process can only be automized partially as these results do not fulfil all requirements. The final optimization of the cross-sections is done manually by checking the various unity checks for all structural elements.

After the structural system is analysed and checked, it results in a list of all cross-sections applied to the different elements. A list of these cross-sections is exported for further processing.

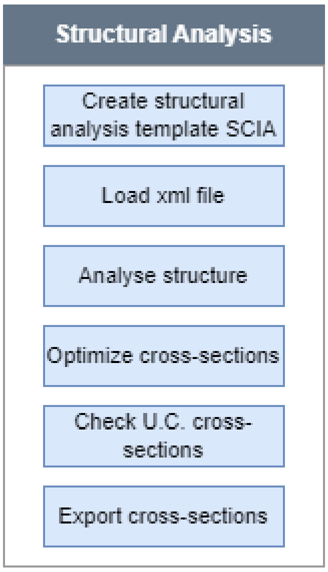


Figure 126. Workflow structural analysis SCIA.

10.3.3 Data Processing

After finishing the structural analysis, it is known what cross-sections are to be used for the different structural elements of the subvariants. However, the goal is to compare the different (sub)variants on their total deck height, weight of the structure and load on the foundation. To come up with these numbers, the characteristics of the various subvariants and their accompanying list of cross-sections is further processed using Python and Excel. Finally Excel is also used to create graphs for the comparison of the various (sub)variants.

First of all, for each subvariant, the total deck height is calculated. This can be either only the thickness of the deck element or the sum of this thickness and its supporting girders.

Second, for each subvariant, for comparison, the self-weight of the structure is calculated for a single level for a length of 4 parking bays (10 m) and a width of the parking deck span. To come up with this weight, the dimensions and weight of every structural element is determined. Next, it is determined how many items of every structural element are present in the standard length and width for the comparison. The number of elements are multiplied by their weight and summed up, resulting in the total weight of the structure.

Third, the load coming from a “standard” column onto the foundation for 1 to 5 levels is calculated. The description of a “standard” column was given in paragraph 7.2.3. This load includes the self-weight of the structural elements, transferring their loads to the column and all loads acting on the parking deck (self-weight, permanent load & imposed load). The permanent loads are multiplied by the factor 1.20 and the variable loads by 1.50.

Finally, the result is an overview of all relevant numbers required for comparison of the various (sub)variants, which is also presented in appendix G.

10.4 Results Analysis

As stated in paragraph 5.1.2, the various (sub)variants for the structural system of the modular and temporary multi-storey car park will be compared to each other considering their total deck height and weight. Therefore, in this paragraph, the three aspects of total deck height, weight per level and load on foundation are discussed for all subvariants. Apart from the results presented in this paragraph, additional tables which support these findings can be found in appendix G.

10.4.1 Total Deck Height

The total deck height for all subvariants for a span of 16.26 m varies from a minimum of 666 mm to a maximum of 1128 mm as can be seen in Figure 127. For design variant 1, both subvariants have the same deck height, since the same long-span deck panels are used with a thickness of 740 mm. This is the second smallest total deck height of all subvariants analysed.

Within design variant 2a, quite a significant difference in total deck height can be seen between the subvariants. The first conclusions that can be drawn is that the use of struts significantly reduces the total deck height as it reduces the height of the main girder, supporting the deck panels. With a column distance of 1 parking bay, a reduction of 280 mm (30%) is realised and with a column distance of 2 parking bays, it is a reduction of 260 mm (23%). The second conclusion is that also reducing the column distance in the transverse direction to the main parking deck span significantly reduces the total deck height, as the height of the main girder is reduced when it has to carry a smaller area of the deck. Without the use of struts, using a column distance of only one parking bay reduces the total deck height by 182 mm (16%) and with the use of struts, the reduction is 202 mm (23%). In total, the subvariant with a column width of a single parking bay and with the use of struts results in the smallest total deck height amongst all subvariants for a span of 16.26 m.

Design variant 2b results in a total deck height of 986 mm. It uses a column distance of two parking bay widths and has no struts. When compared to design variant 2a, it has a significant smaller height than subvariant 2a with a column distance of two parking bay widths and no struts; a reduction of 142 mm (13%). Compared to the subvariant 2a with a column distance of only a single parking bay width and no struts, its total deck height is only 40 mm (4%) larger.

For design variant 3 it can be seen that the distance between the joists does not have a very big impact on the total deck height. However, the use of struts again significantly reduces the total deck height. With a joist spacing of max 2.5 m, a reduction in total deck height of 190 mm (17%) is achieved. By using struts, the total deck height is slightly smaller than design variant 2b, which uses the same column distance. Without the use of struts, the total deck height is the same as the subvariant of variant 2a without struts and using the same column distance.

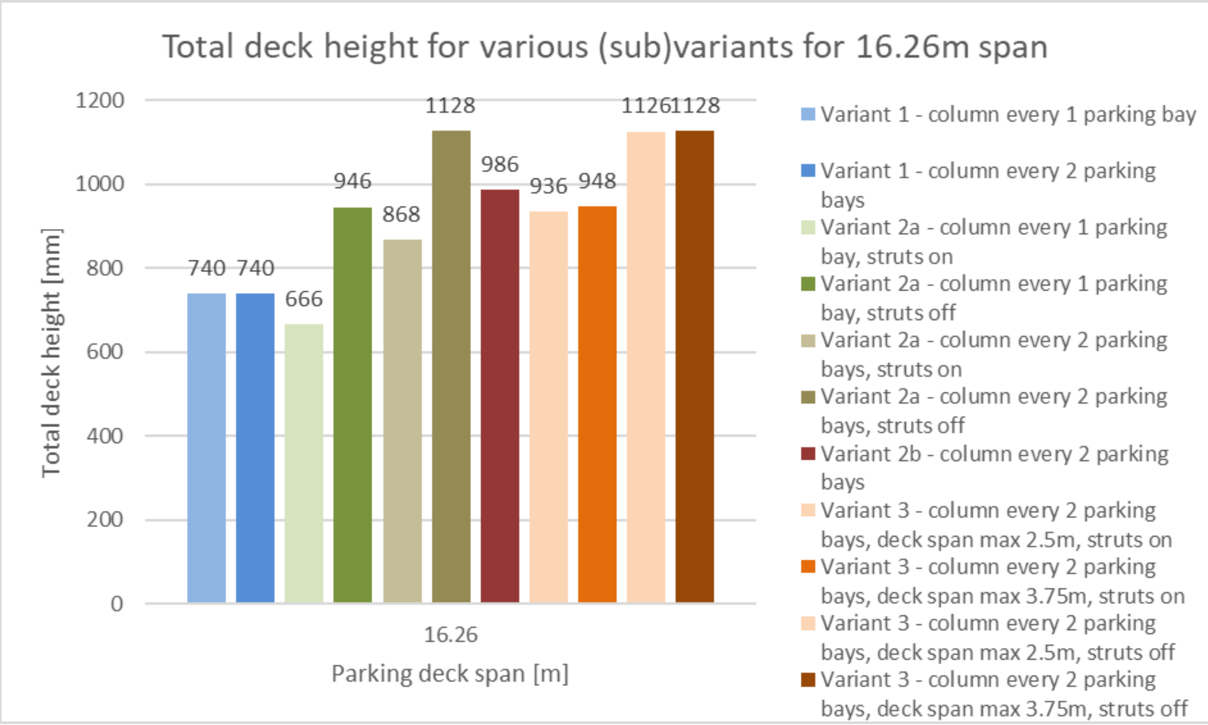


Figure 127. Total deck height for various (sub)variants for 16.26 m span.

The final parameter that is studied is the parking deck span. In Figure 128, an overview is presented of all subvariants and their total deck heights for the different parking deck spans. What can be seen is that for all subvariants, the total deck height significantly increases with the parking deck span, although it differs per variant by how much. The total deck height of design variant 1 increases the most: 75% between the smallest an largest span considered. It performs better for spans smaller than 15 m and afterwards increases even faster. The subvariants of design variant 2a perform the best around a span of 16 m. Between the smallest and biggest span, a total deck height difference between 40% and 45% can be seen. Design variant 2b displays a very linear line of the total deck height with a height difference between smallest and biggest span of 41%. The subvariants of design variant 3 also show rather straight lines with height differences around 40% with the use of struts and around 55% without the use of struts.

The different subvariants can also be compared to each other considering their performance relative to each other for different spans. An important finding is that variant 1 is the best performing (sub)variant in terms of total deck height for spans up to 15 m. From 15 m onwards, subvariant 2a with struts and a column distance of a single parking bay is the best performing. Another finding is that subvariant 2a without struts and with a columns distance of two parking bay widths starts to perform better than some subvariants of design variant 3 for longer spans. Otherwise, no real changes in the ranking of the various subvariants can be observed.

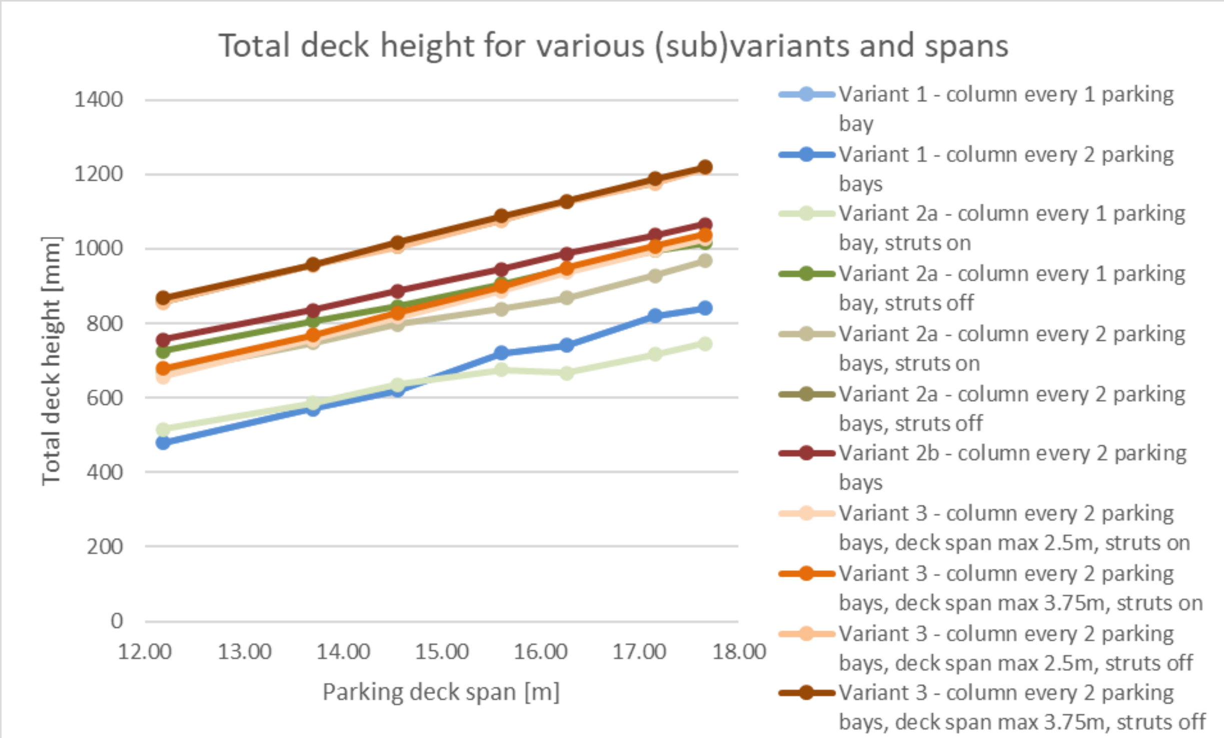


Figure 128. Total deck height for various (sub)variants and spans.

10.4.2 Weight per Level

The weight per level for a section of the structural system of the subvariants for a span of 16.26 m is presented in Figure 129. What immediately stands out is the much bigger weight of design variant 1, which is around twice as large compared to the other variants. It can be concluded that using prefab long-span deck panels is not an efficient way to span the parking decks in terms of weight compared to short-span panels in combination with girders supporting the panels. The other (sub)variants are closer to each other in terms of weight. They all weigh around 15,000 kg per level for the section of the car park considered. The difference between the lightest and heaviest of these subvariants is 20%.

Focussing on design variant 2, in terms of weight, a difference can be seen for the use of struts. The biggest reduction in weight by the use of struts is for a column distance of one parking bay: 11% (1970 kg). For a column distance of two parking bays, the reduction is smaller: 5% (824 kg). Furthermore, a smaller column distance results in a higher weight of the structure. Without the use of struts, a column distance of a single parking bay width results in an increase of the weight of 13% (1932 kg) compared to using a column distance of two parking bays. With the use of struts this is an increase of 5% (786 kg).

Design variant 2b is the worst performing variant amongst the subvariants with a column distance of two parking bays in terms of weight (without taking variant 1 into consideration). Compared to subvariant 2a without struts and a two parking bay column distance, it is 12% (1948 kg) heavier.

Amongst the subvariants of design variant 3, it can be seen that increasing the maximum distance between the joists (and therefore reducing the number of joists) results in a reduction of the weight of the structure. If struts are used, increasing the maximum distance between joists from 2.5 m to 3.75 m results in a reduction in weight of 6% (822 kg) and without the use of struts of 5% (796 kg). Furthermore, again the use of struts reduces the total weight of the structure. It reduces the weight for both joist configurations by 8% (1214 & 1188 kg).

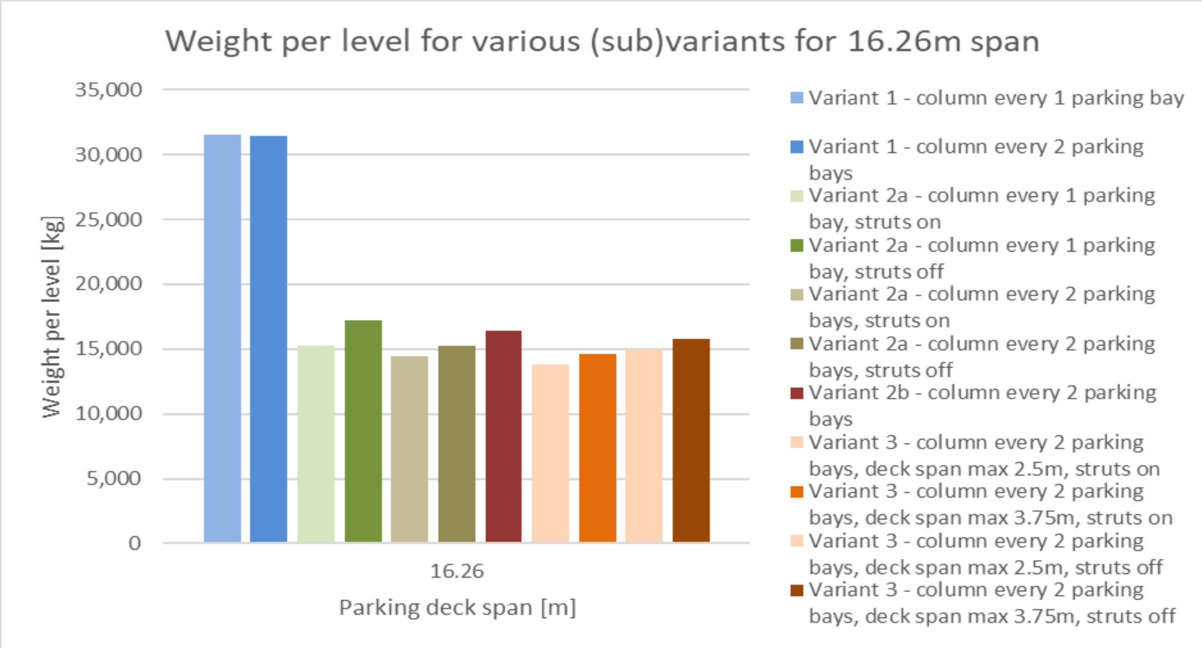


Figure 129. Weight per level for various (sub)variants for 16.26 m span.

Last of all, it is also studied what the effects are of different parking deck spans of the weight of the structure. An overview of all subvariants is presented in Figure 130. From this graph it can again be seen that variant 1 is much heavier compared to the other variants and that for longer spans it becomes even relatively more heavier in comparison to the others. The other subvariants have a more linear behaviour and are presented separately in Figure 131 to give a better overview. The exact number differs per subvariant, but all become between 1.6 and 1.8 times heavier between the smallest and largest span considered.

When comparing the different variants relatively to each other for different spans, almost no changes in the ranking of the performance on weight can be seen. The only change that stands out is the performance of subvariant 2a. For the smallest span considered, it is the second best performing subvariant, while its performance drops towards a span of 16.26 m. From this point onwards it is amongst the middle group in terms of performance.

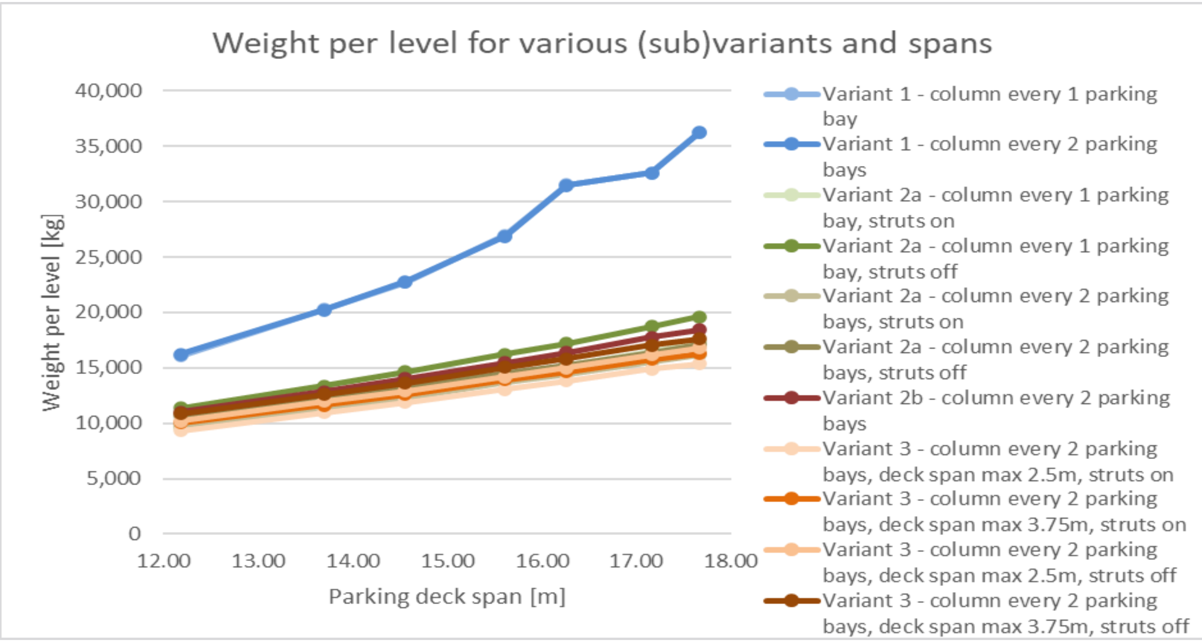


Figure 130. Weight per level for various (sub)variants and spans.

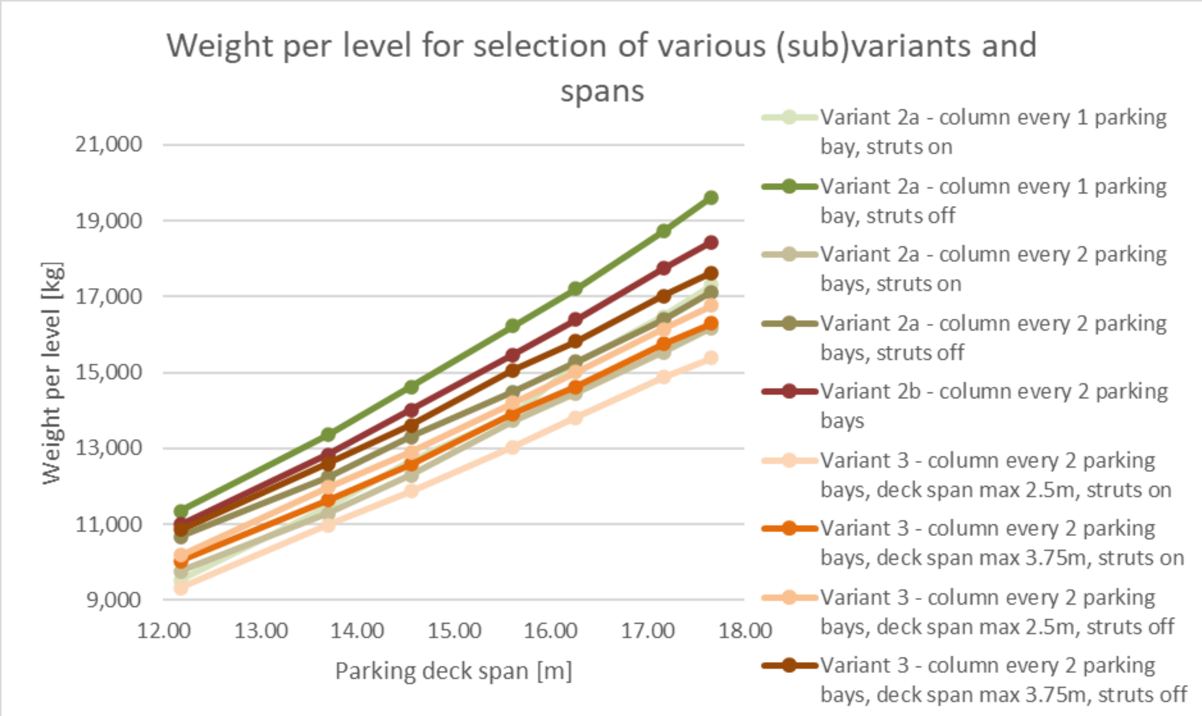


Figure 131. Weight per level for selection of various (sub)variants (not variant 1) and spans.

10.4.3 Load on Foundation

The loads on the foundation of a “standard” column for the different subvariants are compared to each other in Figure 132. The main factor influencing this load is of course the parameter of the column distance in transverse direction of the main parking deck span. The different (sub)variants differ from each other on this parameter which is either the width of a single or two parking bays. Doubling this distance results in doubling of the area that has to be supported by the column. As a result, also the total loads supported by the column for the permanent loads, imposed loads, snow loads and wind loads acting on the parking deck doubles. However, the loads as a result of the self-weight of the structural elements does not simply double, but is dependent on the configuration of the considered (sub)variant. Nevertheless, it can clearly be seen in the graph that doubling the column distance results in (roughly) a doubling of the loads on the foundation.

For variant 1, with its long-span decks, the load on the foundation simply doubles with the doubling of the column distance. For a distance of one parking bay, it is 868 kN for 4 parking levels and for a distance of two parking bays, it is 1735 kN.

Doubling the distance of the columns for variant 2a results in an increase of the load on the foundation by 98% with the use of struts and 95% without the use of struts. For 4 parking levels, the load increases from 678 kN (the smallest load of all subvariants) to 1344 kN with the use of struts and from 702 kN to 1366 kN without the use of struts. The use of struts has only very little influence on the load on the foundations when all other loads acting on the structure are also taken into account. With a column distance of a single parking bay, the use of struts lowers the load on the foundation by just 3% (24 kN for 4 parking levels) and in case of double the distance by just 2% (22 kN for 4 parking levels).

The comparison of design variant 2b for the load on the foundation to the others is similar to the weight comparison. It performs slightly worse than subvariant 2a with also two parking bays column distance and no struts, by 1%.

Similar to the weight comparison of design variant 3, also for the loads on the foundation, a slight increase is found with increasing the distance of the joists and removal of struts. An increase between each subvariant of 1% is found.

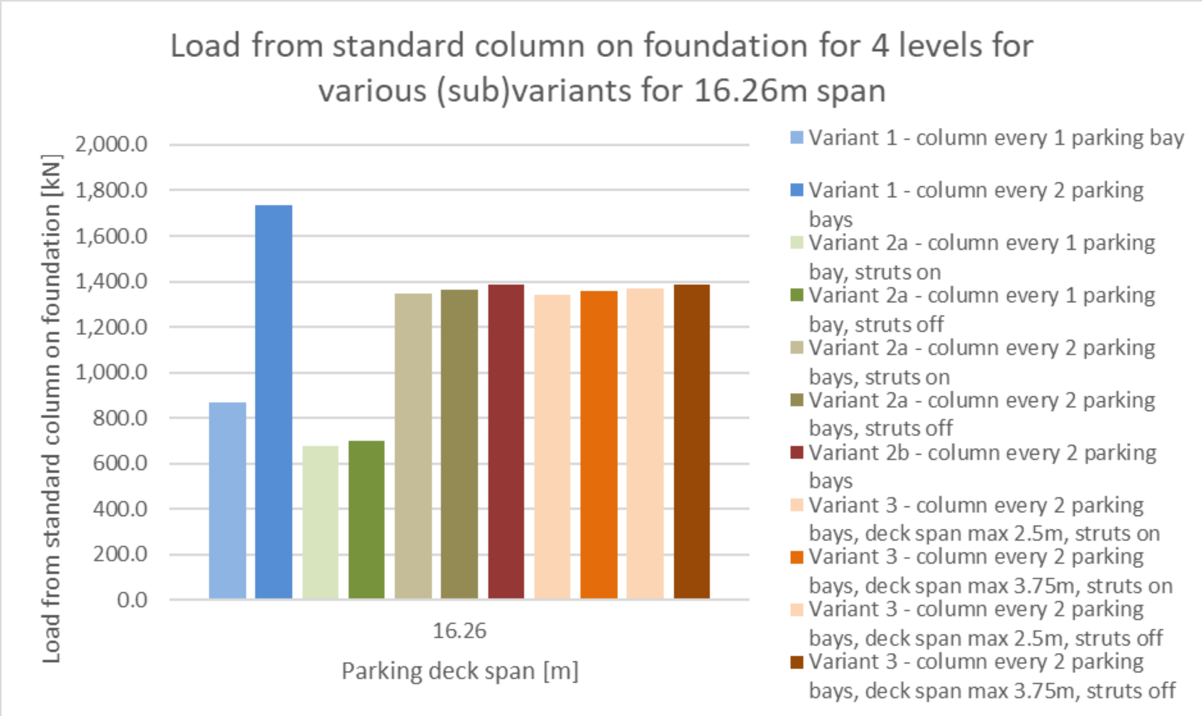


Figure 132. Load from standard column on foundation for 4 levels of various (sub)variants for 16.26 m span.

When studying the effect of the parking deck span on the load on the foundation, for each span, the same ranking of the subvariants is found, see Figure 133. Subvariants 2a with a single parking bay column distance perform best, followed by subvariant 1 with the same column distance. However, the gap between these variants increases as the span becomes bigger. At roughly double the load, the subvariants of design variant 2b and 3 are found, followed by subvariant 1 with its double parking bay width column distance. Again, its relative performance to the subvariants of design variant 2b and 3 decreases with the span.

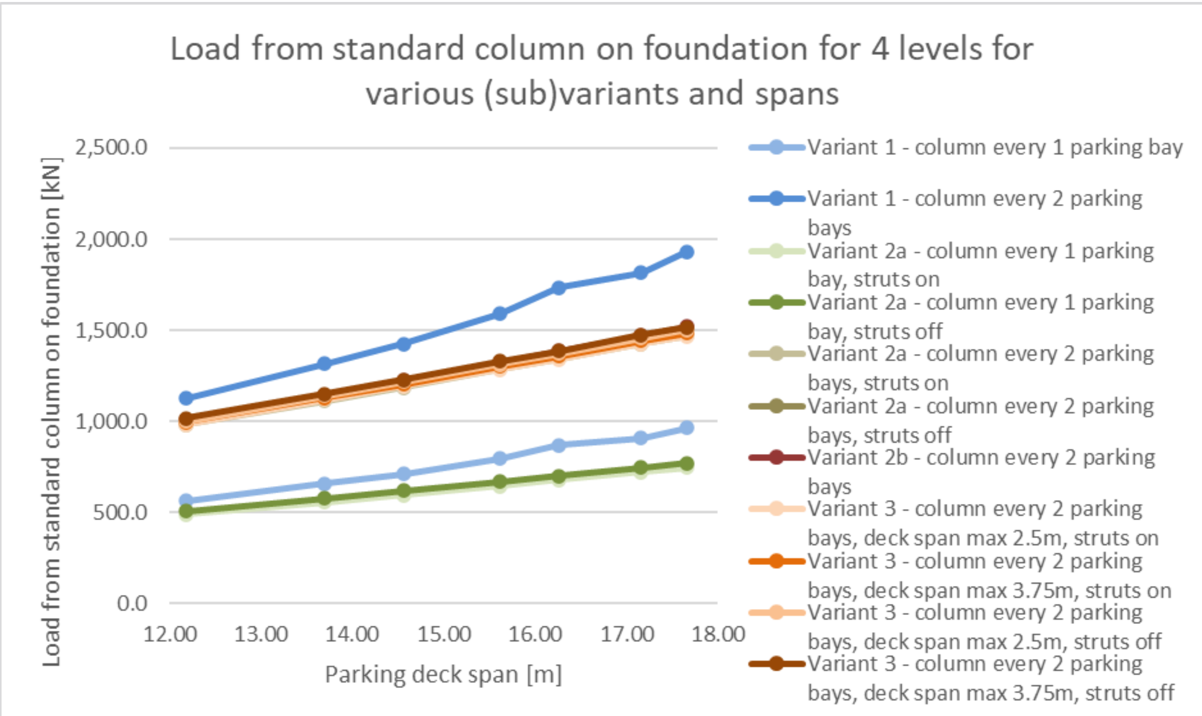


Figure 133. Load from standard column on foundation for 4 levels for various (sub)variants and spans.

10.4.4 Effect of Alternative Assessment Criteria

In the paragraphs above, the effect of changing various geometric parameters of the design variants on its performance has been studied for the specific subgoals of paragraph 5.1.2. Apart from these parameters, for the deck systems, also three parameters related to the assessment criteria for the structural systems were studied. These were the allowable deflections, applied fire-safety class and check on vibrational behaviour. Due to limitations of the software packages used (which make use of automatization), these assessment criteria parameters are not studied in detail for the various design variants including both deck and framing system. However, this does not mean one cannot say anything about what effect is expected in general on the total structural systems when these criteria are altered. Therefore, these expectations are discussed in this paragraph for each design variant.

10.4.4.1 Variant 1

Design variant 1 makes use of the long-span prefab deck system. For these panels, which span the entire parking deck, the effect of alternative assessment criteria on the total deck height as well as total deck weight has already been discussed in paragraph 9.1. Alternative assessment criteria can result in a total reduction in deck height of up to 31% (253 mm for 17.66 m span) as presented in paragraph 9.3.1. In terms of weight, a reduction of up to 49% is possible for the decks under alternative requirements. For this design variant, around 90% of the weight comes from the deck system and only 10% of the rest of the structural system. Therefore, potential reductions in weight of the supporting framing system as a result of alternative assessment criteria is not expected to result in further significant weight reduction of the total structural system of design variant 1. In conclusion, alternative assessment criteria could significantly improve the performance of design variant 1. In terms of total deck height, it would become the best performing variant (other variants under base case scenario) and in terms of weight, it would come into the same range as the other design variants.

10.4.4.2 Variant 2a

Design variant 2a makes use of a short-span prefab deck system, which is supported by a glulam main girder. Since the total deck height for this variant is the summation of the height of the main girder and the thickness of the deck, a different required height of the main girder as a result of alternative assessment criteria can result in a different total deck height. Furthermore, the total weight of the structure can change as a result of different required cross-sections for the elements of the framing system.

Considering the main girders, the ULS strength criterium is governing. Therefore, alternative requirements for deflections, fire-safety and vibrations will not result in a change in cross-section. Also for the other elements of the framing system, the ULS strength criterium is governing. In conclusion, alternative assessment criteria will not result in changing dimensions of the cross-sections of the framing system.

The effect of alternative assessment criteria on the deck system has already been studied in paragraph 9.2. It was concluded in paragraph 9.3.2, that in terms of height and weight, a reduction of up to 39% is possible for a column distance of one parking bay and 16% for a column distance of two parking bays. However, since the total deck is build-up of the short-span deck system in combination with a main girder, in absolute terms, these reductions are less significant. For the height, a maximum reduction of only 36 and 18 mm is achieved for each column distance respectively under alternative assessment criteria. Since no reduction in height of the main girder is possible under alternative requirements, the total height reduction of the deck is rather limited compared to design variant 1 under alternative requirements. Considering the total weight of the structural system, roughly halve of the weight comes from the deck system, the main girders are around 30% and the columns and struts 20% (depending on the specific subvariant). The reduction in weight for the deck under alternative requirements is more significant. In the total structural system, it contributes to a maximum reduction of around 20% in self-weight.

10.4.4.3 Variant 2b

Similar to design variant 2a, its conclusions also stand for design variant 2b. It uses the same short-span deck system, for which a reduction in total deck height under alternative requirements is rather limited. Also for this variant, the ULS strength criteria are governing for the other structural elements and therefore reductions of cross-sections dimensions under alternative assessment criteria are not possible. Bigger savings can be realised in terms of weight. Again, the deck system is around halve the weight of the total structural system, the main girders are around 40% and the transverse girders and columns the remaining 10%. This again results in a potential reduction of the self-weight of the structural system of 20% under alternative assessment criteria.

10.4.4.4 Variant 3

Design variant 3 again makes use of the short-span deck system, but only has a span of 5 m. Again, the ULS strength criteria are governing for the elements of the framing system and therefore, reduction in total deck height and weight under alternative requirements has to come from the deck system. For a span of 5 m, the possible reduction in height and weight are only 16% for the decks. This is only 18 mm reduction in height, which is not significant. In terms of weight, the deck makes up roughly 60% of the weight of the total structural system, the main girders and joists are around 30% and the columns and struts 10%. As a result, a total reduction in self-weight of the structural system under alternative assessment criteria of up to 10% is possible.

10.4.4.5 Conclusion

In conclusion, it can be stated that alternative assessment criteria have a different effect on the various design variants. For variant 1, it can result in a significant reduction in both height and weight, making it able the best performing variant in terms of height and closer to the other variants in terms of weight compared to the base case scenario. For the other design variants, the reduction in total deck height is not significant, however in terms of weight, a reduction in self-weight of around 20% can be achieved for variants 2a and 2b and 10% for variant 3.

10.5 Concluding Remarks

Based on the results of the structural analysis of the various designs for the frame system and its comparison, several conclusions can be drawn. Hypotheses H1 till H4 a & b can be tested and the best performing design subvariant can be chosen. In conclusion of this chapter, sub-question 3c can further be answered: “What is the impact of altering geometric parameters and limit state requirements on the performance of the different designs for a temporary multi-storey car park using timber as primary structural material?”.

10.5.1 Total Deck Height

Considering the total deck height of the different design variants, hypotheses H1a, H2a, H3a and H4a can be tested.

H1a: Increasing the parking deck span will result in an increase of the total deck height.

Focussing on the parking deck span versus the total deck height, as expected a bigger span results in an increase in total deck height, confirming hypothesis H1a.

H2a: The use of struts in the main span of the structural system of the car park will result in a decrease of the total deck height.

The comparison of the deck heights of the various subvariants has clearly shown that the use of struts significantly reduces the total deck height (between 17 and 30% depending on the variant). Already with a small inward distance of the struts of 0.67 m, this reduction is achieved. Using a subvariant with stuts if possible is therefore preferred. This confirms hypothesis H2a.

H3a: Increasing the distance between columns in transverse direction will result in an increase of the total deck height.

The comparison has also proved that using a column distance in the transverse direction of the main parking deck span of one parking bay width results in a lower total deck height compared to a distance of two parking bay widths (reduction of 16 to 23% in total deck height). To minimize the total deck height, a smaller column distance is therefore preferred. This confirms hypothesis H3a.

H4a: Increasing the distance between joists will result in an increase of the total deck height.

For design variant 3, the results have shown that for a larger distance between the joists, the total deck height increases. However, this increase in height is very small. Nevertheless, this confirms hypothesis H4a.

In terms of performance, design variant 1 is the best performing variant for spans up to 15 m, closely followed by subvariant 2a – column every 1 parking bay, struts on. This subvariant performs best for spans larger than 15 m. The other subvariants perform significantly worse in terms of total deck height (>> 100 mm increase in height). Subvariant 2a – column every 2 parking bays, struts off and the subvariants of variant 3 without struts are the worst performers and can be ruled out as potential framing systems.

10.5.2 Weight per Level

With the focus on the weight per level, hypothesis H1b, H2b, H3b and H4b can be tested.

H1b: Increasing the parking deck span will result in an increase of the total construction weight.

Hypothesis H1b can be confirmed as the results have shown that an increase in parking deck span results in an increase of the structures weight.

H2b: The use of struts in the main span of the structural system of the car park will result in a decrease of the total construction weight.

What can be concluded for the comparison of the weight per level of the structural systems is that again, the use of struts is preferred to reduce the structures weight (5-11% reduction). This confirms hypothesis H2b.

H3b: Increasing the distance between columns in transverse direction will result in an increase of the total construction weight.

It was expected that a bigger column distance in the transverse direction to the main parking deck span would result in an increase of the weight, as the larger loads on the elements would increase its size and weight. However, based on the results, hypothesis H3b cannot be confirmed. A column distance of two parking bay widths reduces the weight of the structure between 5 and 13% compared to a distance of a single parking bay width. The reduction in weight however is smaller compared to the increase in total deck height in the case of this parameter.

H4b: Increasing the distance between joists will result in an increase of the total construction weight.

Focussing on design variant 3, the use of a small maximum joist distance is preferred as this reduces the weight of the structure by around 5%. This confirms hypothesis H4b.

For the potential remaining framing systems for the car park design, variant 1 can be ruled out as an option, since its weight is around twice as big as the weight of the other variants.

10.5.3 Load on Foundation

In chapter 8, possible foundation types were studied for the structural foundation of the car park. It was concluded that by limiting the load on the foundation to 1,000 kN, it is possible to use a shallow foundation in most places in The Netherlands, where sand layers are found near the surface. For such a load, it is also often possible to create a pile foundation using multiple timber piles. Therefore, the loads on the foundation of the various subvariants are compared to this number.

For all spans and number of levels, doubling the column distance in the transverse direction of the main parking deck span results in roughly a doubling of the load on the foundation. Therefore, reducing this distance to a single parking bay width is preferred when the loads on the foundation have to be reduced. The subvariants using such a distance are the best performing and within these, subvariant 2a outperforms subvariant 1 by 22% (190 kN for 4 levels). The variation between the subvariants of variant 2b and 3 is not significant and lies around 1%. With subvariant 2a – column every 1 parking bay, struts on, 5 levels can be constructed with a load on the foundation lying still far below the 1,000 kN at 848 kN.

10.5.4 Choice of Framing System

The final choice for the best performing framing system is subvariant 2a – column every 1 parking bay, struts on. It is the best performing subvariant in terms of both total deck height and column load on the foundation. Amongst the weight of the structure, it performs in the same range as most other good performing subvariants.

Under alternative requirements, the biggest shift in performance is expected from design variant 1. It significantly improves in terms of total deck height and furthermore its self-weight is expected to be almost halved. Further study should prove if it is possible to use alternative assessment criteria and if indeed variant 1 in this case is the best performing design alternative.

PART V: Evaluation and Conclusions

11
Case Study

The previous chapters have proven it is possible to design a structural system for a temporary and multi-storey car park primarily in timber. However, it is also very relevant to compare such a design against alternative (existing) concepts and compare how it performs relative to these alternatives. This chapter therefore makes this comparison and answers the fourth sub- research question: "How does the proof of concept for a temporary multi-storey car park, using timber as primary structural material compare to alternative modular and/or timber multi-storey car park concepts?".

11.1 Reference Projects

In chapter 7, a total of five different car park structures have already been compared to each other on the same aspects as the different timber design variants in previous chapter. These car parks have either modular structural systems, are designed for disassembly and reuse or are designed using timber as primary material. These five car park designs are used as reference projects for comparison with the chosen design in timber, developed in this thesis. The five reference car park projects are:

- ModuPark (steel frame with concrete TT-panels as deck);
- Car park Koopman int. (steel frame with fibreglass deck);
- Car park Morspoort (steel frame with steel/concrete composite deck);
- Concept in BauBuche (timber frame with prefab concrete deck panels);
- B&O-Holzparkhaus (timber frame with timber CLT deck).

11.2 Comparison

The comparison between the reference projects and the developed timber concept are made on total deck height and weight, see Table 48. For the weight, the weight for a standard area of the car park for one level is compared as well as the load on the foundation coming from a "standard" column for four levels (see paragraph 7.2.3 for explanation of a "standard" column).

Table 48. Comparison between developed concept in timber vs reference projects including relative performance.

Car park	**Floor depth**		**Weight /level**		**Load standard column on foundation 4 levels**		
	[mm]	**[%]**	**[kg]**	**[%]**	**[kN]**	**[%]**	
ModuPark	550	83%	61,261	402%	2,472	365%	*(5.0 m column distance)*
Koopman int.	640	96%	10,477	69%	1,258	186%	*(5.0 m column distance)*
Morspoort	651	98%	35,110	230%	1,845	272%	*(5.0 m column distance)*
BauBuche concept	730	110%	59,341	389%	1,193	176%	*(2.5 m column distance)*
B&O Holzparkhaus	935	140%	17,547	115%	717	106%	*(2.5 m column distance)*
Variant 2a - column every 1 parking bay, struts on	666	100%	15,242	100%	678	100%	

11.2.1 Total Deck Height

The total deck height of the chosen design variant for the timber car park is 666 mm and falls in the middle of the reference projects as can be seen in Figure 134. All values used in the comparison are also presented in Table 48, which also gives the performance of the developed concept relatively to the reference projects. The developed concept performs similar to the concepts with lightweight fibreglass and steel/concrete composite decks of the Koopman int. and Morspoort car parks. Its total deck height is only 4% and 2% higher. Relatively to the ModuPark concept, it is 17% thicker, but relatively to the BauBuche concept it is 10% thinner. In comparison with the realised car park in timber (B&O-Holzparkhaus), it outperforms it in terms of total deck height by 40%. In conclusion, it can be stated that the total deck height of the developed concept is an improvement compared to the existing car park in timber (B&O-Holzparkhaus) and it falls into the same range as the alternative concepts in the comparison.

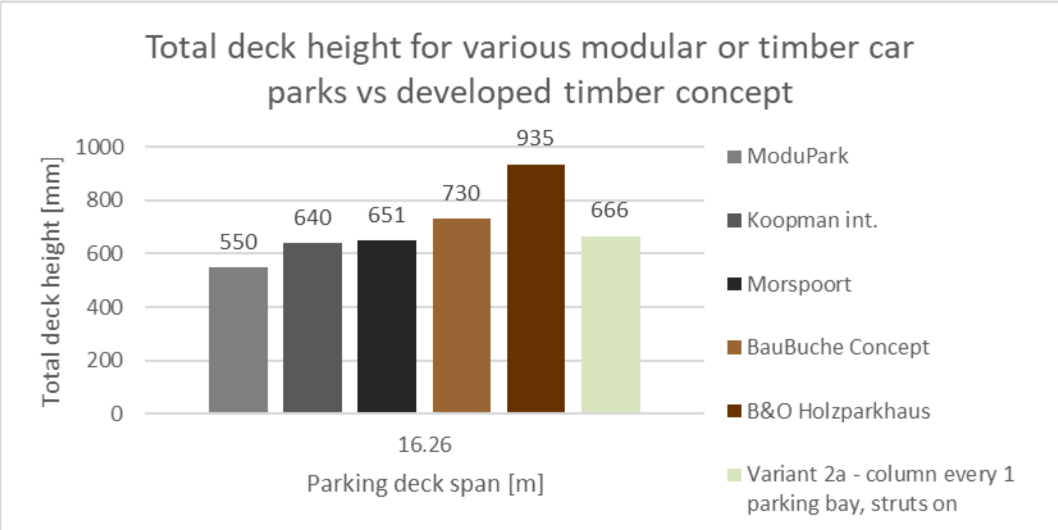


Figure 134. Comparison of total deck height between developed modular timber concept vs concepts from case study.

11.2.2 Weight per Level

The weight for the given area of a single level of the chosen variant for the timber car park is 15,242 kg. Timber again proves itself to be a very lightweight material. The only structural system of a reference project that results in a lower weight is that of the Koopman int. car park, which uses extremely lightweight fibreglass deck panels, see Figure 135. The developed timber concept is 31% heavier compared to this concept, but is much lighter than the structures of most other reference projects. In comparison with the existing timber car park, it is 15% lighter. In comparison, the Morspoort car park with its steel/concrete composite deck is already more than twice as heavy. The ModuPark concept and BauBuche concept are even around 4 times as heavy. It can therefore only be concluded that the developed timber concept is very lightweight and has a self-weight much lower than most reference projects.

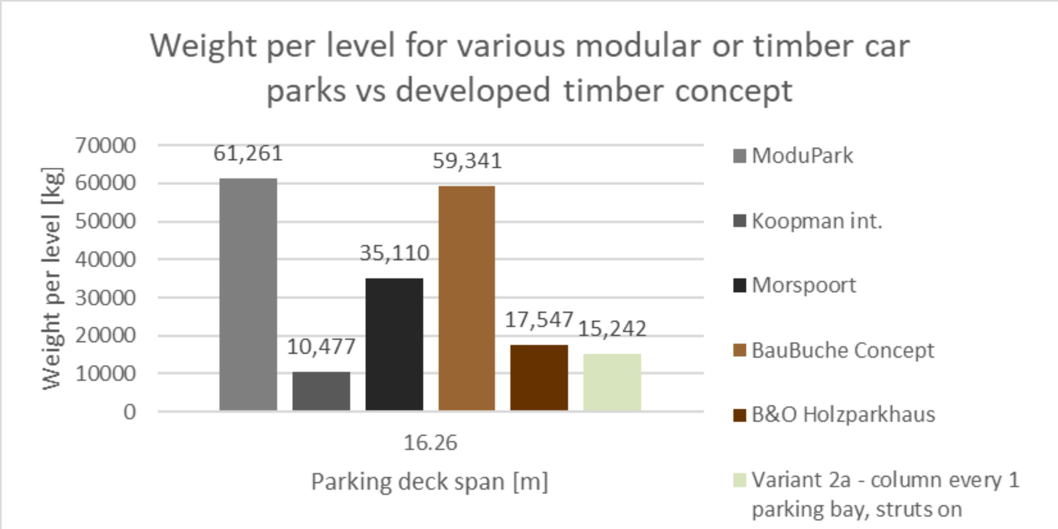


Figure 135. Comparison of weight per level between developed modular timber concept vs concepts from case study.

11.2.3 Load on Foundation

When studying the weight of the structure, also the load of a column of the structure on the foundation is studied. As stated before in the previous chapter, a structural system with a column distance in transverse direction to the main parking deck span of a single parking bay width performs much better than when double this distance is used. Of the reference projects, the ModuPark, Koopman int. car park and Morspoort car park all have a column distance of two parking bays. The BauBuche concept has a distance of a single parking bay. In the overall comparison, the developed timber car park design performs best at 678 kN. The B&O-Holzparkhaus results in a 6% higher load in the foundation. The other projects give much higher loads, see Figure 136. For the BauBuche concept and Kooman int. car park, they result in a load close to two times as large. For the Morspoort car park it is 2.7 times as high and for the ModuPark concept even 3.7 times higher. In conclusion, the developed proof of concept results in the lowest column loads on the foundation in comparison with the reference projects.

In chapter 8, it was concluded that for soils with large sand layers near the surface, a column load of 1,000 kN can be supported by a shallow foundation. The proof of concept with a load of 678 kN for 4 levels can therefore easily be supported by such a soil. For soils with a large sand layer in the first 20 m, a pile load of 678 kN would require between 2 and 5 timber foundation piles. In other cases, 1 up to 3 concrete foundation piles are required.

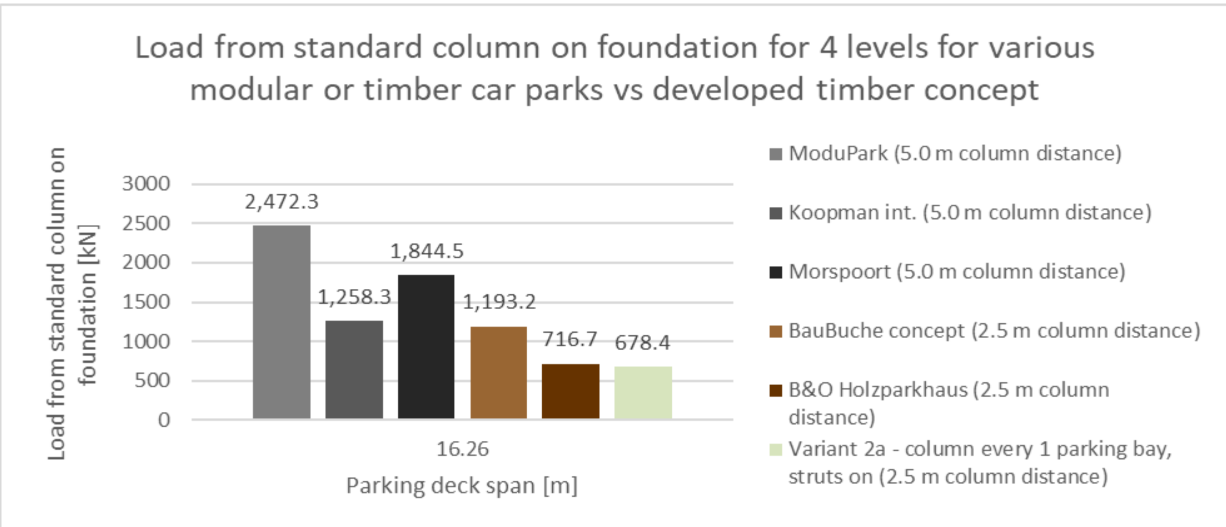


Figure 136. Comparison of load from standard column between developed modular timber concept vs concepts from case study.

11.2.4 Sustainability

In chapter 3, it was already discussed that the building industry has to make a transition towards a more sustainable industry. Circular construction was discussed as a method to come to a more sustainable industry. Furthermore, three themes were presented, focussing on incorporating circular design strategies into the design and construction of new buildings:

- Modularization, prefabrication and standardization;
- Reuse, refurbish and recycle;
- Building materials.

In the developed proof of concept, all of these themes can be found to some extend. Its design is made modular, with standardized elements that are prefabricated. The entire goal of the design is that it can be dismantled and reused and last of all, it is made of primarily timber instead of steel and concrete.

It is not the goal of this thesis to make a detailed sustainability analysis of the developed proof of concept and alternative car park concepts. However, aim is to be able to say something concerning how the proof of concept performs relative to the alternative concepts, which were discussed in chapter 7. Therefore, in this paragraph a simplified analysis is conducted on the embodied carbon in the various studied car park concepts. For this analysis, only the repetitive main structural elements of the deck and framing system of the car parks are considered. This excludes for example wind bracings, façades, connections, foundations and finishing layers.

The results of the embodied carbon analysis are presented in Table 49. It includes the total amount of embodied carbon for the considered area of the car park concepts (1 level, 10.0 m wide, 16.26 m span) for the main structural elements as discussed above. Furthermore, the embodied carbon intensity is presented, which gives the embodied carbon per unit area. The key figures used for the various materials are presented in Table 50. These key figures consider cradle to gate (A1-A3). The use and end-of-life phase are therefore not considered. For the various timber materials, multiple numbers are presented. Since during the growth of a tree, carbon is stored in the timber, it can have a negative embodied carbon number. In such a case, the end-of-life phase becomes important, as at this point, the stored carbon is again released to the atmosphere. For the comparison of the car park concepts, this negative number is not taken into account, however it is good to keep in mind the further potential of timber as a material that can store carbon. For concrete, different numbers apply considering if the steel used for rebar is recycled or not. For the comparison, it is assumed recycled steel is used. Finally, the amount of materials used in the different car park concepts can be found in appendix E.

In Figure 137 a comparison is made between the various car park concepts. From this graph, it becomes clear that the fully timber car park concepts (B&O and proof of concept) have by far the lowest amount of embodied carbon. The combined timber (BauBuche) and concrete concept already is at double the number of the fully timber concepts. Also the car park concepts, which use mainly steel and concrete have an amount of embodied carbon more than twice as high. Finally, the Koopman int. car park has the highest amount of embodied carbon, almost four times higher than the fully timber concepts. This is mainly due to the used GFRP decks, which have a very high embodied carbon rating.

In conclusion it can be stated that the performance in terms of sustainability, based on the simplified embodied carbon analysis of the proof of concept is far better compared to the other non-fully timber car park concepts. Since this analysis is not very detailed, a lot of assumptions are made, and a lot of structural elements of the car park for example are missing in the analysis, the final results can of course change in a more thorough analysis. However, it can also be expected that the performance of the proof of concept in a detailed analysis further improves. For example the possible smaller foundation works can further increase its performance. Also the potential carbon storage in timber was not considered in this brief analysis and brings a bigger potential. In conclusion, this brief analysis shows the potential of the proof of concept in terms of sustainability performance.

Table 49. Embodied carbon from main structural elements in reference area for various car park concepts.

Car park	**Embodied carbon (no carbon storage) [kgCO$_2$e]**	**Embodied carbon intensity (no carbon storage) [kgCO$_2$e/m^2]**
ModuPark	15,643	96.2
Koopman int.	25,697	158.0
Morspoort	17,386	106.9
BauBuche concept	14,250	87.6
B&O Holzparkhaus	6,900	42.4
Variant 2a - column every 1 parking bay, struts on	6,852	42.1

*Table 50. Key figures embodied carbon for various materials (Jones, 2019). *(Crawford, Stephan, & Prideaux, 2019)*

	No carbon storage	**With carbon storage**	
Material	**Embodied Carbon [kgCO$_2$e/kg]**	**Embodied Carbon [kgCO$_2$e/kg]**	**Of which carbon storage [kgCO$_2$e/kg]**
Timber - Softwood	0.263	-1.29	-1.55
Timber - CLT	0.437	-1.20	-1.64
Timber - Glulam	0.512	-0.90	-1.41
Timber - LVL	0.390	-1.25	-1.64
Steel - Rebar	1.990	n.a.	n.a.
Steel - Section	1.550	n.a.	n.a.
Concrete - Precast (non-recycled steel)	0.249	n.a.	n.a.
Concrete - Precast (recycled steel)	0.194	n.a.	n.a.
GFRP	5.0*	n.a.	n.a.

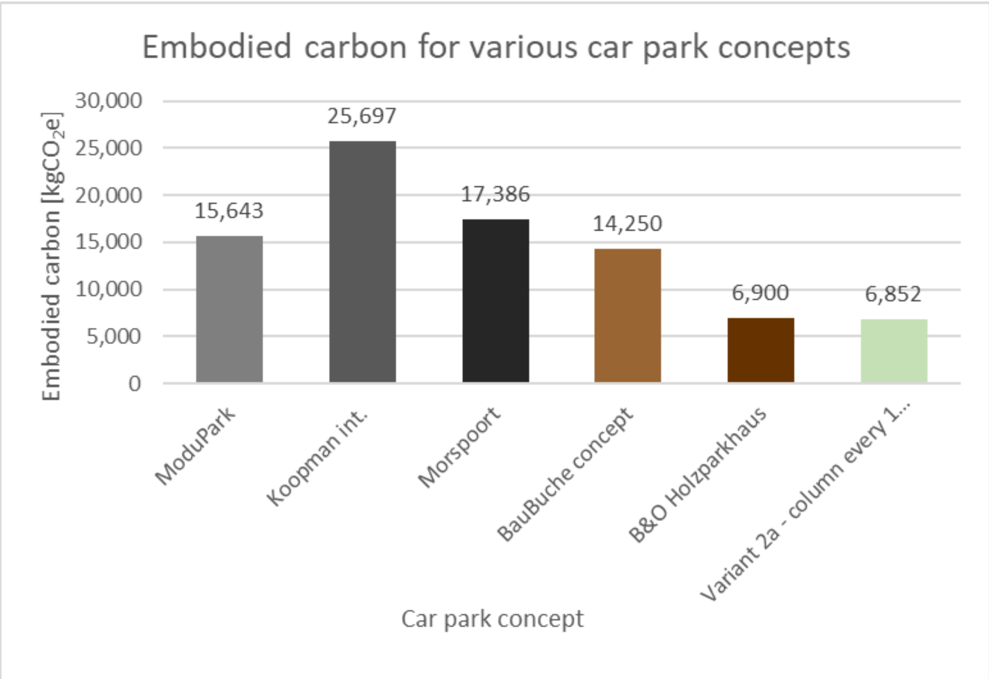


Figure 137. Comparison of embodied carbon from main structural elements in reference area for various car park concepts.

11.3 Concluding Remarks

Concluding the comparison of the case study, the fourth and final sub-question can be answered: “How does the proof of concept for a temporary multi-storey car park, using timber as primary structural material compare to alternative modular and/or timber multi-storey car park concepts?”.

It can be stated that the developed structural system for the modular and temporary car park in timber performs well on the criteria studied in this thesis. In terms of total deck height, it can compete with the various other existing modular and dismantlable concepts. Furthermore, its use of LVL decks results in a smaller deck height compared to other variants in timber. In terms of weight, using timber for almost all structural elements proves to result in a very lightweight car park. It is much lighter compared to other concepts which use concrete. Only the extremely lightweight fibreglass deck can result in a lower self-weight. Furthermore, the reduction in weight creates the potential to use smaller foundation structures. Finally, considering sustainability, it can be stated that a first simplified analysis into the amount of embodied carbon in the structural system of the proof of concept shows that the timber proof of concept is the best performing concept in comparison to the other modular car park concepts. Its amount of embodied carbon is at least halve that of the other non-fully timber build car park concepts, highlighting the potential of timber to bring the building industry further in its transition towards a more sustainable industry.

12 Conclusion

This chapter forms the conclusion of this master thesis. In chapter 1, the objective and research questions were presented. In previous chapters, the various sub-questions have been answered for which the conclusions can be found in the concluding paragraphs of these chapters. Finally, in this chapter, the answer to the main research question is presented.

12.1 Objective & Main Research Question

The objective of this study was to provide a structural system proof of concept for a temporary multi-storey car park, using timber as the primary structural material. The proof of concept presented in this thesis results from a literature study into the construction of a multi-storey car park in timber, followed by a parametric study with structural validation on multiple design variants. Finally, it is compared in a case-study against five alternative multi-storey car park concepts. This has resulted in an answer to the following question:

"To what extent can timber be applied as a structural material in the structural system of a temporary multi-storey car park and how does it compare to alternative concepts?"

12.2 Proof of Concept

The developed proof of concept is the result of a parametric study and consists out of a short-span prefab timber deck system, supported by a timber framing system and is stabilized by steel bracing elements. Its design is presented in Figure 138. The deck is made from solid LVL panels with a thickness of 96 mm, coupled together to allow horizontal distribution of wind loads. These panels are 5 m long and are supported every 2.5 m by glulam main girders (570x280 mm for 16.26 m span). The main girders span the entire parking deck span, creating a column-free parking deck. They are simply supported at the edges by glulam columns (440x360 mm for 16.26 m span), transferring the loads to the foundation. Slanted struts (240x140 mm for 16.26 m span) between the columns and main girders reduce the unsupported length of these girders, resulting in a reduction of their cross-section. Steel L-sections are applied in X-bracings in the façades to vertically stabilize the structure. For the foundation, prefab concrete footings are required, since timber is prone to rot, which may or may not require support by foundation piles. In many locations in The Netherlands, timber foundation piles can be used and only in case no sufficient sand layers are found within the first 20 m of soil, concrete piles are to be used.

In the literature study, it was found that building in timber comes with many challenges, which can limit the applicability of timber in a multi-storey car park. These challenges are related to durability risks and proper detailing of the structure. Based on the literature study, it was concluded that for a temporary and demountable car park, carpentry joints are a potential option for joints with limited loads and which require only a low stiffness. For joints with higher loads or which require higher stiffness, dowel-type fasteners like bolts and slotted in steel plates or bolts in combination with connectors are potential connection types. To protect the timber structure against biological and environmental degradation, it was concluded that natural durability and constructional measures are always preferred over wood preservatives or modification. To protect the timber deck against degradation, the literature study concluded that a coating with aggregate is best applicable considering the dismantlability of the car park.

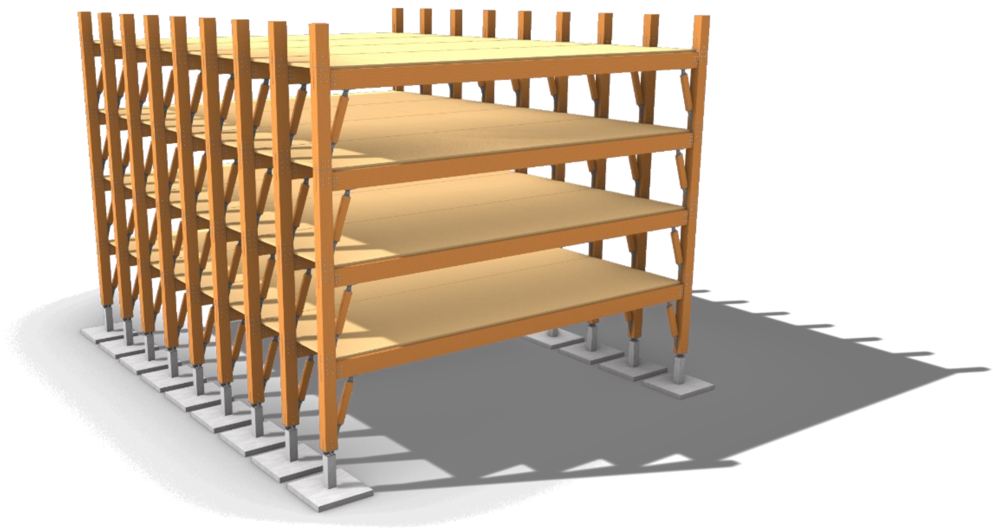


Figure 138. Design for proof of concept temporary multi-storey car park, primarily in timber.

12.3 Variant Study

The proof of concept is the result of a parametric study with structural validation into four different main typologies. Given all relevant aspects and the challenges found in the literature study, it was concluded that the main goal that these designs have to fulfil is threefold: focus on the lifecycle of the structural system, solve the material challenges related to building with timber and take into account the temporary nature of the to be developed concept. To be able to assess the performance of the design variants and compare the resulting proof of concept to alternatives, two specific sub-goals were derived from this main goal: minimization of total deck height and minimization of the weight of the structural system. Given the four main design variants, four geometric parameters were studied, altering the geometry of the main design variants, for their effect on the specific sub-goals. These are: parking deck span, use of struts, distance between columns in transverse direction and minimum distance between joists. Hypotheses were developed in paragraph 5.1.3.1, stating the expected effect of altering these parameters on the outcome of the sub-goals. Furthermore, the effect of altering the assessment criteria for fire resistance, vibrations and deformations on the specific sub-goals was studied. Based on the analysis of all (sub)variants, several conclusions can be drawn.

12.3.1 Parking Deck Span

The results from the parametric study show that an increase in parking deck span results in an increase of both total deck height and total weight of the structural system. For most design variants, this increase in height and weight is roughly linear. Only for the design variants using a long-span deck system, it has more of a logarithmic pattern.

12.3.2 Struts

The parametric study with structural validation has shown that the use of struts significantly reduces the cross-section of the main girder and as a result the total deck height (17-30% depending on design variant) for the design variants that may contain struts. Also a reduction in weight of the total structural system is found, although smaller than the reduction in total deck height (5-11%).

12.3.3 Transverse Column Distance

A transverse column distance of a single parking bay width (2.5 m) results in a smaller total deck height (16-23%) and smaller total weight of the structural system (5-13%) compared to using double the distance. Furthermore, due to the smaller area caried by a column, the load on the foundation is also roughly halved, allowing for smaller foundation works.

12.3.4 Distance between Joists

It is concluded that altering the distance between joists has a negligible effect on the total deck height. In terms of weight reduction, a decrease in maximum joist distance of 50% results in a reduction in weight of 5%.

12.3.5 Alternative Assessment Criteria

The effect of alternative assessment criteria has been studied mainly for different deck systems. The biggest effect is found for long-span deck systems.

The use of a long-span CLT rib panel deck system results in a relatively small total deck height (740 mm for 16.26 m span vs 666 m for proof of concept), but results in a doubling of the self-weight of the structural system in comparison with alternatives that use a short-span solid LVL panel deck system. However, alternative assessment criteria significantly increase the performance of long-span deck systems. When vibrational criteria are not considered and deformations of up to 1/150 x L are allowed, LVL rib panels become possible, resulting in a significant reduction in total deck height (537 mm for 16.26 m span) and almost halving of the self-weight of the deck. This results in a reduction of the weight of the total structural system of more than 40%.

The total deck height for design variants using a solid LVL panel as short-span deck system is highly dependant on the supporting framing system. Regardless of the framing system, the self-weight of the different variants varies within a margin of 20% and is at least halve that of the variant with a long-span deck system. In comparison to the design variants with a long-span deck system, for the design variants with short-span decks, alternative assessment criteria result in smaller reductions in total deck height (3-7%) and total weight of the structural system (10-20%).

12.4 Case Study

The proof of concept is compared to five alternative modular and/or timber multi-storey car park concepts for a span of 16.26 m. These alternatives have different typologies and materials and can be classified as followed: steel frame + concrete TT-plates, steel frame + GFRP deck, steel frame + steel-concrete composite deck, timber frame + concrete deck and fully timber frame and deck.

In terms of total deck height, the proof of concept has a deck height (LVL deck + glulam main girder) smaller than 700 mm (666 mm) and therefore fulfils the goal that was set in paragraph 7.3. It falls into the same range of deck heights as the other modular concepts in steel, concrete and GFRP. It is 17% thicker compared to the smallest deck height of the TT-plates, but 40% thinner compared to the existing alternative in timber.

Considering the self-weight of the structural system, the proof of concept has a self-weight slightly smaller (13% lower) compared to the existing alternative in timber. It cannot compete with the alternative concept, which uses GFRP decks (31% heavier), but is much lighter compared to the other systems which contain (partial) concrete decks. Compared to the steel-concrete composite deck system, the proof of concept has only halve its weight and compared to the fully concrete decks only a quarter.

Also the loads of a column of the structural system on the foundation were studied and with a load of 678 kN, it is lower than the goal set in paragraph 8.4 of 1,000 kN. In comparison with the alternative concepts, this is the lowest load. It is 10% smaller than the existing concept in timber and 43% smaller than the alternative concept in timber and concrete. The other alternatives all have twice the column distance compared to the proof of concept, but all result in a load at least twice that of the proof of concept.

Finally, a simplified sustainability performance analysis on embodied carbon indicates that the performance of the proof of concept is comparable to the existing alternative in timber. All of the other alternatives contain more than twice the amount of embodied carbon compared to the proof of concept and the alternative with GFRP decks almost four times as high. This analysis contains many assumptions, but is a first indication on the sustainability performance of the various concepts.

12.5 Final Word

The results of this thesis have shown through the development of a proof of concept, that a temporary multi-storey car park can be realised using primarily timber as structural material and that such a design can structurally compete with existing alternative modular multi-storey car park concepts in different materials.

13

Discussion & Recommendations

Following the conclusion, in this chapter, the results of this thesis are discussed and recommendations are presented. The discussion focusses on three parts: the results of the literature study, the results of the analysis of the different design variants for the proof of concept and the results of the case study. The discussions include validation and interpretation of the results, limitations of this research and recommendation for future research.

13.1 Discussion of Results Literature Study

In the literature study, three key topics have been addressed, which are concerned with the development of a proof of concept for a temporary and modular multi-storey car park primarily in timber. These are: car park design, challenges towards a sustainable building industry and timber structures. The findings from this literature study have resulted in a broad overview of relevant aspects of these various topics. Based on these results, in the concept development, certain choices have been made and some topics have been elaborated on more in detail, while others have not been further addressed. Therefore, the results from the developed proof of concept do not include all aspects discussed in the literature study. Below, some of these limitations in the results of the proof of concept are discussed.

13.1.1 Car Park Design

The requirements for multi-storey car parks and possible layouts have been extensively discussed in the literature study. For the structural system of the proof of concept, the requirements of the NEN 2443 norm have been considered as mandatory. Deviation of these requirements might result in a further improvement of the performance related to the specified sub-goals. Therefore, it is advised to further study what aspects of the norm limit the performance. Furthermore, the proof of concept is developed for a flat deck or split-level car park. No specific circulation layout has been designed and also the ramps are not included in the proof of concept. It is advised to further develop the proof of concept into different fully developed multi-storey car park designs, based on the findings from the literature study. This allows testing of the modularity and applicability of the concept as a temporary structure.

13.1.2 Building Sustainable

In the literature study, it was studied what principles can be applied to help the building industry in the transition towards a more sustainable industry. It was concluded that many of such principles are found in the approach of a circular economy, for which circular design strategies can be applied. Three key themes have been developed, which combine these circular design strategies and should be applied to the design of a temporary multi-storey car park to result in a sustainable structure:

- Modularization, prefabrication and standardization;
- Reuse, refurbish and recycle;
- Building materials.

These themes have been taken into account in the development of the proof of concept to a certain extend. For the choice of pavement surfacing, modularity has been taken into account and of course timber has been applied as a primary structural material. Additionally, many of the aspects of these themes are to be found in the detailing of the structure; how elements are connected, transported and reused for example. Such details have not been considered in the further development of the proof of concept in this thesis. It is recommended to further develop the proof of concept and apply these principles in the further detailing of the structure.

13.1.3 Building in Timber

Many relevant aspects and challenges of building in timber have been discussed in the literature study. Two important aspects that have not been further developed in the proof of concept and are important to point out are connections and durability.

The connections between different elements of a timber structure can significantly impact the dimensions of those elements. Due to the inhomogeneous material properties of timber, it can be a challenge to transfer loads between elements with different orientations or between elements of different materials. This can require larger cross-sections or use of different materials to be able to transfer the loads, adding complexity to the structure. Furthermore, connections with steel elements can be vulnerable to fire and require proper detailing. All together, the connections of a timber structure are very important and can impact the rest of the structural system. Therefore, it is recommended to further study the detailing of connections of the proof of concept and check what impact they have on the conclusions of this thesis.

Concerning durability of a timber structure, it was concluded that protection by constructional measures is the best solution. Such measures have not been the primary focus in the further development of the proof of concept. Protection can be realised by (slight alteration of) the main structural elements, but also by additional elements like the façade. Therefore, it is advised to further study how the timber elements can be properly protected from durability risks.

13.1.4 Suggestions for Future Research

Based on the discussion of the results of the literature study, the following recommendations for future research are suggested:

- Study what aspects of the NEN 2443 parking norm limit the performance of the proof of concept in terms of the specified sub-goals;
- Further develop the proof of concept into a fully completed design for a multi-storey car park, including ramps and a circulation layout, to study the modularity and performance in terms of user and owner requirements of the concept;
- Incorporate the circular design strategies in the further development of the proof of concept;
- Design the connections between all structural elements of the proof of concept;
- Further develop the design of the proof of concept to incorporate protection of the timber elements from durability risks.

13.2 Discussion of Results Design Variants for Proof of Concept

In total, four main design variants with multiple subvariants have been analysed in the development of the design for the proof of concept for the multi-storey car park. They consist out of a deck and framing system and are compared to each other on total deck height and weight of the structural system. Furthermore, effects of alteration of the assessment criteria for fire-safety, vibrations and deformations on these aspects have been studied. In this paragraph, the results of the comparison of these design alternatives and the effects of alternative assessment criteria are discussed.

13.2.1 Fire-safety

In the literature study, it was discussed that it is possible to allow a fire resistance class of R30 for a multi-storey car park if it is considered to be a single fire compartment. This is allowed based on the principle of equivalence, which has been assumed to be applicable. However, this is yet to be proven and requires further study into the subject. Since timber is often perceived to be a fire risk, requiring approval of a lower fire resistance class for a multi-storey car park in timber might be more difficult compared to one in steel and concrete.

For the proof of concept, only the decks have been analysed in detail for fire-safety. The other structural elements have not been validated for fire-safety. Nevertheless, for the proof of concept it is not expected to have an impact on the results. Verification of fire-safety ULS is expected to impact especially short span elements as was found for the deck elements. Therefore, the long-span main girders of the proof of concept are not expected to change as a result of the check on fire-safety. For other elements, it might result in a slight increase in cross-section, but this is expected to only have a small effect on the total weight of the structural system and not alter the order of the studied variants.

As was concluded in the variant analysis, the impact of altering fire-safety requirements is rather limited. It impacts the short-span deck systems, but in absolute terms, the effect is very small. Therefore, alteration of the fire-safety requirements is not expected to influence the outcomes of the comparison of the proof of concept.

13.2.2 Vibrations

Vibrational criteria have been taken into account in the development of the structural system of the proof of concept for the deck systems. However, the criteria considered are developed with residential and office buildings in mind and may therefore not be applicable to a multi-storey car park. Since people spend only a short amount of time in a car park partially of which duration they are seated in their car (which has additional suspension), the applied criteria are considered to be a conservative upper limit for vibrational behaviour. According to the Dutch Building Degree, no specific criteria concerned with vibrations apply to a car park, but this does not mean vibrations cannot be unpleasant. Especially since timber structures are lightweight and can be vulnerable to vibrational behaviour, it is advised to further study what limitations in terms of vibrational behaviour should be applied to a multi-storey car park.

Apart from the deck systems, the other structural elements have not been validated for vibrational behaviour. For elements with only a short span, vibrational behaviour is not expected to result in different cross-sections than found in this study, since vibrations only became problematic for deck systems for spans of 10 m and longer. However, for the main girders of the various design variants with long spans, validation of vibrations might result in an increase in cross-section.

As the considered vibrational criteria are an upper limit, it is expected that these can be lowered, which influences the outcomes of the analysis. Altering just the vibrational criteria does not affect the short-span deck systems and therefore does not result in a change of the structural system of the proof of concept. Its conclusions therefore still stand. However, it does significantly impact the performance of the design variants with a long-span deck system. These variants should therefore be studied for the found vibrational criteria and again be compared to the proof of concept for its performance. It is expected that this does result in a better performance in terms of total deck height, but not in terms of total weight.

13.2.3 Deformations

The Eurocode gives advice concerning the maximum allowable deflections. As concluded in the literature study, according to the Dutch Building Degree, the designer can deviate from these values. Therefore, in this study, it has been analysed what the effect is of increasing the allowable deflections from 1/250*L to 1/150*L. However, this new value should be validated for what effect is has on the structural system of a modular car park. It is important to check how different elements fit together, to prevent additional impact loads between ramps and floor elements for example. Especially the effect of live loads on the deflections is important in such cases. Since timber is a lightweight material, the live load presents a bigger percentage in the total load on the structure and therefore, it might be concluded that bigger deformations are actually not possible or only for certain elements.

Different limits for deformations mainly impact the choice of long-span deck system. For short-span deck systems, only in combination with alternative criteria for both fire-safety and vibrations, does it result in a different type of panel in which case the effects are still limited. Therefore it is not expected that allowing bigger deformations has a significant effect on the design of the proof of concept. However, it does significantly impact the performance of the design variants with a long-span deck system if also different vibrational criteria apply. In that case, these variants should therefore be studied for the found alternative criteria and again be compared to the proof of concept for its performance. It is expected that this does result in a better performance in terms of total deck height, but not in terms of total weight.

13.2.4 Use of Struts

It was concluded that the use of struts has a significant effect on the total deck height (17-30%) and structural systems weight (5-11%). The applied inward strut distance is rather limited at just 670 mm. Increasing this distance might result in a further decrease of the deck height and weight of the structure.

Furthermore, for increased angles, the struts might also be able to provide vertical stability of the structure. However, the strut might hinder cars in entering and exiting the parking bay or people entering or exiting their vehicles. Therefore it is advised to study what effect increasing the angle of the struts has on the dimensions of the cross-sections of the various elements and if it can act as stabilizing element. Second, it is advised to study how far and at which height the strut can be located without hindering car parks users.

13.2.5 Suggestions for Future Research

Based on the discussion of the results of the design variants for the proof of concept, the following recommendations for future research are suggested:

- Study if fire-safety requirements can be altered based on the principle of equivalence;
- Analyse the framing system of the proof of concept for fire-safety requirements;
- Study what vibrational criteria are best applicable to a multi-storey car park;
- Analyse the framing system of the proof of concept for vibrational behaviour;
- Study what maximum deformations are allowed for the different structural elements of a multi-storey car park, giving special attention to the impact of live loads on the deformations of timber elements;
- Study what effect increasing the angle of the struts has on the dimensions of the cross-sections of the various elements and if it can act as stabilizing element;
- Study how far and at which height a strut can be located without hindering car parks users.

13.3 Discussion of Results Comparison with Alternative Concepts

The developed proof of concept has been compared to five alternative modular and/or timber multi-storey car park concepts on two themes: total deck height and weight of the structural system. These alternatives and the proof of concept have been compared to each other on four specific measurable aspects: total deck height, weight per level, load on foundation and embodied carbon. In this paragraph, the results of the comparison are discussed.

13.3.1 Structural Comparison

All of the alternatives studied have been designed with different starting points and goals in mind for different locations both in The Netherlands and abroad. This results in different structural systems with different geometric configurations and cross-sectional dimensions. To be able to compare the different concepts with each other, only the parking deck span and grid size of the various concepts have been translated to the same dimensions, to be able to make a relevant comparison on deck height and the structures weight. However, this leaves a lot of unknowns considering how the structure was designed and with what performance goals in mind. This can influence the validity of the comparison between the proof of concept and the alternative concepts. Examples of aspects that could affect the comparison are:

- Applied structural rules and norms;
- Applied loads on the structure;
- Applied deformation limits;
- Applied fire-safety class;
- Whether vibrational behaviour has been checked;
- Whether the structure has been over dimensioned for a possible different function or extension in the future;
- Cross-sections based on different grid size than used for comparison.

As a result of the aspects mentioned above, the various alternative concepts could actually perform both better or worse than was found in the comparison. Except for the concepts, which contain timber, all other alternative concepts have been realised in The Netherlands. Therefore, it is expected that these mainly make use of the same rules and norms and as a result are a good comparison. It is expected that the biggest impact on the comparison is the result of how vibrational criteria are applied and whether the structure has a significantly different grid size than used for the comparison. The alternative in timber has been realised in Germany, but is best to compare to since it is also fully made from timber. Such a comparison can be used to interpret the results of the proof of concept.

In terms of total deck height, the proof of concept is 29% thinner compared to the existing alternative in timber, which uses a comparable typology. The deck of the proof of concept is 4 mm thinner (LVL instead of CLT deck), but the biggest difference is found for the height of the main girder. The timber alternative concept has a tapered glulam (BauBuche GL75) beam varying from 600x240 to 760x240 mm and the proof of concept uses a glulam (GL28h) beam of 570x280 mm. Even though the alternative concept uses a stronger material, the use of struts in the proof of concept explains the significantly lower beam height. In comparison, without the use of struts, the beam would require a height of 850 mm, which is somewhat higher than the beam of the timber alternative. This highlights the impact of using struts on the dimensions of the cross-section of the main girder. It also shows that by using BauBuche as a material, which was not considered in the proof of concept development, the cross-sections can be reduced even further. In terms of weight, the proof of concept is slightly lighter as a result of its smaller cross-sections.

13.3.2 Sustainability Assessment

Focussing on the sustainability analysis of the embodied carbon, the analysis is very simplified and brief. Except for the key figure for the GFRP material, all figures come from the same source. Therefore, they have all been derived in the same way and are therefore comparable. Unfortunately, the source of the key figure for GFRP is different, which can affect the comparison, since possibly different criteria are used to derive this figure. The figure applied is in the lower region of the different values found and therefore expected to be a bottom line for the amount of embodied carbon for GFRP. For all alternatives, the same types of structural elements have been considered and materials used for finishing and connections have not been considered. However, this can have a significant effect on the amount of embodied carbon as could already be seen for the use of GFRP, which has a high amount of embodied carbon. Taking into account all materials used in the structure could therefore result in a different view of the comparison.

Considering the embodied carbon of the various concepts, it is based on the weight of the various materials used in their structural designs. Logically, the amount of embodied carbon in the proof of concept is slightly lower than in the fully timber alternative, since they use the same type of material, but the proof of concept is slightly lighter. As expected, the other concepts have significantly higher amounts of embodied carbon. The use of GFRP results in an amount of embodied carbon almost four times as high as the proof of concept. The use of GRRP as main structural material for the decks therefore has a high impact on the sustainability performance.

For timber materials, the possibility of storing carbon inside the timber is not taken into account in the assessment and is therefore expected to perform even better in a more detailed analysis. The presented number for amount of embodied carbon in the main structural elements is therefore expected to be a upper limit. For steel, in the comparison it is assumed the steel has been recycled. If this would not be the case, the amount of embodied carbon in the main structural elements of the concepts using steel would actually be higher. The presented number is therefore expected to be a bottom limit. The types and amounts of additional materials used for other elements like façades, installations, connections, finishing etc. are not expected to have an impact big enough to change the order of the comparison between the concepts. Foundations may be lighter for the timber and GFRP concepts, resulting in a lower additional amount of embodied carbon compared to the concepts using concrete. At the same time, the timber and GFRP concepts require an additional protective layer of their decks, which may result in an additional amount of embodied carbon. A detailed analysis should point out the exact amount of embodied carbon for each concept, but the order of performance of the concepts is not expected to change.

13.3.3 Suggestions for Future Research

Resulting from the discussion of the results of the comparison between the proof of concept and other alternative modular and/or timber multi-storey car park concepts, two suggestions are given for future research:

- Study use of BauBuche and other innovative sustainable materials;
- Make a detailed LCA study to asses the performance on sustainability of the various concepts.

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Appendices

A.

Structural Loads and Limit States

A.1 Load Combinations (ULS/SLS)

Ultimate Limit States (ULS)

Table 51. Partial factors on actions (STR/GEO) (group B) (Nederlands Normalisatie Instituut, 2015)

Design situation	**Permanent**		**Leading variable**	**Accompanying variables**	
Formula for combination of actions	Unfavourable	Favourable		Dominant (if present)	Others
(Eq. 6.10a)	1.35 $G_{k,j,sup}$ [a]	0.9 $G_{k,j,inf}$		1.5 $\psi_{0,1}$ $Q_{k,1}$	1.5 $\psi_{0,1}$ $Q_{k,I}$ (i>1)
(Eq. 6.10b)	1.2 $G_{k,j,sup}$ [b]	0.9 $G_{k,j,inf}$	1.5 $Q_{k,1}$		1.5 $\psi_{0,1}$ $Q_{k,I}$ (i>1)

[a] In the case of fluid pressures with a physically limited value, the following shall suffice: 1.2 $G_{k,j,sup}$
[b] This value is calculated with ξ=0.89

Accompanying equations:

$$\sum_{j\geq 1} \gamma_{Gj} G_{kj} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} \psi_{0,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(Eq. 6.10a)} \qquad 0.1$$

$$\sum_{j\geq 1} \xi_j \gamma_{Gj} G_{kj} \text{"+"} \gamma_P P \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,i} Q_{k,i} \qquad \text{(Eq. 6.10b)} \qquad 0.2$$

Serviceability Limit States (SLS)

Characteristic

Irreversible serviceability limit states are generally assessed using this combination of actions.

$$\sum_{J\geq 1} G_{kj} \text{"+"} P \text{"+"} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{0,i} Q_{k,i} \qquad \text{(Eq. 6.14b)} \qquad 0.3$$

Frequent

Reversible serviceability limit states are generally assessed using this combination of actions.

$$\sum_{J\geq 1} G_{kj} \text{"+"} P \text{"+"} \psi_{1,1} Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad \text{(Eq. 6.15b)} \qquad 0.4$$

Quasi-permanent

Long-term effects and the appearance of the structure are generally assessed using this combination of actions.

$$\sum_{j\geq 1} G_{kj} "+" "P" "+" \sum_{i>1} \psi_{2,i} Q_{k,i} \qquad \text{(Eq. 6.16b)} \qquad 0.5$$

An overview of the combinations of actions can be found in Table 52.

Table 52. Overview of combinations of actions. (Nederlands Normalisatie Instituut, 2015)

Combination of actions	**Permanent (G_d)**		**Prestressing (P_d)**	**Variable**	
	Unfavourable	Favourable		Leading ($Q_{d,1}$)	Accompanying ($Q_{d,j}$)
Characteristic	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$Q_{k,1}$	$\psi_{0,i}\ Q_{k,i}$
Frequent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{1,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$
Quasi-permanent	$G_{k,j,sup}$	$G_{k,j,inf}$	P	$\psi_{2,1}\ Q_{k,1}$	$\psi_{2,i}\ Q_{k,i}$

Combination factors

Table 53. Combination factors for buildings. (Nederlands Normalisatie Instituut, 2015)

Action	$\boldsymbol{\psi_0}$	$\boldsymbol{\psi_1}$	$\boldsymbol{\psi_2}$
Imposed loads on buildings:			
Category F: Traffic area, vehicle weight ≤ 25 kN	0.7	0.7	0.6
Snow loads	0	0.2	0
Wind loads	0	0.2	0
Temperature (not fire)	0	0.5	0
Standing water	0	0	0

A.2 Variable Loads

Snow

The snow load can be calculated using equation 0.6. In The Netherlands, C_e and C_t are equal to 1. The characteristic value s_k in The Netherlands is equal to 0.7 (Nederlands Normalisatie Instituut, 2019b). The snow load shape coefficient μ_i depends on the shape of the roof and is therefore dependant on the design. Equation 0.6 can now be reduced to equation 0.7.

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k \qquad 0.6$$

$$s = 0{,}7 \cdot \mu_i \qquad 0.7$$

Where:

s	is the characteristic value of the snow load [kN/m^2]
s_k	is the characteristic value of the snow load on the ground [kN/m^2]
μ_i	is the snow load shape coefficient [-]
C_e	is the exposure coefficient [-]
C_t	is the thermal coefficient [-]

Wind

The wind load on a structural element can be calculated using equation 0.8. Since the car park will be an open structure, it is assumed the factor c_sc_d is equal to 1. Factor c_f depends on the layout of the building. Factor $q_p(z_e)$ depends on multiple aspects, among which is the height of the building. For now, it is assumed the construction can be located anywhere in The Netherlands. Therefore, the most extreme situation is considered:

- Windregion I
- Coastal area
- Height to be determined

Wind load on structural element:

$$F_w = c_s c_d \cdot c_f \cdot q_p(z_e) \cdot A_{ref} \quad (0.8)$$

Where:

F_w	is the wind force [kN]
c_sc_d	is the structural factor [-]
c_f	is the force coefficient [-]
$q_p(z_e)$	is the peak velocity pressure [kN/m^2]
A_{ref}	is the reference area [m^2]

A.3 Deflections and Displacements

Vertical Deflections

The maximal allowable vertical deflections can be found in Table 54. The different parameters used are displayed in Figure 139.

Table 54. Allowed vertical deflections. (Nederlands Normalisatie Instituut, 2015)

Type	**Allowed additional deflection ($w_2 + w_3$) [$*\ell_{rep}$]**	**Allowed final deflection (w_{max}) [$*\ell_{rep}$]**
Floors	0.003	0.004
Floors with crack-sensitive partitions	0.002	0.004
Roof	0.004	0.004
Roof frequently walked on	0.003	0.004
Floor partitions	0.0067	0.0067

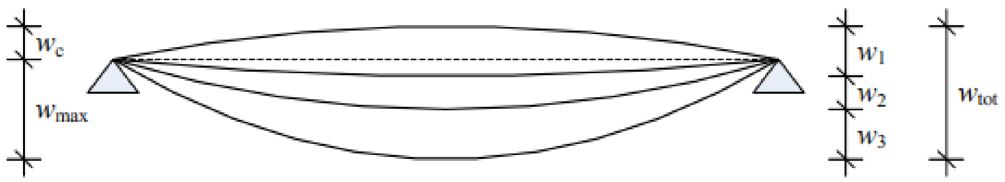

Figure 139. Vertical deflection parameters. (Nederlands Normalisatie Instituut, 2015)

Horizontal Displacements

The total maximum horizontal displacement for buildings with more than one storey under the characteristic combination of actions is:

- h/300 per storey (u_i Figure 140)
- h/500 for the entire building (u Figure 140)

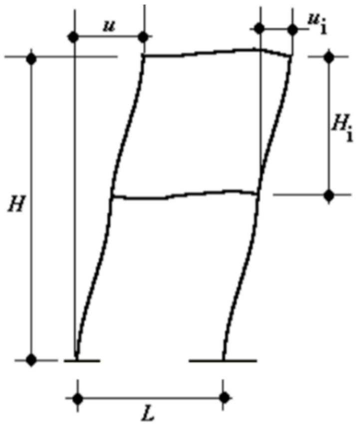

Figure 140. Definition of horizontal displacements. (Nederlands Normalisatie Instituut, 2015)

Gradient of Floors

To prevent the forming of ponds on water draining surfaces, sufficient gradient and camber has to be incorporated in floors and roofs, draining water towards the available drains, see Figure 141. The characteristic combination of actions should be used in which the imposed load or snow load is leading in the variable actions.

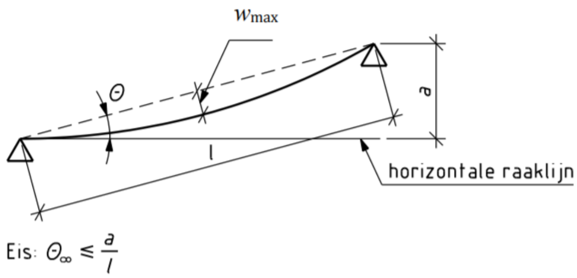

Figure 141. Required gradient as a function of deflection to prevent ponding. (Nederlands Normalisatie Instituut, 2015)

B.

Car Park Circulation Layouts

B.1 Flat Decks

Flat deck car parks are normally built in multiples of a bin width and have an adaptable layout (The Institution of Structural Engineers, 2011). Overall, three different types of flat deck layouts can be distinguished, based on the ramp system used: internal ramps, half external ramps and external ramps. Generally speaking, internal ramps result in low dynamic and static efficiency. Half external ramps cost extra, but reduce the amount of bays used to complete the circulation route, boosting dynamic efficiency. In small car parks, half external ramps may be uneconomic, but with larger car parks, its static efficiency increases. External ramps function independently form the car park and are often used in large car park facilities, providing high dynamic efficiency and for large car parks reasonable static efficiency. (Hill et al., 2005)

The following flat deck layouts can be distinguished:

Internal ramps running across the bins

In a layout with internal ramps running across the bins, all turns are in the same direction with no single turn greater than 90º, recirculation is simple, pedestrians have flat access between adjacent bins and the outflow route is reasonably rapid. Having the internal ramps running across the bins however has a significant impact on the dimensions of the car park, since the ramps have a length around 25 metres, resulting in a minimum width of three bins, see Figure 142. To keep the slope of the ramps acceptable, floor heights should be minimized. Internal ramps also lead to dead ends and the traffic flow pattern has many conflict points, both resulting in a low circulation efficiency. Both static and dynamic efficiency is low, as many bays have to be passed twice to search each deck level. The static efficiency lies around 24 m^2 per car for a deck with 112 bays. (Hill et al., 2005)

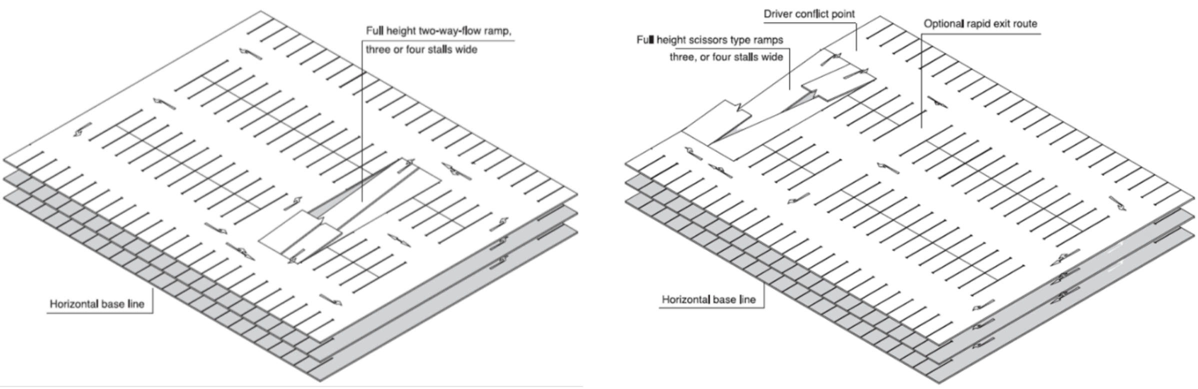

Figure 142. Left: One-way-flow with combined two-way ramps. Right: One-way-flow with side-by-side ramps (scissors type). (Hill et al., 2005)

Internal ramps running parallel with the traffic aisles

Having internal ramps running parallel with the traffic aisles provides simple recirculation capability, flat access for pedestrians between adjacent bins and a reasonably rapid outflow route, like with ramps running across the bins. The different layout of the ramps however eliminates dead ends and allows for any given story height with desired slope of the ramps. This layout can result in potential conflict between traffic and when the ramps are located on the sides of the car park, turns in two directions may follow each other, see Figure 143. The circulation efficiency however is low with a minimum of 44 bays per deck to complete the circulation route. This type of layout should only be used in car parks with large decks, which impacts the static efficiency. For a small layout, the static efficiency drops to around 28 m² per car, which is poor. (Hill et al., 2005)

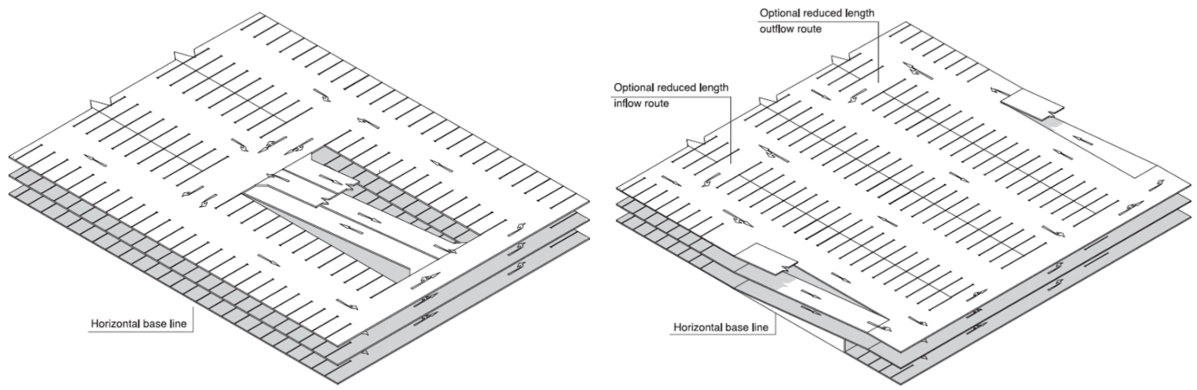

Figure 143. Left: One-way-flow with two-way-flow ramps. Right: One-way-flow with edge ramps. (Hill et al., 2005)

Half external ramps

Different external ramp types exist, half spiral with one-way-flow and different configurations of straight ramps with one-way-flow. All of these types of ramps can be used with a flat parking deck layout. The choice for a certain ramp is more subjective, with spiral often seen as a more attractive, but more expensive option. Using half external ramps, both flow routes can be rapid or extended and the flow routes can be located side-by-side or separated. This layout provides good recirculation capability and the parking decks are clear of ramp obstructions. The circulation route can be kept extremely short with eight bays when the ramps are located on the side of an aisle, see Figure 144, and just four bays when located at the end of an aisle. In these layouts, both inflow and outflow routes combine with the aisle traffic at each parking level, but this is normally not a serious matter. If desired, an extra aisle as part of the ramp system can be added, resulting in an external ramp type at the cost of an extra 10 metres in width. Dynamic efficiency is good (up to 1100 vehicles per hour or 1500 with the full external option), especially when variable message sign systems are incorporated, eliminating the need to search each floor.

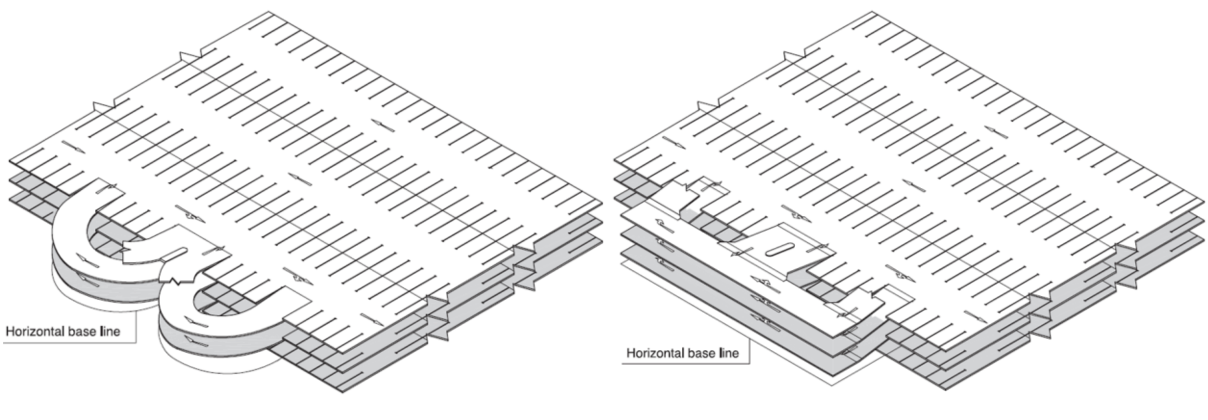

Figure 144. Left: Half spiral with one-way-flow. Right: Straight ramps with one-way-flow (side located) (Hill et al., 2005)

Circular external ramps

Circular external ramps can accommodate minimal width sites, with a minimal width of just 16 metres for the parking deck and 19 metres for the circular ramp (one-way traffic flow), see Figure 145. Generally speaking, they are not popular with the public, which has mostly to do with the minimum diameter. On the in- and outflow routes, decks can be bypassed, resulting in a high dynamic efficiency through the use of variable message signs. The traffic circulation route is vertical rather than horizontal. The static efficiency of the bays is high, but in total it can vary dramatically, depending on the ratio of the size of the parking deck and the size of the ramps. With 56 bays per deck, the static efficiency lies around 28 m^2 per car, which is average. For 100 bays, this increases to 24 m^2 per car, which is good for this type of layout. Using two-way traffic flow ramps results in a crossover condition at the entry of each parking deck. Furthermore, the diameter of the circular ramps increases to 30 metres. Having one structure for both flow routes can prove beneficial in site utilisation. (Hill et al., 2005)

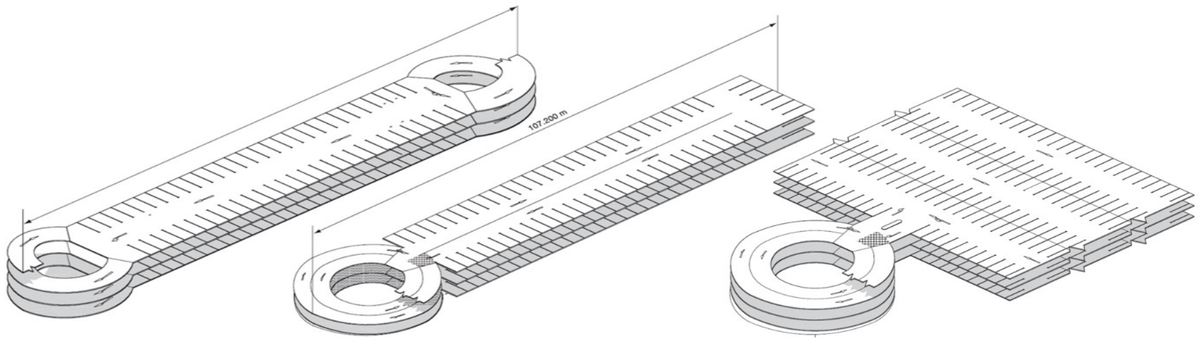

Figure 145. Left: One-way-flow between end ramps. Middle: Two-way-flow with a single two-way-flow ramp. Right: Full circular ramp with two-way-flow. (Hill et al., 2005)

Storey height, straight external ramps

Storey height, straight ramps are another form of external ramps, creating a vertical circulation route, unobstructed by other traffic, apart from joining or leaving the ramp system. It has a good dynamic capacity of 1480 vehicles per hour, which can be optimized with a variable message sign system. As with other external ramp systems, just four bays per deck are required to complete the circulation route. A limiting factor in this layout is the length of the car park, since for each level climbed or descended, roughly 14 bays of length are required, see Figure 146. The total length of the car park therefore sets a limit on the number of levels possible. It is possible to combine both traffic flows on one side, saving 5 metres in width, but this will result in conflicts joining and exiting the ramp system. (Hill et al., 2005)

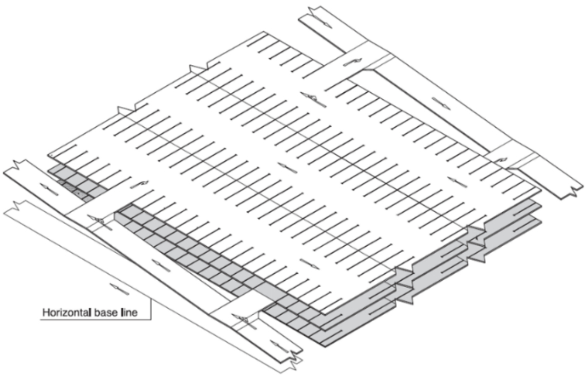

Figure 146. Storey height, straight ramps (Hill et al., 2005)

B.2 Split-level Decks

Split-level deck car parks generally have a good static and dynamic efficiency and are easy to drive around. They can accommodate large capacities with rapid in- and outflow routes, with most variants using one-way aisles, allowing any angle of parking. These layouts with their internal ramp systems are very compact and have a high static efficiency, most varying around 21 m^2 per car. The shortest variant can be constructed with a length of just 24 metres, resulting in a static efficiency of 26.75 m^2 per car. Except for the variants which are three or more bins wide, for drivers all turns are in the same direction and no single turn is greater than 90º. (Hill et al., 2005)

The following split-level deck layouts can be distinguished:

One-way-flow with side-by-side ramps (scissors type)

This layout has a full search plan, with its inflow route passing all bays, resulting in a high circulation efficiency. Unfortunately, the outflow route also passes all bays, resulting in a high degree of conflicts, see Figure 147. Recirculation is simple, but narrow width ramps are not popular and reduce the dynamic efficiency. This layout is a very compact option with a minimal length of 24 metres. Its static efficiency amounts to 21.06 m^2 per car for a length of 28 bays and 26.75 m^2 per car for its minimal length of 24 metres. (Hill et al., 2005; The Institution of Structural Engineers, 2011)

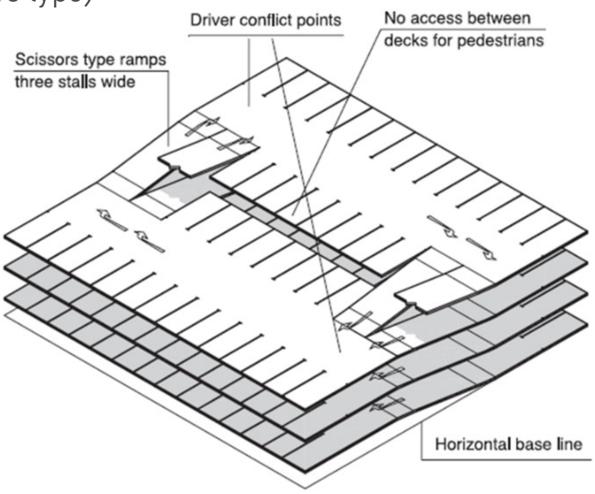

Figure 147. One-way-flow with side-by-side ramps (scissors type) (Hill et al., 2005)

One-way traffic flow with an excluded rapid outflow route

Passing 80% of the bays on the inflow route, this layout has a reasonable search plan. The location of the ramps results in a rapid outflow route. In- and outflow routes are separated, minimizing conflicts, see Figure 148. Also, recirculation is simple. For smaller car parks (<500 spaces), the internal rams can be combined for compatibility of the structure. This layout can be used for large car parks with up to 1100 spaces (for short to medium stay), if rapid inflow routes are incorporated. The static efficiency amounts to 21.84 m^2 per car for a total of 48 spaces per split-level deck. (Hill et al., 2005; The Institution of Structural Engineers, 2011)

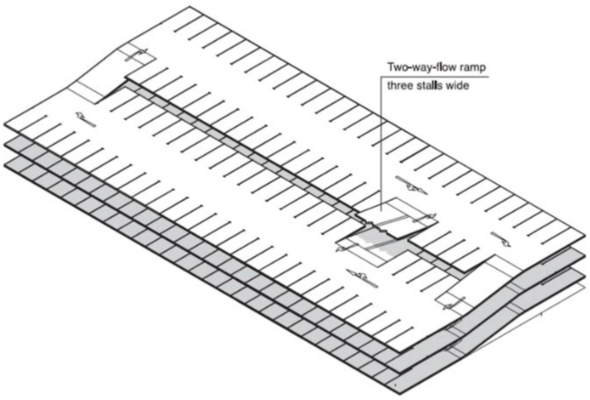

Figure 148. One-way traffic flow with an excluded rapid outflow route (Hill et al., 2005)

One-way traffic flow with an included rapid outflow route

This layout has a highly efficient inflow circulation, passing all bays, but also a rapid outflow route. The result is a reasonable search plan with simple recirculation options. A disadvantage is the combined use of aisles on the in- and outflow routes, which can result in congestion and conflicts, see Figure 149. This layout is suitable for car parks with up to 400 bays (short or medium stay). The static efficiency amounts to 21.84 m^2 per car for a total of 48 spaces per split-level deck. (Hill et al., 2005)

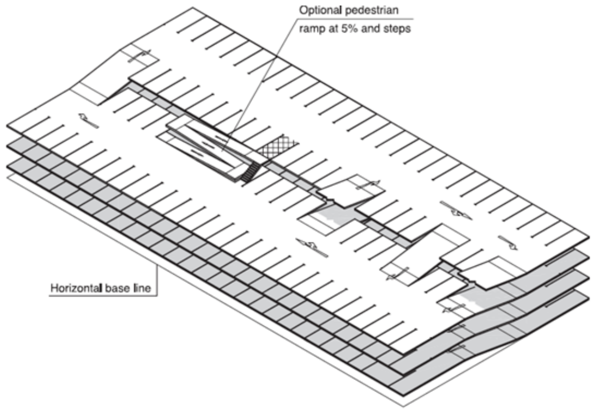

Figure 149. One-way traffic flow with an included rapid outflow route (Hill et al., 2005)

Two-way-flow with 'combined' ramps

In this layout, two-way aisles are used. All bays lay directly off the main inflow route. However, in opposite direction, this is also the outflow route. The two-way traffic flow in this layout is dynamically less efficient, compared to one-way flows and makes recirculation very difficult, see Figure 150. This layout can be very compact (as small as 24 metres in length). As a result, this layout is only suitable for small long stay car parks with up to 300 spaces. The static efficiency amounts to 21.06 m^2 per car for a length of 28 bays and rises to 26.75 m^2 per car in its most compact form. (Hill et al., 2005)

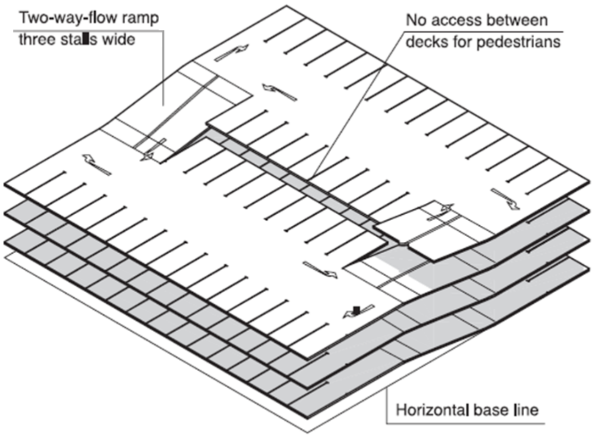

Figure 150. Two-way-flow with 'combined' ramps (Hill, Rhodes, Vollar, & Whapples, 2005)

Three bins or more wide

Layouts with three or more bins in width can also be used. These layouts generally have a poor search pattern. Two thirds of the bays are passed both on the in- and outflow routes, see Figure 151. Entering and exiting the ramps results in conflicts between drivers. Some variants use two-way traffic aisles, requiring the concerned bin to be wider compared to the others. Recirculation in these layouts is easy and the static efficiency is around 21 m^2 per car for a length of 28 bays. (Hill et al., 2005)

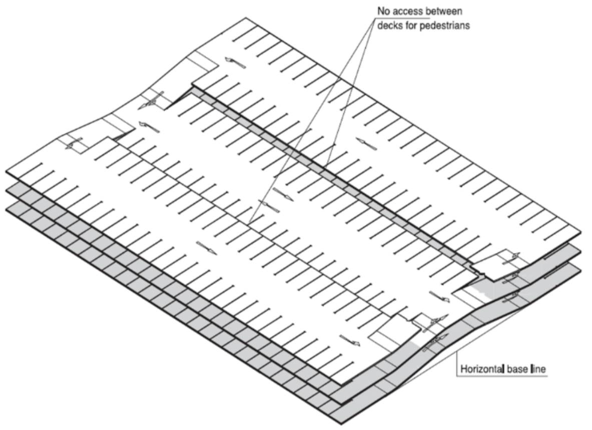

Figure 151. Combined one-way-flows, three bins or more wide (Hill et al., 2005)

B.3 Sloped Parking Decks

Sloped parking decks switch the ways parking decks and ramps are constructed. They use sloped parking decks with a limited slope and flat ramps. The slope of the parking deck is limited (maximum 5%) by the effect of gravity on opening and closing of doors. Also for disabled drivers in wheelchairs and the use of shopping cars, sloped parking decks are a disadvantage. These layouts generally have good static efficiency between 20 and 22 m^2 per car. The dynamic efficiency largely depends on the variant and whether it has a rapid in- and/or outflow route. Sloped parking decks can also be combined with one of the normally sloped decks replaced by a flat deck to compensate some of the disadvantages of a sloped parking deck. This however results in longer parking decks, greater than 72 metres. (Hill et al., 2005)

The following sloped parking deck layouts can be distinguished:

Single helix

According to Stuart (2007), a single helix layout is the simplest and cheapest configuration of all different types of layouts. On the inflow route, it passes all of the bays and only twelve bays per deck are required to complete the circulation route. At each end, flat access for pedestrians is provided. By varying deck length and slope, different storey heights can be incorporated. A minimum overall length of roughly 43 metres is required as a result of the maximum parking slope. The two-way traffic flows are less efficient than one-way traffic flows for both static and dynamic efficiency. The single helix layout doesn't have a rapid outflow route nor is there recirculation ability, which can result in congestion during busy periods. This layout is suitable for up to 300 spaces with a good static efficiency of 22.3 m^2 per car for a deck length of 28 bays. In its shortest configuration (18 bays long), this increases to 23.9 m^2 per car. (Hill et al., 2005)

Some disadvantages of the single helix layout can be improved by adding a half external ramp as rapid outflow route, see Figure 152. This increases dynamic capacity, reduces congestion and provides full recirculation capabilities, making it suitable up to 600 spaces. The external ramp requires extra space, however one-way-flow reduces required aisle width. As a result, from a length of 52 metres onwards, the extra space required by the ramp is compensated by reduction in aisle width. Static efficiency is good with 22.07 m^2 per car for a deck length of 28.5 bays. (Hill et al., 2005)

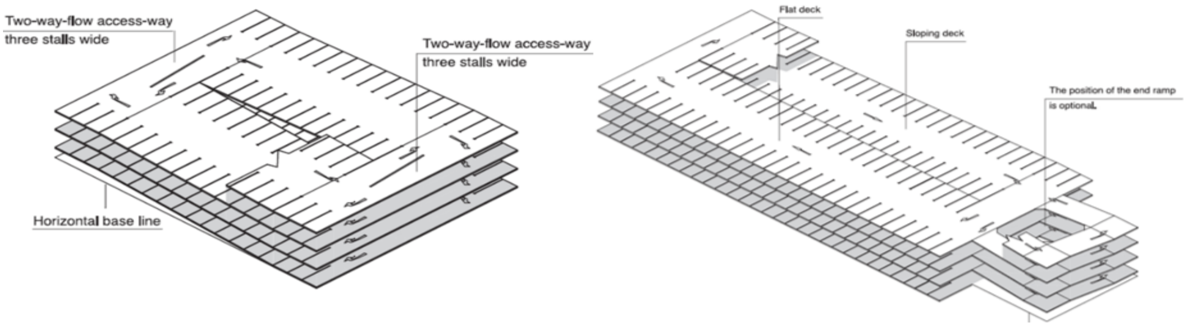

Figure 152. Left: Two-way-flow single helix. Right: One-way-flow single helix rapid outflow route. (Hill et al., 2005)

Double helix

Several types of double helix layouts can be distinguished, but in general the layout consists out of double sloping floor ramps with one-way traffic, providing simple recirculation capability. The inflow route slopes upwards and the outflow route downwards, both passing half of the bays, resulting in a poor search pattern, lacking a rapid in- or outflow route. This together with possible conflicts between in- and outflow traffic entering the central access-way can result in congestion. The circulation route can be completed with 12 to 16 bays per deck. Storey heights are accompanied by varying deck slope and length. The minimal required building length is 72 metres (30 bays), resulting in a good static efficiency of 21.1 m^2 per car. These type of layouts are generally preferred for situations, which involve everyday users (Stuart, 2007). The static efficiency can even be improved up to 20.06 m^2 per car by eliminating the central cross-over, see Figure 153, however this aggravates the disadvantages of circulation and therefore makes the layout unusable for short, medium and long stay car parks. (Hill et al., 2005; Stuart, 2007; The Institution of Structural Engineers, 2011)

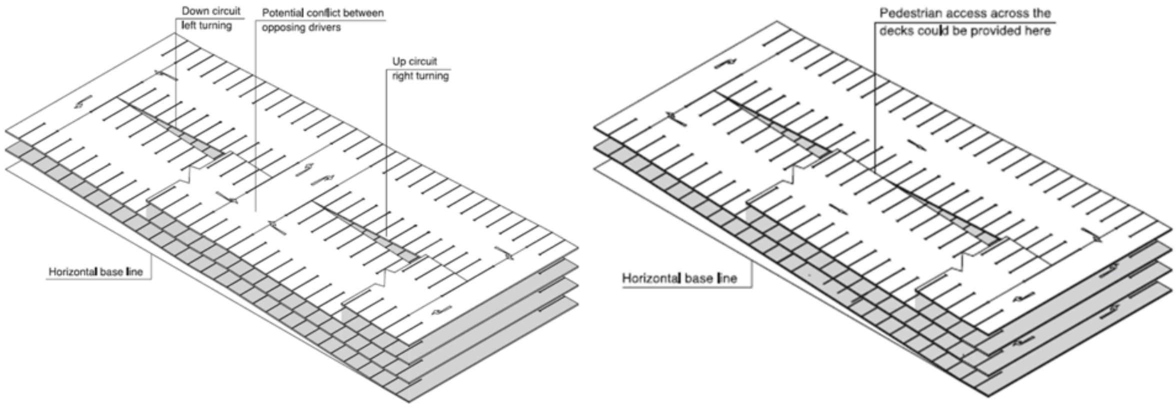

Figure 153. Left: Double helix, end connected with one-way-flow on the central access-way. Right: Interlocking double helix, with one-way-flows. (Hill et al., 2005)

Combined helix

The combined helix layout is three bins wide and combines two-way-flows in the middle aisle and one-way-flow on the outward aisles, see Figure 154. This results in simple circulation and recirculation ability with all turns in the same direction and not bigger than 90º, but lacks a rapid outflow route. On both the in- and outflow route, two thirds of the bays are passed and the central aisle has to be driven through twice. The two-way-flow in the middle aisle is also less efficient both statically and dynamically and can lead to congestion, making this layout not suitable for short and medium stay. The static efficiency is average with 22.9 m^2 per car for a deck length of 16 bays (80 bays per deck). By replacing one of the sloped decks with a flat deck, it is possible to create only one-way-flows. This increases the static efficiency to 21.13 m^2 per car, which is good. However, the long circulation route and driver conflicts remain the same.

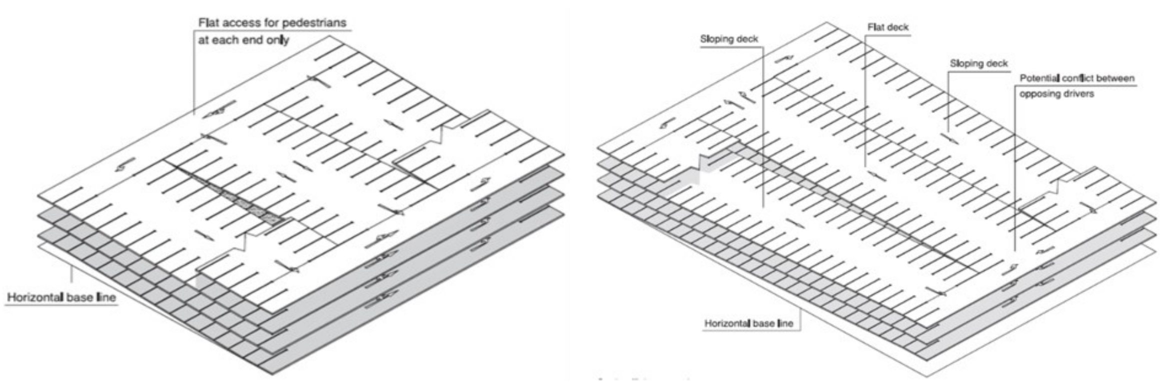

Figure 154. Left: Combined helix, side connected with one- and two-way-flows. Right: Combined helix, side connected with one-way-flow. (Hill et al., 2005)

Combined flat and sloping deck layouts with internal cross-ramps

Combining a vertical circulation module with sloped floors with an extended flat deck results in a car park layout with a significant improvement in cross-deck accessibility for pedestrians. It has good static and dynamic efficiency. In the layout, a rapid inflow route can be incorporated and 80% of the bays can be searched efficiently on the inflow circuit. Furthermore, all turns are in the same direction and no bigger than 90 ° and it has simple recirculation ability, see Figure 155. The layout requires a minimum length of 36 metres (15 bays wide). With 96 bays per deck, the static efficiency is good with 21.84 m^2 per car.

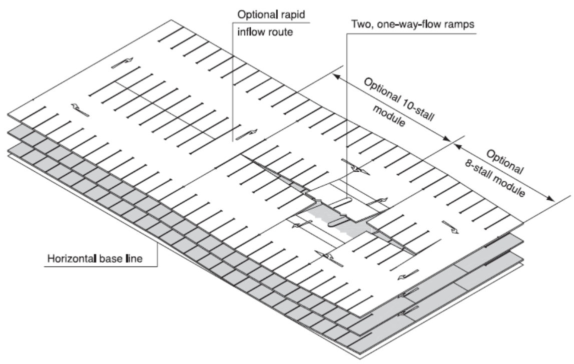

Figure 155. One-way-flow with two one-way-flow ramps. (Hill et al., 2005)

C.

Timber Joints

C.1 Detailed Overview of Timber Joints

C.1.1 Carpentry Joints

Scarf Joint

In a scarf joint, two members are connected to each other by removing material from both members to result in a joint with the thickness of the thickest member, see Figure 156. As material is removed from the members, the cross-section is considerably weakened, resulting in a relatively low load-carrying capacity. Nevertheless, scarf joints are considered to be the strongest form of unglued member lengthening (Harte & Dietsch, 2015). Scarf joints are often applied in older historical buildings in roofs and frames (Herzog et al., 2004).

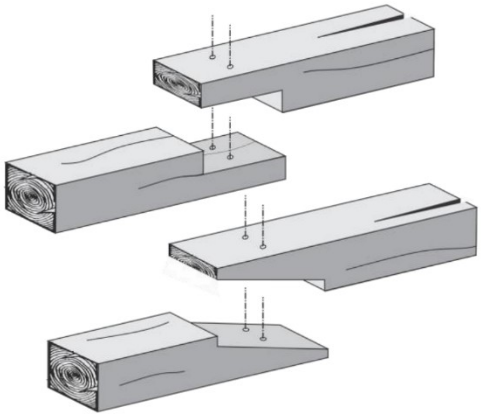

Figure 156. Scarf joint (Harte & Dietsch, 2015)

Lap Joint

Two sorts of lapped joints can be distinguished. The first is the full-lap joint, in which two members are placed over each other and connected to each other by a pin. The second type is a half-lap joint with or without cogs, see Figure 157. In this case, material is removed from both members to make them slide into each other, resulting in a joint with the same thickness as the thickest member in the joint (Harte & Dietsch, 2015). These connections are not used often anymore and were used in floor and roof framing (Blaß & Sandhaas, 2017).

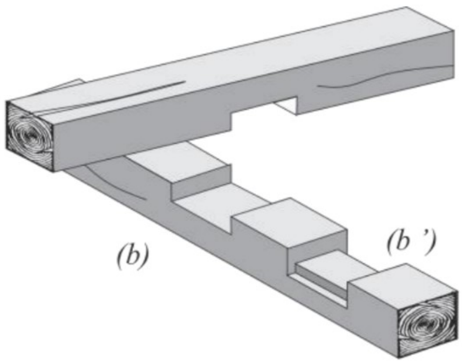

Figure 157. Half-lap joint (Harte & Dietsch, 2015)

Step Joint

Step joints or notched joints are used in timber trusses, transmitting compressive forces of inclined struts to the chords. The joints transfer compressive forces via contact to the front surface of the joint, followed by shear stress in the loaded end to the chord (Blaß & Sandhaas, 2017). Step joints can transfer relatively high compressive forces, however this may require use of a hardwood block, spreading the load, to prevent crushing of the timber perpendicular to the grain (Herzog et al., 2004). Nails, bolts or screws are used to secure the members in their position. Three different types of step joints can be distinguished: single step – notch in front (see Figure 158), notch in heel and double step. Differences between them is the location the forces are transmitted to the chord. A newer multiple step joints has been developed, which is essentially an

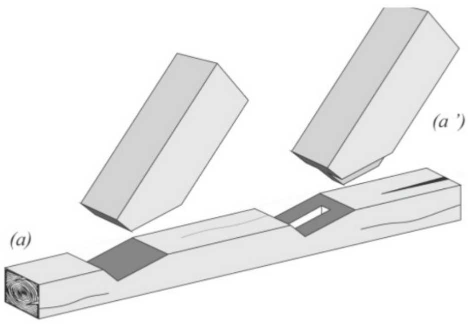

Figure 158. Step joint (Harte & Dietsch, 2015)

improved double step joint, but requiring only a third of the notch depth (Blaß & Sandhaas, 2017).

Mortise and Tenon Joint

A mortise and tenon joint consist out of a hole in one element, the mortise, into which a tongue of another element is slot, the tenon, see Figure 159. As material is removed, the cross-section is weakened. These joints are used to connect individual members in floor, wall and roof constructions with angles between 45º and 90º and transfers shear forces. For the strongest result, the tenon should fit snug into the mortise. It is also possible to fasten the joint using dowels. (Blaß & Sandhaas, 2017; Harte & Dietsch, 2015)

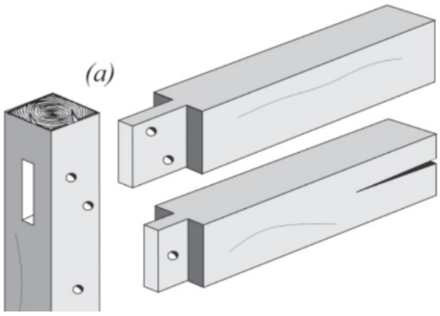

Figure 159. Through pinned mortise and tenon joint (Harte & Dietsch, 2015)

C.1.2 Mechanical Joints

Dowel-type Fasteners

Nails & staples

Nails are the most frequently applied type of fasteners in timber construction and used in for example horizontal and vertical diaphragms and trusses. They come in various shapes and sizes and depending on the type of nail and wood, predrilling of holes might be required to prevent wood from splitting. Nails are used in single-shear joints between timber, steel and wood-based materials. They should not be axially loaded as this can result in withdrawal of the nail or pull-through of the nail head. Ringed shank nails can resist more axial loads than smooth shanked nails, but it is still limited. Nails generally exhibit ductile behaviour, which facilitates a balanced load distribution between nails grouped together. (Blaß & Sandhaas, 2017)

Staples are very similar types of fasteners to nails and have very similar load-bearing behaviour. The steel grade used in staples however is much higher and the angle between staple crown and grain direction of the wood should be monitored. Staples can be assumed as two equivalent nails, but the angle between the staple crown and grain direction should be at least 30º. They are most frequently applied in timber frame buildings as they are very fast to install using special fastener tools (Blaß & Sandhaas, 2017).

Bolts, dowels and threaded rods

Bolts are made of steel and include a head and a nut. They are placed in predrilled holes 1 mm larger than the bolt diameter, resulting in a less stiff connection. Dowels, like bolts consist out of a steel rod, but lack the head and bolt and do not require a wider predrilled hole. As a result, they are often preferred because of their visual appearance and more stiff connection. (Blaß & Sandhaas, 2017)

Dowelled joints are economic, easy to set up and excel in transferring large forces. To ensure that the different members in a joint stay together, at least one bolt is required in a dowelled connection. As a result of the shape of the dowel, dowelled connections cannot be exposed to axial loads. Bolted connections, as a result of their bolts and washers, can take up a limited axial force. Bolted and dowelled connections can be used in both timber-to-timber joints, timber and wood-based materials and steel-to-timber joints (Blaß & Sandhaas, 2017; Herzog et al., 2004).

Screws

Screws are self-tapping fasteners, which come in many variations as they are the fastest evolving type of fasteners. They can be subjected to both axial and lateral loads. As a result of the thread, large forces can be transmitted in axial direction. In comparison with dowels and nails of the same nominal diameter however, the allowed lateral loads on screws are smaller, because of the reduced diameter of threaded part. Screwed joints usually have one shear plane as they are ideal for connections between timber and steel plates, steel members or wood-based materials. They can also be used in timber-to-timber joints. Screws are also used to reinforce other type of joints. Depending on the type and diameter of the screw, predrilling might or might not be required. (Blaß & Sandhaas, 2017)

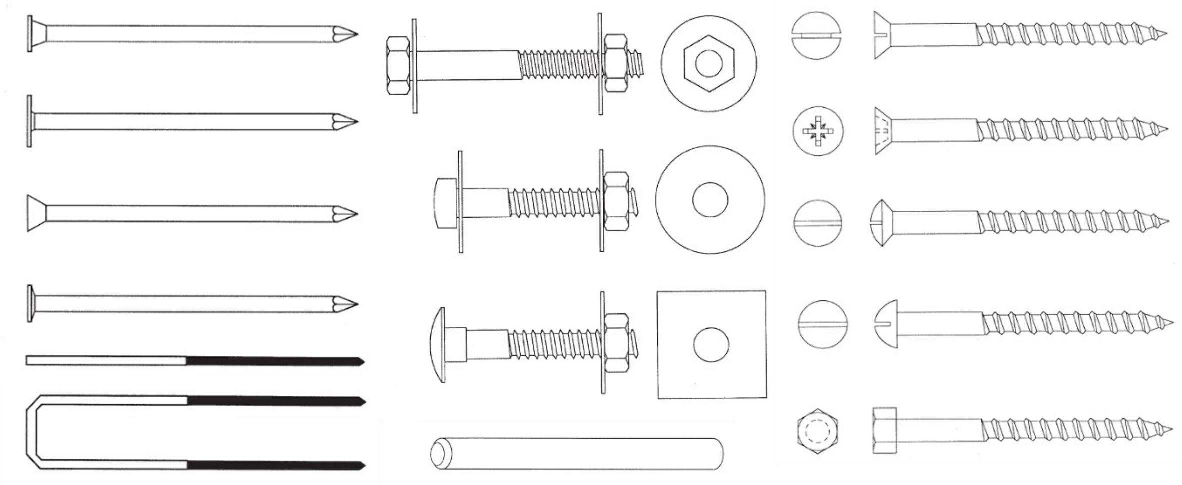

Figure 160. Left: Nails & staples. Middle: Bolts & dowel. Right: Screws. (Herzog et al., 2004)

Joints with multiple shear planes

Dowel-type fasteners are often used in joints in combination with slotted-in steel plates. Slits are sawn into the timber members into which the steel plates are fitted, after which nails, bolts or dowels are pushed through predrilled holes in the timber and steel, see Figure 161. Such joints are categorized as joints with multiple shear planes. The joints are high-performance joints, which can transfer high forces and with correct layout also bending moments. They are often used in heavy trusses and other high-performance joints. (Herzog et al., 2004)

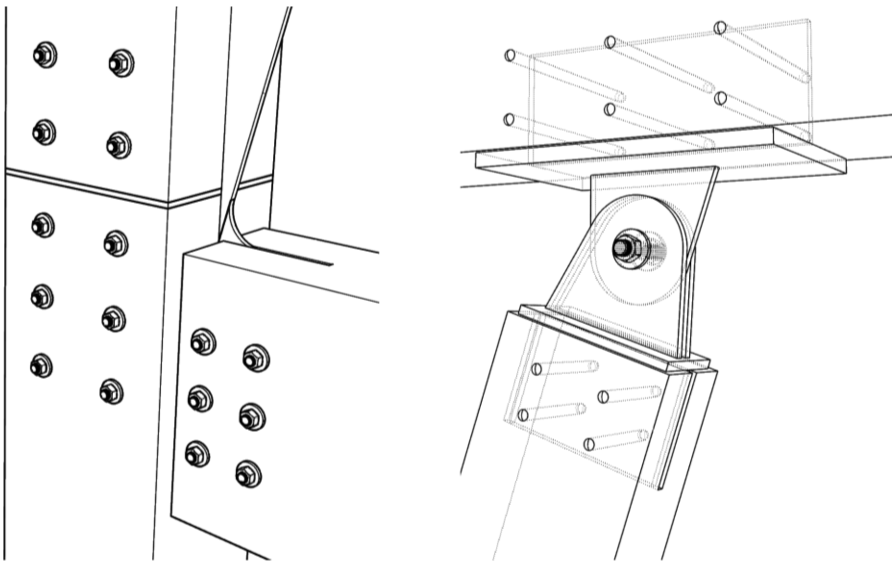

Figure 161. Joints with slotted in steel plates

Cold-formed steel connectors

Traditional carpentry joints have mainly been replaced by cold-formed steel connectors, see Figure 162. Such connectors come in all kind of forms and sizes (plates, straps, angles, brackets, anchors, hangers, etc.) and are connected to both members by nails or screws (Blaß & Sandhaas, 2017). The connectors can be used in timber-to-timber connections as well as timber-to-steel or timber-to-masonry/concrete and are fast and easy to assemble (Herzog et al., 2004).

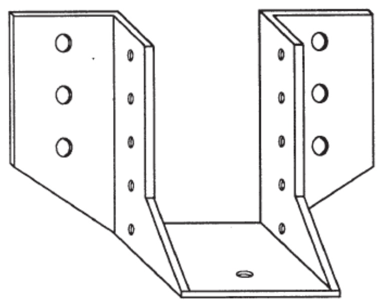

Figure 162. Cold-formed steel joists hangar (Herzog et al., 204).

Surface-type Fasteners

Connectors

Connectors increase the load-bearing area of a connection, therefore increasing the load that can be accommodated. As a result, instead of using multiple dowel-type fasteners, only a single connector might be required creating a perfect hinged joint. Connectors are secured by fasteners like bolts or screws, pressing the timber members together. These bolts or screws absorb the moments generated in the joints, which press the timber members apart, by tensile forces. Joints using connectors can be prefabricated requiring only the installation of bolts on the construction site, making for a very fast erection method. They can just as easily be dismantled. Typical types of connectors are split ring, shear plates and toothed plates. (Blaß & Sandhaas, 2017)

For split ring and shear plate connectors, see Figure 163 left and middle, depressions are milled around the hole for the bold. The connector is placed in this depression and the timber members are connected to each other and secured by a bolt and nut. It is also possible to use screws instead of bolts. (Blaß & Sandhaas, 2017)

Toothed plates differ with split rings and shear plates as they are simply pressed into the timber members and don't require depressions to be milled, see Figure 163 right. A hydraulic press or special high-strength bolt is used for the pressing-in process after which the final bolt or screw is placed. To prevent a gap between the members as a result of the thickness of the base plate of the toothed plate, a recess of the size and thickness of the base plate can be milled. As these plates have to be pressed into the wood, a maximum density of the wood of 500 kg/m^3 is allowed for them to be used. (Blaß & Sandhaas, 2017)

Connector type fasteners transfer shear forces in a joint. Double-sided connectors are used for timber-to-timber joints and single-sided connectors are used for both timber-to-timber and steel-to-timber joints. With double-sided connectors, the force from a member is transferred to the connector via embedment stresses, followed by shear resistance of the connector to the other member. In case of one-sided connectors, the force is again transferred by embedment stresses to the connector, but is then transferred to the bolt by embedment stress between the connector and bolt. The bolt transfers the force via shear resistance to the connector of the other member or directly to the steel plate. With these single-sided connectors, a tolerance of the hole diameter is present, resulting in some slip in the connection.

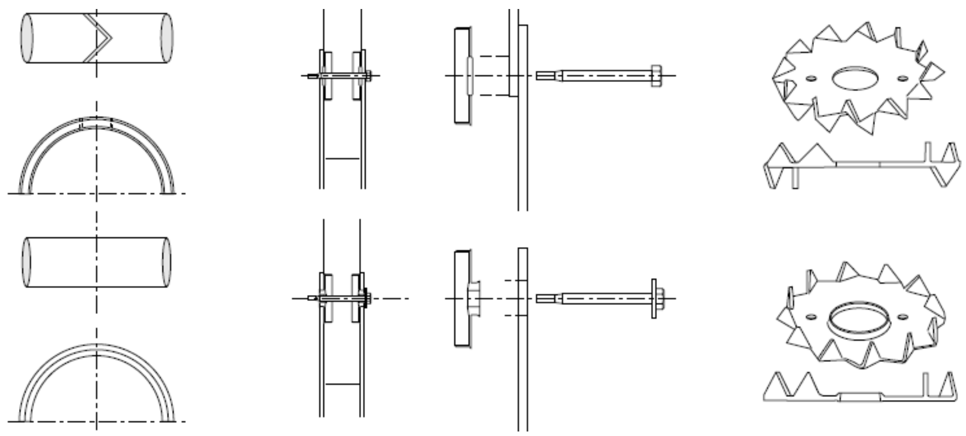

Figure 163. Left: double-sided split ring connector, middle: single-sided shear plate connector, right: double- and single-sided toothed-plate connector (Blaß & Sandhaas, 2017).

Punched metal plate fasteners

Joints between two pieces of wood in the same plane and with the same thickness can be made using punched metal plate fasteners, see Figure 164. These are metal plates with teeth punched in one direction and sticking out of the plate at right angles, which are attached to both sides of the members of a joint. The result is a connection, which for the size of the joint faces can transfer a significantly larger force compared with conventional nailed connections (Herzog et al., 2004). Punched metal plate connections can transfer shear and normal forces and bending moments in the plane of the connection. Forces are transferred from the timber member tot the nails and then to the steel plate. In the same way, these forces are then transferred to the connecting member. They are mainly applied in lightweight timber trusses and in other plane timber structures. For different purposes, different arrangements and shapes of punched metal plates have been developed (Blaß & Sandhaas, 2017).

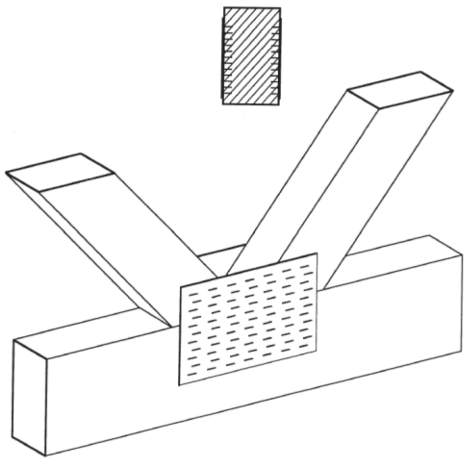

Figure 164. Punched metal plate joint (Herzog et al., 2004)

C.2 Comparison of Timber Joints

The following tables present the motivation for the comparison of the different types of timber joints.

C.2.1 Glued Joints

	Glued joints
Stiffness	Glued joints can be considered as moment-resisting joints. The monolithic joint is very stiff.
Loadbearing capacity	The homogeneous properties of glued finger joints result in a high load bearing capacity.
Types of forces in plane	The monolithic joint allows for bending moments, normal forces and shear forces to be transferred.
Ease of assembly	The gluing process is a specialised process requiring skilled workers and machinery. It is a labour intensive type of joint.
Dismantlability for reuse	The gluing results in a permanent joint, which cannot be undone.
Fire resistance	Glued timber finger joints lack the use of steel and therefore, fire safety is a result of the properties of the wood, which can be deemed good. For glued-in rods, the steel is surrounded by wood, protecting the steel elements from fire.
Costs	The labour intensive production process is expensive.

C.2.2 Carpentry Joints

	Scarf joints
Stiffness	The geometry of a scarf joint in general makes for a hinged joint.
Loadbearing capacity	As material is removed from both members, the cross-section is weakened considerably, limiting the loadbearing capacity of the joint.
Types of forces in plane	Carpentry joints rely on transfer of loads via contact and friction. Scarf joints are mainly suited to transfer shear forces.
Ease of assembly	Assembly of carpentry joints is easy as the members slot into each other. Sometimes an additional steel pin or screw is required.
Dismantlability for reuse	Disassembly of carpentry joints is easy as they are not fixed to each other at all or just by a simple steel pin or screw, which can be removed.
Fire resistance	As these connections rely on contact and friction and not on steel connectors, they have good fire resistant properties.
Costs	The removal of material of member has to be done very precisely to make for a perfect fit between the members. This requires special CNC machinery adding to the costs of these connections.

	Lap joints
Stiffness	The geometry of a lap joint in general makes for a hinged joint.
Loadbearing capacity	As material is removed from both members, the cross-section is weakened considerably, limiting the loadbearing capacity of the joint.
Types of forces in plane	Carpentry joints rely on transfer of loads via contact and friction. Lap joints can mainly transfer shear forces, but adaptation of the geometry like a dovetail crossed lap can result in a connection that is also able to transfer normal forces.
Ease of assembly	Assembly of carpentry joints is easy as the members slot into each other. Sometimes an additional steel pin or screw is required.
Dismantlability for reuse	Disassembly of carpentry joints is easy as they are not fixed to each other at all or just by a simple steel pin or screw, which can be removed.
Fire resistance	As these connections rely on contact and friction and not on steel connectors, they have good fire resistant properties.
Costs	The removal of material of member has to be done very precisely to make for a perfect fit between the members. This requires special CNC machinery adding to the costs of these connections.

	Step joints
Stiffness	The geometry of a step joint in general makes for a hinged joint.
Loadbearing capacity	Step joints can transfer relatively high compressive forces, however this may require use of a hardwood block, spreading the load, to prevent crushing of the timber perpendicular to the grain.
Types of forces in plane	Carpentry joints rely on transfer of loads via contact and friction. The main function of step joints is to transfer normal forces in chords of trusses.
Ease of assembly	Assembly of carpentry joints is easy as the members slot into each other. Sometimes an additional steel pin or screw is required.
Dismantlability for reuse	Disassembly of carpentry joints is easy as they are not fixed to each other at all or just by a simple steel pin or screw, which can be removed.
Fire resistance	As these connections rely on contact and friction and not on steel connectors, they have good fire resistant properties.
Costs	The removal of material of member has to be done very precisely to make for a perfect fit between the members. This requires special CNC machinery adding to the costs of these connections.

	Mortise and tenon joints
Stiffness	Mortise and tenon joints are semi-rigid, but often have significant moment-resisting capacity.
Loadbearing capacity	As material is removed from the members, the cross-section is weakened, limiting the loadbearing capacity of the joint.
Types of forces in plane	Carpentry joints rely on transfer of loads via contact and friction. As the mortise and tenon joints snugly fit into each other, a clamping effect is formed allowing for the transfer of bending moments, normal forces and shear forces.
Ease of assembly	Assembly of carpentry joints is easy as the members slot into each other. Sometimes an additional steel pin or screw is required.
Dismantlability for reuse	Disassembly of carpentry joints is easy as they are not fixed to each other at all or just by a simple steel pin or screw, which can be removed.
Fire resistance	As these connections rely on contact and friction and not on steel connectors, they have good fire resistant properties.
Costs	The removal of material of member has to be done very precisely to make for a perfect fit between the members. This requires special CNC machinery adding to the costs of these connections.

C.2.3 Mechanical Joints

Dowel-type Fasteners

	Nails & staples
Stiffness	Nails and staples exhibit ductile behaviour, facilitating a balanced load distribution between groups of nails or staples. It results in a semi-rigid connection.
Loadbearing capacity	Nails or staples grouped together can transfer medium level forces. It is however important to note they cannot transfer forces axially as this can result in withdrawal of the nail or staple.
Types of forces in plane	Groups of nails and staples are most suited to transfer normal and shear forces.
Ease of assembly	Nails and staples can easily be installed with simple machinery.
Dismantlability for reuse	Nails and staples can be removed, however as they have flat heads, they can be difficult to grasp. When painted over, it can also be difficult to locate them.
Fire resistance	The largest part of the nails or staples is surrounded by wood, protecting them from fire. The heads however are vulnerable to fire and might require protection.
Costs	Applying nails and staples in connections is easy and cheap.

	Bolts, dowels & threaded rods
Stiffness	Bolt or dowel groups result in semi-rigid connections. As dowels do not require wider predrilled holes, they can result in more rigid connections.
Loadbearing capacity	Dowelled connections excel in transferring large forces.
Types of forces in plane	Groups of bolts or dowels can transfer bending moments, normal forces and shear forces.
Ease of assembly	Bolts and dowels are fast and easy to install.
Dismantlability for reuse	Bolts and dowels remain visible and are easy to remove.
Fire resistance	The largest part of the bolts or dowels is surrounded by wood, protecting them from fire. The ends however are vulnerable to fire and might require protection.
Costs	Dowelled joints are economic.

	Screws
Stiffness	Screws exhibit ductile behaviour, facilitating a balanced load distribution between groups of nails or staples. It results in a semi-rigid connection.
Loadbearing capacity	Compared with dowels and nails with the same diameter, screws can transfer smaller forces. As a result of their threads, screws can also transfer axial forces.
Types of forces in plane	Groups of screws are most suited to transfer normal and shear forces.
Ease of assembly	Screws can easily be installed with simple machinery.
Dismantlability for reuse	Screws are easy to remove. Painting however can hide them from sight.
Fire resistance	The largest part of the screws is surrounded by wood, protecting them from fire. The heads however are vulnerable to fire and might require protection.
Costs	Applying screws in connections is easy and cheap.

	Combination with steel plates or connectors
Stiffness	The combination of steel plates with groups of bolts or dowels can result in a semi-rigid connection, but much towards rigid.
Loadbearing capacity	Dowelled connections in combination with steel plates excel in transferring large forces.
Types of forces in plane	Groups of bolts or dowels in combination with steel plates can transfer bending moments, normal forces and shear forces.
Ease of assembly	Bolts and dowels are fast and easy to install.
Dismantlability for reuse	Bolts and dowels remain visible and are easy to remove.
Fire resistance	The steel plates can be exposed to fire conditions. It is also possible to embed the plates into the wood, which can protect the plates from fire conditions.
Costs	The combination of dowelled connections with steel plates can result in economic joints, which can transfer high forces.

Surface-type Fasteners

	Connectors
Stiffness	Connectors with bolt or dowel groups result in semi-rigid connections. The connectors improve the stiffness of the plane between both members.
Loadbearing capacity	The connectors increase the load-bearing area of a connection, increasing the load that can be accommodated.
Types of forces in plane	Groups of connectors with bolts or dowels can transfer bending moments, normal forces and shear forces.
Ease of assembly	Installing connectors might require some additional steps to make recesses for the connectors to fit in. Afterwards, installation is very easy.
Dismantlability for reuse	Bolts and dowels remain visible and are easy to remove.
Fire resistance	The connectors are well protected from fire conditions. The ends of bolts or dowels however might require protection.
Costs	The installation of connectors is easy and cheap.

	Punched metal plate fasteners
Stiffness	Due to the high number of metal pins, the joint can transfer significantly higher loads compared with traditional nailed connections.
Loadbearing capacity	The punched metal plates can transfer medium forces.
Types of forces in plane	The punched metal plates result in a semi-rigid connection, which can transfer bending moments, normal forces and shear forces.
Ease of assembly	The punched metal plates can be easily punched into the timber members.
Dismantlability for reuse	The punched metal plates can simply be peeled of the joints.
Fire resistance	The metal plates are exposed to fire conditions.
Costs	Installation is easy and cheap.

D.

Calculations Technosoft

D.1 Cone Penetration Tests

Sweco Nederland BV Blad: 1

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc01_CPT000000052457

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
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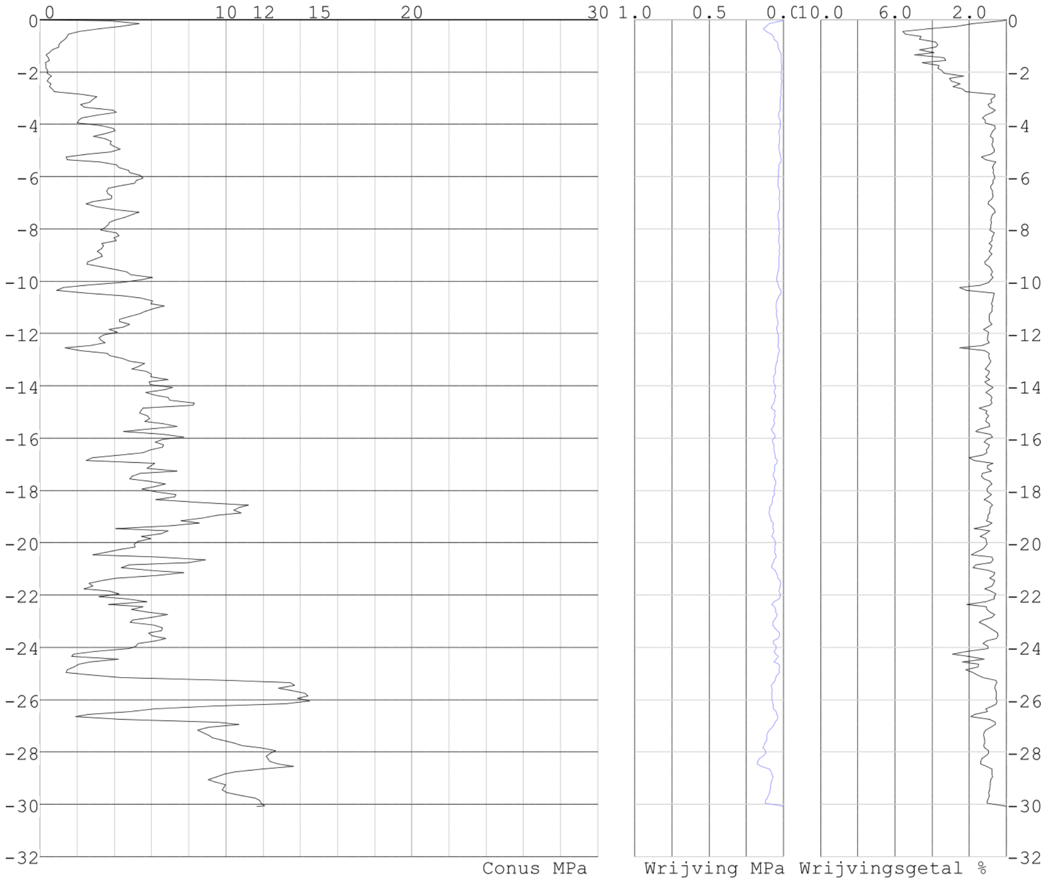


Sweco Nederland BV Blad: 2

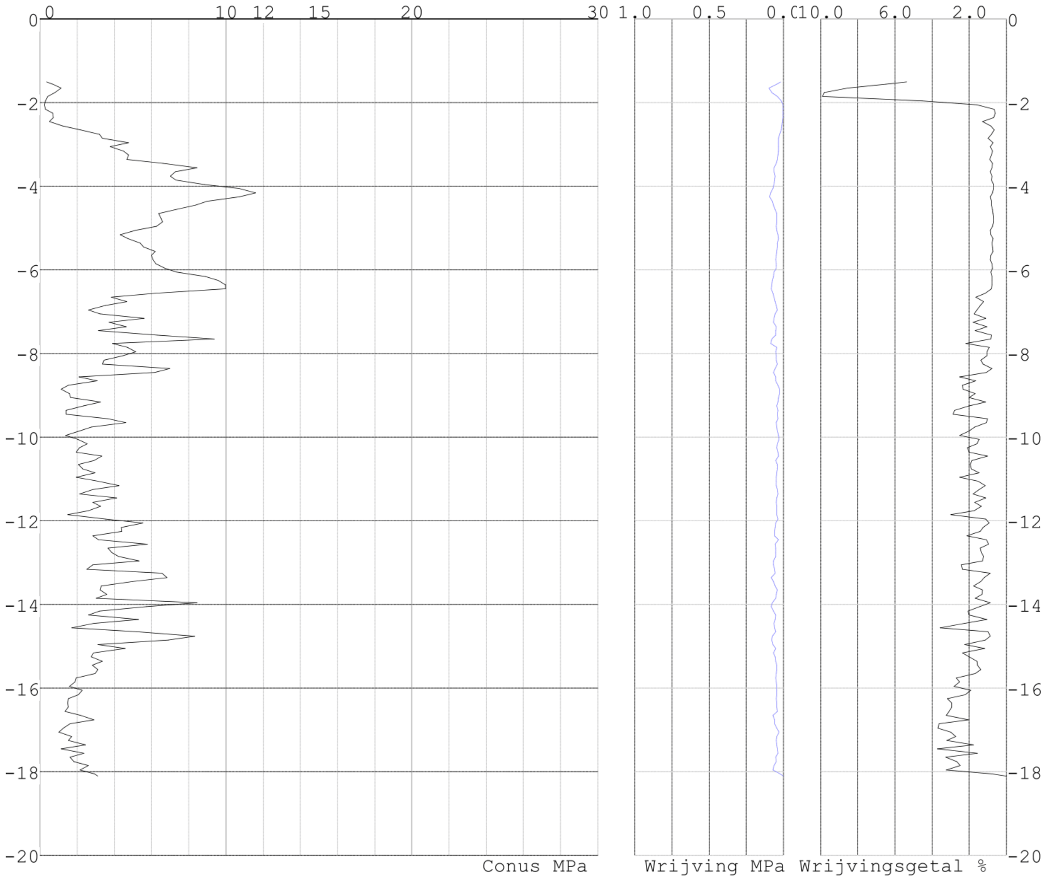
Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc03_CPT000000074906

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc03_CPT000000074906



Sweco Nederland BV Blad: 3

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc04_CPT000000055844

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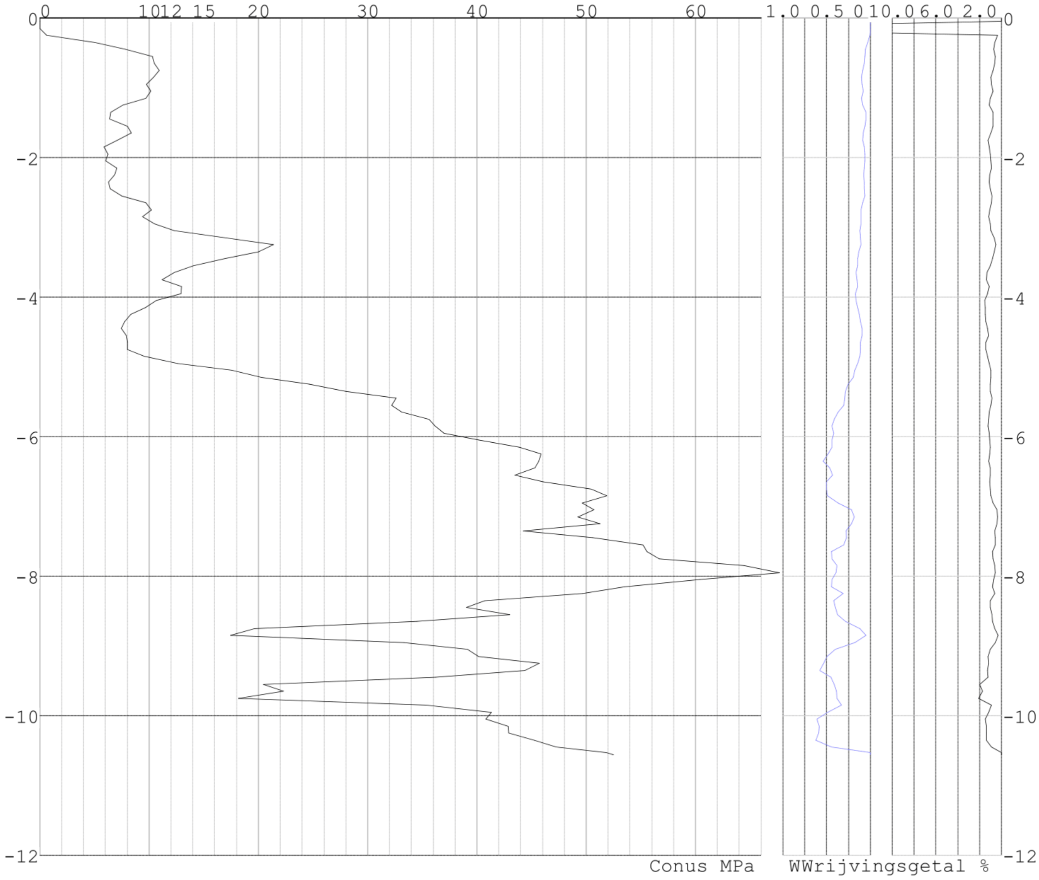


Sweco Nederland BV Blad: 4

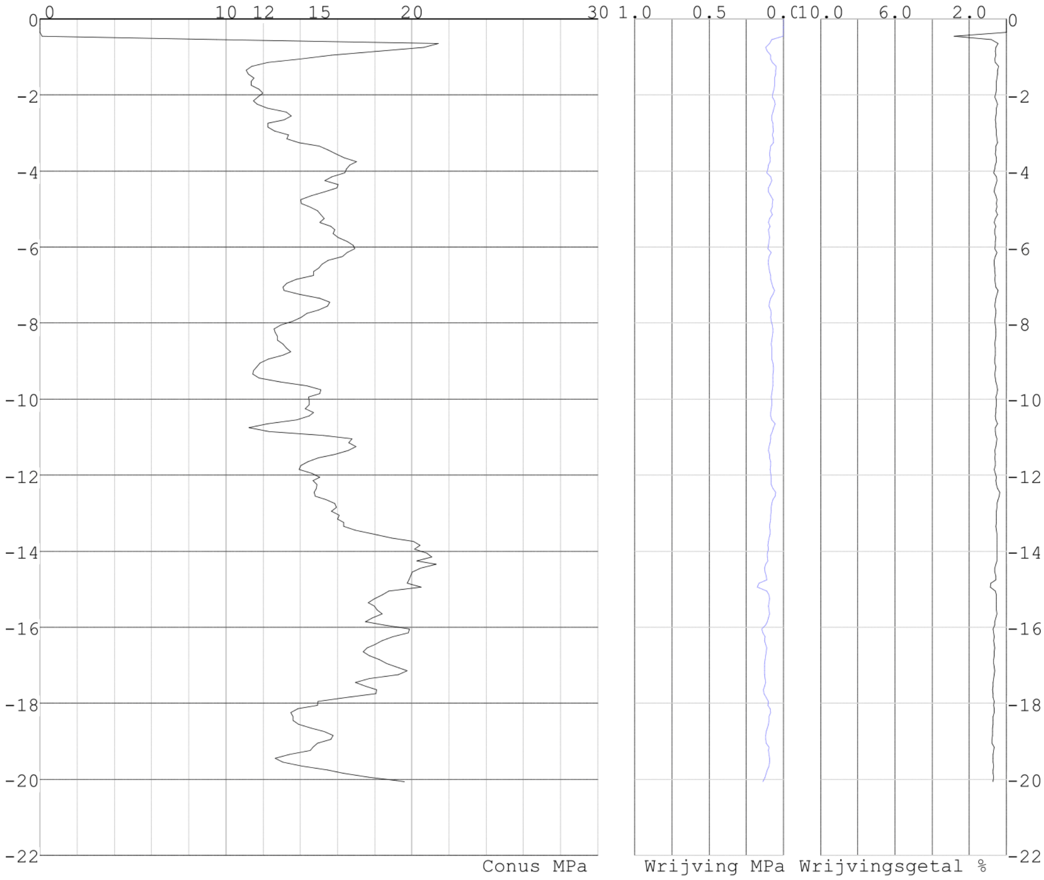
Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc06_CPT000000170226

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc06_CPT000000170226



Sweco Nederland BV Blad: 5

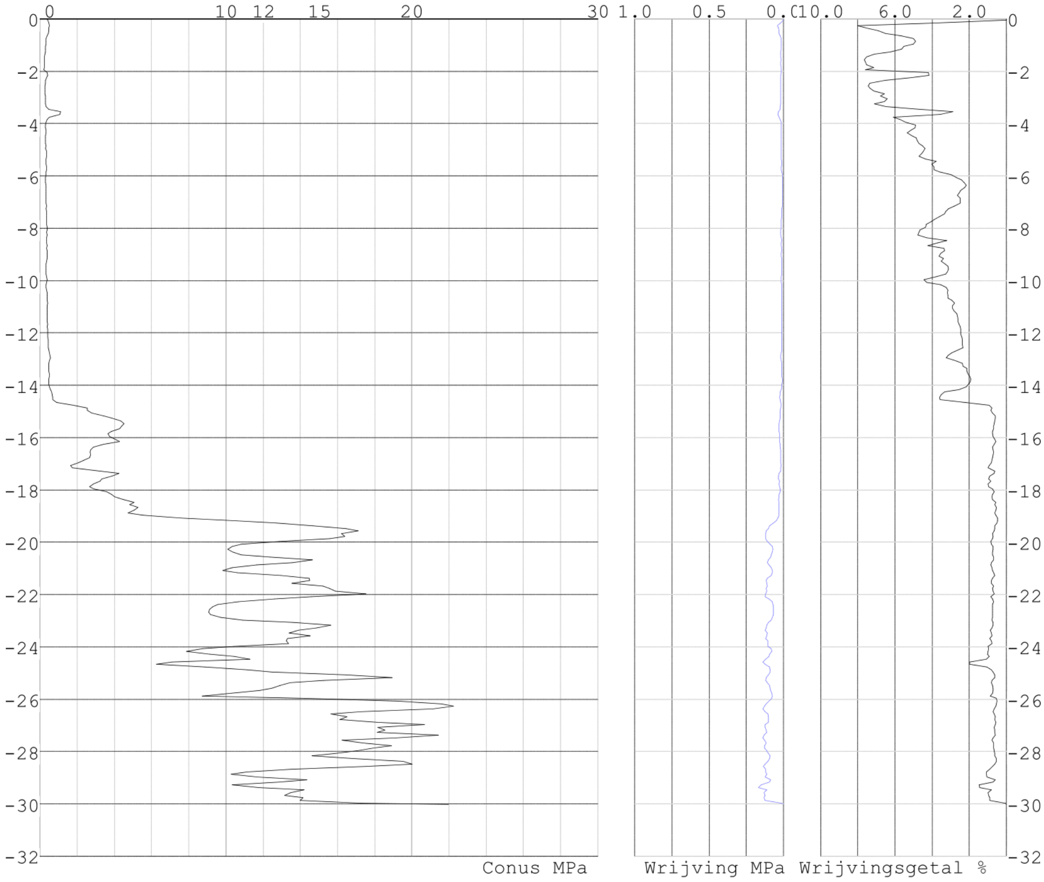
Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc08_CPT000000062979

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc08_CPT000000062979



Sweco Nederland BV Blad: 6

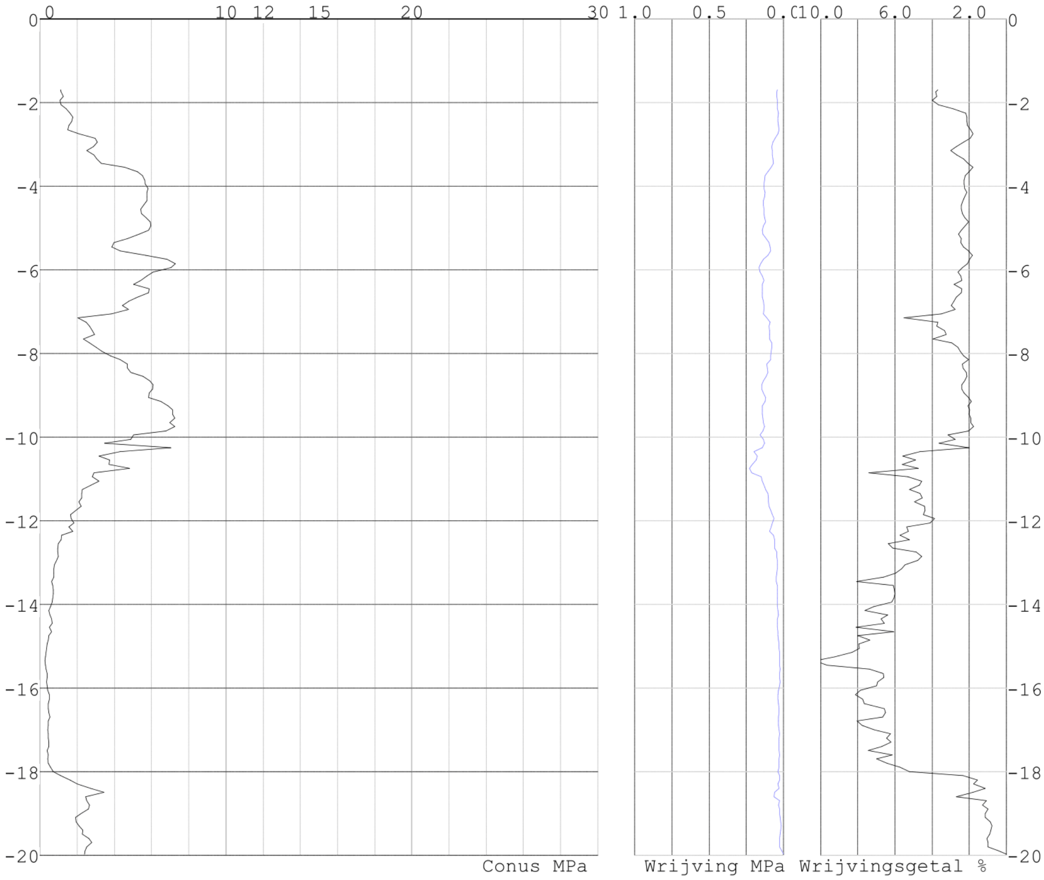
Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc12_CPT000000077437

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc12_CPT000000077437



Sweco Nederland BV Blad: 7

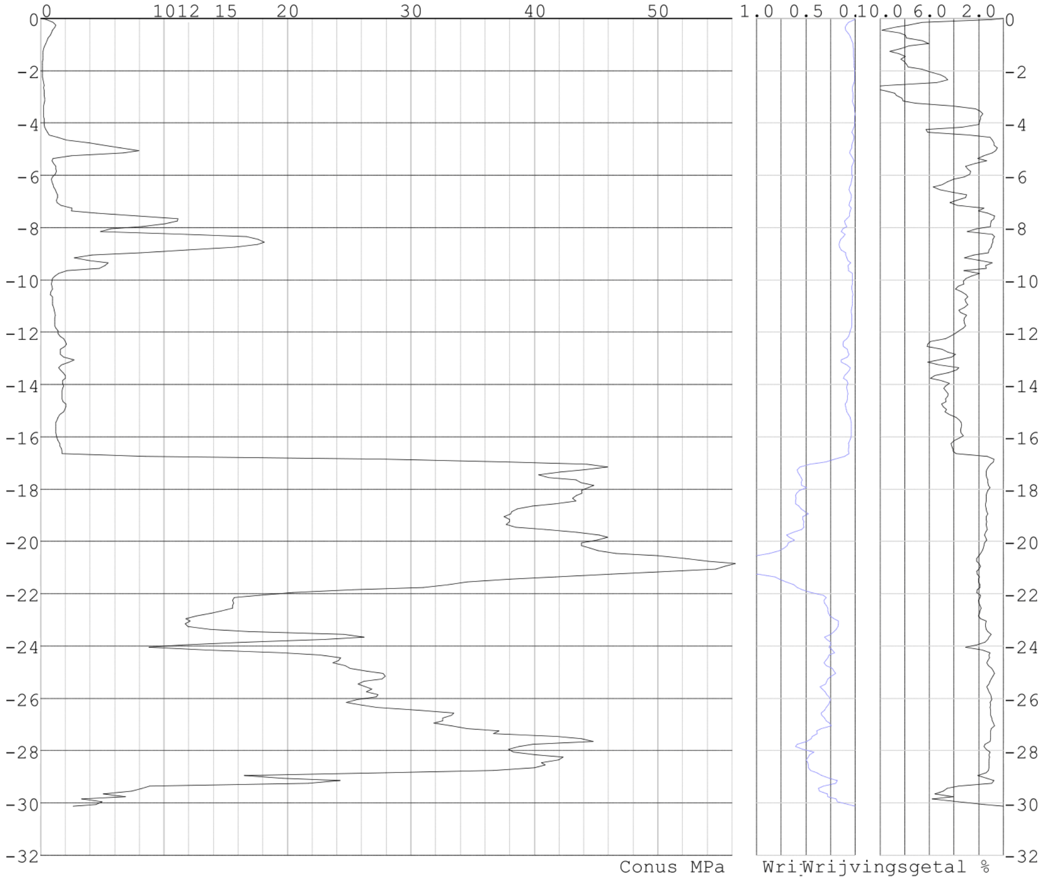
Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc02_CPT000000003023

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc02_CPT000000003023



Sweco Nederland BV Blad: 8

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc05_CPT000000161652

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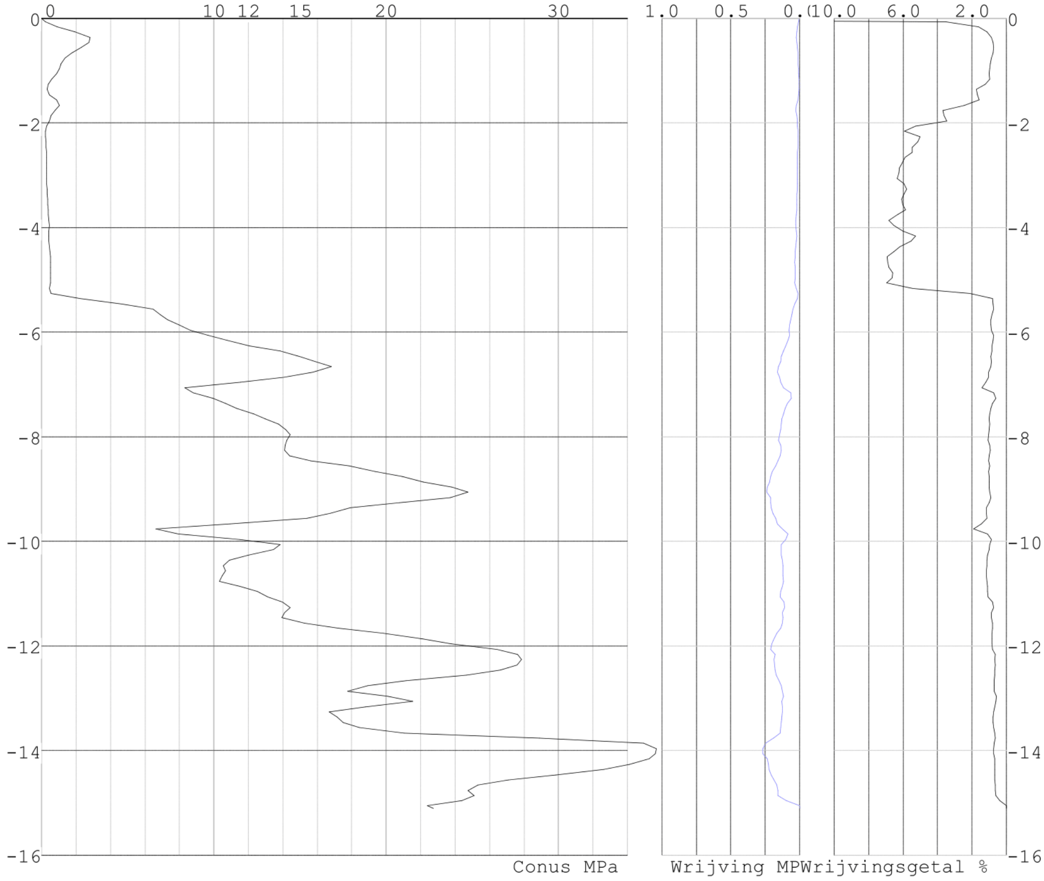
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc05_CPT000000161652



Sweco Nederland BV Blad: 9

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc07CPT000000004530

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

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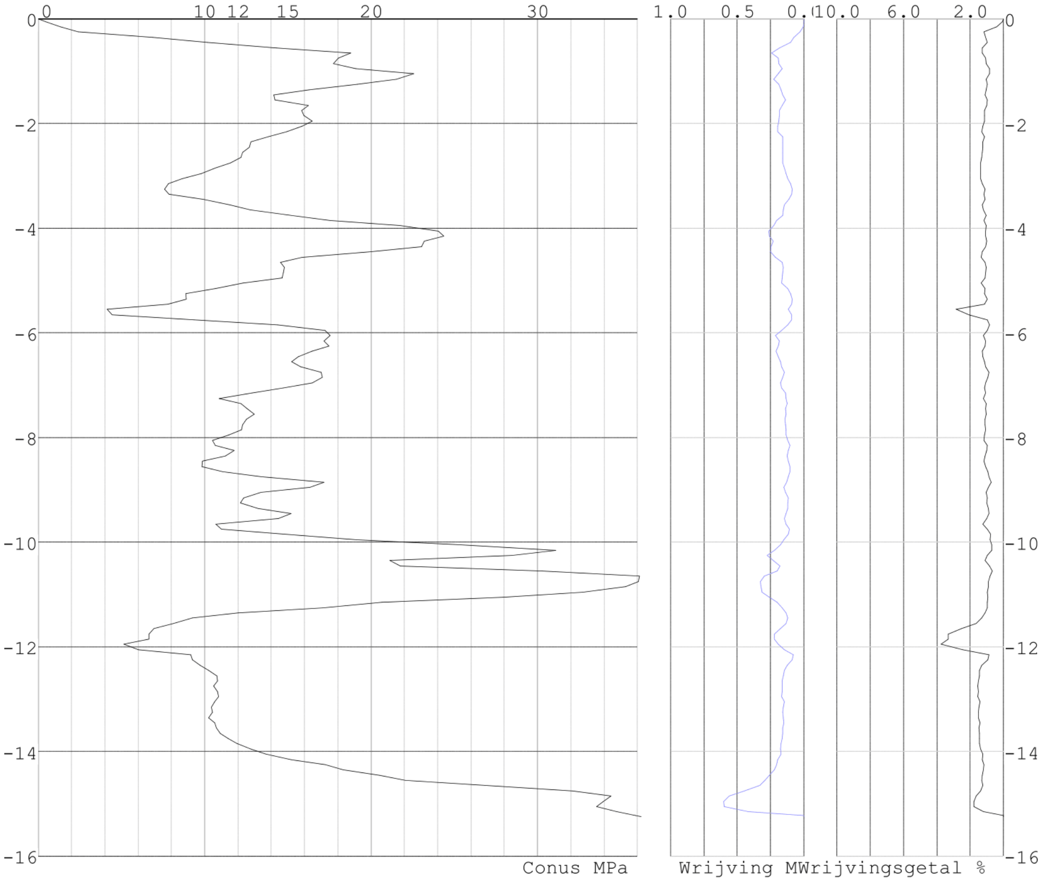


Sweco Nederland BV Blad: 10

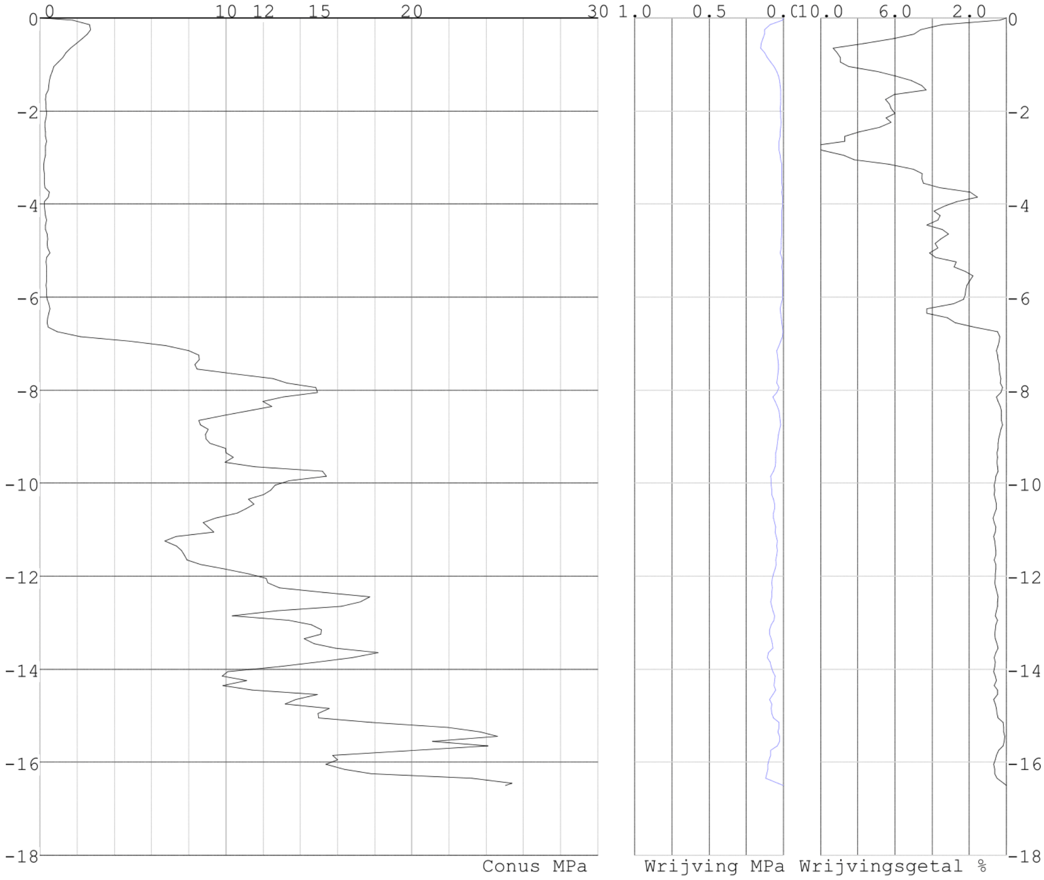
Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc09_CPT000000011161

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc09_CPT000000011161



Sweco Nederland BV Blad: 11

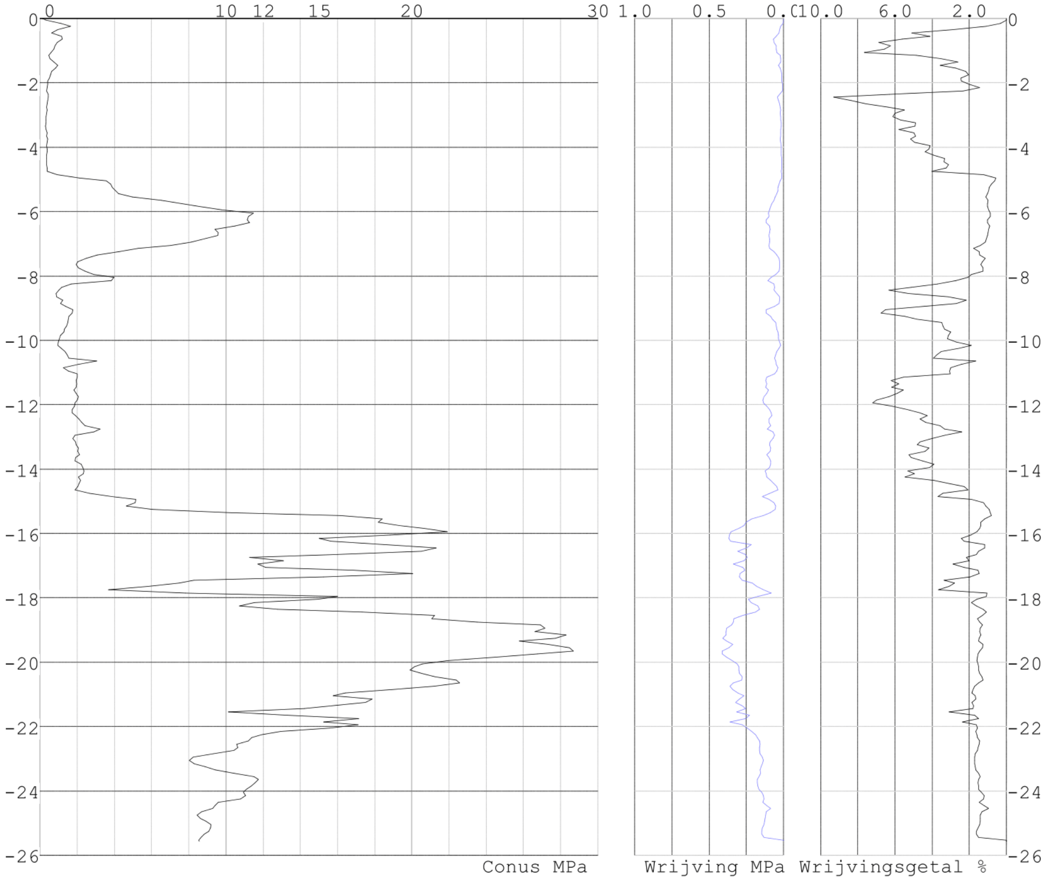
Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc10_CPT000000004377

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
Hoogte maaiveld [m] : 0.00

SONDERINGSGEGEVENS GRAFIEK: Loc10_CPT000000004377



Sweco Nederland BV Blad: 12

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

SONDERINGSGEGEVENS ALGEMEEN: Loc11_CPT000000012135

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld
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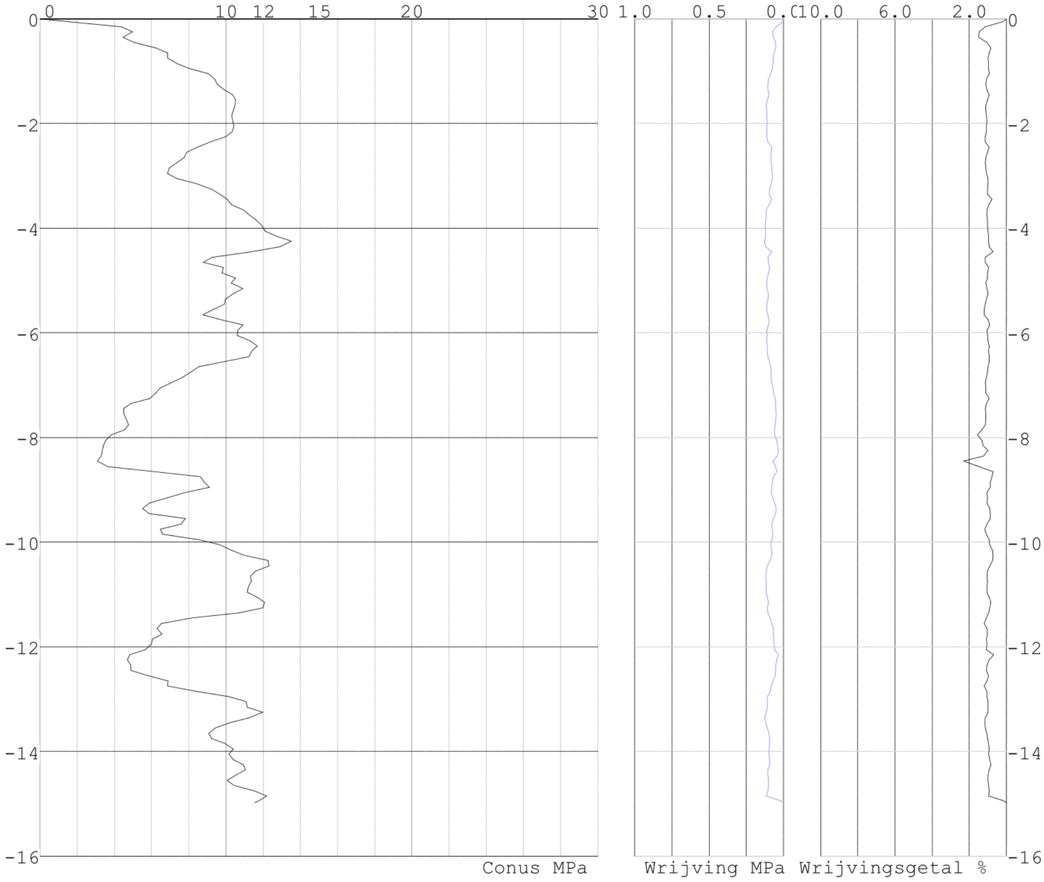


D.2 Output Technosoft Shallow Foundation

Sweco Nederland BV Blad: 1

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

ALGEMENE GEGEVENS

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing
Eenheden : [kN][m][MPa][graden] tenzij anders vermeld
Datum : 05-10-2022
Referentieniveau (RN) : Maaiveld
Referentieperiode : 50 jaar
Bestand : C:\Users\nlpabu\OneDrive - Sweco AB\Documenten\Afstuderen\Technosoft\Draagcapaciteit Fundering op Staal.fsw

Toegepaste normen volgens Eurocode met Nederlandse NB

Beton	NEN-EN 1992-1-1:2011(nl)	C2/A1:2015(nl)	NB:2016(nl)
Geotechniek	EN 1997-1:2004	AC:2009	
	NEN-EN 1997-1:2005	C1+A1:2013	NB:2016
	NEN 9997-1:2016	C2:2017	

PROFIELGEGEVENS Isolated Footing 2000x2000x200

Type : Poer

			Links	Rechts
Breedte	min	[mm]	1000	1000
	max	[mm]	1000	1000
	stap	[mm]	100	100
Lengte	voor	[mm]	1000	
	achter	[mm]	1000	De poer is vierkant.
Hoogte		[mm]	200	
Opstorting	breedte	[mm]	300	
	lengte	[mm]	300	

BELASTINGGEGEVENS Load 350

Permanent

Nr.	Omschrijving	Type	Richting	Waarde [kN,m]	AfstandX [m]	AfstandY [m]	AfstandZ [m]
1	Self-Weight	F/q	Z	-350.00	0.00	0.00	-

Extra lasten t.g.v. eigengewicht poer en opstort staan bij de rekengegevens.

BELASTINGGEGEVENS Load 1000

Permanent

Nr.	Omschrijving	Type	Richting	Waarde [kN,m]	AfstandX [m]	AfstandY [m]	AfstandZ [m]
1	Self-Weight	F/q	Z	-1000.00	0.00	0.00	-

Extra lasten t.g.v. eigengewicht poer en opstort staan bij de rekengegevens.

GRONDSOORTEN

Nr	Naam	γ [kN/m^3]	γ_{sat} [kN/m^3]	φ' [°]	c' [kPa]	c_u [kPa]	$C_c/(1+e_0)$ [-]	C_α [-]	e_0 [-]
1	Zand - Schoon - Los	17.0	19.0	30.0	-	-	0.0115	0.0000	0.83
2	Klei - Schoon - Matig	17.0	17.0	17.5	5.0	50.0	0.1533	0.0061	1.36
3	Zand - Zwak siltig - K..	18.0	20.0	27.0	-	-	0.0051	0.0000	0.65
4	Zand - Schoon - Vast	19.0	21.0	35.0	-	-	0.0023	0.0000	0.50
5	Klei - Schoon - Vast	19.0	19.0	17.5	13.0	100.0	0.0920	0.0037	0.83
6	Veen - Matig voorbelas..	12.0	12.0	15.0	2.5	20.0	0.3067	0.0153	4.90
7	Leem - Zwak zandig - M..	20.0	20.0	27.5	1.0	100.0	0.0511	0.0020	0.65
8	Klei - Sterk zandig	18.0	18.0	27.5	0.0	0.0	0.0920	0.0037	1.06
9	Veen - Niet voorbelast..	10.0	10.0	15.0	1.0	10.0	0.4600	0.0230	15.50
10	Grind - Sterk siltig -..	19.0	21.0	32.5	-	-	0.0038	0.0000	0.50

Sweco Nederland BV Blad: 2

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

GRONDSOORTEN

Nr	Naam	γ [kN/m^3]	γ_{sat} [kN/m^3]	φ' [°]	c' [kPa]	c_u [kPa]	$C_c/(1+e_0)$ [-]	C_α [-]	e_0 [-]
11	Grind - Zwak siltig - ..	19.0	21.0	37.5	-	-	0.0019	0.0000	0.50
12	Klei - Organisch - Matig	15.0	15.0	15.0	0.0	25.0	0.2300	0.0115	2.30
13	Zand - Sterk siltig - ..	18.0	20.0	25.0	-	-	0.0115	0.0000	0.65
14	Grind - Zwak siltig - ..	18.0	20.0	35.0	-	-	0.0023	0.0000	0.65
15	Zand - Schoon - Matig	18.0	20.0	32.5	-	-	0.0038	0.0000	0.65
16	Grind - Zwak siltig - ..	17.0	19.0	32.5	-	-	0.0046	0.0000	0.83
17	Klei - Zwak zandig - S..	15.0	15.0	22.5	0.0	40.0	0.2300	0.0092	2.30
18	Leem - Zwak zandig - V..	21.0	21.0	27.5	2.5	200.0	0.0329	0.0013	0.50
19	Leem - Sterk zandig	19.0	19.0	27.5	0.0	50.0	0.0511	0.0020	0.83
20	Klei - Zwak zandig - M..	18.0	18.0	22.5	5.0	80.0	0.1150	0.0046	1.06
21	Klei - Zwak zandig - V..	20.0	20.0	22.5	13.0	120.0	0.0767	0.0031	0.65
22	Grind - Sterk siltig -..	18.0	20.0	30.0	-	-	0.0058	0.0000	0.65

Sweco Nederland BV Blad: 3

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

```
Project                 : Shallow Foundation for Timber Columns
Onderdeel               : Isolated Footing
```

REKENGEGEVENS Case 1 (d = -0.80m, F=350)

```
Profiel                 : Isolated Footing 2000x2000x200
Belasting               : Load 350
Bodemprofielen          : Loc01_CPT000000052457, Loc03_CPT000000074906
                        : Loc04_CPT000000055844, Loc06_CPT000000170226
                        : Loc08_CPT000000062979, Loc12_CPT000000077437
                        : Loc02_CPT000000003023, Loc05_CPT000000161652
                        : Loc07_CPT000000004530, Loc09_CPT000000011161
                        : Loc10_CPT000000004377, Loc11_CPT000000012135
Wapening                : Wapening 1
Niveau onderkant fnd[m] :   -0.80        Niveau bovenkant [m] :    0.30
Grondwaterniveau    [m] :   -0.20
Opstort                 :    0.80 Zand - Schoon - Los
```

Materiaalfactoren		gunstig	ongunstig
γ_{γ}	gewicht grond :	1.10	1.00
$\gamma_{\varphi'}$	inwendige wrijving:	1.15	
$\gamma_{c'}$	cohesie :	1.60	
γ_{cu}	ongedr. schuifst. :	1.35	
γ_{γ}	gewicht grond BGT :	1.00	

Belastingfactoren	ongunstig	gunstig	Ψ
Permanent :	1.20	0.90	
Variabel :	1.50	0.00	0.40
Grond :		0.90	

Extra belastingen t.g.v. eigengewicht poer en opstort

B-tot [m]	B-li [m]	B-re [m]	L [m]	H [m]	Omschrijving	Type	Rich-ting	Waarde [kN]	AfstX [m]	AfstY [m]	AfstZ [m]
2.00	1.00	1.00	2.00	0.20	E.G poer, plaat	F	Z	20.00	0.00	0.00	-
0.30	0.15	0.15	0.30	0.90	+ opstorting	F	Z	2.02	0.00	0.00	-
2.00	1.00	1.00	2.00	0.60	E.G opstort (droog)	F	Z	40.80	0.00	0.00	-
0.30	0.15	0.15	0.30	0.60	- tpv opstorting	F	Z	-0.92	0.00	0.00	-

REKENGEGEVENS Case 2 (d = -2.00m, F=350)

```
Profiel                 : Isolated Footing 2000x2000x200
Belasting               : Load 350
Bodemprofielen          : Loc01_CPT000000052457, Loc03_CPT000000074906
                        : Loc04_CPT000000055844, Loc06_CPT000000170226
                        : Loc08_CPT000000062979, Loc12_CPT000000077437
                        : Loc02_CPT000000003023, Loc05_CPT000000161652
                        : Loc07_CPT000000004530, Loc09_CPT000000011161
                        : Loc10_CPT000000004377, Loc11_CPT000000012135
Wapening                : Wapening 1
Niveau onderkant fnd[m] :   -2.00        Niveau bovenkant [m] :    0.30
Grondwaterniveau    [m] :   -0.20
Opstort                 :    2.00 Zand - Schoon - Los
```

Materiaalfactoren		gunstig	ongunstig
γ_{γ}	gewicht grond :	1.10	1.00
$\gamma_{\varphi'}$	inwendige wrijving:	1.15	
$\gamma_{c'}$	cohesie :	1.60	
γ_{cu}	ongedr. schuifst. :	1.35	
γ_{γ}	gewicht grond BGT :	1.00	

Belastingfactoren	ongunstig	gunstig	Ψ
Permanent :	1.20	0.90	
Variabel :	1.50	0.00	0.40
Grond :		0.90	

Sweco Nederland BV Blad: 4

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Extra belastingen t.g.v. eigengewicht poer en opstort

B-tot [m]	B-li [m]	B-re [m]	L [m]	H [m]	Omschrijving	Type	Rich-ting	Waarde [kN]	AfstX [m]	AfstY [m]	AfstZ [m]
2.00	1.00	1.00	2.00	0.20	E.G poer, plaat	F	Z	20.00	0.00	0.00	-
0.30	0.15	0.15	0.30	2.10	+ opstorting	F	Z	4.72	0.00	0.00	-
2.00	1.00	1.00	2.00	1.80	E.G opstort (droog)	F	Z	122.40	0.00	0.00	-
0.30	0.15	0.15	0.30	1.80	- tpv opstorting	F	Z	-2.75	0.00	0.00	-

REKENGEGEVENS Case 3 (d = -0.80m, F=1000)

Profiel : Isolated Footing 2000x2000x200
Belasting : Load 1000
Bodemprofielen : Loc01_CPT000000052457, Loc03_CPT000000074906
: Loc04_CPT000000055844, Loc06_CPT000000170226
: Loc08_CPT000000062979, Loc12_CPT000000077437
: Loc02_CPT000000003023, Loc05_CPT000000161652
: Loc07_CPT000000004530, Loc09_CPT000000011161
: Loc10_CPT000000004377, Loc11_CPT000000012135
Wapening : Wapening 1
Niveau onderkant fnd[m] : -0.80 Niveau bovenkant [m] : 0.30
Grondwaterniveau [m] : -0.20
Opstort : 0.80 Zand - Schoon - Los

Materiaalfactoren		gunstig	ongunstig
γ_{γ}	gewicht grond :	1.10	1.00
$\gamma_{\varphi'}$	inwendige wrijving:	1.15	
$\gamma_{c'}$	cohesie :	1.60	
γ_{cu}	ongedr. schuifst. :	1.35	
γ_{γ}	gewicht grond BGT :	1.00	

Belastingfactoren	ongunstig	gunstig	Ψ
Permanent :	1.20	0.90	
Variabel :	1.50	0.00	0.40
Grond :		0.90	

Extra belastingen t.g.v. eigengewicht poer en opstort

B-tot [m]	B-li [m]	B-re [m]	L [m]	H [m]	Omschrijving	Type	Rich-ting	Waarde [kN]	AfstX [m]	AfstY [m]	AfstZ [m]
2.00	1.00	1.00	2.00	0.20	E.G poer, plaat	F	Z	20.00	0.00	0.00	-
0.30	0.15	0.15	0.30	0.90	+ opstorting	F	Z	2.02	0.00	0.00	-
2.00	1.00	1.00	2.00	0.60	E.G opstort (droog)	F	Z	40.80	0.00	0.00	-
0.30	0.15	0.15	0.30	0.60	- tpv opstorting	F	Z	-0.92	0.00	0.00	-

Sweco Nederland BV Blad: 5

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

REKENGEGEVENS Case 4 (d = -2.00m, F=1000)

Profiel : Isolated Footing 2000x2000x200
Belasting : Load 1000
Bodemprofielen : Loc01_CPT000000052457, Loc03_CPT000000074906
: Loc04_CPT000000055844, Loc06_CPT000000170226
: Loc08_CPT000000062979, Loc12_CPT000000077437
: Loc02_CPT000000003023, Loc05_CPT000000161652
: Loc07_CPT000000004530, Loc09_CPT000000011161
: Loc10_CPT000000004377, Loc11_CPT000000012135
Wapening : Wapening 1
Niveau onderkant fnd[m] : -2.00 Niveau bovenkant [m] : 0.30
Grondwaterniveau [m] : -0.20
Opstort : 2.00 Zand - Schoon - Los

Materiaalfactoren		gunstig	ongunstig
γ_{γ}	gewicht grond :	1.10	1.00
$\gamma_{\varphi'}$	inwendige wrijving:	1.15	
$\gamma_{c'}$	cohesie :	1.60	
γ_{cu}	ongedr. schuifst. :	1.35	
γ_{γ}	gewicht grond BGT :	1.00	

Belastingfactoren	ongunstig	gunstig	Ψ
Permanent :	1.20	0.90	
Variabel :	1.50	0.00	0.40
Grond :		0.90	

Extra belastingen t.g.v. eigengewicht poer en opstort

B-tot [m]	B-li [m]	B-re [m]	L [m]	H [m]	Omschrijving	Type	Rich-ting	Waarde [kN]	AfstX [m]	AfstY [m]	AfstZ [m]
2.00	1.00	1.00	2.00	0.20	E.G poer, plaat	F	Z	20.00	0.00	0.00	-
0.30	0.15	0.15	0.30	2.10	+ opstorting	F	Z	4.72	0.00	0.00	-
2.00	1.00	1.00	2.00	1.80	E.G opstort (droog)	F	Z	122.40	0.00	0.00	-
0.30	0.15	0.15	0.30	1.80	- tpv opstorting	F	Z	-2.75	0.00	0.00	-

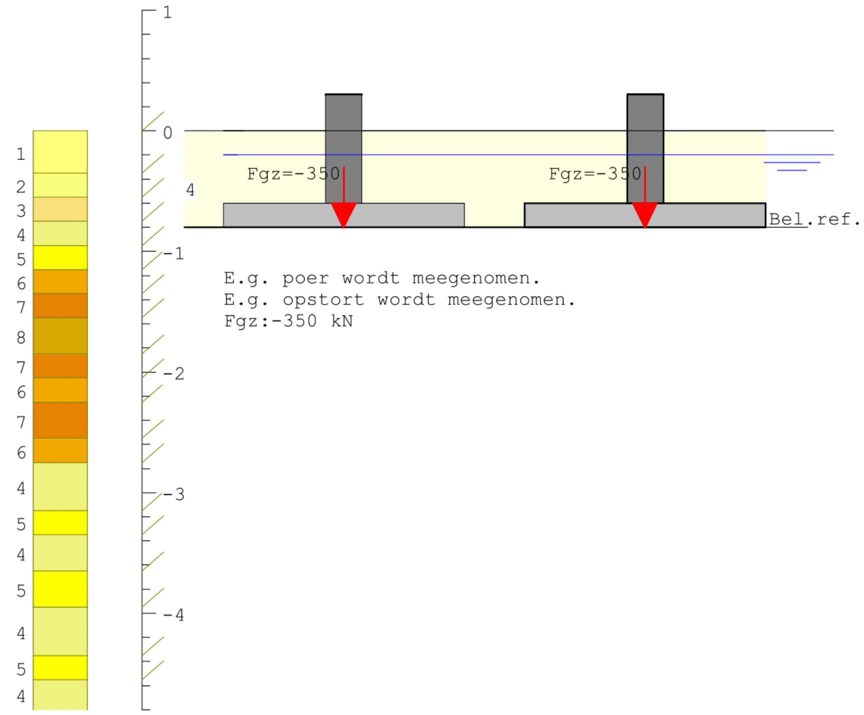
Sweco Nederland BV Blad: 6

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350)

Bodemprofiel: Loc01_CPT000000052457



Legenda

1 : Grind - Zwak siltig - Vast
2 : Zand - Schoon - Matig
3 : Zand - Sterk siltig - Kleiig
4 : Zand - Schoon - Los
5 : Leem - Zwak zandig - Vast
6 : Klei - Zwak zandig - Matig
7 : Klei - Schoon - Matig
8 : Klei - Zwak zandig - Slap
9 : Klei - Schoon - Vast
10 : Klei - Zwak zandig - Vast

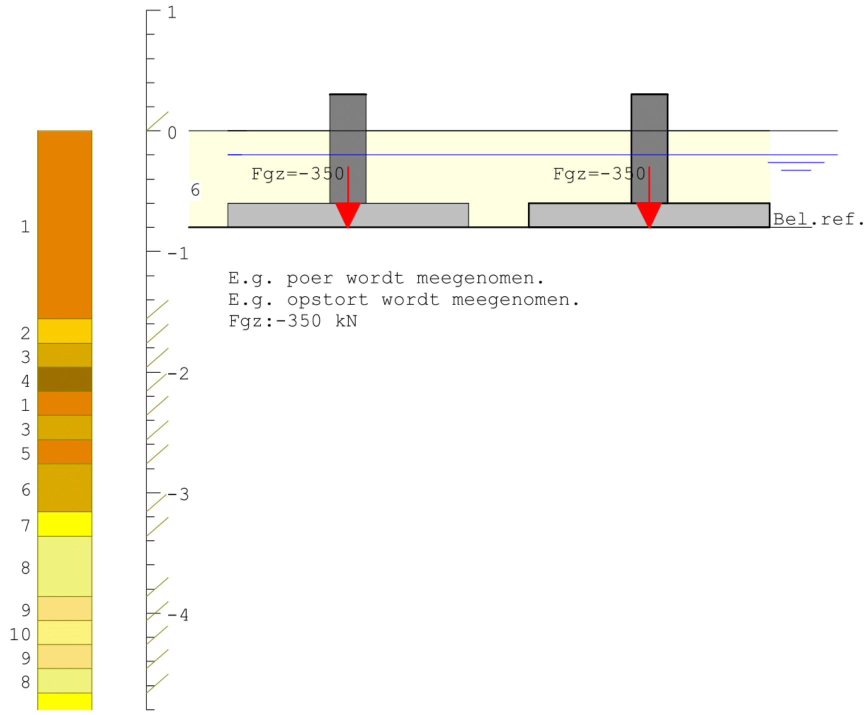
Sweco Nederland BV Blad: 7

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc03_CPT000000074906



Legenda

1 : Klei - Schoon - Matig
2 : Klei - Zwak zandig - Vast
3 : Klei - Zwak zandig - Slap
4 : Veen - Matig voorbelast - Matig
5 : Leem - Zwak zandig - Vast
6 : Zand - Schoon - Los
7 : Zand - Sterk siltig - Kleiig
8 : Grind - Sterk siltig - Los
9 : Zand - Zwak siltig - Kleiig
10 : Zand - Schoon - Matig
11 : Klei - Schoon - Vast
12 : Klei - Zwak zandig - Matig

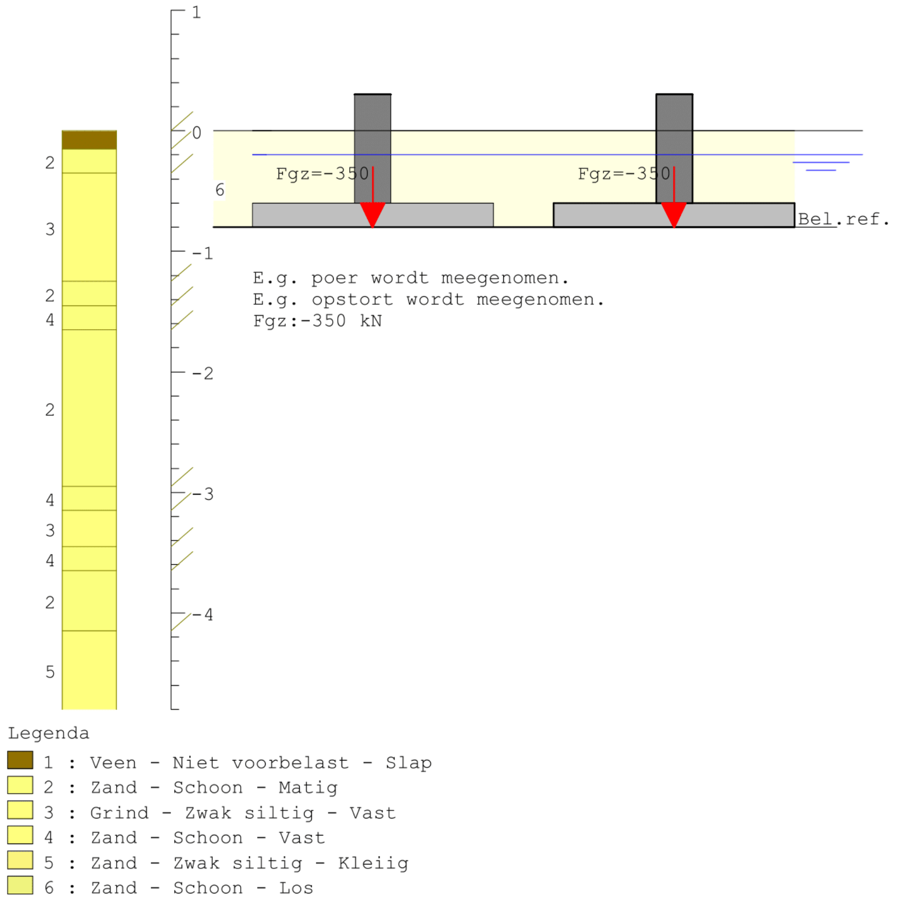
Sweco Nederland BV Blad: 8

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc04_CPT000000055844



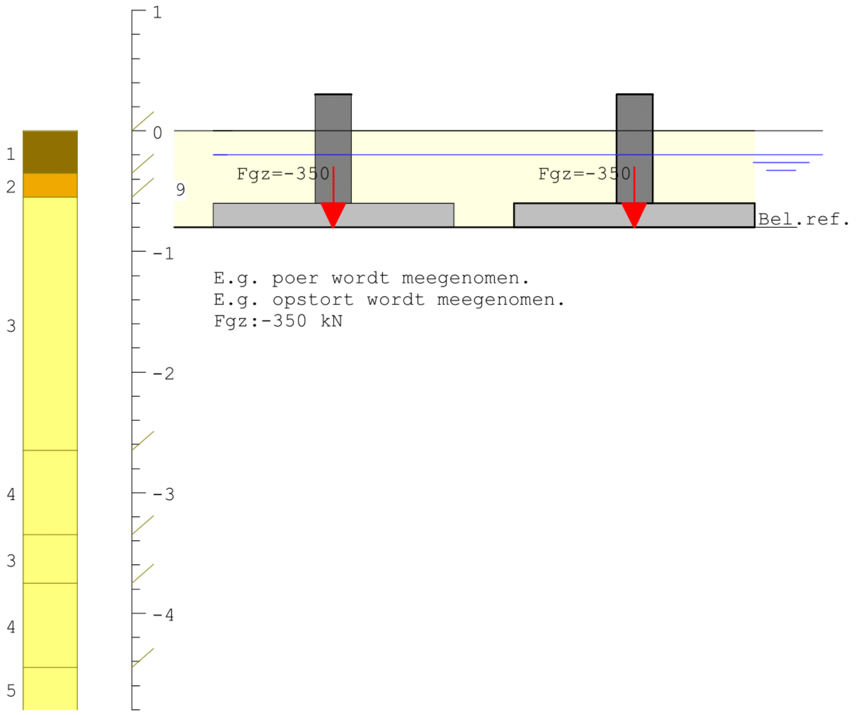
Sweco Nederland BV Blad: 9

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc06_CPT000000170226



Legenda

- 1 : Veen - Niet voorbelast - Slap
- 2 : Klei - Zwak zandig - Matig
- 3 : Grind - Zwak siltig - Vast
- 4 : Zand - Schoon - Vast
- 5 : Zand - Schoon - Matig
- 6 : Zand - Zwak siltig - Kleiig
- 7 : Grind - Sterk siltig - Los
- 8 : Zand - Sterk siltig - Kleiig
- 9 : Zand - Schoon - Los

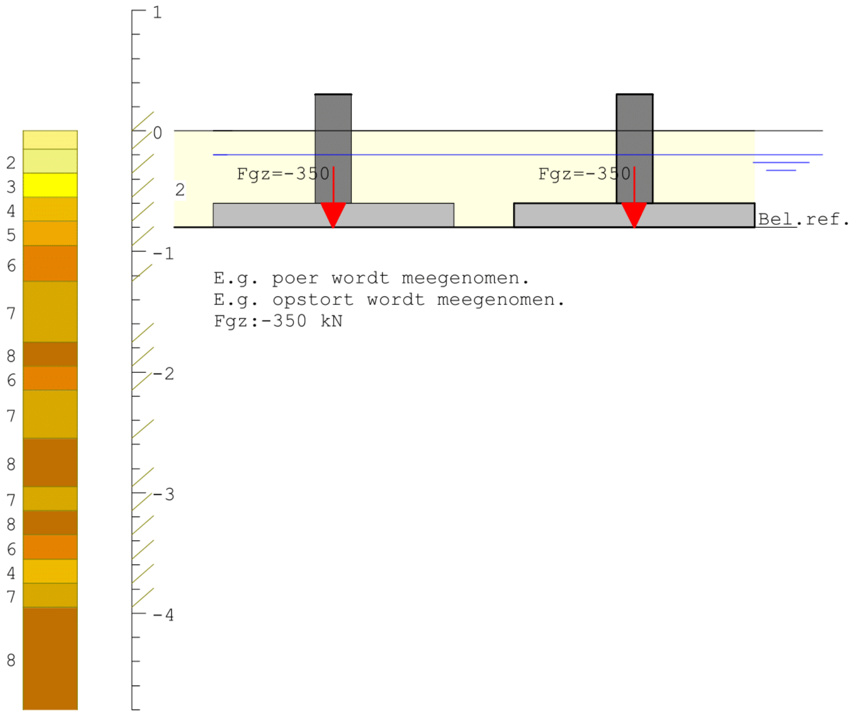
Sweco Nederland BV Blad: 10

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc08_CPT000000062979



Legenda

1 : Zand - Zwak siltig - Kleiig
2 : Zand - Schoon - Los
3 : Leem - Zwak zandig - Vast
4 : Klei - Schoon - Vast
5 : Klei - Zwak zandig - Matig
6 : Klei - Schoon - Matig
7 : Klei - Zwak zandig - Slap
8 : Klei - Organisch - Matig
9 : Veen - Matig voorbelast - Matig
10 : Klei - Zwak zandig - Vast
11 : Grind - Sterk siltig - Los
12 : Zand - Sterk siltig - Kleiig

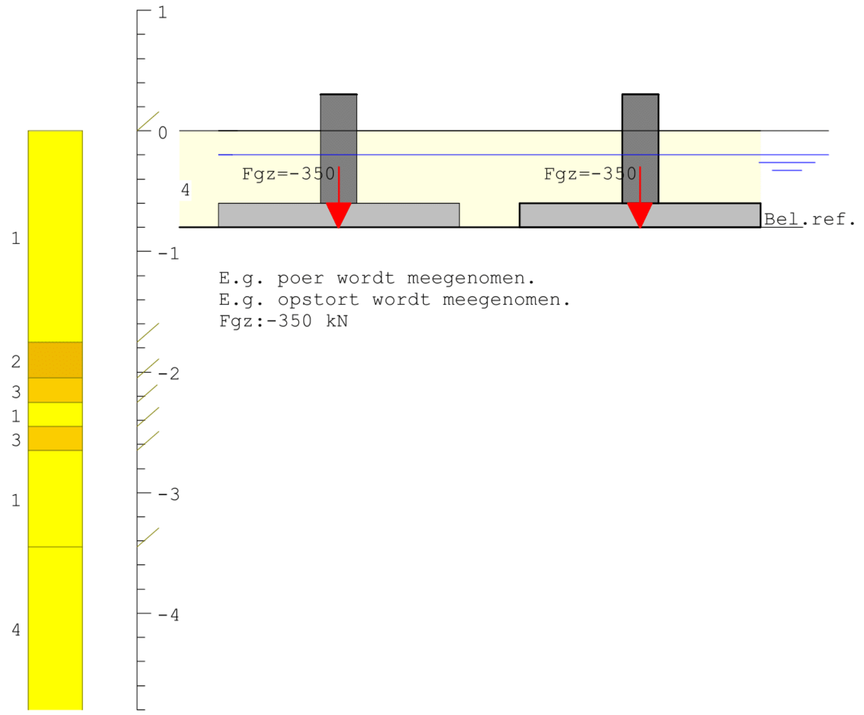
Sweco Nederland BV Blad: 11

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc12_CPT000000077437



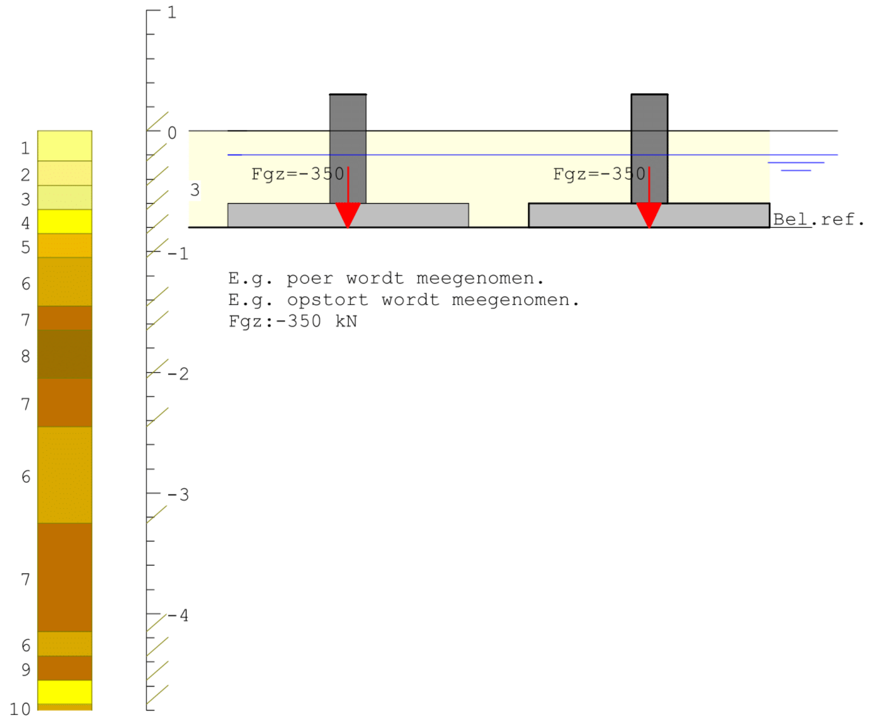
Legenda
1 : Leem - Zwak zandig - Vast
2 : Klei - Schoon - Vast
3 : Klei - Zwak zandig - Vast
4 : Zand - Schoon - Los
5 : Zand - Sterk siltig - Kleiig
6 : Klei - Zwak zandig - Matig
7 : Klei - Schoon - Matig
8 : Klei - Zwak zandig - Slap
9 : Klei - Organisch - Matig
10 : Veen - Matig voorbelast - Matig
11 : Veen - Niet voorbelast - Slap

Sweco Nederland BV Blad: 12

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)
Bodemprofiel: Loc02_CPT000000003023



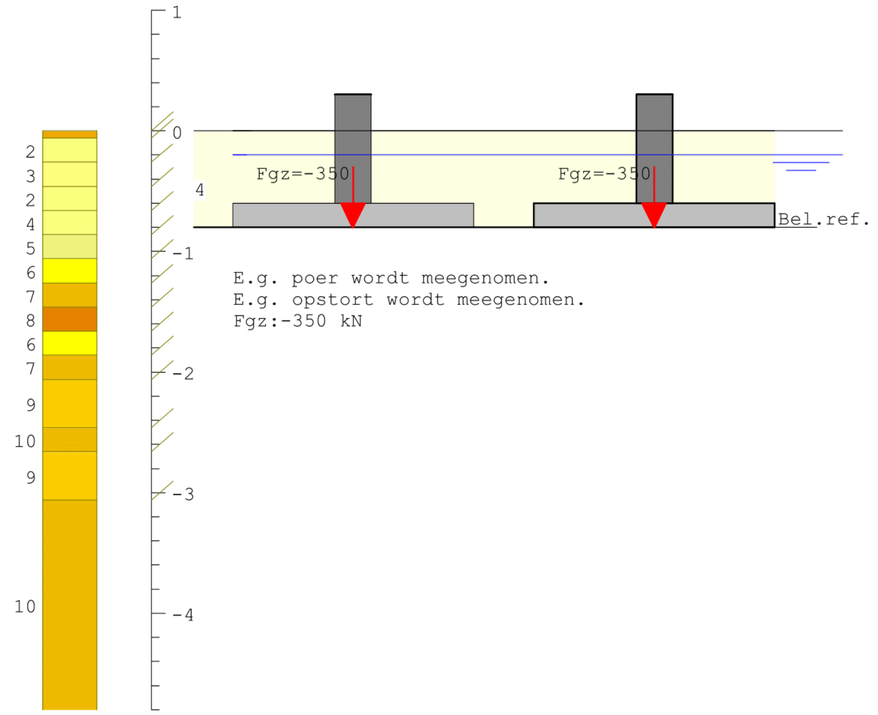
Legenda
1 : Zand - Schoon - Matig
2 : Grind - Sterk siltig - Los
3 : Zand - Schoon - Los
4 : Leem - Zwak zandig - Vast
5 : Klei - Schoon - Vast
6 : Klei - Zwak zandig - Slap
7 : Klei - Organisch - Matig
8 : Veen - Matig voorbelast - Matig
9 : Klei - Schoon - Matig
10 : Zand - Sterk siltig - Kleiig
11 : Zand - Zwak siltig - Kleiig
12 : Klei - Zwak zandig - Matig
13 : Zand - Schoon - Vast
14 : Grind - Zwak siltig - Vast

Sweco Nederland BV Blad: 13

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)
Bodemprofiel: Loc05_CPT000000161652



Legenda
- 1 : Klei - Zwak zandig - Matig
- 2 : Zand - Schoon - Matig
- 3 : Zand - Schoon - Vast
- 4 : Zand - Schoon - Los
- 5 : Leem - Zwak zandig - Vast
- 6 : Klei - Schoon - Vast
- 7 : Klei - Schoon - Matig
- 8 : Klei - Zwak zandig - Vast
- 9 : Klei - Organisch - Matig
- 10 : Klei - Zwak zandig - Slap
- 11 : Grind - Sterk siltig - Los
- 12 : Zand - Zwak siltig - Kleiig
- 13 : Zand - Sterk siltig - Kleiig

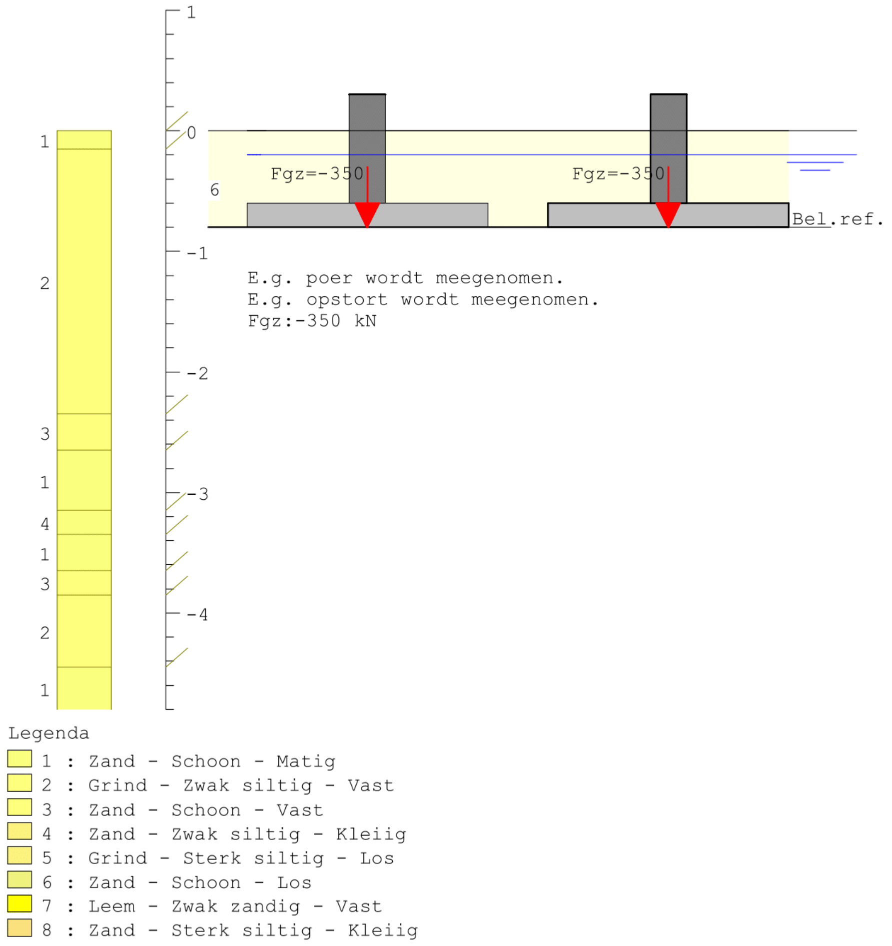
Sweco Nederland BV Blad: 14

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc07_CPT000000004530



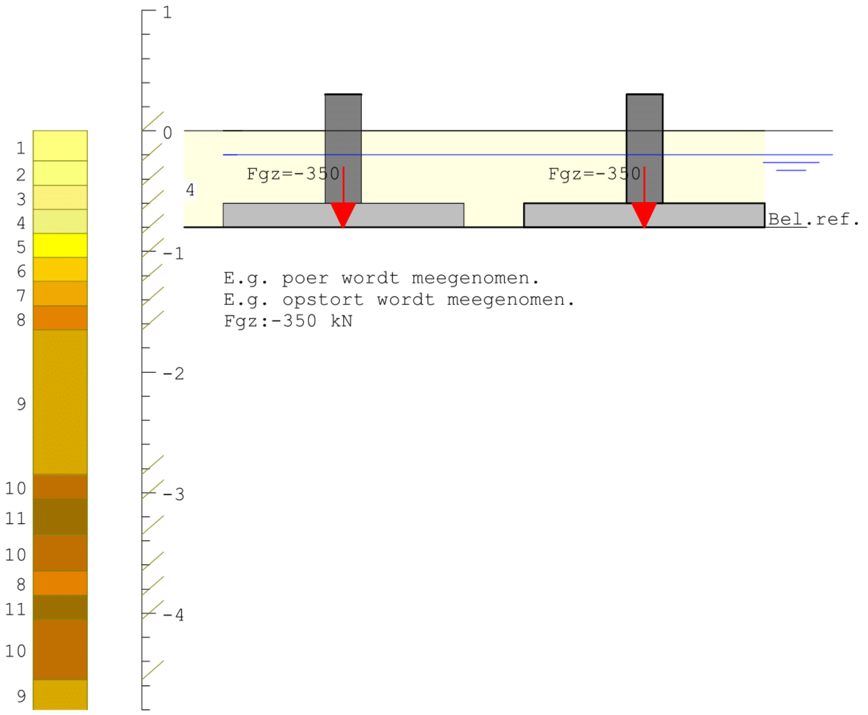
Sweco Nederland BV Blad: 15

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc09_CPT000000011161



Legenda

- 1 : Grind - Zwak siltig - Vast
- 2 : Zand - Schoon - Matig
- 3 : Zand - Zwak siltig - Kleiig
- 4 : Zand - Schoon - Los
- 5 : Leem - Zwak zandig - Vast
- 6 : Klei - Zwak zandig - Vast
- 7 : Klei - Zwak zandig - Matig
- 8 : Klei - Schoon - Matig
- 9 : Klei - Zwak zandig - Slap
- 10 : Klei - Organisch - Matig
- 11 : Veen - Matig voorbelast - Matig
- 12 : Grind - Sterk siltig - Los
- 13 : Zand - Sterk siltig - Kleiig

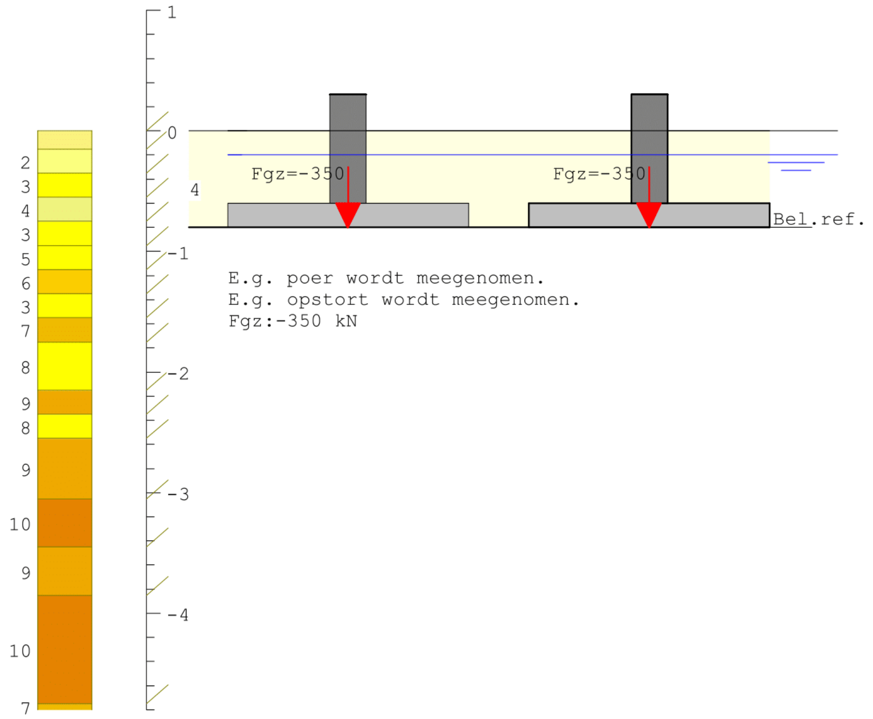
Sweco Nederland BV Blad: 16

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc10_CPT000000004377



Legenda
- 1 : Grind - Sterk siltig - Los
- 2 : Zand - Schoon - Matig
- 3 : Leem - Zwak zandig - Vast
- 4 : Zand - Schoon - Los
- 5 : Klei - Zwak zandig - Vast
- 6 : Klei - Schoon - Vast
- 7 : Klei - Zwak zandig - Matig
- 8 : Klei - Schoon - Matig
- 9 : Klei - Zwak zandig - Slap
- 10 : Klei - Organisch - Matig
- 11 : Zand - Sterk siltig - Kleiig
- 12 : Zand - Zwak siltig - Kleiig

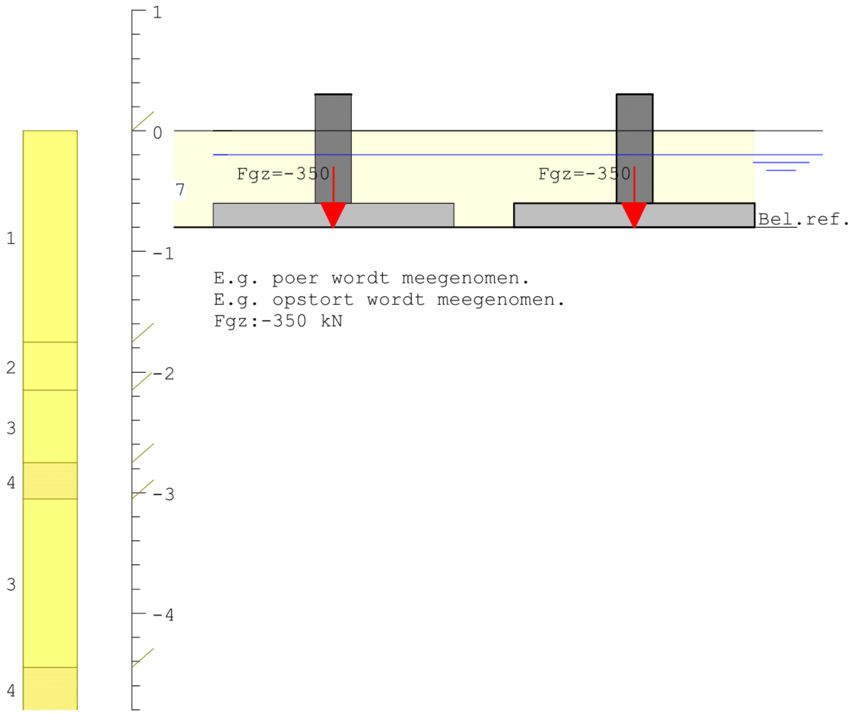
Sweco Nederland BV Blad: 17

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

INVOER GRAFISCH Case 1 (d = -0.80m, F=350) (vervolg)

Bodemprofiel: Loc11_CPT000000012135



Legenda

- 1 : Grind - Zwak siltig - Vast
- 2 : Zand - Schoon - Vast
- 3 : Zand - Schoon - Matig
- 4 : Zand - Zwak siltig - Kleiig
- 5 : Grind - Sterk siltig - Los
- 6 : Zand - Sterk siltig - Kleiig
- 7 : Zand - Schoon - Los
- 8 : Leem - Zwak zandig - Vast

Sweco Nederland BV Blad: 18

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

RESULTATEN ONGEDRAINEERD Case 1 (d = -0.80m, F=350)

Resultaten ongedraineerd gedrag laag 3 (Bodemprofiel Loc01_CPT000000052457)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: c pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-0.95	2.04	2.04	2.04	2.04	4.17	8.5	1.20	1.000	922.6	501	3848

Resultaten ongedraineerd gedrag laag 2 (Bodemprofiel Loc03_CPT000000074906)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: b pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-0.80	2.00	2.00	2.00	2.00	4.00	7.5	1.20	1.000	236.0	494	944

Resultaten ongedraineerd gedrag laag 2 (Bodemprofiel Loc08_CPT000000062979)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: b pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-0.80	2.00	2.00	2.00	2.00	4.00	7.5	1.20	1.000	373.1	494	1492

Resultaten ongedraineerd gedrag laag 2 (Bodemprofiel Loc12_CPT000000077437)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: b pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-0.80	2.00	2.00	2.00	2.00	4.00	7.5	1.20	1.000	921.5	494	3686

Resultaten ongedraineerd gedrag laag 2 (Bodemprofiel Loc02_CPT000000003023)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: b pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-0.80	2.00	2.00	2.00	2.00	4.00	7.5	1.20	1.000	921.5	494	3686

Resultaten ongedraineerd gedrag laag 3 (Bodemprofiel Loc05_CPT000000161652)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: c pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-0.86	2.02	2.02	2.02	2.02	4.07	7.9	1.20	1.000	922.0	497	3750

Resultaten ongedraineerd gedrag laag 31 (Bodemprofiel Loc07_CPT000000004530)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: c pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-11.85	5.11	5.11	5.11	5.11	26.07	100.4	1.20	1.000	1014	1.0e3	26e3

Resultaten ongedraineerd gedrag laag 3 (Bodemprofiel Loc09_CPT000000011161)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: c pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	$\leq R_d$ [kN]
2.00	1.00	1.00	-0.85	2.01	2.01	2.01	2.01	4.06	7.8	1.20	1.000	921.9	496	3740

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten ongedraineerd gedrag laag 2 (Bodemprofiel Loc10_CPT000000004377)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: b pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	-0.80	2.00	2.00	2.00	2.00	4.00	7.5	1.20	1.000	921.5	494	3686

Resultaten ongedraineerd gedrag laag 15 (Bodemprofiel Loc11_CPT000000012135)
Er is gerekend volgens art. 6.5.2.2 (f) Geval: c pons

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	b' [m]	l' [m]	A' [m^2]	$\sigma'_{v;z;d}$ [kPa]	s_c [-]	i_c [-]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	-7.75	3.95	3.95	3.95	3.95	15.63	64.8	1.20	1.000	979	0.8e3	15e3

RESULTATEN GEDRAINEERD Case 1 (d = -0.80m, F=350)

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc01_CPT000000052457)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	4.00	40.6	66.2	18.0	124.8	494	499

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc03_CPT000000074906)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	4.00	50.0	44.9	8.2	103.0	*494*	*412*

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc04_CPT000000055844)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	4.00	0.0	222.8	133.7	356.5	494	1426

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc06_CPT000000170226)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: b

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	4.00	0.0	305.3	211.4	516.6	494	2066

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc08_CPT000000062979)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	4.00	23.4	48.3	7.9	79.7	*494*	*319*

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc12_CPT000000077437)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d [kN]	≤ R_d [kN]
2.00	1.00	1.00	4.00	82.1	85.8	37.2	205.1	494	820

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Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc02_CPT000000003023)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	4.00	27.9	42.8	5.7	76.4	*494*	*306*

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc05_CPT000000161652)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	4.00	68.5	49.6	11.9	129.9	494	520

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc07_CPT000000004530)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	4.00	0.0	273.1	179.9	453.0	494	1812

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc09_CPT000000011161)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	4.00	43.8	64.2	15.8	123.9	494	496

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc10_CPT000000004377)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	4.00	72.8	58.8	16.4	147.9	494	592

Resultaten gedraineerd gedrag alle lagen (Bodemprofiel Loc11_CPT000000012135)
Er is gerekend volgens art. 6.5.2.2 (h) Geval: c

B-tot [m]	B-li [m]	B-re [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	4.00	0.0	234.8	145.4	380.1	494	1521

RESULTATEN GEDRAINEERD PONS Case 1 (d = -0.80m, F=350)
Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc01_CPT000000052457)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-1.15	4.40	42.1	95.7	14.0	151.8	508	668

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc03_CPT000000074906)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-0.80	4.00	50.0	52.9	8.2	111.1	*494*	*444*

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc04_CPT000000055844)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-4.15	8.65	0.0	1341.5	209.3	1550.8	634	13419

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Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc08_CPT000000062979)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-1.75	5.14	14.4	80.2	6.2	100.7	517	518

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-1.75	5.14	94.2	211.2	36.9	342.3	536	1759

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc02_CPT000000003023)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-0.85	4.06	26.9	51.7	5.0	83.6	*496*	*339*

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc05_CPT000000161652)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-1.06	4.30	66.5	62.1	7.7	136.3	505	586

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-3.15	7.08	0.0	847.7	136.8	984.5	596	6969

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc09_CPT000000011161)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-1.05	4.29	40.8	85.2	11.1	137.1	505	587

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc10_CPT000000004377)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-1.15	4.40	51.1	86.7	12.1	150.0	509	660

Resultaten gedraineerd gedrag ponsberekening (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	A' [m^2]	$\sigma'_{max;d;c}$ [kPa]	$\sigma'_{max;d;q}$ [kPa]	$\sigma'_{max;d;\gamma}$ [kPa]	$\sigma'_{max;d}$ [kPa]	V_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-2.75	6.49	0.0	566.2	90.1	656.3	578	4262

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Onderdeel : Isolated Footing

RESULTATEN ZAKKING Case 1 (d = -0.80m, F=350)

Resultaten zakking (Bodemprofiel Loc01_CPT000000052457)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	185.5	44.9	230.4	150.0	2220

Resultaten zakking (Bodemprofiel Loc03_CPT000000074906)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	248.9	53.9	302.8	150.0	1655

Resultaten zakking (Bodemprofiel Loc04_CPT000000055844)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	5.4	0.0	5.4	150.0	76831

Resultaten zakking (Bodemprofiel Loc06_CPT000000170226)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	3.3	0.0	3.3	150.0	125545

Resultaten zakking (Bodemprofiel Loc08_CPT000000062979)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	427.6	199.4	627.0	150.0	963

Resultaten zakking (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	72.9	19.5	92.4	150.0	5651

Resultaten zakking (Bodemprofiel Loc02_CPT000000003023)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	455.8	158.9	614.7	150.0	904

Resultaten zakking (Bodemprofiel Loc05_CPT000000161652)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	293.4	145.4	438.7	150.0	1404

Resultaten zakking (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	4.1	0.0	4.1	150.0	101612

Resultaten zakking (Bodemprofiel Loc09_CPT000000011161)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	366.9	202.1	569.0	150.0	1123

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Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten zakking (Bodemprofiel Loc10_CPT000000004377)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	267.8	129.5	397.3	150.0	1538

Resultaten zakking (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	411.9	103.0	4.7	0.0	4.7	150.0	86917

RESULTATEN ZAKKING Case 2 (d = -2.00m, F=350)

Resultaten zakking (Bodemprofiel Loc01_CPT000000052457)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	99.8	23.5	123.3	150.0	4954

Resultaten zakking (Bodemprofiel Loc03_CPT000000074906)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	118.9	23.1	142.0	150.0	4158

Resultaten zakking (Bodemprofiel Loc04_CPT000000055844)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	5.1	0.0	5.1	150.0	96886

Resultaten zakking (Bodemprofiel Loc06_CPT000000170226)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	2.7	0.0	2.7	150.0	183534

Resultaten zakking (Bodemprofiel Loc08_CPT000000062979)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	370.2	178.9	549.1	150.0	1336

Resultaten zakking (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	55.0	11.9	67.0	150.0	8981

Resultaten zakking (Bodemprofiel Loc02_CPT000000003023)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	359.6	130.7	490.3	150.0	1375

Resultaten zakking (Bodemprofiel Loc05_CPT000000161652)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	362.8	127.6	490.4	150.0	1363

Sweco Nederland BV Blad: 24

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten zakking (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	4.7	0.0	4.7	150.0	104231

Resultaten zakking (Bodemprofiel Loc09_CPT000000011161)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	441.2	206.9	648.0	150.0	1121

Resultaten zakking (Bodemprofiel Loc10_CPT000000004377)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	331.8	112.5	444.3	150.0	1490

Resultaten zakking (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	494.4	123.6	6.3	0.0	6.3	150.0	78638

RESULTATEN ZAKKING Case 3 (d = -0.80m, F=1000)

Resultaten zakking (Bodemprofiel Loc01_CPT000000052457)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	289.6	52.5	342.1	150.0	3667

Resultaten zakking (Bodemprofiel Loc03_CPT000000074906)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	370.9	55.5	426.4	150.0	2863

Resultaten zakking (Bodemprofiel Loc04_CPT000000055844)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	8.9	0.0	8.9	150.0	118689

Resultaten zakking (Bodemprofiel Loc06_CPT000000170226)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	6.6	0.0	6.6	150.0	161359

Resultaten zakking (Bodemprofiel Loc08_CPT000000062979)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	758.3	314.9	1073.1	150.0	1400

Resultaten zakking (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	117.5	20.6	138.1	150.0	9035

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten zakking (Bodemprofiel Loc02_CPT000000003023)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	741.5	189.9	931.4	150.0	1432

Resultaten zakking (Bodemprofiel Loc05_CPT000000161652)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	502.6	145.4	647.9	150.0	2113

Resultaten zakking (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	7.9	0.0	7.9	150.0	135238

Resultaten zakking (Bodemprofiel Loc09_CPT000000011161)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	649.8	231.8	881.6	150.0	1634

Resultaten zakking (Bodemprofiel Loc10_CPT000000004377)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	455.8	129.5	585.3	150.0	2330

Resultaten zakking (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1061.9	265.5	9.2	0.0	9.2	150.0	115841

RESULTATEN ZAKKING Case 4 (d = -2.00m, F=1000)

Resultaten zakking (Bodemprofiel Loc01_CPT000000052457)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	150.4	28.2	178.6	150.0	7609

Resultaten zakking (Bodemprofiel Loc03_CPT000000074906)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	171.0	25.7	196.7	150.0	6693

Resultaten zakking (Bodemprofiel Loc04_CPT000000055844)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	8.7	0.0	8.7	150.0	132186

Resultaten zakking (Bodemprofiel Loc06_CPT000000170226)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	6.2	0.0	6.2	150.0	184120

Sweco Nederland BV Blad: 26

Technosoft Funderingen op Staal release 6.70a 7 okt 2022

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten zakking (Bodemprofiel Loc08_CPT000000062979)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	670.0	315.8	985.8	150.0	1708

Resultaten zakking (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	91.4	25.6	117.0	150.0	12520

Resultaten zakking (Bodemprofiel Loc02_CPT000000003023)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	554.9	141.3	696.2	150.0	2062

Resultaten zakking (Bodemprofiel Loc05_CPT000000161652)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	561.8	127.6	689.4	150.0	2037

Resultaten zakking (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	8.8	0.0	8.8	150.0	130573

Resultaten zakking (Bodemprofiel Loc09_CPT000000011161)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	692.4	206.9	899.3	150.0	1653

Resultaten zakking (Bodemprofiel Loc10_CPT000000004377)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	510.2	120.0	630.2	150.0	2243

Resultaten zakking (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	b' [m]	l' [m]	V_d [kN]	$\sigma_{gem;d}$ [kPa]	s_1 [mm]	s_2 [mm]	s [mm]	≤ s_{req} [mm]	Veerw. [kN/m]
2.00	1.00	1.00	2.00	2.00	1144.4	286.1	11.5	0.0	11.5	150.0	99765

Horizontal Loading

Sweco Nederland BV Blad: 1

Technosoft Funderingen op Staal release 6.70a 29 mrt 2023

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

ALGEMENE GEGEVENS

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing
Eenheden : [kN][m][MPa][graden] tenzij anders vermeld
Datum : 05-10-2022
Referentieniveau (RN) : Maaiveld
Referentieperiode : 50 jaar
Bestand : C:\Users\nlpabu\OneDrive - Sweco AB\Documenten\Afstuderen\Technosoft\Draagcapaciteit Fundering op Staal (incl. hor).fsw

Toegepaste normen volgens Eurocode met Nederlandse NB

Beton	NEN-EN 1992-1-1:2011(nl)	C2/A1:2015(nl)	NB:2016(nl)
Geotechniek	EN 1997-1:2004	AC:2009	
	NEN-EN 1997-1:2005	C1+A1:2013	NB:2016
	NEN 9997-1:2016	C2:2017	

PROFIELGEGEVENS Isolated Footing 2000x2000x200

			Links	Rechts
Type			Poer	
Breedte	min	[mm]	1000	1000
	max	[mm]	1000	1000
	stap	[mm]	100	100
Lengte	voor	[mm]	1000	
	achter	[mm]	1000	De poer is vierkant.
Hoogte		[mm]	200	
Opstorting	breedte	[mm]	300	
	lengte	[mm]	300	

BELASTINGGEGEVENS Load hor

Permanent

Nr.	Omschrijving	Type	Richting	Waarde [kN,m]	AfstandX [m]	AfstandY [m]	AfstandZ [m]
1	Wind	F/q	X	63.00	-	-	0.00
2	Self-Weight	F/q	Z	-344.00	0.00	0.00	-

Extra lasten t.g.v. eigengewicht poer en opstort staan bij de rekengegevens.

RESULTATEN ONGEDRAINEERDE AFSCHUIVING Case 1 (d = -0.80m, Fz=344, Fh=63)

Resultaten ongedraineerde afschuiving (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	V_d [kN]	b' [m]	l' [m]	A' [m^2]	$c_{u;d}$ [kPa]	H_d [kN]	≤	R_d [kN]
2.00	1.00	1.00	-0.80	2.00	2.00	365.3	2.00	2.00	4.00	148.1	75.6		592.6

Resultaten ongedraineerde afschuiving (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	V_d [kN]	b' [m]	l' [m]	A' [m^2]	$c_{u;d}$ [kPa]	H_d [kN]	≤	R_d [kN]
2.00	1.00	1.00	-11.85	5.11	5.11	773.0	2.94	5.11	15.03	148.1	75.6		2227.4

Sweco Nederland BV Blad: 2

Technosoft Funderingen op Staal release 6.70a 29 mrt 2023

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten ongedraineerde afschuiving (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	V_d [kN]	b' [m]	l' [m]	A' [m^2]	$c_{u;d}$ [kPa]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-7.75	3.95	3.95	617.5	2.25	3.95	8.90	148.1	75.6	1318.9

RESULTATEN GEDRAINEERDE AFSCHUIVING Case 1 (d = -0.80m, Fz=344, Fh=63)

Resultaten gedraineerde afschuiving (Bodemprofiel Loc04_CPT000000055844)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	33.7	22.5	365.3	75.6	151.1

Resultaten gedraineerde afschuiving (Bodemprofiel Loc06_CPT000000170226)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	33.7	22.5	365.3	75.6	151.1

Resultaten gedraineerde afschuiving (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	24.4	16.2	365.3	75.6	106.4

Resultaten gedraineerde afschuiving (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	33.7	22.5	365.3	75.6	151.1

Resultaten gedraineerde afschuiving (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	33.7	22.5	365.3	75.6	151.1

RESULTATEN ONGEDRAINEERDE AFSCHUIVING Case 2 (d = -2.00m, Fz=344, Fh=63)

Resultaten ongedraineerde afschuiving (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	V_d [kN]	b' [m]	l' [m]	A' [m^2]	$c_{u;d}$ [kPa]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-2.00	2.00	2.00	439.5	2.00	2.00	4.00	74.1	75.6	296.3

Resultaten ongedraineerde afschuiving (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	V_d [kN]	b' [m]	l' [m]	A' [m^2]	$c_{u;d}$ [kPa]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-11.85	4.77	4.77	799.7	2.91	4.77	13.86	148.1	75.6	2053.2

Resultaten ongedraineerde afschuiving (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	Niv. [m]	B_{fic} [m]	L_{fic} [m]	V_d [kN]	b' [m]	l' [m]	A' [m^2]	$c_{u;d}$ [kPa]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	-7.75	3.62	3.62	644.2	2.27	3.62	8.20	148.1	75.6	1214.3

RESULTATEN GEDRAINEERDE AFSCHUIVING Case 2 (d = -2.00m, Fz=344, Fh=63)

Resultaten gedraineerde afschuiving (Bodemprofiel Loc04_CPT000000055844)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d ≤ [kN]	R_d [kN]
2.00	1.00	1.00	29.0	19.3	439.5	75.6	154.1

Sweco Nederland BV Blad: 3

Technosoft Funderingen op Staal release 6.70a 29 mrt 2023

Project : Shallow Foundation for Timber Columns
Onderdeel : Isolated Footing

Resultaten gedraineerde afschuiving (Bodemprofiel Loc06_CPT000000170226)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d [kN]	≤	R_d [kN]
2.00	1.00	1.00	33.7	22.5	439.5	75.6		181.8

Resultaten gedraineerde afschuiving (Bodemprofiel Loc12_CPT000000077437)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d [kN]	≤	R_d [kN]
2.00	1.00	1.00	15.3	10.2	439.5	75.6		79.3

Resultaten gedraineerde afschuiving (Bodemprofiel Loc07_CPT000000004530)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d [kN]	≤	R_d [kN]
2.00	1.00	1.00	33.7	22.5	439.5	75.6		181.8

Resultaten gedraineerde afschuiving (Bodemprofiel Loc11_CPT000000012135)

B-tot [m]	B-li [m]	B-re [m]	$\varphi'_{cv;d}$ [°]	δ_d [°]	V'_d [kN]	H_d [kN]	≤	R_d [kN]
2.00	1.00	1.00	31.3	20.9	439.5	75.6		167.8

D.3 Output Technosoft Pile Foundation

Example Calculation Pile Foundation

Sweco Nederland BV Blad: 1

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

DETAIL BER. DRAAGVERMOGEN Case 1; Loc01_CPT000000052457; Maaiveld-4.00

Uitgangspunten

- gehanteerde sondering : Loc01_CPT000000052457
- gehanteerde paal : Timber pile d=160
- paalpuntniveau : Maaiveld-4.00 m
- traject positieve kleef : Maaiveld-13.50 m
  tot: Maaiveld -4.00 m

Maximale draagkracht van de paalpunt

De maximale puntweerstand volgens art. 7.6.2.3 (e) bedraagt :

$q_{b;max}$ $= 0.5 * \alpha_p * \beta * s * ((q_{c;I;gem} + q_{c;II;gem})/2 + q_{c;III;gem})$
$=$ 1.758 MPa

waarin :		in dit geval :
$q_{c;I;gem}$	= de gemiddelde waarde van de conusweerstanden over traject I	= 3.60 MPa
$q_{c;II;gem}$	= de gemiddelde waarde van de conusweerstanden over traject II	= 2.75 MPa
$q_{c;III;gem}$	= de gemiddelde waarde van de conusweerstanden over traject III	= 1.85 MPa
α_p	= paalklassefactor	= 0.70 -
β	= factor voor de paalvoetvorm	= 1.00 -
φ	= hoek van de inwendige wrijving	= 27.5 -
r	= verhouding b/a	= 1.00 -
s	= factor voor de vorm van de voet	= 1.00 -

Voor een uitgebreide beschrijving van het bepalen van de gemiddelde conusweerstanden in de gebieden I, II en III wordt verwezen naar art. 7.6.2.3 (e) in de norm.

De maximale draagkracht van de paalpunt volgens art. 7.6.2.3 (c) bedraagt:

$R_{b;cal;max;i} = A_b * q_{b;max;i}$
$=$ 35 kN

waarin :		in dit geval :
A_b	= oppervlak van de paalvoet	= 0.0201 m^2

Maximale paalschachtwrijving

De maximale paalschachtwrijving volgens art. 7.6.2.3 (i) bedraagt:

$q_{s;max;z} = \alpha_s * q_{c;z;a}$

De maximale schachtwrijvingskracht volgens art. 7.6.2.3 (c) bedraagt:

$R_{s;cal;max;i} = O_{s;\Delta l;gem} * \Sigma\, q_{s;max;z;i} * d_z$
$=$ 0 kN

Sweco Nederland BV Blad: 2

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

Per laag

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

Nr	Laag	Nivo [m]	$O_{s;gem}$ [m^1]	α_s	Perc. [%]	$q_{c;z;a}$ [MPa]	$q_{s;max}$ [MPa]	d_z [m]	$R_{c;cal}$ [kN]
--	----	-13.50	--	--	--	--	--	--	--
1	Leem - Zwak zandig - Vast	-13.70	0.50	0.0000	0	5.94	0.000	0.20	0.0
totaal			0.50	0.0000		0.00	0.000	-9.50	0.0

Maximale draagkracht

De maximale draagkracht van de paal volgens art. 7.6.2.3 (c) bedraagt:

$R_{c;cal;i}$ = $R_{b;cal;max;i}$ + $R_{s;cal;max;i}$
= 35 kN (= 35 + 0)

De karakteristieke waarde van de maximale draagkracht van de paal volgens art. 7.6.2.3 (b) bedraagt:

$R_{c;k}$ = $R_{c;cal}$ / $\xi_{3\ (n=1)}$
= 25 kN

waarin : in dit geval :
$\xi_{3\ (n=1)}$ = factor volgens art. A.3.3.3 bij 1 sondering = 1.39 -

Voor de rekenwaarde van de maximale draagkracht van de paal kan volgens art. 2.4.7.3.3 worden aangehouden :

$R_{c;d}$ = $R_{c;k}$ / γ_R
= 21 kN

waarin : in dit geval :
γ_R = partiële weerstandsfactor volgens art. A.3.3.2
tabel A.6, A.7 of A.8 = 1.20 -

Sweco Nederland BV Blad: 3

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

DETAIL BER. NEGATIEVE KLEEF Case 1; Loc01_CPT000000052457; Maaiveld-4.00

Uitgangspunten

- gehanteerde sondering : Loc01_CPT000000052457
- gehanteerde paal : Timber pile d=160
- paalpuntniveau : Maaiveld -4.00 m
- paalkopniveau : Maaiveld 0.30 m
- traject negatieve kleef : Maaiveld 0.00 m tot : Maaiveld-13.50 m
- $p_{sur;k}$: 10.00 kN/m²

Berekening negatieve kleef

De karakteristieke waarde van de maximale negatieve kleefbelasting v.e. alleenstaande paal volgens art. 7.3.2.2 (d) bedraagt:

$F_{nk;k}$ = $O_{s;gem} * \sum d_j * K_{0;j;k} * \tan \delta_{j;k} * (\sigma'_{v;j-1;k} + \sigma'_{v;j;k})/2.0$
= -16.6 kN

waarin :

O_s;gem = omtrek van de dwarsdoorsnede van de paalschacht
d_j = de dikte van de grondlaag i
$K_{0;j;k}$ = de karakteristieke waarde van de neutrale gronddrukfactor in laag i
$\delta_{j;k}$ = de karakteristieke waarde van de wrijvingshoek
$\sigma'_{v;j;k}$ = de karakteristieke waarde van de effectieve verticale spanning onder in laag j

Per laag

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

Nr	Laag	Nivo [m]	Hoogte [m]	$O_{s;gem}$ [m¹]	$K_{0;j}*\tan(\delta_i)$	$\sigma'_{v;j;k}$ [kN/m²]
--	----	0.00	--	--	--	10.00
1	Grind - Zwak siltig - Vast	-0.20	0.20	0.50	0.25	14.00
2	Grind - Zwak siltig - Vast	-0.32	0.12	0.50	0.25	15.44
3	Zand - Schoon - Vast	-0.46	0.14	0.50	0.25	17.12
4	Zand - Zwak siltig - Kleiig	-0.60	0.14	0.50	0.25	18.66
5	Zand - Sterk siltig - Kleiig	-0.74	0.14	0.50	0.25	20.20
6	Zand - Schoon - Los	-0.88	0.14	0.50	0.25	21.60
7	Leem - Zwak zandig - Vast	-1.08	0.20	0.50	0.27	24.00
8	Klei - Zwak zandig - Vast	-1.22	0.14	0.50	0.25	25.54
9	Klei - Zwak zandig - Matig	-1.34	0.12	0.50	0.25	26.74
10	Klei - Zwak zandig - Slap	-1.48	0.14	0.50	0.25	27.86
11	Klei - Schoon - Vast	-1.60	0.12	0.50	0.25	29.06
12	Klei - Schoon - Matig	-1.74	0.14	0.50	0.25	30.32
13	Klei - Zwak zandig - Slap	-1.88	0.14	0.50	0.25	31.44
14	Klei - Schoon - Matig	-2.00	0.12	0.50	0.25	32.52
15	Klei - Zwak zandig - Slap	-2.12	0.12	0.50	0.25	33.48
16	Klei - Zwak zandig - Matig	-2.26	0.14	0.50	0.25	34.88
17	Klei - Schoon - Matig	-2.40	0.14	0.50	0.25	36.14
18	Klei - Zwak zandig - Matig	-2.52	0.12	0.50	0.25	37.34
19	Klei - Schoon - Matig	-2.64	0.12	0.50	0.25	38.42

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

Per laag
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

Nr	Laag	Nivo [m]	Hoogte [m]	$O_{s;gem}$ [m^1]	$K_{0;j}$*tan(δ_i)	$\sigma'_{v;j;k}$ [kN/m^2]
20	Klei - Zwak zandig - Matig	-2.78	0.14	0.50	0.25	39.82
21	Leem - Zwak zandig - Vast	-2.90	0.12	0.50	0.27	41.26
22	Zand - Schoon - Los	-3.22	0.32	0.50	0.25	44.46
23	Leem - Zwak zandig - Vast	-3.36	0.14	0.50	0.27	46.14
24	Zand - Schoon - Los	-3.48	0.12	0.50	0.25	47.34
25	Zand - Sterk siltig - Kleiig	-3.60	0.12	0.50	0.25	48.66
26	Zand - Schoon - Los	-3.72	0.12	0.50	0.25	49.86
27	Leem - Zwak zandig - Vast	-4.00	0.28	0.50	0.27	53.22

Rekenwaarde
De rekenwaarde van de maximale negatieve kleefbelasting van een alleenstaande paal bedraagt :
$F_{nk;d}$ = $F_{nk;k}$ * $\gamma_{f;nk}$ = -16.6 kN

waarin : in dit geval :
$\gamma_{f;nk}$ = belastingfactor voor de negatieve kleef (art. 7.3.2.2 (b)) 1.0 -

Results Pile Foundation

Sweco Nederland BV Blad: 1

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

ALGEMENE GEGEVENS

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile
Datum : 05-10-2022
Bestand : C:\Users\nlpabu\OneDrive - Sweco AB\Documenten\Afstuderen\Technosoft\Draagcapaciteit Houten Paalfundering d160.pvw
Berekeningstype : Verticaal belaste paal
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

Toegepaste normen volgens Eurocode met Nederlandse NB

Geotechniek	EN 1997-1:2004	AC:2009	
	NEN-EN 1997-1:2005	C1+A1:2013	NB:2016
	NEN 9997-1:2016	C2:2017	

GRONDSOORTEN

Nr.	Omschrijving	$\gamma_{k;1}$ [kN/m³]	$\gamma_{sat;k;1}$ [kN/m³]	$\varphi'_{k;1}$ [°]	$\gamma_{k;2}$ [kN/m³]	$\gamma_{sat;k;2}$ [kN/m³]	$\varphi'_{k;2}$ [°]
1	Grind - Zwak siltig - Los	17.00	19.00	32.50	18.00	20.00	35.00
2	Grind - Zwak siltig - Matig	18.00	20.00	35.00	19.00	21.00	37.50
3	Grind - Zwak siltig - Vast	19.00	21.00	37.50	20.00	22.00	40.00
4	Grind - Sterk siltig - Los	18.00	20.00	30.00	19.00	21.00	32.50
5	Grind - Sterk siltig - Matig	19.00	21.00	32.50	20.00	22.00	35.00
6	Zand - Schoon - Los	17.00	19.00	30.00	18.00	20.00	32.50
7	Zand - Schoon - Matig	18.00	20.00	32.50	19.00	21.00	35.00
8	Zand - Schoon - Vast	19.00	21.00	35.00	20.00	22.00	40.00
9	Zand - Zwak siltig - Kleiig	18.00	20.00	27.00	19.00	21.00	32.50
10	Zand - Sterk siltig - Kleiig	18.00	20.00	25.00	19.00	21.00	30.00
11	Leem - Zwak zandig - Matig	20.00	20.00	27.50	21.00	21.00	32.50
12	Leem - Zwak zandig - Vast	21.00	21.00	27.50	22.00	22.00	35.00
13	Leem - Sterk zandig	19.00	19.00	27.50	20.00	20.00	35.00
14	Klei - Schoon - Matig	17.00	17.00	17.50	19.00	19.00	17.50
15	Klei - Schoon - Vast	19.00	19.00	17.50	20.00	20.00	25.00
16	Klei - Zwak zandig - Slap	15.00	15.00	22.50	18.00	18.00	22.50
17	Klei - Zwak zandig - Matig	18.00	18.00	22.50	20.00	20.00	22.50
18	Klei - Zwak zandig - Vast	20.00	20.00	22.50	21.00	21.00	27.50
19	Klei - Sterk zandig	18.00	18.00	27.50	20.00	20.00	32.50
20	Klei - Organisch - Slap	13.00	13.00	15.00	15.00	15.00	15.00
21	Klei - Organisch - Matig	15.00	15.00	15.00	16.00	16.00	15.00
22	Veen - Niet voorbelast - Slap	10.00	10.00	15.00	12.00	12.00	15.00
23	Veen - Matig voorbelast - Matig	12.00	12.00	15.00	13.00	13.00	15.00

PAALGEGEVENS Timber pile d=160

Type : Houten paal (constant)
Wijze van installeren : Heien
Diameter [m] : 0.160
Elasticiteitsmodulus [N/mm²] : 3600
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Factor α_t (tabel 7.c EC 7.1) : 0.0070 (zandlagen; voor kleilagen zie tabel 7.d)
Paalklassefactor α_p : 0.70
Paalvoetvormfactor β : 1.00
Type lastzakkingsdiagram : Grondverdringende paal
Verm.factor * $\varphi'_{j;k}$: 0.75

Sweco Nederland BV Blad: 2

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project	: Pile Foundation for Timber Columns
Onderdeel	: Timber Pile

PAALGEGEVENS Timber pile d=200

Type		: Houten paal (constant)
Wijze van installeren		: Heien
Diameter	[m]	: 0.200
Elasticiteitsmodulus	[N/mm²]	: 3600
Factor α_s (tabel 7.c EC 7.1)		: 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Factor α_t (tabel 7.c EC 7.1)		: 0.0070 (zandlagen; voor kleilagen zie tabel 7.d)
Paalklassefactor α_p		: 0.70
Paalvoetvormfactor β		: 1.00
Type lastzakkingsdiagram		: Grondverdringende paal
Verm.factor * $\varphi'_{j;k}$		: 0.75

PAALGEGEVENS Concrete pile #400

Type		: Geheide paal (beton)
Wijze van installeren		: Heien
Afmeting a	[m]	: 0.400
Afmeting b	[m]	: 0.400
Elasticiteitsmodulus	[N/mm²]	: 20000
Factor α_s (tabel 7.c EC 7.1)		: 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Factor α_t (tabel 7.c EC 7.1)		: 0.0070 (zandlagen; voor kleilagen zie tabel 7.d)
Paalklassefactor α_p		: 0.70
Paalvoetvormfactor β		: 1.00
Type lastzakkingsdiagram		: Grondverdringende paal
Verm.factor * $\varphi'_{j;k}$		: 0.75

Sweco Nederland BV Blad: 3

Technosoft Paalfunderingen release 6.70a 14 okt 2022

```
Project               : Pile Foundation for Timber Columns
Onderdeel             : Timber Pile
```

REKENGEGEVENS Case 1

```
Berekening            : Ontwerpend
Rekenmethode          : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en)         : Loc01_CPT000000052457

Stijf bouwwerk        : NEE
Paalgroep             : NEE
Aantal sonderingen    :    1
```

Factor $\xi_{3\ (n=1)}$: 1.39

Factor $\xi_{3\ (gem)}$: 1.39

Factor $\xi_{4\ (min)}$: 1.39

Weerstandsfactor γ_R : 1.20

$\gamma_{f;nk}$: 1.0

$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE

UGT draagvermogen zonder negatieve kleef : NEE

```
Paal                  : Timber pile d=160
Niveau paalkop    [m] : Maaiveld   0.30
Paalpuntniveau        : Maaiveld  -4.00
Bovenbel.   [kN/m²]   :  -10.00
```

Sweco Nederland BV Blad: 4

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 1 (n=1)

Uitgangspunten
- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc01_CPT0000000	0.00	-4.00	35.4	0.0	35.4	21.2	-16.6	4.6

REKENGEGEVENS Case 2

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc02_CPT000000003023

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -20.70
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 5

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 2 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc02_CPT0000000	0.00	-20.70	301.6	164.4	466.0	279.4	-188.1	91.2

REKENGEGEVENS Case 3

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc03_CPT000000074906

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -5.90
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 6

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 3 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc03_CPT0000000	0.00	-5.90	81.6	76.8	158.3	94.9	-9.5	85.4

REKENGEGEVENS Case 4

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc04_CPT000000055844

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -8.00
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 7

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 4 (n=1)

Uitgangspunten
- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc04_CPT0000000	0.00	-8.00	281.5	147.7	429.2	257.3	0.0	257.3

REKENGEGEVENS Case 5

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc05_CPT000000161652

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -14.30
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 8

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 5 (n=1)

Uitgangspunten
- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc05_CPT0000001	0.00	-14.30	301.6	503.5	805.1	482.7	-25.9	456.8

REKENGEGEVENS Case 6

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc06_CPT000000170226

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -19.30
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 9

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 6 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc06_CPT0000001	0.00	-19.30	178.6	919.9	1098.5	658.6	0.0	658.6

REKENGEGEVENS Case 7

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc07_CPT000000004530

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -14.50
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 10

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 7 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc07_CPT0000000	0.00	-14.50	251.1	562.7	813.9	487.9	0.0	487.9

REKENGEGEVENS Case 8

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc08_CPT000000062979

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -2.00
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 11

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 8 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc08_CPT0000000	0.00	-2.00	3.5	0.0	3.5	2.1	-5.5	-3.4

REKENGEGEVENS Case 9

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc09_CPT000000011161

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -15.70
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 12

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 9 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc09_CPT0000000	0.00	-15.70	212.6	327.0	539.6	323.5	-35.9	287.7

REKENGEGEVENS Case 10

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc10_CPT000000004377

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -20.10
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 13

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 10 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc10_CPT0000000	0.00	-20.10	279.8	249.0	528.8	317.0	-156.8	160.2

REKENGEGEVENS Case 11

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc11_CPT000000012135

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -14.20
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 14

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 11 (n=1)

Uitgangspunten

- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc11_CPT0000000	0.00	-14.20	138.6	404.5	543.1	325.6	0.0	325.6

REKENGEGEVENS Case 12

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc12_CPT000000077437

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=160
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -9.30
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 15
Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 12 (n=1)

Uitgangspunten
- paal : Timber pile d=160
- paaltype : Houten paal (constant)
- schachtafmeting : 160 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc12_CPT0000000	0.00	-9.30	75.2	102.7	178.0	106.7	-14.9	91.8

Sweco Nederland BV Blad: 16

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

OVERZICHT NETTO DRAAGVERMOGEN DRUKPALEN

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

	maaiveld	paalpunt	$R_{c;netto;d}$	[kN]			
sondering	niveau	niveau	Case 1	Case 2	Case 3	Case 4	Case 5
Loc01_CPT0000000	0.00	-4.00	4				
Loc03_CPT0000000	0.00	-5.90			85		
Loc04_CPT0000000	0.00	-8.00				257	
Loc02_CPT0000000	0.00	-20.70		91			
Loc05_CPT0000001	0.00	-14.30					456

Sweco Nederland BV Blad: 17

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	$R_{c;netto;d}$ [kN] Case 6	Case 7	Case 8	Case 9	Case 10
Loc06_CPT0000001	0.00	-19.30	658				
Loc08_CPT0000000	0.00	-2.00			-4		
Loc07_CPT0000000	0.00	-14.50		487			
Loc09_CPT0000000	0.00	-15.70				287	
Loc10_CPT0000000	0.00	-20.10					160

Sweco Nederland BV Blad: 18

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	$R_{c;netto;d}$ [kN] Case 11	Case 12
Loc12_CPT0000000	0.00	-9.30		91
Loc11_CPT0000000	0.00	-14.20	325	

Sweco Nederland BV Blad: 21

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

REKENGEGEVENS Case 1

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc01_CPT000000052457

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -4.00
Bovenbel. [kN/m²] : -10.00

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 1 (n=1)

Uitgangspunten
- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc01_CPT0000000	0.00	-4.00	52.3	0.0	52.3	31.4	-20.7	10.6

REKENGEGEVENS Case 2

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc02_CPT000000003023

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -20.60
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 23

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 2 (n=1)

Uitgangspunten

- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc02_CPT0000000	0.00	-20.60	471.2	205.5	676.7	405.7	-235.1	170.6

REKENGEGEVENS Case 3

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc03_CPT000000074906

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -5.70
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 24

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 3 (n=1)

Uitgangspunten

- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc03_CPT0000000	0.00	-5.70	116.0	89.0	205.1	122.9	-11.9	111.0

REKENGEGEVENS Case 4

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc04_CPT000000055844

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -7.80
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 25

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 4 (n=1)

Uitgangspunten

- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc04_CPT0000000	0.00	-7.80	439.8	184.6	624.4	374.4	0.0	374.4

REKENGEGEVENS Case 5

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc05_CPT000000161652

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -14.10
Bovenbel. [kN/m²] : -10.00

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 5 (n=1)

Uitgangspunten
- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc05_CPT0000001	0.00	-14.10	471.2	610.5	1081.8	648.5	-32.4	616.2

REKENGEGEVENS Case 6

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc06_CPT000000170226

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -19.10
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 27

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 6 (n=1)

Uitgangspunten

- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc06_CPT0000001	0.00	-19.10	283.1	1134.6	1417.8	850.0	0.0	850.0

REKENGEGEVENS Case 7

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc07_CPT000000004530

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -14.30
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 28

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 7 (n=1)

Uitgangspunten

- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc07_CPT0000000	0.00	-14.30	334.4	684.6	1019.0	610.9	0.0	610.9

REKENGEGEVENS Case 8

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc08_CPT000000062979

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -2.00
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 29

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 8 (n=1)

Uitgangspunten

- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc08_CPT0000000	0.00	-2.00	5.3	0.0	5.3	3.2	-6.8	-3.6

REKENGEGEVENS Case 9

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc09_CPT000000011161

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -15.50
Bovenbel. [kN/m²] : -10.00

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 9 (n=1)

Uitgangspunten
- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc09_CPT0000000	0.00	-15.50	324.0	392.7	716.6	429.6	-44.8	384.8

REKENGEGEVENS Case 10

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc10_CPT000000004377

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -19.90
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 31

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 10 (n=1)

Uitgangspunten

- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc10_CPT0000000	0.00	-19.90	434.1	292.4	726.5	435.5	-196.0	239.5

REKENGEGEVENS Case 11

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc11_CPT000000012135

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -14.00
Bovenbel. [kN/m²] : -10.00

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 11 (n=1)

Uitgangspunten
- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc11_CPT0000000	0.00	-14.00	204.1	494.0	698.2	418.6	0.0	418.6

REKENGEGEVENS Case 12

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc12_CPT000000077437

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Timber pile d=200
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -9.10
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 33

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Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

SAMENVATTINGSTABEL Case 12 (n=1)

Uitgangspunten
- paal : Timber pile d=200
- paaltype : Houten paal (constant)
- schachtafmeting : 200 mm

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc12_CPT0000000	0.00	-9.10	113.7	120.8	234.5	140.6	-18.6	122.0

Sweco Nederland BV Blad: 34

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

OVERZICHT NETTO DRAAGVERMOGEN DRUKPALEN

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

	maaiveld	paalpunt	$R_{c;netto;d}$	[kN]			
sondering	niveau	niveau	Case 1	Case 2	Case 3	Case 4	Case 5
Loc01_CPT0000000	0.00	-4.00	10				
Loc03_CPT0000000	0.00	-5.70			111		
Loc04_CPT0000000	0.00	-7.80				374	
Loc02_CPT0000000	0.00	-20.60		170			
Loc05_CPT0000001	0.00	-14.10					616

Sweco Nederland BV Blad: 35

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	$R_{c;netto;d}$ [kN] Case 6	Case 7	Case 8	Case 9	Case 10
Loc06_CPT0000001	0.00	-19.10	849				
Loc08_CPT0000000	0.00	-2.00			-4		
Loc07_CPT0000000	0.00	-14.30		610			
Loc09_CPT0000000	0.00	-15.50				384	
Loc10_CPT0000000	0.00	-19.90					239

Sweco Nederland BV Blad: 36

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Timber Columns
Onderdeel : Timber Pile

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	$R_{c;netto;d}$ [kN] Case 11	Case 12
Loc12_CPT0000000	0.00	-9.10		122
Loc11_CPT0000000	0.00	-14.00	418	

Sweco Nederland BV Blad: 37

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

REKENGEGEVENS Case 1

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc01_CPT000000052457

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -28.20
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 38

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 1 (n=1)

Uitgangspunten

- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc01_CPT0000000	0.00	-28.20	818.4	522.6	1341.0	804.0	-462.9	341.0

REKENGEGEVENS Case 2

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc02_CPT000000003023

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -27.10
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 39

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 2 (n=1)

Uitgangspunten

- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc02_CPT0000000	0.00	-27.10	2400.0	1740.9	4140.9	2482.6	-598.8	1883.8

REKENGEGEVENS Case 3

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc03_CPT000000074906

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -6.10
Bovenbel. [kN/m²] : -10.00

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 3 (n=1)

Uitgangspunten
- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc03_CPT0000000	0.00	-6.10	331.7	264.4	596.1	357.4	-30.3	327.1

REKENGEGEVENS Case 4

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc04_CPT000000055844

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -7.10
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 41

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 4 (n=1)

Uitgangspunten

- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc04_CPT0000000	0.00	-7.10	2369.9	470.1	2840.1	1702.7	0.0	1702.7

REKENGEGEVENS Case 5

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc05_CPT000000161652

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -13.20
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 42

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 5 (n=1)

Uitgangspunten

- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc05_CPT0000001	0.00	-13.20	1703.7	1338.8	3042.5	1824.1	-82.4	1741.7

REKENGEGEVENS Case 6

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc06_CPT000000170226

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -16.00
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 43

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Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 6 (n=1)

Uitgangspunten
- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc06_CPT0000001	0.00	-16.00	1916.2	2415.0	4331.2	2596.6	0.0	2596.6

REKENGEGEVENS Case 7

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc07_CPT000000004530

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -13.40
Bovenbel. [kN/m²] : -10.00

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 7 (n=1)

Uitgangspunten
- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc07_CPT0000000	0.00	-13.40	962.1	1612.4	2574.5	1543.5	0.0	1543.5

REKENGEGEVENS Case 8

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc08_CPT000000062979

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -26.80
Bovenbel. [kN/m²] : -10.00

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 8 (n=1)

Uitgangspunten
- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc08_CPT0000000	0.00	-26.80	1377.5	1041.9	2419.4	1450.5	-584.9	865.5

REKENGEGEVENS Case 9

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc09_CPT000000011161

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -14.60
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 46

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Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 9 (n=1)

Uitgangspunten

- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc09_CPT0000000	0.00	-14.60	1290.1	871.9	2162.0	1296.2	-114.1	1182.1

REKENGEGEVENS Case 10

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc10_CPT000000004377

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -23.70
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 47

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 10 (n=1)

Uitgangspunten

- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc10_CPT0000000	0.00	-23.70	960.7	1314.5	2275.2	1364.0	-499.2	864.8

REKENGEGEVENS Case 11

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc11_CPT000000012135

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -13.10
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 48

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Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 11 (n=1)

Uitgangspunten
- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc11_CPT0000000	0.00	-13.10	839.7	1124.6	1964.3	1177.6	0.0	1177.6

REKENGEGEVENS Case 12

Berekening : Ontwerpend
Rekenmethode : Drukpalen volgens NEN-EN 1997-1, art. 7.6.2
Sondering(en) : Loc12_CPT000000077437

Stijf bouwwerk : NEE
Paalgroep : NEE
Aantal sonderingen : 1
Factor $\xi_{3\ (n=1)}$: 1.39
Factor $\xi_{3\ (gem)}$: 1.39
Factor $\xi_{4\ (min)}$: 1.39
Weerstandsfactor γ_R : 1.20
$\gamma_{f;nk}$: 1.0
$R_{s;cal;max;i}$ begrenzen op 0.75 * $R_{b;cal;max;i}$: NEE
UGT draagvermogen zonder negatieve kleef : NEE

Paal : Concrete Pile 400x400
Niveau paalkop [m] : Maaiveld 0.30
Paalpuntniveau : Maaiveld -9.30
Bovenbel. [kN/m²] : -10.00

Sweco Nederland BV Blad: 49

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Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

SAMENVATTINGSTABEL Case 12 (n=1)

Uitgangspunten

- paal : Concrete Pile 400x400
- paaltype : Geheide paal (beton)
- schachtafmeting : 400 x 400

Paalklassefactor α_p : 0.70
Factor α_s (tabel 7.c EC 7.1) : 0.010 (zandlagen; voor kleilagen zie tabel 7.d)
Correlatiefactor $\xi_{3(n=1)}$: 1.39

Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	Bezwijkdraagvermogen $R_{b;cal}$ [kN]	$R_{s;cal}$ [kN]	$R_{c;cal}$ [kN]	Rekenwaarden $R_{c;d}$ [kN]	$F_{nk;d}$ [kN]	$R_{c;netto;d}$ [kN]
Loc12_CPT0000000	0.00	-9.30	344.5	327.0	671.4	402.5	-47.3	355.3

Sweco Nederland BV Blad: 50

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Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

OVERZICHT NETTO DRAAGVERMOGEN DRUKPALEN

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	$R_{c;netto;d}$ [kN] Case 1	Case 2	Case 3	Case 4	Case 5
Loc01_CPT0000000	0.00	-28.20	341				
Loc03_CPT0000000	0.00	-6.10			327		
Loc04_CPT0000000	0.00	-7.10				1702	
Loc02_CPT0000000	0.00	-27.10		1883			
Loc05_CPT0000001	0.00	-13.20					1741

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	$R_{c;netto;d}$ [kN] Case 6	Case 7	Case 8	Case 9	Case 10
Loc06_CPT0000001	0.00	-16.00	2596				
Loc08_CPT0000000	0.00	-26.80			865		
Loc07_CPT0000000	0.00	-13.40		1543			
Loc09_CPT0000000	0.00	-14.60				1182	
Loc10_CPT0000000	0.00	-23.70					864

Sweco Nederland BV Blad: 52

Technosoft Paalfunderingen release 6.70a 14 okt 2022

Project : Pile Foundation for Concrete Columns
Onderdeel : Concrete Pile

Netto paaldraagvermogen(s) zijn naar beneden toe afgerond op: 1.0 kN nauwkeurig
Alle niveaus/hoogtes/peilmaten zijn t.o.v.: Maaiveld

sondering	maaiveld niveau	paalpunt niveau	$R_{c;netto;d}$ [kN] Case 11	Case 12
Loc12_CPT0000000	0.00	-9.30		355
Loc11_CPT0000000	0.00	-13.10	1177	

E.

Comparison Modular or Timber Car Park Concepts

Table 55. Data modular or timber car park concepts for comparison

ModuPark

IDXX	Modupark, 16.50m (16.26m), column every 2 parking bays (corrected for comparable dimensions)						
Car Park Characteristics							
Parking bay width	2.5 m	Span parking deck	16.26 m	Floor depth	550 mm	Load standard column - 1 level	618.1 kN
Parking bay length	5.13 m	Column free parking deck	Yes	Total weight	61,261 kg	Load standard column - 2 levels	1,236.1 kN
Aisle width	6.00 m	Clearance height	2.3 m			Load standard column - 3 levels	1,854.2 kN
Parking angle	90°	Number of parking levels	4			Load standard column - 4 levels	2,472.3 kN
						Load standard column - 5 levels	3,090.3 kN

Description	Profile	Material	Length [mm]	Width [mm]	Height [mm]	Cross-area [mm²]	A [m²]	Volume [mm³]	V [m³]	Density [kg/m³]	Weight [kg]	Weight [kN]	Nr. of elements	Weight structure [kg]	Volume [m³]
TT-slab	TT-550+0	C50/60	16,260	2,500	550	374,702	0.37	6,092,654,520	6.09	2400	14,622.4	146.22	4	58,489.5	24.4
Transverse main girder	IPE 360	S355	5,000	170	360	7,273	0.01	36,365,000	0.04	7850	291.0	2.91	4	1,164.0	0.1
Column	SHS-HF 250x250x12.5(8-20)	S355	2,850	250	250	17,971	0.02	51,217,350	0.05	7850	401.9	4.02	4	1,607.4	0.2

Koopman int., Amsterdam

IDXX	Koopman int. (steel + fibreglass), 15.65m (16.26m), column every 2 parking bays (corrected for comparable dimensions)						
Car Park Characteristics							
Parking bay width	2.5 m	Span parking deck	16.26 m	Floor depth	640 mm	Load standard column - 1 level	314.6 kN
Parking bay length	5.13 m	Column free parking deck	Yes	Total weight	10,477 kg	Load standard column - 2 levels	629.2 kN
Aisle width	6.00 m	Clearance height	2.3 m			Load standard column - 3 levels	943.7 kN
Parking angle	90°	Number of parking levels	4			Load standard column - 4 levels	1,258.3 kN
						Load standard column - 5 levels	1,572.9 kN

Description	Profile	Material	Length [mm]	Width [mm]	Height [mm]	Cross-area [mm²]	A [m²]	Volume [mm³]	V [m³]	Density [kg/m³]	Weight [kg]	Weight [kN]	Nr. of elements	Weight structure [kg]	Volume [m³]
Deck	Fiberline Plank 40HD	GFRP	16,260	500	40	4,783	0.00	77,771,580	0.08		137.1	1.37	20	2,741.4	1.6
Secondary girder	HEA 140	S355	5,000	140	133	3,142	0.00	15,710,000	0.02	7850	125.5	1.26	24	3,012.0	0.4
Main	IPE 600	S355	16,260	220	600	15,598	0.02	253,623,480	0.25	7850	2,032.5	20.33	2	4,065.0	0.5
Column	K 260x260x7.1	S355	2,940	260	260	7,128	0.01	20,956,320	0.02	7850	164.6	1.65	4	658.6	0.1

Morspoort, Leiden

IDXX	Morspoort (steel + composite), 14.50 (16.26m), column every 2 parking bays (corrected for comparable dimensions)						
Car Park Characteristics							
Parking bay width	2.5 m	Span parking deck	16.26 m	Floor depth	651 mm	Load standard column - 1 level	461.1 kN
Parking bay length	5.13 m	Column free parking deck	Yes	Total weight	35,110 kg	Load standard column - 2 levels	922.2 kN
Aisle width	6.00 m	Clearance height	2.3 m			Load standard column - 3 levels	1,383.4 kN
Parking angle	90°	Number of parking levels	4			Load standard column - 4 levels	1,844.5 kN
						Load standard column - 5 levels	2,305.6 kN

Description	Profile	Material	Length [mm]	Width [mm]	Height [mm]	Cross-area [mm²]	A [m²]	Volume [mm³]	V [m³]	Density [kg/m³]	Weight [kg]	Weight [kN]	Nr. of elements	Weight structure [kg]	Volume [m³]
Deck	Quantum Deck Floor	e (Concret	5,000	16,260	250		0.00	0	0.00		14,634.0	146.34	2	29,268.0	0.0
Joist	K 120x120x5	S275	5,000	120	120	2,273	0.00	11,365,000	0.01	7850	89.0	0.89	8	712.0	0.1
Main	IPE600	S355	16,260	220	600	15,598	0.02	253,623,480	0.25	7850	2,032.5	20.33	2	4,065.0	0.5
Column	K 300x300x10	S275	2,951	300	300	11,493	0.01	33,915,843	0.03	7850	266.2	2.66	4	1,064.7	0.1

Table 56. Data modular or timber car park concepts for comparison

Construction system for multi-storey car park in BauBuche

IDXX	BauBuche concept (timber + concrete), 16.50m (16.26m), column every 1 parking bay (corrected for comparable dimensions)						
Car Park Characteristics							
Parking bay width	2.5 m	Span parking deck	16.26 m	Floor depth	730 mm	Load standard column - 1 level	298.3 kN
Parking bay length	5.13 m	Column free parking deck	Yes	Total weight	59,341 kg	Load standard column - 2 levels	596.6 kN
Aisle width	6.00 m	Clearance height	2.3 m			Load standard column - 3 levels	894.9 kN
Parking angle	90°	Number of parking levels	4			Load standard column - 4 levels	1,193.2 kN
						Load standard column - 5 levels	1,491.5 kN

Description	Profile	Material	Length [mm]	Width [mm]	Height [mm]	Cross-area [mm²]	A [m²]	Volume [mm³]	V [m³]	Density [kg/m³]	Weight [kg]	Weight [kN]	Nr. of elements	Weight structure [kg]	Volume [m³]
Deck	Precast concrete 130 mm	C50/60	2,500	16,260	130	2,113,800	2.11	5,284,500,000	5.28	2400	12,682.8	126.83	4	50,731.2	21.1
Main	240x600, cambered 40 mm	BB GL75	16,260	240	600	144,000	0.14	2,341,440,000	2.34	800	1,873.2	18.73	4	7,492.6	9.4
Column	240x240	BB GL75	3,030	240	240	57,600	0.06	174,528,000	0.17	800	139.6	1.40	8	1,117.0	1.4

B&O-Holzparkhaus, Bad Aibling

IDXX	B&O-Holzparkhaus (timber), 16.24m (16.26m), column every 1 parking bay (corrected for comparable dimensions)						
Car Park Characteristics							
Parking bay width	2.5 m	Span parking deck	16.26 m	Floor depth	935 mm	Load standard column - 1 level	179.2 kN
Parking bay length	5.13 m	Column free parking deck	Yes	Total weight	17,547 kg	Load standard column - 2 levels	358.3 kN
Aisle width	6.00 m	Clearance height	2.3 m			Load standard column - 3 levels	537.5 kN
Parking angle	90°	Number of parking levels	4			Load standard column - 4 levels	716.7 kN
						Load standard column - 5 levels	895.8 kN

Description	Profile	Material	Length [mm]	Width [mm]	Height [mm]	Cross-area [mm²]	A [m²]	Volume [mm³]	V [m³]	Density [kg/m³]	Weight [kg]	Weight [kN]	Nr. of elements	Weight structure [kg]	Volume [m³]
Deck	BSP 100 3s 40/20/40	C24	2,500	16,260	100	1,626,000	1.63	4,065,000,000	4.07	500	2,032.5	20.33	4	8,130.0	16.3
Joist	400x60	C24	2,500	60	400	24,000	0.02	60,000,000	0.06	500	30.0	0.30	8	240.0	0.5
Main	240x600-760	BB GL75	16,260	240	600-760	182,400	0.18	2,653,632,000	2.65	800	2,122.9	21.23	4	8,491.6	10.6
Column	240x240	GL24h	3,235	240	240	57,600	0.06	186,336,000	0.19	460	85.7	0.86	8	685.7	1.5

Table 57. Data proof of concept: variant 2a, columns every 1 parking bay, struts on.

Timber Proof-of-concept

ID19	**Variant 2a, 16.26m, struts On, column every 1 parking bay(s)**						
Car Park Characteristics							
Parking bay width	2.5 m	Span parking deck	16.26 m	Floor depth	666 mm	Load standard column - 1 level	169.6 kN
Parking bay length	5.13 m	Column free parking deck	Yes	Total weight	15,242 kg	Load standard column - 2 levels	339.2 kN
Aisle width	6.00 m	Clearance height	2.3 m			Load standard column - 3 levels	508.8 kN
Parking angle	90°	Number of parking levels	4			Load standard column - 4 levels	678.4 kN
						Load standard column - 5 levels	848.0 kN

Description	Profile	Material	Length [mm]	Width [mm]	Height [mm]	Cross-area [mm²]	A [m²]	Volume [mm³]	V [m³]	Density [kg/m³]	Weight [kg]	Weight [kN]	Nr. of elements	Weight structure [kg]	Volume [m³]
Multi-span LVL panel	LVL G 96 2s	C24	2,500	16,260	96	1,560,960	1.56	3,902,400,000	3.90	500	1,951.2	19.51	4	7,804.8	15.6
Transverse main girder	440x140	GL28h	2,500	140	440	61,600	0.06	154,000,000	0.15	460	70.8	0.71	8	566.7	1.2
Column	440x360	GL28h	2,966	360	440	158,400	0.16	469,814,400	0.47	460	216.1	2.16	8	1,728.9	3.8
Main	570x280	GL28h	16,260	280	570	159,600	0.16	2,595,096,000	2.60	460	1,193.7	11.94	4	4,775.0	10.4
Strut	240x140	GL28h	2,966	140	240	33,600	0.03	99,661,395	0.10	460	45.8	0.46	8	366.8	0.8

F.

Additional Calculations & Results Deck System

F.1 Long-span Deck Systems

F.1.1 Example Calculations

Example Calculation Solid CLT Panel

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1x12.18m span		02.02.2023
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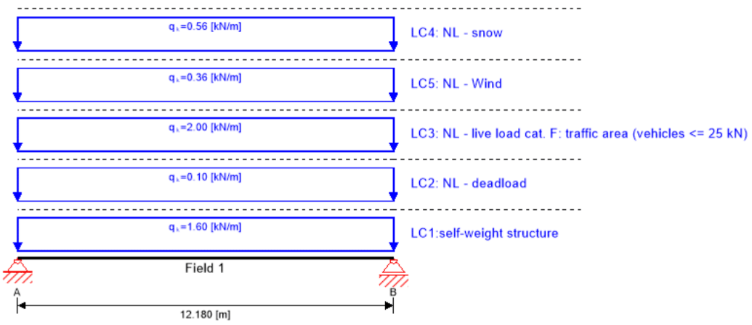
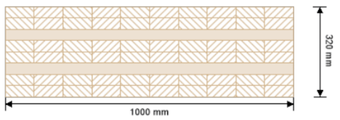
System



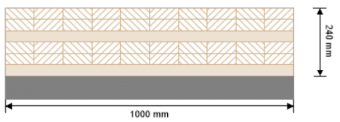
Global utilization ratio									**178 %**
ULS	38 %	ULS Fire	27 %	SLS	142 %	SLS Vibration	178 %	Support	-1 %

Section: CLT 320 L8s - 2



Layer	Thickness	Orientation	Material
1	40.0 mm	0°	C24 spruce ETA (2019)
2	40.0 mm	0°	C24 spruce ETA (2019)
3	40.0 mm	90°	C24 spruce ETA (2019)
4	40.0 mm	0°	C24 spruce ETA (2019)
5	40.0 mm	0°	C24 spruce ETA (2019)
6	40.0 mm	90°	C24 spruce ETA (2019)
7	40.0 mm	0°	C24 spruce ETA (2019)
8	40.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**320.0 mm**		

Section Fire: CLT 320 L8s - 2



Layer	Thickness	Orientation	Material
1	40.0 mm	0°	C24 spruce ETA (2019)
2	40.0 mm	0°	C24 spruce ETA (2019)
3	40.0 mm	90°	C24 spruce ETA (2019)
4	40.0 mm	0°	C24 spruce ETA (2019)
5	40.0 mm	0°	C24 spruce ETA (2019)
6	40.0 mm	90°	C24 spruce ETA (2019)
t_{CLT}	**240.0 mm**		
Time	**90 min**		

Fire resistance class:R 90
Fire protection layering : no additional fire protection

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	71.0	78.0	0.0	0.0

Multi-storey car park_v2

1x12.18m span

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Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	1.60

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00

LC4: NL - snow

continuous load

Field	Load at start [kN/m]
1	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36

Multi-storey car park_v2

1x12.18m span

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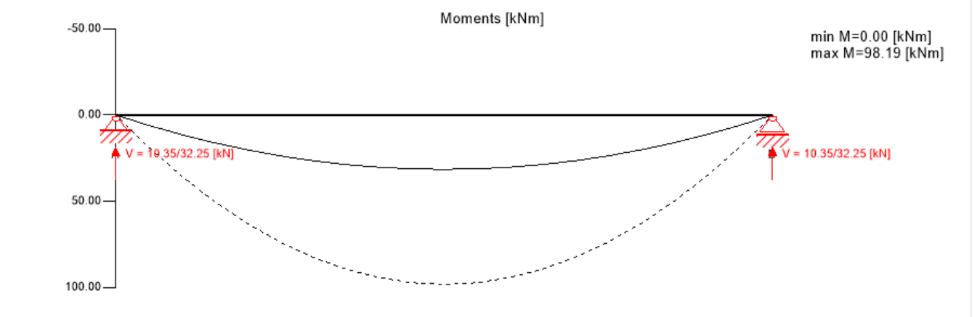
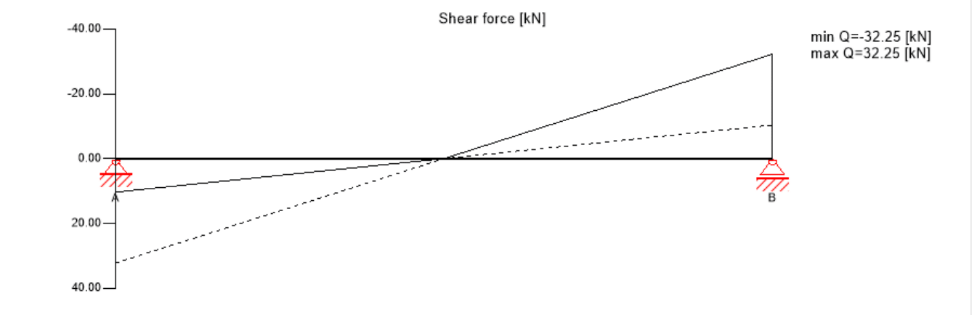
Ultimate limit state (ULS) - design results





ULS Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,y,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$\sigma_{m,y,d}$ [N/mm²]	Ratio	
1	6.09	24.00	1.25	0.80	1.10	16.90	98.19	-6.46	38 %	LCO2

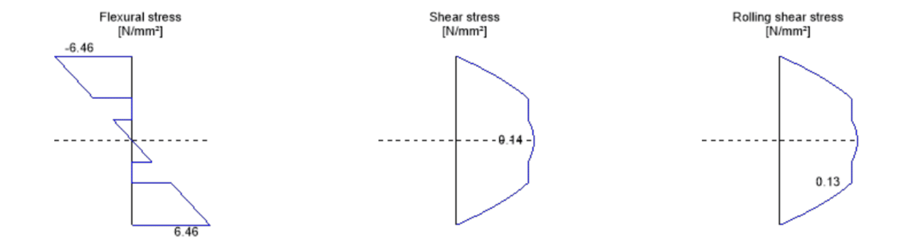
ULS Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	12.18	4.00	1.25	0.80	2.56	-32.25	0.14	5 %	LCO2

ULS Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	12.18	1.05	1.25	0.80	0.67	-32.25	0.13	19 %	LCO2

Stress diagram



Multi-storey car park_v2

1x12.18m span

02.02.2023

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Flexural stress analysis

$M_{y,d}$ =	98.19	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	8.96	N/mm²	
$\sigma_{m,y,d}$ =	-6.46	N/mm²		$f_{m,y,d}$ =	16.90	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓

Utilization ratio 38 %

Shear stress analysis

V_d =	-32.25	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.14	N/mm²	<	$f_{v,d}$ =	2.56	N/mm²	✓

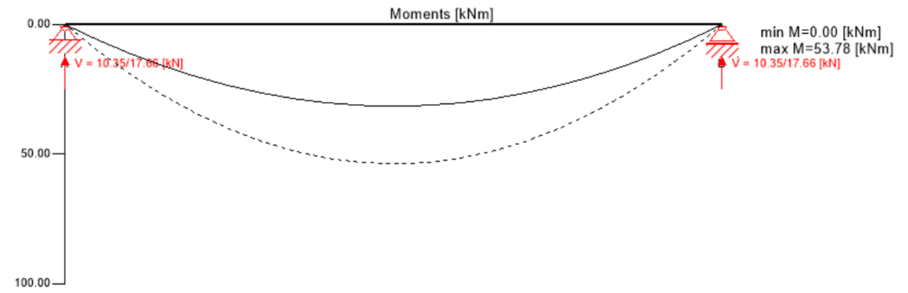
Utilization ratio 5 %

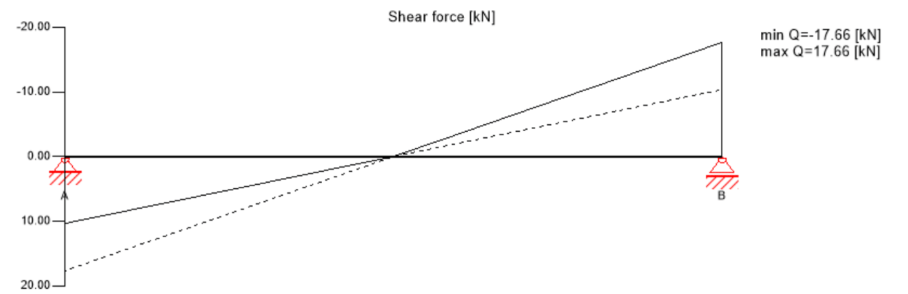
Rolling shear analysis

V_d =	-32.25	kN		$f_{r,k}$ =	1.05	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.13	N/mm²	<	$f_{r,d}$ =	0.67	N/mm²	✓

Utilization ratio 19 %

Ultimate limit state (ULS) fire design - results





Multi-storey car park_v2 | Page 5/8
1x12.18m span | 02.02.2023
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ULS Fire Flexural design

Field	Dist.	$f_{m,k}$	γ_m	k_{mod}	$k_{sys,y}$	k_{fi}	$f_{m,y,d}$	$M_{y,d}$	$\sigma_{m,y,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[-]	[N/mm²]	[kNm]	[N/mm²]		
1	6.09	24.00	1.00	1.00	1.10	1.15	30.36	53.78	-8.13	27 %	LCO12

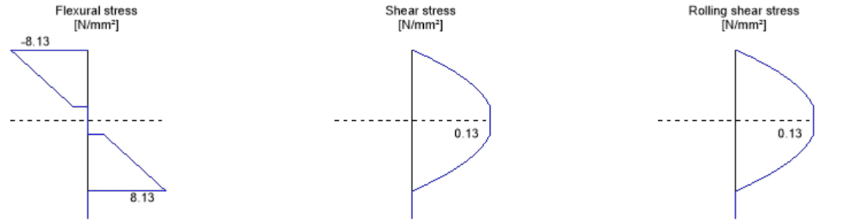
ULS Fire Shear analysis

Field	Dist.	$f_{v,k}$	γ_m	k_{mod}	k_{fi}	$f_{v,d}$	V_d	$\tau_{v,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	12.18	4.00	1.00	1.00	1.15	4.60	-17.66	0.13	3 %	LCO12

ULS Fire Rolling shear

Field	Dist.	$f_{r,k}$	γ_m	k_{mod}	k_{fi}	$f_{r,d}$	V_d	$\tau_{r,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	12.18	1.05	1.00	1.00	1.15	1.21	-17.66	0.13	11 %	LCO12

Stress diagram



Flexural stress analysis Fire

$M_{y,d}$ =	53.78	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
				k_{fi} =	1.15	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	16.10	N/mm²	
$\sigma_{m,y,d}$ =	-8.13	N/mm²		$f_{m,y,d}$ =	30.36	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						27 %	

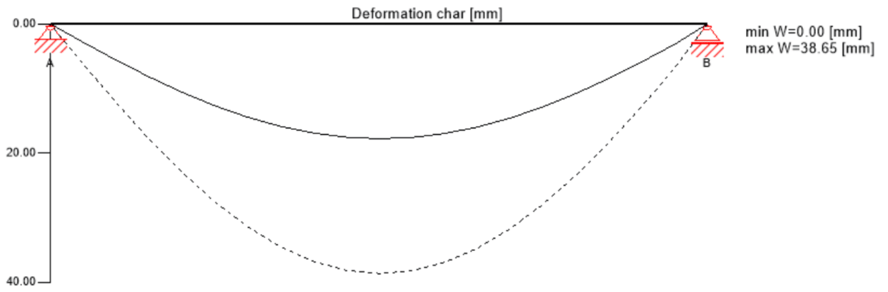
Shear stress analysis Fire

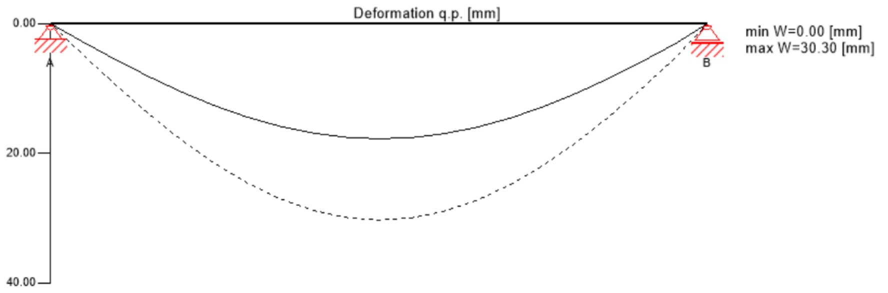
V_d =	-17.66	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{h,v}$ =	0.00	-	
				k_{fi} =	1.15	-	
$\tau_{v,d}$ =	0.13	N/mm²	<	$f_{v,d}$ =	4.60	N/mm²	✓
Utilization ratio						3 %	

Rolling shear analysis Fire

V_d =	-17.66	kN		$f_{r,k}$ =	1.05	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				k_{fi} =	1.15	-	
$\tau_{r,d}$ =	0.13	N/mm²	<	$f_{r,d}$ =	1.21	N/mm²	✓
Utilization ratio						11 %	

Service limit state design (SLS) - design results





w_{inst} = w[char]

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	L/1	12180.0	38.7	0 %

w_{fin} = w[char] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	L/250	48.7	68.9	142 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	L/250	48.7	60.6	124 %

Vibration analysis

General		
Total mass	63.32	[t]
Tributary width	6.6	[m]
Stiffness Longitudinal direction	29184.0	[kNm²]
Stiffness Cross direction	3584.0	[kNm²]
Modal damping	2.0	[%]
α	0.2	[-]
Man weight	700.0	[N]
Modal mass	6910.4	[kg]

Multi-storey car park_v2
1x12.18m span
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Vibration analysis

Analysis				
Criterion	Calc.	Class II	Class II	Cl. II
Frequency criterion min	4.346 [Hz]	4.5 [Hz]	104 %	×
Frequency criterion	4.346 [Hz]	6.0 [Hz]	138 %	
Acceleration criterion	0.178 [m/s²]	0.1 [m/s²]	178 %	×
Stiffness criterion	0.197 [mm]	0.5 [mm]	39 %	✓

Support reaction

Load case category	k_{mod}	A_V	B_V
		[kN]	
self-weight structure	0.6	9.74	9.74
		9.74	9.74
NL - deadload	0.6	0.61	0.61
		0.61	0.61
NL - live load cat. F: traffic area (vehicles <= 25 kN)	0.8	12.18	12.18
		0.00	0.00
NL - snow	0.9	3.41	3.41
		0.00	0.00
NL - Wind	0.9	2.19	2.19
		0.00	0.00

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
Expertise Rolling shear - no edge gluing, H.J. Blass	Expertise on Rolling shear for CLT
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
Technical expertise 122/2011/02: analysis of load bearing capacity and separation performance of CLT elements	Verification of the load bearing capacity and the insulation criterion of CLT structures with Stora Enso CLT
Technical expertise 2434/2012 - BB: failure time tf of gypsum fire boards (GKF) according to ON B 3410	Expertise on failure time tf of gypsum wall fire boards according to ON B3410 and gypsum wall boards type DF according to EN 520
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General-Common rules and rules for buildings
ÖNorm B 1995-1-2 NA	ÖNORM EN 1995-1-2 - Austria - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guideline for Europe; publishes by SP Technical Research Institute of Sweden
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
Expertise Rolling shear, H.J. Blass	Expertise on rolling shear strength and rolling shear modulus of CLT panels
ÖNORM EN 1995-1-1_NA, chapter 7.3	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General-Common rules and rules for buildings; chapter 7.3

Disclaimer

Multi-storey car park_v2

12.18m - 4 ribs - 200mm - CLT120

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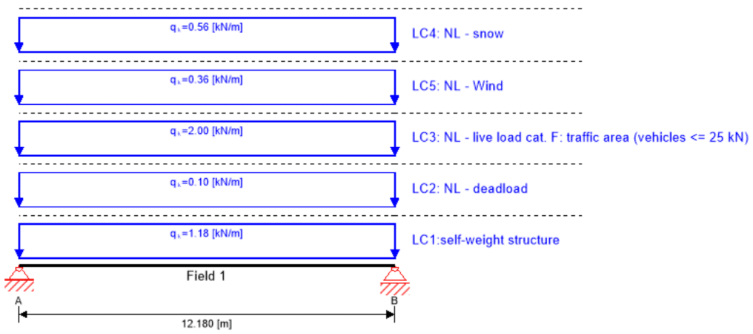
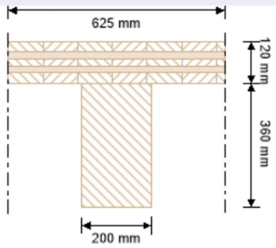
System



Global utilization ratio									100 %
ULS	30 %	ULS Fire	27 %	SLS	68 %	SLS Vibration	76 %	Support	9 %

Section: CLT rib panel by Stora Enso: CLT 120 L5s - 20/36



Ribs width are reduced by 20 mm for the design

Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	30.0	606.0	0°	C24 spruce ETA (2019)
2	20.0	606.0	90°	C24 spruce ETA (2019)
3	20.0	606.0	0°	C24 spruce ETA (2019)
4	20.0	606.0	90°	C24 spruce ETA (2019)
5	30.0	606.0	0°	C24 spruce ETA (2019)
6	360.0	200.0	0°	GL 28h
t_{CLT}	**480.0 mm**			

ULS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	EI,netto [kNm²]
1	Start	0	0.625	51.70	1,937,479,000	93,568	61097.93	0.5527	23249.75
1	Center	0.625	11.555	60.60	2,075,708,000	99,264	65170.56	0.5197	24908.5
1	End	11.555	12.18	51.70	1,937,479,000	93,568	61097.93	0.5527	23249.75

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	9,600	552
2	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	40
3	20.0 mm	606.0 mm	L	C24 spruce ETA (2019)	9,600	552
4	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	40
5	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	9,600	552
6	360.0 mm	200.0 mm	L	GL 28h	10,080	520

ULS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0.625	51.70	1,530,403,000	74,227	48391.95	0.5527	18364.84
1	Center	0.625	11.555	60.60	1,640,238,000	78,608	51524.75	0.5197	19682.86
1	End	11.555	12.18	51.70	1,530,403,000	74,227	48391.95	0.5527	18364.84

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	7,385	425
2	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	31
3	20.0 mm	606.0 mm	L	C24 spruce ETA (2019)	7,385	425
4	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	31
5	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	7,385	425
6	360.0 mm	200.0 mm	L	GL 28h	8,129	419

SLS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	51.70	2,421,849,000	116,960	76372.41	0.5527	29062.19
1	Center	0	12.18	60.60	2,594,635,000	124,080	81463.2	0.5197	31135.62
1	End	12.18	12.18	51.70	2,421,849,000	116,960	76372.41	0.5527	29062.19

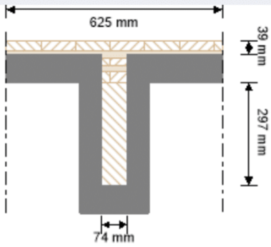
Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	50
3	20.0 mm	606.0 mm	L	C24 spruce ETA (2019)	12,000	690
4	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	50
5	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	12,000	690
6	360.0 mm	200.0 mm	L	GL 28h	12,600	650

SLS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	51.70	2,743,166,000	135,860	88072.41	0.5527	32917.99
1	Center	0	12.18	60.60	2,942,978,000	142,980	93163.2	0.5197	35315.73
1	End	12.18	12.18	51.70	2,743,166,000	135,860	88072.41	0.5527	32917.99

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	50
3	20.0 mm	606.0 mm	L	C24 spruce ETA (2019)	12,000	690
4	20.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	50
5	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	12,000	690
6	360.0 mm	200.0 mm	L	GL 28h	15,750	813

Section Fire: CLT rib panel by Stora Enso: CLT 120 L5s - 20/36



Ribs width are reduced by 20 mm for the design

Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	30.0	606.0	0°	C24 spruce ETA (2019)
2	9.0	606.0	90°	C24 spruce ETA (2019)
3	11.0	74.0	90°	C24 spruce ETA (2019)
4	20.0	74.0	0°	C24 spruce ETA (2019)
5	20.0	74.0	90°	C24 spruce ETA (2019)
6	30.0	74.0	0°	C24 spruce ETA (2019)
7	297.0	74.0	0°	GL 28h

Section Fire: CLT rib panel by Stora Enso: CLT 120 L5s - 20/36

Fire resistance class: enter minutes
Fire protection layering : no additional fire protection

t_{CLT} **417.0 mm**
Time **80 min**

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$
[-]	[mm]	[mm]	[mm]
0	0	0.0	0.0

Field	Range	from	Until	Width	Moment of inertia	Area	G*A	Kappa	Ely,netto
		[m]	[m]	[cm]	[mm⁴]	[mm²]	[kN]		[kNm²]
1	Start	0	0.625	51.70	790,286,500	42,287	27887.95	0.2137	9483.438
1	Center	0.625	11.555	60.60	843,028,100	44,957	29770.3	0.1408	10116.34
1	End	11.555	12.18	51.70	790,286,500	42,287	27887.95	0.2137	9483.438

Layer	Thickness	Width	Type	Material	E	G
	[mm]	[mm]			[N/mm²]	[N/mm²]
1	30.0 mm	606.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	9.0 mm	606.0 mm	C	C24 spruce ETA (2019)	0	50
3	11.0 mm	74.0 mm	C	C24 spruce ETA (2019)	0	50
4	20.0 mm	74.0 mm	L	C24 spruce ETA (2019)	12,000	690
5	20.0 mm	74.0 mm	C	C24 spruce ETA (2019)	0	50
6	30.0 mm	74.0 mm	L	C24 spruce ETA (2019)	12,000	690
7	297.0 mm	74.0 mm	L	GL 28h	12,600	650

Material values

Material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$G_{r,mean}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00
GL 28h	28.00	22.30	0.50	28.00	2.50	2.50	1.20	12,600.00	650.00	65.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start
	[kN/m]
1	1.18

LC2: NL - deadload

continuous load

Field	Load at start
	[kN/m]
1	0.10

Multi-storey car park_v2
12.18m - 4 ribs - 200mm - CLT120

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LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load	
Field	Load at start [kN/m]
1	2.00

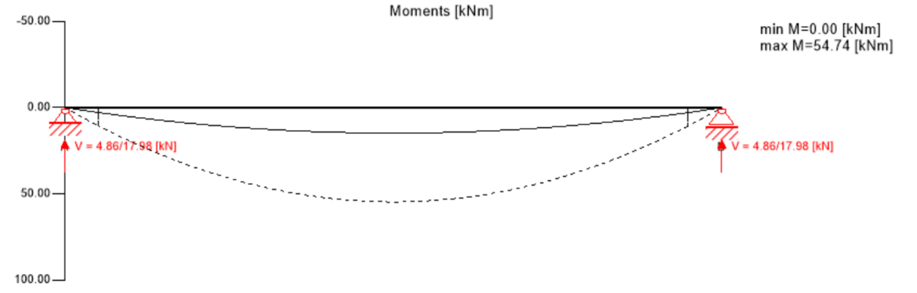
LC4: NL - snow

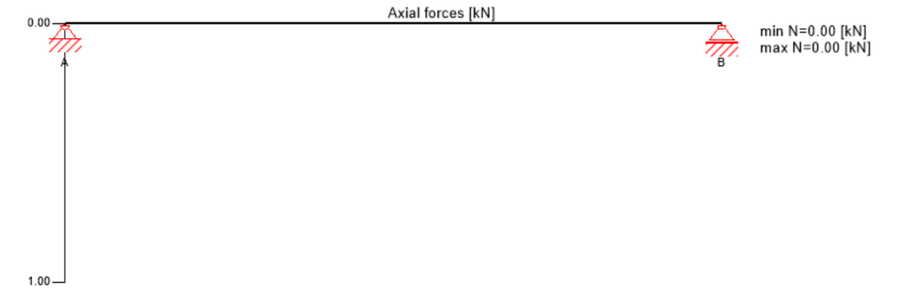
continuous load	
Field	Load at start [kN/m]
1	0.56

LC5: NL - Wind

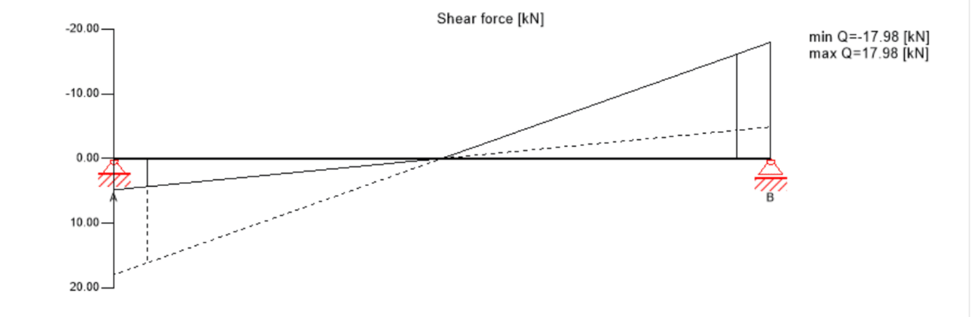
continuous load	
Field	Load at start [kN/m]
1	0.36

Ultimate limit state (ULS) - design results T=0





Ultimate limit state (ULS) - design results T=0



ULS Flexural design T=0

Field	Dist.	γ_m	k_{mod}	$k_{sys,y}$	$k_{h,m,y}$	$k_{h,m,z}$	$f_{m,k}$	$f_{m,y,d}$	$f_{t,0,d}$
	[m]	[-]	[-]	[-]	[-]	[-]	[N/mm²]	[N/mm²]	[N/mm²]
1	5.47	1.25	0.80	1.10	1.05	1.10	28.00	20.75	15.70
Field	$f_{c,0,d}$	$M_{y,d}$	$N_{c,d}$	$N_{t,d}$	$\sigma_{m,y,d}$	$\sigma_{c,d}$	$\sigma_{t,d}$	Ratio	
	[N/mm²]	[kNm]	[kN]	[kN]	[N/mm²]	[N/mm²]	[N/mm²]		
1	17.92	54.74	0.00	0.00	6.06	0.00	0.00	29 %	LCO2

ULS Axial force design T=0

Field	Dist.	$f_{t,0,k}$	$f_{c,0,k}$	γ_m	k_{mod}	$k_{sys,y}$	$k_{h,m,y}$	$k_{h,m,z}$	k_l	$f_{m,y,d}$
	[m]	[N/mm²]	[N/mm²]	[-]	[-]	[-]	[-]	[-]	[-]	[N/mm²]
1	5.47	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90
Field	Dist.	$M_{y,d}$	$f_{c,0,d}$	$N_{c,d}$	$f_{t,0,d}$	$N_{t,d}$	$\sigma_{c,d}$	$\sigma_{t,d}$	Utilization	
	[m]	[kNm]	[N/mm²]	[kN]	[N/mm²]	[kN]	[N/mm²]	[N/mm²]		
1	5.47	54.74	13.44	0.00	8.96	0.00	-4.03	0.00	30 %	LCO2

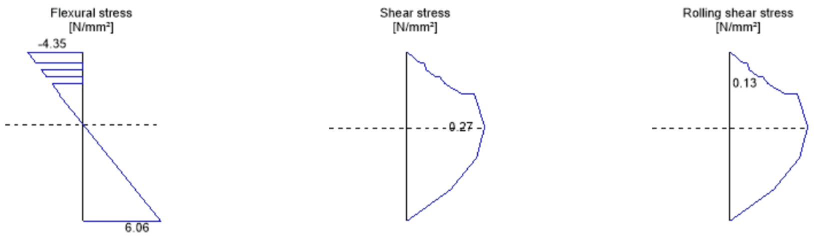
ULS Shear analysis T=0

Field	Dist.	$f_{v,k}$	γ_m	k_{mod}	k_{cr}	$f_{v,d}$	V_d	$\tau_{v,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	0.0	2.50	1.25	0.80	1.00	1.60	17.98	0.27	17 %	LCO2

ULS Rolling shear T=0

Field	Dist.	$f_{r,k}$	γ_m	k_{mod}	$f_{r,d}$	V_d	$\tau_{r,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	0.0	1.25	1.25	0.80	0.80	17.98	0.13	17 %	LCO2

Stress diagram T=0



Multi-storey car park_v2
12.18m - 4 ribs - 200mm - CLT120

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Flexural stress analysis T=0

$M_{y,d}$ =	54.74	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	28.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.05	-	
				$k_{h,m,z}$ =	1.10	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	15.70	N/mm²	
$\sigma_{m,y,d}$ =	6.06	N/mm²		$f_{m,y,d}$ =	20.75	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						**29 %**	

Shear stress analysis T=0

V_d =	17.98	kN		$f_{v,k}$ =	2.50	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.27	N/mm²	<	$f_{v,d}$ =	1.60	N/mm²	✓
Utilization ratio						**17 %**	

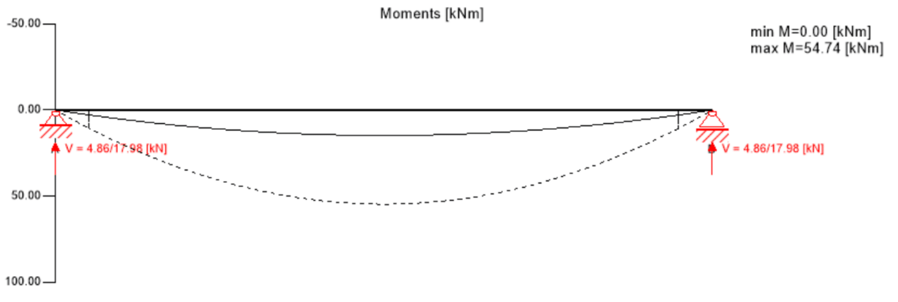
Rolling shear analysis T=0

V_d =	17.98	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.13	N/mm²	<	$f_{r,d}$ =	0.80	N/mm²	✓
Utilization ratio						**17 %**	

Buckling analysis T=0

$M_{y,d}$ =	54.74	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	17.92	N/mm²	
$\sigma_{m,y,d}$ =	4.27	N/mm²		$f_{m,y,d}$ =	17.92	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						**24 %**	

Ultimate limit state (ULS) - design results T=∞



Multi-storey car park_v2

12.18m - 4 ribs - 200mm - CLT120

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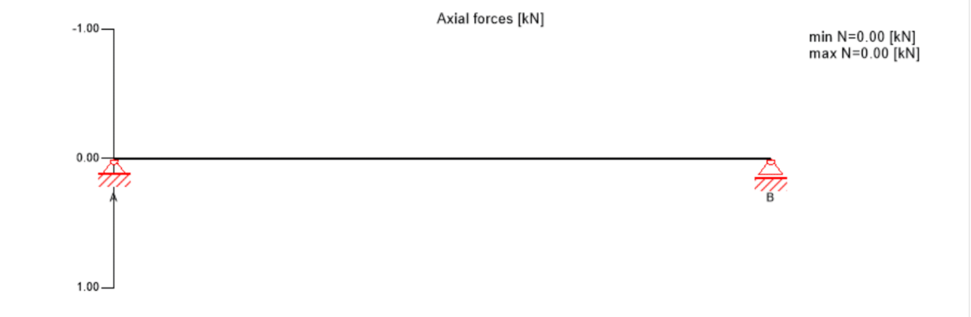
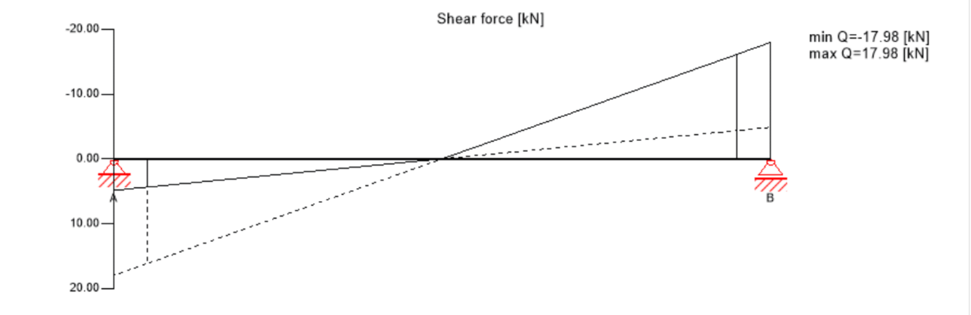
Ultimate limit state (ULS) - design results T=∞





ULS Flexural design T=∞

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]
1	5.47	1.25	0.80	1.10	1.05	1.10	28.00	20.75	15.70
Field	$f_{c,0,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	17.92	54.74	0.00	0.00	6.13	0.00	0.00	30 %	LCO30

ULS Axial force design T=∞

Field	Dist. [m]	$f_{t,0,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]
1	5.47	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90
Field	Dist. [m]	$M_{y,d}$ [kNm]	$f_{c,0,d}$ [N/mm²]	$N_{c,d}$ [kN]	$f_{t,0,d}$ [N/mm²]	$N_{t,d}$ [kN]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Utilization	
1	5.47	54.74	13.44	0.00	8.96	0.00	-3.98	0.00	30 %	LCO30

ULS Shear analysis T=∞

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	2.50	1.25	0.80	1.00	1.60	17.98	0.27	17 %	LCO30

ULS Rolling shear T=∞

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.25	1.25	0.80	0.80	17.98	0.13	17 %	LCO30

Multi-storey car park_v2

12.18m - 4 ribs - 200mm - CLT120

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Stress diagram T=∞

Flexural stress analysis T=∞

$M_{y,d}$ =	54.74	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	28.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.05	-	
				$k_{h,m,z}$ =	1.10	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	15.70	N/mm²	
$\sigma_{m,y,d}$ =	6.13	N/mm²		$f_{m,y,d}$ =	20.75	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						30 %	

Shear stress analysis T=∞

V_d =	17.98	kN		$f_{v,k}$ =	2.50	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.27	N/mm²	<	$f_{v,d}$ =	1.60	N/mm²	✓
Utilization ratio						17 %	

Rolling shear analysis T=∞

V_d =	17.98	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.13	N/mm²	<	$f_{r,d}$ =	0.80	N/mm²	✓
Utilization ratio						17 %	

Buckling analysis T=∞

$M_{y,d}$ =	54.74	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	17.92	N/mm²	
$\sigma_{m,y,d}$ =	4.36	N/mm²		$f_{m,y,d}$ =	17.92	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						24 %	

Multi-storey car park_v2

12.18m - 4 ribs - 200mm - CLT120

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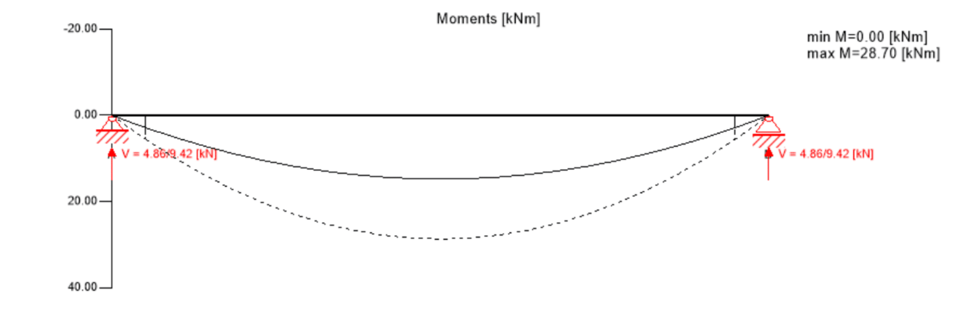
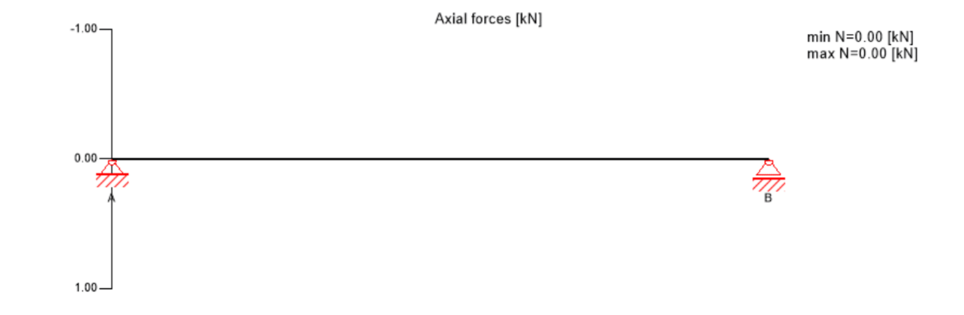
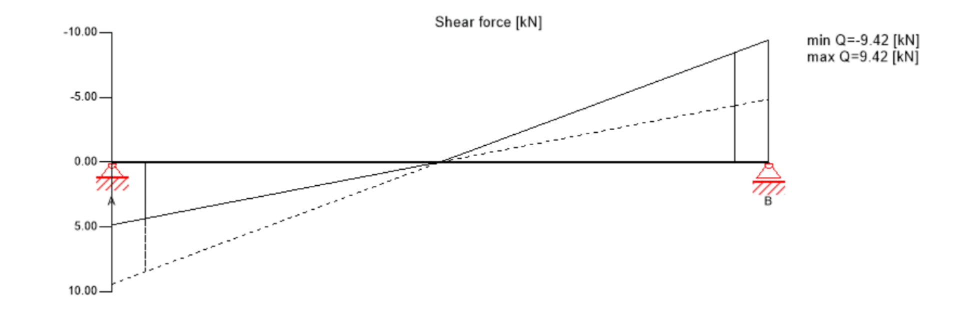
Ultimate limit state (ULS) fire design - results







ULS Fire Flexural design

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	k_{fi} [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	5.47	1.25	0.80	1.10	1.00	28.00	20.75	15.70	17.92

Field	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio
1	54.74	0.00	0.00	6.06	0.00	0.00	29 % LCO2

ULS Fire Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio
1	0.0	2.50	1.00	1.00	1.15	1.00	2.88	9.42	0.41	14 % LCO12

Multi-storey car park_v2

12.18m - 4 ribs - 200mm - CLT120

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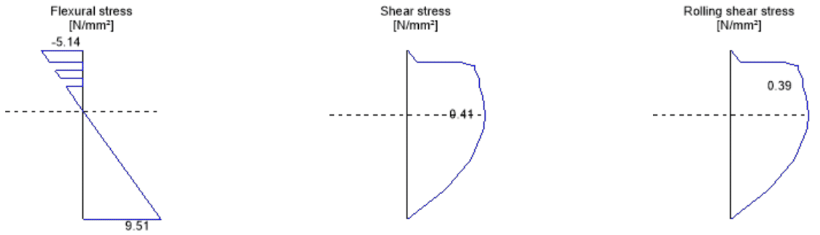
ULS Fire Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.25	1.00	1.00	1.15	1.44	9.42	0.39	27 %	LCO12

Stress diagram



Flexural stress analysis Fire

$M_{y,d}$ =	28.70	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	28.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.07	-	
				$k_{h,m,z}$ =	1.10	-	
				k_l =	1.00	-	
				k_{fi} =	1.15	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	28.21	N/mm²	
$\sigma_{m,y,d}$ =	9.51	N/mm²		$f_{m,y,d}$ =	38.00	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**25 %**

Shear stress analysis Fire

V_d =	9.42	kN		$f_{v,k}$ =	2.50	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{h,v}$ =	0.00	-	
				k_{fi} =	1.15	-	
$\tau_{v,d}$ =	0.41	N/mm²	<	$f_{v,d}$ =	2.88	N/mm²	✓
Utilization ratio							**14 %**

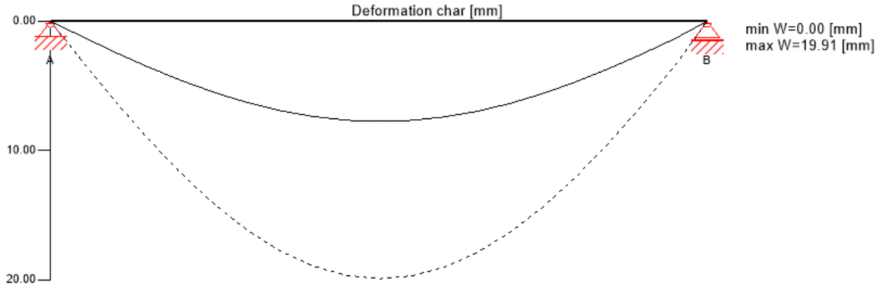
Rolling shear analysis Fire

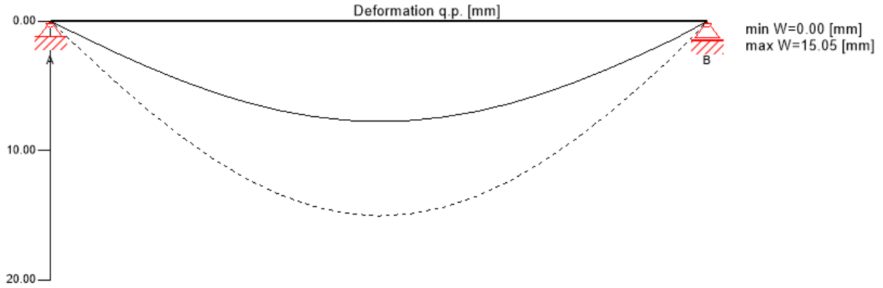
V_d =	9.42	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				k_{fi} =	1.15	-	
$\tau_{r,d}$ =	0.39	N/mm²	<	$f_{r,d}$ =	1.44	N/mm²	✓
Utilization ratio							**27 %**

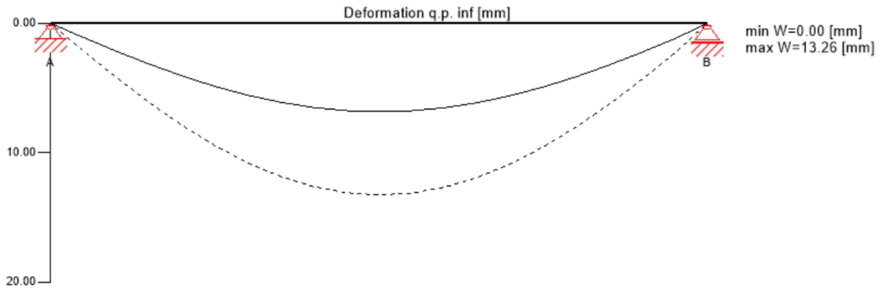
Buckling analysis

$M_{y,d}$ =	28.70	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	32.20	N/mm²	
$\sigma_{m,y,d}$ =	5.66	N/mm²		$f_{m,y,d}$ =	32.20	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**18 %**

Service limit state design (SLS) - design results







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12.18m - 4 ribs - 200mm - CLT120

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w_{inst} = w[char]

Field	$K_{def,top}$	$K_{def,Rib}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	L/1	12180.0	19.9	0 %

w_{fin} = w[char] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	L/250	48.7	33.2	68 %

$w_{net,fin}$ = w[q.p.] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	L/250	48.7	28.3	58 %

Vibration analysis

General		
Total mass	47.53	[t]
Tributary width	3.2	[m]
Stiffness Longitudinal direction	31135.6	[kNm²]
Stiffness Cross direction	208.0	[kNm²]
Modal damping	2.0	[%]
α	0.1	[-]
Man weight	700.0	[N]
Modal mass	2505.0	[kg]

Analysis				
Criterion	Calc.	Class II	Class II	Cl. II
Frequency criterion min	6.554 [Hz]	4.5 [Hz]	69 %	✓
Frequency criterion	6.554 [Hz]	6.0 [Hz]	92 %	
Acceleration criterion	0.203 [m/s²]	0.1 [m/s²]	203 %	
Stiffness criterion	0.382 [mm]	0.5 [mm]	76 %	✓

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V [kN]
self-weight structure	0.6	4.48	4.48
		4.48	4.48
NL - deadload	0.6	0.38	0.38
		0.38	0.38
NL - live load cat. F: traffic area (vehicles <= 25 kN)	0.8	7.61	7.61
		0.00	0.00
NL - snow	0.9	2.13	2.13
		0.00	0.00
NL - Wind	0.9	1.37	1.37
		0.00	0.00

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
Expertise Rolling shear - no edge gluing, H.J. Blass	Expertise on Rolling shear for CLT
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
EN 14080	EN 14080 - Timber Structures - Glued laminated timber and glued solid timber - Requirements
Technical expertise 122/2011/02: analysis of load bearing capacity and separation performance of CLT elements	Verification of the load bearing capacity and the insulation criterion of CLT structures with Stora Enso CLT
Technical expertise 2434/2012 - BB: failure time tf of gypsum fire boards (GKF) according to ON B 3410	Expertise on failure time tf of gypsum wall fire boards according to ON B3410 and gypsum wall boards type DF according to EN 520

Example Calculation CLT Closed Rib Panel

Multi-storey car park_v2
Closed 12.18m - 4 ribs - 200mm - CLT120 copy
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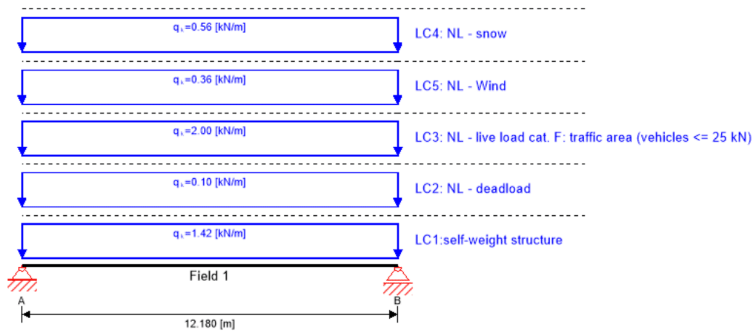
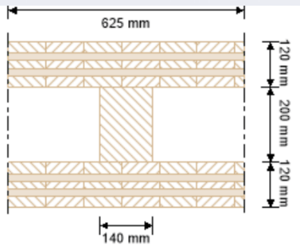
System



Global utilization ratio									76 %
ULS	28 %	ULS Fire	15 %	SLS	76 %	SLS Vibration	73 %	Support	-1 %

Section: CLT rib panel by Stora Enso: CLT 120 L5s - 14/20 - CLT 120 L5s



Ribs width are reduced by 20 mm for the design

Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	30.0	607.0	0°	C24 spruce ETA (2019)
2	20.0	607.0	90°	C24 spruce ETA (2019)
3	20.0	607.0	0°	C24 spruce ETA (2019)
4	20.0	607.0	90°	C24 spruce ETA (2019)
5	30.0	607.0	0°	C24 spruce ETA (2019)
6	200.0	140.0	0°	GL 28h
7	30.0	607.0	0°	C24 spruce ETA (2019)
8	20.0	607.0	90°	C24 spruce ETA (2019)
9	20.0	607.0	0°	C24 spruce ETA (2019)
10	20.0	607.0	90°	C24 spruce ETA (2019)
11	30.0	607.0	0°	C24 spruce ETA (2019)
t_{CLT}	**440.0 mm**			

ULS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	EI,netto [kNm²]
1	Start	0	0.625	48.10	1,752,024,000	85,088	58581.12	0.3739	21024.29
1	Center	0.625	11.555	60.70	2,190,437,000	101,216	70112.65	0.2926	26285.24
1	End	11.555	12.18	48.10	1,752,024,000	85,088	58581.12	0.3739	21024.29

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	9,600	552
2	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	40
3	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	9,600	552
4	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	40

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Closed 12.18m - 4 ribs - 200mm - CLT120 copy 02.02.2023

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Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
5	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	9,600	552
6	200.0 mm	140.0 mm	L	GL 28h	10,080	520
7	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	9,600	552
8	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	40
9	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	9,600	552
10	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	40
11	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	9,600	552

ULS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0.625	48.10	1,350,629,000	66,328	45604.34	0.3739	16207.55
1	Center	0.625	11.555	60.70	1,687,870,000	78,734	54474.74	0.2926	20254.43
1	End	11.555	12.18	48.10	1,350,629,000	66,328	45604.34	0.3739	16207.55

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	7,385	425
2	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	31
3	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	7,385	425
4	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	31
5	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	7,385	425
6	200.0 mm	140.0 mm	L	GL 28h	8,129	419
7	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	7,385	425
8	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	31
9	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	7,385	425
10	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	31
11	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	7,385	425

SLS T=0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	48.10	2,190,030,000	106,360	73226.4	0.3739	26280.36
1	Center	0	12.18	60.70	2,738,046,000	126,520	87640.8	0.2926	32856.55
1	End	12.18	12.18	48.10	2,190,030,000	106,360	73226.4	0.3739	26280.36

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
3	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
4	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
5	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
6	200.0 mm	140.0 mm	L	GL 28h	12,600	650
7	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
8	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
9	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
10	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
11	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690

SLS T=∞

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0	48.10	2,214,530,000	113,710	77776.4	0.3739	26574.36
1	Center	0	12.18	60.70	2,762,546,000	133,870	92190.8	0.2926	33150.55
1	End	12.18	12.18	48.10	2,214,530,000	113,710	77776.4	0.3739	26574.36

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Closed 12.18m - 4 ribs - 200mm - CLT120 copy — 02.02.2023

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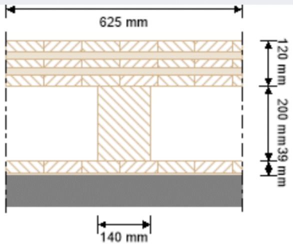
Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
3	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
4	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
5	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
6	200.0 mm	140.0 mm	L	GL 28h	15,750	813
7	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
8	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
9	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
10	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
11	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690

Section Fire: CLT rib panel by Stora Enso: CLT 120 L5s - 14/20 - CLT 120 L5s



Ribs width are reduced by 20 mm for the design

Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	30.0	607.0	0°	C24 spruce ETA (2019)
2	20.0	607.0	90°	C24 spruce ETA (2019)
3	20.0	607.0	0°	C24 spruce ETA (2019)
4	20.0	607.0	90°	C24 spruce ETA (2019)
5	30.0	607.0	0°	C24 spruce ETA (2019)
6	200.0	140.0	0°	GL 28h
7	30.0	607.0	0°	C24 spruce ETA (2019)
8	9.0	607.0	90°	C24 spruce ETA (2019)
9	11.0	140.0	90°	C24 spruce ETA (2019)
t_{CLT}	**370.0 mm**			
Time	**80 min**			

Fire resistance class: enter minutes
Fire protection layering : no additional fire protection

k_0 [-]	d_0 [mm]	$d_{char,0,h}$ [mm]	$d_{ef,h}$ [mm]
0	0	0.0	0.0

Field	Range	from [m]	Until [m]	Width [cm]	Moment of inertia [mm⁴]	Area [mm²]	G*A [kN]	Kappa	Ely,netto [kNm²]
1	Start	0	0.625	48.10	1,090,202,000	82,310	55963.35	0.5368	13082.43
1	Center	0.625	11.555	60.70	1,325,280,000	96,170	65835.45	0.4843	15903.36
1	End	11.555	12.18	48.10	1,090,202,000	82,310	55963.35	0.5368	13082.43

Layer	Thickness [mm]	Width [mm]	Type	Material	E [N/mm²]	G [N/mm²]
1	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
2	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
3	20.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
4	20.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
5	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
6	200.0 mm	140.0 mm	L	GL 28h	12,600	650
7	30.0 mm	607.0 mm	L	C24 spruce ETA (2019)	12,000	690
8	9.0 mm	607.0 mm	C	C24 spruce ETA (2019)	0	50
9	11.0 mm	140.0 mm	C	C24 spruce ETA (2019)	0	50

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\,min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00
GL 28h	28.00	22.30	0.50	28.00	2.50	2.50	1.20	12,600.00	650.00	65.00

Load

Multi-storey car park_v2
Closed 12.18m - 4 ribs - 200mm - CLT120 copy

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Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	1.42

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00

LC4: NL - snow

continuous load

Field	Load at start [kN/m]
1	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36

Multi-storey car park_v2 Page 5/14

Closed 12.18m - 4 ribs - 200mm - CLT120 copy 02.02.2023

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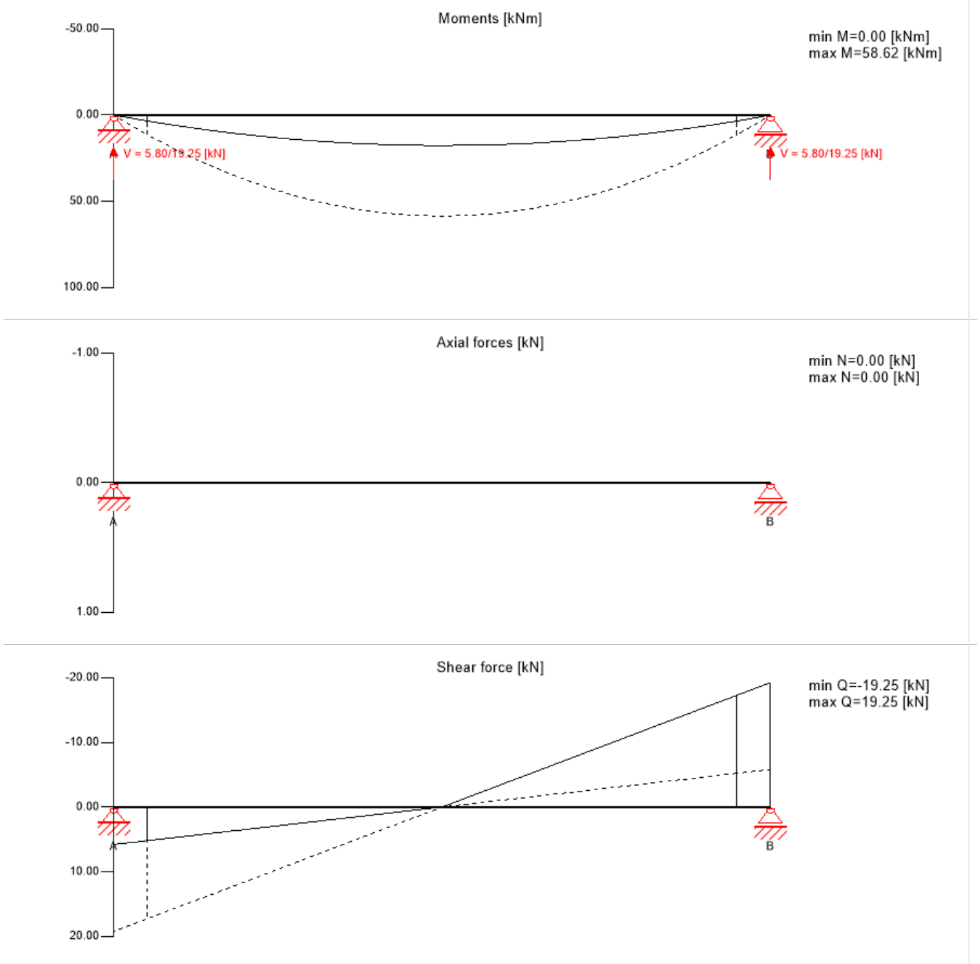
Ultimate limit state (ULS) - design results T=0



ULS Flexural design T=0

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]
1	5.47	1.25	0.80	1.10	1.00	1.00	24.00	16.90	8.96
Field	**$f_{c,0,d}$ [N/mm²]**	**$M_{y,d}$ [kNm]**	**$N_{c,d}$ [kN]**	**$N_{t,d}$ [kN]**	**$\sigma_{m,y,d}$ [N/mm²]**	**$\sigma_{c,d}$ [N/mm²]**	**$\sigma_{t,d}$ [N/mm²]**	**Ratio**	
1	13.44	58.62	0.00	0.00	4.71	0.00	0.00	28 %	LCO2

ULS Axial force design T=0

Field	Dist. [m]	$f_{t,0,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]
1	5.47	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90
Field	**Dist. [m]**	**$M_{y,d}$ [kNm]**	**$f_{c,0,d}$ [N/mm²]**	**$N_{c,d}$ [kN]**	**$f_{t,0,d}$ [N/mm²]**	**$N_{t,d}$ [kN]**	**$\sigma_{c,d}$ [N/mm²]**	**$\sigma_{t,d}$ [N/mm²]**	**Utilization**	
1	5.47	58.62	13.44	0.00	8.96	0.00	0.00	4.39	49 %	LCO2

Multi-storey car park_v2
Closed 12.18m - 4 ribs - 200mm - CLT120 copy
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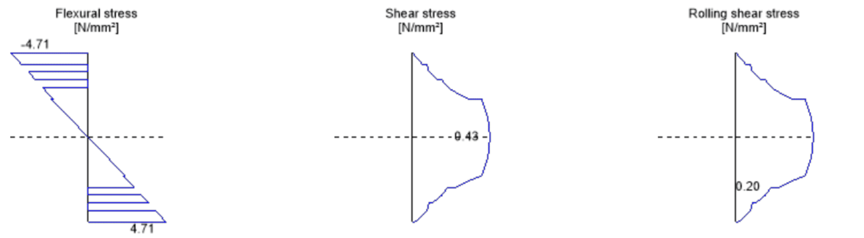
ULS Shear analysis T=0

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	2.50	1.25	0.80	1.00	1.60	19.25	0.43	27 %	LCO2

ULS Rolling shear T=0

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	0.0	1.25	1.25	0.80	0.80	19.25	0.20	25 %	LCO2

Stress diagram T=0



Flexural stress analysis T=0

$M_{y,d}$ =	58.62	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	8.96	N/mm²	
$\sigma_{m,y,d}$ =	4.71	N/mm²		$f_{m,y,d}$ =	16.90	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**28 %**

Shear stress analysis T=0

V_d =	19.25	kN		$f_{v,k}$ =	2.50	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.43	N/mm²	<	$f_{v,d}$ =	1.60	N/mm²	✓
Utilization ratio							**27 %**

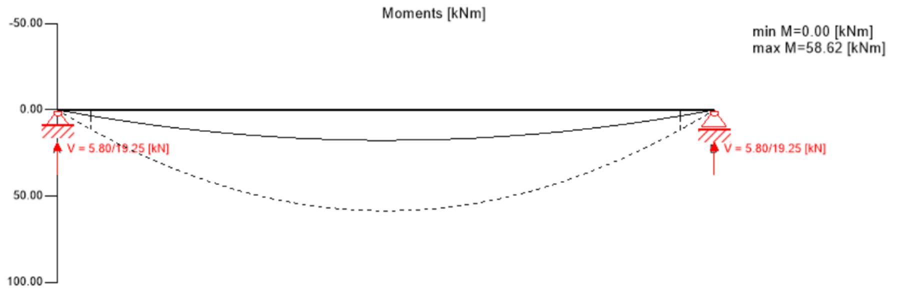
Rolling shear analysis T=0

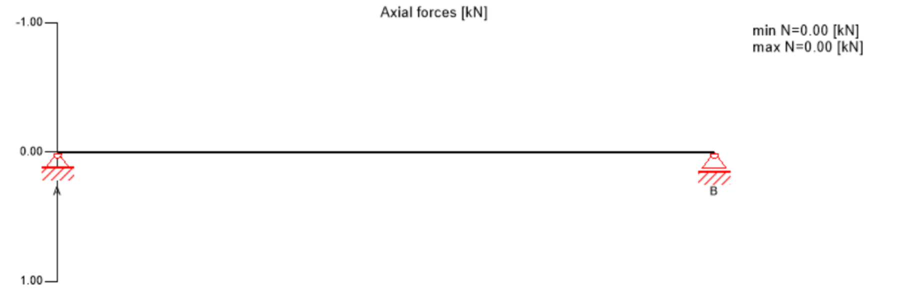
V_d =	19.25	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.20	N/mm²	<	$f_{r,d}$ =	0.80	N/mm²	✓
Utilization ratio							**25 %**

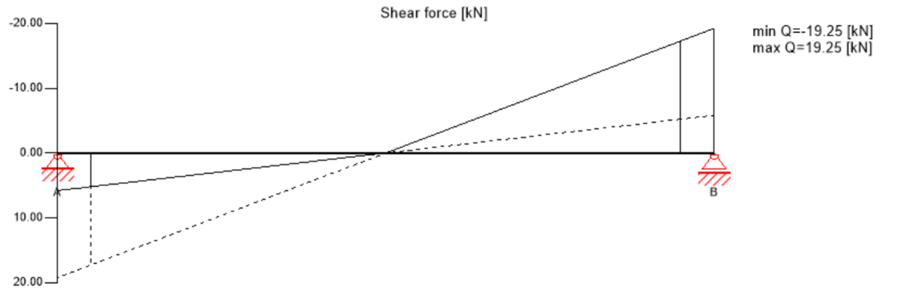
Buckling analysis T=0

$M_{y,d}$ =	58.62	kNm		$f_{m,k}$ =	28.00	N/mm²
$M_{z,d}$ =	0.00	kNm				
$N_{c,d}$ =	0.00	kN		γ_m =	1.25	-
				k_{mod} =	0.80	-
				$k_{sys,y}$ =	1.00	-
				$k_{sys,z}$ =	1.00	-
				$k_{h,m,y}$ =	1.00	-
				$k_{h,m,z}$ =	1.00	-
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	17.92	N/mm²
$\sigma_{m,y,d}$ =	2.81	N/mm²		$f_{m,y,d}$ =	17.92	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm² ✓
Utilization ratio						**16 %**

Ultimate limit state (ULS) - design results T=∞







ULS Flexural design T=∞

Field	Dist.	γ_m	k_{mod}	$k_{sys,y}$	$k_{h,m,y}$	$k_{h,m,z}$	$f_{m,k}$	$f_{m,y,d}$	$f_{t,0,d}$
	[m]	[-]	[-]	[-]	[-]	[-]	[N/mm²]	[N/mm²]	[N/mm²]
1	5.47	1.25	0.80	1.10	1.00	1.00	24.00	16.90	8.96
Field	**$f_{c,0,d}$**	**$M_{y,d}$**	**$N_{c,d}$**	**$N_{t,d}$**	**$\sigma_{m,y,d}$**	**$\sigma_{c,d}$**	**$\sigma_{t,d}$**	**Ratio**	
	[N/mm²]	[kNm]	[kN]	[kN]	[N/mm²]	[N/mm²]	[N/mm²]		
1	13.44	58.62	0.00	0.00	-4.70	0.00	0.00	28 %	LCO30

ULS Axial force design T=∞

Field	Dist.	$f_{t,0,k}$	$f_{c,0,k}$	γ_m	k_{mod}	$k_{sys,y}$	$k_{h,m,y}$	$k_{h,m,z}$	k_l	$f_{m,y,d}$
	[m]	[N/mm²]	[N/mm²]	[-]	[-]	[-]	[-]	[-]	[-]	[N/mm²]
1	5.47	14.00	21.00	1.25	0.80	1.10	1.00	1.00	1.00	16.90
Field	**Dist.**	**$M_{y,d}$**	**$f_{c,0,d}$**	**$N_{c,d}$**	**$f_{t,0,d}$**	**$N_{t,d}$**	**$\sigma_{c,d}$**	**$\sigma_{t,d}$**	**Utilization**	
	[m]	[kNm]	[N/mm²]	[kN]	[N/mm²]	[kN]	[N/mm²]	[N/mm²]		
1	5.47	58.62	13.44	0.00	8.96	0.00	0.00	4.38	49 %	LCO30

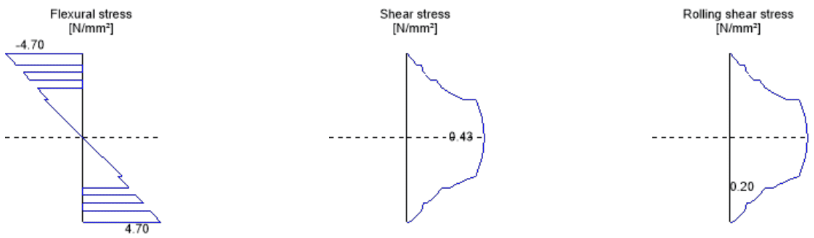
ULS Shear analysis T=∞

Field	Dist.	$f_{v,k}$	γ_m	k_{mod}	k_{cr}	$f_{v,d}$	V_d	$\tau_{v,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	0.0	2.50	1.25	0.80	1.00	1.60	19.25	0.43	27 %	LCO30

ULS Rolling shear T=∞

Field	Dist.	$f_{r,k}$	γ_m	k_{mod}	$f_{r,d}$	V_d	$\tau_{r,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	0.0	1.25	1.25	0.80	0.80	19.25	0.20	25 %	LCO30

Stress diagram T=∞



Flexural stress analysis T=∞

$M_{y,d}$ =	58.62	kNm		$f_{m,k}$ =	24.00	N/mm²		
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²		
$N_{t,d}$ =	0.00	kN		γ_m =	1.25	-		
				k_{mod} =	0.80	-		
				$k_{sys,y}$ =	1.10	-		
				$k_{h,m,y}$ =	1.00	-		
				$k_{h,m,z}$ =	1.00	-		
				k_l =	1.00	-		
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	8.96	N/mm²		
$\sigma_{m,y,d}$ =	-4.70	N/mm²		$f_{m,y,d}$ =	16.90	N/mm²		
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²		✓
Utilization ratio							28 %	

Shear stress analysis T=∞

V_d =	19.25	kN		$f_{v,k}$ =	2.50	N/mm²		
				γ_m =	1.25	-		
				k_{mod} =	0.80	-		
				$k_{h,v}$ =	0.00	-		
$\tau_{v,d}$ =	0.43	N/mm²	<	$f_{v,d}$ =	1.60	N/mm²		✓
Utilization ratio							27 %	

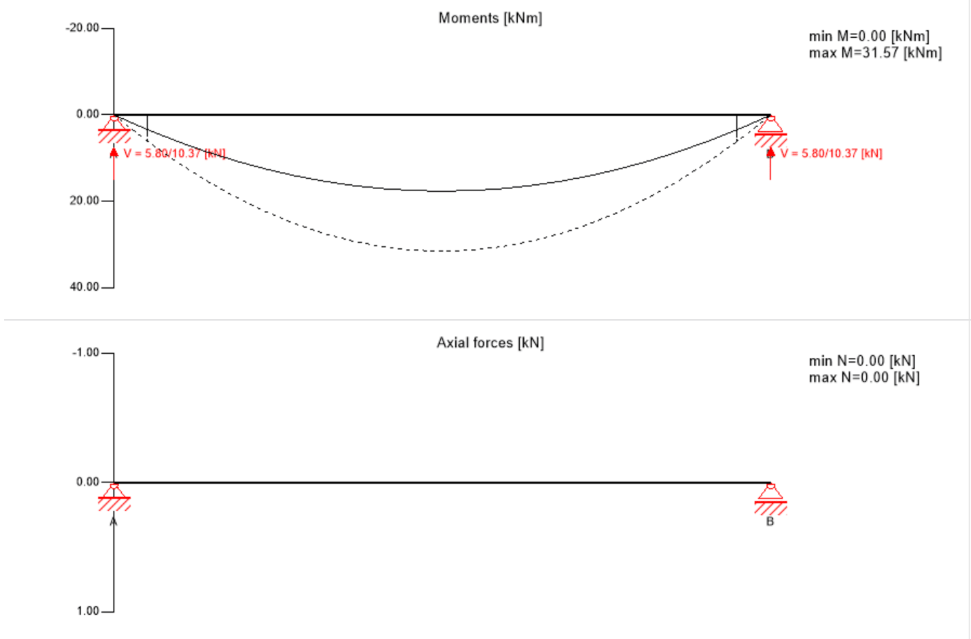
Rolling shear analysis T=∞

V_d =	19.25	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.25	-	
				k_{mod} =	0.80	-	
$\tau_{r,d}$ =	0.20	N/mm²	<	$f_{r,d}$ =	0.80	N/mm²	✓
Utilization ratio							**25 %**

Buckling analysis T=∞

$M_{y,d}$ =	58.62	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.25	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	17.92	N/mm²	
$\sigma_{m,y,d}$ =	2.94	N/mm²		$f_{m,y,d}$ =	17.92	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**16 %**

Ultimate limit state (ULS) fire design - results



Multi-storey car park_v2
Closed 12.18m - 4 ribs - 200mm - CLT120 copy
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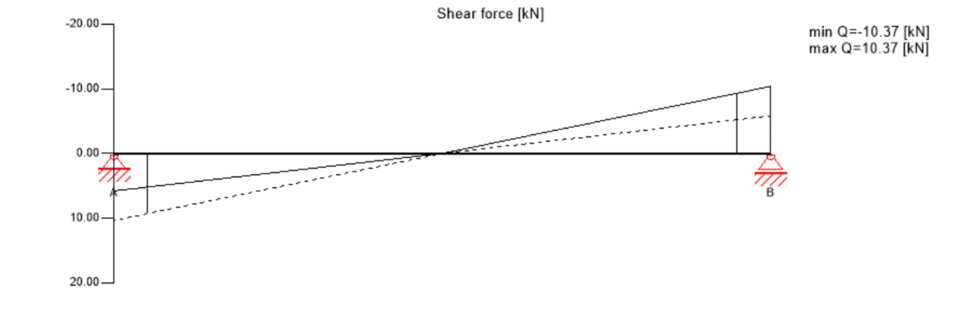
Ultimate limit state (ULS) fire design - results



ULS Fire Flexural design

Field	Dist.	γ_m	k_{mod}	$k_{sys,y}$	k_{fi}	$f_{m,k}$	$f_{m,y,d}$	$f_{t,0,d}$	$f_{c,0,d}$
	[m]	[-]	[-]	[-]	[-]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
1	5.47	1.25	0.80	1.10	1.00	24.00	16.90	8.96	13.44

Field	$M_{y,d}$	$N_{c,d}$	$N_{t,d}$	$\sigma_{m,y,d}$	$\sigma_{c,d}$	$\sigma_{t,d}$	Ratio	
	[kNm]	[kN]	[kN]	[N/mm²]	[N/mm²]	[N/mm²]		
1	58.62	0.00	0.00	4.71	0.00	0.00	28 %	LCO2

ULS Fire Shear analysis

Field	Dist.	$f_{v,k}$	γ_m	k_{mod}	k_{fi}	k_{cr}	$f_{v,d}$	V_d	$\tau_{v,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	0.0	2.50	1.00	1.00	1.15	1.00	2.88	10.37	0.29	10 %	LCO12

ULS Fire Rolling shear

Field	Dist.	$f_{r,k}$	γ_m	k_{mod}	k_{fi}	$f_{r,d}$	V_d	$\tau_{r,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	0.0	1.25	1.00	1.00	1.15	1.44	10.37	0.11	8 %	LCO12

Stress diagram

Multi-storey car park_v2

Closed 12.18m - 4 ribs - 200mm - CLT120 copy

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storaenso

Flexural stress analysis Fire

$M_{y,d}$ =	31.57	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
				k_{fi} =	1.15	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	16.10	N/mm²	
$\sigma_{m,y,d}$ =	4.50	N/mm²		$f_{m,y,d}$ =	30.36	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**15 %**

Shear stress analysis Fire

V_d =	10.37	kN		$f_{v,k}$ =	2.50	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{h,v}$ =	0.00	-	
				k_{fi} =	1.15	-	
$\tau_{v,d}$ =	0.29	N/mm²	<	$f_{v,d}$ =	2.88	N/mm²	✓
Utilization ratio							**10 %**

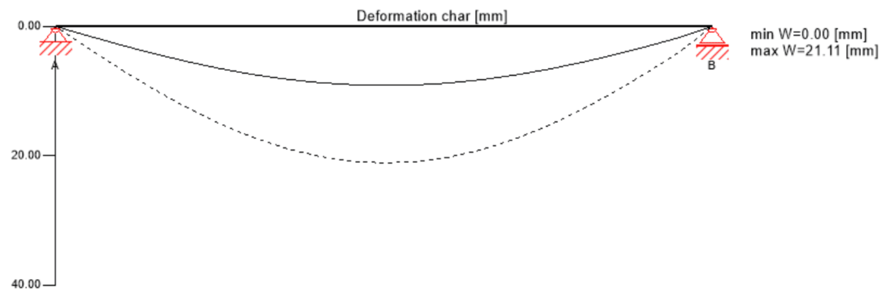
Rolling shear analysis Fire

V_d =	10.37	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				k_{fi} =	1.15	-	
$\tau_{r,d}$ =	0.11	N/mm²	<	$f_{r,d}$ =	1.44	N/mm²	✓
Utilization ratio							**8 %**

Buckling analysis

$M_{y,d}$ =	31.57	kNm		$f_{m,k}$ =	28.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	32.20	N/mm²	
$\sigma_{m,y,d}$ =	3.04	N/mm²		$f_{m,y,d}$ =	32.20	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**9 %**

Service limit state design (SLS) - design results



Multi-storey car park_v2
Closed 12.18m - 4 ribs - 200mm - CLT120 copy
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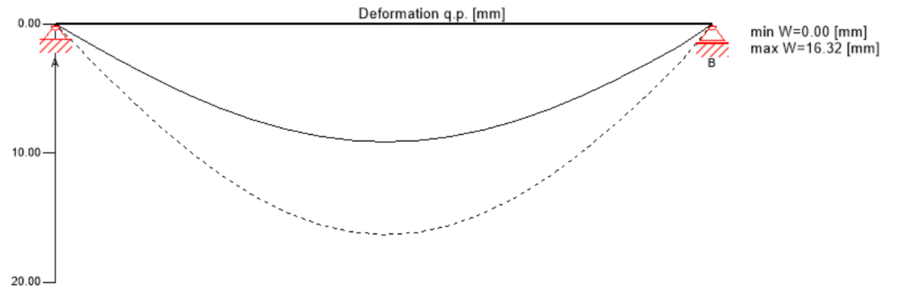
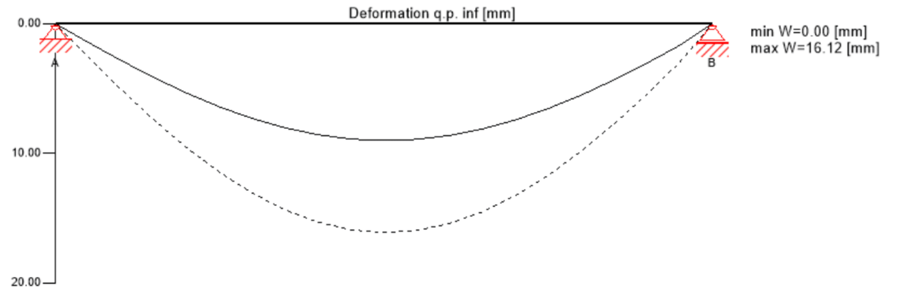
Service limit state design (SLS) - design results





w_{inst} = w[char]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	0.8	L/1	12180.0	21.1	0 %

w_{fin} = w[char] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	0.8	L/250	48.7	37.2	76 %

$w_{net,fin}$ = w[q.p.] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	0.8	L/250	48.7	32.4	67 %

Vibration analysis

General		
Total mass	56.77	[t]
Tributary width	3.1	[m]
Stiffness Longitudinal direction	32856.6	[kNm²]
Stiffness Cross direction	208.0	[kNm²]
Modal damping	2.0	[%]
α	0.1	[-]
Man weight	700.0	[N]
Modal mass	2951.9	[kg]

Vibration analysis

Analysis				
Criterion	Calc.	Class II	Class II	Cl. II
Frequency criterion min	6.161 [Hz]	4.5 [Hz]	73 %	✓
Frequency criterion	6.161 [Hz]	6.0 [Hz]	97 %	
Acceleration criterion	0.202 [m/s²]	0.1 [m/s²]	202 %	
Stiffness criterion	0.367 [mm]	0.5 [mm]	73 %	✓

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V [kN]
self-weight structure	0.6	5.42	5.42
		5.42	5.42
NL - deadload	0.6	0.38	0.38
		0.38	0.38
NL - live load cat. F: traffic area (vehicles <= 25 kN)	0.8	7.61	7.61
		0.00	0.00
NL - snow	0.9	2.13	2.13
		0.00	0.00
NL - Wind	0.9	1.37	1.37
		0.00	0.00

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
Expertise Rolling shear - no edge gluing, H.J. Blass	Expertise on Rolling shear for CLT
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
EN 14080	EN 14080 - Timber Structures - Glued laminated timber and glued solid timber - Requirements
Technical expertise 122/2011/02: analysis of load bearing capacity and separation performance of CLT elements	Verification of the load bearing capacity and the insulation criterion of CLT structures with Stora Enso CLT
Technical expertise 2434/2012 - BB: failure time tf of gypsum fire boards (GKF) according to ON B 3410	Expertise on failure time tf of gypsum wall fire boards according to ON B3410 and gypsum wall boards type DF according to EN 520
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings
ÖNorm B 1995-1-2 NA	ÖNORM EN 1995-1-2 - Austria - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guideline for Europe; publishes by SP Technical Research Institute of Sweden
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
EN 1992-1-1	EN 1992-1-1 - Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings
ETA-11/0030	ETA-11/0030 European Technical Approval; Rothoblaas; Self-tapping screws for use in timber structures
Z-9.1-472	DIBt technical approval Z-9.1-472; SFS intec GmbH; SFS connectors WT-S-6,5, WT-T-6,5, WR-T-9,0 and WR-T-13 as connectors for timber construction
Expertise Rolling shear, H.J. Blass	Expertise on rolling shear strength and rolling shear modulus of CLT panels
Deriviation of the tributary width in CLT-rib deck elements for the engineering practice; TU-Graz, focus_sts 2.2.3_1	Deriviation of the tributary width in CLT-rib deck elements for the engineering practice; TU-Graz, focus_sts 2.2.3_1
ÖNORM EN 1995-1-1_NA, chapter 7.3	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings; chapter 7.3
ETA-11/0190	selftaping screw by Würth

Example Calculation Solid LVL Panel

Multi-storey car park_v2		Page 1/7
1x12.18m span		02.02.2023
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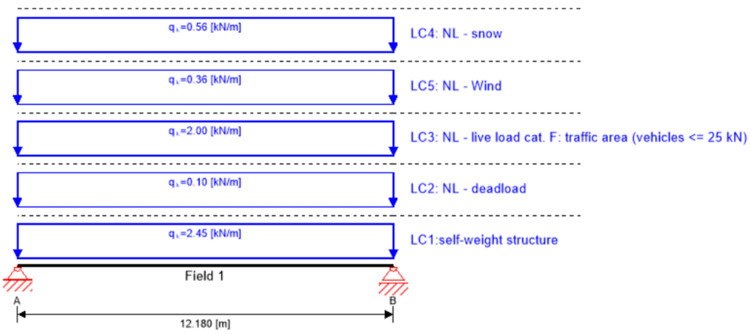
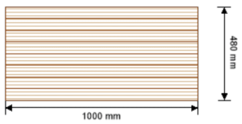
System



Global utilization ratio 70 %

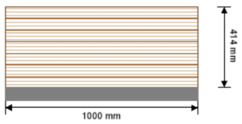
ULS	19 %	ULS Fire	9 %	SLS	48 %	SLS Vibration	70 %	Support	-1 %	Void	-1 %

Section: LVL G-X flatwise 100/48



Layer	Thickness	Orientation	Material
1	60.0 mm	90°	LVL G-X flatwise
2	60.0 mm	90°	LVL G-X flatwise
3	60.0 mm	90°	LVL G-X flatwise
4	60.0 mm	90°	LVL G-X flatwise
5	60.0 mm	90°	LVL G-X flatwise
6	60.0 mm	90°	LVL G-X flatwise
7	60.0 mm	90°	LVL G-X flatwise
8	60.0 mm	90°	LVL G-X flatwise
t_{CLT}	**480.0 mm**		

Section Fire: LVL G-X flatwise 100/48



Layer	Thickness	Orientation	Material
1	60.0 mm	90°	LVL G-X flatwise
2	60.0 mm	90°	LVL G-X flatwise
3	60.0 mm	90°	LVL G-X flatwise
4	60.0 mm	90°	LVL G-X flatwise
5	60.0 mm	90°	LVL G-X flatwise
6	60.0 mm	90°	LVL G-X flatwise
7	54.0 mm	90°	LVL G-X flatwise
t_{CLT}	**414.0 mm**		
Time	**90 min**		

Fire resistance class:R 90
Fire protection layering : no additional fire protection

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	59.0	66.0	58.5	0.0

Multi-storey car park_v2

1x12.18m span

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02.02.2023

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$E_{0,5}$ [N/mm²]
LVL G-X flatwise	31.00	26.00	0.00	26.00	2.20	1.30	0.60	10,100.00	120.00	8,800.00
LVL G-X edgewise	32.00	26.00	6.00	26.00	9.00	4.50	0.00	10,500.00	600.00	8,800.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	2.45

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00

LC4: NL - snow

continuous load

Field	Load at start [kN/m]
1	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36

Multi-storey car park_v2
1x12.18m span
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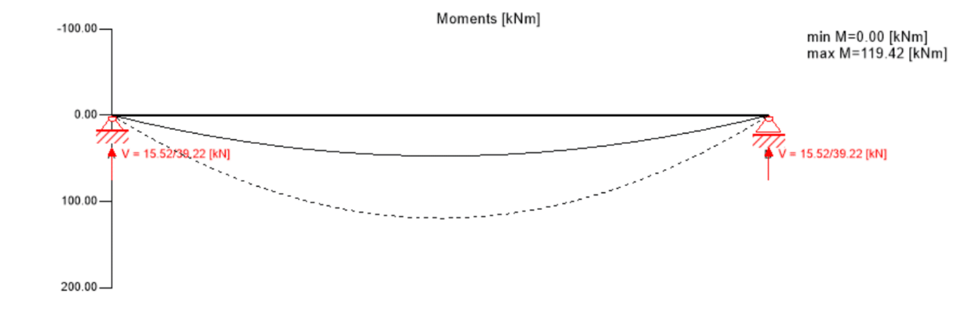
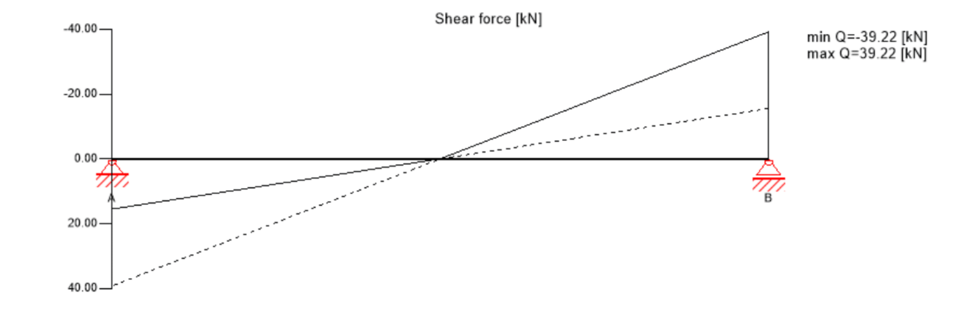
Ultimate limit state (ULS) - design results





ULS Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,z}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]	$f_{m,z,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	31.00	26.00	26.00	1.20	0.80	1.00	0.80	0.87	0.92	16.62	18.46	15.94	17.33

Field	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{m,z,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio
1	119.42	0.00	0.00	0.00	3.11	0.00	0.00	0.00	19 % LCO2

ULS Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{h,v}$ [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio
1	11.7	1.30	1.20	0.80	0.80	0.87	36.13	0.11	13 % LCO2

Flexural stress analysis

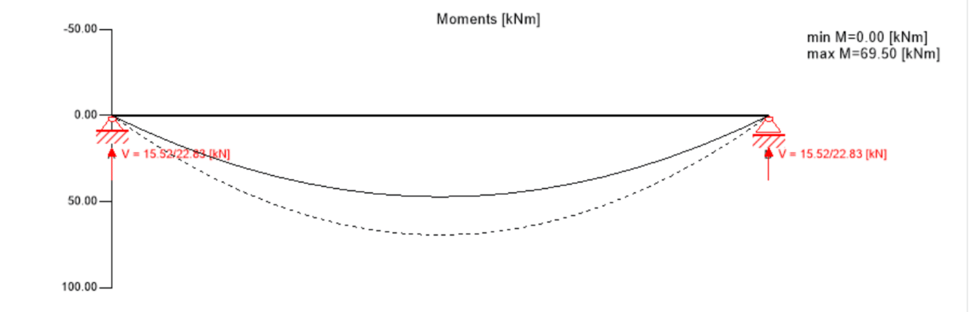
$M_{y,d}$ =	119.42 kNm		$f_{m,k}$ =	31.00 N/mm²	
$M_{z,d}$ =	0.00 kNm		$f_{m,k,z}$ =	31.00 N/mm²	
$N_{t,d}$ =	0.00 kN		γ_m =	1.20 -	
			k_{mod} =	0.80 -	
			$k_{sys,y}$ =	1.00 -	
			$k_{h,m,y}$ =	0.80 -	
			$k_{h,m,z}$ =	0.87 -	
			k_l =	0.92 -	
$\sigma_{t,d}$ =	0.00 N/mm²		$f_{t,0,d}$ =	15.94 N/mm²	
$\sigma_{m,y,d}$ =	3.11 N/mm²		$f_{m,y,d}$ =	16.62 N/mm²	
$\sigma_{m,z,d}$ =	0.00 N/mm²	<	$f_{m,z,d}$ =	18.46 N/mm²	✓

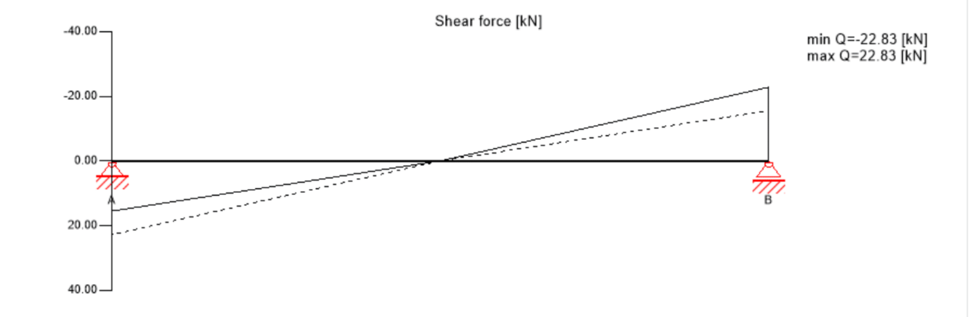
Utilization ratio 19 %

Shear stress analysis

V_d =	36.13	kN	$f_{v,k}$ =	1.30	N/mm²	
			γ_m =	1.20	-	
			k_{mod} =	0.80	-	
			$k_{h,v}$ =	0.80	-	
$\tau_{v,d}$ =	0.11	N/mm² <	$f_{v,d}$ =	0.87	N/mm²	✓
Utilization ratio						13 %

Ultimate limit state (ULS) fire design - results





ULS Fire Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$k_{sys,z}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]	$f_{m,z,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	31.00	26.00	26.00	1.20	0.80	1.00	1.00	0.80	0.87	0.92	16.62	18.46	15.94	17.33

Field	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{m,z,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	119.42	0.00	0.00	0.00	3.11	0.00	0.00	0.00	19 %	LCO2

ULS Fire Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{h,v}$ [-]	k_{fi} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.41	1.30	1.00	1.00	0.82	1.10	1.43	21.27	0.08	5 %	LCO12

Multi-storey car park_v2

1x12.18m span

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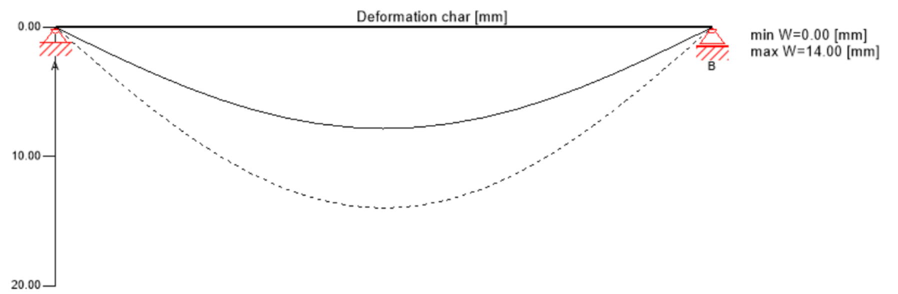
Flexural stress analysis Fire

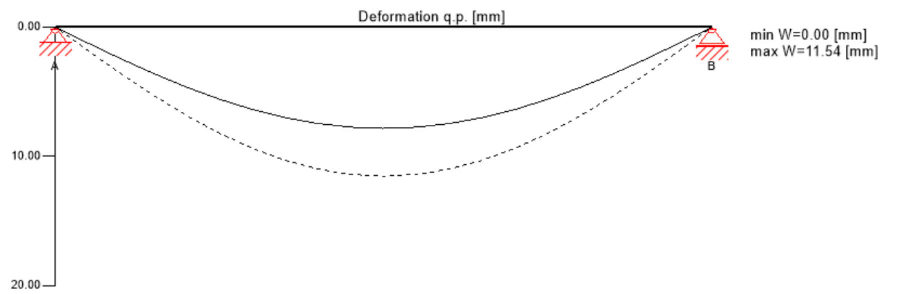
$M_{y,d}$ =	69.50	kNm		$f_{m,k}$ =	31.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	31.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{h,m,y}$ =	0.82	-	
				$k_{h,m,z}$ =	0.87	-	
				k_l =	0.92	-	
				k_{fi} =	1.10	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	26.29	N/mm²	
$\sigma_{m,y,d}$ =	2.43	N/mm²		$f_{m,y,d}$ =	27.96	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	30.46	N/mm²	✓
Utilization ratio							9 %

Shear stress analysis Fire

V_d =	21.27	kN		$f_{v,k}$ =	1.30	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{h,v}$ =	0.82	-	
				k_{fi} =	1.10	-	
$\tau_{v,d}$ =	0.08	N/mm²	<	$f_{v,d}$ =	1.43	N/mm²	✓
Utilization ratio							5 %

Service limit state design (SLS) - design results





w_{inst} = w[char]

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/1	12180.0	14.0	0 %

w_{fin} = w[char] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/250	48.7	23.2	48 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

Field	K_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/250	48.7	20.8	43 %

Vibration analysis

General		
Total mass	38.53	[t]
Tributary width	4.5	[m]
Stiffness Longitudinal direction	95581.6	[kNm²]
Stiffness Cross direction	2500.0	[kNm²]
Modal damping	2.0	[%]
α	0.1	[-]
Man weight	700.0	[N]
Modal mass	7036.3	[kg]

Analysis

Criterion	Calc.	Class I	Class II	Class I	Class II	Cl. I	Cl. II
Frequency criterion min	6.425 [Hz]	4.5 [Hz]	4.5 [Hz]	70 %	70 %	✓	✓
Frequency criterion	6.425 [Hz]	8.0 [Hz]	6.0 [Hz]	125 %	93 %		
Acceleration criterion	0.076 [m/s²]	0.05 [m/s²]	0.1 [m/s²]	152 %	76 %	×	
Stiffness criterion	0.088 [mm]	0.25 [mm]	0.5 [mm]	35 %	18 %	✓	✓

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V
self-weight structure	0.6	14.91	14.91
		14.91	14.91
NL - deadload	0.6	0.61	0.61
		0.61	0.61
NL - live load cat. F: traffic area (vehicles <= 25 kN)	0.8	12.18	12.18
		0.00	0.00
NL - snow	0.9	3.41	3.41
		0.00	0.00
NL - Wind	0.9	2.19	2.19
		0.00	0.00

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
EN 14080	EN 14080 - Timber Structures - Glued laminated timber and glued solid timber - Requirements
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General-Common rules and rules for buildings
ÖNorm B 1995-1-2 NA	ÖNORM EN 1995-1-2 - Austria - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements
ÖNORM EN 1995-1-1_NA, chapter 7.3	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General-Common rules and rules for buildings; chapter 7.3
CERTIFICATE NO. EUFI29-20000564-C	Product certificate

Example Calculation LVL Open Rib Panel

Multi-storey car park_v2

12.18m - 4 ribs - LVL37 - 51

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		02.02.2023
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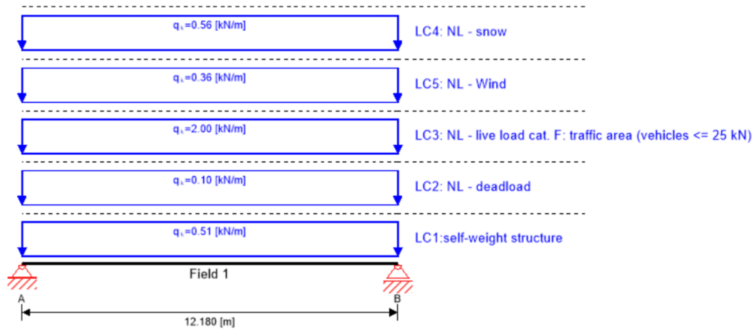
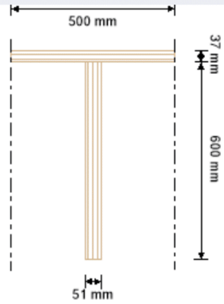
System



Global utilization ratio									**194 %**
ULS	28 %	ULS Fire	0 %	SLS	52 %	SLS Vibration	194 %	Support	-1 %

Section: LVL rib panel by Stora Enso: 1/60



Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	37.0	500.0	0°	LVL X flatwise
2	600.0	51.0	0°	LVL S edgewise
t_{CLT}	**637.0 mm**			

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
LVL S edgewise	44.00	35.00	0.80	35.00	6.00	4.20	0.00	13,800.00	600.00	
LVL X flatwise	36.00	26.00	6.00	26.00	2.20	1.30	0.60	10,500.00	120.00	

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	0.51

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00

LC4: NL - snow

continuous load

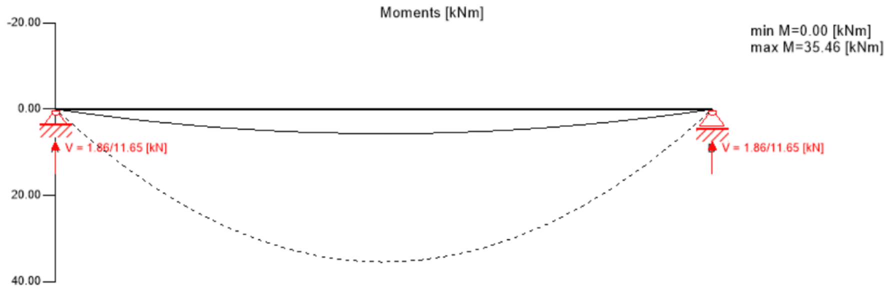
Field	Load at start [kN/m]
1	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36

Ultimate limit state (ULS) - design results T=0



storaenso

Multi-storey car park_v2

12.18m - 4 ribs - LVL37 - 51

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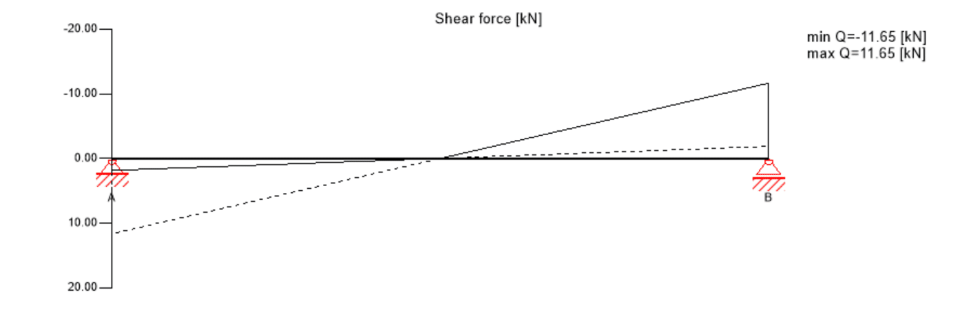
Ultimate limit state (ULS) - design results T=0



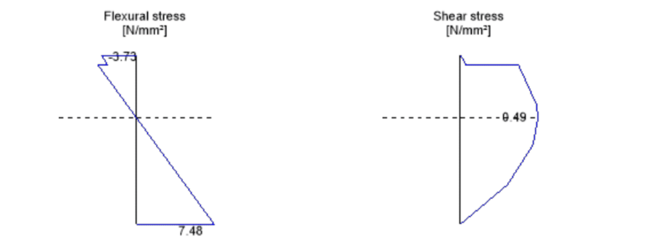
ULS Flexural design T=0

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	1.20	0.80	1.00	44.00	26.99	21.01	23.33
Field	**$M_{y,d}$ [kNm]**	**$N_{c,d}$ [kN]**	**$N_{t,d}$ [kN]**	**$\sigma_{m,y,d}$ [N/mm²]**	**$\sigma_{c,d}$ [N/mm²]**	**$\sigma_{t,d}$ [N/mm²]**	**Ratio**	
1	35.46	0.00	0.00	7.48	0.00	0.00	28 %	LCO2

ULS Shear analysis T=0

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	4.20	1.20	0.80	1.00	2.80	11.65	0.49	18 %	LCO2

Stress diagram T=0



Flexural stress analysis T=0

$M_{y,d}$ =	35.46	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	44.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{h,m,y}$ =	0.92	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	0.90	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	21.01	N/mm²	
$\sigma_{m,y,d}$ =	7.48	N/mm²		$f_{m,y,d}$ =	26.99	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						28 %	

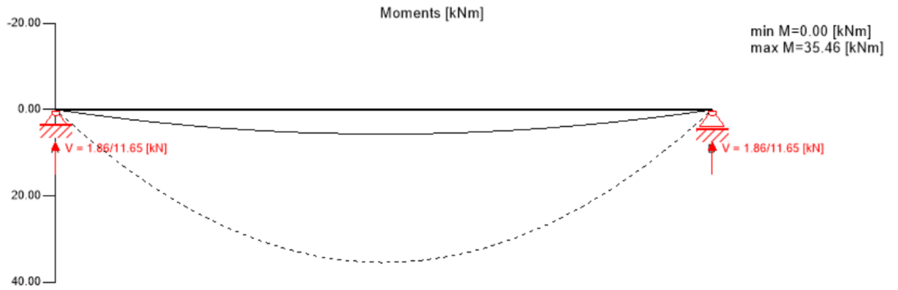
Shear stress analysis T=0

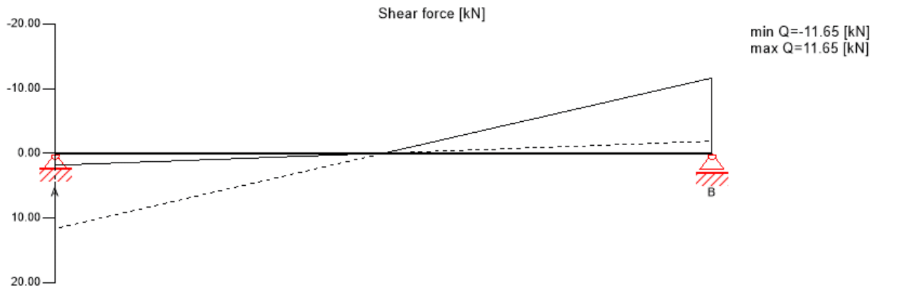
V_d =	11.65	kN		$f_{v,k}$ =	4.20	N/mm²	
				γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.49	N/mm²	<	$f_{v,d}$ =	2.80	N/mm²	✓
Utilization ratio							18 %

Buckling analysis T=0

$M_{y,d}$ =	35.46	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	23.33	N/mm²	
$\sigma_{m,y,d}$ =	5.61	N/mm²		$f_{m,y,d}$ =	29.33	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							19 %

Ultimate limit state (ULS) - design results T=∞





ULS Flexural design T=∞

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	1.20	0.80	1.00	44.00	26.99	21.01	23.33
Field	**$M_{y,d}$ [kNm]**	**$N_{c,d}$ [kN]**	**$N_{t,d}$ [kN]**	**$\sigma_{m,y,d}$ [N/mm²]**	**$\sigma_{c,d}$ [N/mm²]**	**$\sigma_{t,d}$ [N/mm²]**	**Ratio**	
1	35.46	0.00	0.00	7.55	0.00	0.00	28 %	LCO20

Multi-storey car park_v2

12.18m - 4 ribs - LVL37 - 51

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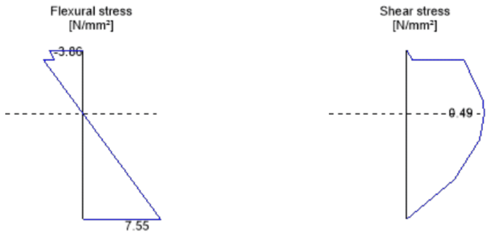
ULS Shear analysis T=∞

Field	Dist.	$f_{v,k}$	γ_m	k_{mod}	k_{cr}	$f_{v,d}$	V_d	$\tau_{v,d}$	Ratio	
	[m]	[N/mm²]	[-]	[-]	[-]	[N/mm²]	[kN]	[N/mm²]		
1	12.18	4.20	1.20	0.80	1.00	2.80	-11.65	0.49	18 %	LCO20

Stress diagram T=∞



Flexural stress analysis T=∞

$M_{y,d}$ =	35.46	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	44.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{h,m,y}$ =	0.92	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	0.90	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	21.01	N/mm²	
$\sigma_{m,y,d}$ =	7.55	N/mm²		$f_{m,y,d}$ =	26.99	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓

Utilization ratio 28 %

Shear stress analysis T=∞

V_d =	-11.65	kN		$f_{v,k}$ =	4.20	N/mm²	
				γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.49	N/mm²	<	$f_{v,d}$ =	2.80	N/mm²	✓

Utilization ratio 18 %

Buckling analysis T=∞

$M_{y,d}$ =	35.46	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	23.33	N/mm²	
$\sigma_{m,y,d}$ =	5.70	N/mm²		$f_{m,y,d}$ =	29.33	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓

Utilization ratio 19 %

Multi-storey car park_v2

12.18m - 4 ribs - LVL37 - 51

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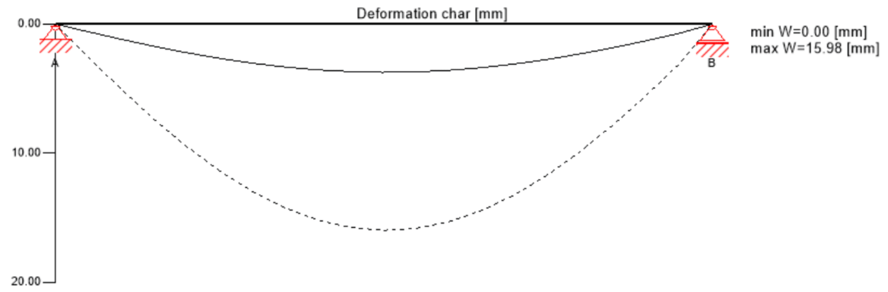
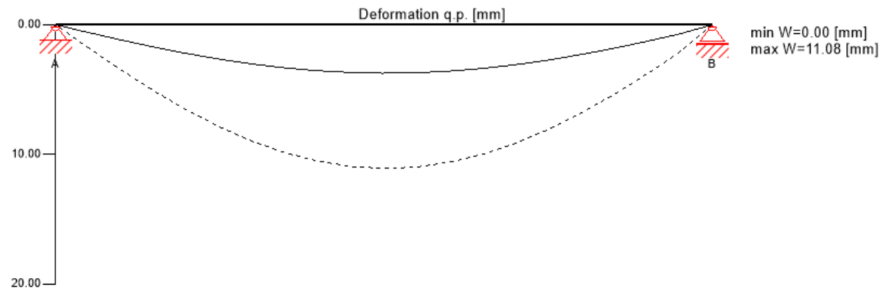
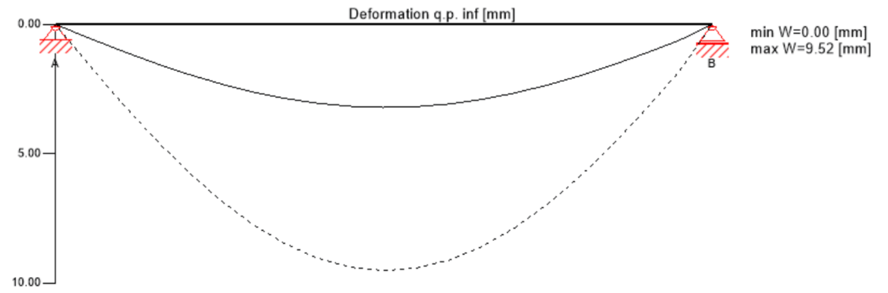
Service limit state design (SLS) - design results







w_{inst} = w[char]

Field	$K_{def,top}$	$K_{def,Rib}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	L/1	12180.0	16.0	0 %

w_{fin} = w[char] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	L/250	48.7	25.5	52 %

$w_{net,fin}$ = w[q.p.] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	L/250	48.7	20.6	42 %

Multi-storey car park_v2

12.18m - 4 ribs - LVL37 - 51

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Vibration analysis

General		
Total mass	22.76	[t]
Tributary width	1.5	[m]
Stiffness Longitudinal direction	26187.2	[kNm²]
Stiffness Cross direction	8.4	[kNm²]
Modal damping	2.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	562.2	[kg]

Analysis				
Criterion	Calc.	Class II	Class II	Cl. II
Frequency criterion min	9.712 [Hz]	4.5 [Hz]	46 %	✓
Frequency criterion	9.712 [Hz]	6.0 [Hz]	62 %	
Acceleration criterion	0.256 [m/s²]	0.1 [m/s²]	256 %	
Stiffness criterion	0.969 [mm]	0.5 [mm]	194 %	×

Support reaction

Load case category	k_{mod}	A_V	B_V
		[kN]	
self-weight structure	0.6	1.56	1.56
		1.56	1.56
NL - deadload	0.6	0.30	0.30
		0.30	0.30
NL - live load cat. F: traffic area (vehicles <= 25 kN)	0.8	6.09	6.09
		0.00	0.00
NL - snow	0.9	1.71	1.71
		0.00	0.00
NL - Wind	0.9	1.10	1.10
		0.00	0.00

Reference documents for this analysis

English title	Description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-11/0190	selftaping screw by Würth

Disclaimer

Multi-storey car park_v2

Semi-open 12.18m - 4 ribs - LVL37 - LVL43 - 51

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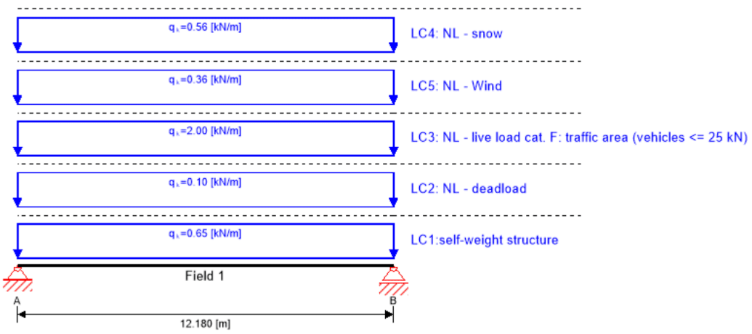
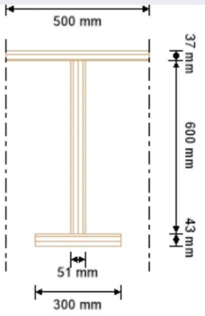
System



Global utilization ratio									127 %
ULS	24 %	ULS Fire	0 %	SLS	34 %	SLS Vibration	127 %	Support	-1 %

Section: LVL rib panel by Stora Enso: 1/60



Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	37.0	500.0	0°	LVL X flatwise
2	600.0	51.0	0°	LVL S edgewise
3	43.0	218.2	0°	LVL X flatwise
t_{CLT}	680.0 mm			

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
LVL S edgewise	44.00	35.00	0.80	35.00	6.00	4.20	0.00	13,800.00	600.00	
LVL X flatwise	36.00	26.00	6.00	26.00	2.20	1.30	0.60	10,500.00	120.00	
LVL X flatwise	36.00	26.00	6.00	26.00	2.20	1.30	0.60	10,500.00	120.00	

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	0.65

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00

LC4: NL - snow

continuous load

Field	Load at start [kN/m]
1	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36

Multi-storey car park_v2

Semi-open 12.18m - 4 ribs - LVL37 - LVL43 - 51

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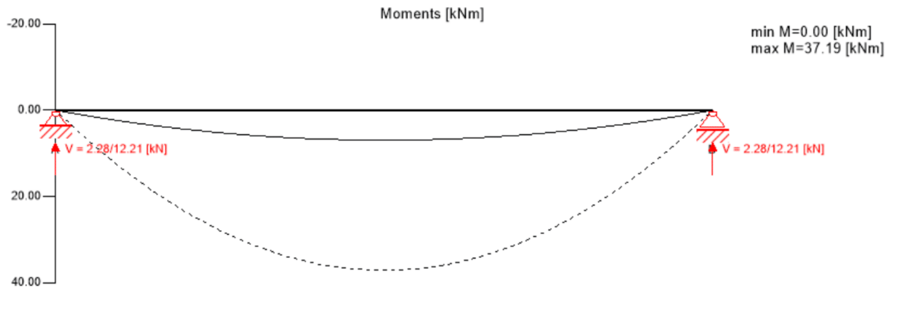
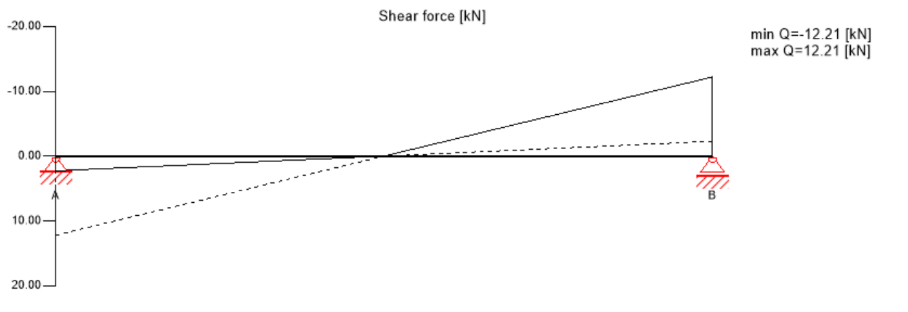
Ultimate limit state (ULS) - design results T=0





ULS Flexural design T=0

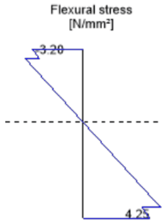
Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	1.20	0.80	1.00	44.00	26.99	21.01	23.33

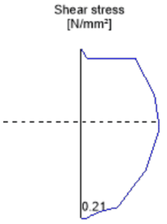
Field	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	37.19	0.00	0.00	4.25	0.00	0.00	16 %	LCO2

ULS Shear analysis T=0

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	1.30	1.20	0.80	1.00	0.87	12.21	0.21	24 %	LCO2

Stress diagram T=0





Multi-storey car park_v2 Page 4/8
Semi-open 12.18m - 4 ribs - LVL37 - LVL43 - 51 02.02.2023
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Flexural stress analysis T=0

$M_{y,d}$ =	37.19	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	44.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{h,m,y}$ =	0.92	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	0.90	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	21.01	N/mm²	
$\sigma_{m,y,d}$ =	4.25	N/mm²		$f_{m,y,d}$ =	26.99	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**16 %**

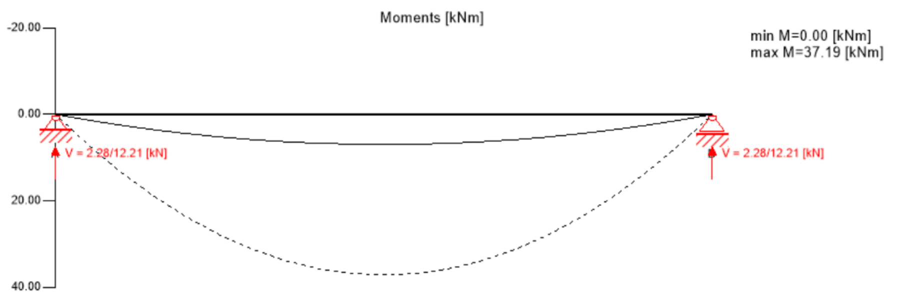
Shear stress analysis T=0

V_d =	12.21	kN		$f_{v,k}$ =	1.30	N/mm²	
				γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.21	N/mm²	<	$f_{v,d}$ =	0.87	N/mm²	✓
Utilization ratio							**24 %**

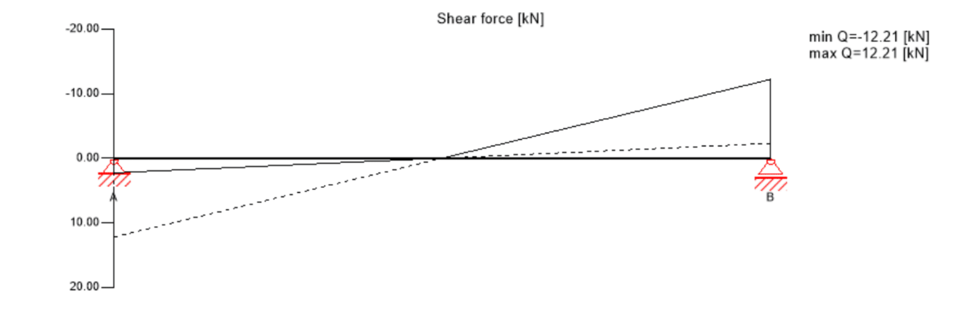
Buckling analysis T=0

$M_{y,d}$ =	37.19	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	23.33	N/mm²	
$\sigma_{m,y,d}$ =	3.73	N/mm²		$f_{m,y,d}$ =	29.33	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**13 %**

Ultimate limit state (ULS) - design results T=∞



Ultimate limit state (ULS) - design results T=∞



ULS Flexural design T=∞

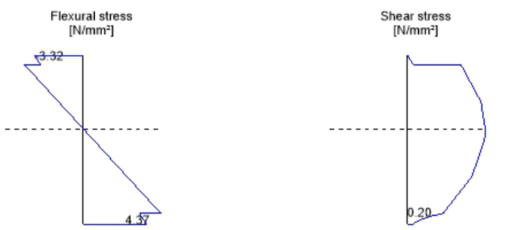
Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	1.20	0.80	1.00	44.00	26.99	21.01	23.33

Field	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	37.19	0.00	0.00	4.37	0.00	0.00	16 %	LCO20

ULS Shear analysis T=∞

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	1.30	1.20	0.80	1.00	0.87	12.21	0.20	24 %	LCO20

Stress diagram T=∞



Flexural stress analysis T=∞

$M_{y,d}$ =	37.19	kNm		$f_{m,k}$ =	44.00	N/mm²
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	44.00	N/mm²
$N_{t,d}$ =	0.00	kN		γ_m =	1.20	-
				k_{mod} =	0.80	-
				$k_{sys,y}$ =	1.00	-
				$k_{h,m,y}$ =	0.92	-
				$k_{h,m,z}$ =	1.00	-
				k_l =	0.90	-
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	21.01	N/mm²
$\sigma_{m,y,d}$ =	4.37	N/mm²		$f_{m,y,d}$ =	26.99	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm² ✓

Utilization ratio 16 %

Multi-storey car park_v2 Page 6/8

Semi-open 12.18m - 4 ribs - LVL37 - LVL43 - 51 02.02.2023

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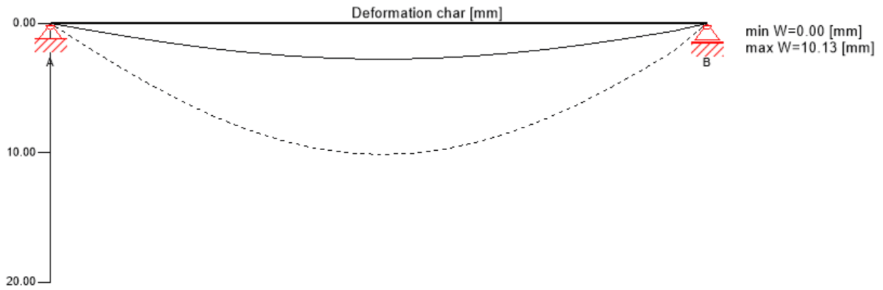
Shear stress analysis T=∞

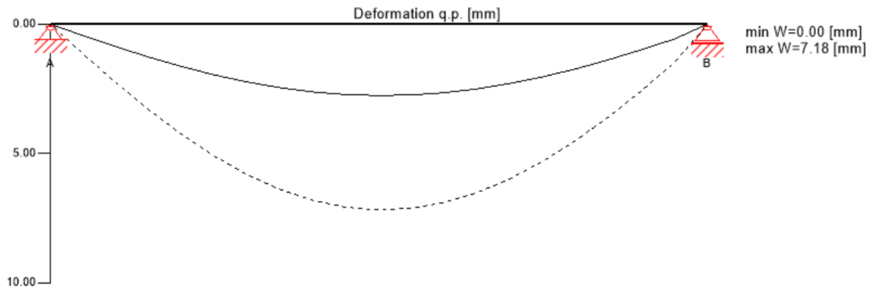
V_d =	12.21	kN		$f_{v,k}$ =	1.30	N/mm²	
				γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.20	N/mm²	<	$f_{v,d}$ =	0.87	N/mm²	✓
Utilization ratio							24 %

Buckling analysis T=∞

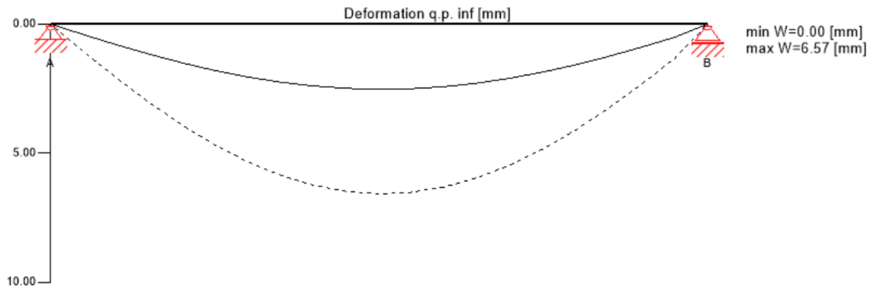
$M_{y,d}$ =	37.19	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	23.33	N/mm²	
$\sigma_{m,y,d}$ =	3.85	N/mm²		$f_{m,y,d}$ =	29.33	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							13 %

Service limit state design (SLS) - design results





Service limit state design (SLS) - design results



w_{inst} = w[char]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	1	L/1	12180.0	10.1	0 %

w_{fin} = w[char] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	1	L/250	48.7	16.7	34 %

$w_{net,fin}$ = w[q.p.] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	1	L/250	48.7	13.8	28 %

Vibration analysis

General		
Total mass	27.89	[t]
Tributary width	1.3	[m]
Stiffness Longitudinal direction	45970.6	[kNm²]
Stiffness Cross direction	8.4	[kNm²]
Modal damping	2.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	598.5	[kg]

Analysis				
Criterion	Calc.	Class II	Class II	Cl. II
Frequency criterion min	11.624 [Hz]	4.5 [Hz]	39 %	✓
Frequency criterion	11.624 [Hz]	6.0 [Hz]	52 %	
Acceleration criterion	0.112 [m/s²]	0.1 [m/s²]	112 %	
Stiffness criterion	0.635 [mm]	0.5 [mm]	127 %	×

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V [kN]
self-weight structure	0.6	1.98	1.98
		1.98	1.98
NL - deadload	0.6	0.30	0.30
		0.30	0.30
NL - live load cat. F: traffic area (vehicles <= 25 kN)	0.8	6.09	6.09
		0.00	0.00
NL - snow	0.9	1.71	1.71
		0.00	0.00

Multi-storey car park_v2

Semi-open 12.18m - 4 ribs - LVL37 - LVL43 - 51

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Support reaction

Load case category	k_{mod}	A_V	B_V
		[kN]	
NL - Wind	0.9	1.10 0.00	1.10 0.00

Reference documents for this analysis

English title	Description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-11/0190	selftaping screw by Würth

Disclaimer

Multi-storey car park_v2

Closed 12.18m - 4 ribs - LVL37 - LVL43 - 51

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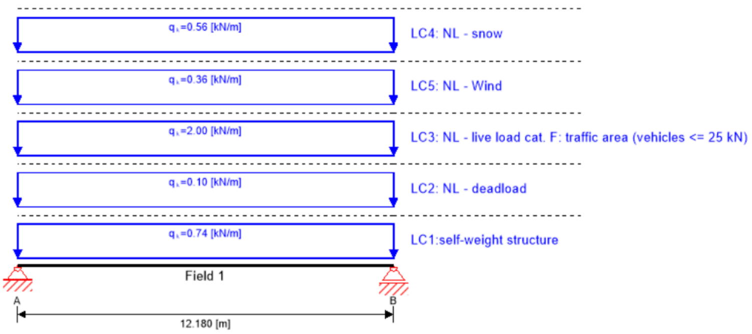
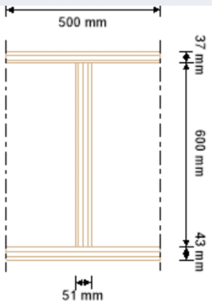
System



Global utilization ratio									87 %
ULS	36 %	ULS Fire	0 %	SLS	30 %	SLS Vibration	87 %	Support	-1 %

Section: LVL rib panel by Stora Enso: 1/60



Layer	Thickness [mm]	Width [mm]	Orientation	Material
1	37.0	500.0	0°	LVL X flatwise
2	600.0	51.0	0°	LVL S edgewise
3	43.0	500.0	0°	LVL X flatwise
t_{CLT}	680.0 mm			

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
LVL S edgewise	44.00	35.00	0.80	35.00	6.00	4.20	0.00	13,800.00	600.00	
LVL X flatwise	36.00	26.00	6.00	26.00	2.20	1.30	0.60	10,500.00	120.00	
LVL X flatwise	36.00	26.00	6.00	26.00	2.20	1.30	0.60	10,500.00	120.00	

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6

Multi-storey car park_v2

Closed 12.18m - 4 ribs - LVL37 - LVL43 - 51

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Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	0.74

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00

LC4: NL - snow

continuous load

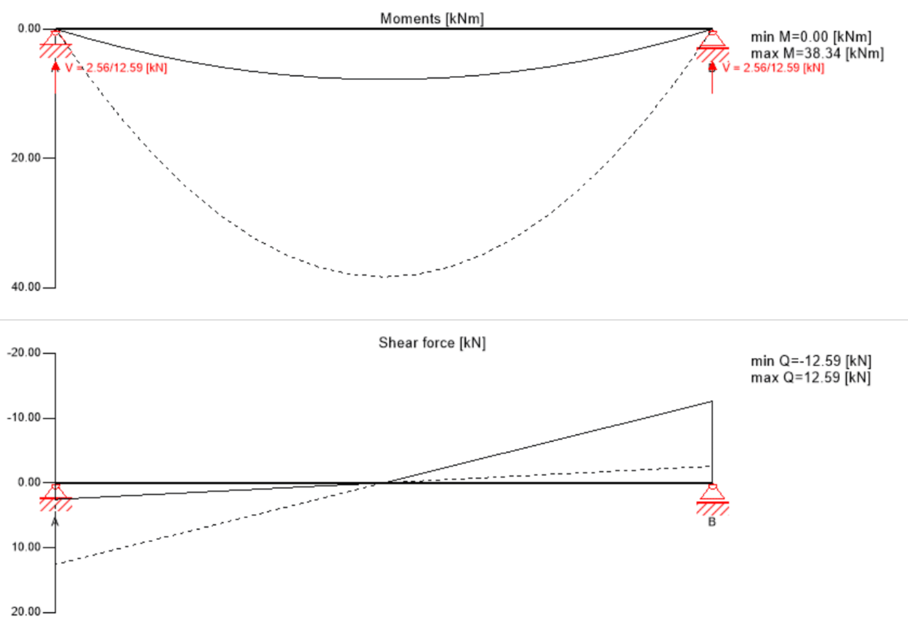
Field	Load at start [kN/m]
1	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36

Ultimate limit state (ULS) - design results T=0



ULS Flexural design T=0

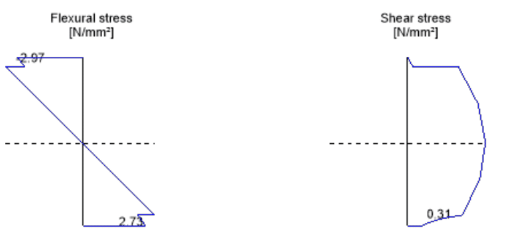
Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	1.20	0.80	1.00	44.00	26.99	21.01	23.33

Field	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	38.34	0.00	0.00	-2.97	0.00	0.00	11 %	LCO2

ULS Shear analysis T=0

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	0.0	1.30	1.20	0.80	1.00	0.87	12.59	0.31	36 %	LCO2

Stress diagram T=0



Flexural stress analysis T=0

$M_{y,d}$ =	38.34	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	44.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{h,m,y}$ =	0.92	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	0.90	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	21.01	N/mm²	
$\sigma_{m,y,d}$ =	-2.97	N/mm²		$f_{m,y,d}$ =	26.99	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						**11 %**	

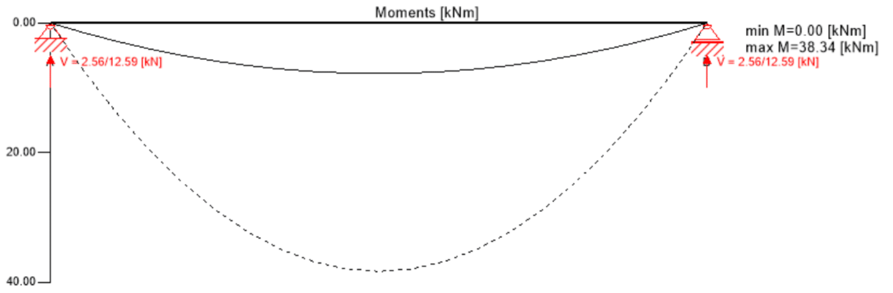
Shear stress analysis T=0

V_d =	12.59	kN		$f_{v,k}$ =	1.30	N/mm²	
				γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{h,v}$ =	0.00	-	
$\tau_{v,d}$ =	0.31	N/mm²	<	$f_{v,d}$ =	0.87	N/mm²	✓
Utilization ratio						**36 %**	

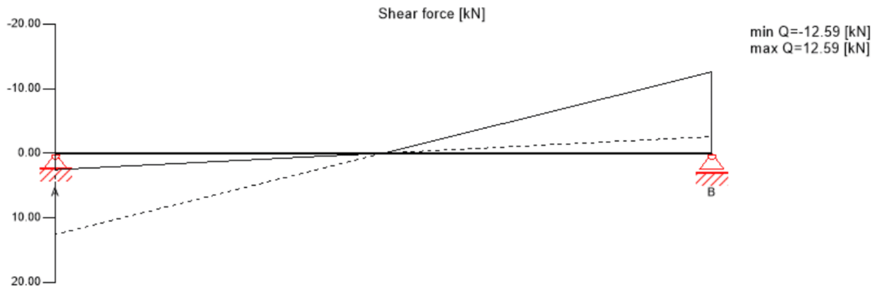
Buckling analysis T=0

$M_{y,d}$ =	38.34	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	23.33	N/mm²	
$\sigma_{m,y,d}$ =	2.85	N/mm²		$f_{m,y,d}$ =	29.33	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						**10 %**	

Ultimate limit state (ULS) - design results T=∞



Ultimate limit state (ULS) - design results T=∞



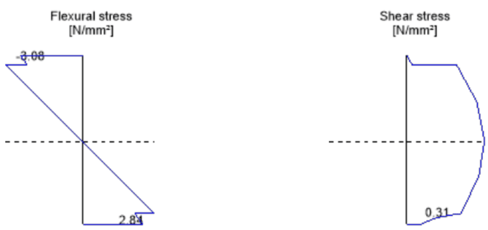
ULS Flexural design T=∞

Field	Dist. [m]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,k}$ [N/mm²]	$f_{m,y,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	6.09	1.20	0.80	1.00	44.00	26.99	21.01	23.33
Field	$M_{y,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	38.34	0.00	0.00	-3.08	0.00	0.00	11 %	LCO20

ULS Shear analysis T=∞

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{cr} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	12.18	1.30	1.20	0.80	1.00	0.87	-12.59	0.31	35 %	LCO20

Stress diagram T=∞



Flexural stress analysis T=∞

$M_{y,d}$ =	38.34	kNm		$f_{m,k}$ =	44.00	N/mm²
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	44.00	N/mm²
$N_{t,d}$ =	0.00	kN		γ_m =	1.20	-
				k_{mod} =	0.80	-
				$k_{sys,y}$ =	1.00	-
				$k_{h,m,y}$ =	0.92	-
				$k_{h,m,z}$ =	1.00	-
				k_l =	0.90	-
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	21.01	N/mm²
$\sigma_{m,y,d}$ =	-3.08	N/mm²		$f_{m,y,d}$ =	26.99	N/mm²
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm² ✓

Utilization ratio 11 %

Multi-storey car park_v2
Closed 12.18m - 4 ribs - LVL37 - LVL43 - 51

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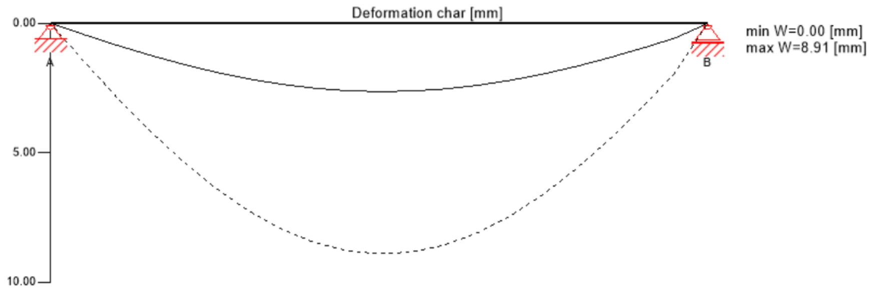
Shear stress analysis T=∞

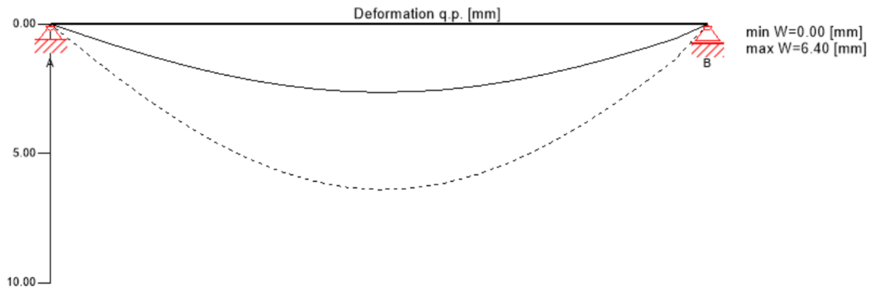
V_d =	-12.59	kN		$f_{v,k}$ =	1.30	N/mm²		
				γ_m =	1.20	-		
				k_{mod} =	0.80	-		
				$k_{h,v}$ =	0.00	-		
$\tau_{v,d}$ =	0.31	N/mm²	<	$f_{v,d}$ =	0.87	N/mm²	✓	
Utilization ratio							**35 %**	

Buckling analysis T=∞

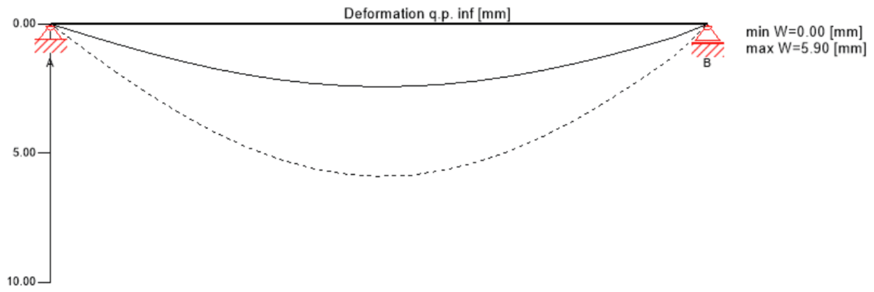
$M_{y,d}$ =	38.34	kNm		$f_{m,k}$ =	44.00	N/mm²	
$M_{z,d}$ =	0.00	kNm					
$N_{c,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{sys,z}$ =	1.00	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
$\sigma_{c,d}$ =	0.00	N/mm²		$f_{c,0,d}$ =	23.33	N/mm²	
$\sigma_{m,y,d}$ =	2.96	N/mm²		$f_{m,y,d}$ =	29.33	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio							**10 %**

Service limit state design (SLS) - design results





Service limit state design (SLS) - design results



w_{inst} = w[char]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	1	L/1	12180.0	8.9	0 %

w_{fin} = w[char] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	1	L/250	48.7	14.8	30 %

$w_{net,fin}$ = w[q.p.] + w[q.p.∞]

Field	$K_{def,top}$	$K_{def,Rib}$	$K_{def,bottom}$	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	1	0.8	1	L/250	48.7	12.3	25 %

Vibration analysis

General		
Total mass	31.31	[t]
Tributary width	1.6	[m]
Stiffness Longitudinal direction	55628.3	[kNm²]
Stiffness Cross direction	21.7	[kNm²]
Modal damping	2.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	811.1	[kg]

Analysis				
Criterion	Calc.	Class II	Class II	Cl. II
Frequency criterion min	12.069 [Hz]	4.5 [Hz]	37 %	✓
Frequency criterion	12.069 [Hz]	6.0 [Hz]	50 %	
Acceleration criterion	0.069 [m/s²]	0.1 [m/s²]	69 %	
Stiffness criterion	0.435 [mm]	0.5 [mm]	87 %	✓

Support reaction

Load case category	k_{mod}	A_V [kN]	B_V
self-weight structure	0.6	2.25	2.25
		2.25	2.25
NL - deadload	0.6	0.30	0.30
		0.30	0.30
NL - live load cat. F: traffic area (vehicles <= 25 kN)	0.8	6.09	6.09
		0.00	0.00
NL - snow	0.9	1.71	1.71
		0.00	0.00

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Closed 12.18m - 4 ribs - LVL37 - LVL43 - 51 02.02.2023

Paul Brink Designer PB Checker

Support reaction

Load case category	k_{mod}	A_V	B_V
		[kN]	
NL - Wind	0.9	1.10 0.00	1.10 0.00

Reference documents for this analysis

English title	Description
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings
ETA-12/0063	SFS intec AG; Self-tapping screws for use in timber constructions
ETA-12/0062	SFA intec AG; ETA-12/0062; selftapping screws for use in timber constructions
ETA-11/0086	GH Various Angle Brackets
ETA-11/0190	selftaping screw by Würth

Disclaimer

F.1.2 Results Minimum Dimensions Long-span Deck Systems

Table 58. Long-span deck systems with required minimum thickness for various span with governing requirement (base case requirements).

	Solid CLT Panel									CLT Open Rib Panel								CLT Closed Rib Panel							
L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height			
[m]	[-]	[-]	[mm]	[kN/m^2]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	xxx							G	G	360	480	1.18	2.94				G	200	440	1.42	3.56				G
13.70	xxx									450	570	1.32	3.30				G	300	540	1.54	3.84				G
14.56	xxx									500	620	1.40	3.50				G	360	600	1.60	4.01				G
15.61	xxx									600	720	1.56	3.90				G	450	690	1.70	4.26				G
16.26	xxx									700	820	1.72	4.30				G	500	740	1.76	4.40				G
17.16	xxx									700	820	1.72	4.30				G	600	840	1.87	4.68				G
17.66	xxx									800	920	1.88	4.70				G	600	840	1.87	4.68				G

	Solid LVL Panel									LVL Open Rib Panel								LVL Semi-open Rib Panel								LVL Closed Rib Panel							
L	$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Normative				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing rib height			
[m]	[-]	[-]	[mm]	[kN/m^2]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	LVL G 480 8s	8	480	2.45	6.13				G	xxx							G	xxx							G	600	737	1.04	2.60				G
13.70	LVL G 540 9s	9	540	2.75	6.88				G	xxx							G	xxx							G	600	737	1.04	2.60				G
14.56	LVL G 540 9s	9	540	2.75	6.88				G	xxx							G	xxx							G	xxx							G
15.61	LVL G 600 10s	10	600	3.06	7.65				G	xxx						G	G	xxx							G	xxx							G
16.26	xxx								G	xxx						G	G	xxx							G	xxx							G
17.16	xxx								G	xxx						G	G	xxx							G	xxx							G
17.66	xxx								G	xxx						G	G	xxx							G	xxx							G

Table 59. Long-span deck systems with required minimum thickness for various span with governing requirement (reduced fire-safety requirements).

	Solid CLT Panel									CLT Open Rib Panel								CLT Closed Rib Panel							
L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[-]	[-]	[mm]	[kN/m^2]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	xxx							G	G	360	480	1.18	2.94				G	200	440	1.42	3.56				G
13.70										450	570	1.32	3.30				G	300	540	1.54	3.84				G
14.56										500	620	1.40	3.50				G	360	600	1.60	4.01				G
15.61										600	720	1.56	3.90				G	450	690	1.70	4.26				G
16.26										700	820	1.72	4.30				G	500	740	1.76	4.40				G
17.16										700	820	1.72	4.30				G	600	840	1.87	4.68				G
17.66										800	920	1.88	4.70				G	600	840	1.87	4.68				G

	Solid LVL Panel									LVL Open Rib Panel								LVL Semi-open Rib Panel								LVL Closed Rib Panel							
L	$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[-]	[-]	[mm]	[kN/m^2]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	LVL G 480 8s	8	480	2.45	6.13				G																								
13.70	LVL G 540 9s	9	540	2.75	6.88				G																								
14.56	LVL G 540 9s	9	540	2.75	6.88				G																								
15.61	LVL G 600 10s	10	600	3.06	7.65				G																								
16.26	xxx								G																								
17.16	xxx								G																								
17.66	xxx								G																								

Table 60. Long-span deck systems with required minimum thickness for various span with governing requirement (no vibrations requirements).

	Solid CLT Panel								CLT Open Rib Panel								CLT Closed Rib Panel							
L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[-]	[-]	[mm]	[kN/m²]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	xxx							G		300	420	1.08	2.70			G		200	440	1.42	3.56			G	
13.70										360	480	1.18	2.94			G		240	480	1.47	3.67			G	
14.56										400	520	1.24	3.10			G		300	540	1.54	3.84			G	
15.61										450	570	1.32	3.30			G		300	540	1.54	3.84			G	
16.26										500	620	1.40	3.50			G		360	600	1.60	4.01			G	
17.16										500	620	1.40	3.50			G		360	600	1.60	4.01			G	
17.66										600	720	1.56	3.90			G		400	640	1.65	4.12			G	

	Solid LVL Panel									LVL Open Rib Panel							
L	$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Normative				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[-]	[-]	[mm]	[kN/m²]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	LVL G 360 6s	6	360	1.84	4.60			G		500	594	0.76	1.91			G	
13.70	LVL G 420 7s	7	420	2.14	5.35			G		600	694	0.82	2.04			G	
14.56	LVL G 480 8s	8	480	2.45	6.13			G		600	694	0.82	2.04			G	
15.61	LVL G 540 9s	9	540	2.75	6.88			G		xxx						G	
16.26	LVL G 540 9s	9	540	2.75	6.88			G		xxx						G	
17.16	LVL G 600 10s	10	600	3.06	7.65			G		xxx						G	
17.66	LVL G 600 10s	10	600	3.06	7.65			G		xxx						G	

	LVL Semi-open Rib Panel								LVL Closed Rib Panel							
L	h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	360	497	0.82	2.06			G		360	497	0.91	2.28			G	
13.70	450	587	0.87	2.18			G		400	537	0.93	2.34			G	
14.56	450	587	0.87	2.18			G		450	587	0.96	2.40			G	
15.61	600	737	0.95	2.38			G		500	637	0.99	2.47			G	
16.26	600	737	0.95	2.38			G		500	637	0.99	2.47			G	
17.16	600	737	0.95	2.38			G		600	737	1.04	2.60			G	
17.66	600	737	0.95	2.38			G		600	737	1.04	2.60			G	

Table 61. Long-span deck systems with required minimum thickness for various span with governing requirement (reduced serviceability limit requirements).

	Solid CLT Panel									CLT Open Rib Panel								CLT Closed Rib Panel							
L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[-]	[-]	[mm]	[kN/m²]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	CLT 300 L8s-2	8	300	1.5	3.75			G		240	360	0.98	2.46			G		200	440	1.42	3.56			G	
13.70	xxx									300	420	1.08	2.70			G		200	440	1.42	3.56			G	
14.56										300	420	1.08	2.70			G		200	440	1.42	3.56			G	
15.61										360	480	1.18	2.94			G		200	440	1.42	3.56			G	
16.26										360	480	1.18	2.94			G		240	480	1.47	3.67			G	
17.16										400	520	1.24	3.10			G		240	480	1.47	3.67			G	
17.66										400	520	1.24	3.10			G		300	540	1.54	3.84			G	

	Solid LVL Panel									LVL Open Rib Panel							
L	$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Normative				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[-]	[-]	[mm]	[kN/m²]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	LVL G 294 7s	7	294	1.50	3.75			G		400	494	0.71	1.77			G	
13.70	LVL G 336 7s	7	336	1.71	4.28			G		450	544	0.74	1.84			G	
14.56	LVL G 360 6s	6	360	1.84	4.60			G		500	594	0.76	1.91			G	
15.61	LVL G 420 7s	7	420	2.14	5.35			G		500	594	0.76	1.91			G	
16.26	LVL G 420 7s	7	420	2.14	5.35			G		600	694	0.82	2.04			G	
17.16	LVL G 480 8s	8	480	2.45	6.13			G		600	694	0.82	2.04			G	
17.66	LVL G 480 8s	8	480	2.45	6.13			G		600	694	0.82	2.04			G	

	LVL Semi-open Rib Panel								LVL Closed Rib Panel							
L	h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
[m]	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	300	437	0.79	1.98			G		200	337	0.83	2.07	G		G	
13.70	360	497	0.82	2.06			G		300	437	0.88	2.20			G	
14.56	360	497	0.82	2.06			G		360	497	0.91	2.28			G	
15.61	400	537	0.84	2.11			G		360	497	0.91	2.28			G	
16.26	450	587	0.87	2.18			G		400	537	0.93	2.34			G	
17.16	450	587	0.87	2.18			G		450	587	0.96	2.40			G	
17.66	500	637	0.90	2.24			G		450	587	0.96	2.40			G	

Table 62. Long-span deck systems with required minimum thickness for various span with governing requirement (reduced fire-safety and serviceability limit requirements).

L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
	Solid CLT Panel									**CLT Open Rib Panel**								**CLT Closed Rib Panel**							
[m]	[-]	[-]	[mm]	[kN/m^2]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	CLT 300 L8s-2	8	300	1.5	3.75			G		240	360	0.98	2.46			G		200	440	1.42	3.56			G	
13.70	xxx									300	420	1.08	2.70			G		200	440	1.42	3.56			G	
14.56										300	420	1.08	2.70			G		200	440	1.42	3.56			G	
15.61										360	480	1.18	2.94			G		200	440	1.42	3.56			G	
16.26										360	480	1.18	2.94			G		240	480	1.47	3.67			G	
17.16										400	520	1.24	3.10			G		240	480	1.47	3.67			G	
17.66										400	520	1.24	3.10			G		300	540	1.54	3.84			G	

L	$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Normative				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing				h_{Rib}	h_{tot}	$g_{0,k}$	$g_{0,k,tot(2500mm)}$	Governing			
	Solid LVL Panel									**LVL Open Rib Panel**								**LVL Semi-open Rib Panel**								**LVL Closed Rib Panel**							
[m]	[-]	[-]	[mm]	[kN/m^2]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations	[mm]	[mm]	[kN/m]	[kN/m]	ULS	ULS Fire	SLS	Vibrations
12.18	LVL G 294 7s	7	294	1.50	3.75			G		400	479	0.56	1.40			G		300	422	0.64	1.60			G		200	322	0.68	1.69	G		G	
13.70	LVL G 336 7s	7	336	1.71	4.28			G		450	529	0.59	1.46			G		360	482	0.67	1.68			G		300	422	0.73	1.83			G	
14.56	LVL G 360 6s	6	360	1.84	4.60			G		500	579	0.61	1.53			G		360	482	0.67	1.68			G		360	482	0.76	1.91			G	
15.61	LVL G 420 7s	7	420	2.14	5.35			G		500	579	0.61	1.53			G		400	522	0.69	1.73			G		360	482	0.76	1.91			G	
16.26	LVL G 420 7s	7	420	2.14	5.35			G		600	679	0.67	1.66			G		450	572	0.72	1.80			G		400	522	0.78	1.96			G	
17.16	LVL G 480 8s	8	480	2.45	6.13			G		600	679	0.67	1.66			G		450	572	0.72	1.80			G		450	572	0.81	2.03			G	
17.66	LVL G 480 8s	8	480	2.45	6.13			G		600	679	0.67	1.66			G		500	622	0.75	1.87			G		450	572	0.81	2.03			G	

F.2 Short-span Deck Systems

F.2.1 Example Calculations

Example Calculation CLT Deck

Multi-storey car park_v2

2x5.0m span

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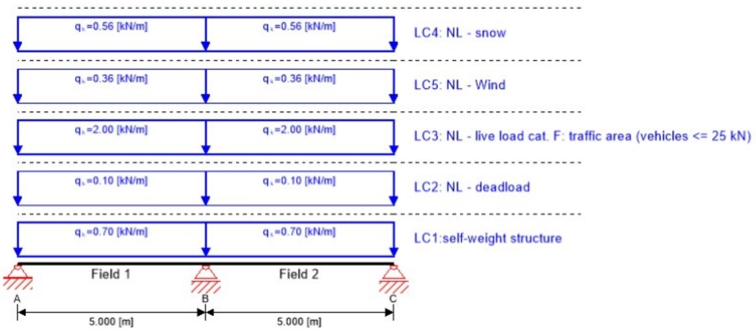
Paul Brink	Designer PB	Checker

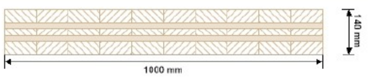
System



Global utilization ratio 84 %

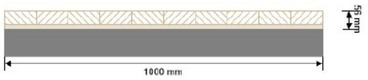
ULS	24 %	ULS Fire	77 %	SLS	54 %	SLS Vibration	84 %	Support	-1 %

Section: CLT 140 L5s



Layer	Thickness	Orientation	Material
1	40.0 mm	0°	C24 spruce ETA (2019)
2	20.0 mm	90°	C24 spruce ETA (2019)
3	20.0 mm	0°	C24 spruce ETA (2019)
4	20.0 mm	90°	C24 spruce ETA (2019)
5	40.0 mm	0°	C24 spruce ETA (2019)
t_{CLT}	**140.0 mm**		

Section Fire: CLT 140 L5s



Layer	Thickness	Orientation	Material
1	40.0 mm	0°	C24 spruce ETA (2019)
2	16.0 mm	90°	C24 spruce ETA (2019)
t_{CLT}	**56.0 mm**		

Fire resistance class:R 90
Fire protection layering : no additional fire protection

Time **90 min**

k_0 [-]	d_0 [mm]	$d_{char,0,h}$ [mm]	$d_{ef,h}$ [mm]	$d_{char,0,v}$ [mm]	$d_{ef,v}$ [mm]
1	7	77.0	84.0	0.0	0.0

Material values

Material	$f_{m,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	$f_{t,90,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{c,90,k}$ [N/mm²]	$f_{v,k}$ [N/mm²]	$f_{r,k\ min}$ [N/mm²]	$E_{0,mean}$ [N/mm²]	G_{mean} [N/mm²]	$G_{r,mean}$ [N/mm²]
C24 spruce ETA (2019)	24.00	14.00	0.12	21.00	2.50	4.00	1.25	12,000.00	690.00	50.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	0.70
2	0.70

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10
2	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00
2	2.00

LC4: NL - snow

continuous load

Field	Load at start [kN/m]
1	0.56
2	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36
2	0.36

Multi-storey car park_v2

2x5.0m span

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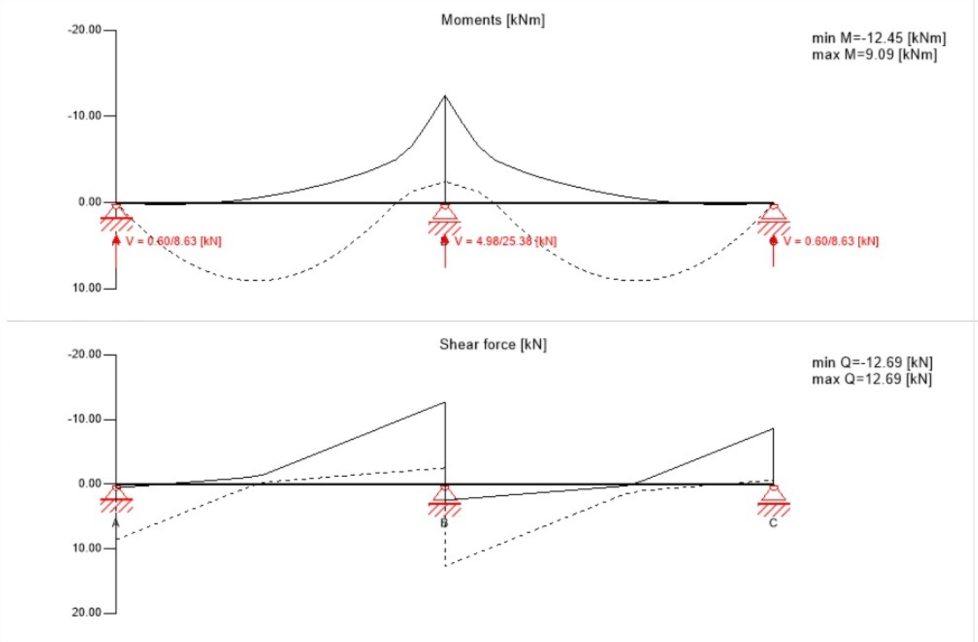
Ultimate limit state (ULS) - design results



ULS Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	$f_{m,y,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$\sigma_{m,y,d}$ [N/mm²]	Ratio	
1	5.0	24.00	1.25	0.80	1.10	16.90	-12.45	4.12	24 %	LCO2
2	0.0	24.00	1.25	0.80	1.10	16.90	-12.45	4.12	24 %	LCO2

ULS Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	5.0	4.00	1.25	0.80	2.56	-12.69	0.12	5 %	LCO2
2	0.0	4.00	1.25	0.80	2.56	12.69	0.12	5 %	LCO2

ULS Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	5.0	1.25	1.25	0.80	0.80	-12.69	0.12	15 %	LCO2
2	0.0	1.25	1.25	0.80	0.80	12.69	0.12	15 %	LCO2

Multi-storey car park_v2
2x5.0m span
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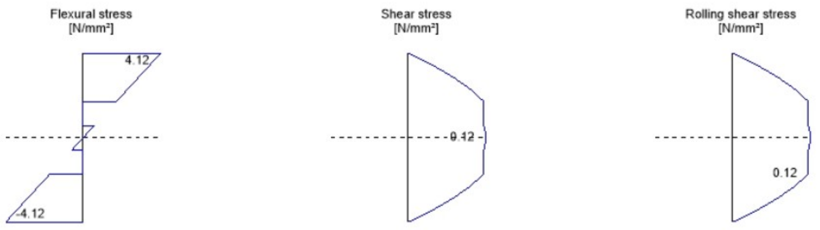
Stress diagram



Flexural stress analysis

$M_{y,d}$ =	-12.45 kNm		$f_{m,k}$ =	24.00 N/mm²	
$M_{z,d}$ =	0.00 kNm		$f_{m,k,z}$ =	24.00 N/mm²	
$N_{t,d}$ =	0.00 kN		γ_m =	1.25 -	
			k_{mod} =	0.80 -	
			$k_{sys,y}$ =	1.10 -	
			$k_{h,m,y}$ =	1.00 -	
			$k_{h,m,z}$ =	1.00 -	
			k_l =	1.00 -	
$\sigma_{t,d}$ =	0.00 N/mm²		$f_{t,0,d}$ =	8.96 N/mm²	
$\sigma_{m,y,d}$ =	4.12 N/mm²		$f_{m,y,d}$ =	16.90 N/mm²	
$\sigma_{m,z,d}$ =	0.00 N/mm²	<	$f_{m,z,d}$ =	0.00 N/mm²	✓
Utilization ratio					24 %

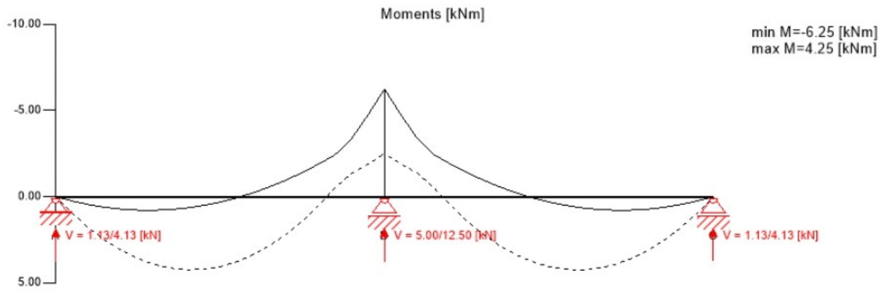
Shear stress analysis

V_d =	-12.69 kN		$f_{v,k}$ =	4.00 N/mm²	
			γ_m =	1.25 -	
			k_{mod} =	0.80 -	
			$k_{h,v}$ =	0.00 -	
$\tau_{v,d}$ =	0.12 N/mm²	<	$f_{v,d}$ =	2.56 N/mm²	✓
Utilization ratio					5 %

Rolling shear analysis

V_d =	-12.69 kN		$f_{r,k}$ =	1.25 N/mm²	
			γ_m =	1.25 -	
			k_{mod} =	0.80 -	
$\tau_{r,d}$ =	0.12 N/mm²	<	$f_{r,d}$ =	0.80 N/mm²	✓
Utilization ratio					15 %

Ultimate limit state (ULS) fire design - results



Multi-storey car park_v2

2x5.0m span

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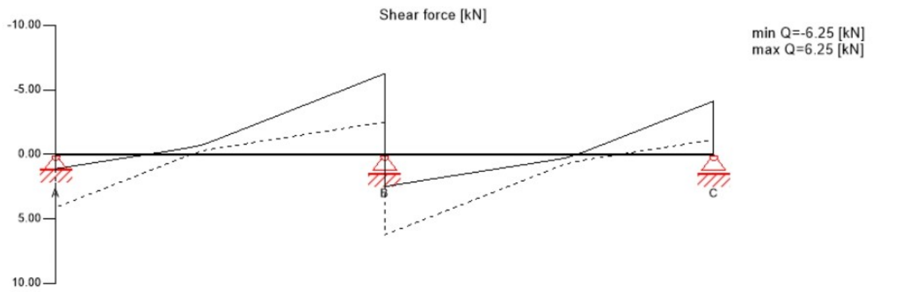
Ultimate limit state (ULS) fire design - results



ULS Fire Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,y}$ [-]	k_{fi} [-]	$f_{m,y,d}$ [N/mm²]	$M_{y,d}$ [kNm]	$\sigma_{m,y,d}$ [N/mm²]	Ratio	
1	5.0	24.00	1.00	1.00	1.10	1.15	30.36	-6.25	23.43	77 %	LCO12
2	0.0	24.00	1.00	1.00	1.10	1.15	30.36	-6.25	23.43	77 %	LCO12

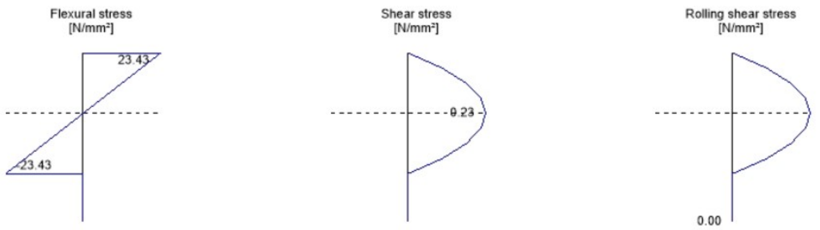
ULS Fire Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	5.0	4.00	1.00	1.00	1.15	4.60	-6.25	0.23	5 %	LCO12
2	0.0	4.00	1.00	1.00	1.15	4.60	6.25	0.23	5 %	LCO12

ULS Fire Rolling shear

Field	Dist. [m]	$f_{r,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$f_{r,d}$ [N/mm²]	V_d [kN]	$\tau_{r,d}$ [N/mm²]	Ratio	
1	5.0	1.25	1.00	1.00	1.15	1.44	-6.25	0.00	0 %	LCO20
2	0.0	1.25	1.00	1.00	1.15	1.44	6.25	0.00	0 %	LCO20

Stress diagram



Flexural stress analysis Fire

$M_{y,d}$ =	-6.25	kNm		$f_{m,k}$ =	24.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	24.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{sys,y}$ =	1.10	-	
				$k_{h,m,y}$ =	1.00	-	
				$k_{h,m,z}$ =	1.00	-	
				k_l =	1.00	-	
				k_{fi} =	1.15	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	16.10	N/mm²	
$\sigma_{m,y,d}$ =	23.43	N/mm²		$f_{m,y,d}$ =	30.36	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	0.00	N/mm²	✓
Utilization ratio						77 %	

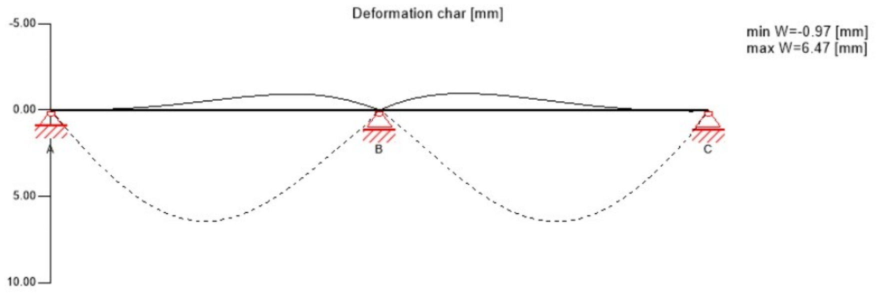
Shear stress analysis Fire

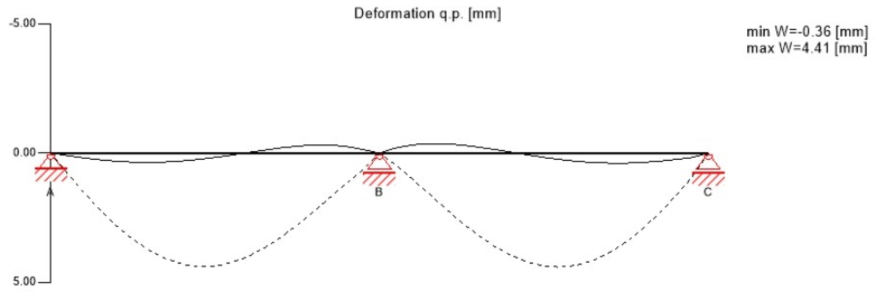
V_d =	-6.25	kN		$f_{v,k}$ =	4.00	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				$k_{h,v}$ =	0.00	-	
				k_{fi} =	1.15	-	
$\tau_{v,d}$ =	0.23	N/mm²	<	$f_{v,d}$ =	4.60	N/mm²	✓
Utilization ratio						5 %	

Rolling shear analysis Fire

V_d =	-6.25	kN		$f_{r,k}$ =	1.25	N/mm²	
				γ_m =	1.00	-	
				k_{mod} =	1.00	-	
				k_{fi} =	1.15	-	
$\tau_{r,d}$ =	0.00	N/mm²	<	$f_{r,d}$ =	1.44	N/mm²	✓
Utilization ratio						0 %	

Service limit state design (SLS) - design results





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w_{inst} = w[char]

Field	K_{def}	Limit	w_{limit}	$w_{calc.}$	Ratio
		[-]	[mm]	[mm]	
1	1	L/1	5000.0	6.4	0 %
2	1	L/1	5000.0	6.5	0 %

w_{fin} = w[char] + w[q.p.]*kdef

Field	K_{def}	Limit	w_{limit}	$w_{calc.}$	Ratio
		[-]	[mm]	[mm]	
1	1	L/250	20.0	10.8	54 %
2	1	L/250	20.0	10.9	54 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

Field	K_{def}	Limit	w_{limit}	$w_{calc.}$	Ratio
		[-]	[mm]	[mm]	
1	1	L/250	20.0	8.8	44 %
2	1	L/250	20.0	8.8	44 %

Vibration analysis

General		
Total mass	9.93	[t]
Tributary width	2.4	[m]
Stiffness Longitudinal direction	2536.0	[kNm²]
Stiffness Cross direction	208.0	[kNm²]
Modal damping	2.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	990.2	[kg]

Analysis				
Criterion	Calc.	Class II	Class II	Cl. II
Frequency criterion min	11.083 [Hz]	4.5 [Hz]	41 %	✓
Frequency criterion	11.083 [Hz]	6.0 [Hz]	54 %	
Acceleration criterion	0.084 [m/s²]	0.1 [m/s²]	84 %	
Stiffness criterion	0.422 [mm]	0.5 [mm]	84 %	✓

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
ETA-14/0349	European Technical Assessment ETA-14/0349 of 02.10.2014
Expertise Rolling shear - no edge gluing, H.J. Blass	Expertise on Rolling shear for CLT
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
Technical expertise 122/2011/02: analysis of load bearing capacity and separation performance of CLT elements	Verification of the load bearing capacity and the insulation criterion of CLT structures with Stora Enso CLT
Technical expertise 2434/2012 - BB: failure time tf of gypsum fire boards (GKF) according to ON B 3410	Expertise on failure time tf of gypsum wall fire boards according to ON B3410 and gypsum wall boards type DF according to EN 520
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General-Common rules and rules for buildings
ÖNorm B 1995-1-2 NA	ÖNORM EN 1995-1-2 - Austria - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements
Fire safety in timber buildings - technical guildeline for Europe	Fire safety in timber buildings - technical guideline for Europe; publishes by SP Technical Research Institute of Sweden
National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12	ÖNORM EN 1995-1-2 - National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements, chapter 12
Expertise Rolling shear, H.J. Blass	Expertise on rolling shear strength and rolling shear modulus of CLT panels

Example Calculation LVL Deck

Multi-storey car park_v2
2x5.0m span
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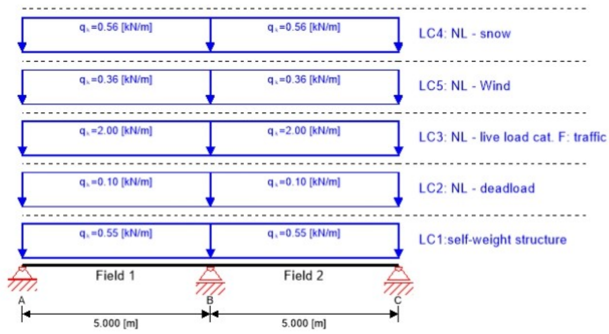
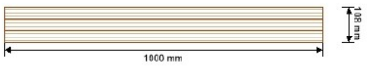
System



Global utilization ratio 98 %

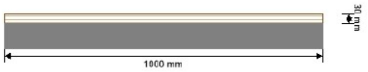
ULS	31 %	ULS Fire	98 %	SLS	98 %	SLS Vibration	35 %	Support	-1 %	Void	-1 %

Section: LVL G-X flatwise 100/10.8



Layer	Thickness	Orientation	Material
1	36.0 mm	90°	LVL G-X flatwise
2	36.0 mm	90°	LVL G-X flatwise
3	36.0 mm	90°	LVL G-X flatwise
t_{CLT}	**108.0 mm**		

Section Fire: LVL G-X flatwise 100/10.8



Layer	Thickness	Orientation	Material
1	30.0 mm	90°	LVL G-X flatwise
t_{CLT}	**30.0 mm**		

Fire resistance class:R 90
Fire protection layering : no additional fire protection

Time **90 min**

k_0	d_0	$d_{char,0,h}$	$d_{ef,h}$	$d_{char,0,v}$	$d_{ef,v}$
[-]	[mm]	[mm]	[mm]	[mm]	[mm]
1	7	71.0	78.0	58.5	0.0

Material values

Material	$f_{m,k}$	$f_{t,0,k}$	$f_{t,90,k}$	$f_{c,0,k}$	$f_{c,90,k}$	$f_{v,k}$	$f_{r,k\ min}$	$E_{0,mean}$	G_{mean}	$E_{0,5}$
	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]	[N/mm²]
LVL G-X flatwise	31.00	26.00	0.00	26.00	2.20	1.30	0.60	10,100.00	120.00	8,800.00
LVL G-X edgewise	32.00	26.00	6.00	26.00	9.00	4.50	0.00	10,500.00	600.00	8,800.00

Load

Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC1	self-weight structure	G	permanent	0.6	1	1.35	1	1	1
LC2	NL - deadload	G	permanent	0.6	1	1.35	1	1	1

Multi-storey car park_v2
2x5.0m span
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Load case groups

	Load case category	Type	Duration	Kmod	γ_{inf}	γ_{sup}	Ψ_0	Ψ_1	Ψ_2
LC3	NL - live load cat. F: traffic area (vehicles <= 25 kN)	Q	medium term	0.8	0	1.5	0.7	0.7	0.6
LC4	NL - snow	Q	short term	0.9	0	1.5	0	0.2	0
LC5	NL - Wind	Q	short term	0.9	0	1.5	0	0.2	0

LC1:self-weight structure

continuous load

Field	Load at start [kN/m]
1	0.55
2	0.55

LC2: NL - deadload

continuous load

Field	Load at start [kN/m]
1	0.10
2	0.10

LC3: NL - live load cat. F: traffic area (vehicles <= 25 kN)

continuous load

Field	Load at start [kN/m]
1	2.00
2	2.00

LC4: NL - snow

continuous load

Field	Load at start [kN/m]
1	0.56
2	0.56

LC5: NL - Wind

continuous load

Field	Load at start [kN/m]
1	0.36
2	0.36

Multi-storey car park_v2 Page 3/7

2x5.0m span 28.11.2022

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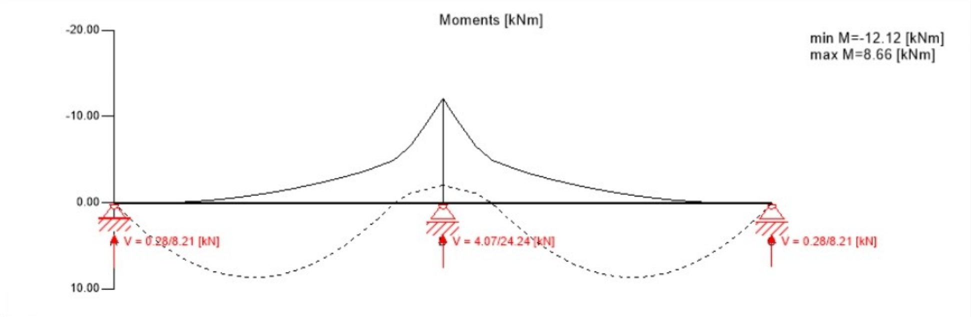
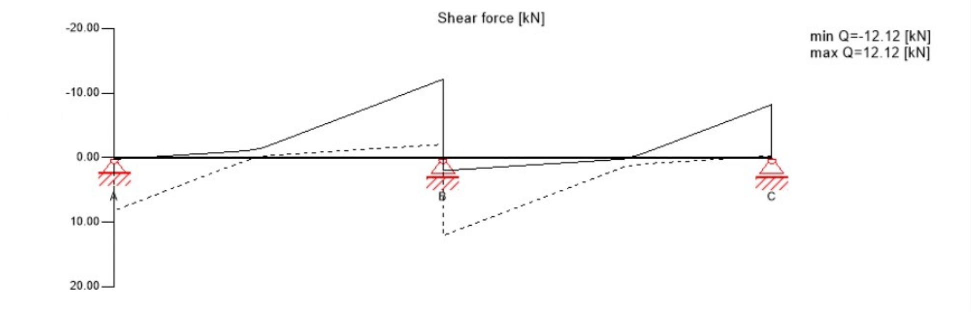
Ultimate limit state (ULS) - design results





ULS Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{sys,z}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]	$f_{m,z,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	5.0	31.00	26.00	26.00	1.20	0.80	1.00	0.98	0.87	0.93	20.18	18.46	16.13	17.33
2	0.0	31.00	26.00	26.00	1.20	0.80	1.00	0.98	0.87	0.93	20.18	18.46	16.13	17.33

Field	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{m,z,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	-12.12	0.00	0.00	0.00	6.23	0.00	0.00	0.00	31 %	LCO2
2	-12.12	0.00	0.00	0.00	6.23	0.00	0.00	0.00	31 %	LCO2

ULS Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{h,v}$ [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	4.89	1.30	1.20	0.80	0.98	0.87	11.70	0.16	19 %	LCO2
2	0.11	1.30	1.20	0.80	0.98	0.87	11.70	0.16	19 %	LCO2

Flexural stress analysis

$M_{y,d}$ =	-12.12	kNm		$f_{m,k}$ =	31.00	N/mm²	
$M_{z,d}$ =	0.00	kNm		$f_{m,k,z}$ =	31.00	N/mm²	
$N_{t,d}$ =	0.00	kN		γ_m =	1.20	-	
				k_{mod} =	0.80	-	
				$k_{sys,y}$ =	1.00	-	
				$k_{h,m,y}$ =	0.98	-	
				$k_{h,m,z}$ =	0.87	-	
				k_l =	0.93	-	
$\sigma_{t,d}$ =	0.00	N/mm²		$f_{t,0,d}$ =	16.13	N/mm²	
$\sigma_{m,y,d}$ =	6.23	N/mm²		$f_{m,y,d}$ =	20.18	N/mm²	
$\sigma_{m,z,d}$ =	0.00	N/mm²	<	$f_{m,z,d}$ =	18.46	N/mm²	✓
Utilization ratio						31 %	

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Multi-storey car park_v2

2x5.0m span

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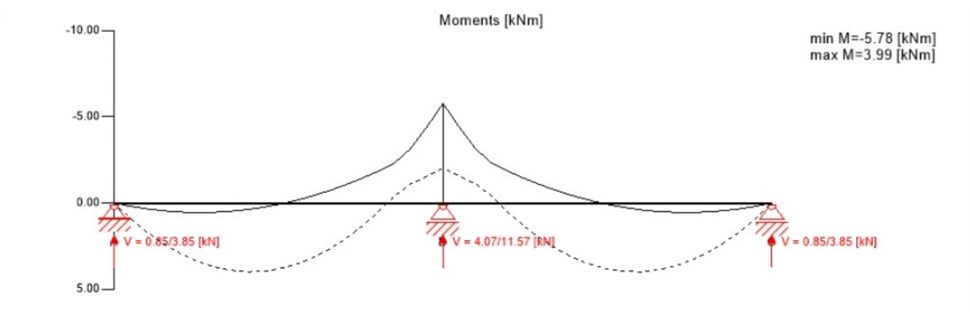
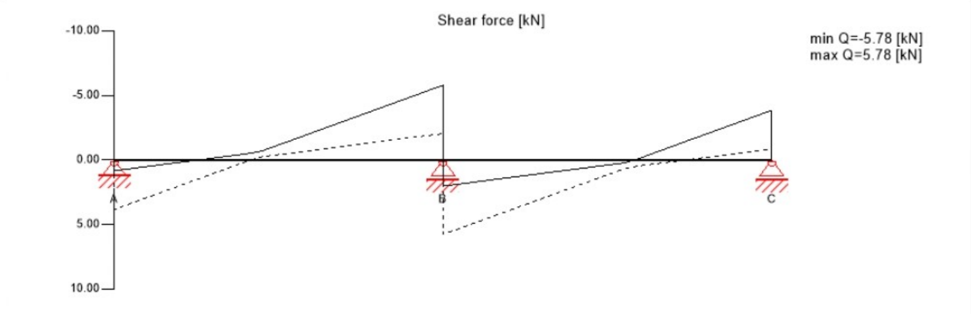
Shear stress analysis

V_d =	11.70 kN	$f_{v,k}$ =	1.30 N/mm²
		γ_m =	1.20 -
		k_{mod} =	0.80 -
		$k_{h,v}$ =	0.98 -
$\tau_{v,d}$ =	0.16 N/mm² <	$f_{v,d}$ =	0.87 N/mm² ✓

Utilization ratio 19 %

Ultimate limit state (ULS) fire design - results





ULS Fire Flexural design

Field	Dist. [m]	$f_{m,k}$ [N/mm²]	$f_{c,0,k}$ [N/mm²]	$f_{t,0,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	k_{fi} [-]	$k_{sys,z}$ [-]	$k_{h,m,y}$ [-]	$k_{h,m,z}$ [-]	k_l [-]	$f_{m,y,d}$ [N/mm²]	$f_{m,z,d}$ [N/mm²]	$f_{t,0,d}$ [N/mm²]	$f_{c,0,d}$ [N/mm²]
1	5.0	31.00	26.00	26.00	1.20	0.80	1.00	1.00	0.98	0.87	0.93	20.18	18.46	16.13	17.33
2	0.0	31.00	26.00	26.00	1.20	0.80	1.00	1.00	0.98	0.87	0.93	20.18	18.46	16.13	17.33

Field	$M_{y,d}$ [kNm]	$M_{z,d}$ [kNm]	$N_{c,d}$ [kN]	$N_{t,d}$ [kN]	$\sigma_{m,y,d}$ [N/mm²]	$\sigma_{m,z,d}$ [N/mm²]	$\sigma_{c,d}$ [N/mm²]	$\sigma_{t,d}$ [N/mm²]	Ratio	
1	-12.12	0.00	0.00	0.00	6.23	0.00	0.00	0.00	31 %	LCO2
2	-12.12	0.00	0.00	0.00	6.23	0.00	0.00	0.00	31 %	LCO2

ULS Fire Shear analysis

Field	Dist. [m]	$f_{v,k}$ [N/mm²]	γ_m [-]	k_{mod} [-]	$k_{h,v}$ [-]	k_{fi} [-]	$f_{v,d}$ [N/mm²]	V_d [kN]	$\tau_{v,d}$ [N/mm²]	Ratio	
1	4.97	1.30	1.00	1.00	1.00	1.10	1.43	5.73	0.29	20 %	LCO12
2	0.03	1.30	1.00	1.00	1.00	1.10	1.43	5.73	0.29	20 %	LCO12

Multi-storey car park_v2

2x5.0m span

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Flexural stress analysis Fire

$M_{y,d}$ =	-5.78 kNm		$f_{m,k}$ =	31.00 N/mm²	
$M_{z,d}$ =	0.00 kNm		$f_{m,k,z}$ =	31.00 N/mm²	
$N_{t,d}$ =	0.00 kN		γ_m =	1.00 -	
			k_{mod} =	1.00 -	
			$k_{sys,y}$ =	1.00 -	
			$k_{h,m,y}$ =	1.15 -	
			$k_{h,m,z}$ =	0.87 -	
			k_l =	0.93 -	
			k_{fi} =	1.10 -	
$\sigma_{t,d}$ =	0.00 N/mm²		$f_{t,0,d}$ =	26.61 N/mm²	
$\sigma_{m,y,d}$ =	38.56 N/mm²		$f_{m,y,d}$ =	39.34 N/mm²	
$\sigma_{m,z,d}$ =	0.00 N/mm²	<	$f_{m,z,d}$ =	30.46 N/mm²	✓

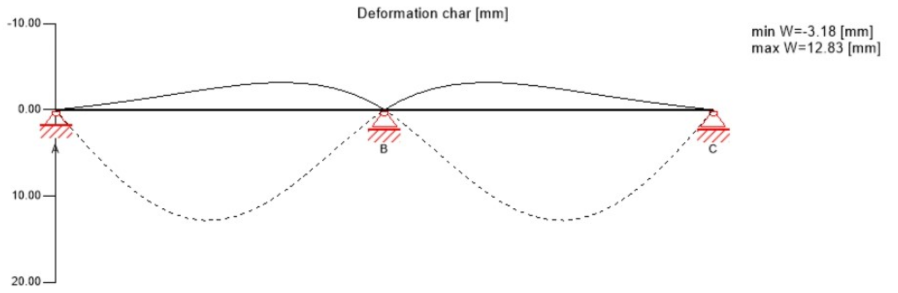
Utilization ratio 98 %

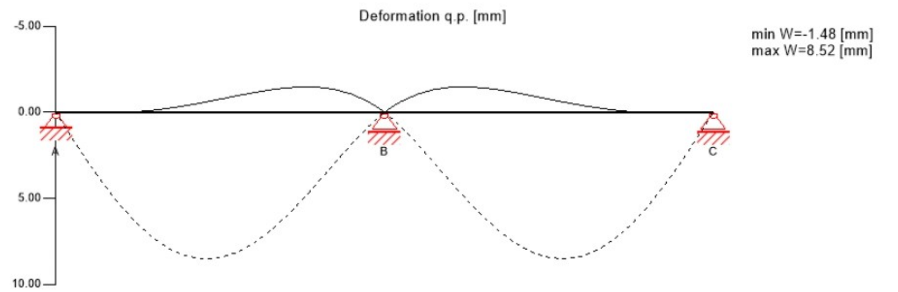
Shear stress analysis Fire

V_d =	5.73 kN		$f_{v,k}$ =	1.30 N/mm²	
			γ_m =	1.00 -	
			k_{mod} =	1.00 -	
			$k_{h,v}$ =	1.00 -	
			k_{fi} =	1.10 -	
$\tau_{v,d}$ =	0.29 N/mm²	<	$f_{v,d}$ =	1.43 N/mm²	✓

Utilization ratio 20 %

Service limit state design (SLS) - design results





w_{inst} = w[char]

Field	k_{def}	Limit [-]	w_{limit} [mm]	$w_{calc.}$ [mm]	Ratio
1	0.8	L/1	5000.0	12.8	0 %
2	0.8	L/1	5000.0	12.8	0 %

Multi-storey car park_v2		Page 6/7
2x5.0m span		28.11.2022
Paul Brink	Designer PB	Checker

w_{fin} = w[char] + w[q.p.]*kdef

Field	K_{def}	Limit	w_{limit}	$w_{calc.}$	Ratio
		[-]	[mm]	[mm]	
1	0.8	L/250	20.0	19.6	98 %
2	0.8	L/250	20.0	19.6	98 %

$w_{net,fin}$ = w[q.p.] + w[q.p.]*kdef

Field	K_{def}	Limit	w_{limit}	$w_{calc.}$	Ratio
		[-]	[mm]	[mm]	
1	0.8	L/250	20.0	15.3	77 %
2	0.8	L/250	20.0	15.3	77 %

Vibration analysis

General		
Total mass	8.08	[t]
Tributary width	4.2	[m]
Stiffness Longitudinal direction	3560.3	[kNm²]
Stiffness Cross direction	2500.0	[kNm²]
Modal damping	2.0	[%]
α	0.0	[-]
Man weight	700.0	[N]
Modal mass	1377.9	[kg]

Analysis							
Criterion	Calc.	Class I	Class II	Class I	Class II	Cl. I	Cl. II
Frequency criterion min	18.995 [Hz]	4.5 [Hz]	4.5 [Hz]	24 %	24 %	✓	✓
Frequency criterion	18.995 [Hz]	8.0 [Hz]	6.0 [Hz]	42 %	32 %		
Acceleration criterion	0.003 [m/s²]	0.05 [m/s²]	0.1 [m/s²]	5 %	3 %		
Stiffness criterion	0.176 [mm]	0.25 [mm]	0.5 [mm]	70 %	35 %	✓	✓

Reference documents for this analysis

English title	Description
EN 338	EN 338 - Structural timber ? Strength classes
EN 1995-1-1	EN 1995-1-1 - Eurocode 5: Design of timber structures - Part 1-1: General - Common rules and rules for buildings
EN 1995-1-2	EN 1995-1-2 - Eurocode 5 — Design of timber structures — Part 1-2: General — Structural fire design
EN 14080	EN 14080 - Timber Structures - Glued laminated timber and glued solid timber - Requirements
EN 1990	EN 1990 - Eurocode ? Basis of structural design
ÖNorm B 1995-1-1 NA	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings
ÖNorm B 1995-1-2 NA	ÖNORM EN 1995-1-2 - Austria - National Annex - Eurocode 5: Design of timber structures ? Part 1-2: General ? Structural fire design ? National specifications concerning ÖNORM EN 1995-1-2, national comments and national supplements
ÖNORM EN 1995-1-1_NA, chapter 7.3	ÖNORM EN 1995-1-1 - Austria - National Annex – Nationally determined parameters – Eurocode 5: Design of timber structures – Part 1-1: General- Common rules and rules for buildings; chapter 7.3
CERTIFICATE NO. EUFI29-20000564-C	Product certificate
LVL G by Stora Enso_Structural design manual column&beam_ V01	Design manual
ETA 20_0291 LVL G by Stora Enso	ETA

Disclaimer

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F.2.2 Results Minimum Dimensions Short-span Deck Systems

Minimum Thickness LVL Decks for Different Requirements

Table 63. CLT decks with required minimum thickness for various span distances and number of spans with governing requirement (base case requirements).

			BASE CASE							
Name	Nr. sub-spans	L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	Governing			
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m²]	ULS	ULS Fire	SLS	Vibrations
1x2.5m span	1	2.50	CLT 100 L3s	3	100	0.5		G		
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.7		G	G	G
1x7.5m span	1	7.50	CLT 220 L7s - 2	7	220	1.1			G	G
1x10.0m span	1	10.00	CLT 300 L8s	8	300	1.5			G	G
2x2.5m span	2	2.50	CLT 100 L3s	3	100	0.5		G		
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.6		G		G
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.7		G		G
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.8		G	G	G
2x7.5m span	2	7.50	CLT 180 L5s	5	180	0.9			G	G
2x8.75m span	2	8.75	CLT 240 L7s - 2	7	240	1.2				G
3x2.5m span	3	2.50	CLT 100 L3s	3	100	0.5		G		
3x3.75m span	3	3.75	CLT 120 L3s	3	120	0.6				G
3x5.0m span	3	5.00	CLT 140 L5s	5	140	0.7				G
3x6.25m span	3	6.25	CLT 160 L5s	5	160	0.8		G	G	G
4x2.5m span	4	2.50	CLT 100 L3s	3	100	0.5		G		
4x3.75m span	4	3.75	CLT 120 L3s	3	120	0.6				G
4x5.0m span	4	5.00	CLT 140 L5s	5	140	0.7				G
5x2.5m span	5	2.50	CLT 100 L3s	3	100	0.5		G		
5x3.75m span	5	3.75	CLT 120 L5s	5	120	0.6				G
6x2.5m span	6	2.50	CLT 100 L3s	3	100	0.5		G		

Table 64. CLT decks with required minimum thickness for various span distances and number of spans with governing requirement (reduced fire-safety requirements).

			Fire to 30 min.							
Name	**Nr. sub-spans**	**L**	**Name$_{CLT}$**	**CLT layers**	**h$_{CLT}$**	**$g_{0,k}$**	**Governing**			
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**	**ULS**	**ULS Fire**	**SLS**	**Vibrations**
1x2.5m span	1	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				**G**
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.70			G	G
1x7.5m span	1	7.50	CLT 220 L7s - 2	7	220	1.10			G	
1x10.0m span	1	10.00	CLT 300 L8s	8	300	1.50			G	G
2x2.5m span	2	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				**G**
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.60				G
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.70				G
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.80			G	G
2x7.5m span	2	7.50	CLT 180 L5s	5	180	0.90			G	G
2x8.75m span	2	8.75	CLT 240 L7s - 2	7	240	1.20				G
3x2.5m span	3	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				**G**
3x3.75m span	3	3.75	CLT 120 L3s	3	120	0.60				G
3x5.0m span	3	5.00	CLT 140 L5s	5	140	0.70				G
3x6.25m span	3	6.25	CLT 160 L5s	5	160	0.80			G	G
4x2.5m span	4	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				**G**
4x3.75m span	4	3.75	CLT 120 L3s	3	120	0.60				G
4x5.0m span	4	5.00	CLT 140 L5s	5	140	0.70				G
5x2.5m span	5	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				**G**
5x3.75m span	5	3.75	CLT 120 L5s	5	120	0.60				G
6x2.5m span	6	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				**G**

Table 65. CLT decks with required minimum thickness for various span distances and number of spans with governing requirement (no vibrations requirements).

			No vibrations check							
Name	**Nr. sub-spans**	**L**	**Name$_{CLT}$**	**CLT layers**	**h$_{CLT}$**	**$g_{0,k}$**	**Governing**			
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**	**ULS**	**ULS Fire**	**SLS**	**Vibrations**
1x2.5m span	1	2.50	CLT 100 L3s	3	100	0.50		G		
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.70		G	G	
1x7.5m span	1	7.50	CLT 220 L7s - 2	7	220	1.10			G	
1x10.0m span	1	10.00	CLT 300 L8s	8	300	1.50			G	
2x2.5m span	2	2.50	CLT 100 L3s	3	100	0.50		G		
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.60		G		
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.70		G		
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.80		G	G	
2x7.5m span	2	7.50	CLT 180 L5s	5	180	0.90			G	
2x8.75m span	2	8.75	**CLT 220 L7s - 2**	**7**	**220**	**1.10**			**G**	
3x2.5m span	3	2.50	CLT 100 L3s	3	100	0.50		G		
3x3.75m span	3	3.75	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**		
3x5.0m span	3	5.00	**CLT 120 L3s**	**3**	**120**	**0.60**		**G**	**G**	
3x6.25m span	3	6.25	CLT 160 L5s	5	160	0.80		G	G	
4x2.5m span	4	2.50	CLT 100 L3s	3	100	0.50		G		
4x3.75m span	4	3.75	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**		
4x5.0m span	4	5.00	**CLT 120 L3s**	**3**	**120**	**0.60**		**G**	**G**	
5x2.5m span	5	2.50	CLT 100 L3s	3	100	0.50		G		
5x3.75m span	5	3.75	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**		
6x2.5m span	6	2.50	CLT 100 L3s	3	100	0.50		G		

Table 66. CLT decks with required minimum thickness for various span distances and number of spans with governing requirement (reduced serviceability limit requirements).

			SLS to 1/150 & no vibrations check							
Name	**Nr. sub-spans**	**L**	**Name$_{CLT}$**	**CLT layers**	**h$_{CLT}$**	**$g_{0,k}$**	**Governing**			
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**	**ULS**	**ULS Fire**	**SLS**	**Vibrations**
1x2.5m span	1	2.50	CLT 100 L3s	3	100	0.50		G		
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.70		G	G	
1x7.5m span	1	7.50	**CLT 180 L7s**	**7**	**180**	**0.90**			**G**	
1x10.0m span	1	10.00	**CLT 240 L7s - 2**	**7**	**240**	**1.20**			**G**	
2x2.5m span	2	2.50	CLT 100 L3s	3	100	0.50		G		
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.60		G		
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.70		G		
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.80		G	G	
2x7.5m span	2	7.50	**CLT 160 L5s - 2**	**5**	**160**	**0.80**		**G**	**G**	
2x8.75m span	2	8.75	**CLT 180 L7s**	**7**	**180**	**0.90**		**G**	**G**	
3x2.5m span	3	2.50	CLT 100 L3s	3	100	0.50		G		
3x3.75m span	3	3.75	CLT 110 L3s	3	110	0.55		G		
3x5.0m span	3	5.00	CLT 120 L3s	3	120	0.60		G		
3x6.25m span	3	6.25	CLT 160 L5s	5	160	0.80		G	G	
4x2.5m span	4	2.50	CLT 100 L3s	3	100	0.50		G		
4x3.75m span	4	3.75	CLT 110 L3s	3	110	0.55		G		
4x5.0m span	4	5.00	CLT 120 L3s	3	120	0.60		G		
5x2.5m span	5	2.50	CLT 100 L3s	3	100	0.50		G		
5x3.75m span	5	3.75	CLT 110 L3s	3	110	0.55		G		
6x2.5m span	6	2.50	CLT 100 L3s	3	100	0.50		G		

Table 67. CLT decks with required minimum thickness for various span distances and number of spans with governing requirement (reduced fire-safety and serviceability limit requirements).

			Fire to 30 min., SLS to 1/150 & no vibrations check								
Name	**Nr. sub-spans**	**L**	**Name$_{CLT}$**	**CLT layers**	**h$_{CLT}$**	**g$_{0,k}$**	**Governing**				
							ULS	**ULS Fire**	**SLS**	**Vibrations**	
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m²]**					
1x2.5m span	1	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					
1x5.0m span	1	5.00	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**	**G**		
1x7.5m span	1	7.50	CLT 180 L7s	7	180	0.90			G		
1x10.0m span	1	10.00	CLT 240 L7s - 2	7	240	1.20			G		
2x2.5m span	2	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					
2x3.75m span	2	3.75	**CLT 90 L3s**	**3**	**90**	**0.45**		**G**			
2x5.0m span	2	5.00	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**			
2x6.25m span	2	6.25	**CLT 120 L5s**	**5**	**120**	**0.60**			**G**		
2x7.5m span	2	7.50	CLT 160 L5s - 2	5	160	0.80		G	G		
2x8.75m span	2	8.75	CLT 180 L7s	7	180	0.90		G	G		
3x2.5m span	3	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					
3x3.75m span	3	3.75	**CLT 90 L3s**	**3**	**90**	**0.45**		**G**			
3x5.0m span	3	5.00	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**			
3x6.25m span	3	6.25	**CLT 120 L3s**	**3**	**120**	**0.60**			**G**		
4x2.5m span	4	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					
4x3.75m span	4	3.75	**CLT 90 L3s**	**3**	**90**	**0.45**		**G**			
4x5.0m span	4	5.00	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**			
5x2.5m span	5	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					
5x3.75m span	5	3.75	**CLT 90 L3s**	**3**	**90**	**0.45**		**G**			
6x2.5m span	6	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					

Minimum Thickness LVL Decks for Different Requirements

Table 68. LVL decks with required minimum thickness for various span distances and number of spans with governing requirement (base case requirements).

			BASE CASE							
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h$_{LVL}$**	**$g_{0,k}$**	**Normative**			
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m²]	**ULS**	**ULS Fire**	**SLS**	**Vibrations**
1x2.5m span	1	2.50	LVL G 96 2s	2	96	0.49		G		
1x5.0m span	1	5.00	LVL G 144 3 s	3	144	0.73			G	
1x7.5m span	1	7.50	LVL G 210 5s	5	210	1.07			G	
1x10.0m span	1	10.00	LVL G 294 7s	7	294	1.50				G
2x2.5m span	2	2.50	LVL G 96 2s	2	96	0.49		G		
2x3.75m span	2	3.75	LVL G 108 3s	3	108	0.55		G		
2x5.0m span	2	5.00	LVL G 108 3s	3	108	0.55		G	G	
2x6.25m span	2	6.25	LVL G 144 3s	3	144	0.73			G	
2x7.5m span	2	7.50	LVL G 168 4s	4	168	0.86			G	
2x8.75m span	2	8.75	LVL G 210 5s	5	210	1.07			G	
3x2.5m span	3	2.50	LVL G 96 2s	2	96	0.49		G		
3x3.75m span	3	3.75	LVL G 108 3s	3	108	0.55		G		
3x5.0m span	3	5.00	LVL G 120 2s	2	120	0.61			G	
3x6.25m span	3	6.25	LVL G 144 3s	3	144	0.73			G	
4x2.5m span	4	2.50	LVL G 96 2s	2	96	0.49		G		
4x3.75m span	4	3.75	LVL G 108 3s	3	108	0.55		G		
4x5.0m span	4	5.00	LVL G 120 2s	2	120	0.61			G	
5x2.5m span	5	2.50	LVL G 96 2s	2	96	0.49		G		
5x3.75m span	5	3.75	LVL G 108 3s	3	108	0.55		G		
6x2.5m span	6	2.50	LVL G 96 2s	2	96	0.49		G		

Table 69. LVL decks with required minimum thickness for various span distances and number of spans with governing requirement (reduced fire-safety requirements).

			Fire to 30 min.							
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h$_{LVL}$**	**$g_{0,k}$**	**Normative**			
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**	**ULS**	**ULS Fire**	**SLS**	**Vibrations**
1x2.5m span	1	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
1x5.0m span	1	5.00	LVL G 144 3 s	3	144	0.73			G	
1x7.5m span	1	7.50	LVL G 210 5s	5	210	1.07			G	
1x10.0m span	1	10.00	LVL G 294 7s	7	294	1.50				G
2x2.5m span	2	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
2x3.75m span	2	3.75	**LVL G 84 2s**	**2**	**84**	**0.43**			**G**	
2x5.0m span	2	5.00	LVL G 108 3s	3	108	0.55		G	G	
2x6.25m span	2	6.25	LVL G 144 3s	3	144	0.73			G	
2x7.5m span	2	7.50	LVL G 168 4s	4	168	0.86			G	
2x8.75m span	2	8.75	LVL G 210 5s	5	210	1.07			G	
3x2.5m span	3	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
3x3.75m span	3	3.75	**LVL G 84 2s**	**2**	**84**	**0.43**			**G**	
3x5.0m span	3	5.00	LVL G 120 2s	2	120	0.61			G	
3x6.25m span	3	6.25	LVL G 144 3s	3	144	0.73			G	
4x2.5m span	4	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
4x3.75m span	4	3.75	**LVL G 84 2s**	**2**	**84**	**0.43**			**G**	
4x5.0m span	4	5.00	LVL G 120 2s	2	120	0.61			G	
5x2.5m span	5	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
5x3.75m span	5	3.75	**LVL G 84 2s**	**2**	**84**	**0.43**			**G**	
6x2.5m span	6	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				

Table 70. LVL decks with required minimum thickness for various span distances and number of spans with governing requirement (no vibrations requirements).

			No vibrations check							
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h_{LVL}**	**$g_{0,k}$**	**Normative**			
							ULS	**ULS Fire**	**SLS**	**Vibrations**
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**				
1x2.5m span	1	2.50	LVL G 96 2s	2	96	0.49		G		
1x5.0m span	1	5.00	LVL G 144 3 s	3	144	0.73			G	
1x7.5m span	1	7.50	LVL G 210 5s	5	210	1.07			G	
1x10.0m span	1	10.00	**LVL G 288 6s**	**6**	**288**	**1.47**			**G**	
2x2.5m span	2	2.50	LVL G 96 2s	2	96	0.49		G		
2x3.75m span	2	3.75	LVL G 108 3s	3	108	0.55		G		
2x5.0m span	2	5.00	LVL G 108 3s	3	108	0.55		G	G	
2x6.25m span	2	6.25	LVL G 144 3s	3	144	0.73			G	
2x7.5m span	2	7.50	LVL G 168 4s	4	168	0.86			G	
2x8.75m span	2	8.75	LVL G 210 5s	5	210	1.07			G	
3x2.5m span	3	2.50	LVL G 96 2s	2	96	0.49		G		
3x3.75m span	3	3.75	LVL G 108 3s	3	108	0.55		G		
3x5.0m span	3	5.00	LVL G 120 2s	2	120	0.61			G	
3x6.25m span	3	6.25	LVL G 144 3s	3	144	0.73			G	
4x2.5m span	4	2.50	LVL G 96 2s	2	96	0.49		G		
4x3.75m span	4	3.75	LVL G 108 3s	3	108	0.55		G		
4x5.0m span	4	5.00	LVL G 120 2s	2	120	0.61			G	
5x2.5m span	5	2.50	LVL G 96 2s	2	96	0.49		G		
5x3.75m span	5	3.75	LVL G 108 3s	3	108	0.55		G		
6x2.5m span	6	2.50	LVL G 96 2s	2	96	0.49		G		

Table 71. LVL decks with required minimum thickness for various span distances and number of spans with governing requirement (reduced serviceability limit requirements).

			SLS to 1/150 & no vibrations check							
Name	**Nr. sub-spans**	**L**	**Name$_{LVL}$**	**Nr. LVL panels**	**h_{LVL}**	**$g_{0,k}$**	**Normative**			
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m^2]**	**ULS**	**ULS Fire**	**SLS**	**Vibrations**
1x2.5m span	1	2.50	**LVL G 96 2s**	**2**	**96**	**0.49**		**G**		
1x5.0m span	1	5.00	**LVL G 108 3s**	**3**	**108**	**0.55**		**G**	**G**	
1x7.5m span	1	7.50	**LVL G 168 4s**	**4**	**168**	**0.86**			**G**	
1x10.0m span	1	10.00	**LVL G 240 4s**	**4**	**240**	**1.22**			**G**	
2x2.5m span	2	2.50	LVL G 96 2s	2	96	0.49		G		
2x3.75m span	2	3.75	LVL G 108 3s	3	108	0.55		G		
2x5.0m span	2	5.00	LVL G 108 3s	3	108	0.55		G		
2x6.25m span	2	6.25	**LVL G 120 2s**	**2**	**120**	**0.61**		**G**	**G**	
2x7.5m span	2	7.50	**LVL G 144 3s**	**3**	**144**	**0.73**			**G**	
2x8.75m span	2	8.75	**LVL G 168 4s**	**4**	**168**	**0.86**			**G**	
3x2.5m span	3	2.50	LVL G 96 2s	2	96	0.49		G		
3x3.75m span	3	3.75	LVL G 108 3s	3	108	0.55		G		
3x5.0m span	3	5.00	**LVL G 108 3s**	**3**	**108**	**0.55**		**G**		
3x6.25m span	3	6.25	**LVL G 120 2s**	**2**	**120**	**0.61**		**G**	**G**	
4x2.5m span	4	2.50	LVL G 96 2s	2	96	0.49		G		
4x3.75m span	4	3.75	LVL G 108 3s	3	108	0.55		G		
4x5.0m span	4	5.00	**LVL G 108 3s**	**3**	**108**	**0.55**		**G**		
5x2.5m span	5	2.50	LVL G 96 2s	2	96	0.46		G		
5x3.75m span	5	3.75	LVL G 108 3s	3	108	0.55		G		
6x2.5m span	6	2.50	LVL G 96 2s	2	96	0.49		G		

Table 72. LVL decks with required minimum thickness for various span distances and number of spans with governing requirement (reduced fire-safety and serviceability limit requirements).

			Fire to 30 min., SLS to 1/150 & no vibrations check							
Name	**Nr. sub-spans**	**L**	**$Name_{LVL}$**	**Nr. LVL panels**	**h_{LVL}**	**$g_{0,k}$**	**Normative**			
[-]	**[-]**	**[m]**	**[-]**	**[-]**	**[mm]**	**[kN/m²]**	**ULS**	**ULS Fire**	**SLS**	**Vibrations**
1x2.5m span	1	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
1x5.0m span	1	5.00	LVL G 108 3s	3	108	0.55			G	
1x7.5m span	1	7.50	LVL G 168 4s	4	168	0.86			G	
1x10.0m span	1	10.00	LVL G 240 4s	4	240	1.22			G	
2x2.5m span	2	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
2x3.75m span	2	3.75	**LVL G 72 1s**	**1**	**72**	**0.37**				
2x5.0m span	2	5.00	**LVL G 90 3s**	**3**	**90**	**0.46**			**G**	
2x6.25m span	2	6.25	**LVL G 120 2s**	**2**	**120**	**0.61**			**G**	
2x7.5m span	2	7.50	**LVL G 144 3s**	**3**	**144**	**0.73**			**G**	
2x8.75m span	2	8.75	**LVL G 168 4s**	**4**	**168**	**0.86**			**G**	
3x2.5m span	3	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
3x3.75m span	3	3.75	**LVL G 72 1s**	**1**	**72**	**0.37**				
3x5.0m span	3	5.00	**LVL G 96 2s**	**2**	**96**	**0.49**			**G**	
3x6.25m span	3	6.25	**LVL G 120 2s**	**2**	**120**	**0.61**			**G**	
4x2.5m span	4	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
4x3.75m span	4	3.75	**LVL G 72 1s**	**1**	**72**	**0.37**				
4x5.0m span	4	5.00	**LVL G 96 2s**	**2**	**96**	**0.49**			**G**	
5x2.5m span	5	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				
5x3.75m span	5	3.75	**LVL G 72 1s**	**1**	**72**	**0.37**				
6x2.5m span	6	2.50	**LVL G 72 1s**	**1**	**72**	**0.37**				

Results Comparison Short-span Deck Systems

Table 73. Comparison short-span CLT and LVL deck system; base case and fire to 30 min.

Name	Nr. sub-spans	L	BASE CASE CLT: $Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	Governing: ULS	ULS Fire	SLS	Vibrations	BASE CASE LVL: $Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	Normative: ULS	ULS Fire	SLS	Vibrations	COMPARISON: Height difference		Weight difference	
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m^2]					[-]	[-]	[mm]	[kN/m^2]					[mm]	[%]	[kN/m^2]	[%]
1x2.5m span	1	2.50	CLT 100 L3s	3	100	0.5		G			LVL G 96 2s	2	96	0.49		G			4	4%	0.01	2%
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.7		G	G	G	LVL G 144 3 s	3	144	0.73			G		-4	-3%	-0.03	-4%
1x7.5m span	1	7.50	CLT 220 L7s - 2	7	220	1.1			G	G	LVL G 210 5s	5	210	1.07			G		10	5%	0.03	3%
1x10.0m span	1	10.00	CLT 300 L8s	8	300	1.5			G	G	LVL G 294 7s	7	294	1.50				G	6	2%	0.00	0%
2x2.5m span	2	2.50	CLT 100 L3s	3	100	0.5		G			LVL G 96 2s	2	96	0.49		G			4	4%	0.01	2%
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.6		G		G	LVL G 108 3s	3	108	0.55		G			12	10%	0.05	8%
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.7		G		G	LVL G 108 3s	3	108	0.55		G	G		32	23%	0.15	21%
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.8		G	G	G	LVL G 144 3s	3	144	0.73			G		16	10%	0.07	9%
2x7.5m span	2	7.50	CLT 180 L5s	5	180	0.9			G	G	LVL G 168 4s	4	168	0.86			G		12	7%	0.04	4%
2x8.75m span	2	8.75	CLT 240 L7s - 2	7	240	1.2				G	LVL G 210 5s	5	210	1.07			G		30	13%	0.13	11%

Name	Nr. sub-spans	L	Fire to 30 min. CLT: $Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	Governing: ULS	ULS Fire	SLS	Vibrations	Fire to 30 min. LVL: $Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	Normative: ULS	ULS Fire	SLS	Vibrations	Comparison: Height difference		Weight difference	
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m^2]					[-]	[-]	[mm]	[kN/m^2]					[mm]	[%]	[kN/m^2]	[%]
1x2.5m span	1	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				G	**LVL G 72 1s**	**1**	**72**	**0.37**					18	20%	0.08	18%
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.70			G	G	LVL G 144 3 s	3	144	0.73			G		-4	-3%	-0.03	-4%
1x7.5m span	1	7.50	CLT 220 L7s - 2	7	220	1.10			G		LVL G 210 5s	5	210	1.07			G		10	5%	0.03	3%
1x10.0m span	1	10.00	CLT 300 L8s	8	300	1.50			G	G	LVL G 294 7s	7	294	1.50				G	6	2%	0.00	0%
2x2.5m span	2	2.50	**CLT 90 L3s**	**3**	**90**	**0.45**				G	**LVL G 72 1s**	**1**	**72**	**0.37**					18	20%	0.08	18%
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.60				G	**LVL G 84 2s**	**2**	**84**	**0.43**			**G**		36	30%	0.17	28%
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.70				G	LVL G 108 3s	3	108	0.55		G	G		32	23%	0.15	21%
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.80			G	G	LVL G 144 3s	3	144	0.73			G		16	10%	0.07	9%
2x7.5m span	2	7.50	CLT 180 L5s	5	180	0.90			G	G	LVL G 168 4s	4	168	0.86			G		12	7%	0.04	4%
2x8.75m span	2	8.75	CLT 240 L7s - 2	7	240	1.20				G	LVL G 210 5s	5	210	1.07			G		30	13%	0.13	11%

Table 74. Comparison short-span CLT and LVL deck system; no vibrations and SLS to 1/150 & no vibrations

			No vibrations check CLT								No vibrations check LVL								Comparison			
Name	Nr. sub-spans	L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	Governing				$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	Normative				Height difference		Weight difference	
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations	[mm]	[%]	[kN/m^2]	[%]
1x2.5m span	1	2.50	CLT 100 L3s	3	100	0.50		G			LVL G 96 2s	2	96	0.49		G			4	4%	0.01	2%
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.70		G	G		LVL G 144 3 s	3	144	0.73			G		-4	-3%	-0.03	-4%
1x7.5m span	1	7.50	CLT 220 L7s - 2	7	220	1.10			G		LVL G 210 5s	5	210	1.07			G		10	5%	0.03	3%
1x10.0m span	1	10.00	CLT 300 L8s	8	300	1.50			G		**LVL G 288 6s**	**6**	**288**	**1.47**			**G**		12	4%	0.03	2%
2x2.5m span	2	2.50	CLT 100 L3s	3	100	0.50		G			LVL G 96 2s	2	96	0.49		G			4	4%	0.01	2%
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.60		G			LVL G 108 3s	3	108	0.55		G			12	10%	0.05	8%
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.70		G			LVL G 108 3s	3	108	0.55		G	G		32	23%	0.15	21%
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.80		G	G		LVL G 144 3s	3	144	0.73			G		16	10%	0.07	9%
2x7.5m span	2	7.50	CLT 180 L5s	5	180	0.90			G		LVL G 168 4s	4	168	0.86			G		12	7%	0.04	4%
2x8.75m span	2	8.75	**CLT 220 L7s - 2**	**7**	**220**	**1.10**			**G**		LVL G 210 5s	5	210	1.07			G		10	5%	0.03	3%

			SLS to 1/150 & no vibrations check CLT								SLS to 1/150 & no vibrations check LVL								Comparison			
Name	Nr. sub-spans	L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	Governing				$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	Normative				Height difference		Weight difference	
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations	[mm]	[%]	[kN/m^2]	[%]
1x2.5m span	1	2.50	CLT 100 L3s	3	100	0.50		G			**LVL G 96 2s**	**2**	**96**	**0.49**		**G**			4	4%	0.01	2%
1x5.0m span	1	5.00	CLT 140 L5s	5	140	0.70		G	G		**LVL G 108 3s**	**3**	**108**	**0.55**		**G**	**G**		32	23%	0.15	21%
1x7.5m span	1	7.50	**CLT 180 L7s**	**7**	**180**	**0.90**			**G**		**LVL G 168 4s**	**4**	**168**	**0.86**			**G**		12	7%	0.04	4%
1x10.0m span	1	10.00	**CLT 240 L7s - 2**	**7**	**240**	**1.20**			**G**		**LVL G 240 4s**	**4**	**240**	**1.22**			**G**		0	0%	-0.02	-2%
2x2.5m span	2	2.50	CLT 100 L3s	3	100	0.50		G			LVL G 96 2s	2	96	0.49		G			4	4%	0.01	2%
2x3.75m span	2	3.75	CLT 120 L5s	5	120	0.60		G			LVL G 108 3s	3	108	0.55		G			12	10%	0.05	8%
2x5.0m span	2	5.00	CLT 140 L5s	5	140	0.70		G			LVL G 108 3s	3	108	0.55		G			32	23%	0.15	21%
2x6.25m span	2	6.25	CLT 160 L5s	5	160	0.80		G	G		**LVL G 120 2s**	**2**	**120**	**0.61**		**G**	**G**		40	25%	0.19	24%
2x7.5m span	2	7.50	**CLT 160 L5s - 2**	**5**	**160**	**0.80**		**G**	**G**		**LVL G 144 3s**	**3**	**144**	**0.73**			**G**		16	10%	0.07	9%
2x8.75m span	2	8.75	**CLT 180 L7s**	**7**	**180**	**0.90**		**G**	**G**		**LVL G 168 4s**	**4**	**168**	**0.86**			**G**		12	7%	0.04	4%

Table 75. Comparison short-span CLT and LVL deck system; fire to 30 min., no vibrations and SLS to 1/150 & no vibrations

			Fire to 30 min., SLS to 1/150 & no vibrations check CLT								Fire to 30 min., SLS to 1/150 & no vibrations check LVL								Comparison			
Name	Nr. sub-spans	L	$Name_{CLT}$	CLT layers	h_{CLT}	$g_{0,k}$	Governing				$Name_{LVL}$	Nr. LVL panels	h_{LVL}	$g_{0,k}$	Normative				Height difference		Weight difference	
[-]	[-]	[m]	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations	[-]	[-]	[mm]	[kN/m^2]	ULS	ULS Fire	SLS	Vibrations	[mm]	[%]	[kN/m^2]	[%]
1x2.5m span	1	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					**LVL G 72 1s**	**1**	**72**	**0.37**					-12	-20%	-0.07	-23%
1x5.0m span	1	5.00	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**	**G**		LVL G 108 3s	3	108	0.55			G		2	2%	0.00	0%
1x7.5m span	1	7.50	CLT 180 L7s	7	180	0.90			G		LVL G 168 4s	4	168	0.86			G		12	7%	0.04	4%
1x10.0m span	1	10.00	CLT 240 L7s - 2	7	240	1.20			G		LVL G 240 4s	4	240	1.22			G		0	0%	-0.02	-2%
2x2.5m span	2	2.50	**CLT 60 L3s**	**3**	**60**	**0.30**					**LVL G 72 1s**	**1**	**72**	**0.37**					-12	-20%	-0.07	-23%
2x3.75m span	2	3.75	**CLT 90 L3s**	**3**	**90**	**0.45**		**G**			**LVL G 72 1s**	**1**	**72**	**0.37**					18	20%	0.08	18%
2x5.0m span	2	5.00	**CLT 110 L3s**	**3**	**110**	**0.55**		**G**			**LVL G 90 3s**	**3**	**90**	**0.46**			**G**		20	18%	0.09	16%
2x6.25m span	2	6.25	**CLT 120 L5s**	**5**	**120**	**0.60**			**G**		**LVL G 120 2s**	**2**	**120**	**0.61**			**G**		0	0%	-0.01	-2%
2x7.5m span	2	7.50	CLT 160 L5s - 2	5	160	0.80		G	G		**LVL G 144 3s**	**3**	**144**	**0.73**			**G**		16	10%	0.07	9%
2x8.75m span	2	8.75	CLT 180 L7s	7	180	0.90		G	G		**LVL G 168 4s**	**4**	**168**	**0.86**			**G**		12	7%	0.04	4%

F.3 Choice of Deck System

Table 76. Choice of long-span deck system for various scenario's.

	BASE CASE				Fire to 30 min.				No vibrations check			
Span [m]	Deck type [-]	h_{tot} [mm]	$g_{0,k}$ [kN/m]	$g_{0,k,tot(2500mm)}$ [kN/m]	Deck type [-]	h_{tot} [mm]	$g_{0,k}$ [kN/m]	$g_{0,k,tot(2500mm)}$ [kN/m]	Deck type [-]	h_{tot} [mm]	$g_{0,k}$ [kN/m]	$g_{0,k,tot(2500mm)}$ [kN/m]
12.18	Open CLT deck: 120mm CLT - 4 ribs 360x200mm	480	1.18	2.95	Open CLT deck: 120mm CLT - 4 ribs 360x200mm	480	1.18	2.95	LVL semi-open rib panel: h_{rib} = 360mm	497	0.82	2.06
13.70	Open CLT deck: 120mm CLT - 4 ribs 450x200mm	570	1.32	3.30	Open CLT deck: 120mm CLT - 4 ribs 450x200mm	570	1.32	3.30	LVL closed rib panel: h_{rib} = 400mm	537	0.93	2.34
14.56	Open CLT deck: 120mm CLT - 4 ribs 500x200mm	620	1.40	3.50	Open CLT deck: 120mm CLT - 4 ribs 500x200mm	620	1.40	3.50	LVL semi-open rib panel: h_{rib} = 450mm	587	0.87	2.18
15.61	Open CLT deck: 120mm CLT - 4 ribs 600x200mm	720	1.56	3.90	Open CLT deck: 120mm CLT - 4 ribs 600x200mm	720	1.56	3.90	LVL closed rib panel: h_{rib} = 500mm	637	0.99	2.47
16.26	Closed CLT deck: 120mm CLT - 4 ribs 500x140mm - 120mm CLT	740	1.76	4.40	Closed CLT deck: 120mm CLT - 4 ribs 500x140mm - 120mm CLT	740	1.76	4.40	LVL closed rib panel: h_{rib} = 500mm	637	0.99	2.47
17.16	Open CLT deck: 120mm CLT - 4 ribs 700x200mm	820	1.72	4.30	Open CLT deck: 120mm CLT - 4 ribs 700x200mm	820	1.72	4.30	LVL semi-open rib panel: h_{rib} = 600mm	737	0.95	2.38
17.66	Closed CLT deck: 120mm CLT - 4 ribs 600x140mm - 120mm CLT	840	1.87	4.68	Closed CLT deck: 120mm CLT - 4 ribs 600x140mm - 120mm CLT	840	1.87	4.68	LVL semi-open rib panel: h_{rib} = 600mm	737	0.95	2.38

	SLS to 1/150 & no vibrations check				Fire to 30 min., SLS to 1/150 & no vibrations check			
Span [m]	Deck type [-]	h_{tot} [mm]	$g_{0,k}$ [kN/m]	$g_{0,k,tot(2500mm)}$ [kN/m]	Deck type [-]	h_{tot} [mm]	$g_{0,k}$ [kN/m]	$g_{0,k,tot(2500mm)}$ [kN/m]
12.18	LVL closed rib panel: h_{rib} = 200mm	337	0.83	2.07	LVL closed rib panel: h_{rib} = 200mm	337	0.83	2.07
13.70	LVL closed rib panel: h_{rib} = 300mm	437	0.88	2.20	LVL closed rib panel: h_{rib} = 300mm	437	0.88	2.20
14.56	LVL semi-open rib panel: h_{rib} = 360mm	497	0.82	2.06	LVL semi-open rib panel: h_{rib} = 360mm	497	0.82	2.06
15.61	LVL closed rib panel: h_{rib} = 360mm	497	0.91	2.28	LVL closed rib panel: h_{rib} = 360mm	497	0.91	2.28
16.26	LVL closed rib panel: h_{rib} = 400mm	537	0.93	2.34	LVL closed rib panel: h_{rib} = 400mm	537	0.93	2.34
17.16	LVL semi-open rib panel: h_{rib} = 450mm	587	0.87	2.18	LVL semi-open rib panel: h_{rib} = 450mm	587	0.87	2.18
17.66	LVL closed rib panel: h_{rib} = 450mm	587	0.96	2.40	LVL closed rib panel: h_{rib} = 450mm	587	0.96	2.40

G.

Additional Calculations & Results Framing System

All Subvariants Total Deck Height, Weight and Column Load

Table 77. Total deck height, weight per level and loads on foundation for all subvariants.

					Load standard column on foundation [kN]				
ID	**Car park (sub)variant**	**Span [m]**	**Floor depth [mm]**	**Weight /level [kg]**	**1 level**	**2 levels**	**3 levels**	**4 levels**	**5 levels**
1	Variant 1, 12.18m, column every 1 parking bay(s)	12.18	480	16,098	140.8	281.6	422.3	563.1	703.9
2	Variant 1, 13.7m, column every 1 parking bay(s)	13.70	570	20,267	164.4	328.8	493.2	657.5	821.9
3	Variant 1, 14.56m, column every 1 parking bay(s)	14.56	620	22,734	178.2	356.5	534.7	713.0	891.2
4	Variant 1, 15.61m, column every 1 parking bay(s)	15.61	720	26,979	198.8	397.5	596.3	795.0	993.8
5	Variant 1, 16.26m, column every 1 parking bay(s)	16.26	740	31,507	217.0	434.0	651.0	868.1	1,085.1
6	Variant 1, 17.16m, column every 1 parking bay(s)	17.16	820	32,595	227.0	454.0	681.0	908.1	1,135.1
7	Variant 1, 17.66m, column every 1 parking bay(s)	17.66	840	36,213	241.6	483.1	724.7	966.2	1,207.8
8	Variant 1, 12.18m, column every 2 parking bay(s)	12.18	480	16,238	282.0	564.0	846.0	1,127.9	1,409.9
9	Variant 1, 13.7m, column every 2 parking bay(s)	13.70	570	20,226	328.6	657.3	985.9	1,314.6	1,643.2
10	Variant 1, 14.56m, column every 2 parking bay(s)	14.56	620	22,747	356.5	713.1	1,069.6	1,426.1	1,782.7
11	Variant 1, 15.61m, column every 2 parking bay(s)	15.61	720	26,880	397.2	794.4	1,191.6	1,588.8	1,986.1
12	Variant 1, 16.26m, column every 2 parking bay(s)	16.26	740	31,439	433.8	867.7	1,301.5	1,735.3	2,169.1
13	Variant 1, 17.16m, column every 2 parking bay(s)	17.16	820	32,577	454.0	907.9	1,361.9	1,815.9	2,269.9
14	Variant 1, 17.66m, column every 2 parking bay(s)	17.66	840	36,211	483.1	966.2	1,449.3	1,932.4	2,415.5
15	Variant 2a, 12.18m, struts On, column every 1 parking bay(s)	12.18	516	9,529	122.1	244.2	366.4	488.5	610.6
16	Variant 2a, 13.7m, struts On, column every 1 parking bay(s)	13.70	586	11,466	139.1	278.3	417.4	556.5	695.7
17	Variant 2a, 14.56m, struts On, column every 1 parking bay(s)	14.56	636	12,679	149.2	298.3	447.5	596.7	745.8
18	Variant 2a, 15.61m, struts On, column every 1 parking bay(s)	15.61	676	13,890	160.8	321.5	482.3	643.0	803.8
19	Variant 2a, 16.26m, struts On, column every 1 parking bay(s)	16.26	666	15,242	169.6	339.2	508.8	678.4	848.0
20	Variant 2a, 17.16m, struts On, column every 1 parking bay(s)	17.16	716	16,479	180.2	360.4	540.6	720.8	901.0

21	Variant 2a, 17.66m, struts On, column every 1 parking bay(s)	17.66	746	17,305	186.4	372.7	559.1	745.4	931.8
22	Variant 2a, 12.18m, struts Off, column every 1 parking bay(s)	12.18	726	11,352	127.1	254.2	381.3	508.4	635.5
23	Variant 2a, 13.7m, struts Off, column every 1 parking bay(s)	13.70	806	13,368	144.7	289.4	434.1	578.8	723.5
24	Variant 2a, 14.56m, struts Off, column every 1 parking bay(s)	14.56	846	14,630	154.9	309.7	464.6	619.5	774.4
25	Variant 2a, 15.61m, struts Off, column every 1 parking bay(s)	15.61	906	16,221	167.6	335.1	502.7	670.3	837.8
26	Variant 2a, 16.26m, struts Off, column every 1 parking bay(s)	16.26	946	17,212	175.5	351.0	526.6	702.1	877.6
27	Variant 2a, 17.16m, struts Off, column every 1 parking bay(s)	17.16	996	18,733	186.8	373.5	560.3	747.0	933.8
28	Variant 2a, 17.66m, struts Off, column every 1 parking bay(s)	17.66	1016	19,626	193.0	386.0	579.0	771.9	964.9
29	Variant 2a, 12.18m, struts On, column every 2 parking bay(s)	12.18	668	9,769	245.5	490.9	736.4	981.9	1,227.3
30	Variant 2a, 13.7m, struts On, column every 2 parking bay(s)	13.70	748	11,292	277.9	555.9	833.8	1,111.7	1,389.6
31	Variant 2a, 14.56m, struts On, column every 2 parking bay(s)	14.56	798	12,291	296.9	593.8	890.7	1,187.6	1,484.5
32	Variant 2a, 15.61m, struts On, column every 2 parking bay(s)	15.61	838	13,739	321.8	643.5	965.3	1,287.1	1,608.8
33	Variant 2a, 16.26m, struts On, column every 2 parking bay(s)	16.26	868	14,456	336.0	672.0	1,008.0	1,344.0	1,680.1
34	Variant 2a, 17.16m, struts On, column every 2 parking bay(s)	17.16	928	15,539	356.3	712.7	1,069.0	1,425.4	1,781.7
35	Variant 2a, 17.66m, struts On, column every 2 parking bay(s)	17.66	968	16,180	368.0	736.0	1,103.9	1,471.9	1,839.9
36	Variant 2a, 12.18m, struts Off, column every 2 parking bay(s)	12.18	858	10,671	250.6	501.1	751.7	1,002.2	1,252.8
37	Variant 2a, 13.7m, struts Off, column every 2 parking bay(s)	13.70	958	12,250	283.8	567.5	851.3	1,135.1	1,418.9
38	Variant 2a, 14.56m, struts Off, column every 2 parking bay(s)	14.56	1008	13,312	303.1	606.2	909.3	1,212.4	1,515.5
39	Variant 2a, 15.61m, struts Off, column every 2 parking bay(s)	15.61	1078	14,494	326.6	653.2	979.8	1,306.4	1,633.0
40	Variant 2a, 16.26m, struts Off, column every 2 parking bay(s)	16.26	1128	15,280	341.5	682.9	1,024.4	1,365.8	1,707.3
41	Variant 2a, 17.16m, struts Off, column every 2 parking bay(s)	17.16	1178	16,392	361.9	723.8	1,085.8	1,447.7	1,809.6
42	Variant 2a, 17.66m, struts Off, column every 2 parking bay(s)	17.66	1218	17,118	373.8	747.7	1,121.5	1,495.3	1,869.2
43	Variant 2b, 12.18m, column every 2 parking bay(s)	12.18	756	11,001	251.9	503.9	755.8	1,007.7	1,259.7

44	Variant 2b, 13.7m, column every 2 parking bay(s)	13.70	836	12,831	286.2	572.4	858.6	1,144.8	1,431.0
45	Variant 2b, 14.56m, column every 2 parking bay(s)	14.56	886	14,016	306.3	612.5	918.8	1,225.1	1,531.3
46	Variant 2b, 15.61m, column every 2 parking bay(s)	15.61	946	15,476	330.9	661.9	992.8	1,323.7	1,654.7
47	Variant 2b, 16.26m, column every 2 parking bay(s)	16.26	986	16,404	346.4	692.9	1,039.3	1,385.7	1,732.2
48	Variant 2b, 17.16m, column every 2 parking bay(s)	17.16	1036	17,745	368.0	736.1	1,104.1	1,472.2	1,840.2
49	Variant 2b, 17.66m, column every 2 parking bay(s)	17.66	1066	18,437	380.0	760.0	1,140.0	1,520.0	1,899.9
50	Variant 3, 12.18m, struts On, column every 2 parking bay(s)	12.18	656	9,319	245.1	490.2	735.3	980.4	1,225.5
51	Variant 3, 13.7m, struts On, column every 2 parking bay(s)	13.70	756	10,986	279.4	558.8	838.2	1,117.6	1,397.0
52	Variant 3, 14.56m, struts On, column every 2 parking bay(s)	14.56	816	11,881	298.0	596.0	893.9	1,191.9	1,489.9
53	Variant 3, 15.61m, struts On, column every 2 parking bay(s)	15.61	886	13,043	321.0	642.0	963.0	1,284.0	1,605.0
54	Variant 3, 16.26m, struts On, column every 2 parking bay(s)	16.26	936	13,808	335.5	671.0	1,006.5	1,342.0	1,677.5
55	Variant 3, 17.16m, struts On, column every 2 parking bay(s)	17.16	996	14,879	356.3	712.6	1,068.9	1,425.2	1,781.5
56	Variant 3, 17.66m, struts On, column every 2 parking bay(s)	17.66	1026	15,380	367.1	734.2	1,101.3	1,468.5	1,835.6
57	Variant 3, 12.18m, struts On, column every 2 parking bay(s)	12.18	678	10,031	249.0	498.1	747.1	996.1	1,245.2
58	Variant 3, 13.7m, struts On, column every 2 parking bay(s)	13.70	768	11,643	282.3	564.7	847.0	1,129.3	1,411.6
59	Variant 3, 14.56m, struts On, column every 2 parking bay(s)	14.56	828	12,590	301.2	602.4	903.7	1,204.9	1,506.1
60	Variant 3, 15.61m, struts On, column every 2 parking bay(s)	15.61	898	13,916	325.8	651.7	977.5	1,303.3	1,629.1
61	Variant 3, 16.26m, struts On, column every 2 parking bay(s)	16.26	948	14,630	339.6	679.3	1,018.9	1,358.6	1,698.2
62	Variant 3, 17.16m, struts On, column every 2 parking bay(s)	17.16	1008	15,757	360.6	721.3	1,081.9	1,442.5	1,803.2
63	Variant 3, 17.66m, struts On, column every 2 parking bay(s)	17.66	1038	16,288	371.6	743.3	1,114.9	1,486.5	1,858.2
64	Variant 3, 12.18m, struts Off, column every 2 parking bay(s)	12.18	856	10,181	250.2	500.4	750.5	1,000.7	1,250.9
65	Variant 3, 13.7m, struts Off, column every 2 parking bay(s)	13.70	956	11,983	285.3	570.5	855.8	1,141.0	1,426.3
66	Variant 3, 14.56m, struts Off, column every 2 parking bay(s)	14.56	1006	12,911	304.0	608.1	912.1	1,216.2	1,520.2
67	Variant 3, 15.61m, struts Off, column every 2 parking bay(s)	15.61	1076	14,199	327.8	655.6	983.4	1,311.2	1,639.0

68	Variant 3, 16.26m, struts Off, column every 2 parking bay(s)	16.26	1126	15,022	342.7	685.3	1,028.0	1,370.7	1,713.4
69	Variant 3, 17.16m, struts Off, column every 2 parking bay(s)	17.16	1176	16,152	363.8	727.6	1,091.5	1,455.3	1,819.1
70	Variant 3, 17.66m, struts Off, column every 2 parking bay(s)	17.66	1216	16,767	375.3	750.6	1,126.0	1,501.3	1,876.6
71	Variant 3, 12.18m, struts Off, column every 2 parking bay(s)	12.18	868	10,859	253.9	507.8	761.7	1,015.6	1,269.5
72	Variant 3, 13.7m, struts Off, column every 2 parking bay(s)	13.70	958	12,598	287.9	575.9	863.8	1,151.8	1,439.7
73	Variant 3, 14.56m, struts Off, column every 2 parking bay(s)	14.56	1018	13,619	307.3	614.6	921.9	1,229.1	1,536.4
74	Variant 3, 15.61m, struts Off, column every 2 parking bay(s)	15.61	1088	15,072	332.6	665.3	997.9	1,330.6	1,663.2
75	Variant 3, 16.26m, struts Off, column every 2 parking bay(s)	16.26	1128	15,818	346.7	693.3	1,040.0	1,386.6	1,733.3
76	Variant 3, 17.16m, struts Off, column every 2 parking bay(s)	17.16	1188	17,029	368.2	736.3	1,104.5	1,472.6	1,840.8
77	Variant 3, 17.66m, struts Off, column every 2 parking bay(s)	17.66	1218	17,621	379.5	759.0	1,138.5	1,518.1	1,897.6
xx	Modupark, 16.50m (16.26m), column every 2 parking bays (corrected for comparable dimensions)	16.26	550	61,261	618.1	1,236.1	1,854.2	2,472.3	3,090.3
xx	Koopman int. (steel + fibreglass), 15.65m (16.26m), column every 2 parking bays (corrected for comparable dimensions)	16.26	640	10,477	314.6	629.2	943.7	1,258.3	1,572.9
xx	Morspoort (steel + composite), 14.50 (16.26m), column every 2 parking bays (corrected for comparable dimensions)	16.26	651	35,110	461.1	922.2	1,383.4	1,844.5	2,305.6
xx	BauBuche concept (timber + concrete), 16.50m (16.26m), column every 1 parking bay (corrected for comparable dimensions)	16.26	730	59,341	298.3	596.6	894.9	1,193.2	1,491.5
xx	B&O-Holzparkhaus (timber), 16.24m (16.26m), column every 1 parking bay (corrected for comparable dimensions)	16.26	935	17,547	179.2	358.3	537.5	716.7	895.8

H.

Parametric Model

This appendix describes the script of the parametric model using Rhinoceros and Grasshopper to create a .xml file for structural analysis in SCIA using the plugin Koala. First, a general overview of the parametric script is given after which the various components are discussed.

H.1 General Overview

An overview of the workflow within the script can be found in Figure 165 and the total script is presented in Figure 166. The script can be separated into two parts. The first part is concerned with creating the geometry of the structural elements to later be used for the second part, preparing a structural analysis model of the subvariant for SCIA, using the plugin Koala.

The script starts with the inputs. These are either parameterised inputs, like the parking deck span and column distance, but also include the specific parking deck system chosen in chapter 9. A second set of inputs are the so-called standardised inputs, which include car park characteristics like the parking bay width and clearance height, inputs for the wind load calculation and general load input values. These input values are further processed to determine often used input settings for the modelling of the geometry like the total story height.

Next, the geometry for the 2D structural model is created. These lines and nodes are later used to create the model for the structural analysis. For each main design variant, the geometry is created by a separate set of objects.

The preparation for the structural analysis model for SCIA starts with setting up the general settings of the analysis. Next, the different items required for the analysis model are created like the layers for the structural elements, load cases & groups and load combinations. For the cross-sections, self-made libraries for the standard available cross-section dimensions are inserted as well as material types and classes for later selection. The created lines and nodes from the geometry section are then used to create beam elements and support points. Cross-sections are linked to the elements and loads are applied. Finally, hinges are added where needed.

With all the inputs for the generation of the xml file by Koala ready, they are inserted into the xml generation component, which combines all elements into an xml file, which can be opened and analysed in SCIA. However, some settings required for the structural analysis of especially timber structures are not yet available in Koala components. Therefore, the created xml file is further processed by text editing in the Grasshopper model, to add certain settings to the file. This finally results in the xml file, which is used for structural analysis in SCIA.

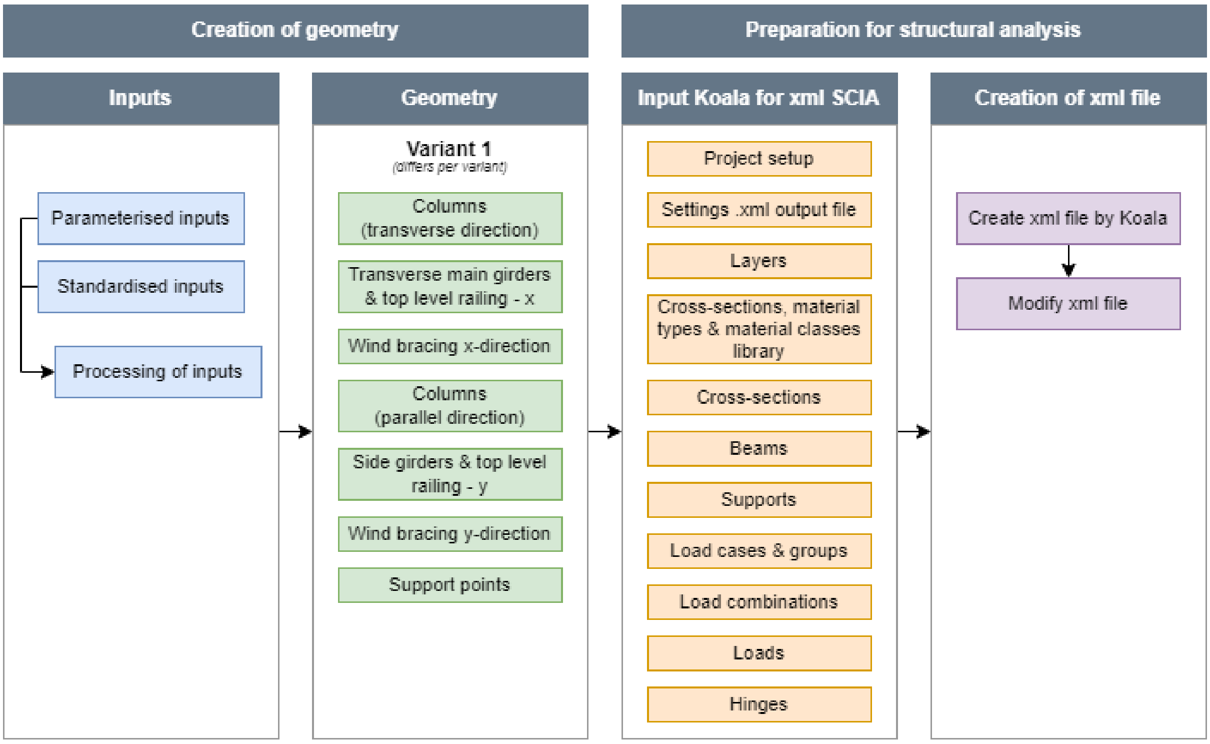

Figure 165. Overview of workflow in parametric script.

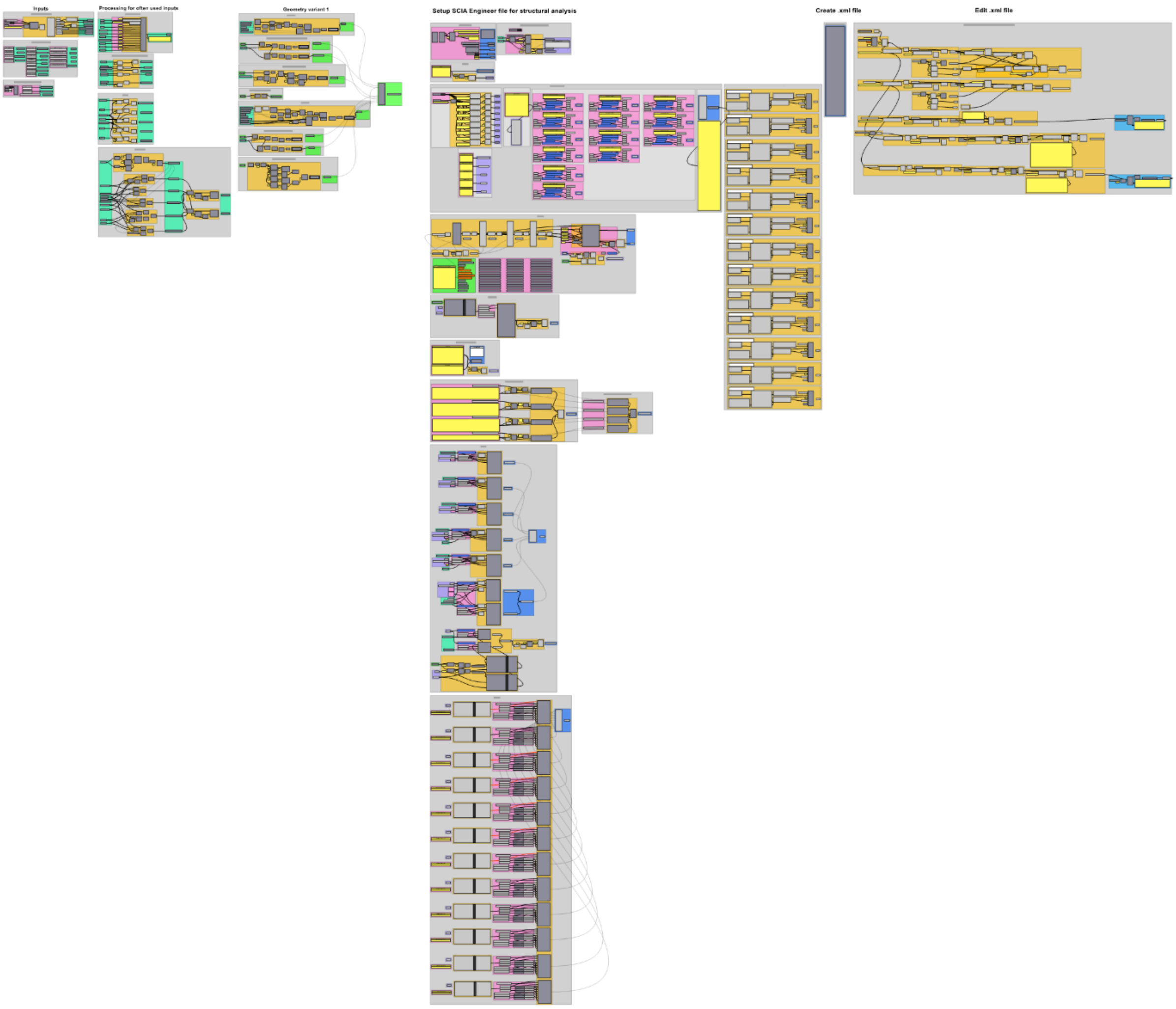

Figure 166. Grasshopper script parametric model.

To further structure the Grasshopper script, various colours are used to indicate the type of logic.

	Input by user		Input for Koala plugin
	General (processed) input parameter		Output .xml file for SCIA
	Geometry output		Operation on logic

H.2 Inputs

User Inputs

Parameterized Inputs

The parameterized input section reads various input parameters from an Excel file. These include for example the parking deck span and weight of the deck. These are then extracted and stored in separate containers for further use, see Figure 167

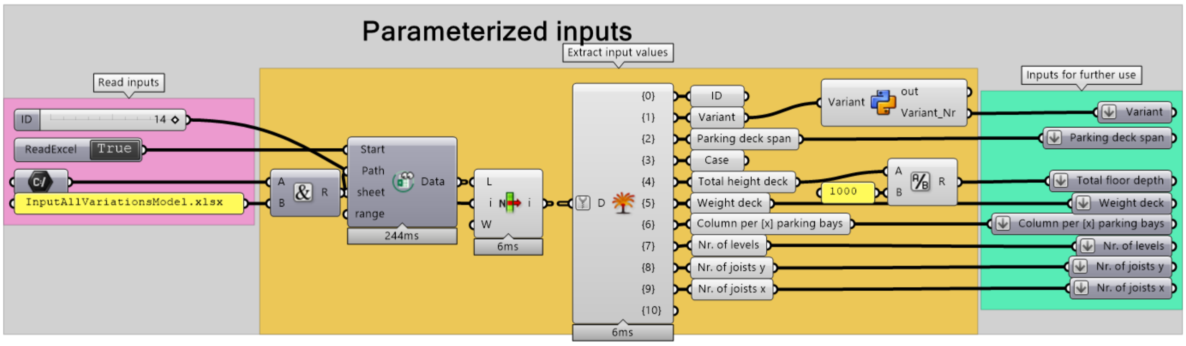

Figure 167. Parameterized inputs.

Standardized inputs

The standardized input section includes three types of inputs, which do not change for the various considered design (sub)variants, see Figure 168. The first type are general settings for the geometry of the structure: parking bay width, clearance height, height railing top deck and inward distance struts. The second type of inputs are related to the inputs required for the wind calculations. These include for example the wind peak force to be considered and wind zone coefficients. The third set of inputs is related to the various loads considered in the structural validation of the structure: dead load, imposed load, snow load, wind load and self-weight of the façade.

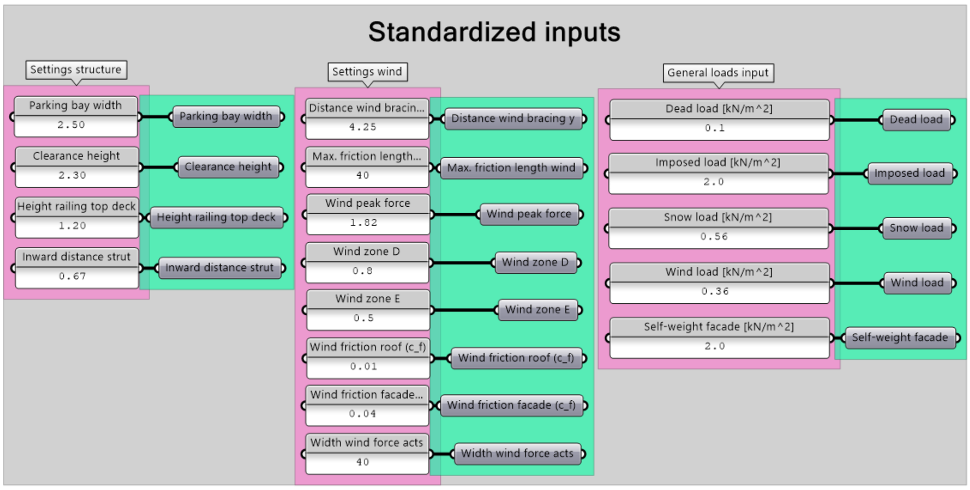

Figure 168. Standardized inputs.

Settings Grasshopper/Rhino

The last section under inputs include various general settings for Grasshopper and Rhinoceros, like the colour of the grasshopper scripts and drawing distance between different sections of the structure, see Figure 169.

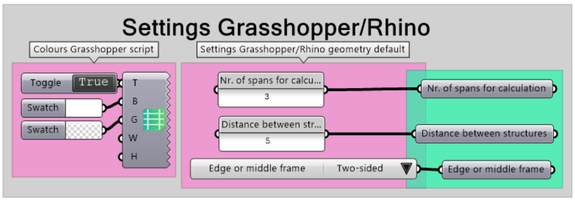

Figure 169. Settings Grasshopper/Rhino.

Processing for Often Used Inputs

Create Filename

Using the various inputs by the user, a filename is created, by combining various inputs and a unique ID into the filename, see Figure 170.

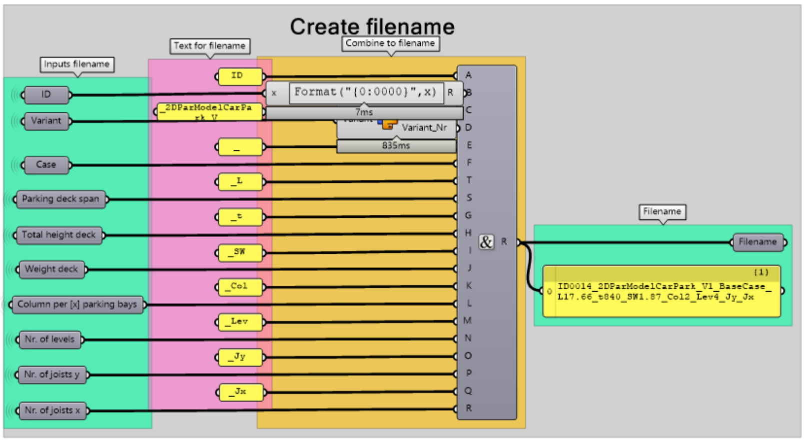

Figure 170. Create filename.

Specification of Dimensions Car Park

For the creation of the geometry of the car park structure, various parameters are required, which depend on multiple input parameters. Some of these parameters, which are often used in the script are therefore specified at this point, see Figure 171.

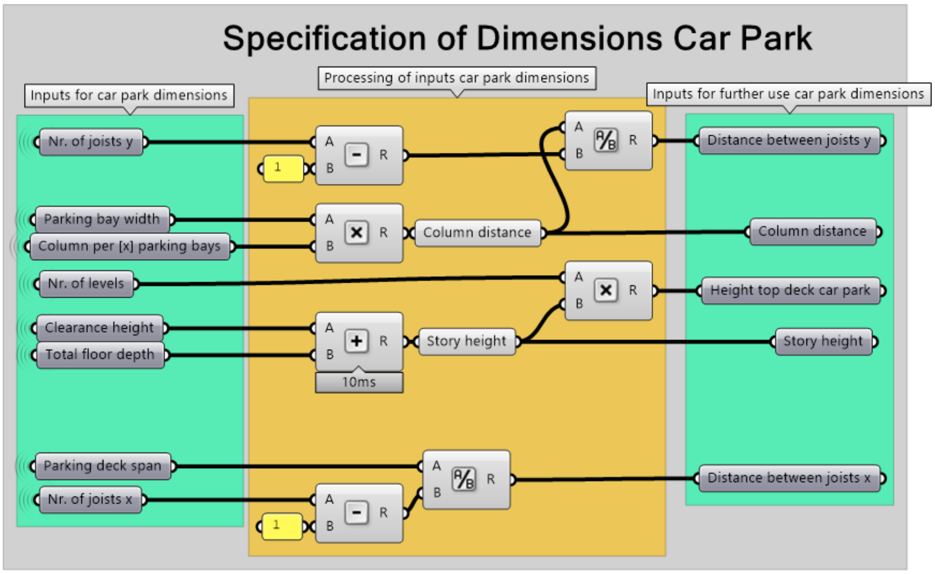

Figure 171. Specification of dimensions car park.

Loads

In the standardized input section, the various loads were already inserted in kN/m². These are now converted to kN/m, see Figure 172.

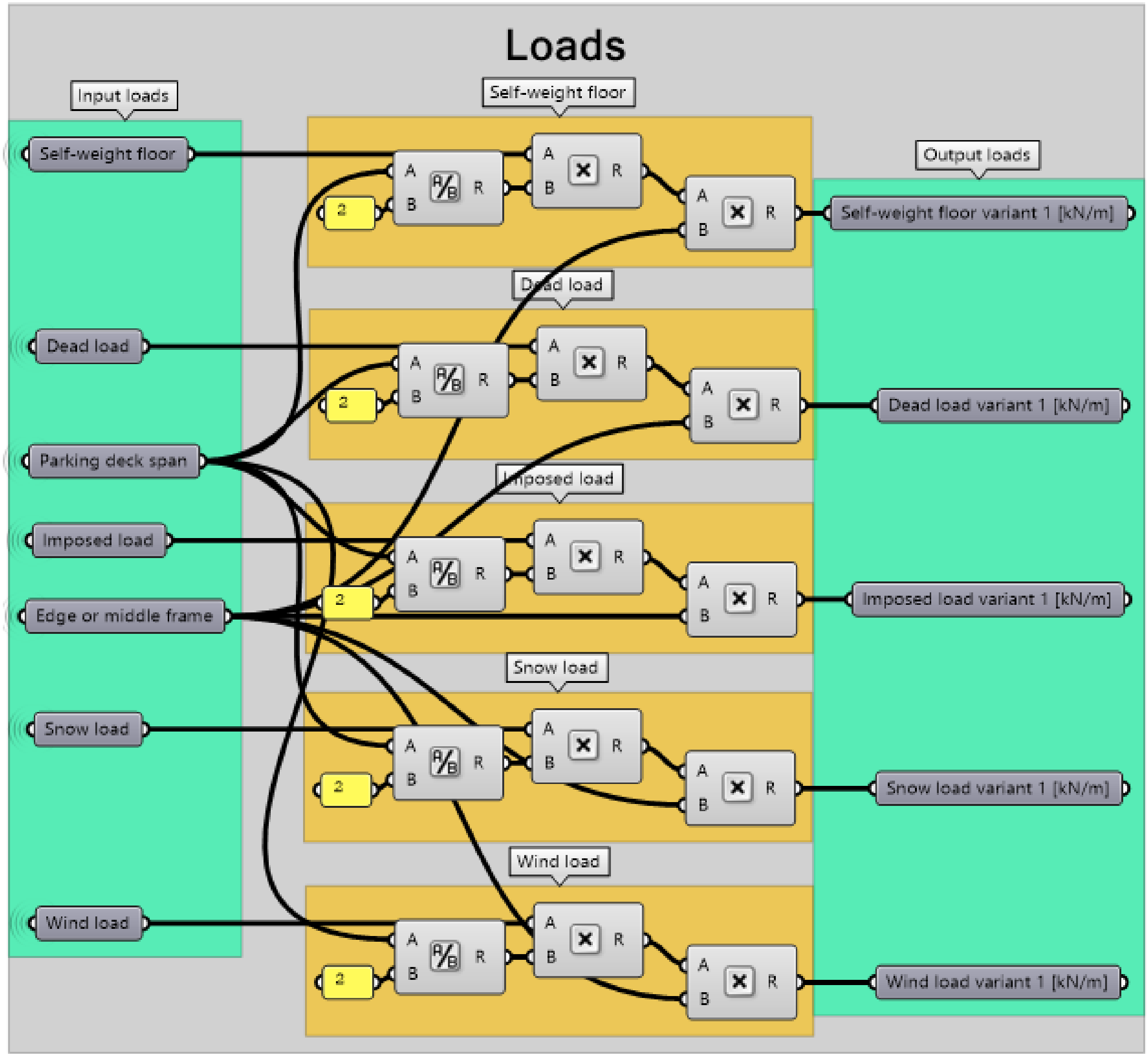

Figure 172. Loads.

Wind Loads

In this section, the various point loads on the structure as a result of wind loading are calculated, see Figure 173. The calculation consist out of various steps. First, the façades are distributed into various sections, which transfer their wind loads to a supporting elements. The heights of these different sections are calculated. Next, the various types of wind loads acting on the structure are calculated. These are the friction load on the roof in x and y direction and the pressure and suction loads on the façades in x and y direction. All of these loads act on a certain point on the structure, where they are supported. Finally, all the wind point loads acting on these support points are added up, resulting in an array of point loads for the wind loads acting on the structure.

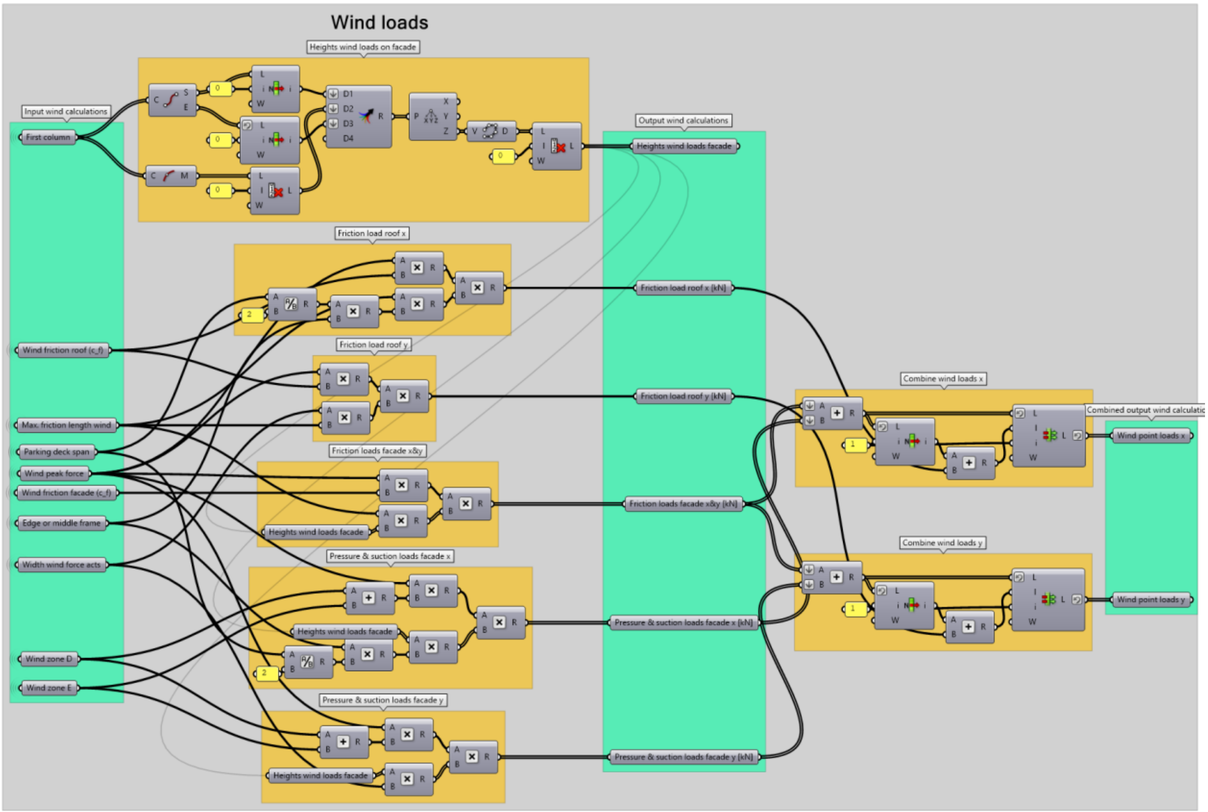

Figure 173. Wind loads.

H.3 Geometry

The second part of the script consists out of the creation of the geometry of the structural system of the design variant. In this paragraph, the creation of the geometry of design variant 1 is described. This results in a set of lines and points as presented in Figure 174.

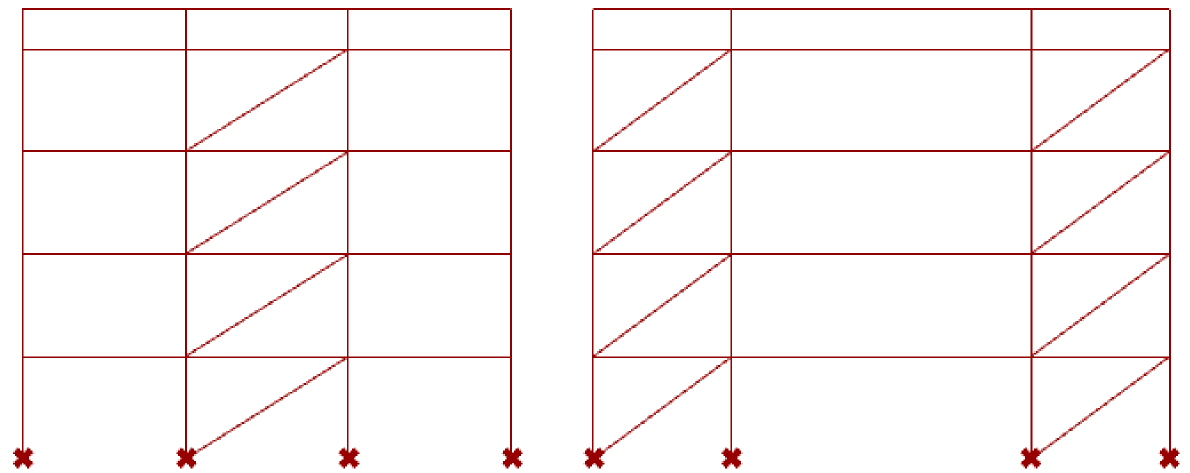

Figure 174. Geometry of design variant 1.

Columns

The main columns following the repetitive pattern in the transverse direction are created in this section, see Figure 175 (left vertical elements in Figure 174).

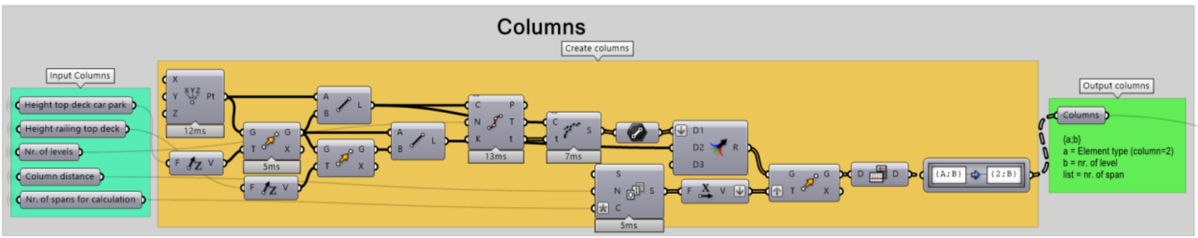

Figure 175. Columns.

Transverse Main Girders & Top Level Railing

In this section, the transverse main girders and top level railing are created, see Figure 176. These are the horizontal elements in Figure 174. The transverse main girders can also be the floor panels in the final design, which can later be selected when choosing the cross-section applied to this element.

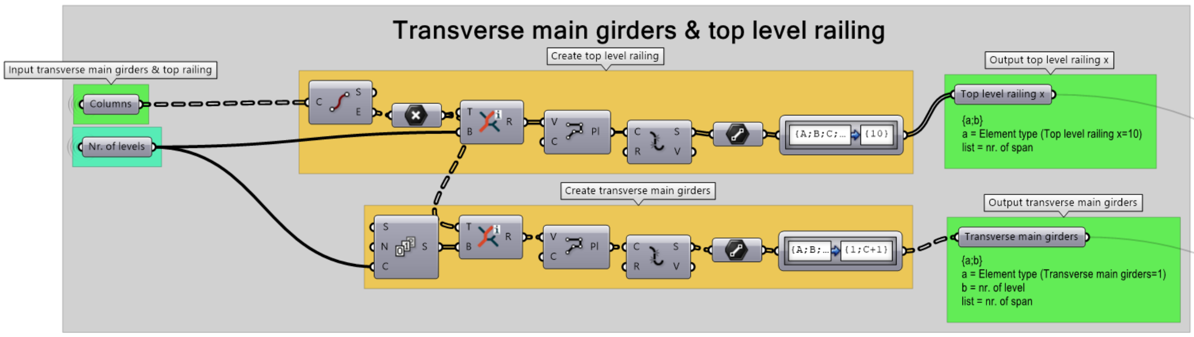

Figure 176. Transverse main girders & top level railing.

Wind Bracing X-direction

For the wind bracings, only slanted elements in one direction are created, see Figure 177. These elements function as tension only elements and in the model, only horizontal loads coming from the left side are considered. In reality, cross bracings will be applied, but for modelling purposes, elements in only one direction are sufficient.

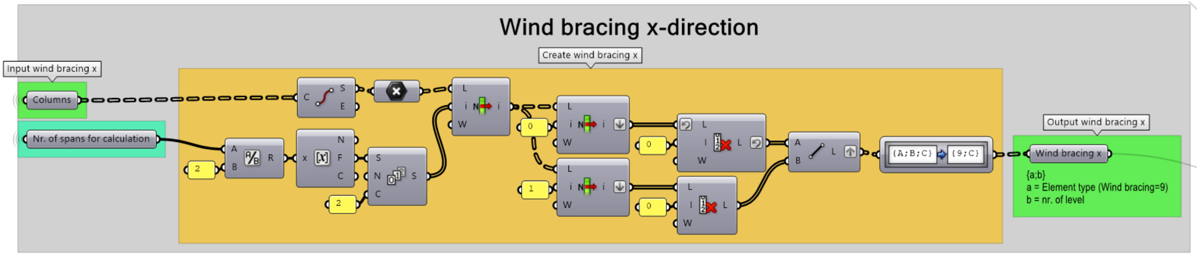

Figure 177. Wind bracing x-direction.

Ground Support Points

Points for the ground supports are created based on the geometry of the columns, see Figure 178.

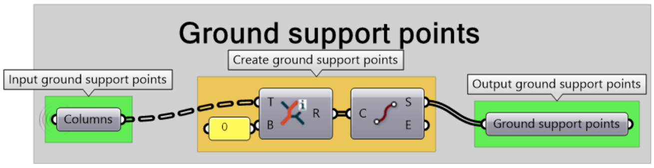

Figure 178. Ground support points.

Columns Sides

The columns at the sides are created to provide a stabilizing frame to the main structure. These columns are found in the right part of Figure 174 and are created by the logic of Figure 179.

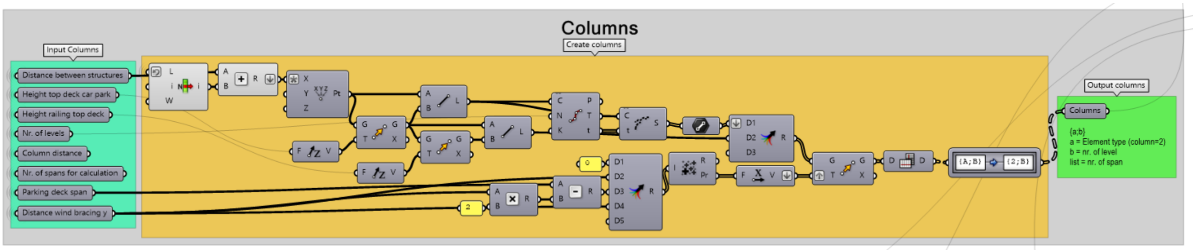

Figure 179. Columns sides.

Side Girders & Top Level Railing

The stabilizing frame also contains side girders and a top level railing, which are created in the logic of Figure 180.

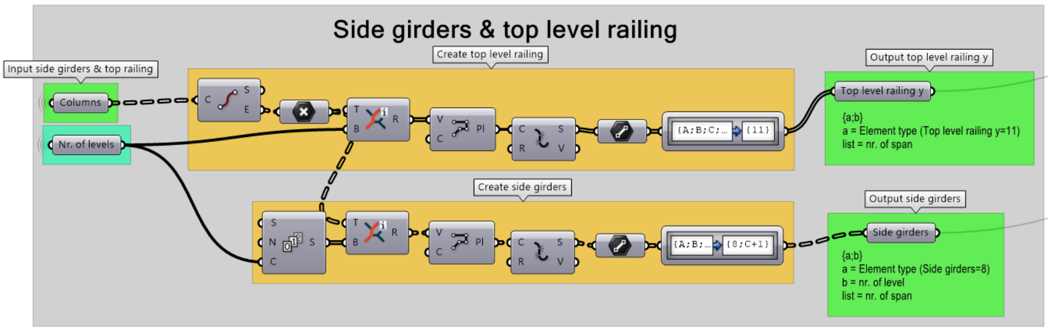

Figure 180. Side girders & top level railing.

Wind Bracing Y-direction

The last elements of the stabilizing side frame are the wind bracings in y-direction, see Figure 181. Again, only slanted elements in one direction are crated, while in reality elements in both directions are used.

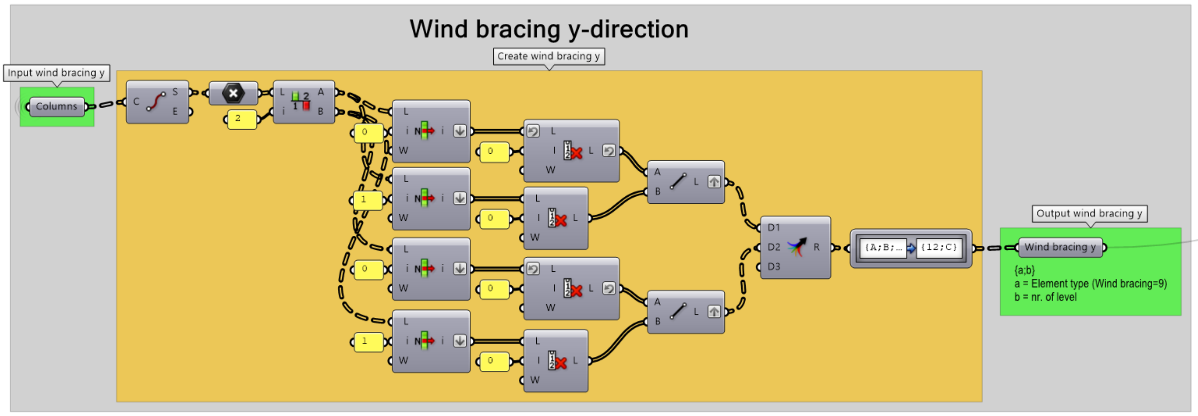

Figure 181. Wind bracing y-direction.

H.4 Setup of XML File for Structural Analysis in SCIA

In this part of the script, using the plugin Koala, a .xml file is to be later used as input for the structural analysis of the system in SCIA. This file contains amongst others geometry including materials and cross-sections, loads and settings for the structural analysis.

Project Setup

The project setup section contains general settings for start up of a SCIA project, see Figure 182.

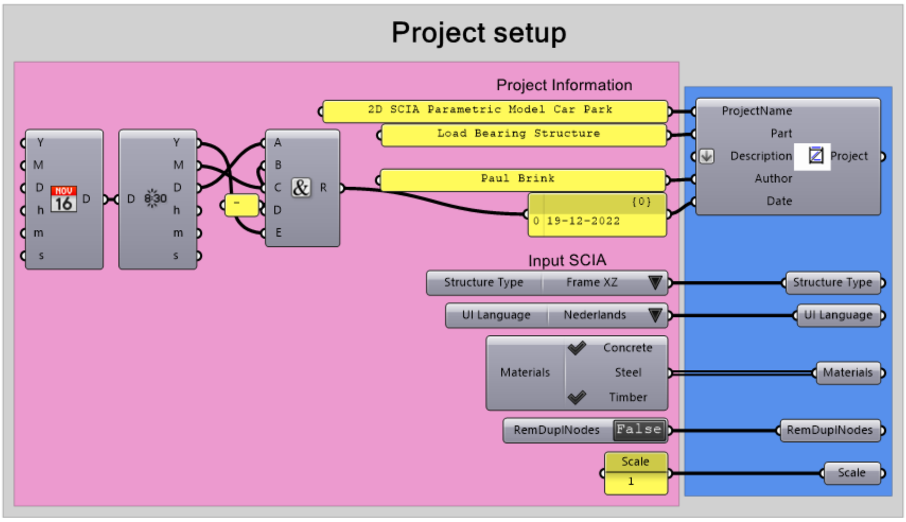

Figure 182. Project setup.

Create .xml Output File

This section includes the logic required for the creation of the .xml output file. This includes a button for creation of the file as well as names and locations of the .xml and xml.def files, see Figure 183.

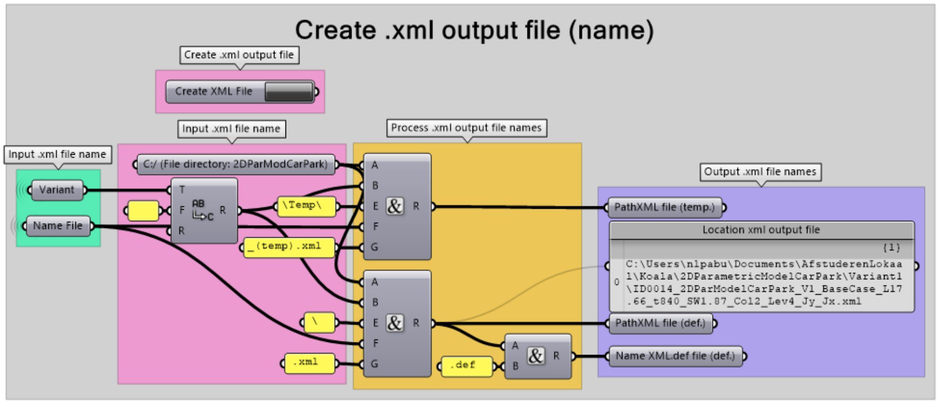

Figure 183. Create .xml output file (name).

Layers

This section creates the different layers of the model in SCIA. This includes a layer for the geometry (frame) and one for the loads, see Figure 184.

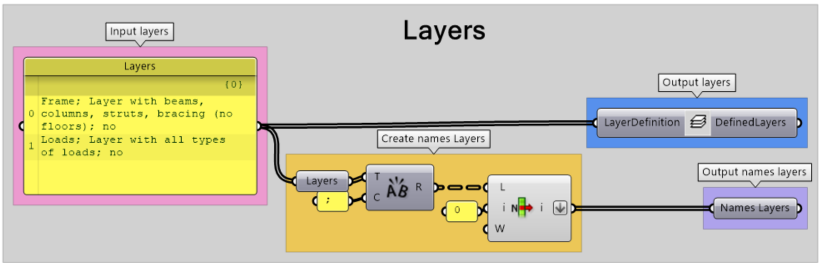

Figure 184. Layers.

Cross-sections

The creation of the cross-sections and applying them to various elements consists out of various sections presented below.

Cross-section Library

In Excel, a library is created, which contains the cross-sections dimensions for various types of timber and steel elements. For timber, different sections are found for glulam, glued solid timber and solid timber beams. For steel, different sections are included like RMS, CMS, L, RD and FL sections. In this section, the library from Excel is imported and split out for the various types of sections, see Figure 185.

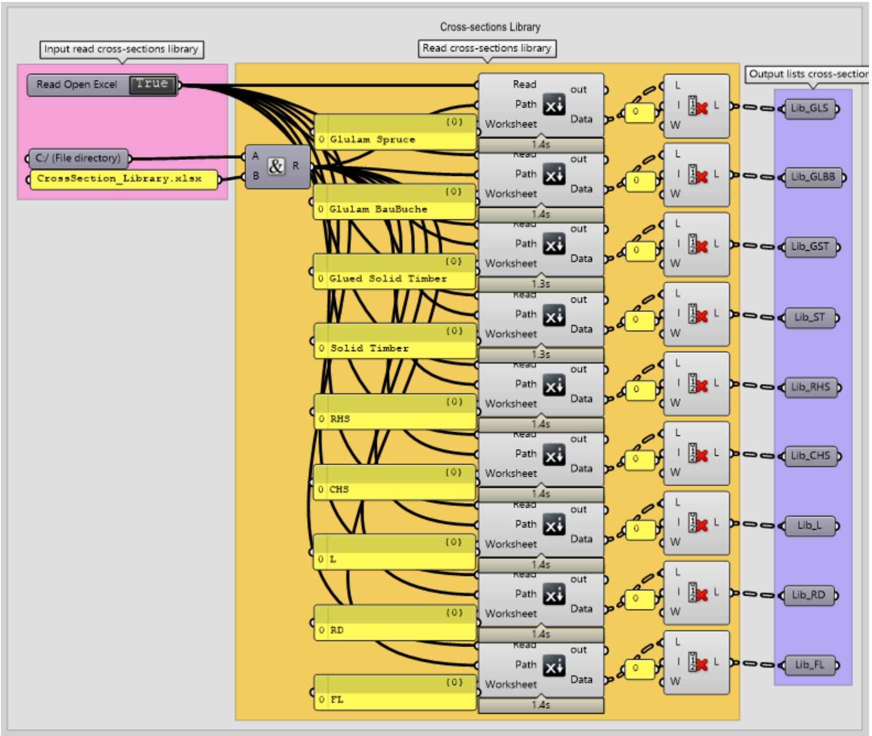

Figure 185. Cross-section library.

Material Types & Classes Library

The different material types and classes for the timber and steel are manually created to form a library to later choose from, see Figure 186.

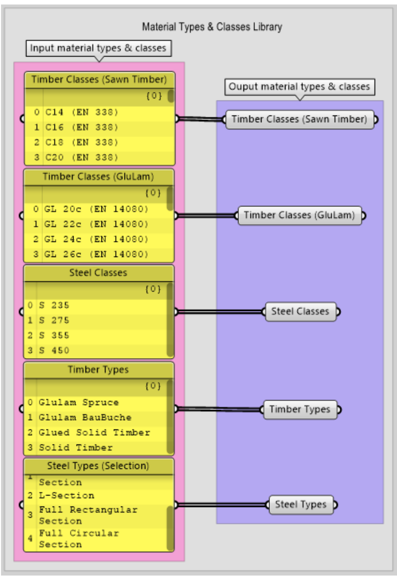

Figure 186. Material types & classes library.

Element Names

The names of all the different elements are manually added to the script and sorted into a tree data structure, see Figure 187.

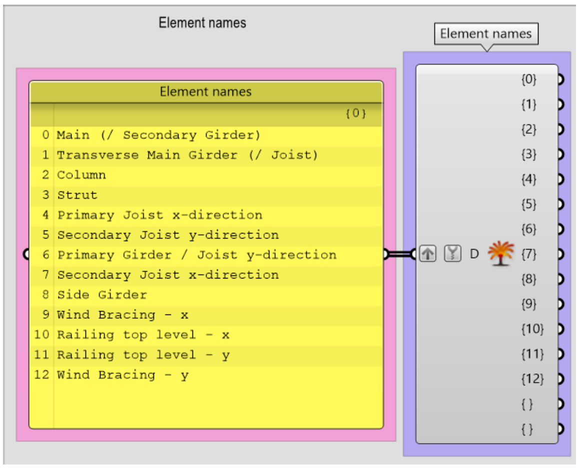

Figure 187. Element names.

Cross-section Selection

For each group of elements, a cross-section is applied for analysis in SCIA. For these elements, the material, material type, material class, height of the cross-section and finally to be applied cross-section has to be selected from a list of inputs as presented in Figure 188. The lists from which can be chosen are updated based on the choice in a previous aspect. Updating these lists is done in the logic from Figure 189, which includes all the libraries presented above and imports the choices from the drop-down menus. This also creates the name of the cross-section in the format required by the Koala plugin.

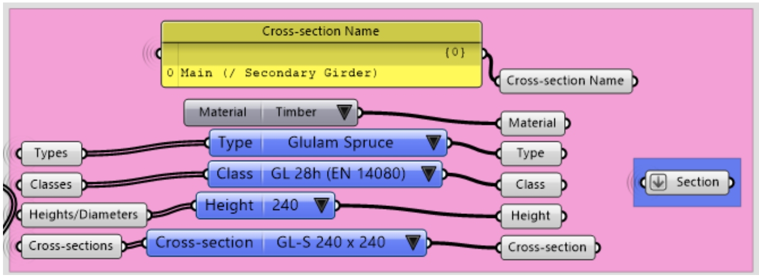

Figure 188. Input for selection of cross-section for main girders.

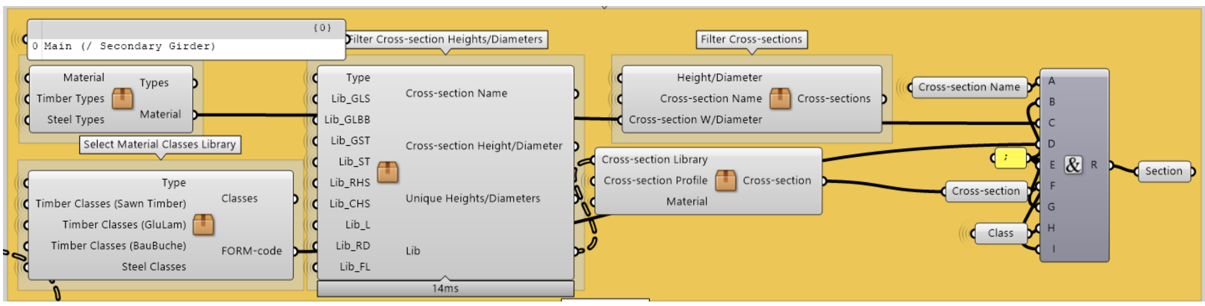

Figure 189. Logic for selection correct libraries to be applied and creation of section name.

Output Sections

Finally, all the different sections are combined and inserted into the Koala sections component (see Figure 190), which then contains a list of all sections to be applied to the various elements.

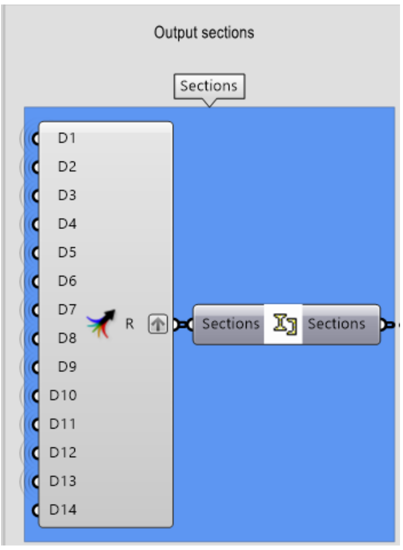

Figure 190. Output with all cross-sections.

Beams

To create the beam elements for the .xml file, first the active geometry for the considered design (sub)variant is fetched (see green block Figure 191). For these elements, the settings for the beam creation can than be selected in the pink block.

Figure 191. Input geometry and select settings for beam creation.

A datatree of all beam elements is created as well as a tree for each type of setting, containing the setting for each beam element, see Figure 192.

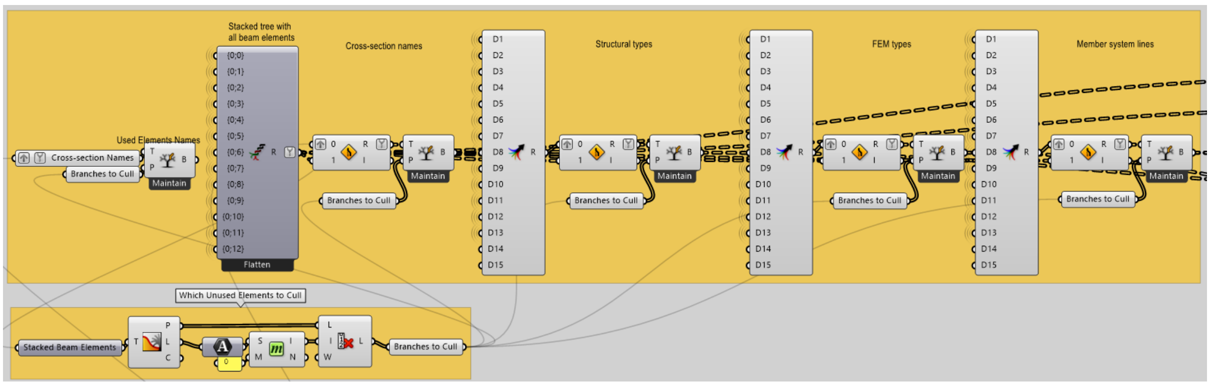

Figure 192. Merging all settings for beam elements.

Now the beam elements can be created (see Figure 193), which requires the input of the various datatrees created before.

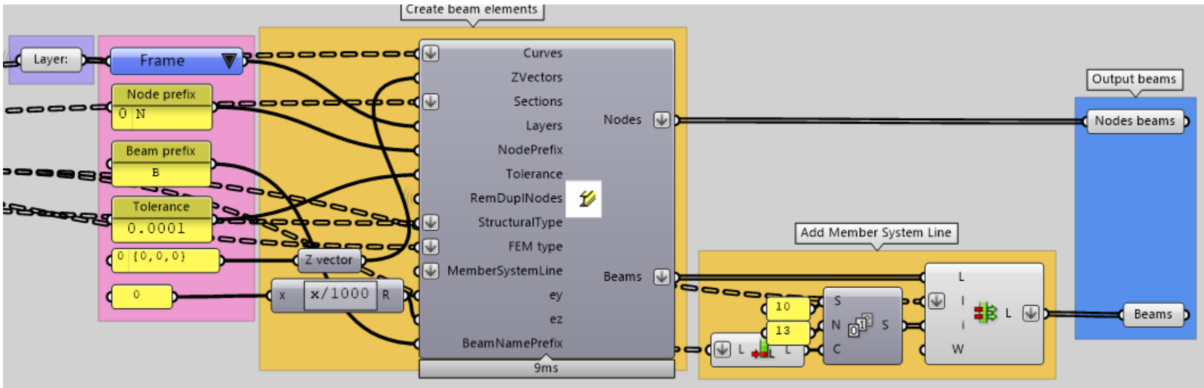

Figure 193. Creation of beam elements.

Finally, the list of beams is sorted according to the data structure that was applied to the creation of geometry, see Figure 194. This later makes it easier to find the correct beam name according to a line selected from the geometry.

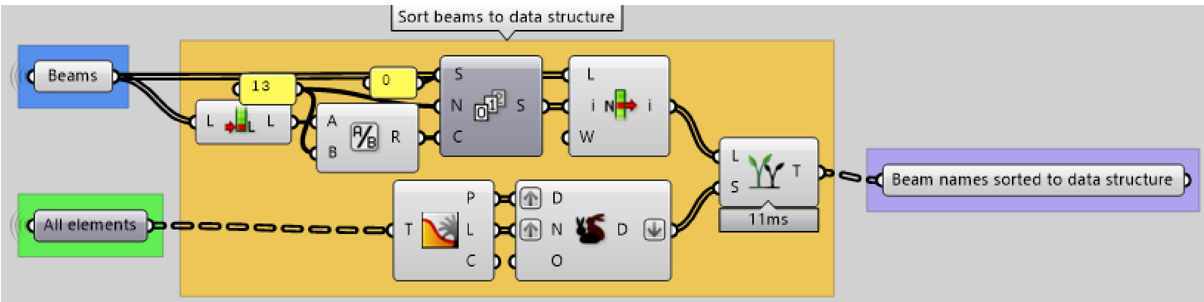

Figure 194. Sorting beams to data structure of geometry.

Supports

Supports are created for the structure. This is done by selection of the support points in the geometry and finding the nodes from the beam and column elements that apply to these nodes. Furthermore, the degrees of freedom for the support points are selected, see Figure 195. Finally, since 2D is considered, Rx and Rz are removed as these otherwise create problems with importing the .xml file in SCIA.

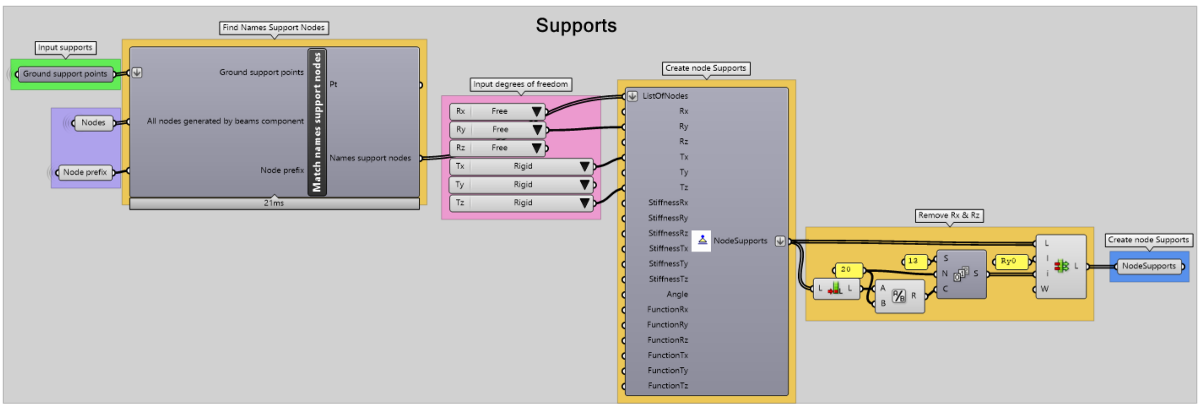

Figure 195. Supports.

Hinges

For the various beam elements, hinges are created. From the list of beams, the beams to be considered are selected based on the name of the elements. Next, the settings for the hinges on one or both sides of the beam can be selected, after which the hinges are created, see Figure 196. All hinges for all elements are finally added into a single data tree.

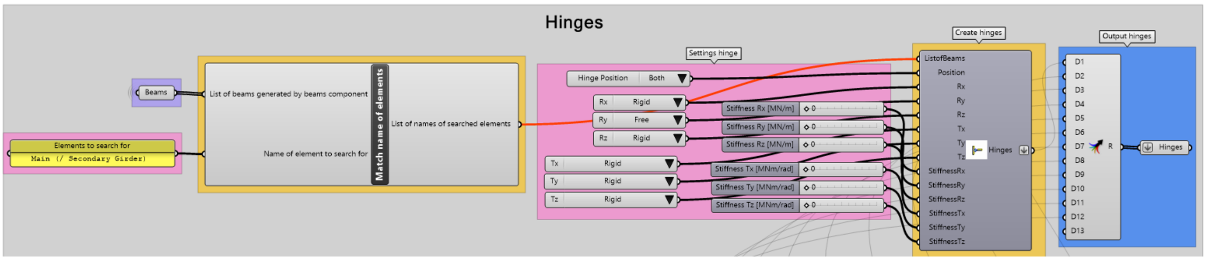

Figure 196. Hinges.

Load Cases & Groups

The different load cases and groups are created in the logic of Figure 197.

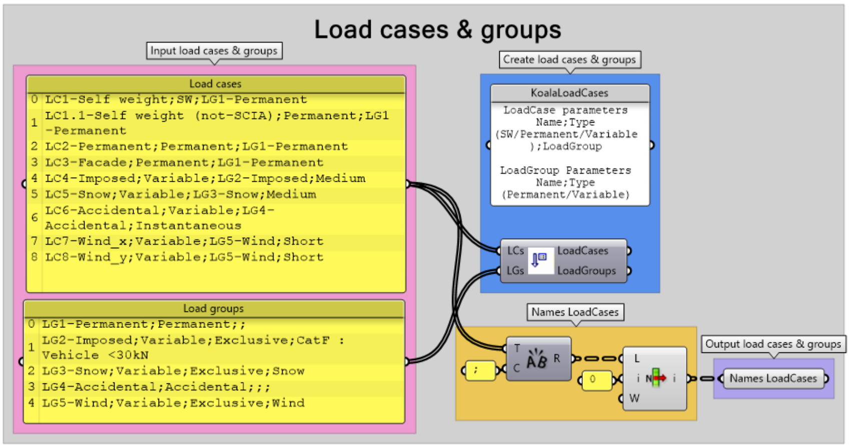

Figure 197. Load cases & groups.

Load Combinations

Different types of linear load combinations are created, see Figure 198. These include load combinations for the Ultimate Limit State and various Serviceability Limit States (characteristic, frequent and quasi).

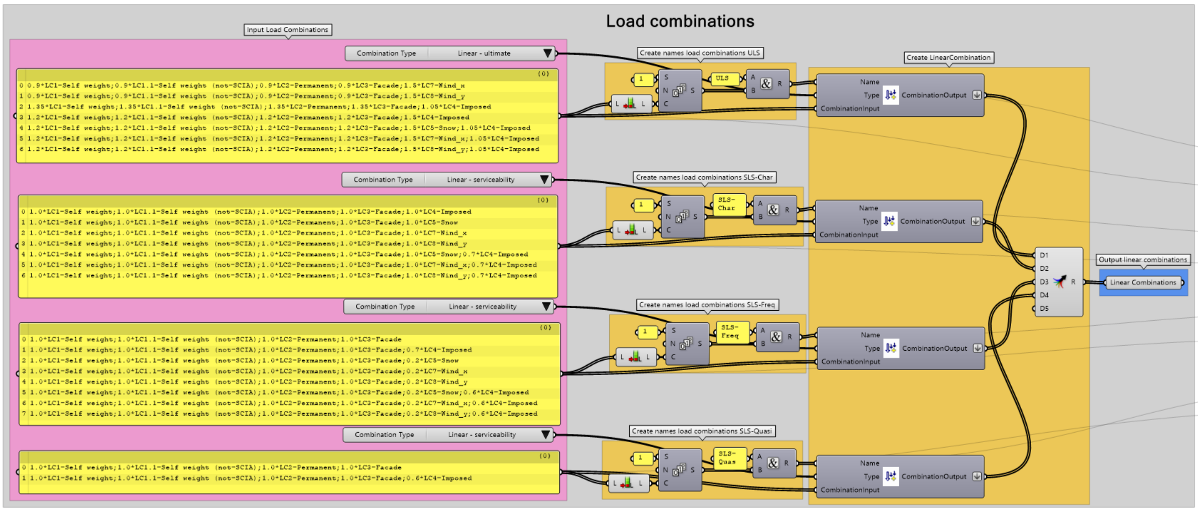

Figure 198. Linear load combinations.

Loads

The various loads are created by the Koala plugin for the .xml file. For the creation of the self-weight of the floors, dead load and imposed load, the logic of Figure 199 is used. This requires input of the value of the load, correct load case and elements the load should be applied to.

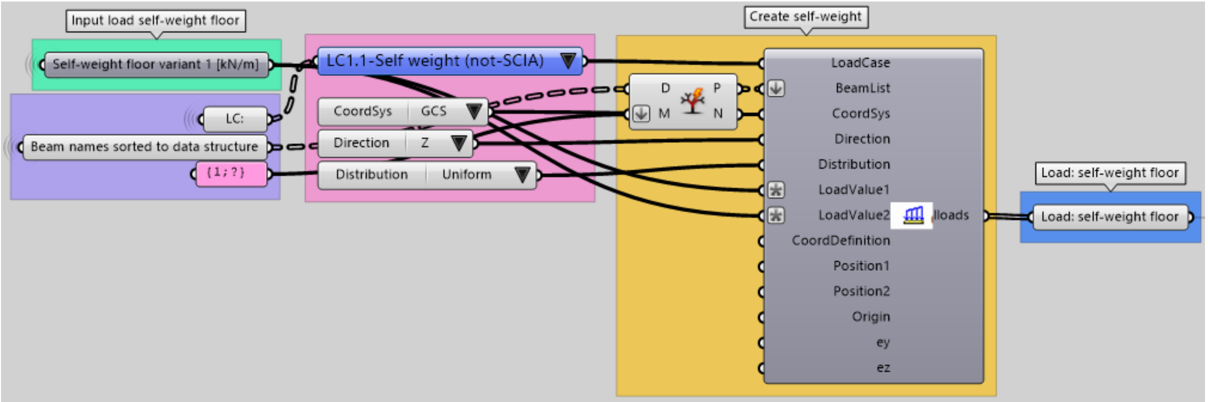

Figure 199. Creation of loads for self-weight floors (same logic for dead load and imposed load).

The snow load and wind line load (for floors) only act on the top level floor. This requires a slight alteration of the logic, see Figure 200.

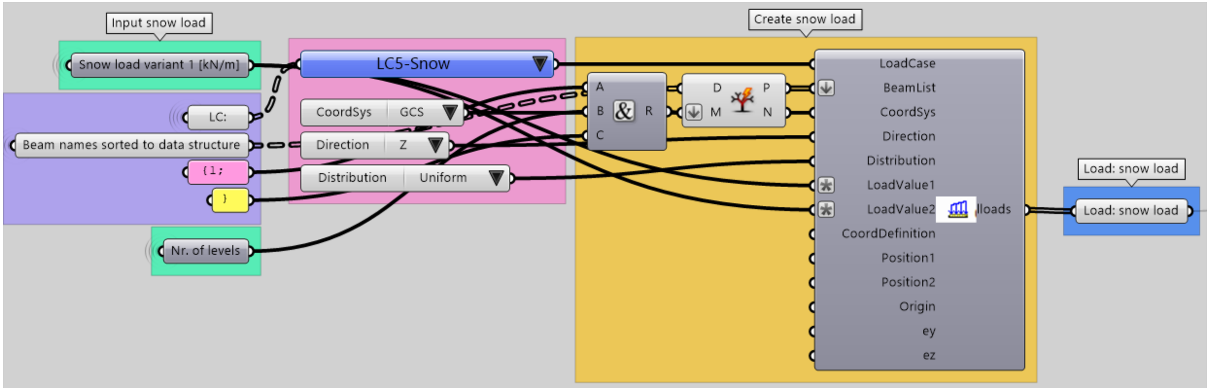

Figure 200. Creation of snow load on top floor level (same logic for wind on top floor level).

The creation of the loads for the self-weight of the façade requires using the input of the different height sections of the façade. These loads were calculated earlier and are applied to the correct load case, see Figure 201.

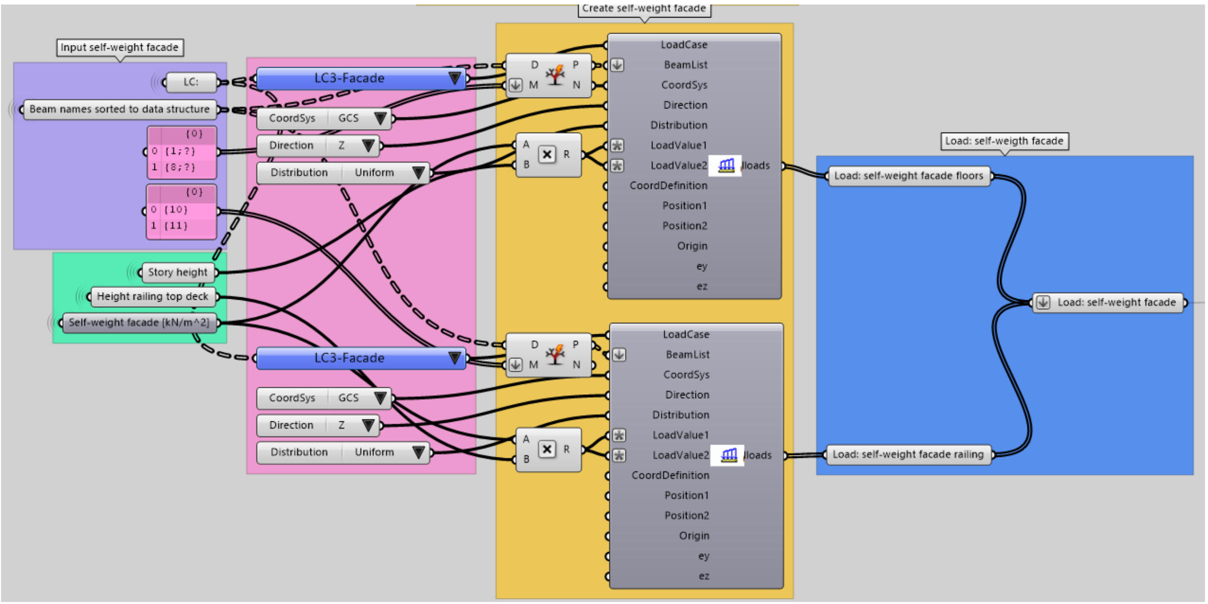

Figure 201. Creation of load self-weight façade.

For the creation of point loads for wind loading, first the correct nodes to which the loads should be applied have to be found. This is done by finding the selection of these point from the geometry and finding the corresponding nodes from the beam (column) elements generated by the Koala element, see Figure 202. Next, for both the x- and y-direction, the point wind loads that were calculated earlier are applied and the wind point loads are created by Koala. Again, for the 2D model, Rx and Rz are removed.

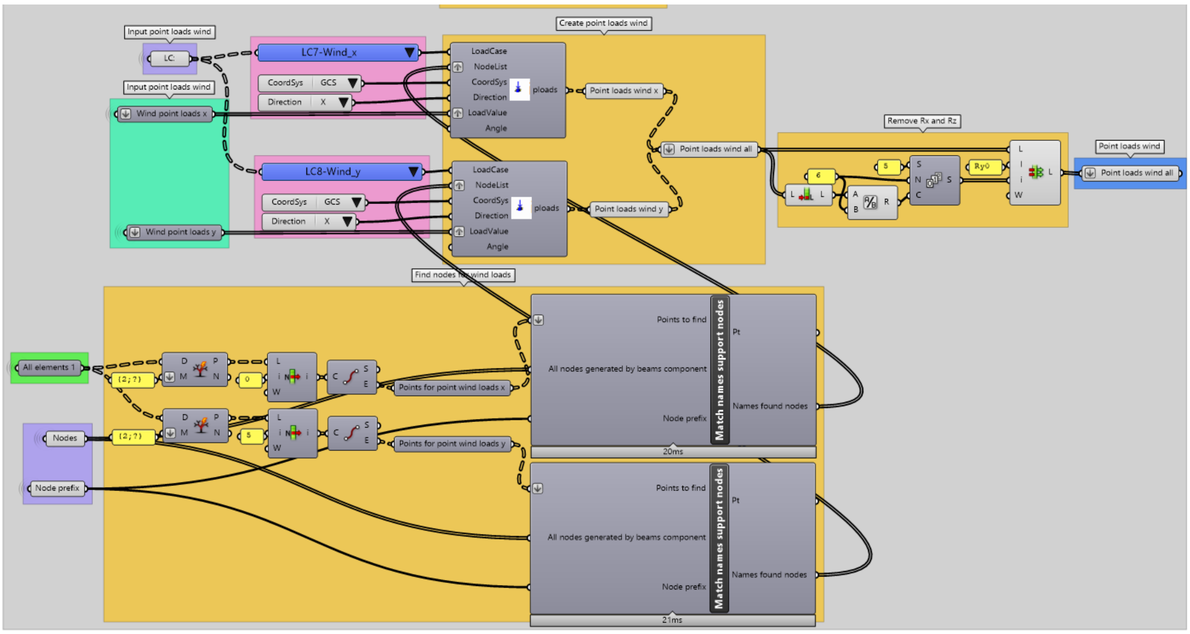

Figure 202. Create point loads wind.

H.5 Creation of .xml File

Create .xml File by Koala

Using all the input created in the previous section, the Koala CreatXML component combines all the input and creates a .xml file for structural analysis in SCIA, see Figure 203.

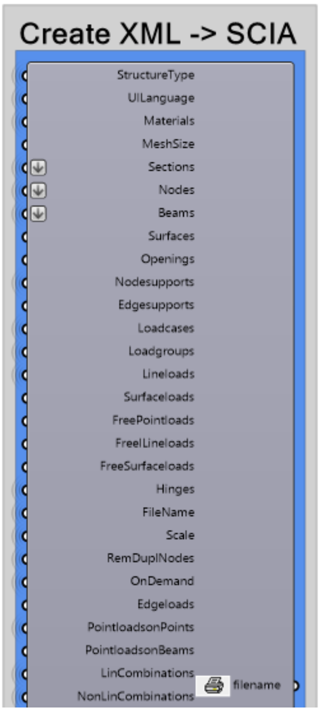

Figure 203. Koala component for creation of .xml file.

Editing of Koala .xml File

The .xml file created by the Koala plugin is not complete. The Koala plugin does not yet contain all possible inputs that are available in SCIA. Some of these inputs are required for proper analysis of the structural models. Such inputs are for example the load group and load duration. Therefore, these are manually added to the .xml file by copying the file generated by Koala and saving a newer version.

Prepare .xml File

The .xml file created by the Koala plugin is read and transferred into a .txt file, see Figure 204. Each .xml file is supported by an additional xml.def file, which also needs to be updated. In the newer .xml file, this updated xml.def file should be used.

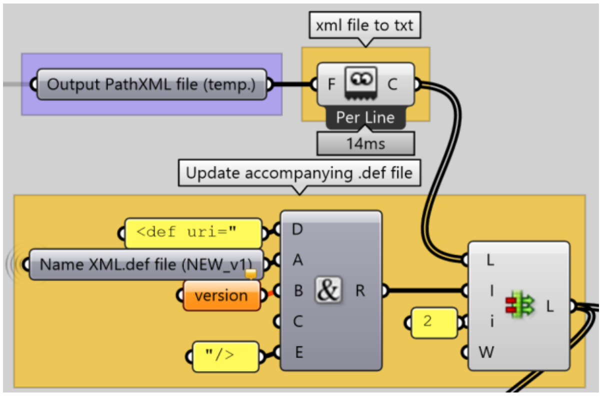

Figure 204. Preparation of .xml file for editing and updating .def file.

Add New Section

Adding a new section to the .xml file is always done in four steps. The first step is to find the section which needs to be edited or location where the new section should be inserted. This is done by finding the unique ID string in the .txt file and snipping the section considered from the .txt file. In Figure 205, this step can be seen for finding the load group section.

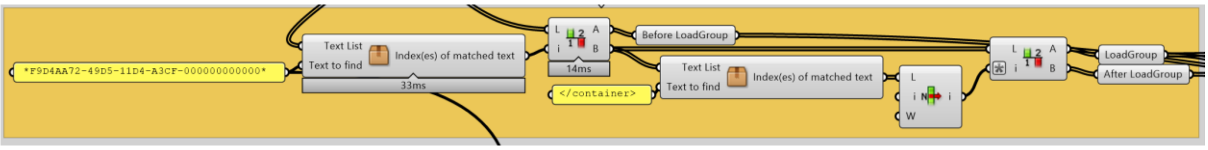

Figure 205. Find load group section.

The second step is to retrieve the input that has to be inserted into the .txt file. See Figure 206 for the retrieval of input for the load groups and types.

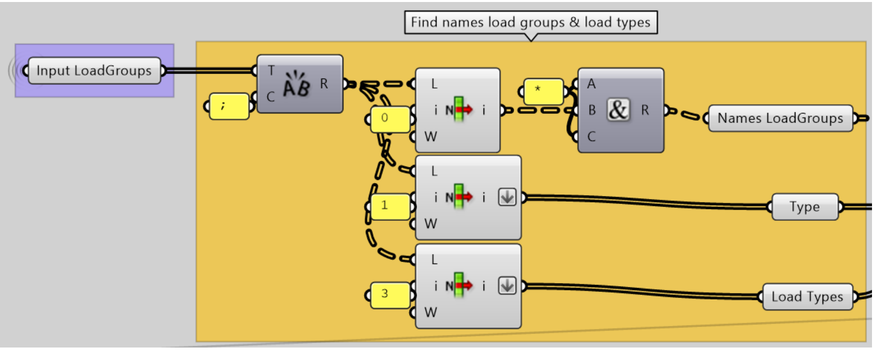

Figure 206. Retrieve input for load groups & types.

The third step is to create the section that has to be inserted into the .txt file. The retrieved input is correctly formatted and a code block is created. Finally as the fourth step, the created section is inserted into the .txt file, replacing the original section, see Figure 207.

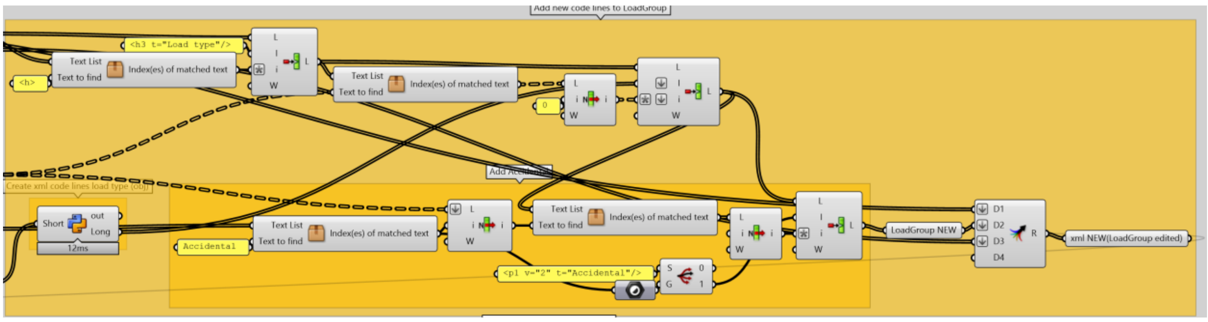

Figure 207. Add new code lines for load groups to file.

Load Case Section

Similar to the steps for adding the section for load groups, the same procedure is followed for adding the load case section, see Figure 208 trough Figure 210.

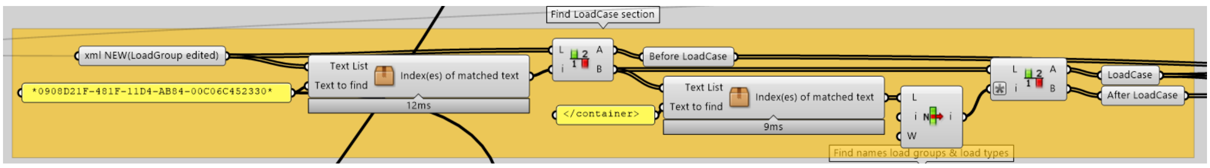

Figure 208. Find LoadCase section.

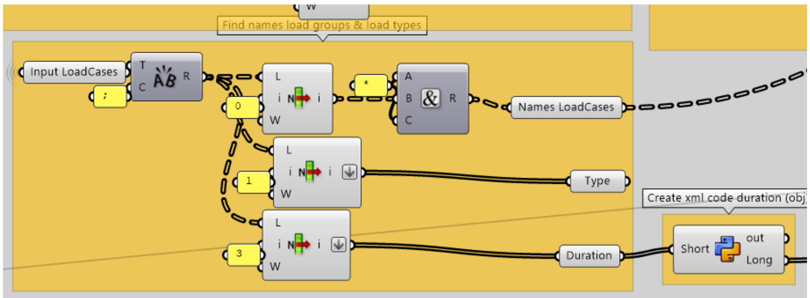

Figure 209. Retrieve input for load case.

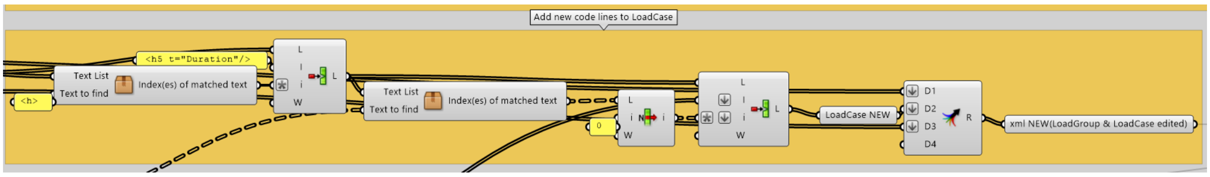

Figure 210. Add new code lines for load cases to file.

ProjectData Section

Also a section for project data needs to be inserted. This time, the input is not retrieved, but directly specified in a panel, see Figure 211 and Figure 212.

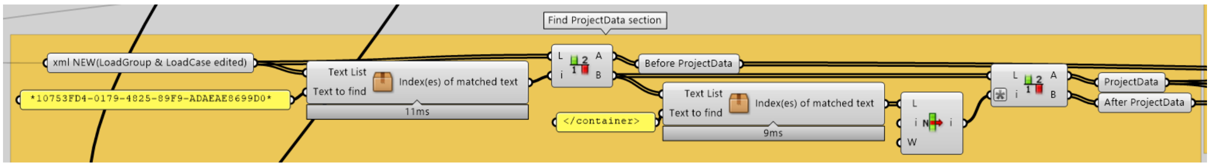

Figure 211. Find ProjectData section.

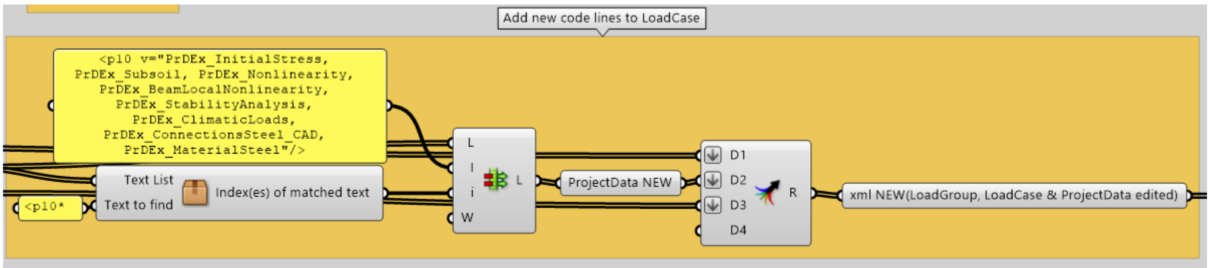

Figure 212. Add new code lines for project data.

Write Updated .xml File

The last step is to create the updated .xml file. This is simply done by transforming the updated .txt file into a .xml file, see Figure 213.

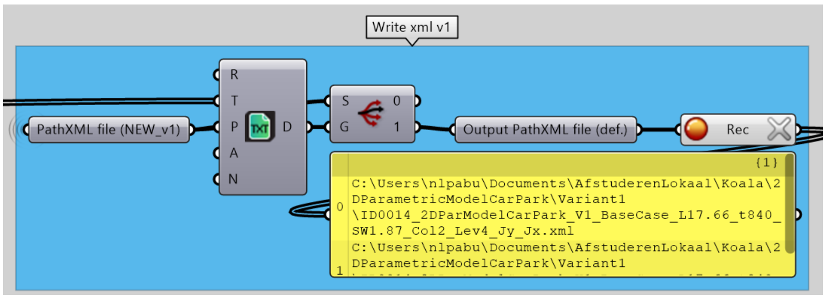

Figure 213. Write updated .xml file.

Editing of Koala .xml.def File

The .xml.def file contains all the building blocks that can be used in the .xml file, which uses this .xml.def file. Within these building blocks, elements are presented, for which input values can be given. However, in the .xml.def file created by the Koala plugin, not all the elements, which are available in SCIA are present in the blocks. Therefore, they need to be added manually, so they can be used.

Prepare .xml.def File

The .xml.def file created by the Koala plugin is read and transferred into a .txt file, see Figure 214.

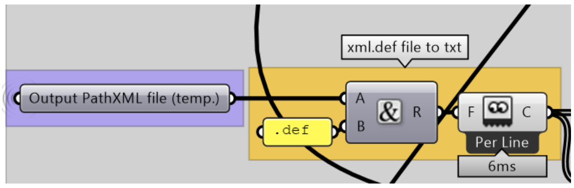

Figure 214. Preparation of .xml.def file for editing.

Add Element to Block

Adding an element to a block is done in three steps. The first step is to find the load group block, see Figure 215.

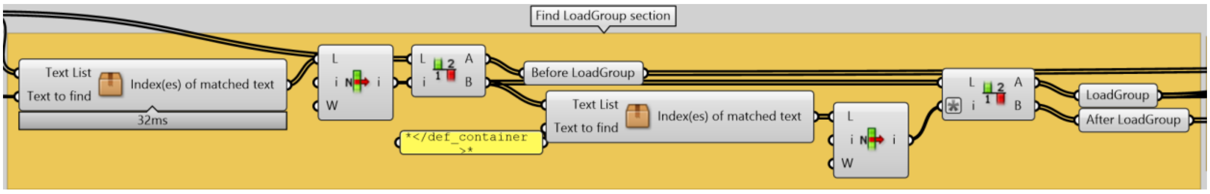

Figure 215. Find load group block.

The second step is to increase the size of the block with the number of elements that are added, see Figure 216.

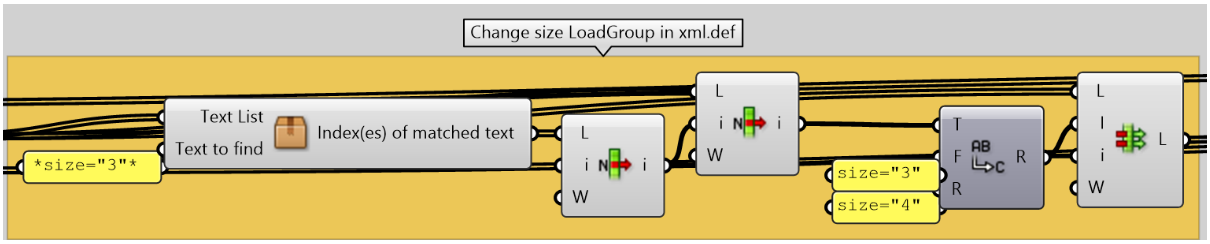

Figure 216. Change size LoadGroup.

The third step is to add the new code lines for the additional element to the building block. This block is then inserted into the .txt file, replacing the old building block, see Figure 217.

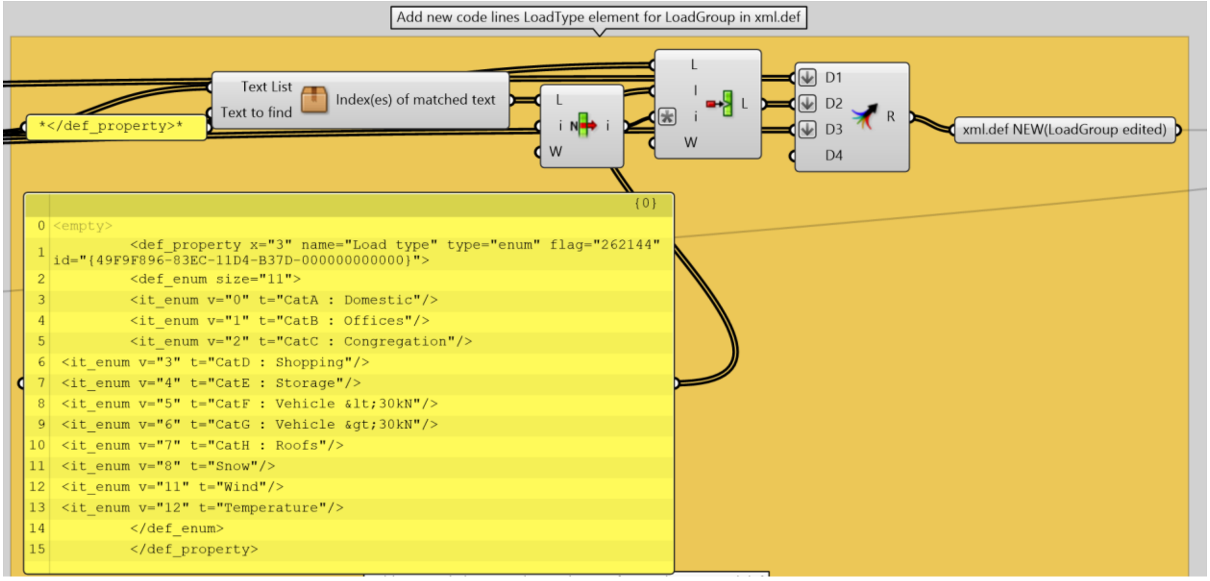

Figure 217. Add new code lines for additional element to building block and replace building block in .txt file.

Load Case Block

Similar to the steps for adding the elements to the block for load groups, the same procedure is followed for adding the load case elements, see Figure 218 through Figure 220.

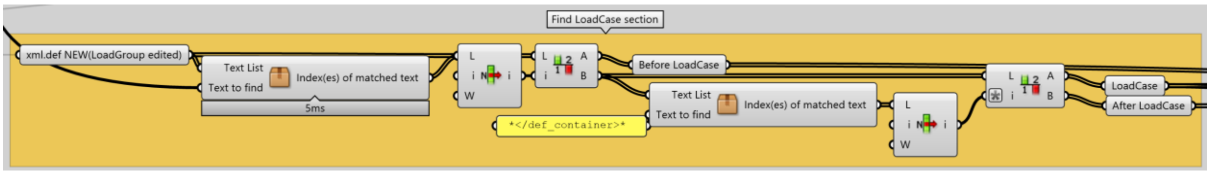

Figure 218. Find load case block.

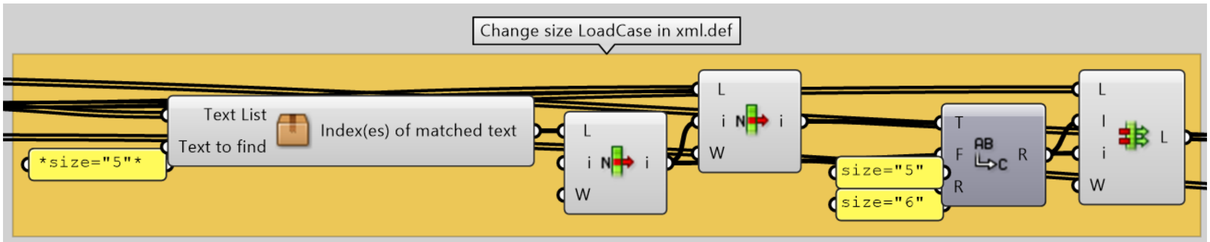

Figure 219. Change size load case block.

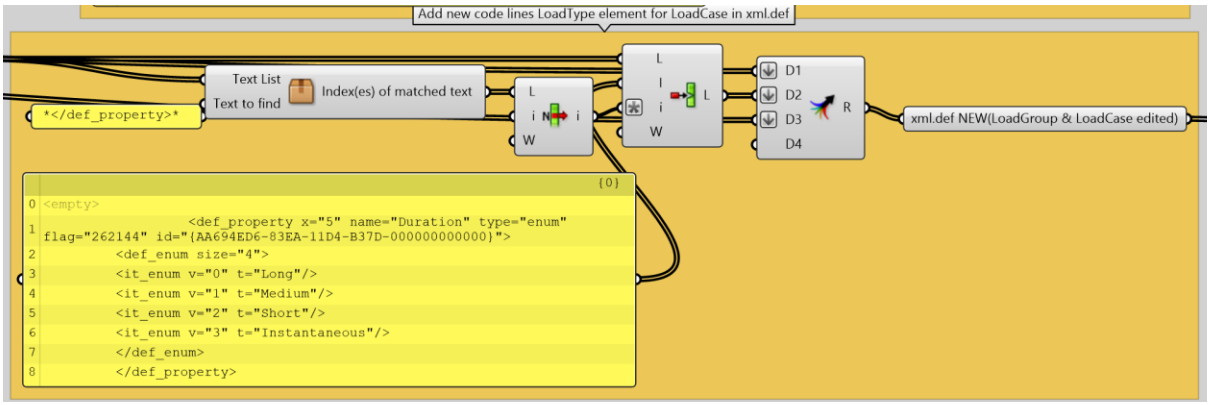

Figure 220. Add new code lines for additional element to building block and replace building block in .txt file.

Write Updated .xml.def File

The last step is to create the updated .xml.def file. This is simply done by transforming the updated .txt file into a .xml.def file, see Figure 221.

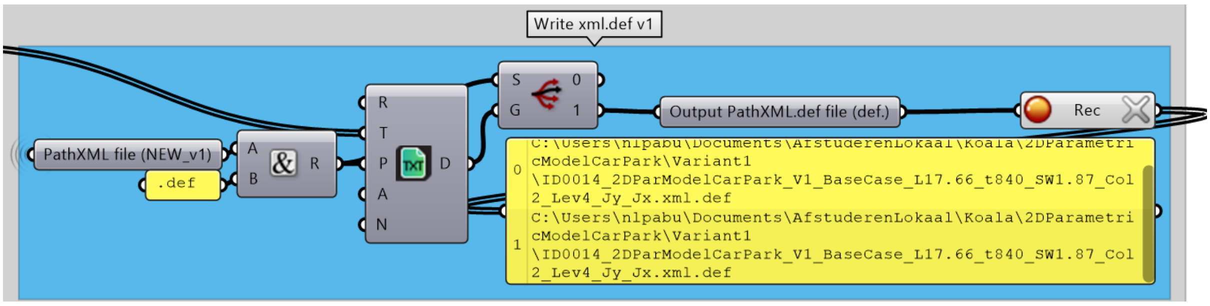

Figure 221. Write updated .xml.def file.