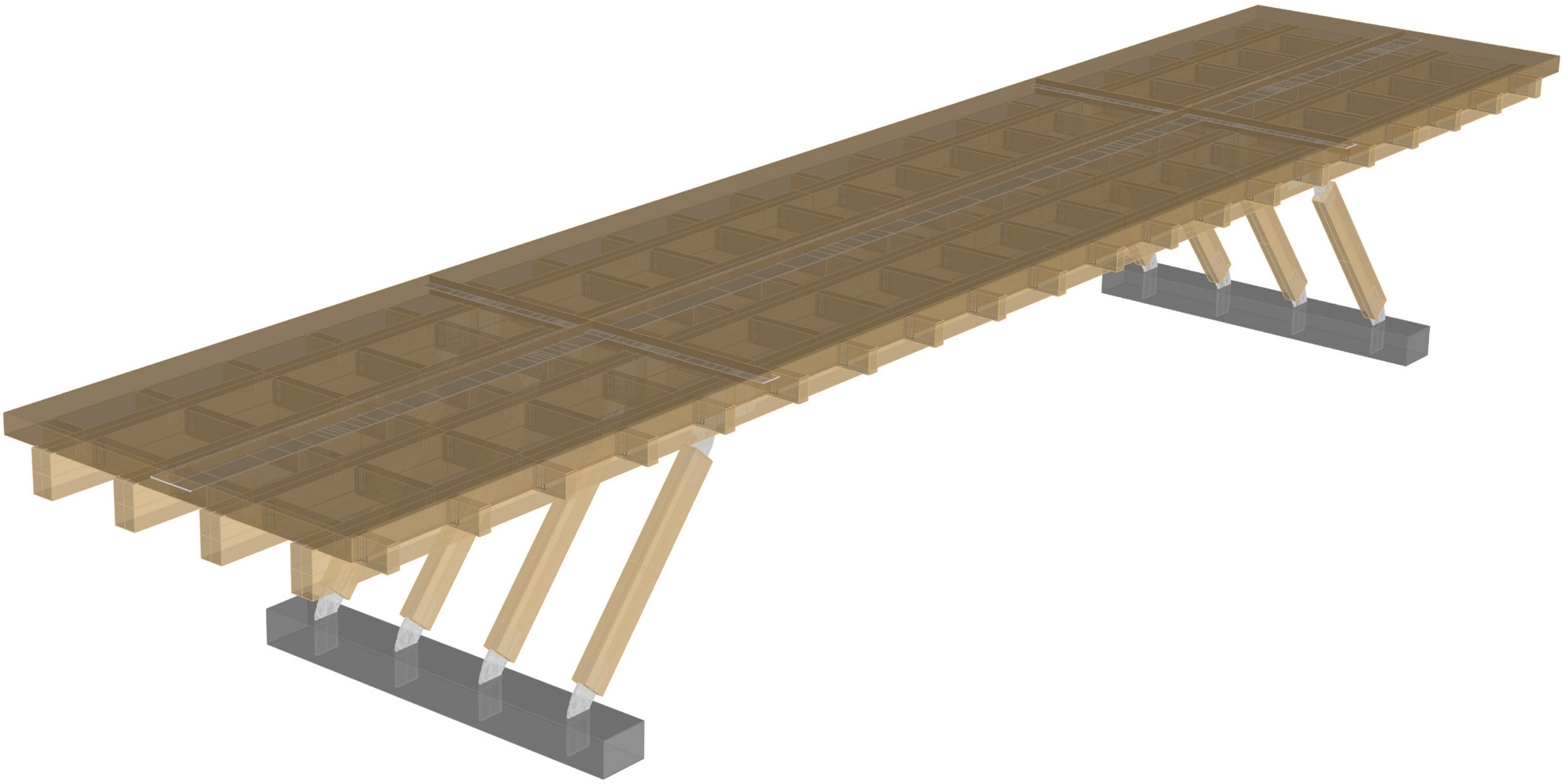

DEVELOPMENT OF A MODULAR SYSTEM FOR CIRCULAR TIMBER HIGHWAY BRIDGES

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ARUP

Development of a modular system for circular timber highway bridges

by

V.H. Sluijter

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at the Delft University of Technology,

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An electronic version of this thesis is available at http://repository.tudelft.nl/.

Preface

This thesis is the last step towards obtaining my Master's degree in Building Engineering at Delft University of Technology. During my studies my two main interests were sustainability and structural design, which I combined in this research.

Currently, the transition to the circular economy is becoming more relevant with the consequences of the linear economy becoming more visible. Change needs to happen in all industries and thus also in the construction industry. Together with Arup, the combination of circular principles with highway bridges was created and resulted in this research topic. During the past ten months, I have enjoyed working on this topic and I have gained a lot of knowledge, both on structural and sustainability level.

My supervisors helped me gaining this knowledge and shaping my research. First, I want to thank Edwin Thie for the daily supervision, feedback on my research, and good discussions on each part of my research. Furthermore, I would like to thank Jan-Willem van der Kuilen for his critical view and guidance in timber engineering. I would like to thank Michele Mirra for the helpful discussions on the connections in the timber and supervision when I needed it. Next, I would like to thank Marco Schuurman for the practical advice on the design and the critical comments from a different point of view which completed the committee. Lastly, thanks to all colleagues in the infrastructure department of Arup Amsterdam, for providing an inspiring environment and showing interest in my research, although we mostly met online. Special thanks to Laetitia Koning for helping me improving my Grasshopper skills and developing the parametric model for this thesis.

Next, I want to thank my parents, sister, friends, and housemates for the support during my research: by reading parts of my report, putting things into perspective, and mostly keeping me motivated.

Vera Sluijter
October 2021

Abstract

The infrastructure industry currently deals with two issues: the deterioration of (highway) bridges, and the urge to reduce emissions by the construction sector. Rijkswaterstaat must repair or replace hundreds of bridges in the upcoming decades. At the same time, the impact of human behaviour on climate change becomes more visible and must be reduced. The most structural solution to diminish the human impact on the environment is switching from a linear economy to a circular economy (CE).

This research aims to develop a system for circular highway bridges by both constructing with timber and extending the lifespan. The first contributes to a lower environmental impact as timber is a renewable material that captures carbon during growth. For an outdoor timber structure, protection is crucial to prevent the timber from deteriorating due to weather influences. The lifespan extension is obtained by applying three circular principles: (1) Design for Material Efficiency (DfME), (2) Design for Adaptability (DfA), and (3) Design for Disassembly (DfD). Variant studies on typology, connections and material optimise for material efficiency. A flexible structure enables DfA: converting in function and expansion in length and width is possible. Furthermore, a modular system is developed to include DfD: connections between modules are demountable to disassemble, adapt and reuse the system. In summary, four design strategies are defined: efficient, protected, adaptable and demountable.

A parametric model simulates the structural behaviour of the modular bridge system in the software Grasshopper, using the Finite Element Model plug-in Karamba3D. This model performs quick structural analyses, explores multiple options, and executes variant studies. The first variant study is on typology: the strut typology with integrated cross girders is best suited for bridges up to a fifty-metre span within the strategies set for this research. Second, a screwed connection between deck and girders performs best when considering structural and feasibility requirements. Third, GL28h glued laminated timber and C24 cross-laminated timber suit the modular system best for respectively the beams and the deck.

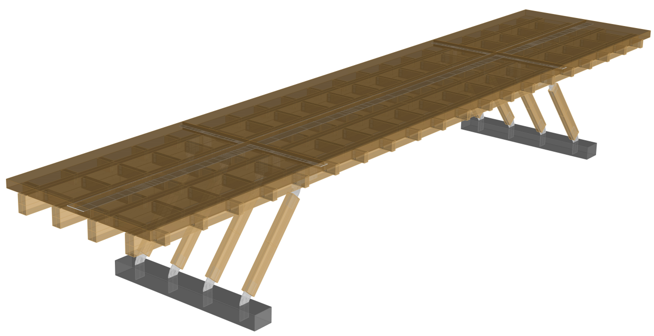


Figure 1: Three-dimensional impression of modular timber bridge with a span of 35 metres

Given the optimised aspects, this research derived a scope for the standardised modules: 5-35 metres. The construction height of the modules is 1.03 metres (Figure 1). To incorporate protection against weather influences in the structure, a cantilevering deck is applied to protect the main girders from getting wet. Additionally, a watertight membrane is added above the deck to protect the entire structure from weather influences. With these protection measures, a technical lifespan of one hundred years can be assumed, according to prEN-1995.

This research aims to provide an alternative for concrete highway bridges with a lower environmental impact. The timber bridge, a circular concrete bridge and a traditional concrete bridge are compared (NIBE Research bv, 2019a). The comparison includes the production (A1-A3), construction (A4-A5) and end of

life (C1-C4) stages. Figure 2 indicates that timber results in a lower carbon footprint for all reference periods and lifespans. The study considers the third scenario most reliable, as the technical lifespans are most substantiated. In this scenario, reductions of 18% and 47% for respectively the circular and traditional concrete bridge are established.

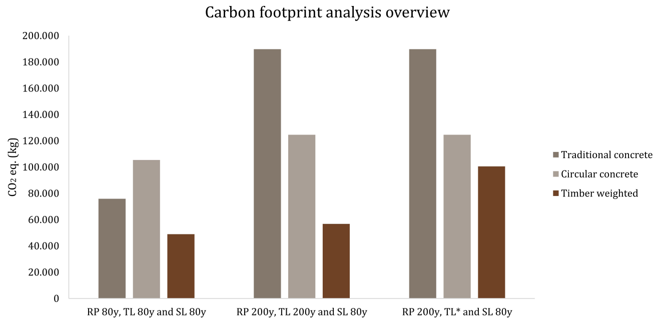


Figure 2: Carbon footprint (kg CO_2 eq.) for weighted end of life scenarios. Accounting for stage A1-A5 and C1-C4, TL* means a technical lifespan is 200 years for concrete and 100 years for timber.

One observes an additional benefit for timber when considering carbon emissions in time (Figure 3). In the short term, timber captures carbon, and emissions are delayed by one hundred and two hundred years. In contrast, concrete emits most carbon at the start of its lifespan. IPCC (2018) showed that reducing the carbon concentration in the atmosphere in the short term is crucial to limit global warming. Therefore, the nuance of carbon emissions plotted in time adds a benefit to the timber bridge compared to concrete.

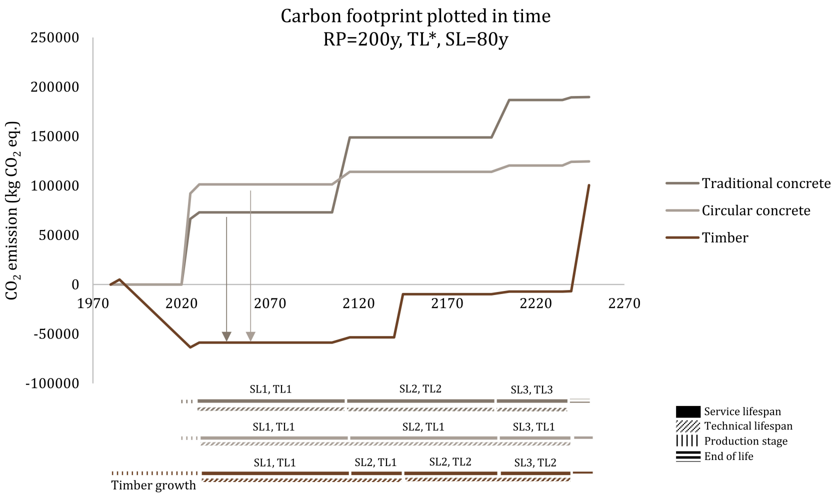


Figure 3: Carbon footprint (kg CO_2 eq.) plotted in time. Accounting for stage A1-A5 and C1-C4, TL* means a technical lifespan is 200 years for concrete and 100 years for timber.

This study concludes that Dutch highway bridges suit replacement with a modular timber alternative. The developed modular system covers 58% of the highway bridges stock owned by Rijkswaterstaat that were built in 1950-1980. Moreover, timber shows substantial environmental benefits compared to concrete. The alternative reduces the carbon footprint by 47% compared to a traditional concrete bridge. Furthermore, this study observes additional benefits when one considers carbon emissions in time, as the structure sequestrates carbon during use. Consequently, building with timber lowers the carbon concentration in the atmosphere, reducing the global warming effect.

This study recommends Rijkswaterstaat to invest in the development of circular timber highway bridges. By changing the 'business as usual' from concrete to a more sustainable alternative, the circular and climate goals can be obtained. Furthermore, the recommendation is made to clarify the regulations on quantification of environmental impact, both on data and methodology level.

Contents

I

Research framework

1 Introduction

The Netherlands has a fine-meshed transportation network. After World War II, the infrastructure developed rapidly by building more roads. To keep the roads safe in this denser network, one constructed multi-level crossings. Consequently, many highway bridges date from 1950 until 1980. Concrete was already a well-known material at that time and widely available. Therefore, most of the highway bridges are constructed in concrete. Nowadays, increased traffic and heavier trucks intensify the use of highway bridges. Consequently, many highway bridges no longer comply with the current regulations. Rijkswaterstaat must improve or replace these bridges (Rijkswaterstaat, 2020b).

The main distinction between bridges and highway bridges (or overpasses) is that the first cross water and the latter cross roads. This distinction influences the context, requirements, and data of existing bridges. Many highway bridges exist in the Netherlands; hence creating a more sustainable alternative can significantly affect the environmental impact of the infrastructure industry. Therefore, this research focuses on highway bridges. For the readability of the report, highway bridges are referred to as bridges.

A more sustainable approach to bridge design can be implemented now, as bridges reach their end of life phase. The world is running short on raw materials and must limit the emission of greenhouse gasses. Many countries signed the Paris Climate Accord (United Nations, 2016) to comply with goals on mitigating climate change. The Dutch goals are reducing the CO_2 emissions by 49% by 2030 and 95% by 2050, compared to the emissions in 1990 (Rijksoverheid, 2018). The construction sector requires significant change to reach these climate goals. In 2019, the global construction industry contributed 10% to the total emission of CO_2, as presented in Figure 1.1 (United Nations Environment Programme, 2020). This share includes emissions in the construction phase, whereas emissions during the lifespan of a structure are not included. Replacing bridges plays an essential role in the activities of the construction sector. By replacing with more sustainable alternatives, the climate impact of this sector can be reduced. The industry works on more sustainable concrete and steel. This innovation takes time, whereas timber is an available alternative which can be implemented in the short term.

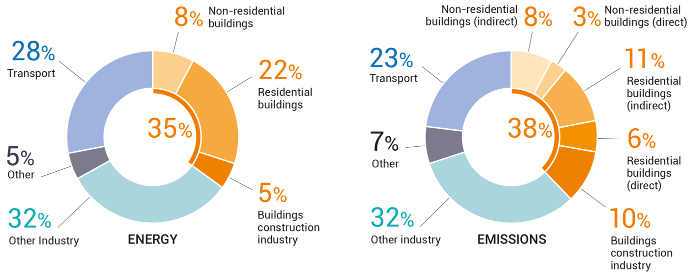


Figure 1.1: Emissions of the construction sector (United Nations Environment Programme, 2020)

1.1. Problem statement

1.1.1. Climate change

A large number of people, combined with the high level of human consumption, puts immense stress on the environment. The consequences of this stress become more and more visible. In 1987, the Brundlandt committee derived a definition for sustainable development (Brundlandt, 1987) "Sustainable development is development that meets the needs of the present without compromising the ability of future generations to meet their own needs." Worldwide, one still considers this the primary definition of sustainable development. The Paris Climate Accord used it in setting concrete aims on emissions for 2030 and 2050 (Rogelj et al., 2016). The previous section provided the general reduction goals for the Netherlands in all sectors. In addition, Rijkswaterstaat set their goals even higher, where Rijkswaterstaat (2020a) aims to work circular by 2030 and be circular by 2050. More information on the circular economy is presented in Section 4.1. At this point, two complementary routes towards a circular economy are (1) to build with renewable materials and (2) to keep products in use for as long as possible. Both are included in this research.

1.1.2. Renewable materials

Building with renewable bio-based materials provides a serious alternative to traditional high carbon materials like steel and concrete. One avoids the depletion of finite materials, and carbon emissions are reduced.

Arup investigates the possibility of using timber as a primary construction material in bridge design. Multiple projects in the building industry are already executed in timber, but application in bridge design occurs sporadically. Timber is a renewable product, resulting avoiding the use of finite resources. Sustainable forest management is a necessary boundary condition, ensuring the constant size of forests. Furthermore, wood captures carbon during growth and retains it when it is used in construction. When timber is at its end of life, the carbon re-enters the atmosphere. However, this carbon capture delays the carbon emission, thus temporarily lowers the concentration in the atmosphere. Figure 1.2 illustrates this principle. The CO_2 pulse (set to 1) in the atmosphere reduces over time as it is absorbed by the ocean, soil and other organisms. The benefit of the pulse delay is defined as the cumulative reduction of carbon load in the atmosphere. With a constant level of construction in timber, the concentration in the atmosphere can be constantly reduced.

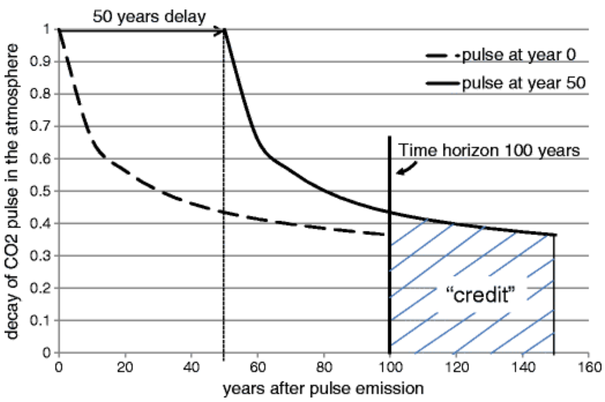


Figure 1.2: Residence time of CO_2 in the atmosphere and the resulting credit of a delayed pulse (Vogtländer et al., 2014)

O'Born (2018) performed a study on the global warming potential (GWP) of the construction material of bridges. The study executed a Life Cycle Assessment (LCA) on two bridges: a superstructure either made of concrete or timber, using the same materials for the substructure. Figure 1.3 presents the conclusion, considering the construction (A) until the demolition (C) stages. Stage D (benefits out of the system boundaries) is not included in this comparison.

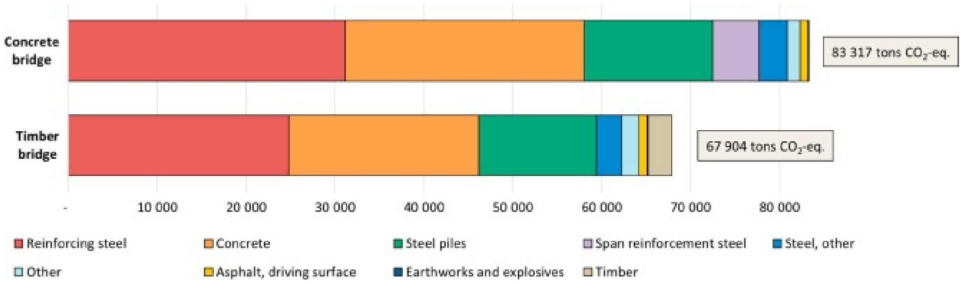


Figure 1.3: Global warming potential of timber and concrete bridge (O'Born, 2018)

The amount of reinforcing steel and concrete cause a significant difference in GWP. As more concrete is used, more reinforcing steel is required. Furthermore, higher loads are placed on the substructure due to its higher self-weight, resulting in more material use. Both increase the GWP of the concrete bridge. In conclusion, the study demonstrates that timber bridges result in a lower GWP.

1.1.3. Service life

Nowadays, one designs infrastructural works in the Netherlands for a service life of 100 years (NEN-EN 1990 - NB). Service life is the desired lifespan, where "the lifespan of a building component can be defined as the period a building component can fulfil its requirements" (Hermans, 1999).

Innovation in transportation led to heavier (freight) traffic. Bridges must withstand these increased loads. Consequently, the requirements for infrastructural works changed multiple times during the last century. Furthermore, roads expanded, and thus the bridges crossing these also must be expanded. However, adjusting is often more expensive than replacing, as bridges are not designed for modifications. New structures are placed due to cost and time limitations, which is not a sustainable solution. New bridges require more material use, and reuse of the old products is not yet done on a high-quality level.

Modular systems are an upcoming solution to make structures adaptable. The design incorporates demountable connections, hence adjustments and replacements can be made. The first modular bridge in the Netherlands applies this principle (Rijkswaterstaat, 2019b). This bridge can be reused in multiple locations and functions. The functional lifespan is no longer governing, the lifespan of a bridge is extended.

1.2. Research objectives

This research aims to develop a sustainable alternative to concrete bridges in line with the circular economy principles. Two strategies are combined to obtain this objective: building with a renewable material and making the bridge modular. The first reduces the impact on the environment and depletion of finite resources. The latter enables reuse of the bridge by the ability to disassemble, adjust and move the system. Both timber and modular bridges exist, but a combination of the two is not researched yet. The objective is to reduce the impact on the environment further by combining these strategies.

1.3. Research questions

The main research question is defined as:

How can a timber bridge lead to a circular alternative for highway bridges when considering the preliminary design phase?

Three sub-questions support this research question:

1. Which principles should be applied to obtain a circular timber bridge in the preliminary design phase?
   (a) Which strategies can be applied to ensure the design is in line with the fundamentals of the circular economy?
   (b) Which aspects of timber structures are governing in the preliminary design?
2. What composition of a timber bridge results in a circular design?
   (a) How to translate the standardisation and boundary conditions of a bridge into a design for a modular system?
   (b) Which structural typology is best suited within the defined design space?
   (c) How is standardisation applied in the context of replacing existing highway bridges?
   (d) Which engineered timber products result in the most material-efficient structure?
   (e) Which connections should be demountable, and which connections are best suited for the bridge?
3. What is the environmental impact of the developed highway bridge?
   (a) Which factors influence the quantification, and how can an objective comparison be obtained?
   (b) What is the carbon footprint of the timber bridge compared to a concrete bridge?

2
Research approach

This chapter describes the approach and methodology for this research. The objectives and research questions were derived in the previous chapter. The scope limitations are described first, followed by a description of the methodology. Furthermore, the structure of the report is visualised, and connected to the research questions.

2.1. Scope limitations

Within the time frame of a Master's thesis, scope limitations are necessary. The first scope limitation is that only the superstructure is designed. The substructure is assumed to be in good condition, made out of concrete. Secondly, the context of this research is highway bridge replacement in the Netherlands. Therefore, the research applies European and Dutch regulations, and assumes timber to originate from Europe.

A global parametric model is developed, containing Finite Element Analyses. With this model, a design for the preliminary design phase is established. However, the connections are not modelled in detail, but equivalent values for stiffness and strength are implemented in the model. Moreover, a static design is made, excluding dynamic effects. Furthermore, long-term effects are not considered: fatigue and deterioration of timber. The influence of temperature, moisture, and protection are not in the scope of this research. However, design measures are taken to protect the timber members, reducing the chance of deterioration.

The environmental impact quantification is done with a simplified Life Cycle Assessment method. Existing data from producers and wood research institutes are used. The reliability of this data is often low, but the gathering of new data is out of the scope of this research. Therefore, a comparison between data is made, choosing the most reliable. Lastly, this research does not include a cost analysis, as costs vary over time, product and country. An in-depth analysis should be performed to make a good comparison of life cycle costs between timber bridges and alternatives, which is not possible in the timeframe of this research.

2.2. Methodology

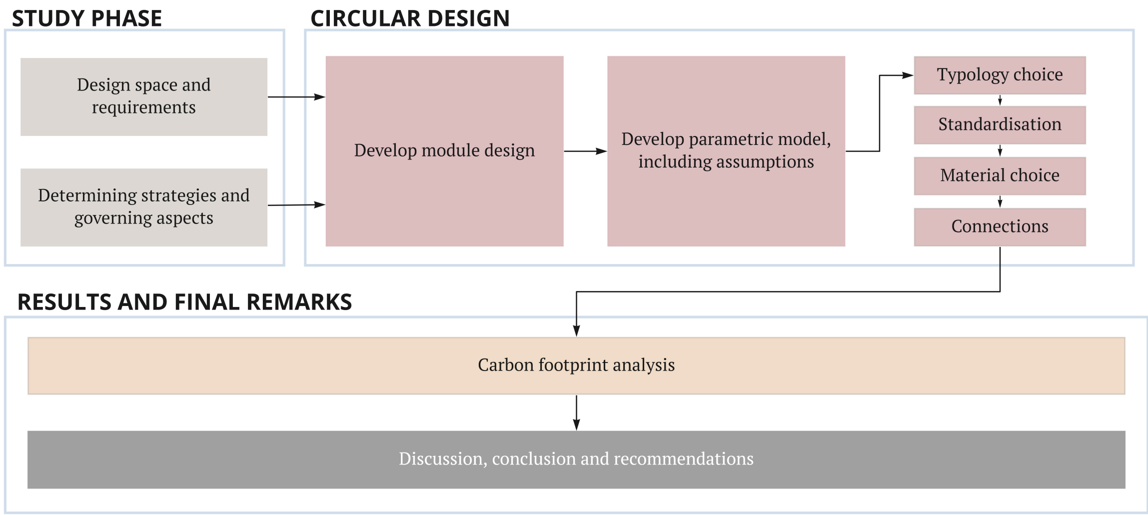


Figure 2.1: Diagram of methodology

Figure 2.1 presents an overview of the methodology of this research. The research is split into three parts: study phase, circular design, and results. First, a study is performed into the future replacement task for Dutch highway bridges of Rijkswaterstaat to define the application area. Second, literature research is performed into the circular economy, timber construction, and timber bridge design options. Design strategies result from this study phase.

Next, the basis for the modular system is derived, and translated into a parametric model. It is chosen to model the bridge design in a parametric manner to make it widely applicable and increase the ease of considering the modular timber bridge in an early design stage. This model is made in the Rhino/Grasshopper environment, allowing fast modelling on a global level. Within Grasshopper, many (open source) plug-ins exist, containing programmed components based on Python coding. One of these plug-ins is Karamba3D: a Finite Element Analysis program, enabling three-dimensional structural analysis within the Grasshopper environment. This plug-in is used for all structural analyses in this research. Karamba3D has a high calculation speed with accurate global results. However, Karamba3D is not suited for detailed FEM analyses, for example, on connection level. Therefore, the values of forces and deflections are derived globally, and connections are designed with hand calculations.

The parametric model is the basis for multiple variant studies for the bridge. The first variant study is on the typology, for which the best suited option is chosen using a trade-off matrix (TOM). When a decision on typology is made, the standardised system is defined. This standardised system is defined by the application area. Using the data from Rijkswaterstaat, an optimised set of modules is defined for the chosen application area. Next, a variant study is performed on timber products, considering their structural behaviour and environmental impact. Consequently, the connections are chosen, designed and verified. After these steps, an iteration is made into the parametric model, including all optimisations. A final design for the modular timber bridge is established.

To verify the design choices made and to support multi-criteria decision making, TOMs are applied. By using a TOM, performance indicators can be set, and a structured consideration can be made. Weight factors are given to the performance indicators, and a score is assigned for each design option. The scores, as presented in Table 2.1, are given to the performance indicators.

Table 2.1: Score factors of TOM

Score	Description	Value
+ +	Favourable	1
+	Neutral	0.67
-	Unfavourable	0.33
- -	Negative	0

When the final design is obtained, a carbon footprint study is performed. With a carbon footprint study, the environmental impact estimation is compared to a concrete bridge. The traditional and circular concrete bridges from Rijkswaterstaat are used for this comparative study. The data for the concrete bridges is obtained from NIBE Research bv (2019a). For the timber environmental data, an Environmental Product Declaration (EPD) is used. This EPD contains data from multiple manufacturers of laminated timber. This data is multiplied by the amount of material required for the bridges. For timber, a bridge with the same span and width is modelled and the required material is used for the comparison.

2.2.1. Assumptions during research

Figure 2.1 illustrates the design options which are investigated after the initial design phase. Therefore, assumptions must be made in the first design. An iterative process is established, resulting in a constantly changing design. Figure 2.2 presents the assumptions made in different stages of the research.

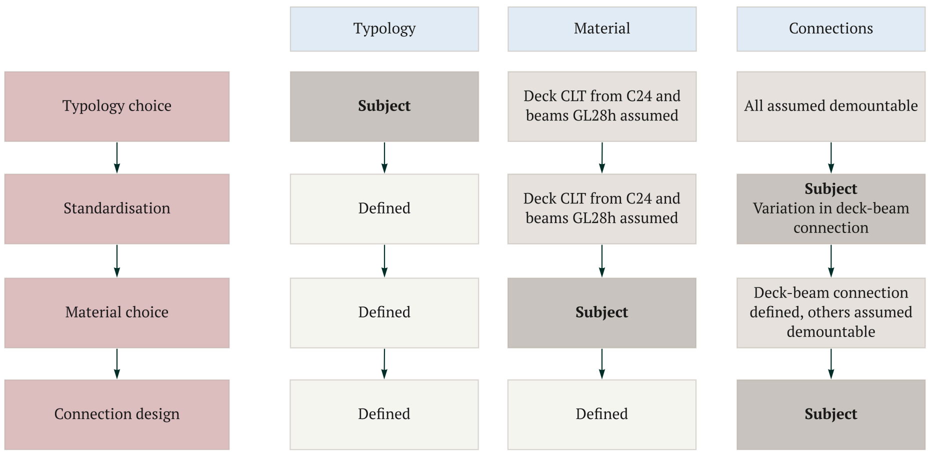


Figure 2.2: Assumptions throughout the research process

The typology is the first study, thus no assumptions are required on typology (Section 7.2). Cross Laminated Timber (CLT) from C24 and Glued Laminated Timber (GL28h) are assumed as materials, based on the reference design of Behrens and Benner (2015). Section 7.4 studies the timber products with the most material-efficient behaviour. The connections are all assumed to be demountable since a modular structure is desired. Section 7.3.2 assess the feasibility of making all connections demountable.

2.3. Structure

This section describes the structure of this thesis report, presented in Figure 2.3.

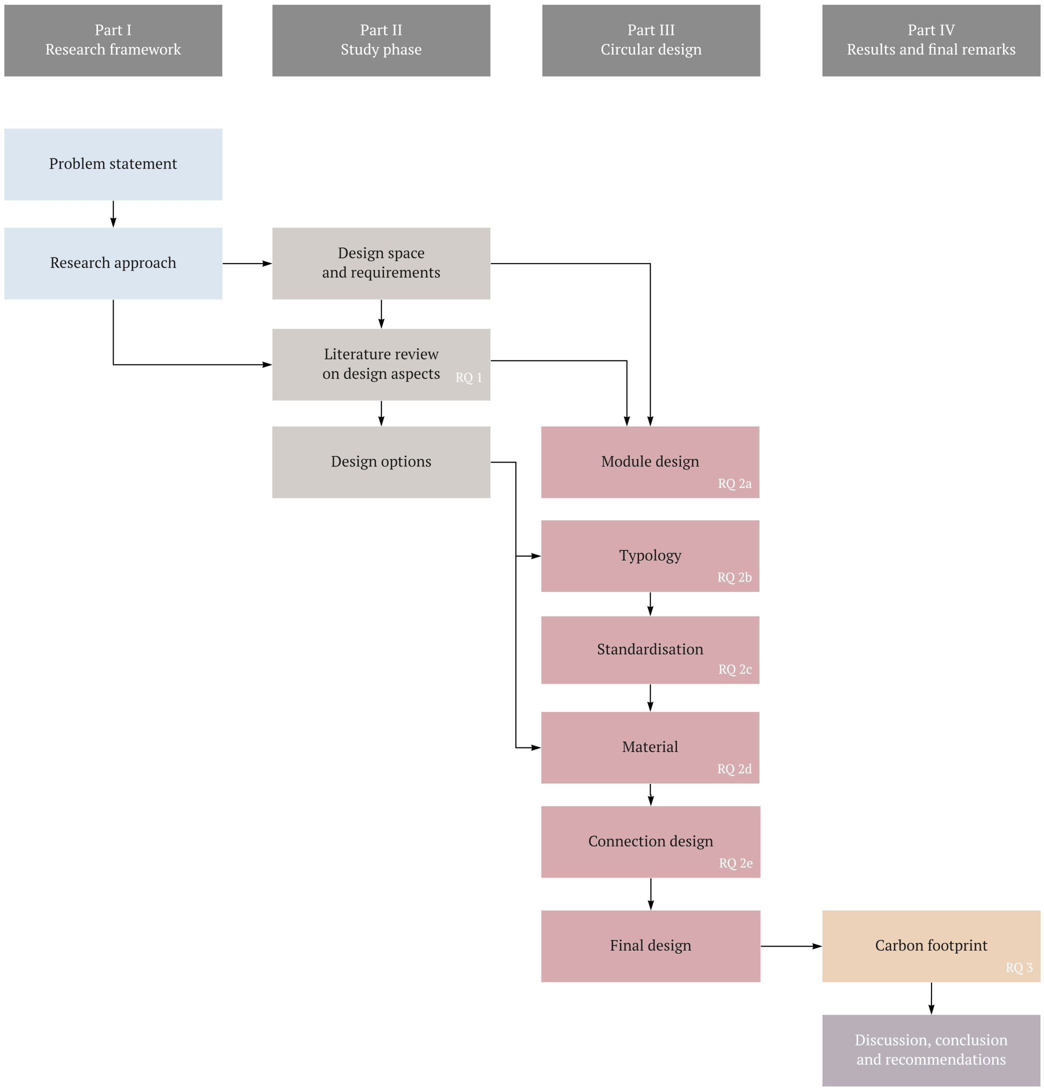


Figure 2.3: Layout of the report

Each part of the report focuses on a research question. Part II studies the design space, aspects and options for the aspects considered. In conclusion, the strategies and governing aspects are determined, answering research question 1.

Following, Part III describes the development of the circular design. Firstly, a module design is determined, together with the principles of the modular system. Secondly, the typology study is performed, resulting in the best-suited typology for the design space. Thirdly, standardisation is applied to the modular system while maintaining a material-efficient structure. Consequently, a material study is performed on the beams and deck, determining the best-suited materials. Finally, connections are designed, and the final design is presented. These aspects together provide an answer to research question 2.

Lastly, the impact of the new bridge system is quantified in Part IV, performing a carbon footprint. Thus, an answer to research question 3 is obtained. To finalise, the discussion, conclusion and recommendations are presented.

II

Study phase

3 Design space and requirements

This chapter starts with describing the context of the replacement task from Rijkswaterstaat, for which the timber bridge design is made. Subsequently, the design codes and boundary conditions are stated. Furthermore, an overview of material properties and load conditions is presented. Furthermore, the load combinations are derived, which are used for all structural analyses in this research.

3.1. Replacement task

Rijkswaterstaat owns most highway bridges in the Netherlands. This study uses a database (Rijkswaterstaat, 2021) with all bridges that Rijkswaterstaat owns, to analyse the existing bridges in the Netherlands. An overview of the most important findings is given in this section.

Most of the infrastructural works in the Netherlands were built in the second half of the twentieth century. Concrete and steel are used in most works, and many bridges do no longer comply with the structural or functional requirements. Consequently, Rijkswaterstaat is planning on repairing or replacing many infrastructural works in the coming decades. A modular timber alternative for the bridges can be part of the replacement task, with the goal to minimise the environmental impact.

The dataset contains both bridges in use or demolished, starting from construction year 1920. The average age for bridges in use is 37 years, where it is 46 years for demolished bridges. Hence, the average demolished age is not near the design service life of 100 years. Two possible reasons can be given: (1) less knowledge on structural behaviour and protection was available at the time of construction, and (2) the bridges no longer fulfil the functional requirements. The latter can be avoided by making the bridge adaptable, thus following the circular strategies as described in Section 4.1.

3.1.1. Design space

As the average demolishing age is 46, the decision is made to consider all bridges built between 1950 and 1980. These bridges are forty to seventy years old and have a high chance of being demolished in the coming years. In total, 2724 highway bridges in the dataset are in use. When one selects the bridges built between 1950 and 1980, 1389 highway bridges remain (Figure 3.1).

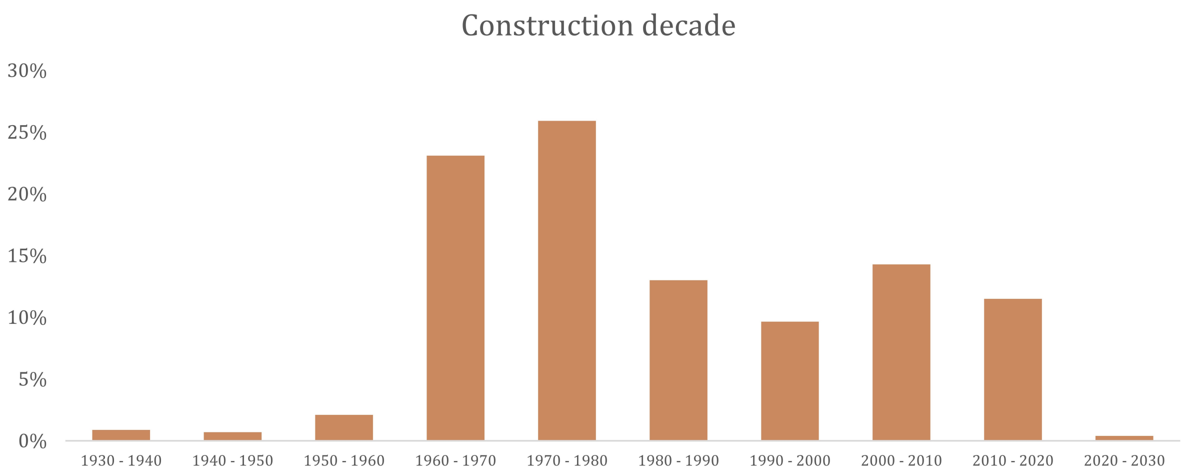


Figure 3.1: Construction decade of bridges in use

Not all bridges are suitable to be replaced by a timber alternative, thus further selection is required. The experience with timber bridges in the Netherlands is low, as it was with concrete in the 1950s. Furthermore, the objective is to make a demountable and adaptable design. This limit the free spans of a bridge, due to required joints in the free span. Furthermore, the study aims for a standardised system, thus the design space should not be too wide. A selection is performed on the following three properties:

1. Free span
2. Number of free spans
3. Skew

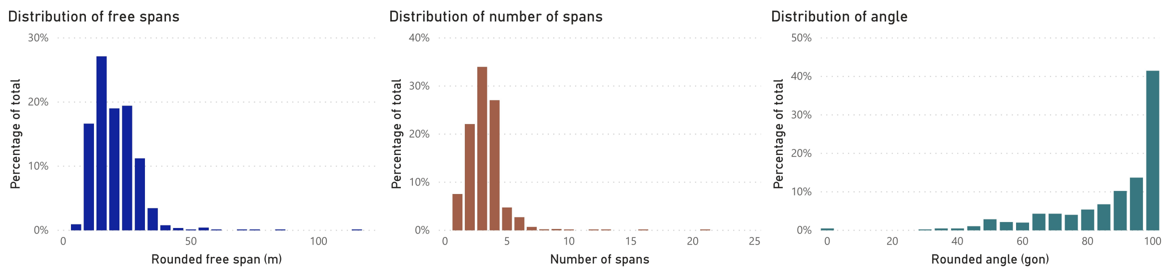


Figure 3.2: Occurrence of selection properties

First, the free span is selected. Figure 3.2 indicates that most free spans are below fifty metres. Fifty metres is an ambitious span for a timber bridge, as was stated in Section 5.1. However, the system's limits can be explored by including spans up to fifty metres and are therefore included. Second, the number of spans is selected: most bridges have one to five spans. No significant change in structural behaviour is observed for three or more spans. However, to narrow the scope, one to five spans are included in the design space. For the third selection property, the skew is analysed. It is important for the modular design, as this results in a significant change in dimensions and layout of the modules. The angles are expressed in gon, a unit that is often used in infrastructure design (100 gon=90 degrees). 40% of the bridges cross perpendicular. However, the aim is to include more than 40% of the bridges in the design space, thus bridges with a non-perpendicular skew are also included. The decision is made to consider all bridges with an angle between 60 and 100 gons (54 to 90 degrees). A reason for this is a balance between a wide variety of bridges and the demand to obtain an efficient modular system. Table 3.1 presents the reduction in bridges by this selection. As a result, 1205 bridges remain, which is 86.8% of the bridges in the replacement task.

Table 3.1: Impact of bridge selection

Property	Selection	Amount before	Amount after	Part remaining	Reduction
Free span	1-50 metre	1389	1376	99,1%	0.9 %
Nr of free spans	1-5	1376	1315	94.7%	4.4 %
Skew	60-100 gon	1315	1205	86.8%	8.4 %

3.2. Design requirements

Road layout

The standard layouts of the road below and over the bridges is based on the ROK: Richtlijn Ontwerp Kunstwerken (Rijkswaterstaat, 2017) and the ROA: Richtlijnen Ontwerp Autowegen (Rijkswaterstaat, 2019a).

- The minimum free height should be 4.6 metres: from the top of the deck on the bottom road to the lowest point of the superstructure.
- The ideal slope of the abutments is 3:2, but it can go up to a slope of 1:1. Most existing bridges in the Netherlands have abutments with a slope of 3:2. As the substructure is not a part of this research, no or minimal adjustments to the abutments are made when the superstructure is replaced by a timber one. However, the possibility for change in slope of the abutment can be helpful for adaptations to the bridge.
- The standard width of the lanes is 3.5 metres.
- Emergency lanes should be 3.5 metres wide as well.
- A free space of one metre should be present, both on the sides of the road and along the middle verge.
- On the edges of the roads, lines of 0.2 metre are required.
- The middle verge should be at least 2.5 metres wide.

The same dimensions apply for the road on top of the bridge, but an emergency lane is not always required. Furthermore, the width of the middle verge is more flexible.

3.2.1. Material properties

In the global design phase, material choices are assumed, based on the reference design of Behrens and Benner (2015): glued laminated beams (GL28h) are applied with a cross laminated timber (CLT) deck made from C24. According to EC1995-1-1, the calculation value of the strength for all timber elements should be taken as:

$$X_d = k_{mod} \frac{X_k}{\gamma_M} \cdot k_{sys} \tag{3.1}$$

where k_{mod} is a modification factor that considers the load duration and moisture content of the timber. γ_M is the partial factor defined for specific materials. In this design, the relevant values for γ_M are:

- Laminated wood - 1.25
- LVL - 1.2
- Connections - 1.3

The mechanical properties of glulam and CLT are presented in Table B.1 in Appendix B. For CLT systems, the additional factor k_{sys} can be included. The value of k_{sys} depends on the number of stressed boards, as displayed in Figure 3.3.

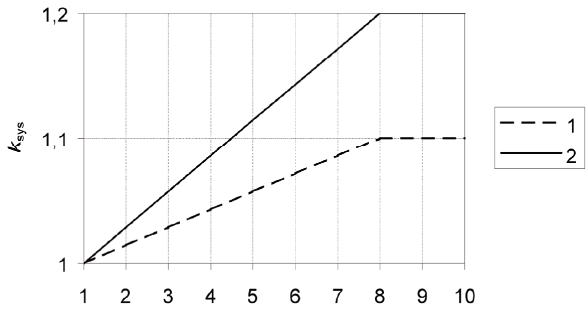


Figure 3.3: Values for k_{sys}

3.2.2. Loads

Traffic loads are considered according to NEN-EN 1991-2 (LM1). Wind, temperature and collision loads are not considered for the reduction of load combinations. The research of Arup stated that this was governing for the timber bridge (Arup; Heijmans, 2021). Next to traffic loads, self-weight and static loads are included. These loads are joined in the load combinations for Ultimate Limit State (ULS) and Serviceability Limit State (SLS).

Load Model 1 (LM1): effects of traffic

Traffic loads mainly depend on two factors: the width of the road and the number of vehicles per year. The width of the road influences the theoretical lanes that should be considered, taken from Figure 3.4. The assumption is made that all roads considered have a width larger than six metres. For this reason, w is always three metres and divided over the width of the road. The National Annex (NEN-EN 1991-2 NA) defines an additional factor for three or more lanes. This additional factor is not included to keep the load cases constant for all bridge sizes. Hence the results can be compared more easily.

Carriageway width w	Number of notional lanes	Width of a notional lane w_l	Width of the remaining area
$w < 5{,}4$ m	$n_1 = 1$	3 m	w - 3 m
$5{,}4 \text{ m} \leq w < 6$ m	$n_1 = 2$	$\frac{w}{2}$	0
$6 \text{ m} \leq w$	$n_1 = Int\left(\frac{w}{3}\right)$	3 m	$w - 3 \times n_1$
NOTE For example, for a carriageway width equal to 11m, $n_1 = Int\left(\frac{w}{3}\right) = 3$, and the width of the remaining area is 11 - 3×3 = 2m.			

Figure 3.4: Theoretical lanes, from NEN-EN 1991-2

LM1 is divided into double-axle concentrated loads (tandem system TS) and a uniformly distributed load. Each load has the magnitude of:

$$\alpha_Q \cdot Q_k \tag{3.2}$$

α_Q are adjustment factors, which are defined in the national annex. The factor depends on the number of heavy vehicles per year and the length of the superstructure. The values for α_Q are presented in Figure 3.5. For this design space, the value of α_{Q1}=1.0, as the maximum amount of heavy traffic is assumed. The values of Q_{ik} and q_{ik} and their locations are displayed in Figure 3.6.

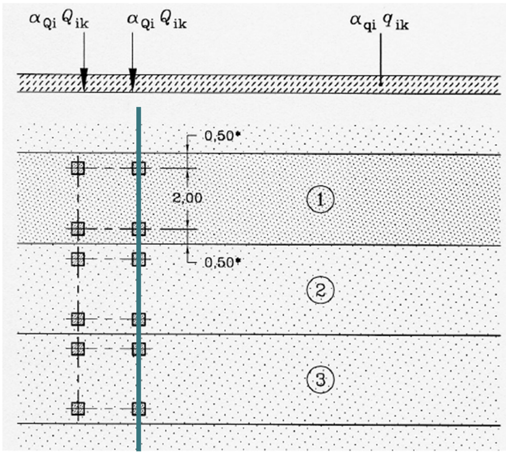
Figure 3.7 presents the forces acting on the section as indicated in Figure 3.6. The values presented are for one wheel: each surface in Figure 3.6 contains a wheel load.

Tabel NB.1 — Correctiefactoren α_{Q1}, α_{q1} en α_{qr}

Aantal vrachtwagens per jaar per rijstrook voor zwaar verkeer N_{obs} [a]	α_{Q1} en α_{q1} Lengte van de overspanning of invloedslengte (L)				α_{qr}
	20 m	50 m	100 m	≥ 200 m	
≥ 2 000 000	1,0	1,0	1,0	1,0	
200 000	0,97	0,97	0,95	0,95	0,90
20 000	0,95	0,94	0,89	0,88	0,80
2 000	0,91	0,91	0,82	0,81	0,70
200	0,88	0,87	0,75	0,74	0,60

[a] Tussengelegen waarden mogen worden geïnterpoleerd.

Figure 3.5: Value of α_Q (NEN-EN 1991-2 NB, 2019)



Key
(1) Lane Nr. 1 : Q_{1k} = 300 kN ; q_{1k} = 9 kN/m^2
(2) Lane Nr. 2 : Q_{2k} = 200 kN ; q_{2k} = 2,5 kN/m^2
(3) Lane Nr. 3 : Q_{3k} = 100 kN ; q_{3k} = 2,5 kN/m^2
* For w_l = 3,00 m

Figure 3.6: Load locations and values in LM1 (NEN-EN 1991-2, 2015)

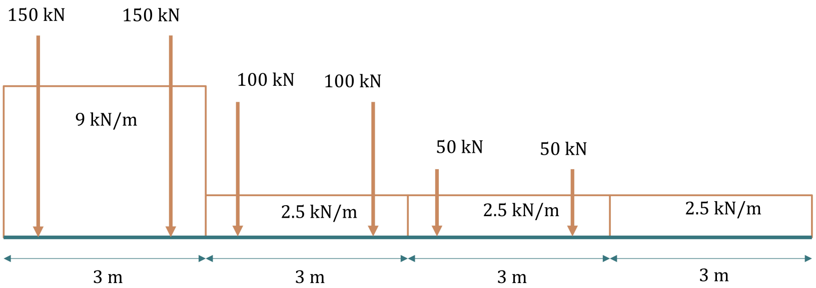


Figure 3.7: Loads acting on an axis of the tandem system in LM1. Concentrated loads are wheel loads. The load areas displayed in Figure 3.6, are wheels.

Static load

Additional static loads are considered due to the self-weight of the bridge, asphalt and guiding rails (Rijkswaterstaat, 2017). The self-weight of the structure follows from the material use. The prescribed load is divided into two parts: loading of borders and rails and loading over the whole area resembling the weight of the asphalt.

Asphalt requires a thickness of (140+a) mm. The value of a is: a=(L-30)/4 where L is the largest free span, and a should be between 0 and 30 mm. Within the defined design space, L will never be above fifty metres, but most of the time lower than thirty metres. For this reason, a=0 and the thickness of the asphalt is thus 140 mm. With the density of asphalt as 23 kN/m^3, this results in a load of $23 \cdot 0.14 = 3.2$ kN/m^2.

For the sides, a value of 6.16 kN/m^2 should be applied. In the load combinations, the static loads are combined with the self-weight of the structure as all are permanent loads.

3.2.3. Load combinations

Multiple loads can occur at the same time, hence load combinations must be made. The final deflection in time is defined by NEN-EN 1995-1-1 Chapter 2.2:

$$u_{fin} = u_{fin,G} + u_{fin,Q1} + \sum u_{fin,Qi} \tag{3.3}$$

$$u_{fin,G} = u_{inst,G} \cdot (1 + k_{def}) \tag{3.4}$$

$$u_{fin,Q1} = u_{inst,Q1} \cdot (1 + \psi_{2,1} \cdot k_{def}) \tag{3.5}$$

$$u_{fin,Qi} = u_{inst,Qi} \cdot (\psi_{0,i} + \psi_{2,i} \cdot k_{def}) \tag{3.6}$$

According to the National Annex of NEN-EN 1990, $\psi_0 = 0.8$ and $\psi_2 = 0$ for traffic loads. k_{def} is 0.8 for glulam in climate class 2. However, as traffic load is the only variable loading considered, this is Q1, and no other Q occurs.

$$u_{fin} = u_{inst,G} \cdot (1.8) + u_{inst,Q1} \cdot (1 + 0 \cdot 0.8) + u_{inst,Qi} \cdot (\psi_{0,i} + 0 \cdot 0.8) \tag{3.7}$$

$$u_{fin} = 1.8 \cdot u_{inst,G} + u_{inst,Q1}$$

To comply with the SLS requirements, only traffic load has to be considered. NEN-EN 1995-1-1 Chapter 7 defines the following value for the final deflection of the structure:

$$u_{net,fin} = u_{inst} + u_{creep} - u_{camber} \tag{3.8}$$

It is assumed that the camber compensates for creep and deflections due to static loading. Section 7.6 elaborates on camber. The requirement for the deflection when camber is applied is:

$$u_{max} = u_{traffic} \leq \frac{l}{400} \tag{3.9}$$

as provided by the National Annex of NEN-EN 1995-2. In ULS, three load combinations must be checked with different governing loads. The scheme of Table 3.2 is applied for this, considering consequence class 3 for traffic bridges. The values are retrieved from the National Annex of NEN-EN 1990 (Appendix A2).

$$\gamma_G \cdot G_k + \gamma_{Q,1} \cdot Q_{1,k} + \sum \gamma_{Q,i} \cdot \psi_{0,i} \cdot Q_{i,k} \tag{3.10}$$

Table 3.2: Load combinations ULS, considering consequence class 3 for traffic bridges.

LC		$\text{factor}_{\text{static}}$	$\text{factor}_{\text{traf}}$	k_{mod}
1a	Static load governing, traffic = Q1	1.5	1.32	0.9
1b	Static load governing	1.5	0	0.6
2b	Traffic governing, static load = Q1	1.35	1.65	0.9

4 Literature review

This chapter presents the literature review which defines governing aspects and strategies relating to research question 1. Section 4.1 defines the principles for a circular structure. Furthermore, literature on the quantification of environmental impact is described. Section 4.2 covers two main aspects of timber structures: durability and fatigue. It is determined whether these aspects are governing in the preliminary design stage. Section 4.3 summarizes the literature in design strategies for this circular design. Section 4.4 provides an answer to research question 1.

4.1. Circular economy

The construction industry is dominated by linear processes. This causes adverse effects: large amounts of waste, depletion of finite resources and high emissions, which have a negative impact on the environment. Therefore, change is required to mitigate the harmful effects of the construction sector. In contrast to the linear economy, the circular economy prevents products ending as waste. Figure 4.1 presents the different processes in the linear and circular economy (CE).

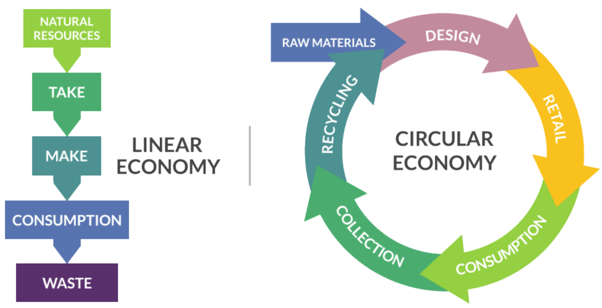


Figure 4.1: Linear vs circular economy from End of waste foundation (2021)

4.1.1. Definition and principles

This section presents multiple definitions of the CE, combined with principles to put it into practice. Many initiatives arose in the past decade to catalyse the transfer of a linear towards a circular economy. The stricter climate agreement of Paris (United Nations, 2016) increased the attention for a CE. Consequently, the institutes focussing on the transition also gained more attention. The definition and principles of the CE are defined by leading institutes. The Ellen MacArthur Foundation is one of these institutes, currently leading in the CE. The Ellen MacArthur Foundation uses the following definition for the CE:

> *"A circular economy is based on the principles of designing out waste and pollution, keeping products and materials in use, and regenerating natural systems"* (Ellen MacArthur Foundation, 2021b).

This is a broad but covering definition of the CE, to be applied to all industries. Ellen MacArthur Foundation (2021a) defines three circular principles: (1) design out waste and pollution, (2) keep products and materials in use, and (3) regenerate natural systems. Within this research, the first and second circular principles are applied. No focus is placed on regenerating natural systems.

Platform CB'23 (Circulair Bouwen 2023) is the leading institute on the CE in the Dutch construction industry. Rijkswaterstaat is one of the initiators of CB'23, joining Dutch academic and commercial organisations with the intention to obtain a framework for the CE by 2023. The following definition of the circular construction industry is used in CB'23:

> *"Building circular means the development, use and re-use of buildings, areas and infrastructure, without unnecessarily depleting natural resources, polluting the environment and damaging ecosystems. Building in a way that is economically and ecologically responsible and contributes to the well-being of people and animals. Here and there, now and later."* (Platform CB'23, 2019)

Timber is a renewable material, and with sustainable forest management, resources are not depleted. Consequently, the aspect of building "without unnecessarily depleting natural resources, polluting the environment and damaging ecosystems" is applied. The other principles are similar to the Ellen MacArthur Foundation.

A third active party in the transition is the PBL Netherlands Environmental Assessment Agency. Their policy report on the CE (Potting et al., 2017) implements circular strategies, called 10R strategies. These were first defined by Cramer (2014) and are presented in Figure 4.2.

Smarter product use and manufacture	R0	Refuse	Make product redundant by abondoning its function or by offering the same function with a radically different product.
	R1	Rethink	Make product use more intensive (e.g. through sharing products or by putting multi-functional products on market)
	R2	Reduce	Increase efficiency in product manufacture or use by consuming fewer natural resources.
Extend lifespan of product and its parts	R3	Reuse	Re-use by another consumer of discarded product which is still in good condition and fulfils its original function.
	R4	Repair	Repair and maintenance of defective product so it can be used with its original function
	R5	Refurbish	Restore an old product and bring it up to date.
	R6	Remanufacture	Use parts of discarded product in a new product with the same function.
	R7	Repurpose	Use discarded products or its parts in a new product with a different function.
Useful applications of materials	R8	Recycle	Process materials to obtain the same (high grade) or lower (low grade) quality.
	R9	Recover	Incineration of material with energy recovery.

Figure 4.2: 10R principles, derived from Cramer (2014)

The study focuses on design with strategy R2: Reduce. Reduction is applied in three ways. Firstly, the study focuses on timber as the primary construction material, which is a renewable resource. Consequently, one reduces the use of primary abiotic materials and the carbon footprint of the bridge. Secondly, the design applies the circular strategy Design for Material Efficiency (DfME): optimisation is performed to reduce material use (Chapter 7). Lastly, one implements the Design for Adaptability (DfA) strategy. Adapting the bridges according to the needs at that time extends the lifespan. Hence, lower demand arises for total replacement of bridges and thus the material use is reduced.

Almost all R-strategies can be applied to a timber bridge. However, the scope of this research is limited to the design phase. Nevertheless, Design for Disassembly (DfD) and DfA enable strategy R3 until R7. Applying these two circular strategies enables lifespan extension as the structure is more flexible and can be used in multiple functions.

4.1.2. Environmental impact

The objective of this research is to develop an alternative for concrete highway bridges with reduced environmental impact. According to European standards, the environmental impact is quantified by a Life Cycle Assessment (LCA). One requires extensive knowledge on the production process, energy use, origin of the materials and emissions during all stages to perform a detailed LCA. However, this study uses existing data from Environmental Product Declarations (EPDs). EPDs present an LCA for a standard unit of a specific product from manufacturers.

An LCA considers several stages through the lifespan of a product. Figure 4.3 shows that stage D represents potential benefits and loads, which are out of the system boundaries. Therefore, stage D is not included in an international context.

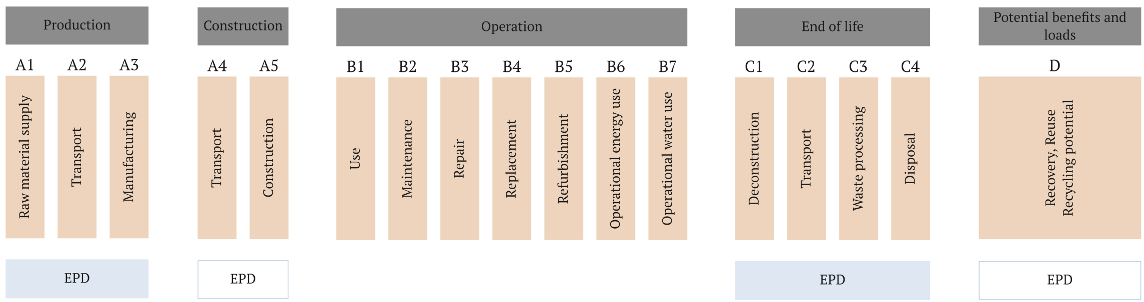


Figure 4.3: Stages in an LCA, adapted from Kuijpers (2021)

Carbon cycle

Stage A1-A3 covers the production of the material, including the impact of the raw material. Carbon is captured during the growth of wood, which results in the temporary storage of carbon in wood products (Vogtländer et al., 2014). Consequently, the carbon concentration is reduced in the atmosphere and thus the global warming effect is reduced (Van Wijnen, 2020). However, the carbon is assumed to re-enter the atmosphere later in the lifespan of the product (C3), as incineration of wood is presumed. Since 2021, the carbon capture (A1) and release (C3) should be included in the EPDs (Keijzer et al., 2021). This study applies this method. Next to carbon capture in the structure, more environmental benefits of building with timber can be established, as stated in the following paragraphs.

Vogtländer et al. (2014) state that "Extra demand of boreal and temperate softwood from Europe and North America leads to a better forest management and an increase in forest area therefore more sequestered carbon". This statement is based on converting unmanaged forests into sustainably managed forests. Together with increased demand for timber products for the construction industry, this development is accelerated. Consequently, carbon storage increased in European and North American forests since 1990.

Kurz and Apps (1999) and Perez-Garcia et al. (2005) state that tree growth occurs rapidly between the age of twenty and sixty. With sustainable forest management, the age of the trees is monitored. Above a certain age, trees are felled. Consequently, new trees can be planted and a more constant and rapid growth is ensured. Perez-Garcia et al. (2005) states the total carbon storage increases most with a rotation time of 45 years. An increase of 50% is obtained in a period of 165 years, including carbon storage in construction. Furthermore, the statement considers the replacement of concrete structures by timber structures, thus avoiding carbon emissions of concrete.

In conclusion, the carbon cycle of timber structures can result in substantial environmental benefits. These benefits are mainly caused by the delay of carbon emissions, and the stimulation of more carbon capture by planting new trees. Subsequently, the environmental goals on carbon can be obtained, reducing global warming.

Impact of circular strategies

The strategies of the CE, as defined in Section 4.1.1, also influence the environmental impact. Reduction of material (and thus environmental impact) can be realised by DfME. Furthermore, the lifespan of a structure can be extended by applying DfA and DfD. Kuijpers (2021) applied these circular strategies in LCAs for buildings. The context and expected lifespans differ, but they similarly impact the LCA outcome for bridges.

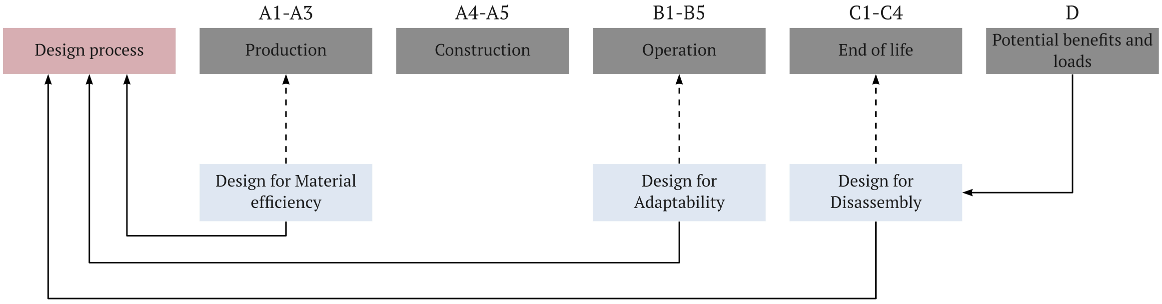


Figure 4.4: Relation between the LCA stages and the circular design strategies which are included in this research, adapted from Kuijpers (2021)

Figure 4.4 presents the influence of the circular strategies on the LCA modules. DfME relates to stages A1-A3: production. DfA links to B1-B5 (operation) in the context of building design. EPDs do not include stages B1-B5, due to the wide range of applications. However, DfA extends the lifespan, and thus the environmental impact of the structure can be spread out over a longer timespan. DfD is linked to stages C1-C4, which is included in the EPDs. By applying DfD, reuse is enabled. This extends the lifespan and changes the occurrence of end of life scenarios.

For the current (linear) economy, Stichting Bouwkwaliteit (2014) defined the end of life scenarios for wooden constructions in the 'GWW sector' (civil engineering works). It is stated that 90% of the timber used in civil engineering works is incinerated, thus burned to generate energy. The other 10% end up as landfills. EPDs also include the recycling scenario, which does not occur in the Netherlands, according to Stichting Bouwkwaliteit (2014). However, the circular economy ensures that reuse and recycling occur more often. Accordingly, incineration and landfill occur less often. This is beneficial for the emissions and material use, which can both be spread over a longer period of time.

Quantification

The environmental impact is assessed multiple categories, each expressed in the equivalent of a harmful emission. In EPDs, this is done in six impact categories, as displayed in Figure 4.5.

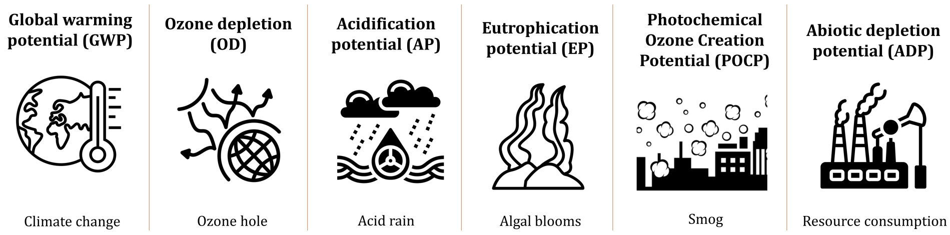


Figure 4.5: Environmental impact categories. Own figure, icons retrieved from www.nounproject.com

The Environmental Cost Indicator (ECI) is a Dutch method to quantify the environmental impact in one number. Each equivalent kilogram of a harmful emission has a shadow price assigned. This shadow price represents the costs to compensate for this emission. Therefore, one considers the ECI as an approximation in costs of the negative impact of a structure. The shadow costs, as presented in Table 4.1, originate from Stichting Bouwkwaliteit (2014). In the EPDs, the impact for ADP fossil is given in megajoule (MJ), not in equivalent kilograms antimony (Sb). However, this value can be converted by $4.81 \cdot 10^{-4}$.

Table 4.1: Shadow costs for each category (Stichting Bouwkwaliteit, 2014)

	Unit	Costs
GWP	kg CO_2 eq.	€0.05
ODP	kg CFC-11 eq.	€30
AP	kg SO_2 eq.	€4
EP	kg PO_4 eq.	€9
POCP	kg Ethene eq.	€2
ADP elements	kg Sb eq.	€0.16
ADP fossil	kg Sb eq.	€0.16

Section 5.2 compares the environmental impact of several timber products. Finally, Chapter 8 quantifies the environmental performance, comparing the timber and concrete circular bridges.

4.2. Timber structures

This section describes two design aspects related to timber bridges: durability (Section 4.2.1) and fatigue (Section 4.2.2). Durability is essential for all structures but deserves additional attention for timber. The climate (temperature and moisture level) is of high influence on the durability of timber. Furthermore, fatigue is researched as it highly depends on the material properties. Both sections assess to which extend the aspects should be considered in the preliminary design stage.

4.2.1. Durability

Two main factors influence the durability of timber: fungi growth and the microstructure of the timber. Timber is susceptible to fungal growth, decreasing the strength and stiffness of the structural members over time. Furthermore, the microstructure of the timber determines the durability of the construction. This section describes fungal growth and measures against it first, followed by the durability of the timber on micro scale.

Fungi cause the deterioration of the primary particles of wood, which are hemicellulose, cellulose and lignin (Simon and Koch, 2016). Fungal growth can occur at a moisture content above 20% (mass percentage). However, prEN 1995-2 states that for protected structures, a moisture content of 20% is allowed for four months per year, and exceeding 28% should not occur.

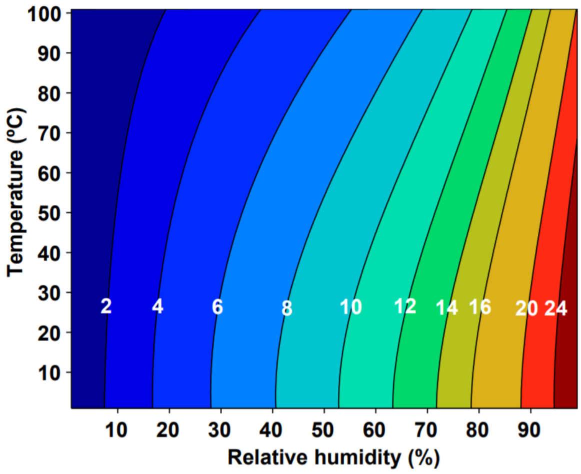


Figure 4.6: Equilibrium moisture content wood compared to the humidity of the air (Glass and Zelinka, 1999)

In the Netherlands, the relative humidity is 95% in winter and 50% in summer. The average humidity was 80% between 2016 and 2019 (Statista, 2020), and the average temperature in the Netherlands was

11 °C in 2020 (Rijksoverheid). Figure 4.6 depicts that with a humidity of 80% and a temperature of 10 °C, the moisture content in the wood is 16%. With humidity of 95% in winter, the moisture content is 22%. This level exceeds the boundary of 20%, but not over four months a year. Next to the moisture content, oxygen availability and temperature above 0 °C must be present for fungi to grow (Mahnert and Hundhausen, 2018). These circumstances are generally satisfied for bridges in the Netherlands.

However, one must still take measures to prevent fungal growth. Protection of the structural members can be ensured by the geometry of the bridge. Furthermore, the choice of wood species and its treatment influence the durability of the timber.

Physical protection is often mentioned in literature on outdoor timber structures. The previous paragraphs indicated that the humidity levels do not exceed the boundary values. However, this is on the condition that the timber is protected. Full protection against water and moisture is hard to obtain. However, a waterproofing membrane can protect the deck. For the beams and girders, design measures in geometry can be taken to keep them dry. Some often applied solutions are (Simon and Koch, 2016):

- Overhanging parts on top of the construction. An angle of 30 degrees must be maintained as water does not fall vertically.
- Cantilevering deck (which is not made of timber or can have a shorter lifespan) in combination with protected edges.
- Open lamellas that cover the structure, inspection and maintenance are still possible.
- Closed lamellas that cover the structure, which should be taken away for inspection or maintenance.

Simon et al. (2019) state that protected structures in Germany maintained a moisture level below 20% during the year. Slight exceeding of the maximum moisture level was indicated during winter, but was shorter than four months, which is allowed. The unprotected bridge showed higher moisture levels in the timber. Bridges close to the Dutch border were included in the study, thus the statements can be applied to Dutch bridge design. It is concluded that physical protection reduces the moisture level in timber and is therefore applied as a strategy in this research.

4.2.2. Fatigue

> "Fatigue is the process of progressive damaging of a material subjected to repeated loading, resulting eventually in failure at stress levels smaller than the corresponding quasi-static strength."
> (Pousette et al., 2017)

Fatigue behaviour highly depends on the material. The fatigue strength of structures is often determined by the $S - \log_{10} N$ curves. In these curves, S are stress ranges, and N is the number of loading cycles until failure. Smith et al. (2003) published an overview of the fatigue development of timber, aluminium, concrete and steel. Figure 4.7 illustrates that wood has a relatively long fatigue life compared to the other materials; the development of fatigue is gradual, and the strength remains relatively high.

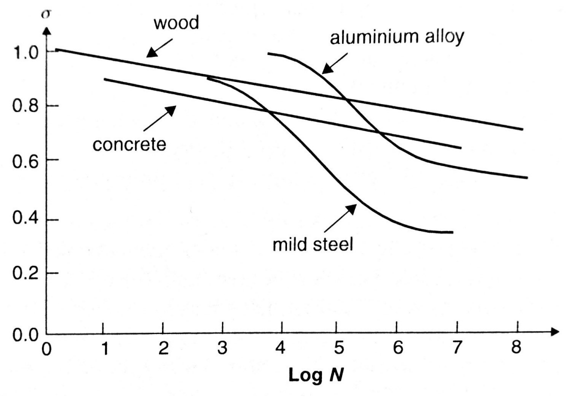


Figure 4.7: Fatigue development of several materials (Smith et al., 2003)

Connections in timber bridges are often made with steel. Limited studies are performed on fatigue of mechanical fasteners in timber structures. However, Malo et al. (2006) performed a study on the slotted-in steel plate connection. For 10^7 cycles, a strength loss of 50% is observed with R=0.1. For Dutch highway structures, 10^7 cycles are generally considered. However, the study does not observe a limit below which no fatigue strength losses occur. Therefore, the strength can reduce further with more load cycles.

Figure 4.7 illustrates that the development of fatigue in timber is gradual compared to steel. To accurately assess fatigue in bridge design, dynamic behaviour must be studied. Furthermore, many factors influence the fatigue behaviour of timber bridges, among which moisture content is essential (Malo et al., 2006). Therefore, a separate study on fatigue in timber bridge design is needed, but out of the scope of this research. The most crucial connection in the bridge, where the main girders between modules meet, is overdimensioned to avoid fatigue failure.

4.3. Design strategies

This section elaborates on the design strategies, following from the literature study. Circular design principles are defined, together with design aspects for timber constructions.

The previous sections provided literature on circular and timber structures. From this literature, the following design strategies are stated: efficient (DfME), protected (durable), adaptable (DfA) and demountable (DfD). This section elaborates on the design strategies, leading to design actions. These design strategies and actions are guiding through the circular design process.

Efficient

The efficient strategy is focused on the circular strategy DfME. Within this research, efficiency is measured by two properties: mass, and construction height of the structure.

Mass relates directly to the amount of material used. Therefore, the strategy DfME focuses on reducing mass. Furthermore, less material use requires less transportation and production. Consequently, the costs and environmental impact are lowered.

One should consider the construction height with more nuance. A structure with a large height is often efficient as the deflection is related to the height to the power three. However, a high construction height results in difficulties in attaching to the existing infrastructure. From experience of engineers at Arup, it is stated that attaching to the existing infrastructure is an important aspect in the replacement task. With an increased structural height, additional costs and materials are induced. The driveways towards the bridge need to be heightened, resulting in a wider ramp. In conclusion, a large construction height results in a material-efficient superstructure, but in difficulties attaching to the infrastructure. Therefore, this study considers mass as indicator for material-efficiency of the superstructure itself, and construction height for efficient fitting in the surrounding infrastructure.

Protected

As explained in Section 4.2.1, the protection of timber members is essential to enlarge the lifespan of a structure. The humidity in the air in the Netherlands during winter is reaching the border for maximum humidity levels. Therefore, measures are required to ensure that the timber elements do not become too humid. These measures are taken in two ways:

- Protect the structure from rain and other weather influences;
- Make sure that the structure can dry.

A roof can fully protect the structure from weather influences. However, this requires much material and often does not fit the dense infrastructure system. Nevertheless, the deck can be designed to act as a shield for the girders. A watertight membrane is applied between the asphalt and deck, preventing water from entering the deck. This protection increases by applying a cantilever (Figure 4.8) and edge details for additional coverage.

For construction parts above the deck, such as a truss, the protection against rain is more difficult without a roof. The only way to fully protect it against rain is by applying boards or slats to keep the rain away. Adjustable slats still allow for maintenance and monitoring of the construction.

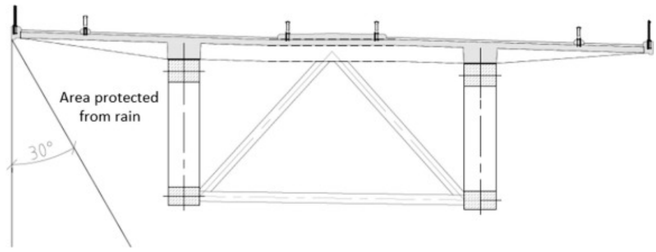


Figure 4.8: Physical protection against weather influences (Simon and Koch, 2016)

Figure 4.9: Adjustable slats can be applied on a truss (Simon and Koch, 2016)

Enabling the structure to dry is done by the following measures (Pousette et al., 2017):

- Avoid still-standing water in voids;
- Reduce contact surface between timber beams, especially avoid horizontal contact surfaces;
- Allow for ventilation.

Flexible

The service life of a structure can be extended by enabling adaptation (DfA). DfA results in a flexible construction over time, which can adapt to the current needs (Coenen, 2019). NEN-ISO 20887 2020 defines three levels of DfA: versatility, convertibility and expandability. The first focuses on minor function changes, therefore, the latter two are relevant for bridges.

Converting

Converting is about changes to the bridge in terms of function and loading. Constant strength and stiffness should be present over the area of the bridge to accompany for change in function. Furthermore, the modules should be over-dimensioned to account for changes in loading. This often results in more material use, which is not desired in the short term. However, when reuse can be applied, and the functional lifespan can be extended. Consequently, the structure is more material-efficient in relation to its lifespan. Furthermore, one can strengthen the deck of the modules to increase the load-bearing capacity. An additional layer of deck panels can be added on top of the existing deck for strengthening.

Expanding

Expansion requires more significant changes. A standardised set of modules is derived in Section 7.3.1. All modules are dimensioned to fit in the standardised system, but not all configurations are utilised to the limits. Therefore, expansion is possible for that part of the configurations. Moreover, strengthening can be performed similar to the converting strategy.

Demountable

The principles of adaptable and reusable structures are based on the fact that the structure is demountable. Consequently, the connections should be demountable. Literature on (demountable) connections is described in Section 5.3 and implemented into the design in Section 7.6.

To make the demountability of the structure feasible, the amount of labour required should be minimised. Therefore, the connections should be made simple and easy to mount and demount. The latter is influenced by the design of the connection itself and the number of structural joints.

4.4. Conclusions

This chapter focused on research question 1:

Which principles should be applied to obtain a circular timber bridge in the preliminary design phase?

- Which strategies can be applied to ensure the design is in line with the fundamentals of the circular economy?
- Which aspects of timber structures are governing in the preliminary design?

Figure 4.10 summarises the answer to the first sub-question.

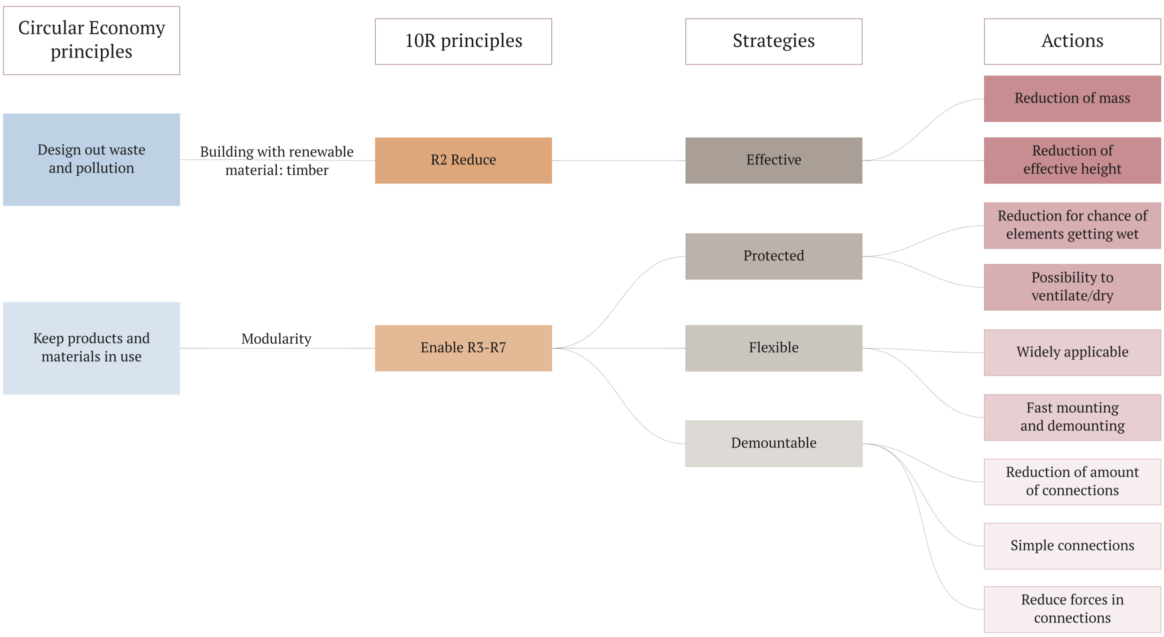


Figure 4.10: Design principles, strategies and actions

The second sub-question is answered by literature on timber structures. Two governing aspects in each early design stage of a bridge are typology and material choice. Furthermore, connections are designed as they are crucial for a demountable system. It is concluded that fatigue is not considered in the preliminary design stage. Lastly, the protection of the structure requires additional attention, as deterioration of timber members is highly influenced by weather circumstances. When the structure can be protected against weather influences, the lifespan can be extended.

5

Design options

This chapter explores the design options for typologies, materials and connections. Firstly, typology options for the set design space are defined, related to research question 2b. Secondly, timber products are explored, related to research question 2d. Information is provided on structural behaviour, production process, environmental impact and feasibility in timber bridges. Finally, connections that can be used in the timber bridge design are explored. For each category of connections, multiple options are provided and scored, providing a basis for research question 2e.

5.1. Typologies

The first variant study is the typology study. Section 3.1 defines the design space for the modular system: up to a free span of fifty metres. This design space is used to select suitable typologies. Pousette (2016) studied typical timber bridge typologies, called Swedish typologies. The infrastructure network in Sweden is different from the Netherlands: a denser network is observed in the Netherlands. Due to the denser network, some typologies take too much space to fit in the existing infrastructure. Therefore, not all bridge typologies can be implemented in the Dutch highway bridges. Figure 5.1 presents the bridge typologies including their typical span. Given that this figure is based on pedestrian bridges, one should use the typical spans to estimate the span obtained in highway bridge design.

Two main reasons for suitable typologies in this research are flexibility for expansion and appropriate typical span. When expansion can not be applied to a typology, it does not fit the strategies for adaptation. Furthermore, the suitable typical span should fit in the design space. Typologies for large bridges are not material-efficient for small bridges.

The first criterion excludes the arch and king post structure from the consideration because expansion in width and length is not possible for both typologies. The second criterion eliminates the suspension and cable-stayed bridge. These are generally suited for larger spans and are also not adaptable.

Five typologies are left, all suited for the flexibility aim and the design space. However, the SLTD and beam structure have a similar structural system. The same applies to the strut frame and V-support typologies. It is decided to consider the stress-laminated deck as decking material, but not as typology. Therefore, the beam structure is included in the study. According to Crocetti (2014), this typology has a lower typical span than the design space. However, it is observed that most existing highway bridges have a span below thirty metres. Therefore, the beam structure can still fulfil part of the replacement task of Rijkswaterstaat. The difference between the strut frame and V-supports is the location of additional supports. Generally, limited space is available next to the Dutch roads, hence the strut frame is implemented easier. Lastly, the truss structure is investigated, as it can be applied in modules and fits the proper spans for the scope. To conclude, three bridge types are considered: beams, truss and strut frame. For the global design, a simulation of these three typologies is made.

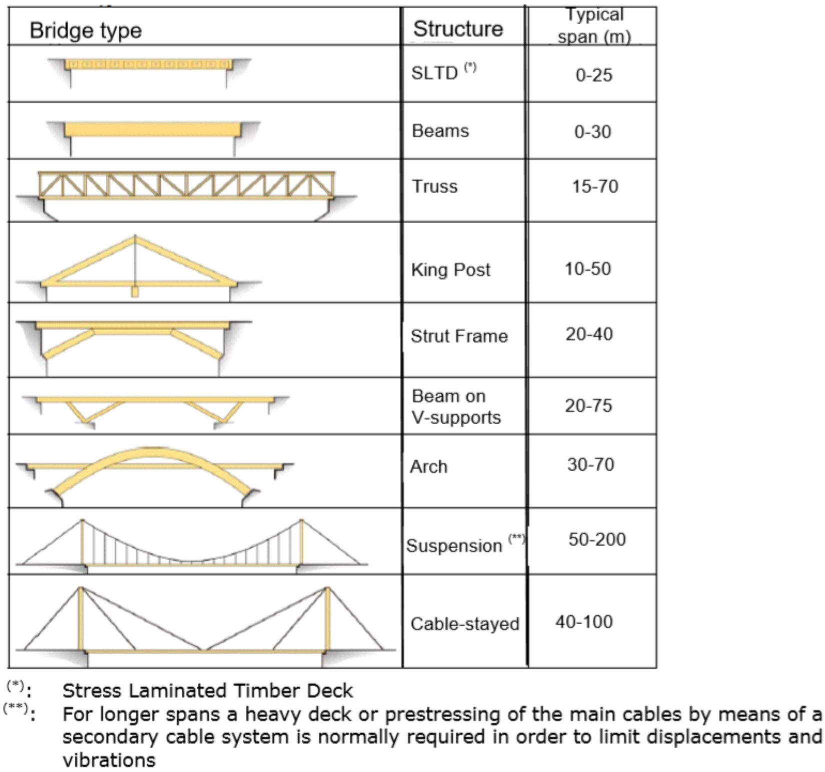


Figure 5.1: Typical types of timber bridges (Crocetti, 2014)

5.2. Engineered Timber Products

Timber is the definition for all wood used in constructions. Since 2000, the consumption of engineered timber products has increased significantly (Hildebrandt et al., 2017). Table 5.1 presents an overview of wood construction products and their components.

Table 5.1: Timber products, retrieved from Glos et al. (1995)

Wood product	Components
Logs	Stems
Sawn timber	Squared timber, planks, boards and battens
Glued laminated timber (glulam)	Boards
Laminated Veneer Lumber (LVL)	Veneers
Plywood	Veneers or sawn timber
Parallel strand lumber	Veneer strands
Particleboards	Particles (chips)
Fibreboards	Fibres

Two advantages of engineered timber products are the possibility of obtaining larger dimensions than sawn wood, and imperfections can be taken out of the wood. Large dimensions are required for the scope of this study, as free spans up to fifty metres are considered. Furthermore, by taking out the imperfections, the homogeneity of the material increases and thus the characteristic strength. Along with taking out imperfections, the amount of manufacturing increases over the height in Table 5.1. Fibreboards require the highest grade of manufacturing and thus also have the highest costs (Glos et al., 1995). As the aim is to apply the modular system on a large scale, costs and manufacturing should be reduced to make it economically feasible. Sawn timber, glulam (and CLT) and LVL are considered to obtain a balance between costs, quality, and design freedom.

A clear distinction between wood species exists: soft- and hardwood. The categorisation of wood as soft or hardwood is based on the structure on cell level. The distinction is not based on the density of the wood, although hardwood often has a higher density than softwood. Due to the high density, bonding between the timber and glue is more complicated, and hardwood is less suited to be applied in laminated products (Aicher and Stapf, 2014). Appendix A presents an environmental impact study on soft and hardwood, combined with literature on mechanical properties of soft and hardwood. The conclusion is drawn that the environmental impact of hardwood is significantly larger than softwood. Furthermore, Ramage et al. (2017) established no clear benefit in stiffness or strength for hardwood. In conclusion, softwood products are considered in the timber product analysis, as lamination can be applied easier, and its environmental impact is lower than hardwood.

Criteria

Criteria must be selected first to provide an assessment of the best-suited timber products. The amount of factors that influence the material choice is larger than the scope of this research. The ones included are described, followed by a list of not included aspects and their reasoning.

First, the strength and stiffness of the material are included in the comparison to assess the structural behaviour. The bridge is analysed with different beam and deck materials, and deflection and strength are compared. The strength and stiffness are essential aspects in the material choice, as less material is required when the material is stiffer and stronger, which is in line with the strategy Design for Material Efficiency. Second, the grade of prefabrication of the material is included. When more prefabrication is applied, less labour is required on site. Furthermore, the quality is more constant, as the circumstances in factories are controlled. However, less prefabrication can result in more flexibility and thus adaptability of the elements. Therefore, the grade of prefabrication should be considered for the deck material. For the beam elements, adaptability is less determined by material choice, but creating a curvature in the beams is important. When curvature can be obtained over the length of the beams, camber can be applied. By applying camber, the deflection due to self-weight can be compensated, improving the functionality and aesthetics. As mentioned before, the available dimensions of the products are essential. When dimensions of the elements can be large, fewer connections are required, and large modules can be used. As the last aspect, the environmental impact is included. EPDs are used to assess the environmental impact.

An aspect that is not included in the material study is the availability and origin of the timber products. The stock of these types of wood is hard to assess and fluctuates over time. The origin also influences the environmental impact by transportation emissions. In the data as used for the environmental impact, an average transportation distance is assumed. Furthermore, the actual costs of the materials are not included, as these fluctuate over time, and no economic analysis is performed in this research. However, rough indications are taken from experienced timber bridge designers at Arup, giving an indication for comparison. Lastly, the influence of moisture and temperature is not considered, as a detailed long-term analysis is required, which is out of scope.

Chapter 7.4 gives a three-dimensional structural analysis of the materials. This analysis is considered for the material choice, and therefore a conclusion is drawn there. The environmental impact and production processes are described in this literature study.

5.2.1. Beam materials

Sawn timber, glulam and LVL are considered for the beam material. The build-up of a beam of each of the materials is illustrated in Figure 5.2.

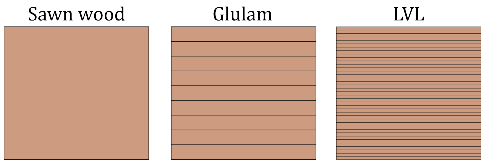


Figure 5.2: Build-up of a beam of sawn wood, glulam or LVL

Figures 5.3 until 5.5 present the production processes of sawn timber, glulam and LVL.

Figure 5.3 is based on the production process of Metsä Fibre (2021). The steps in the production process for sawn wood are performed for each type of engineered timber. It is the shortest production process, and therefore also the cheapest. A relatively low amount of labour, energy and additional materials are required to make this product.

The dimensions of sawn wood beams are limited, as it is limited to the dimensions of a tree. According to Hasslacher Norica Timber (2020) the maximum dimensions are thickness = 145 mm, width = 305 mm and length = 5.10 metre. Hence, a high number of connections in the span of the bridge are required. Consequently, the structure is less efficient, and the level of prefabrication decreases. Furthermore, the cross-sections must be larger than available for a structure in the scope of this research. In conclusion, sawn wood performs badly on available sizes.

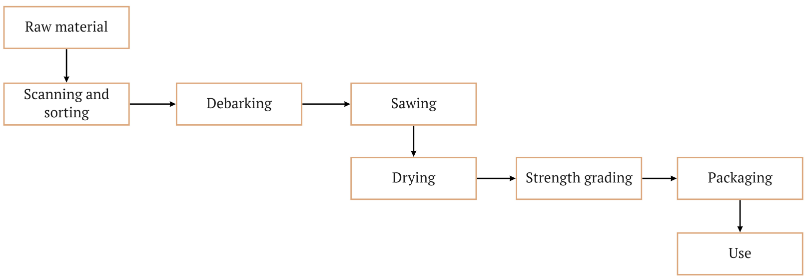


Figure 5.3: Production process sawn wood

Figure 5.4 is based on Ghiyasinasab et al. (2018). After the drying and strength grading, finger joints are made to obtain large dimensions and an effective glueing process. Next, the boards must be planed to apply the adhesive and start the glueing process. An optimal bonding is obtained by pressing the boards, and consequently, the boards start acting as one beam structure. However, this process requires a high amount of labour compared to sawn wood. Furthermore, by applying glue, the environmental impact and costs increase.

Glulam beams can be produced up to lengths of eighteen metres (Derix, 2019), which significantly reduces the number of connections compared to sawn wood. The cross-sections can be up to 30 cm wide and 100 cm high. Furthermore, block-glulam can be applied to increase the cross-sections even further (Aicher and Stapf, 2014). In block-glulam, multiple glulam beams are glued next to each other to realise a larger cross-section in the width direction. No size adjustments are defined for block-glulam by the Eurocode, thus it can be considered as a solid cross-section.

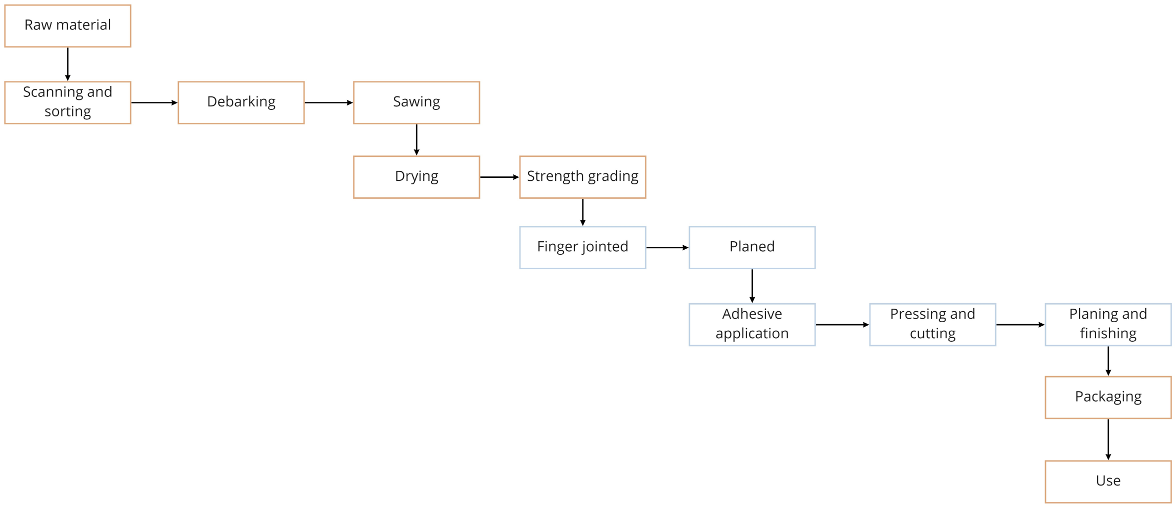


Figure 5.4: Production process glulam

Figure 5.5 is based on Metsä Wood (2021). Veneers are thin layers of wood, hence more adhesive is required to make cross sections out of it, compared to glulam. Furthermore, the layers of veneers are pressed twice, first cold and then hot, which also increases the duration of the production process. Along with labour and duration, costs increase as well. High accuracy and a controlled environment are required to glue all small particles together. Therefore the price of LVL is the largest compared to sawn wood and glulam.

A length of 24.5 metres is possible for LVL beams, according to Metsä Wood (2020). However, the standard cross-sections of LVL are limited: 2500 mm wide and 75 mm thick. Especially the thickness is an issue, as much thicker cross-sections are required. This issue can be solved by combining multiple LVL beams and joining them with glue or mechanical fasteners, similar to block-glulam.

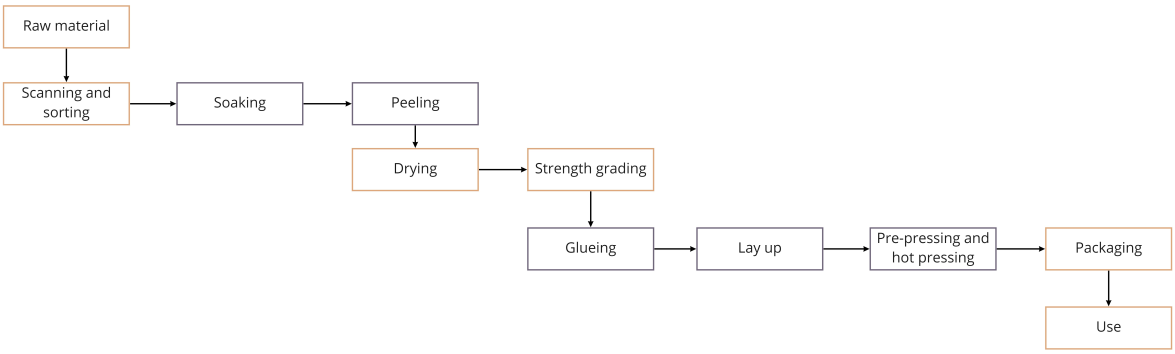


Figure 5.5: Production process laminated veneer lumber

5.2.2. Deck panels

Two requirements are set for the structural behaviour of deck panels: transfer high concentrated loads and provide stiffness. Three viable options arise for a timber deck: Cross Laminated Timber (CLT), Stress Laminated Timber (SLT) and Laminated Veneer Lumber (LVL).

CLT is build up from boards, similar to glulam. However, in the lamination process, the boards are not placed parallel but are orientated perpendicular, as illustrated in Figure 5.6. By placing the boards in two directions, stiffness and strength are obtained in two directions. However, the connections between the deck panels should be designed to transfer loads between deck panels. Otherwise, the deck panels still function as beams. When this is taken care of, high axle loads can be transferred in two directions, and thus the local deflection of the deck can be reduced.

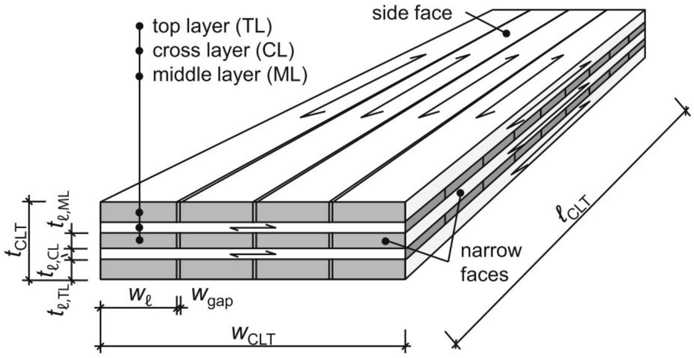


Figure 5.6: Build-up of a CLT panel (Brandner et al., 2016)

In CLT production, the arrangement of layers can be adjusted according to the required strength in both directions. One must always choose a dominant direction, as the cross-section of CLT should be symmetrical over the height. However, variations can be made in thickness, number of layers and arrangement. These variations give the designer much freedom in using CLT panels. Another advantage of CLT is the high grade of prefabrication (Brandner et al., 2016). The panels can be fully prefabricated, transported to the site and assembled in a fast manner.

The second option for the deck material is LVL, which production is similar to the beam material but orientated differently. LVL can be orientated as a plate with one dominant direction (LVL-S), or multiple plates can be cross-laminated (LVL-X) (Ardalany et al., 2011). LVL-X combines the aspects of LVL-S and cross-lamination of CLT, thus containing the same advantages and disadvantages.

The third option is SLT: timber boards are arranged in the same direction, after which tensioning is applied (Figure 5.7). The main advantage of SLT is that a continuous deck panel can be made in any dimension, which acts as a continuous slab (Ritter, 1990).

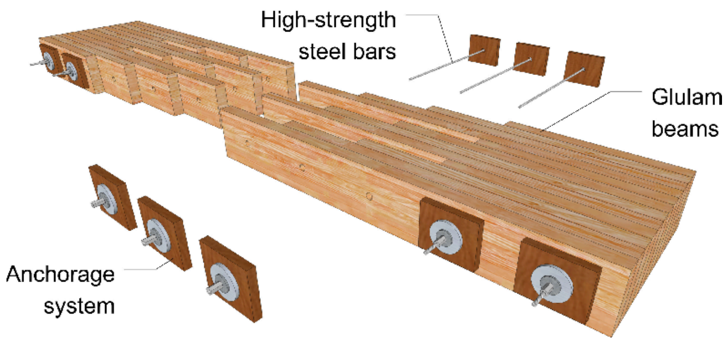


Figure 5.7: Build-up of SLT slab (Crocetti et al., 2016)

The stressing of the boards result in high vertical friction between the members. Consequently, high local forces can be distributed well (Massaro and Malo, 2014). Still, the strength and stiffness are dominantly present in one direction (Crocetti et al., 2016). Bending moments can only be taken in one direction, and limited shear strength is provided in the transverse direction.

The grade of prefabrication is lower than CLT or LVL panels, as stressing should be done on-site. In general, the main disadvantage of SLT is the amount of labour required for stressing the panels (Behrens and Benner, 2015). On the other hand, the adaptability of this deck material is high, as adjustments to the stressing can be made during the lifespan of the bridge. When the bridge should be expanded, the entire deck should be taken apart, boards must be added, and the stressing must be performed again. Hindering the infrastructure should be avoided or reduced, as high costs are generated by hinder. When adapting an SLT deck, the entire structure can not be used by traffic, thus hinder occurs. This procedure is different from the other deck materials, in which the modules can just be placed next to the existing ones, and the bridge does not have to be closed off entirely. Furthermore, stressing should be done over the life of the bridge, as stress losses occur. According to Ritter (1990), the tensile stress in the bars should remain at least 40%. When this is maintained, the bridge deck performs well.

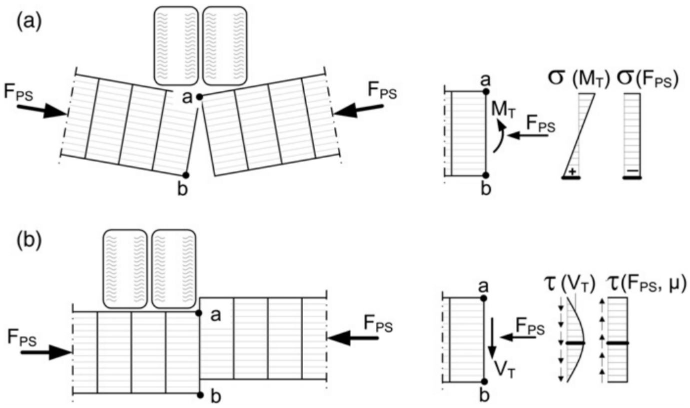


Figure 5.8: Failure modes in SLT deck (Ekholm et al., 2013)

Modelling an SLT deck is more complicated than the other materials considered. As the transverse stiffness is obtained by friction between the boards, slip can occur in vertical and horizontal directions. Ekholm et al. (2013) describe that slip between laminations results in non-linear behaviour in the deck panels. Slip occurs when one of the failures modes of Figure 5.8 occur. Therefore, a non-linear analysis is required in order to model SLT accurately. This study is performed in Karamba3D, which only performs orthotropic, linear elastic analysis for two-dimensional elements. Therefore, assumptions must be made to model the material.

Oliva and Dimakis (1989) performed an analysis on the distribution width of stress-laminated panels. The distribution width is defined as the width in which the load resistance is (almost) uniform across the width. A distribution width of 1.78 metres was found. When the centre-to-centre distances of the beams are not more than 1.78 metres, the deck can be considered an orthotropic plate. Carlberg and Toyib (2012) performed a study on the slip in the SLT panels. The material was modelled linearly, but the interaction between the boards, thus slipping, was modelled in a non-linear manner. The difference between the linear analysis and non-linear analysis for different prestress levels is illustrated in Figure 5.9.

The analysis from Carlberg and Toyib (2012) is used to include the influence of slip in the linear analysis. The deck material is modelled as an orthotropic plate, with the board material's material properties applied (either C24 or GL28h). The properties of these orthotropic materials are displayed in Appendix B. With the deflection obtained from the model, the deflection as presented in Figure 5.9 is used to transfer the deflection from the linear model to a non-linear approximation.

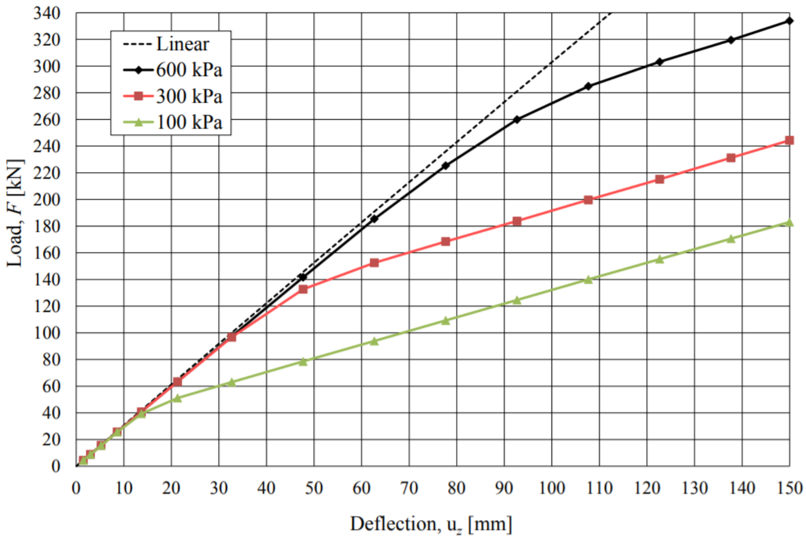


Figure 5.9: Linear and non-linear modelling of SLT deck (Carlberg and Toyib, 2012)

5.2.3. Environmental impact

This section describes an environmental impact analysis of beam/deck materials. All data as used in the figures in this section are stated in Appendix A. The data used is according to the following EPDs and combined with the shadow costs presented in Table 4.1.

- Sawn timber (PE International and Wood for Good, 2013a);
- Glulam (PE International and Wood for Good, 2013c);
- LVL (PE International and Wood for Good, 2013d);
- CLT (PE International and Wood for Good, 2013b).

Figure 4.3 presented the stages of an LCA. These EPDs include stages A1-A5, C1-C4 and D. As the function of the material is unknown, the use phase is not included. The data used can be found in Appendix A and is for one m^3. The environmental impact should be considered in combination with the structural efficiency of the material. As LVL is stronger than sawn wood, less material is required to obtain a structure that complies with the regulations. Section 7.4 elaborates on this.

Table 5.2: Properties of materials used in EPDs

Property	Sawn wood	Glulam	LVL	CLT
Moisture content	15%	12%	12%	12%
Density	483 kg/m^3	490 kg/m^3	488 kg/m^3	488 kg/m^3
Adhesive content	0%	2.1%	2.5%	2.0%
Transportation by sea	Imported pine: 1849 km Imported spruce: 1099 km	643 km	2808 km	300 km
Transportation by road	Imported pine: 796 km Imported spruce: 685 km From UK: 130 km	959 km	721 km	1216 km

Table 5.2 presents the material properties and boundary conditions used for the EPDs. The timber products are all produced in Europe and transported to the UK. However, part of the sawn wood originates from the UK (40.4%), reducing the environmental impact of transportation for sawn wood.

Figure 5.10 clearly indicates that the GWP is more negative for sawn wood than for the other wood products. The EPD used does not make a distinction between fossil GWP and biogenic GWP. Biogenic GWP represents the carbon captured during growth (negative impact), and fossil GWP represents the impact of fossil fuels during production. For fossil GWP the contributors are: "in decreasing order, the use of fossil fuels during tree felling, production of adhesives, transport to the factory and used energy" (Van Wijnen, 2020). The significant difference between softwood and the other products can be explained accordingly: adhesives are not applied in sawn wood, where it is in the others. Furthermore, the sawn wood has a higher moisture content, thus less drying has occurred. Accordingly, it has a smaller impact on AP and EP. The tree felling can also cause a higher impact on AP and EP, but according to the EPDs, the same procedure is used for sawn wood.

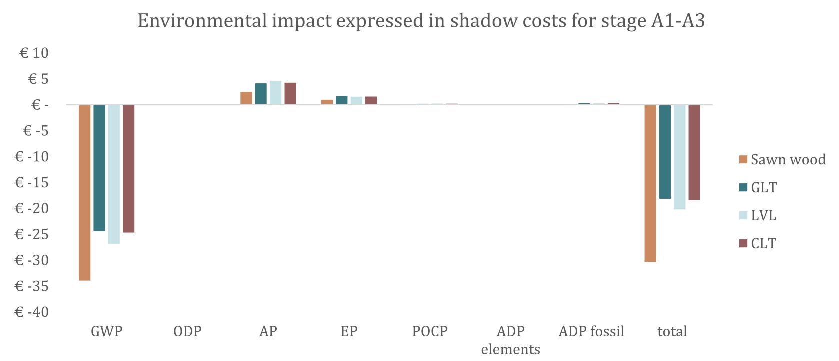


Figure 5.10: Environmental costs per m^3 for four timber products in stage A1-A3

Figure 5.11 illustrates a significantly higher environmental impact for LVL, mainly caused by the higher transportation (over sea). Furthermore, as part of the sawn wood originates from the UK, a significantly lower impact due to transportation can be observed there. However, the influence from stages A4-A5 is minor on the total environmental impact, so the results are still considered comparable.

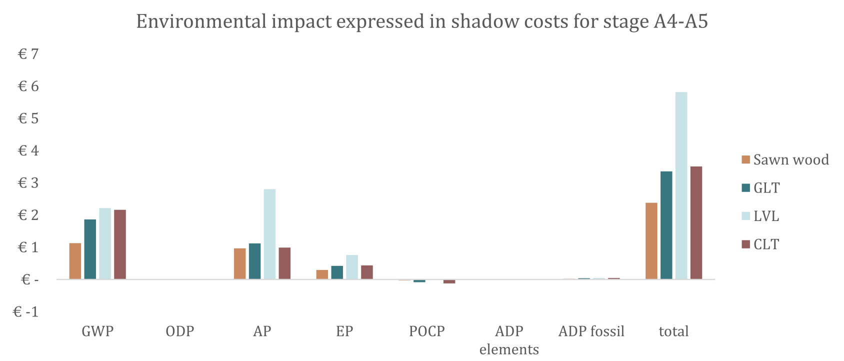


Figure 5.11: Environmental costs per m^3 for four timber products in stage A4-A5

The occurrence of end of life scenarios as stated in Section 4.1.2 is applied to the environmental costs, resulting in Figures 5.12 and 5.13. The environmental impact of both stages C and D are similar for all products. For stage C, the main impact is the release of CO_2 in phase C3, causing the high influence of GWP on the total environmental costs. All contain one m^3 of wood that releases CO_2, explaining the similarity between the four timber products. The current Dutch and European regulations do not account for any carbon delay to be included (Keijzer et al., 2021).

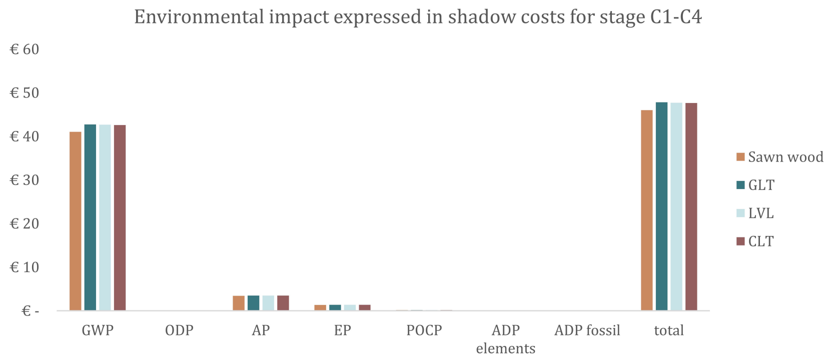


Figure 5.12: Environmental costs per m^3 for four timber products in stage C1-C4

The negative costs in stage D have a significant impact on the total environmental costs. All products end up with negative costs, which would mean that more material use is beneficial for the environment. However, the remark must be made that reduction of material use is always desired, and high uncertainty lies in the end of life scenarios.

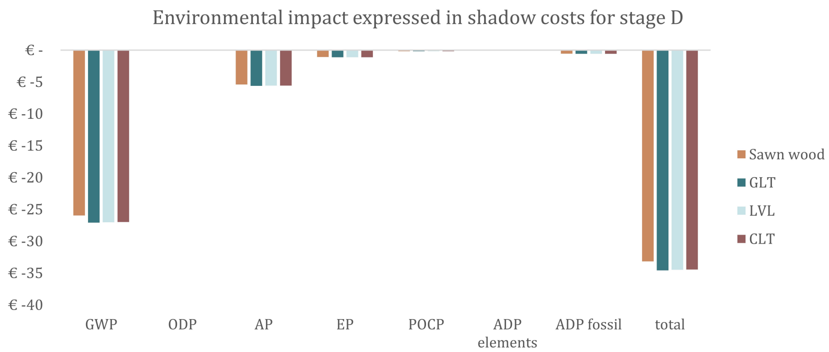


Figure 5.13: Environmental costs per m^3 for four timber products in stage D

As stage D represents benefits out of the system, this stage is not considered in the environmental costs for the beam material comparison (Figure 5.14). More on stage D is included in the discussion (Chapter 9).

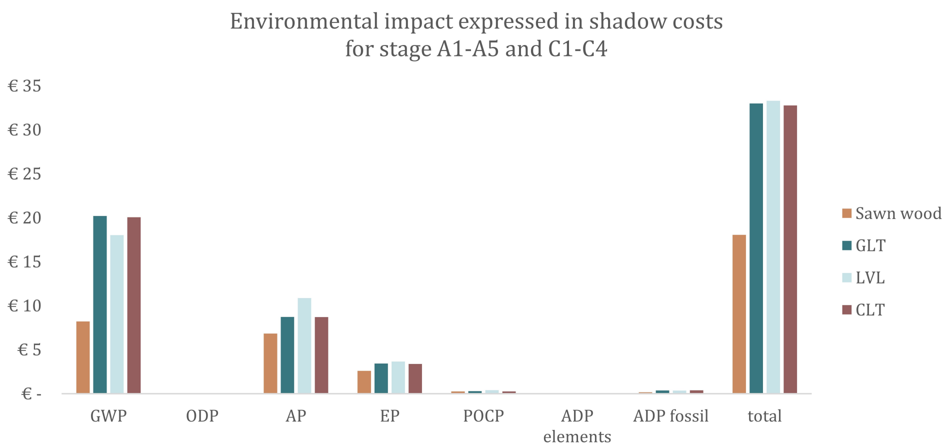


Figure 5.14: Environmental costs per m^3 for four timber products in stages A1-A5 and C1-C5

It can be seen that sawn wood has the lowest environmental impact than the other materials. Glulam, LVL and CLT are similar in the total environmental costs. The environmental costs as presented in Figure 5.14 are per cubic metre and should be combined with the material efficiency. An analysis of material efficiency is performed in Section 7.4.

5.3. Connections

As stated in the literature review on the CE, demountable connections are crucial to obtain a circular bridge. By applying demountable connections, the circular design strategies can be obtained, as stated in Section 4.1. However, not all connections need to be demountable, which is elaborated further in Section 7.3.2.

In this section, the types of connections as required in the bridge design are stated. The criteria on the connections are described, to which each connection is tested. Consequently, options for the types of connections are described, followed by an assessment based on the criteria. The complete scoring of the connections is presented in Appendix C. This section provides a short introduction to the connections and the primary literature found. At the end of the chapter, an overview of the scores of the connection is given. Their calculation and behaviour are described in Section 7.6, providing the answer to research question 2e.

5.3.1. Criteria on connections

As a modular system must be obtained, additional criteria are stated, compared to a solid structure that does not have to be taken apart. Not all connections have to be demountable, but all connections are scored on their ability to function in a demountable structure in this chapter. The suitability for demounting is expressed in two criteria:

- Amount of labour required to mount;
- Reuse potential.

The amount of labour required is indicated by estimating the number of fasteners that should be mounted and the grade of prefabrication. The possibility for demounting and reuse is mainly influenced by the type of fasteners applied. For example, glue cannot be demounted, where dowels and bolts are relatively easy to demount.

Furthermore, four criteria apply to every connection (demountable or not):

- Structural efficiency;
- Stiffness;
- Amount of steel required;
- Reliability.

With high structural efficiency, less material is required to obtain sufficient strength. Rotational stiffness is required as connections between modules must be made in the free span. When the rotational stiffness of the connections is low, the deflection increases significantly. Translational stiffness (slip) is important for all connections, as slip in the connection causes less efficient load transfer. No extensive environmental impact analysis is performed on the connections, but the amount of steel used is a good indicator. The environmental impact of timber is relatively low compared to steel (Hassan and Johansson, 2018). Thus most of the environmental impact of the connections will be caused by the amount of steel. Finally, the reliability of the connection is included. The reliability is influenced by the available knowledge level and the warning given when likely to fail. The latter is influenced by brittle or ductile behaviour.
Similar to the material variants, more criteria can be considered, but a more detailed study is required. Interesting criteria are costs and structural behaviour in the long term.

5.3.2. Type of connections

This section describes the different types of connections and their functions. Following this, options for the types of connection are given, with their advantages and disadvantages. Three types of connections that are applied in this bridge design are:

- Beam-beam connection: Both in the same direction (beam splice) and perpendicular to the beam.
- Panel-panel connection: Deck panels are connected along the sides of the panels, preventing uneven deformations and providing transfer of forces.
- Beam-panel connection: The bridge deck should be connected to the main beams, by which different levels of composite behaviour can be ensured.

5.3.3. Beam-beam connection

Moment-resisting connection

A beam splice should be made between the main girders. As modules are applied to obtain the span of the bridge, splices must be made in the longitudinal direction. These connections must have a significant moment resisting capacity to transfer bending moments and provide rotational stiffness. When this capacity is too low, the deflection of the bridge increases significantly: regulations will not be met, and people will not feel safe on the bridge. Three options arise for this connection:

- Slotted-in steel plate with dowelled connection;
- Glued-in rods;
- End-plate connection.

In all the connections as described below, steel elements are applied. Two reasons can be given for this: increasing the stiffness of the connection and obtaining ductile failure. Brittle failure occurs in timber when it fails on tension or shear (Rouger, 1995). In ductile failure, significant displacements occur, giving warnings before failure (Pokluda and Šandera, 2010). Consequently, measures can be taken on time, and total failure of the structure can be avoided.

Slotted-in steel plate

Within the slotted-in steel plate concept, one or more steel plates are inserted in the timber beams, as illustrated in Figure 5.15.

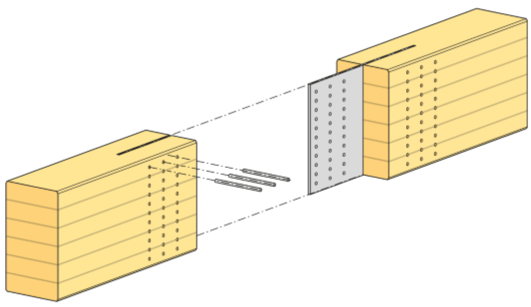


Figure 5.15: Slotted-in steel plate connection (Setra, 2020)

With bolts or dowels, rotational stiffness can be obtained when placed in a rectangular or circular pattern. The stiffness and strength of the connection increase further when a steel plate is added. Consequently, this is applied in the beam splice connection. Jorissen (1998) states that the effective number of fasteners is lower than the number of fasteners applied, as the distribution between the fasteners is not equal. Therefore, not all fasteners are utilised effectively, which should be considered for the efficiency of the connection.

Glued-in steel rods

A wide variety of connections exist under the name of glued-in rods. The common characteristic is that it contains timber, steel and an adhesive (Tlustochowicz et al., 2011). The rods are often made out of steel as ductile failure can be obtained. Glued-in rods are often used for strengthening structures and new works (Serrano et al., 2008). By embedding rods in the timber with an adhesive, the timber-steel connection is strong. However, as this connection is so strong, either the rod or the timber itself will fail. When the rod is pulled out, a layer of glued timber can be pulled out as well. Azinović et al. (2018) demonstrates this for CLT, but it can occur in sawn wood, glulam or LVL as well.

In the application of a splice connection, the rods can be connected to a steel plate, as illustrated in Figure 5.16.

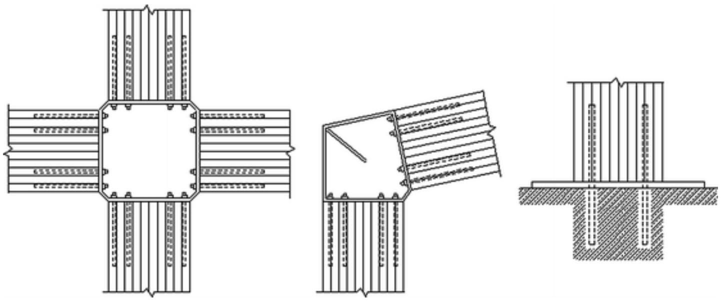


Figure 5.16: Glued-in steel rods connection (Tlustochowicz et al., 2011)

End-plate connection
A variant of the previous two connections is the end plate connection. Steel plates or rods are glued into the timber elements and welded to a steel end-plate (Cepelka and Malo, 2014). This procedure is done on both sides of the connection, and the endplates are bolted together. The application is not often applied yet, and further research on the embedding of the endplate to the timber member must be performed.

Hinged connections

In the locations where no rotational stiffness is required, hinged connections are wise to apply, as less steel and fasteners are required. Furthermore, the bending moments in the timber members can be reduced by applying hinged connections, as the bending moment in the connection should be zero. In bridge structure, hinged connections are mainly applied between beams that are oriented perpendicular to each other. Furthermore, hinged connections are applied to connect the deck panels in this bridge.

For the beam to beam connections, a wide variety of connections are available on the market. The ones considered here are:

- Joist hangers and concealed brackets;
- Interlocking joints.

In the first connections, steel elements are combined with fasteners, where only timber is used for the latter. The use of steel has various advantages and disadvantages, described below.

Joist hangers and concealed brackets
Joist hangers are a simple and often applied way of connecting two beams that intersect. The continuous beam is connected to the other beams by a steel hanger, where the other beam is placed in. The joist hanger is attached to the continuous beam by screws or nails. When the attaching beam is placed into the hanger, screws, nails or bolts are put through the beam and the hanger to assemble it.

The concealed bracket connection works similar, but the hanger is inserted in the beam. According to the MyProject software from Rothoblaas, larger forces can be transferred by the concealed bracket, compared to the joist hanger. Both connections have a similar connection to the continuous beam and are presented in Figures 5.17 and 5.18.

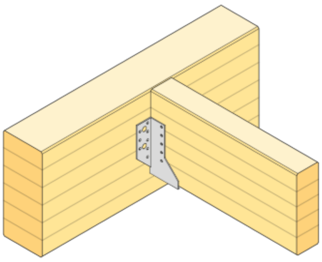


Figure 5.17: Joist hanger connection (Setra, 2020)

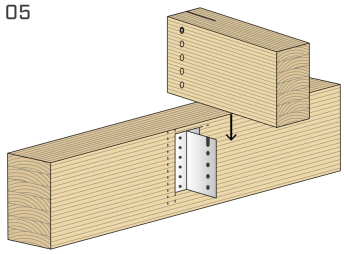


Figure 5.18: Concealed bracket connection (Rothoblaas, 2021b)

Interlocking joints

In Japanese timber structures, interlocking or carpentry joints are often applied. The timber is sawn into a fine and specific pattern, which resembles a pattern in the attaching member (Erman, 2002). High accuracy is crucial in this manufacturing, and therefore these joints were not applied in the European market for a long time. However, due to digital manufacturing, the level of accuracy can now be obtained by computers and becomes more economically feasible (Koning, 2018).

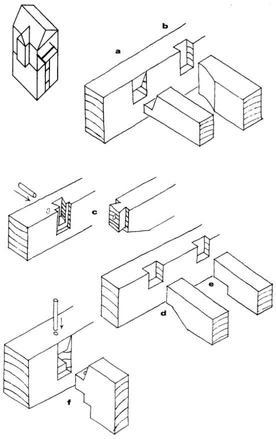


Figure 5.19: Carpentry joints (Erman, 2002)

Strut connection

Crocetti (2016) describes connections for the bottom of a timber arch used in bridge design (Figure 5.20). The bottom of an arch is a hinged connection, which has to transfer large compression forces to the support, similar to the requirements for a strut. As the connection is hinged, no bending moments occur in the struts, acting as a two-forced member. Therefore, this connection is suited to be applied on both ends of the struts.

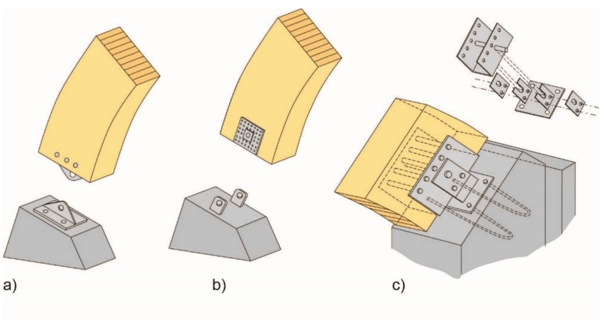


Figure 5.20: Strut connection (Crocetti, 2016)

5.3.4. Panel-panel connection

It is decided that the connection between the panels is hinged. No demountable moment-resisting connections can be found, which can be applied in the geometry of this bridge. The options that are investigated are:

- Screws;
- Slotted-in steel plate;
- X-RAD.

Screws

The most often applied connection between timber panels is screwed. The panels are half-lapped, and the overlapping part is connected by screws (Figure 5.21). The main disadvantage of this connection is that it is not demountable, and splitting of the deck panel can occur (Muñoz et al., 2010). However, it is quick and easy to assemble.

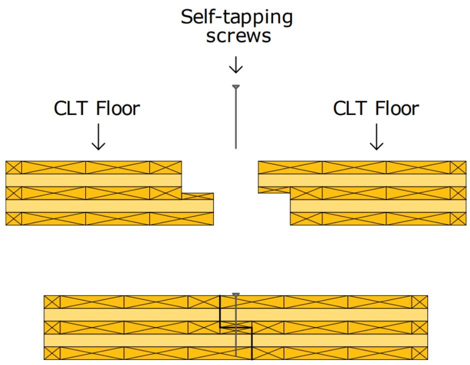


Figure 5.21: Half lapped screwed joint (Mohammad et al., 2013)

Slotted-in steel plate

The same principle as used for the beam splice can be applied for deck panels as well. A steel plate is slotted in the deck panels and connected by bolts. A resilient and robust connection is obtained. The plate is inserted in the plane of the deck panel, not perpendicular to it as done for the beam splices.

X-RAD connection

The X-RAD connection is an often applied connection in modular building design. This connection is mainly prefabricated; only bolts in the outer parts are mounted on site. The screws in the X-RAD are placed to be subjected to a tension force (Polastri and Angeli, 2014). Therefore, shear is not dominant, and the connection is more stiff and strong than traditional connectors.

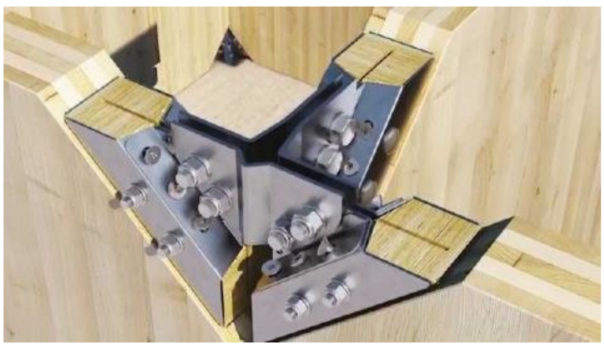


Figure 5.22: X-RAD connection (Rothoblaas, 2021a)

5.3.5. Panel-beam connection

The stiffness of the structure can be increased significantly by ensuring composite behaviour between the deck and girders. To obtain composite behaviour, slip between the elements must be limited. Multiple studies are done on ensuring composite behaviour between concrete decks and timber girders, which can also be applied to timber decks. Several connection types are indicated by Ceccotti (2002) to obtain composite behaviour between a timber-concrete structure, as presented in Figure 5.23.

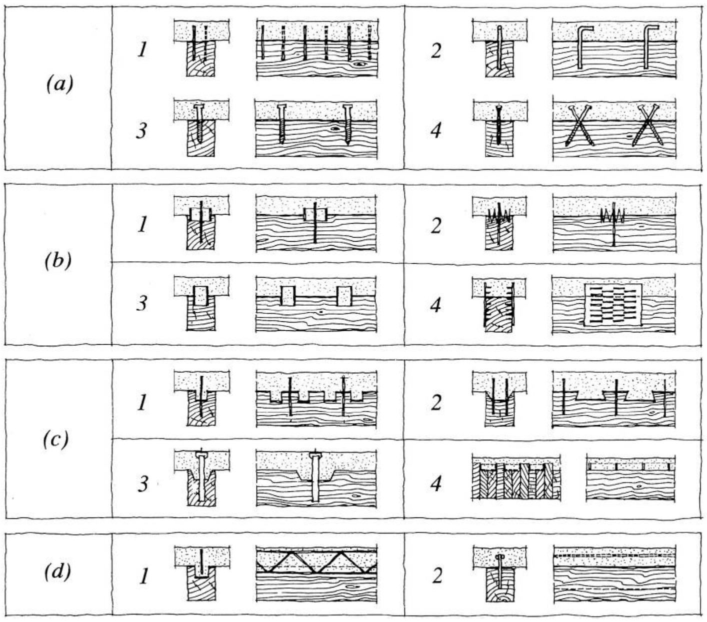


Figure 5.23: Examples of connections for composite behaviour (Ceccotti, 2002)

Most connections, as presented in Figure 5.23, can not be considered reusable. Dowels can be applied in a circular structure but are less stiff. Inclined screws can, in theory, be demounted but not remounted and therefore not reused (Mariller, 2020). Inclined screws obtain higher stiffness compared to dowels. Furthermore, the stiffest option is to glue the members together. The considered options are:

- Dowels;
- Inclined screws;
- Glue.

Dowels

The connection with dowels (a.2) can be executed in a demountable manner. However, a relatively large slip occurs when dowels are applied due to the limited stiffness of this connection. As more slip occurs, the composite behaviour decreases. Furthermore, large deformations occur over time, which makes the demounting of the dowels difficult in practice.

Inclined screws

Inclined screws can be applied to connect the deck to the girders (a.4). The main advantage of screws compared to dowels is the increased embedment in the timber, which increases both strength and stiffness. In addition, when the screws are placed in an inclined manner, flexural stiffness increases further, and therefore less slip occurs (Symons et al., 2010). However, as stated before, screws are not reusable.

Glued connection

The last, again not demountable, option is to glue the deck panels together. By glueing the connection, high strength and stiffness are obtained (Grunwald et al., 2014). No slip occurs in the adhesive, but delamination can occur in the adjacent timber elements. Furthermore, in case of failure, no warnings are given, and brittle failure occurs.

5.3.6. Overview of connection study

Appendix C provides a multi-criteria assessment on all connection categories, as summarized below. In Chapter 7, assumptions are made for the use of connections in the global design. Consequently, in Section 7.6 the connections are worked out in detail for the final design.

Table 5.3: Overview moment-resisting beam-beam connection

Criterion	Slotted-in steel plate	Glued-in rods	Endplate
Amount of labour required	-	++	++
Possibility for reuse	++	++	++
Structural efficiency	++	-	+
Stiffness	++	++	-
Amount of steel required	-	-	- -
Reliability	++	-	-
Score	47	40	33

Table 5.4: Overview hinged beam-beam connection

Criterion	Joist hanger	Interlocking joint
Amount of labour required	++	+
Possibility for reuse	++	++
Structural efficiency	+	-
Stiffness	-	-
Amount of steel required	- -	++
Reliability	++	- -
Score	40	33

Table 5.5: Overview panel-panel connection

Criterion	Screws	Slotted-in steel plate	X-RAD
Amount of labour required	+	++	+
Possibility for reuse	- -	+	++
Structural efficiency	-	++	+
Stiffness	- -	++	+
Amount of steel required	+	-	-
Reliability	++	++	-
Score	27	50	37

Table 5.6: Overview shear connection for composite behaviour

Criterion	Dowels	Inclined screws	Glue
Amount of labour required	- -	+	++
Possibility for reuse	+	- -	- -
Structural efficiency	-	+	+
Stiffness	-	+	++
Amount of steel required	-	+	++
Reliability	++	++	- -
Score	27	37	37

III

Circular design

6 Module design

In this chapter, the application of modules within the bridge system is elaborated. An optimisation is performed on the dimensions of the modules. Furthermore, the build-up of modules for the typologies is made. This chapter provides the answer to research question 2a.

6.1. Arrangement of modules

As was stated in the definition of the design space, perpendicular and non-perpendicular crossings are included. For perpendicular crossings, rectangular modules work well. However, additional elements are required for skewed crossings to attach to the existing infrastructure. As this is needed for both ends, a rectangular module can be sawn on the right angle, or custom made elements can be produced. Modules are made rectangular, as illustrated in Figure 6.1, and can be completed with triangular elements.

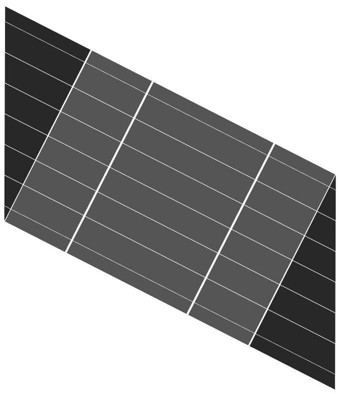


Figure 6.1: Arrangement of modules when crossing is not perpendicular

For the number of standard modules, a balance must be found between a grade of standardisation and the fitting in the replacement task. When many different modules are produced, the advantage of having standardised elements is reduced, and costs increase. However, when very few are produced, it is hard to fit them within the existing infrastructure. Consequently a high amount of excess material is used. Two primary goals define the effectiveness of the modules:

- Reduce the excessive length and width of the bridges;
- Reduce the number of connections.

An economic study can be performed into this balance, combining the economic and the functional aspect. Economic aspects are out of the scope, thus it is decided to work with two standard dimensions in length and one standard dimension in the width direction. The modules can also be produced with half of the dimension to create more flexibility. As a result, four lengths exist and two widths, thus eight standard sizes.

6.1.1. Placing of modules

The arrangement of the modules in the spans is based on the shapes of the moment and shear diagrams. The shape of the bending moment and shear lines are presented in Figure 6.2 for a beam with a constant distributed load.

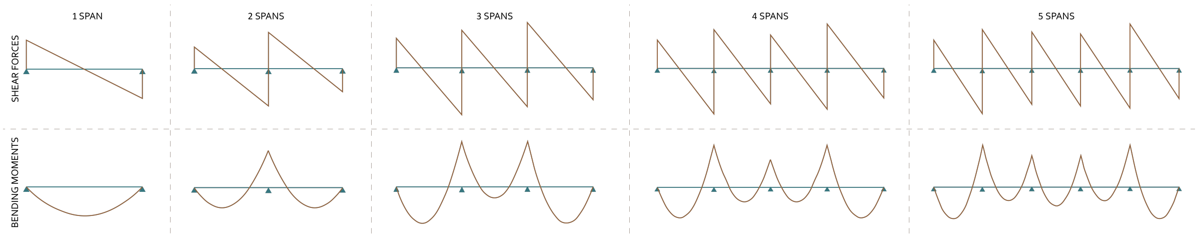


Figure 6.2: Shear and bending moment diagrams for continuous beams with one to five spans, subjected to a constant distributed load

The bending moment is often at its maximum value at the location of a support point. The same applies to the shear force. In bridges that are not part of a modular system, a hinged connection is often used at the supports, thus the beams act as simply supported beams. However, no standardisation is found in the distance between the supports for the bridges in the design space. Consequently, connections can not be applied on top of the supports for all bridges: one must make connections between modules in the free span, with rotational stiffness and bending moment capacity. One should avoid beam splices in the middle of a span to reduce the deflection and large forces in the connections. In principle, the script as written places the large modules in the middle. However, if the free span is approximately equal to the size of the large module, the module splice is at the supports.

A similar but simplified script is written for the width direction (Figure 6.4). Two different dimensions are used in this direction, and again the largest modules are placed in the middle. In the following section, the dimensions of the modules are derived.

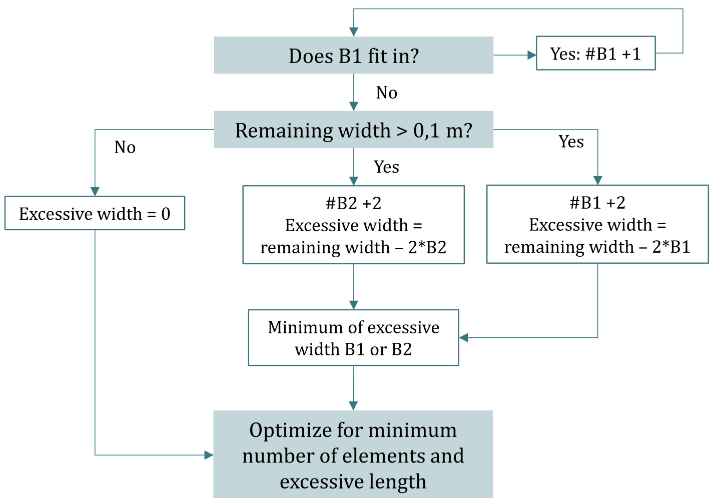


Figure 6.4: Script for arranging modules in width

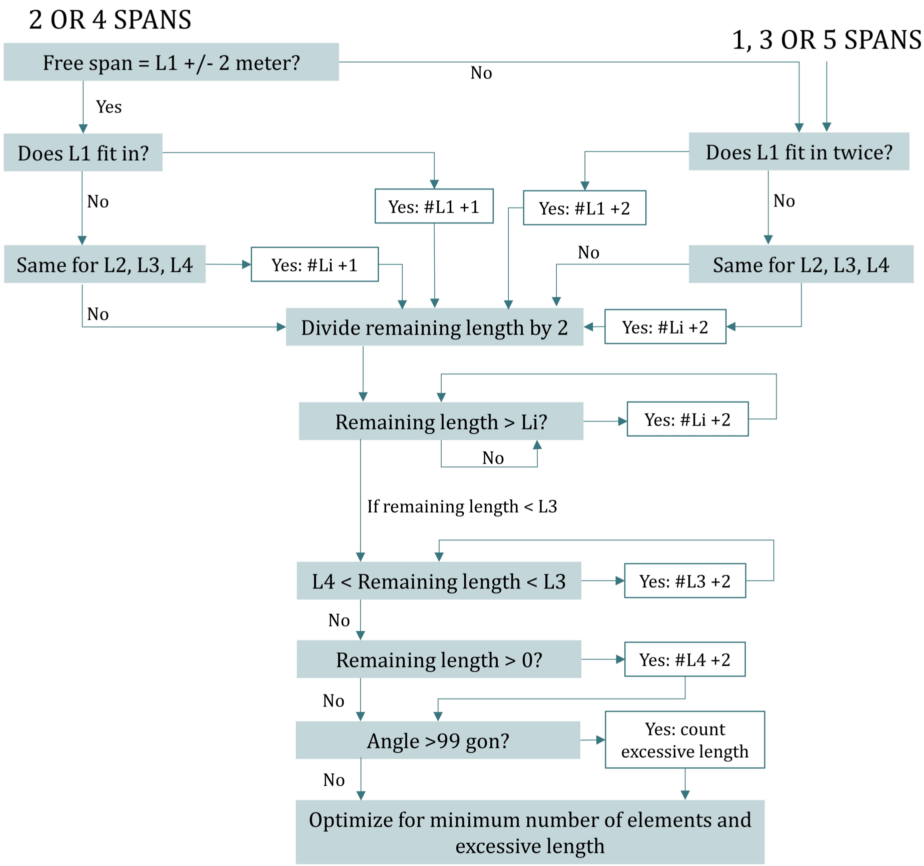


Figure 6.3: Script for arranging modules

6.2. Dimensions of modules

The replacement of existing structures and the adaptable aspects of the timber structures are considered to determine the dimensions of the modules. The aspects to consider for the replacement structure were mentioned before: reduce the excessive length/width of the bridges and reduce the number of connections. To make the dimensions suitable for the adaptable structure, it is convenient when the module's dimensions match the dimensions of a lane. A lane is 3.5 metres wide, and as expansion can occur in two directions, a square module of 3.5 metres is suitable. For the second width of the modules, 1.75 metres is applied (half of 3.5). With two beams placed in the width direction for the wide module, the narrow module has one beam. The same centre-to-centre distance between girders is maintained for both module sizes and thus throughout the entire structure.

As the dimensions in width are determined, the lengths of the modules should be derived. One module is 3.5 metres square to function as the expansion module: expansion is covered for perpendicular crossings. In expanding a bridge with a non-perpendicular crossing, compensation length can be obtained from the freedom in the abutments. It is advised to make the abutments with a slope of 3:2, but it can go up to 1:1. With a minimum free height of 4.6 metres, this results in 2.3 metres of extra length on both sides to compensate.

Similar to the width, the modules can also be divided into two modules in length. One should define the dimensions of the large module (L1), and half of it is called L2. An optimisation is performed to minimise the number of connections and fit the modules in the existing structures in an optimal manner. The excessive length and number of connections are obtained for the bridges in the design space, using the Python script from Figure 6.3. These values can be transferred into a score to define the most optimal combination. L3 is given as fixed input, the variable to optimise for is L1, and respectively L4 and L2 are half of it.

An optimisation is performed with the Galapagos optimiser in Grasshopper. Two outputs are combined to measure the fitness of the solution. Firstly, this is the number of modules needed to generate all 1205 bridges within the design space. Secondly, the total of the excessive length [m] that is generated is counted for the bridges without a skew. The latter is multiplied by ten to make them equally important for the score. The bridges with a skew are not considered in the score, as custom-made elements must be produced and do not have to fit in with the module dimensions.

As a result of this optimisation, it turns out that L1=17.5 metre results in the best-suited set of module lengths, which also fits in the production (and thus transportation) requirements. Table 6.1 presents an overview of the number of modules needed for the design space. It can be seen that there is an equal distribution between all module lengths. In conclusion, the eight modules are used, as illustrated in Figure 6.5. All modules fit within transportation limits.

Table 6.1: Modules after optimisation

Module	Length (m)	Amount of modules used when all bridges in design space are modelled
L1	17.5	2579
L2	8.25	1177
L3	3.5	1474
L4	1.75	2114

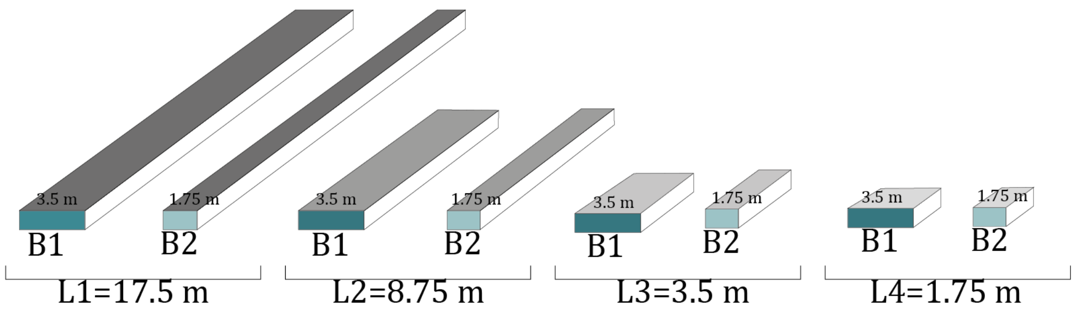


Figure 6.5: Eight modules that are used and their dimensions

Figure 6.6 illustrates an example of the arrangement of modules. Two spans of 22 metres are applied, with a width of 23 metres. In orange, the support points and original length are indicated. One can see that the length is exceeded slightly by the small (L4) modules. Section 7.3 elaborates on the arrangement of modules in standard road configurations.

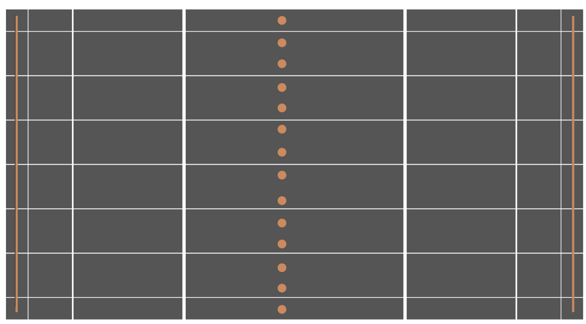


Figure 6.6: Arrangement of modules in a structure with 2x22 metre span, width 23 metre

6.3. Build-up of modules

Different modules are designed for the beam/strut typology and the truss typology. The difference is determined by the direction in which the loads are transferred. The beam/strut typology transfers from the deck to the main girders, thus in the longitudinal direction to the abutments. The deck in the truss structure transfers in the transverse direction to the trusses, and the trusses transfer in the longitudinal direction to the abutments.

6.3.1. Beam and strut frame

The beam and strut frame structures are similar, with free span differences (reduced by the struts). At the module level, no distinction is made in the design. On the global level, some differences occur in connections, magnitude of forces and scale on which one can apply it.

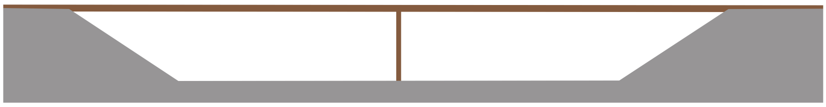


Figure 6.7: Sideview of beam typology

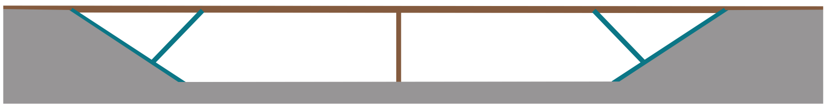


Figure 6.8: Sideview of strut typology

Arup made an extensive trade-off on the design on module level for a beam structure. Multiple configurations of elements are considered. Arup set similar design strategies for their design, but the demand for demountability was less crucial. The findings and results of this option study are used to assess the configuration of elements. Not all configurations suited a modular system, thus taken out of the comparison. Below, the options and most important findings are stated. In Appendix D, the complete trade-off from Arup is presented. The configurations that are considered are:

1. Beams with cross beams on top and a deck on top of the cross beams.
2. Beams with integrated cross beams, with a deck on top.
3. Only beams in the longitudinal direction, with a deck on top.
4. Block glulam beams, which cover the entire width. Only a thin deck on top for continuity.
5. Hollow rectangular timber beams, with a deck on top.

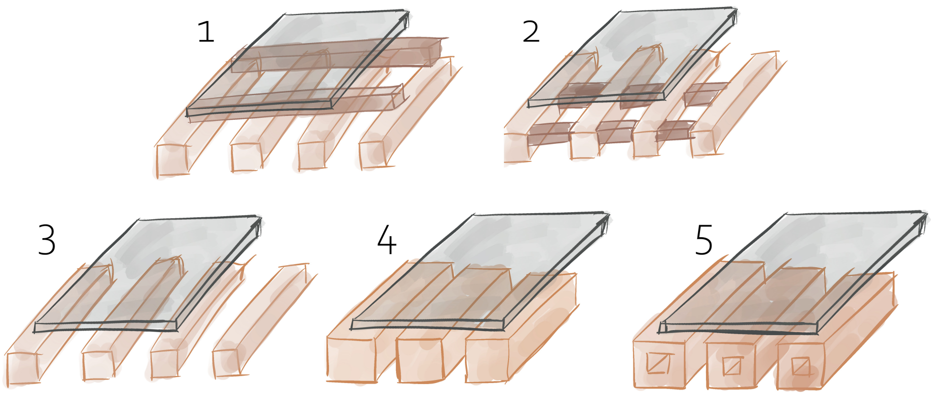


Figure 6.9: Options for configurations of beams and deck

Figure 6.9 illustrates the configurations, all have their advantages and disadvantages. These are described shortly per option, based on Appendix D.

Table 6.2: Advantages and disadvantages of beam-deck configurations

Option	Advantages	Disadvantages
1	Cross girders ensure distribution of concentrated forces over multiple main girders, reducing deformation differences in the deck.	Large construction height due to cross girders.
	High possibility for ventilation is created due to spacial structure.	No composite behaviour between deck and main girders can be obtained.
2	Cross girders ensure distribution of concentrated forces over multiple main girders, reducing deformation differences in the deck.	Limited ventilation is possible due to dense structure.
	Reduced construction height due to integrated cross girders and composite behaviour.	A high number of connections are required.
3	Composite behaviour can be obtained between the deck and girders.	Large deformation differences in the deck since only main girders are applied.
	Possibility for ventilation.	The main girders become very high because no redistribution between girders can occur.
4	Because of large width of girders, the height can be reduced.	Large deformation differences in the deck since only main girders are applied.
	Composite behaviour can be obtained between deck and girders.	Limited possibility for ventilation.
5	Reduced mass of structure due to hollow sections.	Large deformation differences in the deck since only main girders are applied.
	Composite behaviour can be obtained between the deck and girders.	Peak stresses occur in hollow sections around the sharp edges, not covered in Eurocode.

As indicated in the Arup's trade-off, the peak stresses and differences in deformation are important aspects. The redistribution of axle loads can cause these differences and lead to the failure of the deck. Furthermore, ventilation is an important aspect to prevent moisture problems.

In the typology study, configurations 1 and 2 are applied. These perform best on the distribution of local loads to reduce the stresses and deformations in the deck. The main difference between these two is the effective construction height and the possibility of ventilation.

The main girders are placed in the longitudinal direction, as well as the grain of the timber. For configuration 1, the secondary beams are placed on top of this, spanning over the width of the module. For configuration 2, these are placed in smaller parts between the main girders. On top of the beams, the deck is placed. The build-up of the modules is presented in Figures 6.10 until 6.13.

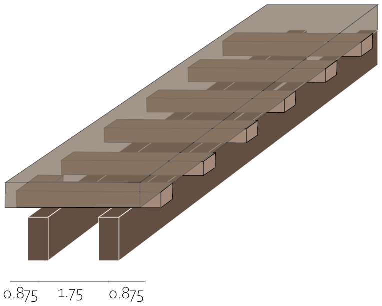


Figure 6.10: Configuration 1

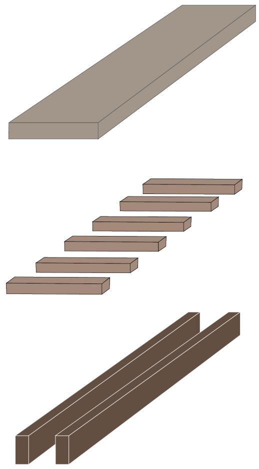


Figure 6.11: Exploded view of configuration 1

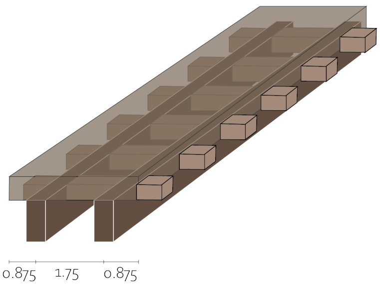


Figure 6.12: Configuration 2

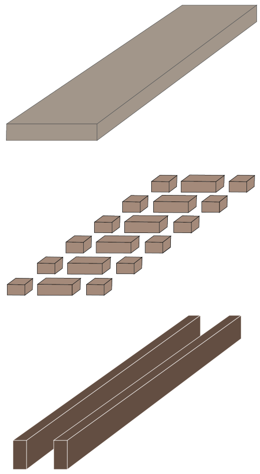


Figure 6.13: Exploded view of configuration 2

6.3.2. Truss structure

The configuration of load-bearing elements is different for a truss structure compared to a beam structure. All forces are transferred to the trusses, done by beams in the transverse direction, together with the deck.

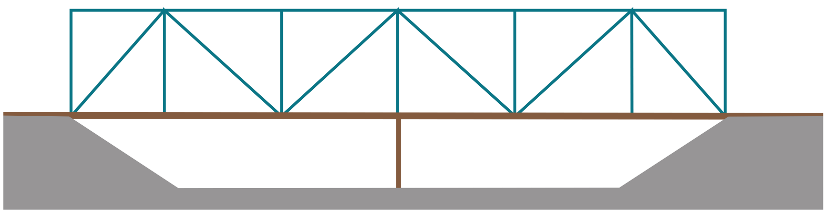


Figure 6.14: Sideview of truss typology

The truss structure is only effective when the width between the trusses is not too large. The efficiency depends on the stiffness of the cross girders and the length and width of the bridge. When the bridge exceeds a certain width, a truss in the middle of the bridge can also be applied. In Section 3.2, the typical road layout is described. When the bridge contains at least two lanes in each direction, traffic in the opposite direction can be separated by a verge. Thus the minimum width of the bridge with three trusses is: 2*free space (1 m) + 2*line (0.2 m) + 4*lane (3.5 m) + 2*redress lane (0.6 m) + middle verge (2.5 m) = 20.1 metre. In conclusion, a bridge with a width of more than twenty metres can have an additional truss in the middle verge of the bridge.

As the maximum distance between the trusses is set to twenty metres, a configuration for the modules can be derived. Still, twenty metres is a significant span for a deck and girders with concentrated axle loads on it. The truss works most efficient when all loads are transferred directly to the trusses, thus beams in the longitudinal direction is not desired. By only applying girders perpendicular to the truss, the deflection of the deck between these girders becomes too large. The concentrated axle loads cause high local deflection in the deck when cross-wise girders are not present. Therefore, a combination of cross-wise girders and girders in transverse direction can be applied to obtain sufficient stiffness in the structure. The configuration is presented in Figure 6.16.

The Warren truss with verticals is applied, as this fits in a modular system. The direction of the diagonals are alternated, thus one can easily add a module in the length direction. However, the verticals of the truss should always be above support points. For the replacement task, attaching to the existing infrastructure can be difficult. The sizes of the truss elements resemble the modules, applied in the following way:

- Module L1: two truss parts are made.
- Module L2: one truss part is made.
- Module L3: when two modules of L3 are adjacent, one truss part is made. If only one L3 is made, one truss part is made as well.
- With only one module L4 at the end, it is joined to the module next to it, and they form one truss part together.

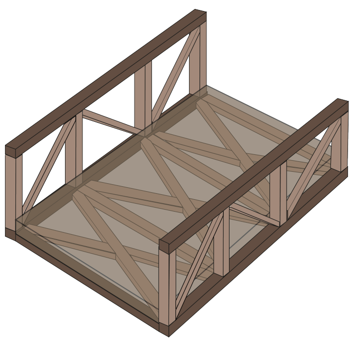


Figure 6.15: Configuration of truss

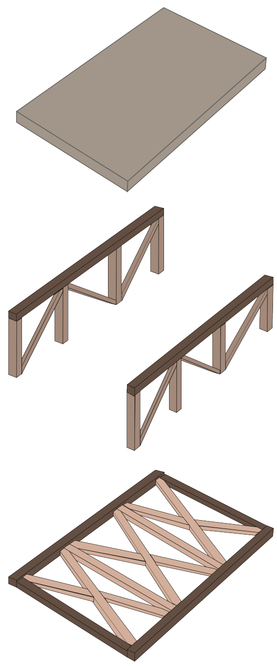


Figure 6.16: Exploded view of truss configuration

6.4. Conclusions

In conclusion of this chapter, research question 2a can be answered:

How to translate the standardisation and boundary conditions a bridge into a design for a modular system?

As stated in the design strategies, a modular system is derived, leading to a circular timber bridge. The modular system is a rectangular system, with eight sizes of modules: four variations in the length and two in the width direction. With this modular system, a balance is found between fitting in the existing infrastructure and standardisation. Furthermore, the module's arrangement is based on the acting forces in the structure. This fulfils the objective to reduce forces in the connection. Finally, module designs are made for all three typologies. These are studied in the next chapter, after which a conclusion is drawn on the applied typology. Section 7.3.1 elaborates on the applied standardisation and the scope of the final design.

7
Bridge design

This chapter investigates the design aspects as stated in the literature review. First, the methodology for design and variant studies is elaborated. Then, the typology, standardisation, materials and connections are determined for an optimised system. In conclusion, the final design is presented, providing an answer to research question 2.

7.1. Methodology

This section describes the method of the structural analyses. First, the performance indicators for the trade-off are defined. Subsequently, the parametric workflow is given in short. An extensive description of the model is found in Appendix E. This section is the basis for all structural analysis and variant studies performed.

7.1.1. Trade-off matrix

After considering the design choices of Chapter 6, five options for the bridge's design remain, as displayed in Figure 6.10 until 6.16.

- Cross girders on top of main girders (hinged), with the deck connected to the cross girders
  - Regular beam structure (v1b)
  - Strut frame (v1s)
- Cross girders between main girders (hinged), with the deck connected to the main girders
  - Regular beam structure (v2b)
  - Strut frame (v2s)
- Truss structure (T)

The five options are investigated further, modelled and finally assessed by a trade-off matrix (TOM). They are named by the abbreviations mentioned above.

In the TOM, several performance indicators are set, and a weight is added to them. The design strategies and actions determine the performance indicators. Four categories are made according to the strategies from Section 4.3. First, the performance indicators and their weights are given in Table 7.1, the criteria are stated in Table 7.2. The weight factors are described relative to each other, furthermore the total score is set to 100. The explanation of weights to the scores is explained in Appendix G.2.

Table 7.1: Weight factors of TOM

Strategy	Performance indicator	Weight
Effective	Mass - deflection	15
	Mass - strength	10
	Construction height	10
Protected	Moisture	10
Flexible	Widely applicable	10
	Adaptability	15
Demountable	Reaching connections	5
	Complexity	15
	Connection rigidity	10
Total		100

Table 7.2: Criteria for TOM

Strategy	Performance indicator	Criteria
Effective	Mass - deflection	How much mass is needed to satisfy the SLS requirements? Is the deflection evenly distributed?
	Mass - strength	How much mass is needed to satisfy the ULS requirements? Are the stresses evenly distributed? Are there very large peaks in the stresses?
	Construction height	What is the required construction height to satisfy both SLS and ULS requirements? Is this height used efficiently?
Protected	Moisture	Is the structure protected against weather influences? Can the structure ventilate and dry when it got wet?
Flexible	Widely applicable	Is the structure applicable for all skews/lengths/widths within the scope?
	Adaptability	Is the structure adaptable in width of the bridge? Is the structure adaptable in length of the bridge?
Demountable	Reaching connections	Are the connections easy to reach? Is the space available to place the connections that are needed?
	Complexity	Are there many connections in the construction? How complex are the connections (how many members join in the connection)?
	Connection rigidity	Do large bending moments occur at the location of the connections? Is there much rotational stiffness needed in these connections?

7.1.2. Parametric model

This section describes the structure and options in the parametric model, made in the Rhino-Grasshopper environment. Appendix E describes the full script and input values. A total of five options are modelled as described in the beginning of the chapter: v1b, v1s, v2b, v2s and T.

The model is made to function for all Rijkswaterstaat bridges that are in the design space of this research. This results in a total of 1205 bridges, each with five configurations. To reduce the amount of results, five bridges are chosen to analyse, illustrated in Table 7.3 and Figure 7.1. This set of bridges cover all different properties of the bridges, which are variations in length, number of spans, width and skew. By making the model parametric, all bridges can be modelled. Furthermore, bridges outside of the dataset can be modelled as well by changing the input manually.

Table 7.3: Five selected bridges

Number	Nr of spans	Length (m)	Length per free span (m)	Skew (gon)	Width (m)
1	2	63	31.5	100	8.2
2	1	48.6	48.6	75	17
3	1	26.3	26.3	100	21.8
4	2	48.9	24.5	100	34.3
5	4	93.4	23.3	100	20.8

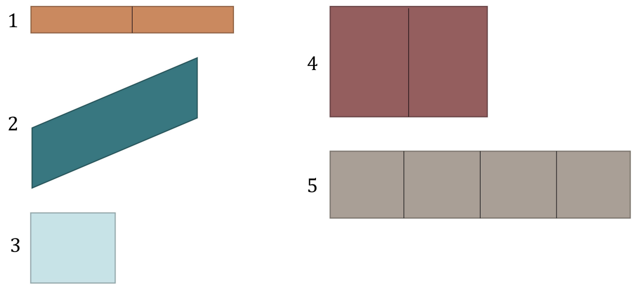


Figure 7.1: Schematic top view of five analysed bridges

Karamba3D

Appendix E describes all steps in the build-up of the model, and this section describes the general workflow and possibilities of the model.

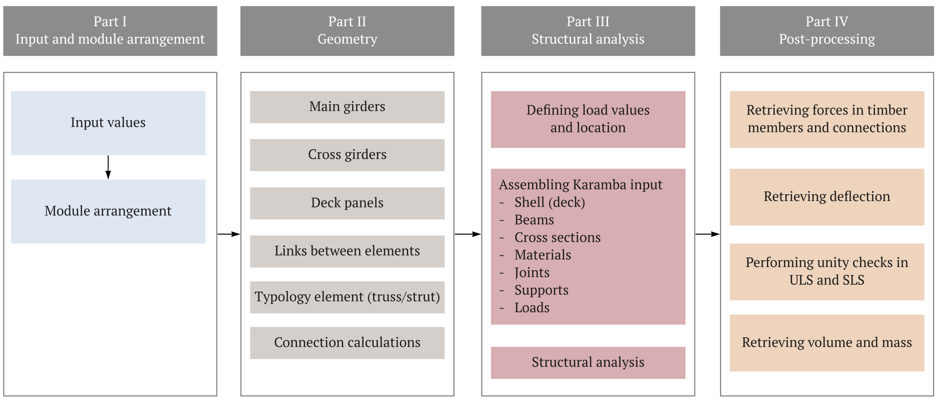


Figure 7.2: Workflow in the parametric model

A construction modelled in Karamba3D consists of lines, meshes and nodes. Joints in the structure can only be made between connected nodes. Therefore, lines and meshes must be split up into several nodes, connected with lines. These lines can be made infinite stiff, or a specific stiffness can be assigned to them. The same is done for nodes: degrees of freedom can be assigned to the nodes with complete freedom or a defined stiffness.

When all joints are modelled as stiff connections, the structure behaves stiff, but this is not according to the actual behaviour of the connections.The small lines between elements, which cover the difference in the elements' central axes, are modelled as infinite stiff beams. Therefore, no bending occurs in these beams, and the loads are directly transferred to the nodes. For the joints on both sides of these lines, the degrees of freedom and stiffnesses are used to make a realistic structural model. These stiffnesses are expressed in vectors, which represent the stiffness in either translation or rotation.

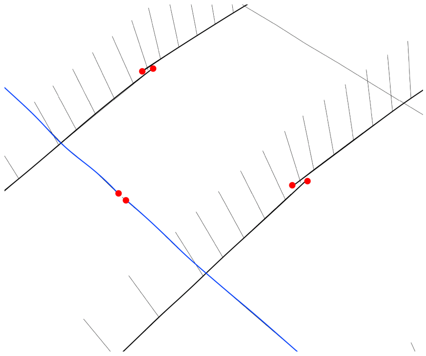


Figure 7.3: Node displacement around connections that are not infinitely stiff

Figure 7.3 illustrates the node displacement around connections with a specific stiffness. The displaced structure of the Karamba model is illustrated, exaggerated 100 times. In black, the main girders are displayed, and the cross girders are blue. The location of the connections is for both beams on the beam splice between modules. Both translational and rotational freedom are observed in the figure. Slip is observed, both in tension and in compression. It is concluded that the connections behave as expected.

The materials for the typology study are GL28h for the beams and CLT made from C24 for the deck. In Chapter 7.4, further analysis of the materials and their influence is performed. The material GL28h is put into Karamba easily, but for CLT, the material properties should be converted into orthotropic material properties. This calculation is performed in Appendix B.

7.2. Typology study

This section provides the first results of this research: the typology study. First, results from the parametric model are given, followed by an assessment of the typology. The assessment is performed using the TOM, as explained in the previous chapter. In conclusion, the answer to research question 2b is obtained.

The variant study on typology is performed for the five bridge configurations as stated in Table 7.3. The geometry, layout and materials of the timber members and connections are defined in Appendix E. The input values are defined in Appendix H. For comparison of the typologies, each typology is modelled for the five configurations. Cross-sections are adjusted to lead to a unity check close to, but maximum, 1.0 in SLS or ULS. The structural height and mass are compared. The definition of the structural height is presented in Figure 7.4.

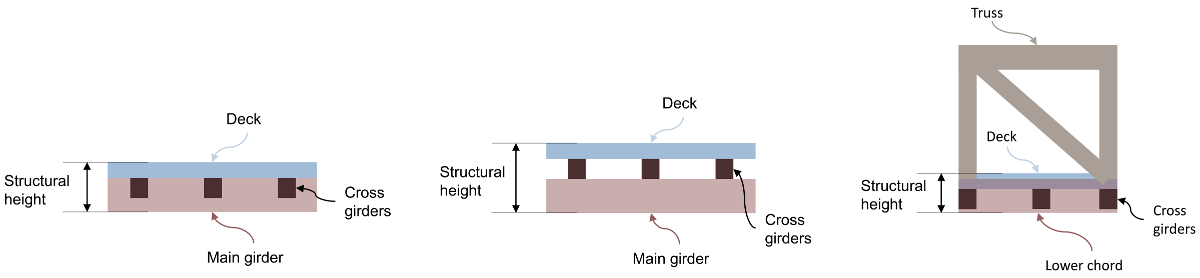


Figure 7.4: Definition of structural height

Results from the structural analyses define the efficiency. The efficiency is expressed in ratios to compare multiple bridges. Mass/area is a commonly applied ratio in bridge design, indicating the material efficiency. Furthermore, slenderness is often expressed in construction height/length. However, bridges with one or multiple free spans are considered. The profile of free space indicates the functional part of the bridge. The free height is kept constant, thus the profile of free width is used for functional span. Therefore, the slenderness ratio is converted to a height/free width ratio.

The definition of free width is defined in Figure 7.5. It is indicated that the structure does not benefit from the struts for four spans, as the middle spans are governing. For one or two spans, the free width does not decrease with the introduction of struts.

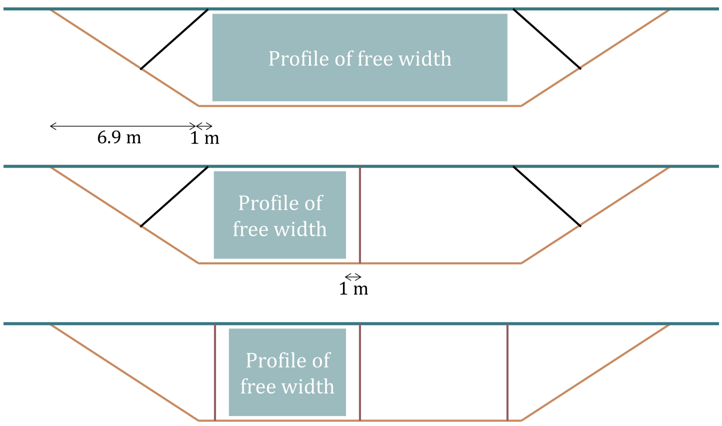


Figure 7.5: Definition of free width for multiple spans

7.2.1. Structural systems of explored options

The mechanical schemes of the three typologies are displayed in Figure 7.6 until 7.9.

Beam structure

The beam structure is the basis for all typologies. Figure 7.6 presents the mechanical scheme of the beam structure. Rotational springs are applied where the modules meet, simulating a beam splice in the main girders. The stiffness of this beam splice is defined in Appendix E.

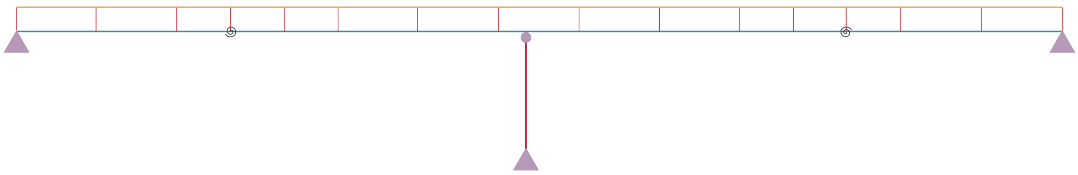


Figure 7.6: Mechanical scheme beam structure

Strut frame

The strut frame typology is applied to minimise the free span of the bridge. It is decided that the struts are only used at the outer spans. In that case, the profile of free space does not coincide with the struts. Struts can also be applied to the columns, but the columns should also be designed to withstand these forces. However, the substructure is not researched, and the strength of the columns can not be guaranteed. Therefore, struts are not applied to the columns.

A maximum horizontal distance is set for the struts to ensure that the struts do not coincide with the profile of free space. This distance is determined at 1.5+6.9=8.4 metres. The free space between the road and the foot of the talud is at least 1.5 metres, and the talud has a width of 6.9 metres. This calculation is accurate when a slope of 3:2 is applied with a free height of 4.6 metres. By doing this, the free span can be reduced by 8.4 metres on both sides.

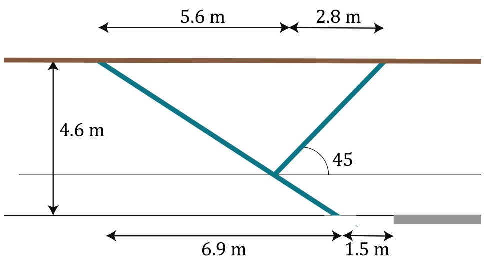


Figure 7.7: Geometry of strut structure

Figure 7.8 presents the mechanical scheme of the strut structure. Rotational springs are applied to the location where the modules meet, similar to the beam structure. The struts are modelled as two-forced members, having hinged connections at both outer ends.

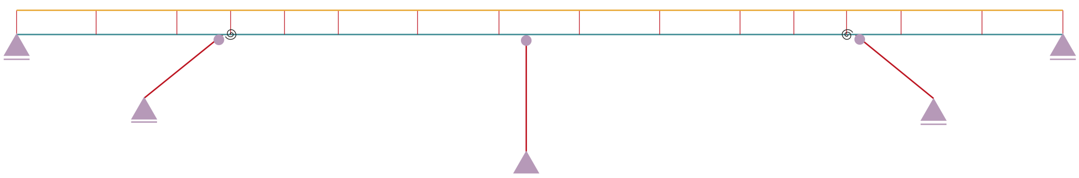


Figure 7.8: Mechanical scheme strut structure

Truss structure

When using a truss, a lot of the effective height of the construction can be placed above the deck. The main advantage of this is that the structure can be embedded more easily because the effective height below the deck does not have to increase compared to the existing concrete structure. The configuration of the truss and beams under the deck was described in the previous chapter.

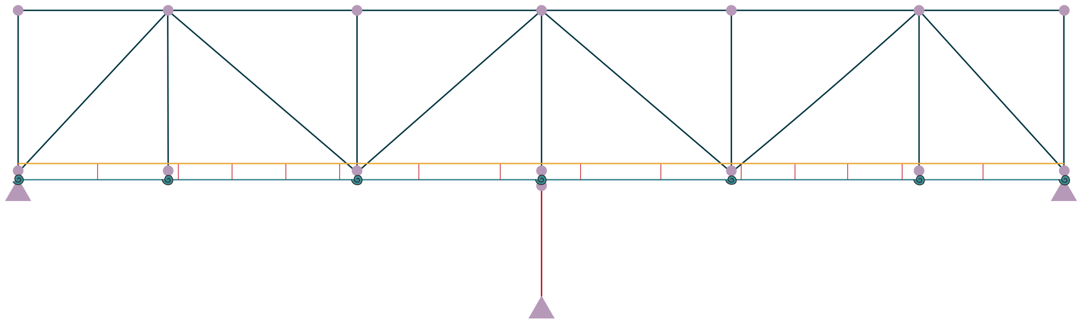


Figure 7.9: Mechanical scheme truss structure

7.2.2. Structural performance

The results, as presented in the following sections, are according to the bridges as illustrated in Figure 7.10. The properties of these bridges were mentioned in Section 7.1.2.

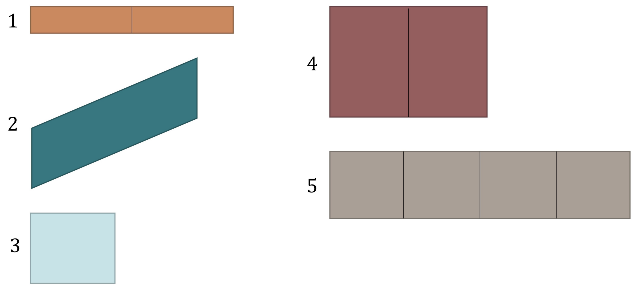


Figure 7.10: Schematic top view of five analysed bridges

Results from the analysis are visualised in Figures 7.11 until 7.13, the numerical values are stated in Appendix G. The ratios of mass/area are multiplied by the unity checks to make a distinction between the behaviour in SLS and ULS. In SLS, results for the global deflection of the system are presented. Therefore, the structural component with the largest deflection is governing. For configuration 1, the main girders are governing. For configuration 2, the cross girders are governing, following the deflection of the main girders, with additional local deflection due to axle loads. The truss typology is governed by the deflection of the cross girders. In ULS, unity checks for the main girders are presented for the beam and strut typology. The ULS behaviour of the truss typology is expressed in unity checks of the beams below the deck (cross girders), as they are governing. Especially for the truss typology, a significant difference is observed between the SLS and ULS behaviour.

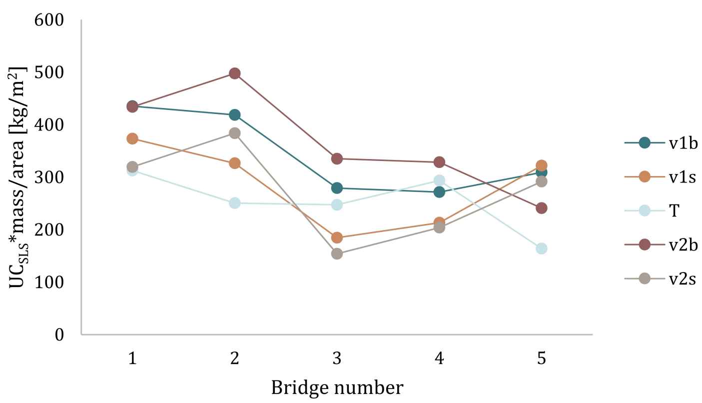


Figure 7.11: Mass-deflection behaviour, expressed in UC of the whole bridge in SLS * mass/area

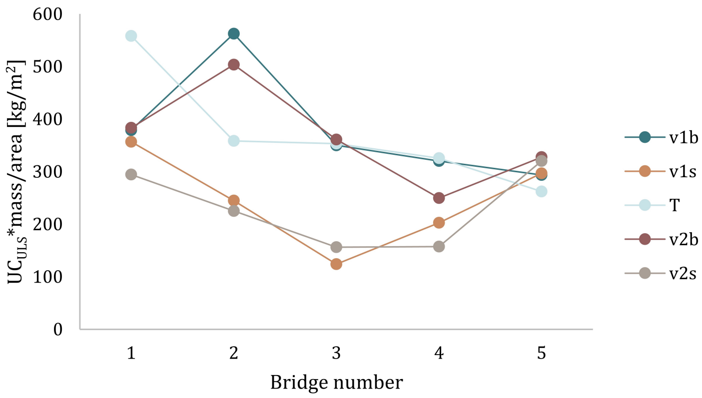


Figure 7.12: Mass-strength behaviour, expressed in UC of main girders in ULS * mass/area

The most remarkable results are explained and discussed. Bridge 3 in ULS performs the same as bridge 4 in mass/area, thus it can not be seen in the figure. For both ULS and SLS, it is indicated that the strut typology has the most efficient behaviour for almost all bridges within the design space, explained by the reduction of the free span. However, for bridge 5, the struts do not benefit the structural behaviour. Struts are only applied to the outer two spans, but the middle two spans are governing. Furthermore, the construction starts with a higher inclination as the geometry is not symmetrical on both sides of the support point. The conclusion can be drawn that the strut structure is more efficient for one or two free spans, but not for more than that.

Between configurations 1 and 2, no significant difference is observed in mass/area ratio. This is explained because configuration 1 has less composite behaviour (less stiff), but higher construction height. A higher construction height due to the cross girders increases the moment of inertia and thus the stiffness. Configuration 2 has the opposite benefits, and they cancel each other out for the amount of material required. The truss structure behaves well in SLS, but in ULS, the behaviour is less efficient. The latter is caused by high unity checks in the cross girders beneath the deck. However, for bridge 5, the truss behaves most material-efficient. This result confirms the assumption that truss bridges perform better on larger bridges.

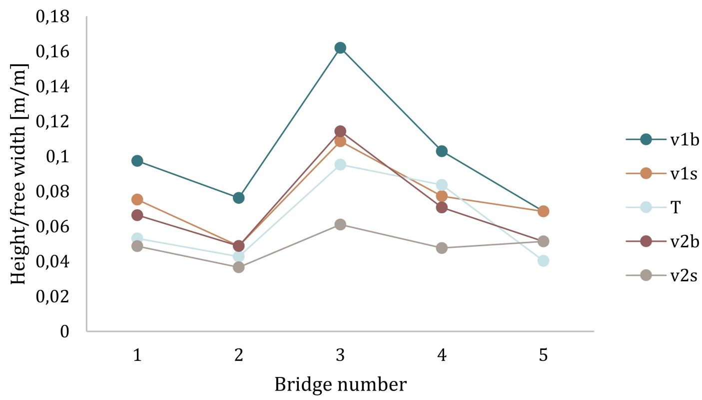


Figure 7.13: Construction height behaviour, expressed in height/free width ratio

Bridge 1 in v2s performs equal to bridge 4 on height/free width, hence it can not be seen in the figure. The truss structure performs well on height/free width ratio, especially for bridge 5 (94 metres length). This can be explained by the benefit of a truss structure for larger and multiple spans, as Crocetti (2014) stated. Additionally, trusses perform well in multiple-span bridges, as indicated by the difference in performance between bridges 4 and 5. Furthermore, for bridge 1, better behaviour is demonstrated for the truss structure than for the other typologies. This can be explained by the efficient behaviour for a structure with a small width. For both configurations 1 and 2, it is indicated that the strut typology performs better, explained by the reduction of the free span while maintaining the free width. Figure 7.13 clearly indicates that v2s has the lowest height/free width ratio. This behaviour is explained by two factors that benefit this ratio: the integrated cross beams and the struts reducing the free span.

7.2.3. Force distribution

For the five bridges as mentioned above, the force distributions in the main girders are obtained. The minimum and maximum values are taken and used as input for the trade-off matrix. The results are presented in Figures 7.14 until 7.16.

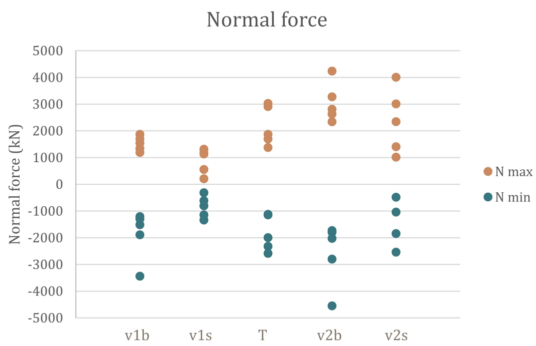


Figure 7.14: Maximum and minimum normal forces in main girders

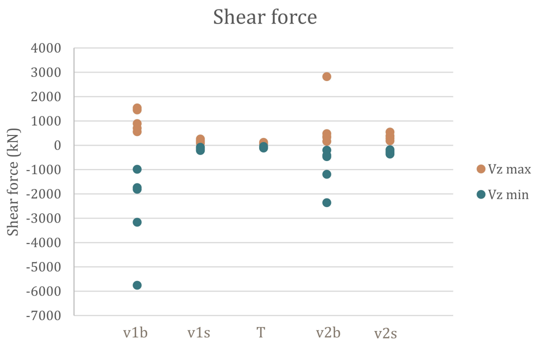


Figure 7.15: Maximum and minimum shear forces in main girders

A clear difference is observed between the normal force between configurations 1 and 2. The difference can be explained by the way the composite behaviour is modelled. As described before, the beams are modelled by lines, connected by links to other elements. More composite behaviour is assumed for configuration 2, as the deck and girders are connected over the entire length of the bridge. For configuration 1, composite behaviour is only generated through the cross girders, which is considered negligible. Therefore, a larger normal force is created for configuration 1, as it creates the bending moment together with the normal force in the deck. The normal forces in the strut typology are smaller than for the beam typology, again caused by the reduced free span.

For the shear force distribution, large shear forces are observed for the beam typology in configuration 1, caused by the concentrated distribution of forces between cross girders and main girders. These are transferred as shear forces to the main girders. The shear forces are relatively small for the other typologies, explained by the direct contact between the deck and beams, causing distributed force transfer.

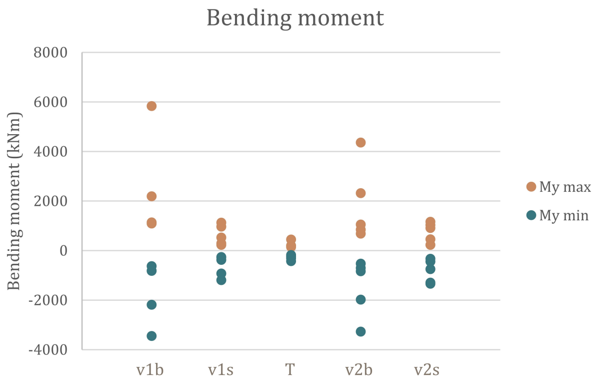


Figure 7.16: Maximum and minimum bending moments in main girders

The bending moment diagram contains a distribution as expected. The beam typologies have the highest bending moment due to the lowest stiffness and largest free span. The truss typology has a low bending moment, which is how a truss should behave, as most forces should be transferred as normal forces in the trusses. Configuration 2 has slightly smaller bending moments than 1, caused by the increased stiffness and composite behaviour.

7.2.4. Result

In Section 7.1.1, the performance indicators and weight factors of the trade-off are explained. In this section, the results of the analysis and the literature study are combined into a trade-off. Appendix G explains the scores in the TOM in detail, a summary is given in Table 7.4.

Table 7.4: Score factors of TOM

Category	Performance indicator	Weight	v1b	v1s	T	v2b	v2s
Effective	Mass - deflection	15	-	+	+ +	- -	+
	Mass - strength	15	- -	+	- -	- -	+ +
	Construction height	10	- -	-	+ +	-	+ +
Protected	Moisture	10	+ +	+ +	- -	+	+
Flexible	Widely applicable	10	- -	-	+	-	+ +
	Adaptability	15	+ +	+	- -	+ +	+
Demountable	Reaching connections	5	+ +	+ +	+	-	-
	Complexity	15	+ +	+	- -	+	-
	Connection rigidity	10	- -	+	+ +	-	+
Total			50	65	45	43	70

7.2.5. Conclusions

Research question 2b is answered by the trade-off as stated above:

Which structural typology is best suited within the defined design space?

From Table 7.4, the conclusion is drawn that option v2 with the strut frame (v2s) performs the best. The struts ensure that the free span is reduced, while maintaining the profile of free width. Furthermore, configuration 2 enables composite behaviour between the deck and main girders, which makes the structure stiffer. This system is also easy to adapt and widely applicable. The ventilation of the structure requires further research.

7.3. Standardisation

After the design is determined for the modules and a typology is chosen, the actual modular system should be defined. This chapter describes the applicability of the modules. Furthermore, the grade of demountability is determined, related to research question 2e.

7.3.1. Scope of the modular system

Figure 7.17 presents for all bridges in the design space the number of free spans and their length. The free spans are rounded to a multiple of five. The development of free spans in the decades is presented in Figure 7.18.

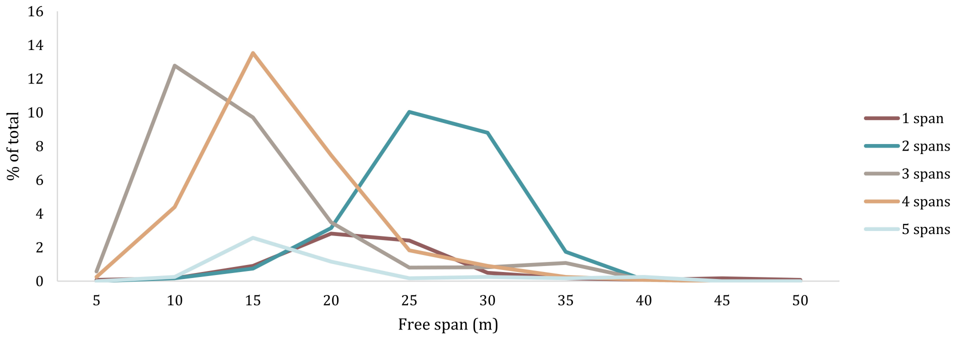


Figure 7.17: Relation number of spans and free span

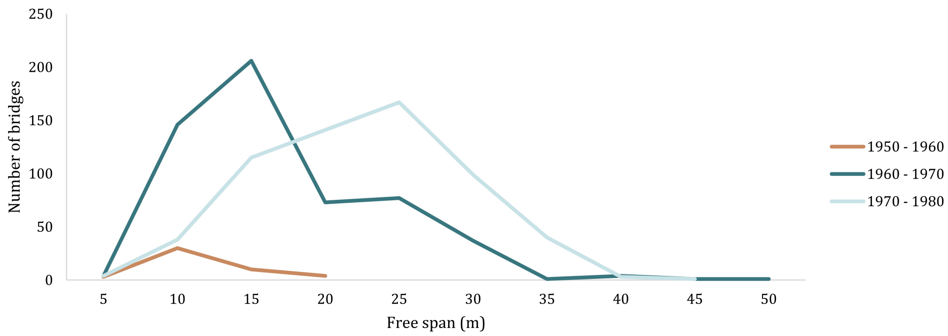


Figure 7.18: Relation decade and free span

With struts applied at the ends of the length of the bridge, only the outer two spans benefit from the struts. The results of Section 7.2 confirms this assumption. Standardisation must be applied to benefit from a modular system, resulting in economic benefits. For an efficient, standardised system, the difference in free span should not be too large. Therefore, a more narrow design space is defined.

The definition of a span is set to the distance between support points in the existing infrastructure. For example, for a bridge spanning between the abutments, one span is considered. However, the struts provide additional support points. As stated in the typology analysis, no reduction of profile of free space occurs. Therefore, the distance between existing support points is called the span in the coming chapters in the report.

Figure 7.17 indicates that bridges with a free span above 25 metres mostly have one or two spans. Furthermore, only a few bridges exist with free spans over 40 metres, and no bridges with more than one span exist over 45 metres. Furthermore, Figure 7.18 indicates that the bridges with a longer free span were constructed in the last considered decade. The older bridges, which probably require replacement first, have smaller spans. The first generation of timber highway bridges can replace the bridges with smaller spans. Afterwards, the spans of the bridges can increase, as more experience in timber bridges is obtained. This process is similar to the concrete bridge process in the 1950s.

The struts in this design cover approximately eight metres, and thus it is decided that for bridges with one or two spans below twenty metres, no struts are applied. Furthermore, bridges with three or more spans do not benefit from struts. Figure 7.19 that the tandem loads must be placed on the least favourable location, the part without struts.

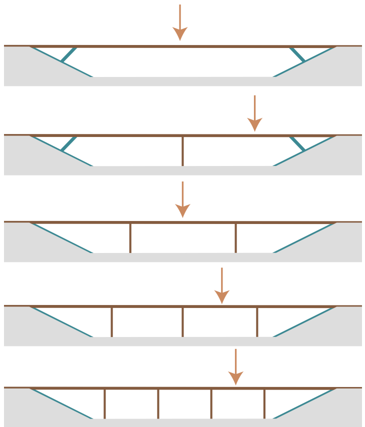


Figure 7.19: Rule of thumb for applying struts. The tandem loads are applied on the locations as indicated.

A structural analysis is made to define a module with standard cross-sections. A balance must be found between the range of bridges that can be covered modules while minimising the over-dimensioning of the bridges. This analysis is based on deflection requirements, as this is governing in most situations. All bridges stated in Figure 7.17 are analysed in Karamba for ULS and SLS. Since rounded spans are used, the upper limit is implemented into the model. For example, for bridges with a rounded span of twenty metres, 22 metres is modelled. The values for the unity checks are stated in Appendix I for a width of fifteen metres, which is the average. The cross-sections and variables in the model for this analysis are presented in Appendix H, Table H.1. The system is standardised for one cross-section. Further optimisation can be obtained when multiple standardised cross-sections are applied but considered in combination with an economic perspective.

Table 7.5 presents the structures with struts (s) and those without, classic beam structures (b). In orange, the lengths and spans that are within the standardised system are indicated.

Table 7.5: Scope of the modular system with application of struts or beam structure

	1	2	3	4	5
5	b	b	b	b	b
10	b	b	b	b	b
15	b	b	b	b	b
20	s	s	b	b	b
25	s	s	b	b	b
30	s	s	b	b	b
35	s	s	b	b	b
40	s	s	b	b	b
45	s	s	b	b	b
50	s	s	b	b	b

In Figures 7.20 and 7.21, the unity checks are displayed in a graph. Appendix I presents the data in a table. The sizes of the dots are according to the percentage of bridges with that span and the number of spans.

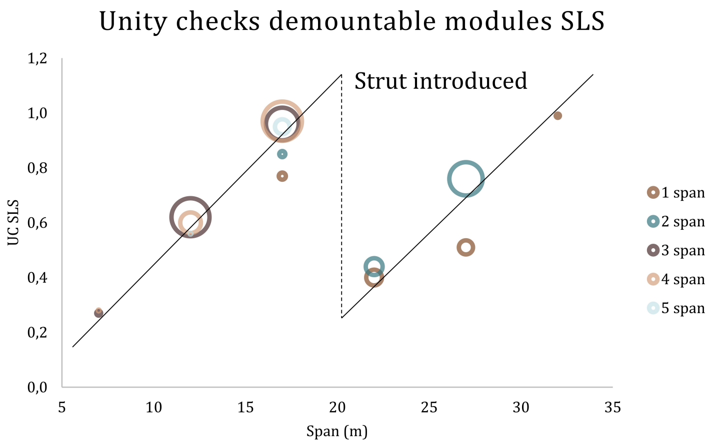


Figure 7.20: Unity checks for SLS with demountable modules

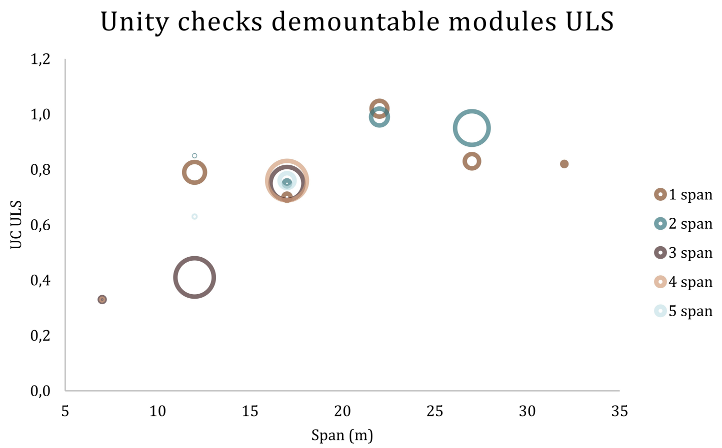


Figure 7.21: Unity checks for ULS with demountable modules

Figure 7.20 indicates that SLS requirements govern. With the spans included in the system, the most occurring bridges have an efficient structure: UC close to 1. A significant jump is observed in SLS results between 17 metres and 22 metres span, which is explained by applying struts from 22 metres on. The free span is reduced by 16 metres, and thus the deflection is reduced significantly. In the ULS results, no significant jumps or deviations are observed. For ULS, the main girders are governing in the combination of bending moment and tension, with and without struts.

When the standardised modules are designed to function in larger spans, the cross-sections have to be enlarged, and thus more material is used in the modules. As the spans of 35 metres and higher do not occur often in the database, these should not be governing. To conclude, the structures are included for the modular system (orange) and with or without struts (Table 7.5). Up to a rounded span of 30 metres is included, thus a span of 32 metres can be obtained. However, due to the standardised system, a span of 32 metres results in a bridge with a length of 35 metres. In conclusion, a maximum free span of 35 metres is stated. 67% of the bridges from Figure 7.17 are included for the modules, derived from Table I.3. From the acreage of Rijkswaterstaat from 1950-1980, 58% is included.

7.3.2. Grade of demountability

In the previous section, a modular system is developed, where several options remain within this system. The bridge system is build up from modules, which consist of main girders, cross girders and a deck panel. The grade of demountability highly influences the level of circularity: depending on the applied connections (van Vliet et al., 2019). When the connections are easy to demount, high circularity is obtained. On the other hand, when the grade of demountability is lower, more efficient connections are used in terms of stiffness and strength. For example, glue is a very stiff connection for timber structures but can not be demounted. In this module design, the connection that mainly influences the grade of demountability is the connection between the deck and main girders. As stated in the literature review, three options arise for this connection: dowels, inclined screws or glue.

The three options for the deck-main girder connection are presented in Figure 7.22, with increasing stiffness from left to right. It was already mentioned in Section 5.3 that screwed and glued connections are not demountable. In theory, the dowelled connection can be performed in a demountable manner. However, a steel dowel yields when high forces are transferred. Consequently, deformations occur, and it would be hard to demount the dowels. This should be included in considering the grade of demountability.

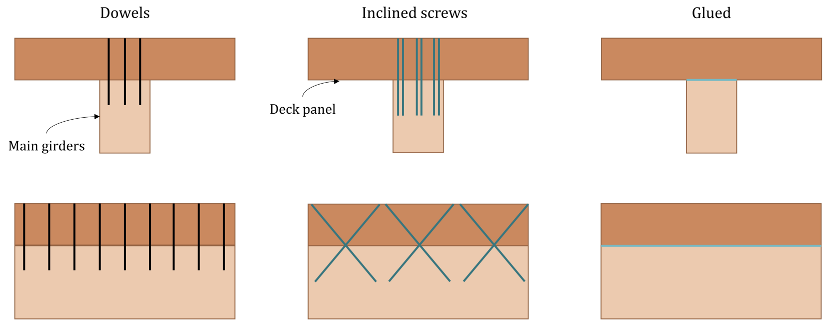


Figure 7.22: Options for connection deck-main girder

An analysis should be performed to assess the difference in behaviour and efficiency of the bridge. As described in the previous section, the standard element is based on a span of 35 metres, as stated in Table H.5. When this structure is considered again, an analysis can be made for a dowelled connection (demountable), a connection with inclined screws (fixed) and a glued connection (fixed). With glued connections, one can assume that the glue is stronger than the timber, so the connection is infinite stiff, and no slip occurs.

First, an analysis is performed where the cross-sections and other connections are kept constant, and only the slip between the deck and main girders varies. The cross-sections used are presented in Table H.1 in Appendix H.

The calculation of these slip moduli is performed in Appendix I, and the result is presented in Table 7.6. For the dowelled connection, six dowels are applied per link in the model, which is the number allowed according to the minimum centre-to-centre distances of EC1995-1-1. For the inclined screws, three pairs of inclined screws are applied over the width of the main girder, as this results in a favourable result for both SLS and ULS unity checks.

Table 7.6: Unity checks of three connection options, with constant cross-sections

Connection	Slip modulus	SLS	ULS deck	ULS MG	ULS CG
Dowelled	6*10295 N/mm	0.99	0.51	0.71	0.78
Inclined screws	6*31176 N/mm	0.86	0.53	0.90	0.38
Glued	Infinite stiff	0.74	0.61	0.92	0.31

As presented in Table 7.6, a higher slip modulus results in better performance in SLS (stiffer) but slightly worse performance in ULS. This is caused by the increased cooperation between the main girders and the deck. As a result of this, larger tension stresses occur in the beams. Both for the glued and screwed connection, a combination of tension and bending is governing in the main girders. The literature review stated that the glued and screwed connection scored equally on the performance criteria. The screwed connection has the lowest unity check when SLS and ULS are considered, thus this connection is chosen for the deck panel-main girder connection. Cross-sections are not adjusted as the unity checks are desired, and changes will occur in the other connections, as described in Section 7.6.

The cross-sections for the modules with a screwed connection are based on Table H.1. For the new structural analysis, only the connection has changed, resulting in the variables presented in Table H.3, the unity checks as presented in Figures 7.23 and 7.24 are the result of this structural analysis.

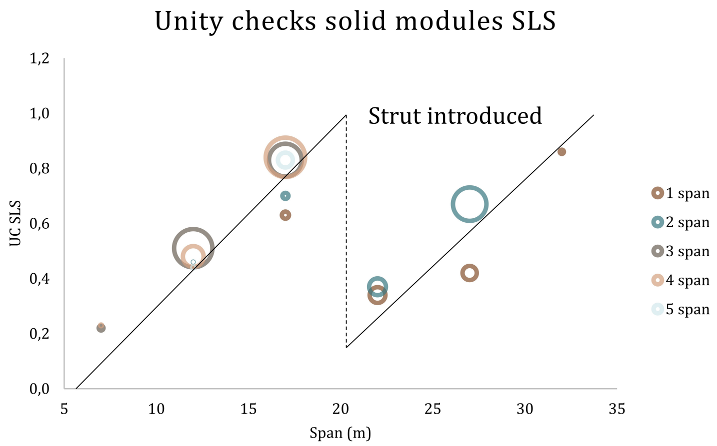


Figure 7.23: Unity checks for SLS with solid modules

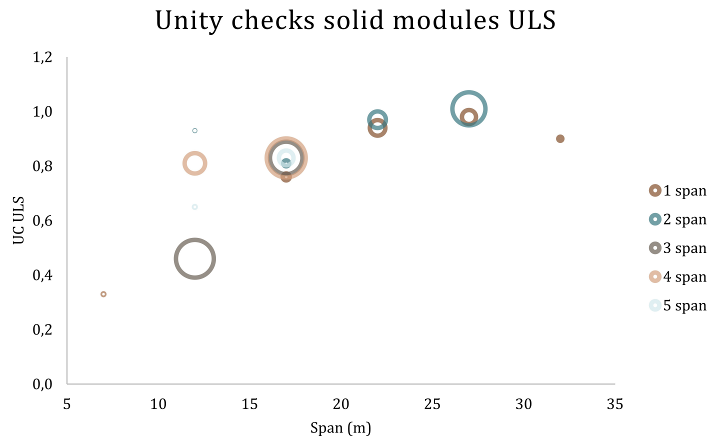


Figure 7.24: Unity checks for ULS with solid modules

7.3.3. Application in practice

This research focuses on an alternative on highway bridges to be used in the replacement task of Rijkswaterstaat. However, the timber bridges are also suited for application in new-build bridges. This section illustrates the required modules for a bridge on the most often applied configurations of roads. First, the modules required for the underpasses is stated, followed by the modules required for the road on top of the bridge. The visual representation of this data is presented in Appendix I.

Table 7.7: Modules required for configuration of roads beneath the highway bridge

Lanes	Required length	Modules	Number of spans	Application of struts
2x1	9.6 m	L4 L2 L4	1	No
2x1 with bike lanes	14 m	L1	1	No
2x1 with taluds	21.4 m	L3 L1 L3	1	Yes
2x2 with middle verge	19.5 m	L4 L1 L4	1	No
2x2 with middle verge and taluds	28.8 m	L3 L3 L1 L3 L3	1	Yes
2x3 with middle verge	24.4 m	L3 L1 L3	1	Yes
2x3 with middle verge and taluds	36.2 m	L3 L2 L1 L2 L3	2	Yes

Table 7.8: Modules required for configuration of roads beneath the highway bridge

Lanes	Required width	Modules
2x1	9.6 m	B2 2*B1 B2
2x1 with bike lanes	14 m	4*B1
2x2 with middle verge	19.5 m	B2 5*B1 B2
2x3 with middle verge	24.4 m	B2 7*B1 B2

7.3.4. Conclusions

Consequently, research question 2c is answered:

How can standardisation be applied in the context of replacement of existing highway bridges?

Standardisation on module size is defined in Chapter 6: eight modules are defined, which can be used as stock products. This section defined the application of standardised modules in the context of Dutch highway bridges. With the defined scope of the standardised system, 67% of the bridges in the design space are included. This covers 58% of the bridge stock of Rijkswaterstaat. Consequently, bridges with a span up to 35 metres can be constructed. Struts are applied for free spans above twenty metres. This study defines one standardised module, which can be applied to the bridges in the scope. However, when the cross-section dimensions of the modules are varied, more efficient behaviour can be obtained. A study into the benefits of standardisation and material efficiency is required, including the financial aspect of the bridges.

The second aspect in standardisation is the grade to which the bridge is demountable. This is determined by the connection between the main girders and the deck. Three options are considered: dowelled, inclined screwed and glued. In conclusion, the connection with inclined screws is best suited. The most efficient structural performance is obtained, and a reliable connection is obtained. However, screws are not demountable, and therefore the modules are not demountable. Consequently, modules are used as prefabricated building blocks, which can be mounted and demounted to each other.

7.4. Timber products

This chapter elaborates on the choice of timber products, related to research question 2d. In the study phase, the options for timber products were explored, with their environmental impact. To quantify the effects of different materials, one bridge is chosen to analyse. As presented in Table 7.5, the structure with one span of 35 metres is the largest within the modular system. This construction is worked out further in this chapter. First, the optimal beam material is chosen, followed by the deck material analysis. For all calculations, the values for the variables as presented in Table H.3 are used unless stated differently.

7.4.1. Beam material analysis

As described in the literature review, three beam materials were considered: sawn timber, glulam and LVL. The information on environmental impact and production process is considered in the decision for the beam material, combined with the structural analysis as performed in this section. As sawn timber is restricted to the sizes of the tree, cross-sections can not be made as large as needed, and imperfections can not be removed. Therefore, sawn timber is not considered further for the beam material. The materials that are considered are GL24h, GL28h, GL32h and LVL-S.

The structural analysis is performed in the global model in Karamba3D. For the deck material, CLT is applied, similar to the typology study. The choice of deck material is performed after the choice for beam material. The mechanical properties used are presented in Table B.1.

Results Karamba3D

A structural analysis is performed with the dimensions as mentioned in Table H.3. The stiffness properties used are stated in Table 7.9.

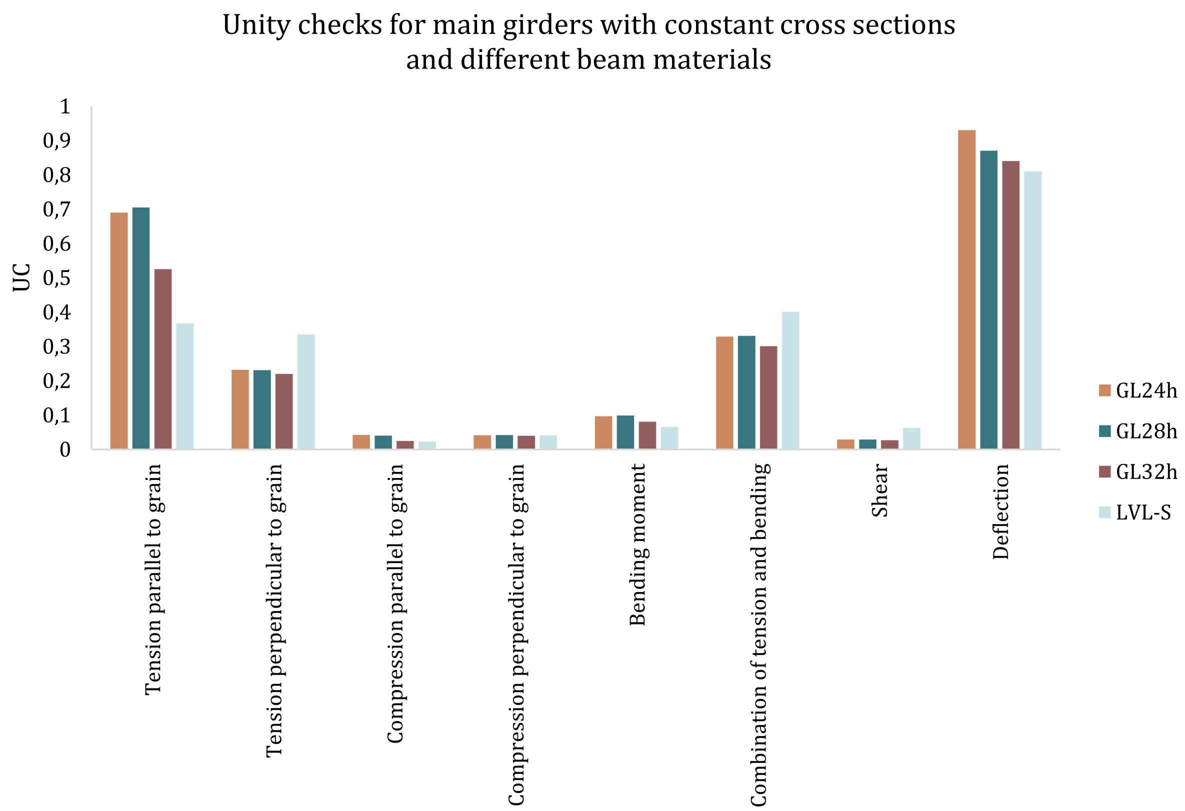


Figure 7.25: Comparison of unity checks for beam materials

Table 7.9: Stiffness properties beam material analysis

Stiffness properties	GL24h	GL28h	GL32h	LVL-S
E1 [N/mm$_2$]	11500	12600	14200	13800
E2 [N/mm$_2$]	300	300	300	0
G12 [N/mm$_2$]	650	650	650	600
G13 [N/mm$_2$]	65	65	65	460
G23 [N/mm$_2$]	65	65	65	0

The result of this analysis for the main girders are presented in Figure 7.25, the data is listed in Table J.1. It is concluded that SLS requirements govern the design of the beams. Therefore, the Young's modulus of the material is governing for the dimensioning of the members. When all material variants are set to obtain a unity check in SLS of 0.95, the cross-sections as presented in Table 7.10 are applied. The price is the shadow price for a beam of 1-metre length, with the area required. No separate environmental prices are found for different grades of glulam, so the same shadow price is applied. The density differs between the three types of glulam, which influences the capture of carbon during growth. However, as this is included as negative impact (A1) and as positive impact (C3), this difference is cancelled out.

Table 7.10: Environmental costs beam materials

	GL24h	GL28h	GL32h	LVL-S
h (m)	0.71	0.68	0.66	0.64
b (m)	0.4	0.4	0.4	0.4
A (m^2)	0.284	0.272	0.264	0.256
Shadow price per m^3	€29.59	€29.59	€29.59	€29.82
Environmental costs per metre beam	€8.40	€8.05	€7.81	€7.63

The difference in cross-sections is limited, the height only differs some centimetres. A similar result is also observed for the environmental costs, which are close to each other. Therefore, at this moment, no significant proof is found to use either glulam or LVL. In Europe, LVL is more expensive: the price is approximately 40% higher than for GL28h, according to experienced timber engineers at Arup. However, costs change over time, depending on demand and supply, and as costs are not in the scope of this research, one can make no concise decision. Still, glulam production is larger in Europe and, at the moment, less expensive than LVL. Based on this, glulam is chosen as building material.

Within the glulam products, three strength grades are researched. GL32h is slightly stiffer compared to GL24h and GL28h. However, GL28h is applied most often nowadays, thus production occurs on a larger scale. Furthermore, for the circular system, it is desired to work with commonly used materials, as it is more likely to be used in another construction. In conclusion, GL28h is used as the beam material. To make a well-substantiated choice on the beam material, more research should be performed into fatigue behaviour, long-term degradation due to moisture and temperature, and costs.

7.4.2. Deck material analysis

As described in the literature review, five types of deck materials can be applied: CLT, LVL-S, LVL-X, SLT (from sawn wood C24 or glulam GL28h). The structural behaviour is derived from the stiffness of the materials that is put into the model.

Cross-laminated materials

The derivation of the equivalent stiffness of cross-laminated materials is presented in Appendix B.2. Usually, the stiffness of a material is given, and the moment of inertia varies for both directions. However, Karamba 3D calculates the moment of inertia automatically according to the solid cross-section, the stiffness should be adjusted. Therefore, the equivalent material stiffness of E and G in both directions is derived, both for panels with nine layers. The equivalent material stiffness as used in the model is presented in Table 7.11.

Table 7.11: Deck material properties as input for Karamba analysis in N/mm^2

Property	CLT	LVL-X	LVL-S	SLT C24	SLT GL28h
$E_{ef,strong}$	$9.753 \cdot 10^3$	$1.241 \cdot 10^4$	$1.38 \cdot 10^4$	$1.10 \cdot 10^4$	$1.26 \cdot 10^4$
$E_{ef,weak}$	$3.807 \cdot 10^3$	$4.547 \cdot 10^3$	0	$3.70 \cdot 10^2$	$3.00 \cdot 10^2$
$G_{ef,strong}$	$1.562 \cdot 10^2$	$1.004 \cdot 10^2$	$4.6 \cdot 10^2$	$6.9 \cdot 10^2$	$6.5 \cdot 10^2$
$G_{ef,weak}$	$6.649 \cdot 10^1$	$3.764 \cdot 10^1$	$1 \cdot 10^1$	$6.9 \cdot 10^1$	$6.5 \cdot 10^1$

Stress laminated materials

The literature review presented information on the three-dimensional modelling of stress laminated timber (SLT). It is stated that for an accurate analysis, non-linear modelling is required. However, Karamba3D can only model linearly, and a non-linear analysis is out of the scope of this research. To transfer the deflection from the linear model to a non-linear approximation, Figure 5.9 is used, where pre-stressing of 600 kPa is assumed. With this assumption, the SLT deck is assumed to work as a solid panel, thus assuming a large enough pre-stressing force. When a bridge is realised with an SLT deck, the failure modes, as stated in Figure 5.8, should be checked, and the prestressing should be verified.

Results

A structural analysis is performed with the dimensions as mentioned in Table H.3. Only the material properties of the deck material are varied. Figures 7.27 and 7.26 present the structural results, expressed as unity checks for the deck. The unity check in SLS is based on global deflection. The data is listed in Appendix J.

Figure 5.14 stated that the environmental impact for CLT, glulam and LVL is almost similar, thus no distinction is made on environmental level. Figure 7.26 shows that the SLS unity check are governing. CLT performs best in SLS, which is explained by the relatively high stiffness in both directions. CLT is followed by LVL-X, again a material that is cross-laminated. The shear modulus of LVL is significantly lower than CLT, and thus more deflection occurs with the same thickness. Figure 7.27 indicates significant differences between the unity checks in ULS, but all unity checks stay within limits. LVL costs are significantly higher than CLT, thus CLT is more favourable from an economic point of view. Therefore, CLT is applied in all deck parts.

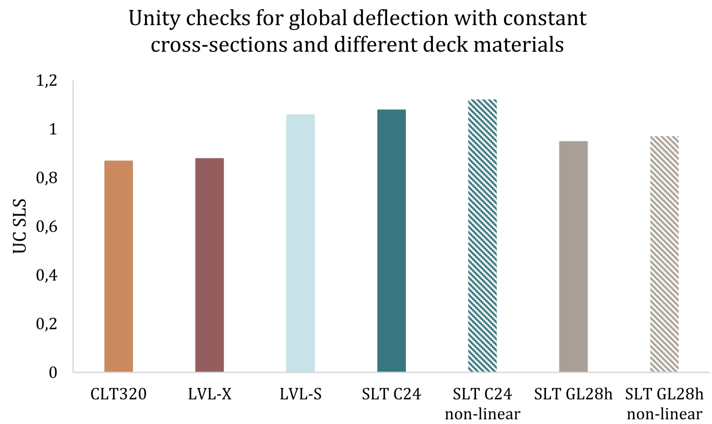


Figure 7.26: Comparison of unity checks for deflection

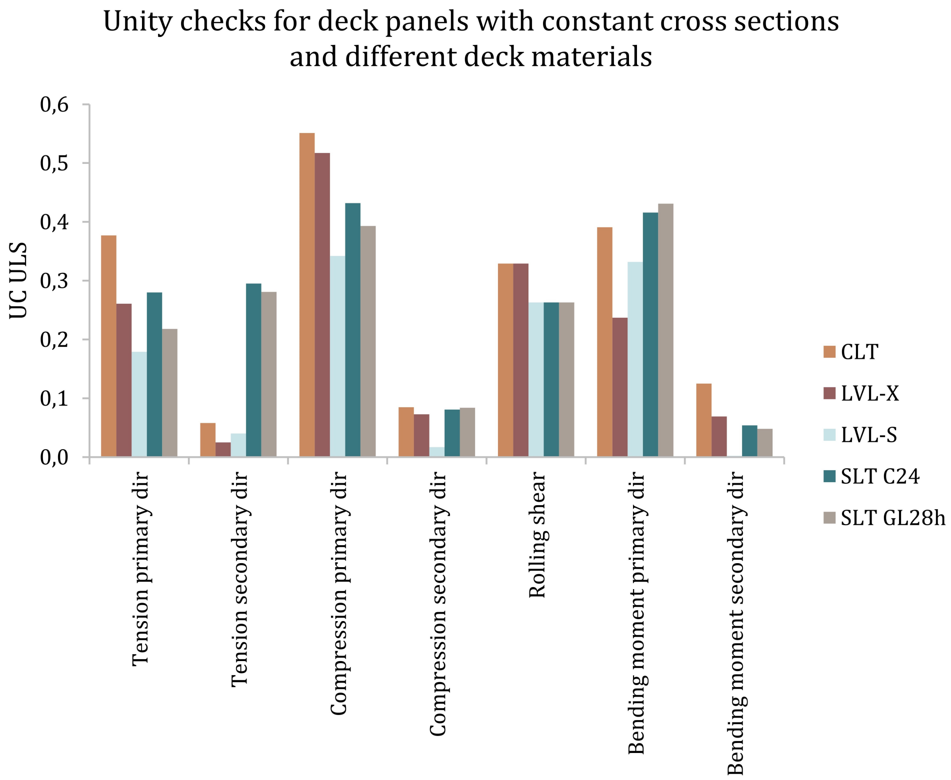


Figure 7.27: Comparison of unity checks for strength

7.4.3. Conclusions

This section focused on research question 2d:

Which engineered timber products result in the most material-efficient structure?

Sawn wood is not suited for beam elements for a bridge on this scale, as the dimensions are not large enough. Three types of glulam (GL24h, GL28h and GL32h) and LVL were studied in a structural analysis. Differences in both SLS and ULS behaviour are found, but no substantiated choice can be made with the analyses in the scope of this research. An indication of a higher price of 40% and lower production in Europe is used for LVL. Consequently, glulam is chosen as beam material for this research, but further research is required. Additional research should be performed into the long-term behaviour of the timber products, including fatigue, moisture and temperature influences.

A clear difference is observed in SLS results for the deck materials. The most material-efficient behaviour is observed for CLT, with the lowest unity check in SLS. Therefore, CLT is the most favourable choice for the deck material, considering structural behaviour, costs and production possibilities.

7.5. Connection design

This section focuses on research question 2e. Figure J.1 presents an exploded view of the modules, indicating the connections in the bridge. These connections are design and calculated in this section. The next section describes the final design of the timber members, using the connections as determined here.

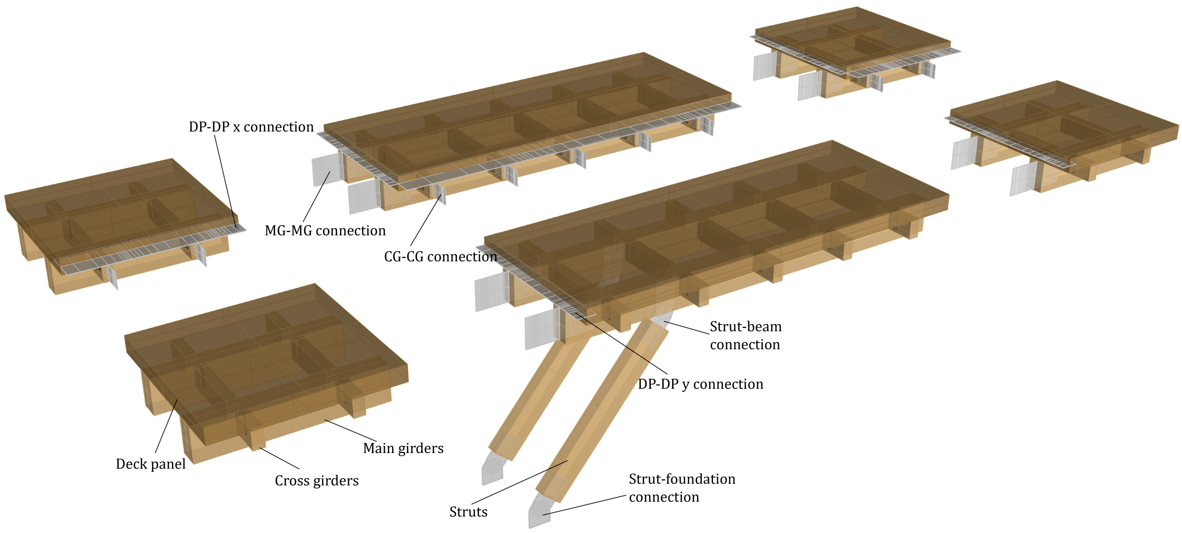


Figure 7.28: Location of connections

Appendix J describes the connections one by one. The calculation of the connection is provided, with the values for the forces in the connections at that point. As a basis, the values of Appendix H are applied and changed in the order of the report. In this section, an overview of the structure and its connection is given. Furthermore, an overview of the unity checks is provided. The technical drawings in this report can be found on scale in Appendix K.

Figure 7.29 illustrates a side view of the governing bridge for this design (35 metres in length).

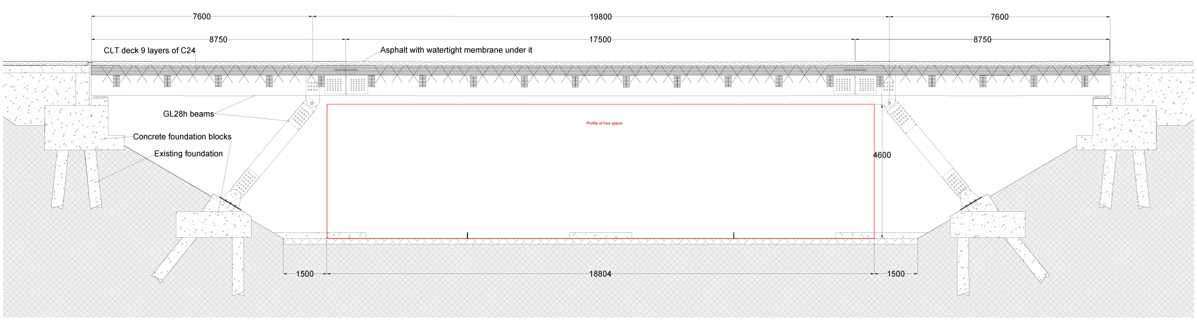


Figure 7.29: Side view of bridge

Figure 7.30 illustrates the connection between the struts and the main girders. This is performed with a pin connection, where the steel plates are attached to the timber by bolts. Due to the high normal forces in the beams, a high number of bolts is required.

For the bridge of 35 metres, the beam splice is located close to the location where the bending moment is zero. However, the beam splice connection is crucial for the load-bearing behaviour of the bridge. As the location of the beam splices differ for each bridge, the connection is designed to 50% of the beam's capacity. This requirement is used for steel connections, and ensures a more redundant design. The beam splice connection is executed as a slotted-in steel plate connection, assembled with bolts.

Furthermore, the connection between the deck and main girders is presented in the figure, executed with inclined screws. An angle of 35 degrees is chosen, to satisfy the strength requirements, and still obtain the desired stiffness. A high number of screws is applied, which can be optimised further.

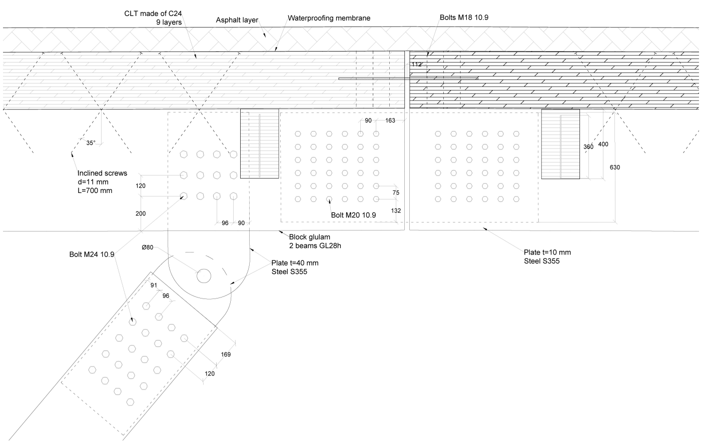


Figure 7.30: Section of connection strut to main girders. Also illustrates the connection between the main girders and the deck

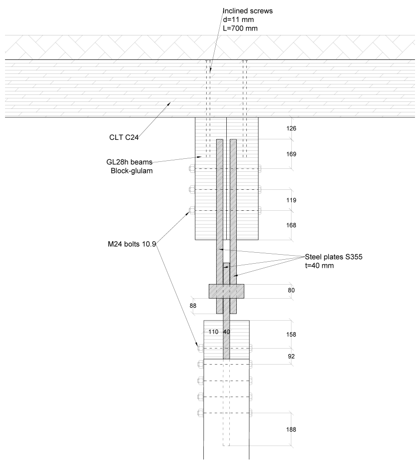


Figure 7.31: Side view of connection strut to main girders

On the other side of the strut, the pin connection is used again and attached to a concrete block in the substructure. As the substructure is out of the scope of this research, no calculations are performed for this attachment to the foundation.

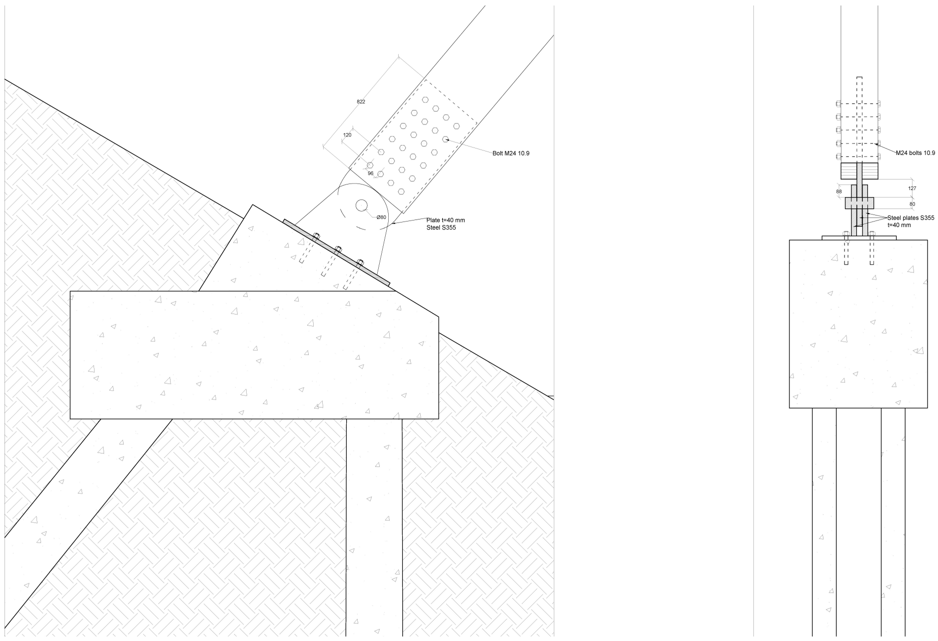


Figure 7.32: Connection of strut to the substructure

For the attachment to the abutments, a design is made which is based on reference projects. The design of Arup is used, together with bridges from Miebach. Figure 7.33 presents the preliminary design for the attachment to the abutments, including a void for movements over time.

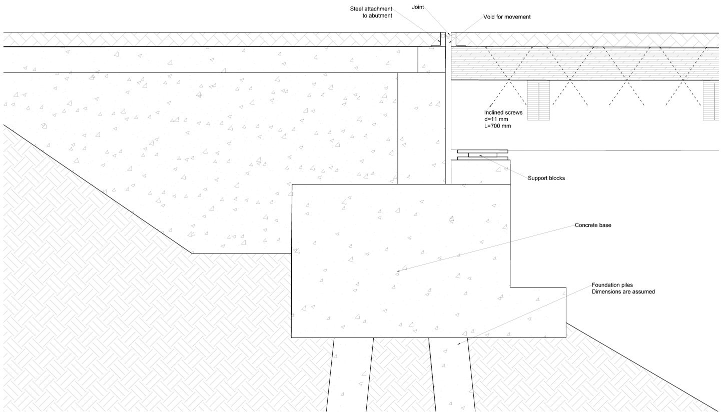


Figure 7.33: Connection to abutment

Lastly, Figure 7.34 displays a section in the global y-direction. The connection between cross girders is illustrated, as well as the connection of the cross girders to the main girders. Furthermore, the connection between deck panels is presented.

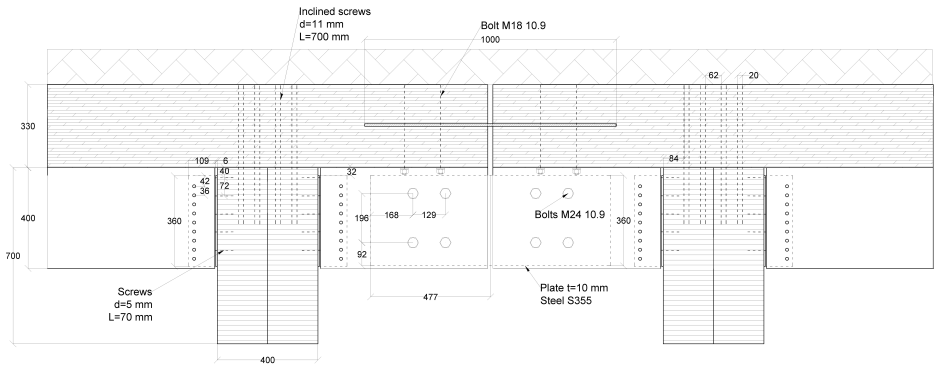


Figure 7.34: Connection of cross girders to cross girders and cross girders to main girders. Joint between deck panels in the y-direction is presented as well.

7.6. Final design

This section elaborates on the final design of the modular bridge system.

7.6.1. Global behaviour

As a result of the connection design, the structural behaviour of the timber elements changes as well. The dimension of the main girder is adjusted to 700 * 400 mm, to satisfy deflection requirements. A unity check of 0.92 is obtained for SLS. In conclusion, the variables as expressed in Table H.4 are used. Appendix J presents the check of all timber members and connections. The appendix also explains the calculation of the unity checks. Table 7.12 until 7.14 present the occurring stresses, the allowed stresses and the resulting unity check. All calculations are based on the equations in Appendix J, and are automated in the Karamba3D model.

Main girders

Table 7.12: Unity checks for main girders

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	12.10	17.8	0.68
Tension in transverse direction	0.06	0.38	0.17
Compression in longitudinal direction	1.34	22.4	0.06
Compression in transverse direction	0.08	2.0	0.04
Bending	2.24	22.4	0.10
Shear	0.06	2.8	0.02
Combined bending and tension	-	-	0.78
Combined bending and compression	-	-	0.10

Cross girders

Table 7.13: Unity checks for cross girders

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	4.27	17.8	0.24
Tension in transverse direction	0.14	0.38	0.38
Compression in longitudinal direction	0.67	22.4	0.03
Compression in transverse direction	0.16	2.0	0.08
Bending	1.34	22.4	0.06
Shear	0.45	2.8	0.16
Combined bending and tension	-	-	0.30
Combined bending and compression	-	-	0.06

Struts

Table 7.14: Unity checks for struts

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	-	-	-
Tension in transverse direction	0.02	0.38	0.04
Compression in longitudinal direction	9.86	22.4	0.44
Compression in transverse direction	0.02	2.0	0.01
Shear	0.11	2.8	0.04

Deck

Table 7.15: Unity checks for deck

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	4.26	11.2	0.38
Tension in transverse direction	2.83	11.2	0.25
Compression in longitudinal direction	9.07	16.8	0.54
Compression in transverse direction	3.02	16.8	0.18
Bending in longitudinal direction	8.45	19.2	0.44
Bending in transverse direction	4.22	19.2	0.22
Rolling shear	0.66	2	0.33

Expansion due to temperature differences

The expansion of timber due to temperature differences should be considered. With large expansion, high concentrated forces are induced on fasteners, which can lead to failure. The execution phase is not included in this research, but an estimation is provided for the expansion.

The expansion coefficient for timber parallel to the grain is $5 \cdot 10^{-6}$ per °C. With a length of 35 metres and an assumed temperature difference of 40°Cin the Netherlands, 7 mm expansion is obtained. This is a small expansion in comparison to the length of the bridge. Therefore, it is assumed that with the allowance for movement at the abutments, the expansion does not cause problems.

7.6.2. Camber

Design codes for bridges assume a camber to be applied. By applying an upward curvature, the deflection of the bridge is compensated. The upward curvature ensures that the beams do not have downward curvature due to deflection. Consequently, camber improves both aesthetics and function. The bridge looks more robust (aesthetics), and the profile of free space is not reduced (function).

Camber is determined by deflection, thus mainly influenced by the span of the bridge. Spans between 5 and 35 metres are included in the standardised system. Therefore, the camber must also be standardised.

Appendix I presents the calculation of the required camber, the cross-sections of the standardised modules are used, as presented in Table H.3. The required camber is calculated for all bridges in the selection of the modular system. Figure I.8 presents the results of the calculation. The percentage of bridges with a specific rounded free span in the acreage of Rijkswaterstaat is represented in the size of the dots. Within the system, a minimum radius of 1981 metres is obtained. For ease of calculation, the radius is rounded to 2000 metres.

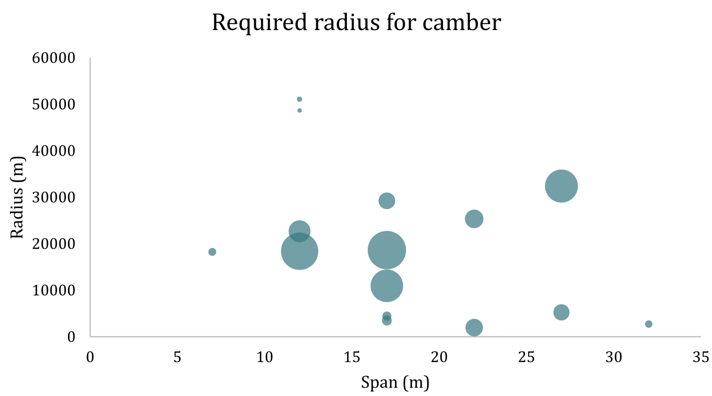


Figure 7.35: Required radius for bridges in standardised system

The camber generates a height difference of 19.3 mm for the module of 17.5 metre in length. Glulam can easily be produced with curvature, but for CLT, this is not common practice yet. The difference in stiffness between the glulam beams and CLT deck, and therefore deflection, can be used to realise this camber. The deflection due to the self-weight of the CLT deck is 37.38 mm, larger than the camber applied. The CLT panel will not deflect more than the camber, as it touches the glulam beams and is screwed to that. The execution of the connection process is presented in Figure 7.36.

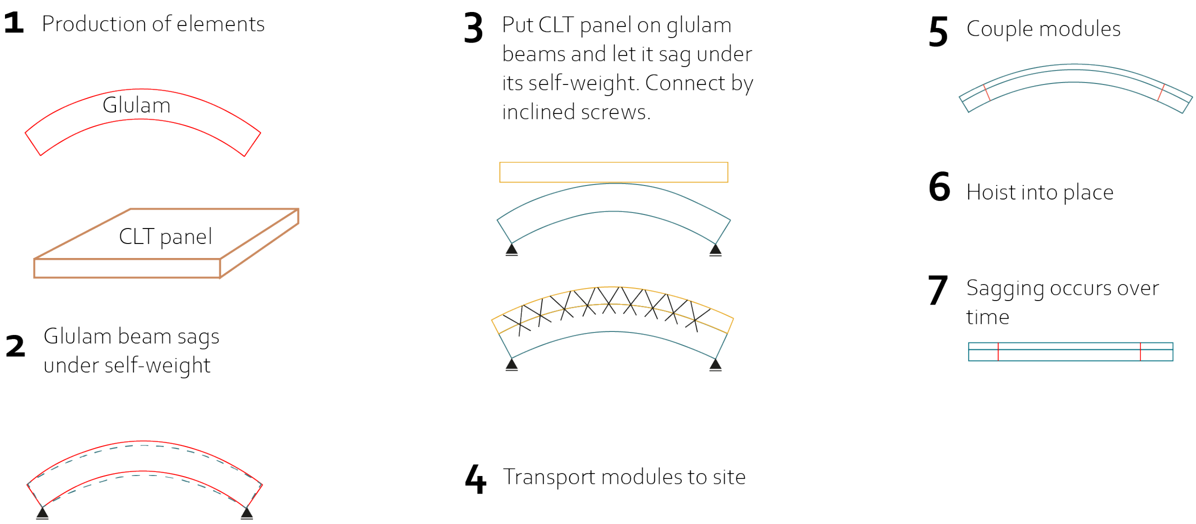


Figure 7.36: Phasing of applying camber

IV

Results and final remarks

8 Carbon footprint

This section elaborates on the environmental impact of the timber bridge compared to the concrete bridge to come to an answer to research question 3b. Rijkswaterstaat constructed a circular highway bridge in 2019, made in concrete (Figure 8.1). NIBE Research bv (2019a) performed a Life Cycle Assessment (LCA), quantifying the environmental impact of the circular concrete bridge compared to a traditional concrete bridge. This traditional (non-circular) bridge is also included in this comparative study.

First, the factors of influence on the outcome of the environmental impact are stated. Second, the comparison on environmental impact is elaborated.

Figure 8.1: Circular concrete highway bridge (Rijkswaterstaat, 2019c)

8.1. Influence factors on environmental impact

The scope and boundary conditions for a comparative environmental impact study are essential to perform an objective comparison. This section defines the most critical factors in the comparison and defines the values used in the comparison. As stated in the methodology, a comparison is made to the traditional and circular concrete bridge of Rijkswaterstaat, for which a study is performed by NIBE research bv. The boundary conditions set for this comparison are used for this research and expanded when required.

For the environmental data of the timber bridge, an Environmental Product Declaration (EPD) is used, as mentioned in Section 5.2. This comparative study uses the EPD of Wood for Good, which combines EPDs from multiple manufacturers. This EPD contains all end of life scenarios that must be considered for timber.

Functional unit

The functional units of the compared cases should be equal. For EPD data, environmental impact is expressed per cubic metre or kilogram. The EPD data used for timber represents the impact per cubic metre. The functional unit used for the NIBE research is a bridge of 20 metres long and 7.5 metres wide. The concrete modules are designed for a span of 15-22.5 metres.

The timber modules are designed for a maximum span of 35 metres. The concrete modules are designed for a smaller free span. A span of twenty metres is in the standardised timber system, and therefore the cross-sections are not adjusted. In total, 130.24 m^3 of timber is used in the design. Thus, the data from the EPD (per m^3) is multiplied by 130.24 m^3 of timber used in the bridge.

The steel used in the connections is not included in the environmental impact analysis. The study of NIBE Research bv (2019a) does not include connections; only reinforcing and prestressing steel is included. Furthermore, asphalt surfacing and substructure are not included. Therefore, these components are not included in the timber carbon footprint (CF).

Life cycle stages

The life cycle stages can influence the outcome in both a negative and positive manner. As shown in the literature research, no data is provided by timber EPDs on the use phase (stage B). Similarly, for the concrete bridge, the use stage is not considered. Consequently, the use stage is not included for all bridges.

In phase D, the potential benefits out of the system boundaries are accounted for. The benefits or loads to account for are defined by the end of life scenario considered. For timber, three end of life scenarios are included in EPDs: recycling, incineration and landfill. The benefit out of the system means that the residual product can replace another product after the end-of-life of the bridge. By recycling, wood chips from softwood are generated. By incineration, the replacement of energy generation by fossil fuels is accounted for. For landfill, the assumption is made to have 50% gas extraction from the landfill (used for energy). A sizeable negative impact is considered mainly for incineration, as energy generated by fossil fuels has a high environmental impact, which is avoided. The alternative for this scenario can explain a small effect for recycling in phase D: woodchips from softwood. Woodchips from softwood have a relatively small environmental impact and are less rewarded than replacing energy from fossil fuels.

On a European level, stage D is not allowed to be included in LCAs, where it is in the Netherlands. However, as the impact of multiple stages can be added up to one value for environmental impact, a biased result is obtained. For bridges, the potential benefits in stage D occur in 100 years or longer, containing high uncertainty. It is allowed to include compensation for energy generation by fossil fuels, but it is not likely that energy generation will have the same emissions in 100 years. Furthermore, as it is not included on a European level, it is decided to not include stage D in the environmental comparison. In conclusion, stages A1-A5 and C1-C4 are included.

The occurrence of the end of life scenarios is defined by Stichting Bouwkwaliteit (2014). The NIBE research used these scenarios to obtain one weighted scenario, considering 99% recycling and 1% landfill. For timber, this scenario is currently 90% incineration and 10% landfill.

Timeframe

It is essential to consider the environmental impact combined with the timeframe for which it fulfils a specific function. A structure can have a low impact, but it is not a durable solution when it requires replacement in a short timeframe. Three indicators are governing in the timeframe:

- Reference period;
- Service lifespan;
- Technical lifespan.

The NIBE research defined a service lifespan of 80 years and a technical lifespan of 200 years for concrete. For the concrete comparison, two scenarios are worked out for a reference period of 80 and 200 years.

Pr-EN 1995-2 states one assumes a design life of 100 years for protected bridges. For comparative reasons, a technical lifespan of 200 years is considered as well. Like the concrete bridge, a scenario where the service lifespan is equal to the technical lifespan is also considered. The scenarios considered are expressed in Figure 8.2.

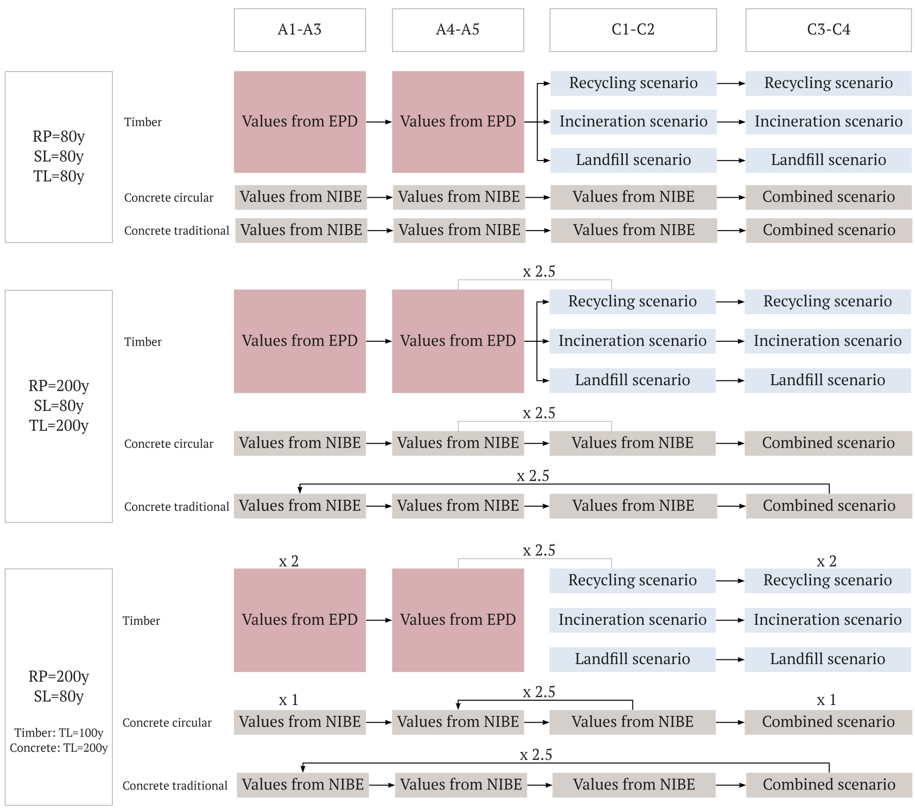


Figure 8.2: Scenarios to consider in comparative carbon footprint study

Available environmental data

EPDs provide data on the environmental impact of a specified product, complying with EN15804. However, the level of detail and boundary conditions are not specified in the regulations. Figure 5.14 showed that the environmental impacts of CLT and glulam correspond well, thus the EPD data for glulam is used for all timber (PE International and Wood for Good, 2013c). Boundary conditions influence the data, for example, by the assumed transportation.

The EPD represents timber structures in the UK, but the timber is produced on the mainland of Europe. As a consequence, 643 km transportation by sea is included and 959 km by road. The distance by road is within expectations, but the high distance by sea would not be required in the Netherlands. No separate information is present on the environmental impact on transportation by sea. Therefore, the environmental impact considered is slightly higher than in the Netherlands. However, this difference is neglected in this analysis. The level of detail leads to a second limitation: only one value is provided for all end of life stages, summarising C1-C4. For the reuse scenarios used, C1-C2 should be separated from C3-C4. However, based on another EPD from Stora Enso (2020a), one can conclude that C1-C2 cause a small part (0.3%) of the environmental impact of timber, compared to C3-C4. The main reason for this is that CO_2 is captured in A1 and released in stage C3, resulting in a high environmental impact for C3. In conclusion, the percentage of C1-C2 in the EPD of Stora Enso (2020a) is assigned to C1-C2 and the remaining part to C3-C4.

8.2. Carbon footprint results

The analyses as described in the previous section are performed in this section. The following abbreviations are used in the graphs: TL=technical lifespan, SL=service lifespan and RP=reference period. Figure 8.3 presents all analyses as stated in Figure 8.2. Three end of life scenarios for timber are considered. The weighted end of life scenario is presented as well, according to Stichting Bouwkwaliteit (2014).

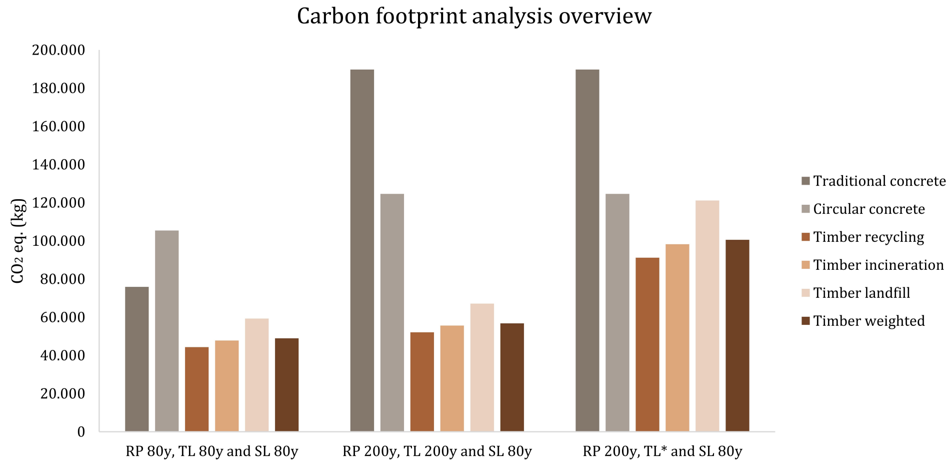


Figure 8.3: Carbon footprint (kg CO_2 eq.) for all stated scenarios. Variations are applied in reference period and technical lifespan. Stage A1-A5 and C1-C4 in the environmental impact are accounted for.

As expected, the reference period and technical lifespans influence the outcome of the analyses significantly. In the short term, the traditional concrete bridge has a lower footprint than the circular concrete bridge. This is caused by the over-dimensioning of the reusable bridge, thus more concrete is used. However, when an extended reference period is considered, the benefit of the circular system is shown. The timber bridges have a significantly lower footprint when the technical lifespan is equal to the reference period. However, when the technical lifespan of 100 years is considered, the landfill scenario is similar to concrete.

With the weighted scenario, the timber bridge contains a lower carbon footprint than both concrete bridges. Even for a technical lifespan of 100 years, less CF is generated. A better insight into the carbon emissions is obtained by plotting the CF in time. Figure 8.4 shows the CF in time for a technical lifespan of 200 years, whereas Figure 10.3 shows the CF for a technical lifespan of 200 years for concrete and 100 for timber.

The carbon footprint plotted in time shows clearly where the main distinction between footprints of timber and concrete is established. Timber contains captured carbon for almost 200 years, lowering the concentration in the atmosphere. On the other hand, concrete emits carbon at the start of its lifespan. IPCC (2018) defined the influence on global warming compared to the year where a carbon-neutral economy is obtained. It is shown that net-zero emissions by 2040 can limit global warming to 1.5 °C. When net-zero is estimated by 2055, global warming is likely to end up 0.2 °C higher.

Furthermore, the emissions in the short term have a high reliability, as the techniques will be similar to the current. However, for the carbon emission of timber, occurring in 100 or 200 years, innovation can occur in capturing the carbon. Developments occur on this topic, however the economic feasibility must be improved to apply it on a large scale.

In conclusion, the IPCC shows the relevance of mitigating carbon emissions now. Carbon capture in timber elements ensures the reduction of carbon in the atmosphere for 100 or 200 years. Consequently, timber structures can reduce global warming now, where concrete structures increase global warming in the short term. Furthermore, as the carbon release occurs in 100 or 200 years, innovations in reducing carbon release are likely to occur. Therefore, the end of life impact can only be reduced. The majority of the timber impact occurs at the beginning of the lifespan, allowing for less innovation.

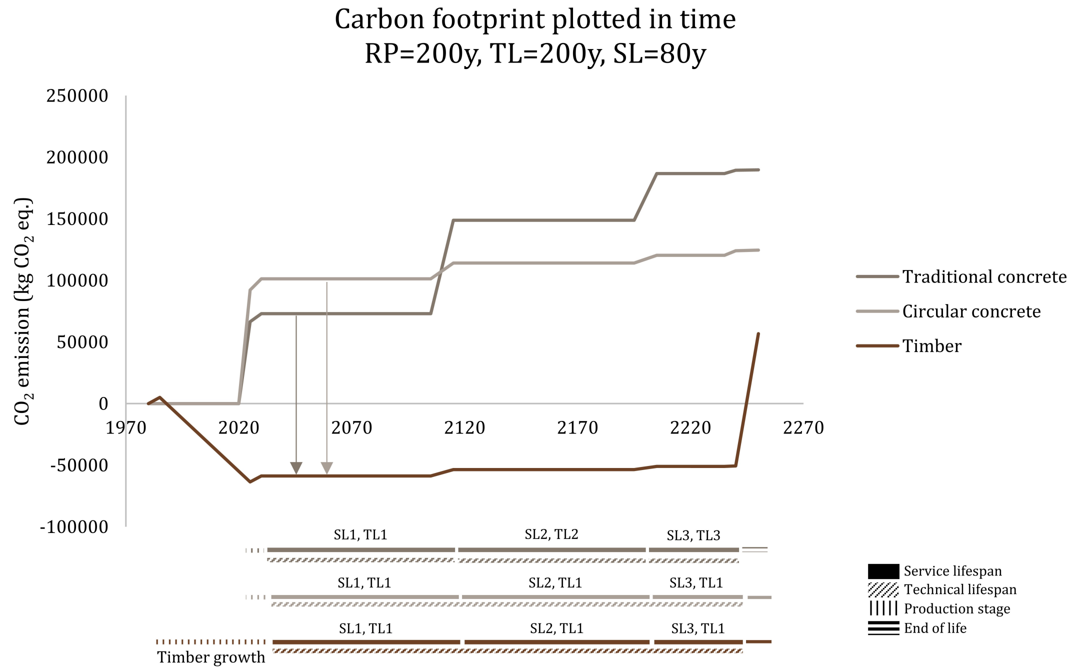


Figure 8.4: Carbon footprint (kg CO_2 eq.) in time. Stage A1-A5 and C1-C4 in the environmental impact are accounted for. Technical lifespan is 200 years for both concrete and timber.

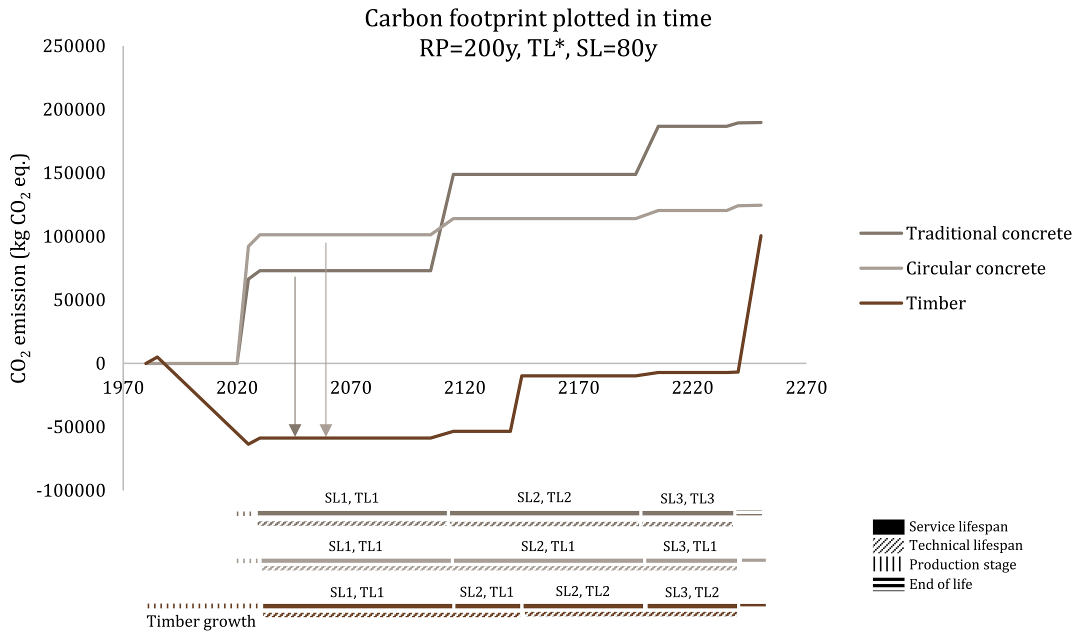


Figure 8.5: Carbon footprint (kg CO_2 eq.) in time. Stage A1-A5 and C1-C4 in the environmental impact are accounted for. Technical lifespan is 200 years for concrete and 100 for timber.

8.3. Conclusions

This chapter focused on research question 3:

What is the environmental impact of the developed highway bridge?

- Which factors influence the quantification, and how can an objective comparison be obtained?
- What is the carbon footprint of the timber bridge compared to a concrete bridge?

It is shown that functional units, stages considered, and time frames mainly influence the outcome of the environmental impact analysis. Furthermore, the quality of the available data is of importance. The discussion (Chapter 9) elaborates on these aspects.

A comparison is made between the circular timber bridge, a traditional concrete bridge and a circular concrete bridge. The production (A1-A3), construction (A4-A5) and end of life (C1-C4) stages are considered in the carbon footprint analyses. For each considered scenario, timber results in a lower carbon footprint than both concrete bridges. The most reliable timeframes are set to a reference period of 200 years, with a technical lifespan of respectively 100 and 200 years for timber and concrete. In this scenario, timber reduces the carbon footprint by 18% compared to the circular concrete bridge and 47% for the traditional.

Next to the expression of footprint in one number, a benefit in time is shown for timber bridges. During the majority of the timber's lifespan, a negative carbon footprint is observed. Drastic reduction of carbon emissions is required soon to mitigate global warming. Reduction of carbon in the atmosphere occurs in the short term due to carbon storage in the timber bridge. For concrete, carbon is emitted at the beginning of its lifespan. Reductions are required now, introducing a second benefit for timber compared to concrete.

9 Discussion

This chapter provides a discussion on the presented results. First, the structural analysis is discussed. Second, the outcome of the carbon footprint analysis is put to discussion. The points of discussion are structured by indicating the limitation, defining the influence of the limitation and assessing the reliability of the results. The outcome of this chapter is implemented in the conclusions and recommendations (Chapter 10).

9.1. Structural analysis

Chapter 7 presents multiple variant studies which led to the developed modular system. All variant studies are based on the Karamba3D model. The geometry is generated parametrically, hence a wide variety of bridges can be modelled. The parametric setup makes Karamba3D suited for the preliminary design stage. However, Karamba3D focuses on global analysis, containing less detailed aspects. The consequences of this accuracy are discussed in this section.

Simplification of details

The structural model includes equivalent strength and stiffness values for connections and material properties on a local scale. Connections are designed by hand, and not modelled locally. Consequently, the connections are likely to be over-dimensioned, due to the conservative approach of the Eurocode. This is substantiated by the review of the timber Eurocode by Jockwer and Jorissen (2018), stating that the stiffness of fasteners is often conservative.

Engineered timber products are non-homogeneous materials, thus simplifications are performed by modelling with equivalent values. The characteristic strength is conservative, leading to a less material-efficient design. Furthermore, the interaction between the timber elements and steel fasteners is not modelled in detail. Local failure modes are now included in the Eurocode requirements. However, modelling local interaction influences the occurring failure modes and thus can lead to different results. However, for the preliminary design phase, these material assumptions are substantiated.

In the deck material analysis, Stress Laminated Timber (SLT) is considered. SLT is modelled as a solid plate material, which is not according to reality. Slip can occur between the stressed boards, introducing additional failure modes. To model the slip, a non-linear analysis is required, which can not be performed in Karamba3D. Therefore, a linear approximation is applied, as stated in Section 5.3. This approximation is based on literature, and is therefore considered reliable for the preliminary design phase. However, for a detailed material analysis, non-linear modelling is required. For this study, SLT did not prove to be the best-suited deck material, hence it is not included in the final design.

Mesh density

The parametric model is an approximation of reality, where the level of similarity relies on the mesh densities. The governing mesh density is the deck panel's, defining the mesh size of the entire model. Nodes are created at each mesh intersection and results are created for each node. Therefore, a denser mesh leads to much more computations. Hence, the calculation time increases significantly. For example, when the mesh density increases in both direction with factor four, the number of calculations increases with sixteen.

The resulting forces from structural analyses (Section 7.4) are as expected for the timber elements, except for the shear force in the deck. The deck mesh density is indicated by the number of nodes in the v and u direction (local coordinate system). With a low mesh density, the centre-to-centre distance between the nodes increases. Consequently, the links between beams and elements have high concentrated forces entering the deck panels. Therefore, concentrated forces are introduced in the deck plate.

To quantify the influence of the mesh density, the number of nodes in the longitudinal (v) direction is varied from 2-25. The mesh density used for the analyses is five. Figure 9.1 shows that the shear force decreases for increasing mesh density. A power trendline is fitted to forecast the behaviour for an infinite dense deck. It is shown that a dense deck results in shear forces according to expectation. Hence, a further detailed model is required for accurate shear force results in the deck.

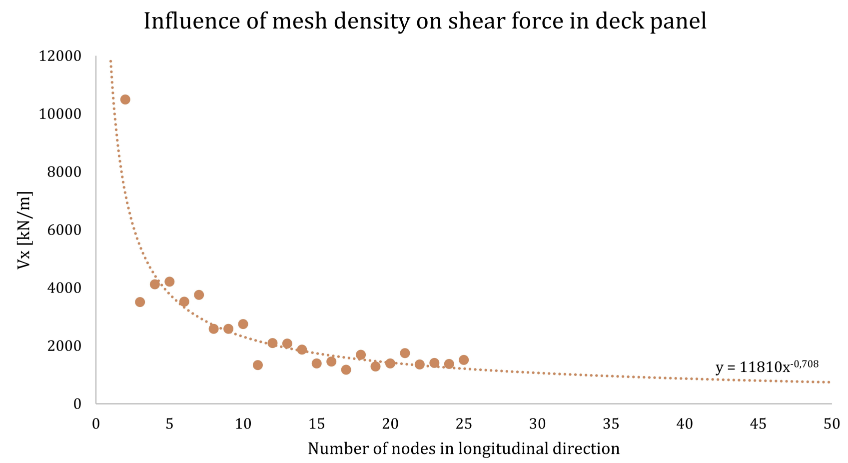


Figure 9.1: Influence of mesh density on the shear force in the deck

Number of studied bridge configurations

In the typology study, five bridge configurations are used for the structural analysis. The optimised cross-sections for each bridge and typology needed to be determined by hand, introducing a time-consuming process. Therefore, time limitations controlled the number of studied bridges. Within the design space, a wide spread of bridge configurations is chosen to provide reliable results. However, with a low number of results, it is hard to determine the properties that influence the typology behaviour. More results lead to better insights on which typology is best-suited for which bridge configuration.

Structural analyses out of scope

Section 2.1 stated that the superstructure is out of the scope of this research. This leads to the assumption of a stiff foundation, which causes no translations or rotations. Furthermore, it is assumed that the new bridge can be attached to the old foundation. As the timber bridge results in a lower self-weight, compared to concrete, it is assumed to be reused. However, the struts introduce two additional support points, hence new foundations need to be constructed there.

Furthermore, this research focuses on traffic loads on the highway bridges. Hence, wind, temperature and accidental loads are not included in the structural assessment of the system. This is based on the study of Arup; Heijmans (2020), where traffic and permanent loads turned out to be governing. However, when for example horizontal collision loads are included, different results can be found for the variant studies.

9.2. Carbon footprint analysis

This section discusses the methodology in environmental impact quantification. The environmental impact of the highway bridges is expressed in carbon footprint. The other impact categories are neglected, which is put to discussion in this section. This study considers two circular principles which are not commonly applied in environmental assessments: reuse and building with a renewable material. The existing methods lack clarity on how to account for these principles, which discussed in this section. Furthermore, the stages to consider and available data for environmental assessment are put to discussion.

Environmental impact categories

The national and international climate goals for 2030 and 2050 focus on global warming, mainly influenced by carbon emissions. Therefore, the goals are expressed in the environmental impact category global warming potential (GWP), expressed in CO_2 equivalents. Nevertheless, other environmental impact categories exist, representing its own harmful effects on the environment. However, as the goals focus on GWP, this research quantifies the benefits in GWP (thus carbon footprint).

The quantification in one number, as provided by the carbon footprint, gives a simplified but clear overview. The Netherlands expresses the impact of multiple categories in one number: the Environmental Cost Indicator (ECI). Section 4.1.2 elaborated on this method. For carbon, a shadow price of €0.05/kg CO_2 is prescribed. Figure 9.2 presents the ECI, including all impact categories and including only the carbon footprint. It is shown that the total ECI and GWP part are in proportion to each other. For concrete, the GWP causes 55% of the ECI, where it is 60% for timber. Considering the total ECI, a reduction of respectively 48% and 24% is obtained compared to the traditional and circular concrete bridge. This is a slightly larger reduction than in CF.

The conclusion is drawn that the carbon footprint is a good representation of environmental impact when comparing concrete and timber bridges.

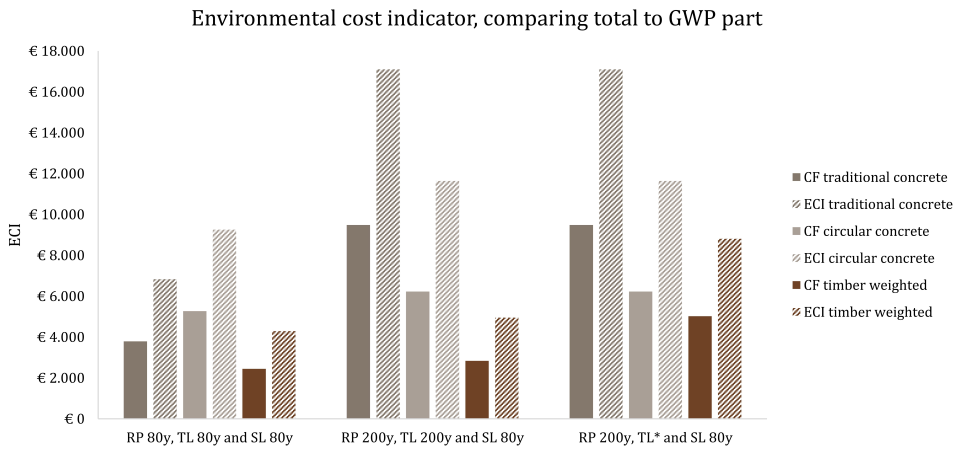


Figure 9.2: Similarity in ECI and CF for the analysed scenarios

Circular structures

Current methods on quantification of environmental impact do not include two positive consequences of building with timber: (1) carbon is captured in the building material, lowering the concentration in the atmosphere, and (2) the benefits of building with a renewable material. The first is not awarded in the Life Cycle Assessment (LCA), as the carbon is captured in the production stage (A) and released in the end of life stage (C). However, this assumes that all carbon re-enters the atmosphere due to incineration. Currently, technologies are in development to capture the carbon from the timber. The carbon emission is based on common practice, but this is likely to change over a lifespan of 100 years.

The avoidance of depleting finite resources by building with renewable materials are also not awarded. The impact category Abiotic Depletion Potential (ADP) considers the resource use of a product. However, this considers the amount of material used, but does not include the renewability of timber. Furthermore, the ADP impact of timber and concrete is minor in the total ECI, as shown in Section 5.2. Therefore, the ADP is not studied in this research, but the stock of raw materials should be further researched.

Stages to consider

A weighted end of life scenario is considered for both concrete and timber bridges. These scenarios are prescribed by Stichting Bouwkwaliteit (2014) and substantiated on current practice. This practice occurs nowadays, which is reliable for products with a short lifespan. However, bridges are expected to be used for 100 or 200 years, where its end of life practice is hard to predict.

The current end of life scenario of incineration occurs for 90% of structural timber. However, rapid change occurs in the energy generation, increasing the demand for green energy. Furthermore, the view on biofuels is becoming less popular, hence it is likely that the energy generation by incinerating timber decreases over time. This will change the end of life scenario of timber. Furthermore, recycling and reuse are stimulated more, which will increase the share of these scenarios. Lastly, carbon capturing during incineration is stimulated and likely to be implemented more in the future. This significantly decreases the carbon emissions in stage C. The main obstacle for large scale application is the high costs that it entails. Concluding, the (weighted) end of life scenario should be observed with care, as these scenarios are likely to change in the coming decades.

Where stage A-C account for emissions directly related to the lifespan of the structure, stage D relates to benefits outside of the system boundaries. The residual product of the structure replaces another product, which emissions are avoided. According to NIBE Research bv (2019b), avoiding emissions in stage D has the highest uncertainty in an LCA. Therefore, it is stated that more focus should be placed on the emissions that are certain now and not on possible benefits in the future. Furthermore, the Netherlands accounts for stage D in an LCA, where the European regulations do not allow including stage D. Therefore, this research decided to exclude stage D of the analysis. However, the results including stage D are interesting as this is according to the Dutch standard method and provides insight into the prevention of other emissions after its end of life.

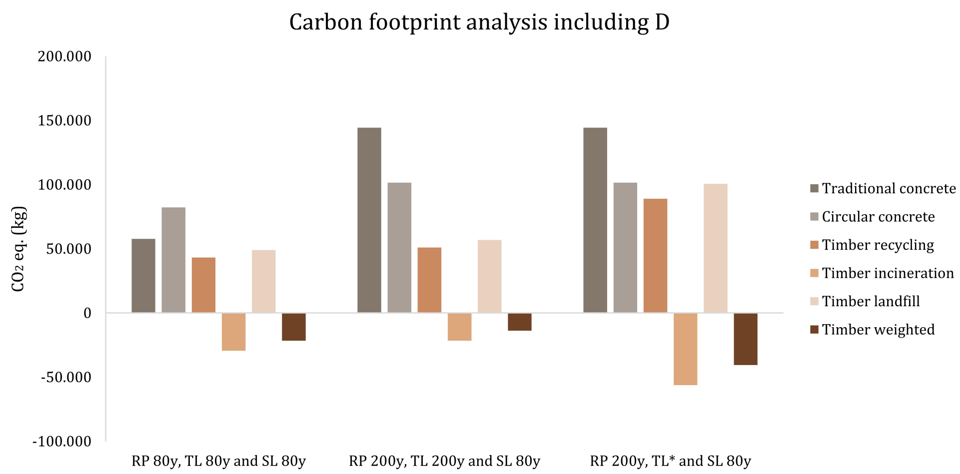


Figure 9.3: Carbon footprint (kg CO_2 eq.) for all stated scenarios. Variations are applied in reference period and technical lifespan. Stage A1-A5, C1-C4 and D in the environmental impact are accounted for.

For concrete, recycling results in the avoidance of gravel production. The result for incineration (and thus the weighted scenario) are most notable. A negative footprint is obtained, as high negative emissions are accounted for in stage D. These negative emissions are not reduction of emissions, but prevention of high emissions due to energy generation by fossil fuels. The incineration of timber is assumed to replace fossil fuel energy, and therefore 'prevents' high emissions. However, this is based fully on fossil fuel energy, where green energy is not included. Furthermore, reuse and recycling should always be preferable over incineration, as incinerating is the lowest grade of using timber. Therefore, the incineration scenario is not desirable, which is implicated by the LCA. For the other scenarios, the difference is less notable, as less significant emissions are prevented. The distorted result of the LCA is proven by comparing Figures 8.3 and 9.3. This observation substantiates the decision to exclude stage D from the carbon comparison since the result is not relevant for reducing carbon emissions.

Reliability of EPD data

The data used for the CF analysis originate from an Environmental Product Declarations (EPD). However, the EPD data is subjective as manufacturers provide it. The EPD used from the institution Wood for Good (WFG) is based on data from multiple suppliers in the United Kingdom. As various suppliers are used, the reliability of this data is higher than from one supplier, but no background information on the data is provided. The data from the EPD (WFG) is compared to the EPD from a German producer: Stora Enso. Figure 9.4 shows the CF for several end of life scenarios. The EPD from Wood for Good is indicated by WFG and Stora Enso by SE.

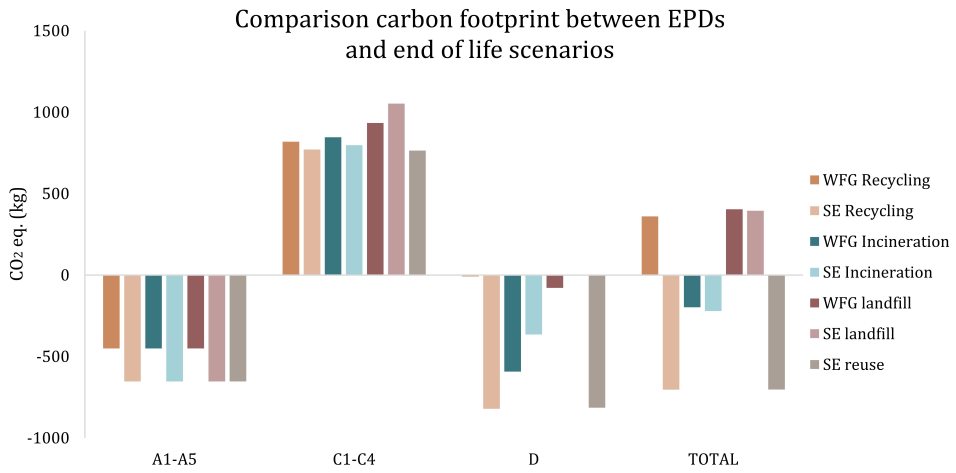


Figure 9.4: Comparison of EPD data per m^3 for carbon footprint, using PE International and Wood for Good (2013c) and Stora Enso (2020a)

A significant difference is shown in stages A1-A5, where the negative CF is approximately 1.5 times larger at Stora Enso. The higher amount of transportation explains the additional emissions for the UK (WFG), as the timber originates from the European mainland. The results for stages C1-C4 correspond well. However, stage D shows the most prominent differences. For recycling, almost no benefits are accounted by WFG, where SE accounts for a large advantage due to recycling. Both EPDs state that recycling into woodchips is assumed in the recycling scenario, but it is not stated which emissions are avoided. Therefore, the high benefits accounted for by Stora Enso are not substantiated and not considered reliable. Consequently, WFG results in a positive CF and SE in a negative CF. For incineration, WFG accounts for more benefits, but less negative footprint is accounted for in stages A1-A5, thus this counterbalances the total footprint. The landfill scenarios correspond well.

These large deviations in data show that the EPD data lacks clarity. Especially the quantification of benefits out of the system boundaries (stage D) is uncertain and contains a high level of interpretation. Furthermore, the source of the EPD should examined critically.

10 Concluding remarks

This chapter presents the conclusions and recommendations of this research. First, the conclusions are stated. The main research question is supported by three sub-questions (Section 1.3). Within the report, the conclusions on the sub-questions were provided at the end of the chapters. This chapter summarizes these conclusions, leading to the answer on the main research question. The conclusions are followed by recommendations for further research, policymakers and Rijkswaterstaat.

10.1. Conclusion

1. Which principles should be applied to obtain a circular timber bridge in the preliminary design phase?

(a) Which strategies can be applied to ensure the design is in line with the fundamentals of the circular economy?
(b) Which aspects of timber structures are governing in the preliminary design?

This study started with a literature study on design codes and regulations to define the design requirements. Eurocode requirements are applied, together with the Dutch ROK (2017) and ROA (2019a). The design space is defined by analysing data on the total bridge stock of Rijkswaterstaat (Section 3.1). The average demolition age of highway bridges is 46 years. This is significantly lower than the technical design lifespan and often caused by functional requirements. Due to the average demolition age, bridges built between 1950 and 1980 are considered in the replacement task. The design space is set with a free span up to fifty metres is set, and a skew between 54 and 90 degrees. 86.8% of bridges within the replacement task are included in this design space.

Four design strategies are defined as guidelines: efficient, protected, flexible, and demountable (Section 4.3). These strategies are defined by literature research into the circular economy (Section 4.1) and outdoor timber constructions (Section 4.2). Multiple variant studies on structural aspects are performed to obtain a material-efficient construction. Design measures are implemented to protect the timber against weather influences. Lastly, the bridge is converted into a modular system to obtain a flexible and demountable structure.

Next to the strategies, essential aspects for the preliminary design phase are established. These are defined by characteristics for (1) general bridge design, (2) modular design and (3) timber design. The first defines that typology, material, and connection choice are governing for all highway bridges in the preliminary design stage. The second leads to the development of a standardised system, including the implementation of the system in the infrastructure. The third aspect leads to additional attention towards protection of the timber members.

2. What composition of a timber bridge results in a circular design?

(a) How to translate the standardisation and boundary conditions of a bridge into a design for a modular system?
(b) Which structural typology is best suited within the defined design space?
(c) How is standardisation applied in the context of replacing existing highway bridges?
(d) Which engineered timber products result in the most material-efficient structure?
(e) Which connections should be demountable, and which connections are best suited for the bridge?

A system with standardised rectangular modules is created, where triangular elements can be added at the ends for skewed crossings (Chapter 6). Standardised modules are applied for three reasons: (1) economic feasibility, (2) low construction time, and (3) flexibility for exchange between bridges. The module's arrangement is based on load paths in the structure to minimise the shear forces and bending moments in the connections between the modules.

Four module sizes in length and two in width direction are established. The largest modules are placed in the middle, combined with smaller modules towards the abutments. The most likely change to the bridge is adaptation in width or length, which calls for an expansion module. Lanes are 3.5 metre wide, thus a square module is defined: 3.5 × 3.5 metres. To determine the length of the large module, an optimisation is performed for all bridges in the design space. The goal of this optimisation was to reduce the number of modules and excessive length. As a result, an optimal length of 17.5 metres for the large module is obtained. Both described dimensions can be cut in half, for more flexibility, hence eight modules are developed.

Three typology options are analysed: beam structure, strut structure and a truss bridge. The beam and strut structure are developed with two different module configurations: (1) cross beams on top of the main beams or (2) integrated cross beams. On the bridge level, the strut typology is best suited in the defined design space (Section 7.2). The free span is reduced by the struts while maintaining the envelope of free space. Furthermore, high flexibility is maintained, both for expansion in length and width. For the module configuration, the integrated cross girders are most material-efficient. A compact structure is obtained, where composite behaviour is enabled between the deck and main girders. Composite behaviour increases the strength of the total cross-section. Figure 10.1 presents the established global design.

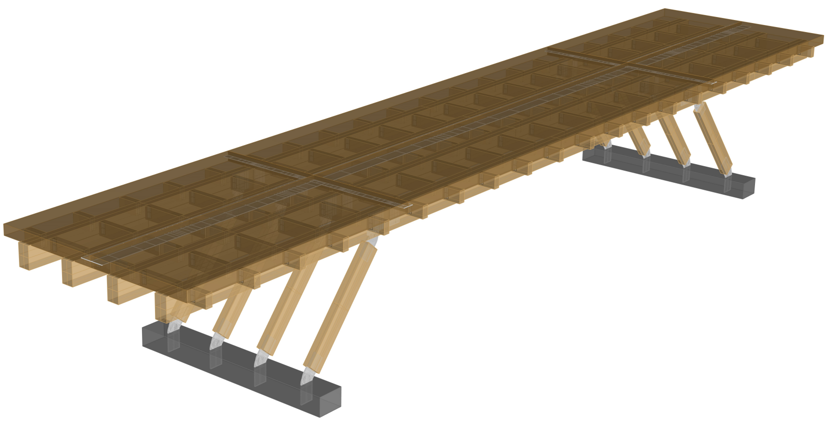


Figure 10.1: Best suited typology: strut frame with integrated cross girders

Standardisation is defined by the scope of the system (Section 7.3). Material efficiency and range of application are considered to define the spans included in the standardised system. A maximum free span of 35 metres is established, using two struts. This scope covers 58% of the Rijkswaterstaat bridge stock on highway bridges, built between 1950 and 1980. One standardised set of modules is developed, with constant cross-sections. Consequently, all modules can be interchanged, as they have a similar stiffness. With more module configurations designed, the scope of the modular system can be expanded.

The best-suited material is defined as the material with the lowest environmental impact while satisfying the strength and stiffness requirements (Section 7.4). Both beam and deck materials are researched. For beam materials, four timber products are analysed: GL24h, GL28h, GL32h and LVL. The literature study showed that all four products have a similar environmental impact per cubic metre (Section 5.2). Deflection is governing for dimensioning the main girders, and both glulam and LVL perform similar on deflection requirements. Hence, the considered aspects can not establish a substantiated result for the most efficient beam material. With a European price indication where LVL is 40% more expensive than glulam, the decision is made to consider glulam as the best-suited material. More specific, GL28h is chosen, as it is produced on the largest scale in Europe. Large scale production improves the economic feasibility of the bridge, where it also increases the potential for reuse.

Dimensioning the deck is also governed by deflection requirements. Five deck materials are considered in the comparison: CLT, LVL-X, LVL-S, SLT from C24 and SLT from GL28h. CLT made from C24 is best suited as deck material when considering stiffness. The smallest deflection is observed for CLT due to load transfer in two directions. Compared to other cross-laminated materials, CLT has a high shear stiffness, reducing the occurring deflection further.

The ability for demounting determines the level of circularity of the construction. The connection between the main girders and deck is considered governing for the level of demountability. This connection highly influences the global structural behaviour, as composite behaviour can be obtained. Furthermore, to make the link demountable, many fasteners are required. Consequently, much labour is required for demounting, and it is not a feasible solution. This study considered three types of connections - dowels, inclined screws, and glue - of which the inclined screws perform best on structural behaviour. This connection is not demountable, thus the modules are prefabricated as one solid element. Consequently, the other connections within the module do not have to be demountable. Only the (demountable) connections between the modules are assembled on site.

The final design is based on the maximum free span in the scope of the modular system: 35 metres. This bridge contains two × two roads below and above the bridge. All connections are designed and calculated to comply with the boundary conditions and requirements (Section 7.6). The final design is shown in Figure 10.2. A larger scale drawing is included in Appendix K, which also presents detailed drawings of the connections.

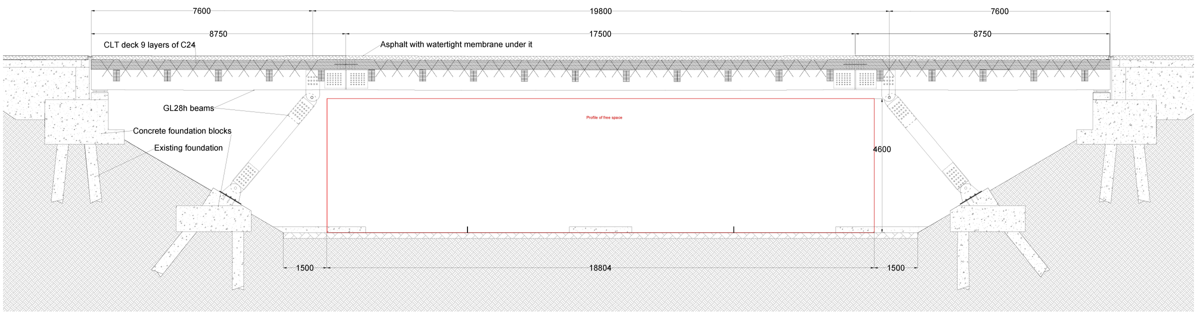


Figure 10.2: Final design of the bridge, including connections

3. What is the environmental impact of the developed highway bridge?

(a) Which factors influence the quantification, and how can an objective comparison be obtained?
(b) What is the carbon footprint of the timber bridge compared to a concrete bridge?

The environmental impact is mainly influenced by: (1) functional unit, (2) life cycle stages, and (3) timeframe. For an objective environmental impact, a careful statement of these factors is required. This study compares the carbon footprint of a traditional concrete bridge, a circular concrete bridge and the circular timber bridge. Stages A1-A5 (production and construction) and C1-C4 (end of life) are considered in the footprint. For the end of life scenario, the most reliable scenario from Stichting Bouwkwaliteit (2014) is applied for timber (90% incineration and 10% landfill). The following timeframe is considered most reliable: reference period = 200 years, service life = 80 years, technical lifespan = 200 years for concrete and 100 years for timber. Reductions of respectively 47% and 18% are obtained in comparison to the traditional and circular concrete bridge. The conclusion is drawn that the timber alternative shows a significant benefit compared to the current highway bridges, traditional concrete.

The carbon footprint plotted in time gives more insight into the impact in the short and long term, presented in Figure 10.3. In the short term, the timber bridge contributes to the reduction of carbon in the atmosphere. On the other hand, concrete emits carbon at the beginning of the timeframe, contributing to global warming. According to IPCC (2018), a quick reduction of carbon concentration in the atmosphere is eminent to limit global warming. The carbon reduction of replacing a concrete bridge by a timber bridge is illustrated in Figure 10.3 by the difference in carbon level during its lifespan.

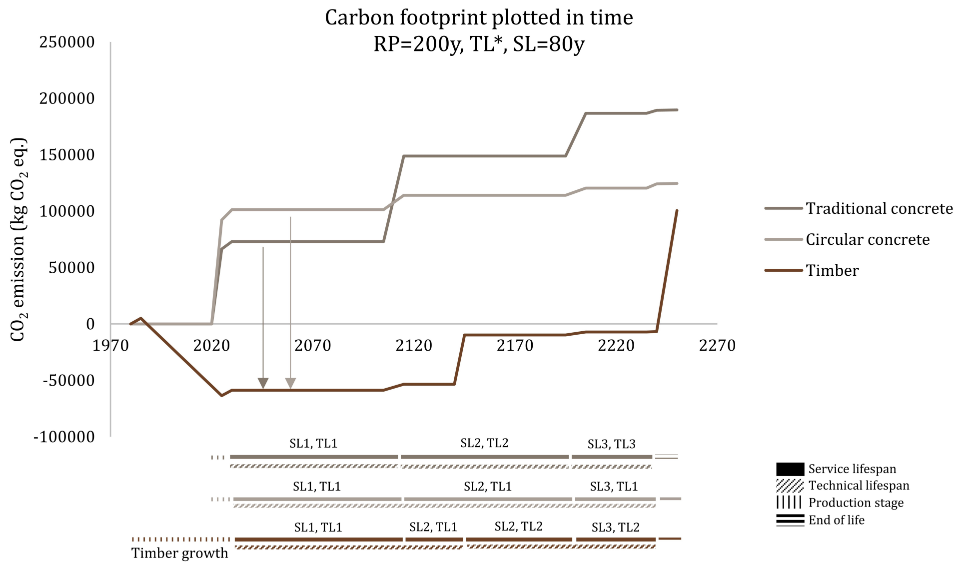


Figure 10.3: Carbon footprint (kg CO_2 eq.) in time. Stage A1-A5 and C1-C4 in the environmental impact are accounted for. Technical lifespan is 200 years for concrete and 100 for timber.

How can a modular timber bridge lead to a circular alternative for highway bridges when considering the preliminary design phase?

A standardised system of modules is developed, where the links between the modules can be disassembled. Consequently, the bridge can be adapted or the modules can be reused multiple times during their lifespan. The lifespan of products is extended, as the functional lifespan is not governing due to flexibility.

The benefits of this new bridge system emerge on three levels: (1) a renewable resource is used, avoiding depletion of finite resources, (2) the carbon concentration is the atmosphere is lowered due to carbon storage in the bridge, and (3) the lifespan is extended by the modular system, as adaptation and reuse are enabled. The carbon footprint resulted in reductions of 18% and 47% for respectively the circular and traditional concrete bridge. Plotting the carbon footprint in time reveals a second benefit: carbon emissions are avoided in the short term. Consequently, the timber bridge has a negative carbon footprint during the main part of its lifespan, reducing the carbon concentration in the atmosphere. Global warming must be reduced in the short term, hence significant reduction of carbon in the atmosphere is relevant now.

Change is required in the construction industry to meet the climate and circular goals of the Dutch government and Rijkswaterstaat. The developed timber bridge contributes to the reduction of negative impacts on the environment and use of primary abiotic materials. It is a sustainable, circular and durable solution, both in the short and long term.

10.2. Recommendations

This section provides recommendations on the topic of circular timber bridges. These are split in recommendations for further research, policymakers and Rijkswaterstaat.

10.2.1. Recommendations for further research

The timeframe set for a graduation thesis leads to scope limitations and assumptions. This research is the first study in circular timber highway bridges for the Netherlands. Therefore, aspects are left out of the scope, which brings demand for further research.

Karamba3D is chosen as software for structural analysis and optimisation. However, it is recommended to verify the Karamba3D model with a FEM model in different software. Additionally, settings in the Karamba3D model can be verified, among others the mesh density of the deck as expressed in the discussion. Next to verifying, a more detailed structural analysis improves the reliability of local behaviour. Connections are now designed by hand, and equivalent properties are put in the model. Sufficient level of detail is obtained for the preliminary design stage, but other FEM software provides more accurate results on connection and composite behaviour.

Long-term behaviour is not studied in this research. Important long-term aspects are fatigue, and the influence of moisture and waterproofing systems on timber deterioration. Literature is studied on these aspects, and design measures are taken. However, no modelling of these aspects is performed. It is recommended to undertake further research on fatigue behaviour of steel-timber connections. Research into waterproofing systems and their effectiveness is recommended as well. Furthermore, monitoring of existing timber bridges can substantiate the expected lifespan.

This research performed optimisations on structural aspects. A modular system is developed with one set of modules, with a constant cross-section. By performing an economic study, the balance between standardisation and flexibility in the modular system can be researched. This can further increase the feasibility of the circular bridge. A study into costs, environmental impact, benefits of standardisation and demand for flexible bridges is recommended for further development and implementation of the concept.

10.2.2. Recommendation for policymakers

In the results and discussion on environmental impact, recommendations arose for policymakers. It is shown that high uncertainty lies in environmental impact data and regulations. Stricter requirements on the Dutch and European level are required for more transparency in the environmental impact of products. Greenwashing occurs easily in environmental data, especially when regulations are loose. With the clear ambitions to reduce emissions in the coming decades, clear regulations are required for EPDs. Especially carbon sequestration (stage A), end of life scenarios (stage C) and additional benefits (stage D) require guidance in data composition.

Furthermore, additional guidance and regulations are required on the methodology of LCAs. More guidance is needed on how and which end of life scenarios to include for objective comparison. Moreover, with the current methodology, environmental data of different stages is summed up, but not all data is equally reliable. The impact in the production stage occurs in the short term, where on the other hand, the end of life impact occurs in 100 or 200 years, containing high uncertainty. Especially for stage D, where benefits out of the system boundary are examined, more nuance is required as the direct benefits to the system are unsure.

Two benefits of building with timber are not valued in the current regulations: (1) no finite resources are depleted with sustainably managed forests, and (2) the timber in construction works as a natural carbon capture and storage system. Concrete and steel are well-known, durable and economically feasible materials, and therefore often applied in infrastructure design. When regulations reward the aforementioned benefits of timber, an incentive is created to construct with timber.

Changing regulations to account for benefits of circular and timber structures, supports the ambition for transition to a climate neutral and circular economy. It is recommended to obligate the consideration of emissions in time, adding a distinction between short and long term emissions. Furthermore, more nuance is provided to the (un)certainty of the used data. Furthermore, this study recommends to include the distinction between use of finite or renewable resources in the environmental assessment. Depleting finite resources impacts the environment as well, thus should be included in environmental assessments.

10.2.3. Recommendations for Rijkswaterstaat

This research focused on the replacement task of highway bridges, which is the responsibility of Rijkswaterstaat. Rijkswaterstaat aims to work circular by 2030, which is in nine years. Therefore, significant change in the construction of highway bridges is required. This study shows that significant environmental benefits are obtained by the circular timber bridge. The IPCC (2018) shows that the carbon emissions must be reduced in the short term, to limit global warming. Consequently, Rijkswaterstaat has the obligation to reduce carbon emissions on the short term, which is enabled by this circular timber bridge.

Many Dutch highway bridges are designed similarly, as the regulations support the ’business as usual’ highway bridges. This study recommends Rijkswaterstaat to invest in the development of circular timber bridges. All disciplines in the infrastructure sector should be involved in this process, to establish a feasible, environmental friendly alternative. Rijkswaterstaat must also adapt their design manuals, enabling timber bridges to be a viable alternative to concrete/steel bridge.

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Wood Solutions. Environmental Product Declaration Softwood Timber, 2020b.

List of Symbols and Abbreviations

List of Abbreviations

ADP	Abiotic Depletion Potential
AP	Acidification Potential
CB'23	Circulair Bouwen 2023
CE	Circular economy
CF	Carbon footprint
CG	Cross girder
CLT	Cross laminated timber
DfA	Design for Adaptability
DfD	Design for Disassembly
DfME	Design for Material Efficiency
DP	Deck panel
EC	Eurocode
EPD	Environmental Product Declaration
EP	Eutrophication Potential
Glulam	Glued laminated timber
GWP	Global Warming Potential
LCA	Life Cycle Assessment
LM1	Load model 1

LVL	Laminated veneer lumber	
MG	Main girder	
ODP	Ozone Depletion Potential	
POCP	Photochemical Ozone Creation Potential	
RP	Reference period	
SLS	Serviceability limit state	
SL	Service lifespan	
TL	Technical lifespan	
TS	Tandem system	
UC	Unity check	
UDL	Uniformly distributed load	
ULS	Ultimate limit state	

Nomenclature

α	Angle of fastener to the grain direction of the timber	rad
α_Q	Adjustment factor for traffic load	-
β	Ratio of embedment strength element 1 and 2	-
γ_M	Partial factor	-
ψ_0	Factor for quasi-permanent loading	-
ρ_k	Characteristic density of the timber	kg/m^3
ρ_m	Mean density of timber	kg/m^3
$\sigma_{c,0,d}$	Design value of compression stress parallel to grain	N/mm^2
$\sigma_{c,90,d}$	Design value of compression stress perpendicular to grain	N/mm^2

$\sigma_{m,y,d}$	Design value of bending stress around y-axis	N/mm^2
$\sigma_{m,z,d}$	Design value of bending stress around z-axis	N/mm^2
$\sigma_{t,0,d}$	Design value of tension stress parallel to grain	N/mm^2
τ_d	Design value of shear stress	N/mm^2
A	Area	mm^2
a_1	Distance between fasteners, perpendicular to the fastener	mm
A_{ef}	Effective contact area for compression perpendicular to the grain	mm^2
C	Height of camber	m
E	Modulus of elasticity	N/mm^2
e_i	Distance to center	mm
f_u	Tensile strength of the fastener	N/mm^2
$F_{ax,Ed}$	Design axial stress	N
$f_{ax,k}$	Characteristic axial strength, perpendicular to the grain direction	N/mm^2
$F_{ax,Rd}$	Design axial strength	N
$f_{c,0,d}$	Design value of compression strength parallel to grain	N/mm^2
$F_{c,90,d}$	Design value of the compression force perpendicular to the grain	N
$f_{c,90,d}$	Design value of compression strength perpendicular to grain	N/mm^2
$f_{m,y,d}$	Design value of bending strength around y-axis	N/mm^2
$f_{m,z,d}$	Design value of bending strength around z-axis	N/mm^2
$f_{t,0,d}$	Design value of tension strength parallel to grain	N/mm^2
$f_{v,d}$	Design value of shear strength	N/mm^2
$F_{v,Ed}$	Design shear stress	N
$F_{v,Rd}$	Design shear strength	N

G	Shear modulus	N/mm^2
G_k	Characteristic value of permanent load	kN orkN/m^2
I	Moment of inertia	mm^4
$k_{c,90,d}$	Factor for compressive strength	-
k_{mod}	Modification factor for timber strength	-
k_m	Factor for bending strength that takes into account redistribution of forces	-
K_r	Rotational stiffness of group of fasteners	kNm/rad
K_{ser}	Slip modulus of fastener	Nmm
k_{sys}	Modification factor for system strength	-
l_{ef}	Effective length of a screw	mm
n_{ef}	Effective number of fasteners	-
Q_k	Axle load	kN
q_k	Uniformly distributed load	kN/m^2
$Q_{1,k}$	Characteristic value of governing variable load	kN orkN/m^2
$Q_{i,k}$	Characteristic value of other variable loads	kN orkN/m^2
R	Radius of camber arc	m
t_1	Thickness of timber element 1 in which the fastener is present	mm
t_2	Thickness of timber element 2 in which the fastener is present	mm
u_{camber}	Height of camber	mm
u_{creep}	Deflection due to creep on long term	mm
u_{inst}	Instant deflection	mm
$u_{net,fin}$	Net deflection on long term	mm
d	Diameter of fastener, including thread	mm

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Appendices

A

Environmental impact

The environmental impact data is converted to environmental impact costs, in order to quantify the actual impact on the environment in one unit. The costs per unit for each environmental impact category are shown in Table A.1.

Table A.1: Shadow costs for each category (Stichting Bouwkwaliteit, 2014)

	Unit	Costs
GWP	kg CO_2 eq.	€0.05
ODP	kg CFC-11 eq.	€30
AP	kg SO_2 eq.	€4
EP	kg PO_4 eq.	€9
POCP	kg Ethene eq.	€2
ADP elements	kg Sb eq.	€0.16
ADP fossil	kg Sb eq.	€0.16

A.1. Soft- and hardwood comparison

Both sawn wood, CLT and glulam can be produced in either soft- or hardwood. The difference between soft- and hardwood is in the structure on cell level. Both have their advantages and disadvantages. In this section, the focus is placed on the environmental impact, mechanical properties and durability.

No EPDs can be found that are performed for both sawn hardwood and sawn softwood within Europe. Therefore, EPDs are used from Australia (Wood Solutions, 2020a) and (Wood Solutions, 2020b). These EPDs are according to the Eurocode, but based on Australian wood for application in Australia. This influences the transportation included, and the properties of the wood. However, as both EPDs are made for Australian wood and application, a comparison can be made. Only stage A1-A3, C3 and C4 are included in the EPD. As the construction process and use are not included, the data is more objective for all types of constructions. For both studies, the data is provided by multiple wood suppliers. In the EPD that is used, an average is taken from Australian hardwood, with an average density of 735 kg/m^3. For the softwood that is analysed, again an average is taken from Australian softwood species, of which the average density is 551 kg/m^3. For European softwood species, an average density of 483 kg/m^3 is observed (PE International and Wood for Good, 2013a). The density of wood influences the biomass, and thus the carbon captured. Therefore, results for wood species with different densities can deviate from this comparison.

Table A.2: EPD data phase A1-A3 for soft- and hardwood

	Softwood	Hardwood
GWP	-699	-731
ODP	4.72 E-11	8.90 E-11
AP	1.1	2.54
EP	0.275	0.565
POCP	0.68	3.88
ADP elements	7.86 E-5	1.10 E-5
ADP fossil	2250	3830

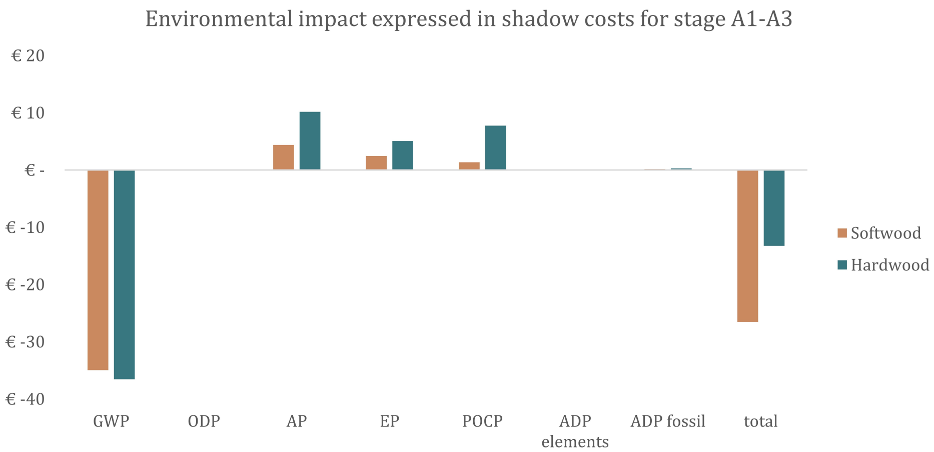


Figure A.1: Environmental costs of hard- and softwood stage A1-A3

Two scenarios can be established for the end of life:

- Energy recovery
- Landfill

The end of life scenario of the construction material defines the environmental impact of phase C1-C4. Stichting Bouwkwaliteit (2014) defined the end of life scenarios for wooden constructions in the ’GWW sector’ (civil engineering works). It is stated that 90% of the timber used in civil engineering works ends up to be incinerated, thus burned to generate energy. The other 10% ends up as landfill. In EPDs, the recycling scenario is often included as well, but according to Stichting Bouwkwaliteit (2014), this does not occur in the Netherlands. When the scenario of energy recovery is considered, environmental impact is assigned to stage C3, for landfill the impact is assigned to C4. Table A.3 presents the combined environmental impact of both scenarios, with 0.9*incineration and 0.1*landfill.

Table A.3: EPD data phase C3-C4 for soft- and hardwood combined incineration and lanfill scenario

	Softwood	Hardwood
GWP	821.52	1112.84
ODP	3.01 E-12	3.1 E-12
AP	4.98 E-2	6.08 E-2
EP	9.74 E-3	1.23 E-2
POCP	3.90 E-3	4.80 E-3
ADP elements	1.22 E-6	1.20 E-6
ADP fossil	149.41	172.08

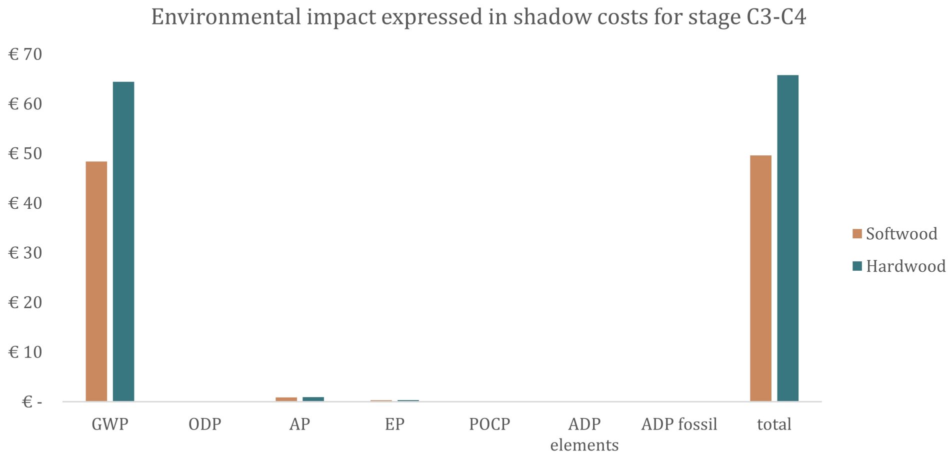


Figure A.2: Environmental costs of hard- and softwood stage C3-C4

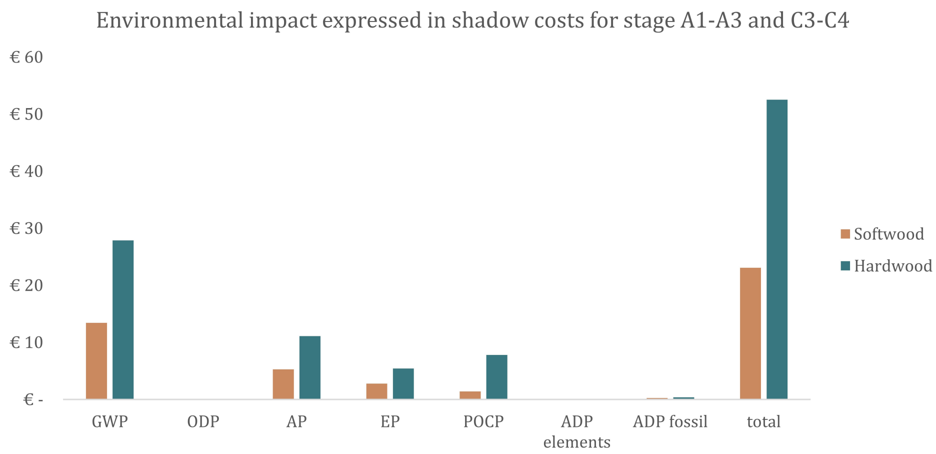


Figure A.3: Environmental costs of hard- and softwood total

The negative global warming potential (GWP) is almost equal for both types of wood in stage A1-A3, caused by the growth of the wood. In stage C3-C4, only GWP has a significant influence on the environmental impact. However, in stage A1-A3, a significant difference is shown in the impact of AP, EP and POCP. According to Van Wijnen (2020), the impact in these categories is mostly caused by 'the combustion of fossil fuels during tree felling and biomass combustion for the heat generation of the drying process'. Both EPDs are based on kiln-dried wood, but the remark must be made that hardwood is based on a moisture content of 10%, where softwood is used with a moisture content of 12%. It can be concluded that a more extensive drying process was required for the hardwood, partly because a lower moisture content was obtained, but also because of the higher density and different structure of the wood. Despite this difference, the conclusion can be drawn that the environmental impact per m^3 is significantly higher for hardwood than for softwood.

In addition to the environmental impact, the mechanical properties of softwood and hardwood are considered. Better mechanical properties (stronger or stiffer) can result in less material required, and thus can reduce the environmental impact. However, increased strength and stiffness makes the lamination harder. For laminated materials, manufacturing with hardwood is difficult, as the bonding between the members is hard to obtain due to the higher density (Aicher et al., 2014). Ramage et al. (2017) normalized the stiffness and compression strength of several materials, relative to the density of the wood. The results of this study are summarized in Figure A.4. Figure A.5 shows the durability of hardwood and softwood species, including resistance against water ingress and fungi growth.

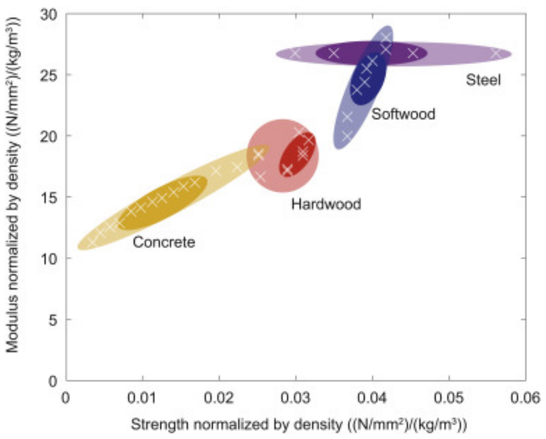


Figure A.4: Strength and stiffness, relative to density, of soft- and hardwoods (Ramage et al., 2017)

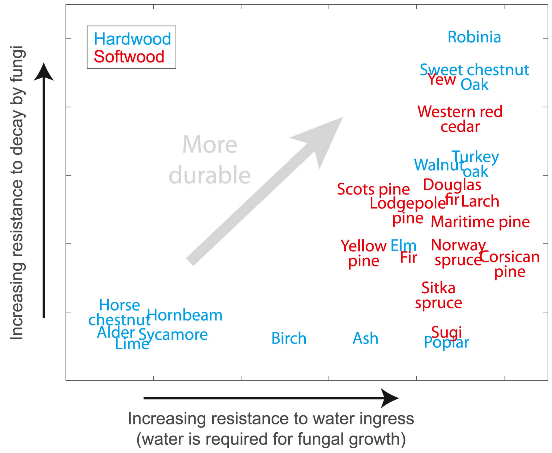


Figure A.5: Durability of soft- and hardwoods (Ramage et al., 2017)

It is shown that softwood has a more efficient strength/density relation, as well as a better stiffness/density relation. The hardwood used in the EPD has an average density of 735 kg/m^3, compared to 551 kg/m^3 for the softwood. This is 33% higher for hardwood, where the stiffness/density relation is 25% lower. Therefore, the hardwood has a slightly higher stiffness than the softwood considered. For strength, both types of wood end up approximately with the same strength, as the strength/density relation is 33% higher for softwood. In conclusion, the stiffness and strength are not significantly higher for the considered hardwoods than for the softwoods, so the large difference in environmental impact is not compensated by this.

Figure A.5 shows that some hardwood species are more durable than softwood in terms of fungi growth. As stated in the section on durability, no fungi grows without water ingress. In terms of water ingress, most soft- and hardwood species are performing similar. Therefore, softwood can be as durable as hardwood, and no significant distinction is made here.

Taking into account the mechanical properties, environmental impact and durability, softwood is better suited to use for the bridge design.

A.2. Timber product comparison

The data is according to the following EPDs:

- Sawn timber (PE International and Wood for Good, 2013a)
- Glulam (PE International and Wood for Good, 2013c)
- LVL (PE International and Wood for Good, 2013d)
- CLT (PE International and Wood for Good, 2013b)

A.2.1. Boundary conditions of EPDs

No adhesives are used in sawn wood. A moisture content of 15% is used, with an average density of 483 kg/m^3.
For the glulam EPD, an average adhesive mix is assumed, consisting of (PE International and Wood for Good, 2013c):

- 1.94% melamine urea formaldehyde (MUF);
- 0.09% phenol resorcinol formaldehyde (PRF);
- 0.03% polyurethane (PUR).

A moisture content of 12% is used, with an average density of 490 kg/m^3.
In the EPD for LVL, a mix of phenol formaldehyde (PF) and phenol resorcinol formaldehyde (PRF) is used, which add up to an adhesive content of 2.5% (PE International and Wood for Good, 2013d) . A moisture content of 12% is used, with an average density of 488 kg/m^3.
For the CLT EPD, an average adhesive mix is assumed, consisting of (PE International and Wood for Good, 2013b):

- 1.4% melamine urea formaldehyde (MUF);
- 0.1% emulsion polymer isocyanate (EPI);
- 0.5% polyurethane (PUR).

A moisture content of 12% is used, with an average density of 488 kg/m^3.

A.2.2. Results

Table A.4: EPD data phase A1-A3

	Unit	Sawn wood	GLT	LVL	CLT
GWP	kg CO_2 eq.	-679	-488	-537	-494
ODP	kg CFC-11 eq.	2.98 E-9	1.66 E-8	1.90 E-8	2.32 E-8
AP	kg SO_2 eq.	0.61	1.03	1.14	1.06
EP	kg PO_4 eq.	0.11	0.18	0.17	0.175
POCP	kg Ethene eq.	4.86 E-2	8.90 E-2	0.11	0.0928
ADP elements	kg Sb eq.	7.81 E-6	8.42 E-5	5.81 E-5	1.41 E-4
ADP fossil	MJ	1390	3860	3540	3990

Table A.5: EPD data phase A4-A5

	Unit	Sawn wood	GLT	LVL	CLT
GWP	kg CO_2 eq.	22.5	37.20	44.30	43.2
ODP	kg CFC-11 eq.	7.88 E-11	1.50 E-10	1.65 E-10	1.82 E-10
AP	kg SO_2 eq.	0.24	0.28	0.70	0.247
EP	kg PO_4 eq.	3.29 E-2	4.66 E-2	8.41 E-2	4.84 E-2
POCP	kg Ethene eq.	-1.39 E-2	-4.33 E-2	-2.48 E-3	-6.21 E-2
ADP elements	kg Sb eq.	6.40 E-7	1.209 E-6	1.289 E-6	1.46 E-6
ADP fossil	MJ	300	504	579	590

Table A.6: EPD data phase C1-C4 weighted to scenarios (90% incineration, 10% landfill)

	Unit	Sawn wood	GLT	LVL	CLT
GWP	kg CO_2 eq.	820.60	854.80	853.30	851.90
ODP	kg CFC-11 eq.	2.57 E-10	3.23 E-10	3.30 E-10	3.20 E-10
AP	kg SO_2 eq.	0.86	0.87	0.87	0.87
EP	kg PO_4 eq.	0.15	0.15	0.15	0.15
POCP	kg Ethene eq.	9.38 E-2	9.52 E-2	9.46 E-2	9.49 E-2
ADP elements	kg Sb eq.	1.01 E-6	2.52 E-6	2.80 E-6	2.46 E-6
ADP fossil	MJ	323.8	335.5	336.0	334.4

Table A.7: EPD data phase D weighted to scenarios (90% incineration, 10% landfill)

	Unit	Sawn wood	GLT	LVL	CLT
GWP	kg CO_2 eq.	-519.19	-541.61	-540.66	-539.79
ODP	kg CFC-11 eq.	2.21 E-8	2.30 E-8	2.30 E-8	2.30 E-8
AP	kg SO_2 eq.	-1.35	-1.40	-1.39	-1.40
EP	kg PO_4 eq.	-0.12	-0.12	-0.12	-0.12
POCP	kg Ethene eq.	-8.37 E-2	-8.70 E-2	-8.68 E-2	-8.67 E-2
ADP elements	kg Sb eq.	1.28 E-5	1.28 E-5	1.30 E-5	1.30 E-5
ADP fossil	MJ	-7221	-7553	-7543	-7526

Table A.8: EPD data phase A1-A5 and C1-C4

	Unit	Sawn wood	GLT	LVL	CLT
GWP	kg CO_2 eq.	164.1	404.0	360.6	401.1
ODP	kg CFC-11 eq.	3.32 E-9	1.70 E-8	1.90 E-8	2.37 E-8
AP	kg SO_2 eq.	1.71	2.18	2.72	2.18
EP	kg PO_4 eq.	0.29	0.38	0.41	0.38
POCP	kg Ethene eq.	0.13	0.14	0.20	0.13
ADP elements	kg Sb eq.	9.46 E-6	8.80 E-5	6.20 E-5	1.45 E-4
ADP fossil	MJ	2013.8	4699.5	4455.0	4914.4

Table A.9: Environmental costs phase A1-A5 and C1-C4

	Unit	Sawn wood	GLT	LVL	CLT
GWP	€0.05 / kg CO_2 eq.	€8.21	€20.20	€18.03	€20.06
ODP	€30 / kg CFC-11 eq.	€0.00	€0.00	€0.00	€0.00
AP	€4 / kg SO_2 eq.	€6.84	€8.73	€10.87	€8.71
EP	€9 / kg PO_4 eq.	€2.59	€3.42	€3.65	€3.38
POCP	€2 / kg Ethene eq.	€0.26	€0.28	€0.39	€0.25
ADP elements	€0.16 / kg Sb eq.	€0.00	€0.00	€0.00	€0.00
ADP fossil	€0.16 * 4.81 E-4 / MJ	€0.15	€0.36	€0.34	€0.38
Total		€18.05	€33.00	€33.29	€32.77

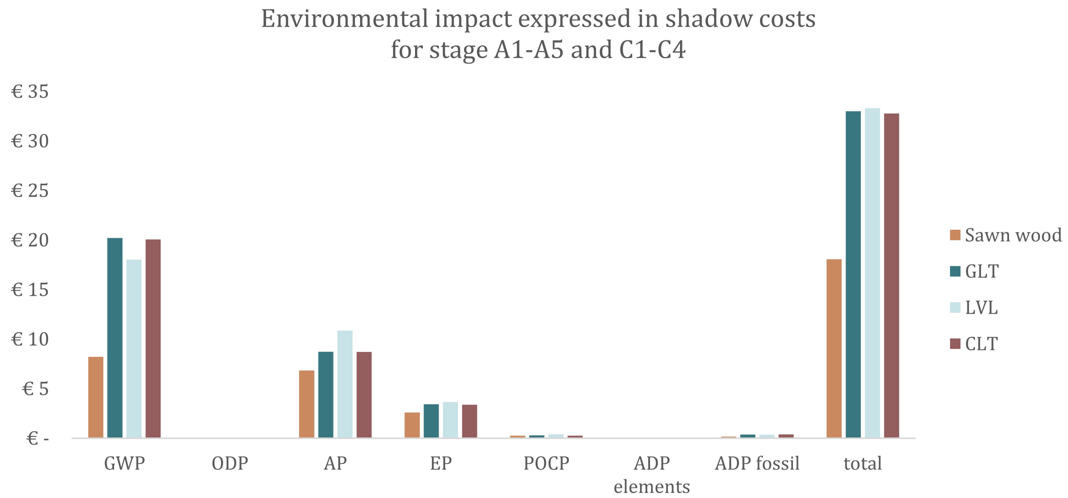


Figure A.6: Environmental costs of timber products total

A.3. Carbon footprint analysis

This section provides the tables with data as used in Chapter 8.

Table A.10: Carbon footprint (kg CO_2 eq.) for one functional lifespan

Stage	Traditional concrete	Circular concrete	Timber - recycling	Timber - incineration	Timber - landfill
A1-A3	66444	92176	-63557	-63557	-63557
A4-A5	6501	9111	4845	4845	4845
C1-C2	2637	3666	0	0	0
C3-C4	302	447	106667	110183	121644
D	-18146	-23090	-1095	-77232	-10302
Total incl D	57738	82310	43193	-29427	48964
Total excl D	75884	105400	44288	47805	59266

Table A.11: Multiplication factor for reference period of two hundred years, with technical lifespan of two hundred years

Stage	Traditional structures	Circular structures
A1-A3	2.5	1
A4-A5	2.5	2.5
C1-C2	2.5	2.5
C3-C4	2.5	1
D	2.5	1

Table A.12: Carbon footprint (kg CO_2 eq.) for a reference period of 200 years, TL=100y, SL=80y

Stage	Traditional concrete	Circular concrete	Timber - recycling	Timber - incineration	Timber - landfill
A1-A3	166110	92176	-63557	-63557	-63557
A4-A5	16253	22778	12112	12112	12112
C1-C2	6593	3666	0	0	0
C3-C4	755	447	106667	110183	121644
D	-45365	-23090	-1095	-77232	-10302
Total incl D	144345	101476	50970	-21633	56815
Total excl D	189710	124566	52066	55599	67117

Table A.13: Multiplication factor for reference period of two hundred years, with technical lifespan of two hundred years for concrete and one hundred for timber

Stage	Traditional concrete	Circular concrete	Circular timber
A1-A3	2.5	1	2
A4-A5	2.5	2.5	2.5
C1-C2	2.5	2.5	2.5
C3-C4	2.5	1	2
D	2.5	1	2

Table A.14: Carbon footprint (kg CO_2 eq.) for a reference period of 200 years. Concrete: TL=200y, SL=80y and timber: TL=100y, SL=80y

Stage	Traditional concrete	Circular concrete	Timber - recycling	Timber - incineration	Timber - landfill
A1-A3	166110	92176	-127114	-127114	-127114
A4-A5	16253	22778	12112	12112	12112
C1-C2	6593	9165	0	0	0
C3-C4	755	447	213333	220366	243288
D	-45365	-23090	-2191	-154465	-20604
Total incl D	144345	101476	88979	-56257	100545
Total excl D	189710	124566	91169	98208	121149

B
Mechanical properties

B.1. Beam material

The values as presented in Table B.1 originate from:

- Glulam: NEN-EN 14080 (2013)
- LVL: Stora Enso (2020b)

Table B.1: Mechanical properties of beam materials

Property	Unit	GL24h	GL28h	GL32h	LVL-S
$f_{m,k}$	N/mm^2	24	28	32	44
$f_{t,0,k}$	N/mm^2	19.2	22.3	25.6	35
$f_{t,90,k}$	N/mm^2	0.5	0.5	0.5	0.8
$f_{c,0,k}$	N/mm^2	24	28	32	35
$f_{c,90,k}$	N/mm^2	2.5	2.5	2.5	6
$f_{v,k}$	N/mm^2	3.5	3.5	3.5	2.2
$E_{0,mean}$	N/mm^2	11500	12600	14200	13800
$E_{90,mean}$	N/mm^2	300	300	300	0
$G_{0,mean}$	N/mm^2	650	650	650	600
$G_{90,mean}$	N/mm^2	65	65	65	16
ρ_{mean}	kg/m^3	420	460	490	510
ρ_k	kg/m^3	385	425	440	480

Figure B.1 presents the comparison of the mechanical properties of the three beam materials. They are all presented relative to the properties of GL28h, which is set at 100%. When the strength parallel to the grain is considered, it is clearly shown that LVL performs best, followed by GL32h, GL28h and GL24h. However, when the properties perpendicular to the grain are considered, LVL performs poor. LVL is not able to provide any stiffness or strength perpendicular to the grain.

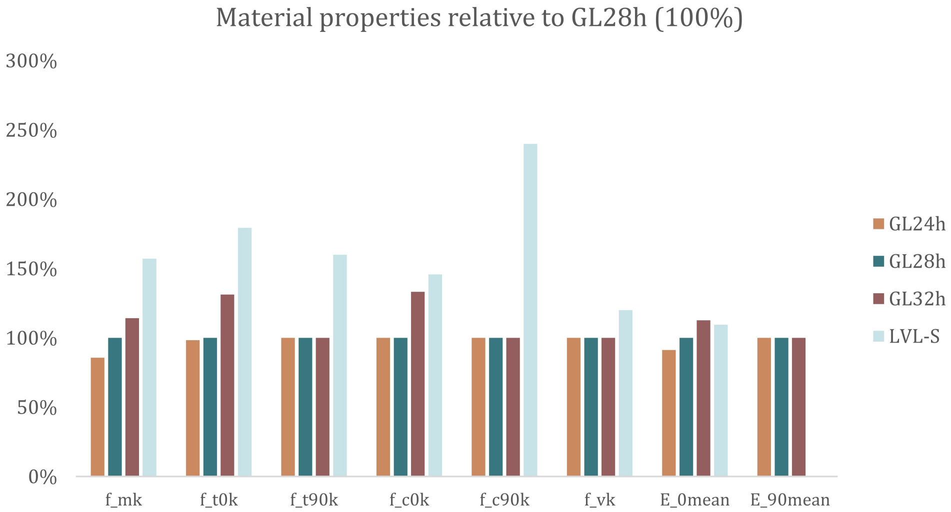


Figure B.1: Relative mechanical properties of GL24h, GL28h, GL32h and LVL

B.2. Deck material

As stated in Section 7.4.2, five options for deck materials are researched:

- Cross Laminated Timber
- Laminated Veneer Lumber
  - Parallel orientated
  - Cross-wise orientated
- Stress Laminated Timber
  - Made from sawn wood
  - Made from laminated timber

The mechanical properties of the materials are described in Table B.1. To derive the stiffness properties of the cross-laminated deck materials, calculations have to be performed. The calculations are similar for CLT panels and LVL-X panels, but with different stiffness parameters. The calculation for the CLT panel is described in detail, followed by a concise description for LVL.

The build-up of a cross-laminated panel is shown in Figure B.2. An excel file is used to calculate the mechanical properties of this CLT panel. The panel is symmetrical over the height, the top half of the layers are shown in Table B.2.

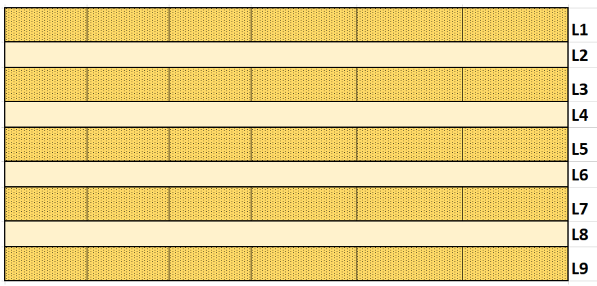


Figure B.2: Build-up of CLT deck with nine layers

Table B.2: Build-up of upper part of CLT deck

Layer	Thickness (mm)	Direction
L1	40	main
L2	30	cross
L3	40	main
L4	30	cross
L5	40	main

To calculate the mechanical properties of the CLT panel, a calculation is made for the upper half, and is multiplied by two in the end. In this case, i is between 1 and 5.

$$e_i^2 = ((0,5 \cdot t_i) + t_{i+1} + t_{i+2} + t_{i+3} + t_{i+4} + (0,5 \cdot t_{i+5}))^2 \tag{B.1a}$$

$$E_i A_i = E_i \cdot t_i \cdot 1000 \tag{B.1b}$$

$$E_i A_i e_i^2 = E_i A_i \cdot e_i^2 \tag{B.1c}$$

$$E_i I_i = E_i \cdot 1000 * (t_i^3)/12 \tag{B.1d}$$

$$(EI)_{ef,strong} = 2 \cdot \left(\sum_{i=4} E_i A_i e_i^2 + E_i I_i \right) + E_5 A_5 e_5^2 + E_5 I_5 \tag{B.1e}$$

$$G_i A_i = G_i \cdot t_i \cdot 1000 \tag{B.1f}$$

$$(GA)_{ef,strong} = 0.25 \cdot \left(2 \cdot \left(\sum_{i=4} G_i A_i \right) + G_5 I_5 \right) \tag{B.1g}$$

The same calculations are performed for the weak direction, where the E and G are switched for the layers in different directions.

To convert these mechanical properties to input values for Karamba3D, further calculations are performed. Again, the same is performed for the weak direction.

$$I_{kar} = \frac{1}{12} \cdot 1000 \cdot \left(\left(2 * \sum (t_1 ... t_4) + t_5 \right)^3 \right) \tag{B.2a}$$

$$kA_{kar} = \frac{5}{6} \cdot 1000 \cdot \left(\left(2 * \sum (t_1 ... t_4) + t_5 \right) \right) \tag{B.2b}$$

$$E_{kar,strong} = \frac{(EI)_{ef,strong}}{I_{kar}} \tag{B.2c}$$

$$G_{kar,strong} = \frac{(GA)_{ef,strong}}{kA_{kar}} \tag{B.2d}$$

To obtain the properties of the panels, the mechanical properties for C24 are used, as shown in Table B.3.

Table B.3: Properties of C24

Property	Value	Unit
E_0	10800	N/mm^2
E_{90}	370	N/mm^2
G_0	690	N/mm^2
G_{90}	69	N/mm^2

The properties for CLT are obtained as presented in Table B.4.

Table B.4: Properties of CLT panel

Property	Value	Unit
$(EI)_{ef,strong}$	$2.173 \cdot 10^{13}$	Nmm^2
$(EI)_{ef,weak}$	$8.770 \cdot 10^{12}$	Nmm^2
$(GA)_{ef,strong}$	$3.657 \cdot 10^{7}$	N
$(GA)_{ef,weak}$	$2.277 \cdot 10^{7}$	N
$E_{ef,strong}$	$7.958 \cdot 10^{3}$	N/mm^2
$E_{ef,weak}$	$3.212 \cdot 10^{3}$	N/mm^2
$G_{ef,strong}$	$1.371 \cdot 10^{2}$	N/mm^2
$G_{ef,weak}$	$8.539 \cdot 10^{1}$	N/mm^2

For the modular system, a stiffer deck is required. Therefore, the configuration of Table B.5 is applied. The same configuration is applied to the LVL-X panel, resulting in the properties as stated in Table B.7. The mechanical properties of LVL are stated in Table B.6.

Table B.5: Build-up of upper part of CLT deck

Layer	Thickness (mm)	Direction
L1	40	main
L2	40	main
L3	30	cross
L4	40	main
L5	30	cross

Table B.6: Properties of LVL

Property	Value	Unit
E_0	13800	N/mm^2
E_{90}	0	N/mm^2
G_0	460	N/mm^2
G_{90}	0	N/mm^2

Table B.7: Properties of CLT and LVL-X panel

Property	CLT	LVL-X	Unit
$(EI)_{ef,strong}$	$2.921 \cdot 10^{13}$	$3.718 \cdot 10^{13}$	Nmm^2
$(EI)_{ef,weak}$	$1.140 \cdot 10^{13}$	$1.362 \cdot 10^{13}$	Nmm^2
$(GA)_{ef,strong}$	$4.295 \cdot 10^{7}$	$2.760 \cdot 10^{7}$	N
$(GA)_{ef,weak}$	$1.829 \cdot 10^{7}$	$1.035 \cdot 10^{7}$	N
$E_{ef,strong}$	$9.753 \cdot 10^{3}$	$1.241 \cdot 10^{4}$	N/mm^2
$E_{ef,weak}$	$3.807 \cdot 10^{3}$	$4.547 \cdot 10^{3}$	N/mm^2
$G_{ef,strong}$	$1.562 \cdot 10^{2}$	$1.004 \cdot 10^{2}$	N/mm^2
$G_{ef,weak}$	$6.649 \cdot 10^{1}$	$3.764 \cdot 10^{1}$	N/mm^2

C

Options for connection design

C.1. Beam-beam connection

C.1.1. Moment resisting connection

Slotted-in steel plate

Table C.1: Behaviour on criteria slotted-in steel plate

Amount of labour required	-	A relative high number of fasteners is required, which increases the labour.
Possibility for reuse	++	Good, as the bolts can be demounted easily, and the steel plates are not glued to the timber.
Structural efficiency	++	High due to combination of fasteners and steel plates.
Stiffness	++	Due to thecombination of wide spacing of fasteners and steel plates the stiffness is high.
Amount of steel required	-	By applying one or more steel plates, relatively high amount of steel is applied.
Reliability	++	Extensive knowledge is available on this connection, ductile behaviour can be obtained thus the connection will not fail unexpectedly.

Glued-in steel rods

Table C.2: Behaviour on criteria glued-in steel rods

Amount of labour required	++	The inserting of the rods into the timber can be done in the factory. Only the assembly of the rods to the steel plates must be done on site, this is done by nuts. Therefore, limited labour is required on site.
Possibility for reuse	++	Good, only the rods must be disassembled from the steel coupler, which is done by removing the nuts.
Structural efficiency	-	Moderate, the rods are stiff, but have concentrated load transfer to the timber.
Stiffness	++	High as a bending stiffness can be obtained by the lever arms between the rods. Furthermore, almost no slip occurs due to the glued connection between the rods and the timber (Harvey and Ansell, 2000).
Amount of steel required	-	Relatively high due to combination of rods and steel plates.
Reliability	-	Limited knowledge available on the actual behaviour of the joint due to the interactions between the three materials (Harvey and Ansell, 2000). Furthermore, the glued-in rods connection is not present in Eurocode on timber connections. Glued-in rods are likely to have brittle failure between the glue and timber.

End-plate connection

Table C.3: Behaviour on criteria end-plate connection

Amount of labour required	++	Low amount of labour required as only the bolts in the end plates have to be assembled.
Possibility for reuse	++	High possibility for demounting as only the bolts in the end plates should be demounted.
Structural efficiency	+	Good, as the steel plates in the timber and the end plates work well together.
Stiffness	-	Moderate as it all comes down to the end plates. Furthermore, brittle failure is likely to occur due to the welding of the steel plates in the timber members to the end plates.
Amount of steel required	- -	Very high because of the combination of end plates and steel in the timber members.
Reliability	-	For the application in timber structures, limited knowledge is available and it is not widely applied. Again, glued-in rods are likely to have brittle failure between the glue and timber.

C.1.2. Hinged connections

Joist hangers

Table C.4: Behaviour on criteria joist hanger connection

Amount of labour required	++	Low, as the beam only needs to be placed into the hanger and the bolts have to be assembled.
Possibility for reuse	++	High possibility for demounting and reuse as only the bolts in the joist hanger should be demounted.
Structural efficiency	+	Good, as the steel can transfer the forces in an efficient manner.
Stiffness	-	Some slip occurs between the steel hanger and the continuous beam.
Amount of steel required	- -	Relatively high as fasteners are applied, as well as the steel brackets or hangers.
Reliability	++	High, as it is often applied in practice. Ductile behaviour can be ensured.

Interlocking joints

Table C.5: Behaviour on criteria joist hanger connection

Amount of labour required	+	With digital fabrication, the amount of labour required is low.
Possibility for reuse	++	High, the joints can just be taken apart.
Structural efficiency	-	Moderate, reversed loads will occur, as well as uplift in the timber members. Therefore, the connection should withstand compression, tension and shear forces which can cause problems for the interlocking joints.
Stiffness	-	Slip can occur between the timber members, as some tolerances are also needed for changes in moisture content.
Amount of steel required	++	None.
Reliability	- -	The application into modern structures and the digital fabrication is in development, but not applied often. Failure is hard to predict.

C.2. Panel-panel connections

Screws

Table C.6: Behaviour on criteria deck panel screwed connection

Amount of labour required	+	A lot of screws have to be assembled, but a it is a simple connection.
Possibility for reuse	- -	Demounting is not possible
Structural efficiency	-	Moderate, all forces that need to be transferred need to go through the screw.
Stiffness	- -	High deformation can occur as screws are not that stiff.
Amount of steel required	+	Only the screws are made of steel.
Reliability	++	An often applied solution in floor and deck design. Failure can be predicted well.

Slotted-in steel plate

Table C.7: Behaviour on criteria glued-in rods with tube connection

Amount of labour required	++	Low, only the tubes have to be assembled by two nuts.
Possibility for reuse	+	Only the bolts have to be demounted and remounted. The connection is stiff so deformations in the bolts will stay limited.
Structural efficiency	++	High due to combination of bolts and steel plates.
Stiffness	++	Similar to efficiency, high due to combination of bolts and steel plates.
Amount of steel required	-	The bolts and steel plate are made out of steel.
Reliability	++	Slotted-in steel plate is an often applied connection in timber design, clear design codes are defined. Failure can be predicted well.

X-RAD connection

Table C.8: Behaviour on criteria X-RAD connection

Amount of labour required	+	Low, only the tubes have to be assembled by nuts.
Possibility for reuse	++	The bolts can be demounted well, it is already often applied as a demountable connection.
Structural efficiency	+	Efficient load transfer in the X-RAD component.
Stiffness	+	The X-RAD component is relatively stiff due to its geometry.
Amount of steel required	-	The X-RAD component is made out of steel.
Reliability	-	Limited knowledge on applying it on larger cross sections. Product is intended to be used in modular building design and thus on a smaller scale.

C.3. Shear connections

C.3.1. Dowels

Table C.9: Behaviour on criteria dowelled deck-girder connection

Amount of labour required	- -	Very high, as a lot of dowels should be applied.
Possibility for reuse	+	Can be demounted, although it requires a lot of labour to demount all the dowels.
Structural efficiency	-	Not very efficient, as dowels have a relative low strength.
Stiffness	-	Relatively low due to the slip that occurs in every dowel.
Amount of steel required	-	Depending on the material of the dowels, quite a lot of steel is required. Steel dowels are stiffer and stronger than timber ones so that can be desirable.
Reliability	++	The behaviour of dowelled connections is well known. Failure can be predicted well.

C.3.2. Inclined screws

Table C.10: Behaviour on criteria screwed deck-girder connection

Amount of labour required	+	The inclined screw connection is stiffer compared to the dowelled connection, thus a lower number of fasteners is required. Consequently the amount of labour will also be reduced.
Possibility for reuse	- -	Reuse is not possible as the screws can not be assembled again. Furthermore, detaching the screws is hard for long screws.
Structural efficiency	+	Relatively efficient, as screws are stronger than other mechanical fasteners (dowels).
Stiffness	+	Still some slip occurs, but significantly lower compared to dowels.
Amount of steel required	+	The screws are made out of steel, but can have a smaller diameter than dowels.
Reliability	++	The behaviour of inclined screwed connections is well known. Failure can be predicted well.

C.3.3. Glued connection

Table C.11: Behaviour on criteria screwed deck-girder connection

Amount of labour required	++	Can be fully pre-fabricated.
Possibility for reuse	- -	Not possible
Structural efficiency	+	Glue has a high strength, thus the connection is stronger than the timber. However, the strength of the deck panel will become critical, as high tension and compression forces arise in the deck.
Stiffness	++	No slip occurs
Amount of steel required	++	No steel is applied.
Reliability	- -	Well known in smaller structures, but not common in bridge design. Failure is likely to be brittle and thus hard to predict.

Overview of connection study

Table C.12: Overview moment-resisting beam-beam connection

Criterion	Slotted-in steel plate	Glued-in rods	End plate
Amount of labour required	-	++	++
Possibility for reuse	++	++	++
Structural efficiency	++	-	+
Stiffness	++	++	-
Amount of steel required	-	-	- -
Reliability	++	-	-
Score	47	40	33

Table C.13: Overview hinged beam-beam connection

Criterion	Joist hanger	Interlocking joint
Amount of labour required	++	+
Possibility for reuse	++	++
Structural efficiency	+	-
Stiffness	-	-
Amount of steel required	- -	++
Reliability	++	- -
Score	40	33

Table C.14: Overview panel-panel connection

Criterion	Screws	Slotted-in steel plate	X-RAD
Amount of labour required	+	++	+
Possibility for reuse	- -	+	++
Structural efficiency	-	++	+
Stiffness	- -	++	+
Amount of steel required	+	-	-
Reliability	++	++	-
Score	27	50	37

Table C.15: Overview shear connection for composite behaviour

Criterion	Dowels	Inclined screws	Glue
Amount of labour required	- -	+	++
Possibility for reuse	+	- -	- -
Structural efficiency	-	+	+
Stiffness	-	+	++
Amount of steel required	-	+	++
Reliability	++	++	- -
Score	27	37	37

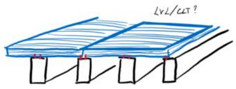
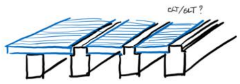
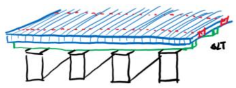
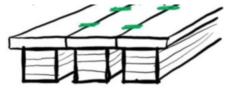
D

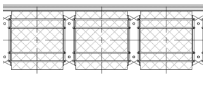
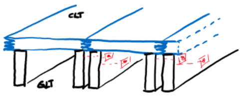
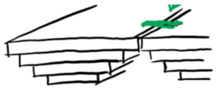
Trade-off matrix Arup

Voorlopig

TRADE-OFF-MATRIX	**: BOLT, bovenbouw opties**	**Legend**	
Projectnummer	:	**score / risk**	**Weight**
Objectnaam	:		
Objectomschrijving	: Houten bovenbouw van langsliggers/dek	(++) favourable / none	3
Versie	: 1.0	(+) neutral / small	2
Documentnummer	:	(-) unfavourable / medium	1
Datum	: 2-12-2020	(--) negative / large	0

160 180

VARIANTEN :	**Weegfactor**	**Optie 1**	Score	Rating	Score	Rating	**Optie 2**	Score	Rating	**Optie 3**	Score	Rating	**Optie 4**	Score	Rating
															
Omschrijving		**Dek op block glulam liggers** Dekplaten op block glulam langsliggers (ca. 1m breed, hoh 2m). Platen in delen in langs- of dwarsrichting. Mogelijke samenwerking tussen dek en ligger	**Platen in langsrichting**		**Platen in dwarsrichting**		**Dek tussen block glulam liggers** Dekplaten tussen de block glulam langsliggers. Dek werkt niet samen in langsrichting.			**Voorgespannen dek op dwarsliggers die op of tussen block glulam liggers** SLT dek op dwarsdragers, die op hun beurt weer op of tussen block glulam langsliggers liggen. Geen samenwerking tussen dek en ligger			**Massieve block glulam liggers** Block glulam langsliggers met een (gelijmde of losmaakbare) meewerkende deklaag die zij aan zij liggen (met koppeling voor krachtsoverdracht in dwarsrichting)		
CRITERIUM :															
Technische Haalbaarheid															
Technology Readiness Level (TRL) Zijn er al voorbeelden van toepassing in de wereld Is het al onderdeel van onderzoek derden	**5**	Er zijn niet of nauwelijks voorbeelden van deze toepassing in infra, maar dit is wel standaard opbouw voor hout-beton composiet verkeersbruggen. Een houten CLT dek wordt wel vaker gebruikt om in dwarsrichting te overspannen. Bij de Mistissini verkeersbrug in Canada (40m overspanningen) ook een houten dek op slanke glulam houten liggers toegepast. Dekplaten in dwarsrichting geplaatst en met diagonale schroeven vast gezet. Daar is niet van samenwerking uitgegaan.	+	10	+	10	Op deze manier een houten dek met een houten ligger verbinden is nog niet eerder gezien. Kleine ruimte voor verbinding	-	5	SLT is een standaard deksysteem voor veel voetgangersbruggen, maar ook wel voor rurale verkeersbruggen.Post-tensioning van het dek wordt wel regelmatig toegepas bij kleinere verkeersbruggen als zelfdragende consdtructie en als dek op dwarsliggers.	++	15	Block glulam wordt regelmatig gebruikt bij (kleinere) verkeersbruggen. Wanneer meerdere blocklam liggers naast elkaar worden geplaatst, wordt meestal gebruik gemaakt van een betonnen dek (hout-beton composiet bruggen).	+	10
Sterkte Dient de globale belastingen te dragen Dient de lokale belastingen op het dek te dragen Verbinding zorgt voor composiet gedrag Vermoeiingsgevoeligheid verbinding	**10**	Met dek in langsrichting kan het goed samenwerken bij de juiste detailering van de verbinding. Volledige samenwerking is eigenlijk alleen bereikbaar met een verlijmde verbinding. Bij dek in dwarsrichting is geen samenwerking in langsrichting mogelijk. Maar juist weer wel goede spreiding van belasitng over meerdere liggers. Verbinding zijn in het algemeen vermoeiingsgevoelig, Dek zal nabij de hoofdligger onderhevig zijn aan spanningen in meerdere richtingen. Langsspanningen door globale werking en dwarsspanning en schuif door lokale afdracht. Hoge wiellast op CLT dek is een risico vanwege lage 'rolling shear' canciteit.	-	10	-	10	Bijna geen afdracht van krachten op het dek naar hoofdligger. Geen composiet gedrag mogelijk. Verbinding veel op buiging belast, waardoor zeer vermoeiingsgevoelig	--	0	Goede en gelijke verdeling van deklasten over de dwarsliggers naar de hoofdliggers. Eenvoudige verbinding met weinig vermoeiingsgevoeligheid. Door werking van het hout en vooral dwars op de vezel treedt er voorspanningsverlies op. Dit is een risico voor de werking van het SLT dek. Spanningsconcentratie op aansluitvlak tussen primaire en secundaire liggers is een aandachtspunt.	+	20	De globale belasting zal eenvoudig voldoen, maar de lokale belasting bij de voeg kan een probleem worden. De verbinding wordt zwaar belast en ook op vermoeiing, verdient dus nog veel aandacht.	-	10
Vervorming Vervorming binnen de eisen Lokale zakking tussen liggers Beperking trillingen (comfort voor voetgangers) Langdurig gedrag (krimp, zwel, kruip)	**5**	Samenwerking is gunstig voor beperken doorbuiging in langsrichting. Dit is bij dek in langsrichting mogelijk. Verbinding is in het algemeen echter lastig zo stijf te krijgen dat goede samenwerking wordt bereikt. Bij dek in dwarsrichting zorgen de dwarsnaden dat samenwerking niet op treed. Langsnaden dienen zo gedetaileerd te zijn dat ze in dwarsrichting stijf zijn voor spreiding belasting over meerdere langsliggers. Bij dek in dwarsrichting gaat dit makkelijker. Hout heeft in langsrichting glulam beperkte krimp/zwel, een CLT dek is ook zeer krimp/zwel arm in beide richtingen.	-	5	-	5	Geen goede samenwerking tussen liggers en dek te behalen. Geen stijf dek in dwarsrichting waardoor geen spreiding van dek belasting over meerdere liggers. Tenzij extra dwarsliggers worden toegevoegd. Lage totaal stijfheid dus lage frequentie en mogelijk in gevoeliggebied voor hinderlijke trilling/versnellingen. Indien dek GLT in dwarsrichting dan verschil in krim/zwel tov GLT liggers.	--	0	Geen samenwerking dek en liggers, dus niet stijf. Dwarsliggers spreiden belasting over meerdere liggers, dat is gunstig. Door krimp zal er voorspanningsverlies ontstaan en risico op slip en vervorming tussen dek dekbalken. Lage totaal stijfheid dus lage frequentie en mogelijk in gevoeliggebied voor hinderlijke trilling/versnellingen	--	0	Balk geprefabriceerd, dus door gelijmde verbinding goede samenwerking van dek en ligger. Met goed dwarskracht verbinding is er spreiding van verticale belasting tussen liggers. Zonder moment verbinding wel risico op torsie. Mogelijk zwaardere koppeling nodig.	+	10
Levensduur Hoofdligger 100jr, Dek >50jr Toepassing waterdicht membraam Onderhoud intensiteit	**10**	Hoofdliggers zijn beschermd voor weer en wind, met behandeling levensduur >100jr (spray van onderliggend verkeer en dooizouten mogelijk een issue). Dek dient te worden bescherm door een waterdichtmembraam (mogelijk nog een 2de bescherming in de vorm van behandeling voor levensduur >50jr). Naden zijn mogelijk een risico voor levensduur membraam als daar lokale vervormingen zijn.	+	20	+	20	In dit ontwerp zijn de hoofdliggers ook niet beschermd door het dek. Dus zal een 100jr levensduur van de hoofdligger een grote uitdaging zijn. De hoofdliggers zullen ook onderling verplaatsen en daardoor zal het membraan snel scheuren. Ook is de opening tussen dek en ligger een locatie waar vocht kan blijven hangen.	--	0	Goede garantie voor 100 jaar levensduur van de hoofdliggers door de 'cavity', maar het dek zal een grotere uitdaging zijn. De voorspanning zal gedurende de jaren afnemen door kruipen van hout. De dwarsdragers zullen daarentegen een goede beschermlaag zijn voor mogelijk vochtophoping tussen dek en ligger (vanwege cavity), wat vaak een probleem is. Kopse kant van dwarsdrager dient wel beschermd te worden.	++	30	Door onderlinge vervormingen van de hoofdliggers zou het membraan kunnen scheuren. Daarentegen is er geen mogelijkeid tot vochtophoping tussen dek en ligger, wat voordelig is. Een nadeel is de integraliteit van hoofdligger en dek. Indien lokaal het dek vervangen dient te worden, zal de hoofdligger ook vervangen worden.	-	10
Uitvoerbaarheid Prefabricatie van onderdelen Transport Materieel	**10**	Liggers kunnen in 1 lengte worden getranporteerd. Dek kan in delen. De detailering van de verbindingen is belangrijk. Hout is relatief licht dus is lichter materieel nodig dan bij beton.	+	20	+	20	Vergelijkbaar met Optie 1 is het een eenvoudige uitvoering, waar de liggers in de volledige lengte kunnen worden getransporteerd en geplaatst.	+	20	De onderdelen kunnen geprefabriceerd worden en eenvoudig getranporteerd worden. Nadelig ten opzichte van de voorgaande opties is nog een liggerconstructie. Dit zal voor een langer installatietijd zorgen. Daarnaast moet het dek ook voorgespannen worden. wat meer tijd zal kosten.	-	10	Een bijna volledig geprefabiceerd ontwerp waar alleen de verbinding geplaatst moet worden.	++	30
Economisch perspectief															
Kosten Bouwkosten (bovenbouw en gehele systeem) Onderhoudskosten Afschaffingskosten	**15**	Bouwkosten: materiaal gebruik vergelijkbaar met andere balk dek oplossingen: score neutraal Onderhoud: vergelijkbaar met andere ligger - dek oplossingen : score neutraal	+	30	+	30	Bouwkosten: materiaal gebruik vergelijkbaar met andere balk dek oplossingen: score neutraal Onderhoud: vergelijkbaar met andere ligger - dek oplossingen : score neutraal	+	30	Bouwkosten: meer materiaal in dek, complexer monteren (boven verkeer ?): score - Onderhoud: Dwarsvoorspanning zal mogelijk vaker gecontroleerd en nagespannen moeten worden: score -	-	15	Bouwkosten: materiaal gebruik vergelijkbaar met andere balk dek oplossingen: score neutraal Onderhoud: vergelijkbaar met andere ligger - dek oplossingen : score neutraal	+	30
Baten Schaalbaarheid, markt grote (ruim inzetbaar, ook andere opdrachtgevers)	**15**	Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30	+	30	Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30	Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30	Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30
Duurzame Impact															
Circulariteit Beperking afval en vervuiling (MKI en CO2) Aanpasbaar tijdens levensduur Herbruikbaar einde levensduur Regeneratief voor ecosystemen	**20**	Zolang de dekplaat niet in dwarsrichting over volledige breedte ligt, kan makkelijk dek en liggers worden verwijderd en toegevoegd, zonder al het verkeer van de brug te halen. Hoofdliggers hebben lange levensduur en zijn makkelijk her te gebruiken bij de juiste losmaakbare verbindingen. Het dek is bij gebruik en detailering van waterdichtmembraam ook een redelijke lange levensduur en kan worden hergebruikt. Mogelijk niet als dek, maar als andere functie in de bouw.	++	60	+	40	Levensduur opmerking bovenstaand behoorlijk bepalend. Kleine elementen is meer verbindingen. Aangezien deze wellicht niet van hout zijn, zal daar dan een relatief hogere impact aan zitten. In breedte makkelijk aanpasbaar zonder al het verkeer van de brug te halen. Hoofdliggers hebben lange levensduur en zijn makkelijk her te gebruiken bij de juiste losmaakbare verbindingen. Het dek is bij gebruik en detailering van waterdichtmembraam ook een redelijke lange levensduur en kan worden hergebruikt. Mogelijk niet als dek, maar als andere functie in de bouw.	+	40	Wellicht voordeel dat doordat dek en liggers niet samenwerken er ook beter gedemonteerd kan worden(ntb). Nadeel is dat er meer materiaal benodigd is. In breedte wel aanpasbaar, maar dan moet de dwarsvoorspanning worden ontkoppeld en kan er geen verkeer op brug. Hoofdliggers hebben lange levensduur en zijn makkelijk her te gebruiken bij de juiste losmaakbare verbindingen. Het dek is bij gebruik en detailering van waterdichtmembraam ook een redelijke lange levensduur en kan worden hergebruikt. Mogelijk niet als dek, maar als andere functie in de bouw.	+	40	Elementen goed te demonteren mocht het 'dekdeel sneller slijten dan de ligger. Verbinding dek-ligger is niet demonteerbaar. Bij verwijderen dek van ligger is de ligger weer inzetbaar voor andere functie als balk in de bouw. Voor dek is waarschijnlijk minder hoogwaardig hergebruik mogelijk. In breedte makkelijk aanpasbaar zonder al het verkeer van de brug te halen.	-	20
Hinder Te bouwen met beperkte hinder verkeer / omgeving Aanpasbaar met beperkte verkeershinder	**10**	Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden, aandachtpunt zijn de dekplaten waardoor meer stops nodig om : score 0 (neutraal) Aanpasbaarheid: makkelijk uit te breiden door toepassen losse liggers die onderling te koppelen zijn: score +	+	20	+	20	Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden, aandachtpunt zijn de dekplaten waardoor meer stops nodig om : score 0 (neutraal) Aanpasbaarheid: makkelijk uit te breiden door toepassen losse liggers die onderling te koppelen zijn: score +	+	20	Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden. Echter de de stapeling met dwarsbalken en dekplaten leidt tot meer hijsacties. Het dwarsvoorspannen boven de onderliggende weg met een spankar is ivm veiligheid ongewenst, bijvoorkeur rijstroken afsluiten Aanpasbaarheid: Door aanwezigheid dwarsvoorspanning is verbreden lastiger, de spankoppen moeten waarschiojnlijk altijd geinspecteerd en nagespannen worden waardoor verlengen niet eenvoudig is. Vervangen dwarsvoorspannign betekent dat verkeer van het dek moet: score -	--	0	Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden: score + Aanpasbaarheid: makkelijk uit te breiden door toepassen losse liggers die onderling te koppelen zijn: score +	++	30
TOTAAL	**100**														
Score :				205		185			145			160			180
Beoordeling : :															
Voorkeur :															

185			185			165			180					
Optie 5			**Optie 6**			**Optie 7**			**Optie 8**			**Optie 9**		
	Score	Rating		Score	Rating		Score	Rating		Score	Rating		Score	Rating
Gekoppelde langsliggers met secundaire dekplaat Gelamineerde houten liggers gekoppeld met stalen elementen (massief of kokers) + secundaire CLT deklaag (dun) in dwarsrichting.			**prefab n- of m-liggers** CLT dek gelijmd op twee of drie GLT balken. Dek werkt samen met liggers in langsrichting. Koppeling van elementen voor spreiding bealsting in dwarsrichting.			**Massieve block glulam platen, trapezium** In trapezium vorm gelamineerde platen, breedte 2-3m. Koppeling voor beheersing verschil doorbuiging. Platen liggen op 2 oplegblokken per zijde voor rotatiestijfheid.			**Massieve block glulam platen, rechthoekig** In rechthoekige vorm gelamineerde platen, breedte 2-3m. Eenvoudige koppeling voor beheersing verschil doorbuiging. Platen liggen op 2 oplegblokken per zijde voor			**Houten boogbrug** 50m (dubbele overspanning) Houten boogbrug met SLT dek op dwarsdragers		
Dergelijke oplossing nog niet tegen gekomen. Deze verbinding tussen de liggers ook nog niet eerder op deze manier gezien, maar stalen verbindingenbij hout constructies zijn wel een bewezen concept.	-	5	Redelijk vergelijkbaar principe is in het verleden op enkele locaties toegepast voor bruggen. Soms met een glt dek tussen de hoofdliggers met dwarsvoorspanning. En zoms ook met een onderflens, waardoor het gesloten kokers worden.	+	10	Grote gelamineerde platen worden af en toe toegepast voor fietsbruggen, zoals bijvoorbeeld voor een fietsbrug te Leeuwarden. Deze bestaan dan uit 1 massief blok met daarop een losse of vaste rijdek van beton. Het samenvoegen van deze platen tot een verkeersdek komt nog niet voor.	+	10	Grote gelamineerde platen worden af en toe toegepast voor fietsbruggen, zoals bijvoorbeeld voor een fietsbrug te Leeuwarden. Deze bestaan dan uit 1 massief blok met daarop een losse of vaste rijdek van beton. Het samenvoegen van deze platen tot een verkeersdek komt nog niet voor.	+	10	Wordt veel toegepast voor verkeersbruggen (vooral in Noorwegen).	++	15
Sterk hoofddraagsysteem met een sterke verbinding. Verbinding verdient nog veel aandacht. Er is slechte toegang voor inspectie en onderhoud tijdens gebruik. Verbiningen zijn in het algemeen vermoeiingsgevoelig.	+	20	Sterk ontwerp als dek in fabriek vast aan ligger kan worden gemaakt (bv gelijmd), dan goede samenwerking dek en liggers. De verbinding verdient nog aandacht om goede spreiding van belasting in dwarsrichting mogelijk te maken. De verbinding kan wel robust in de liggers plaats vinden. Mogelijk problemen met asbelasting op de dekverbinding. Hoge wiellast op CLT dek is een risico vanwege lage 'rolling shear' capciteit.	-	10	Afzondelijke platen zullen makkelijk de krachtenafdracht aankunnen. Smallere onderflens is wat ongunstig in vergelijk met optie 8. De verbinding lijkt zwak en zal doorslaggevend zijn. Door platen op 2 oplegblokken te leggen is deze rotatie stabiel, wel torsie in ligger bij wiellast nabij naad.	-	10	Afzondelijke platen zullen makkelijk de krachtenafdracht aankunnen, maar de verbinding dient nog verder onderzocht te worden. Wel voldoende materiaal voor een dwarskracht verbining. Door platen op 2 oplegblokken te leggen is deze rotatie stabiel.	+	20	Het ontwerp kan de krachtenafdracht zowel lokaal als globaal aan. Dragende elementen voornamelijk in druk belast, waardoor weinig vermoeiingsgevoeligheid. Bij brede dekken is een extra boog nodig om de dwarsdrager te ondersteunen. Er dient rekening te worden gehouden met hanger uitval (door aanrijding of onderhoud)	++	30
Vele elementen resulteerd in grotere stijfheid per m2, dus gunstig voor beperken doorbuiging. Robuste verbinding zorgt voor spreiding van belasting over meerdere liggers. Is wel relatief zwaar, maar nog steeds licht in vergelijking tot de verkeersbelasting.	++	15	Balk geprefabriceerd, dus door gelijmde verbinding goede samenwerking van dek en ligger. Gunstig voor beperken doorbuiging. Met goed dwarskracht verbinding is er spreiding van verticale belasting tussen liggers. Zonder moment verbinding wel risico op torsie. Mogelijk zwaardere koppeling nodig, bijvoorbeeld door dubbele rij verbinding en enkele dwarsbalken.	++	15	Massieve liggers zijn stijve elementen resulteerd in grotere stijfheid per m2,, dus gunstig voor beperken doorbuiging. Smallere onderflens is wat ongunstig in vergelijk met optie 8. Voor samenwerking tussen de liggers is een dwarskracht verbinding noodzakelijk. Dit is uitvoerbaar. Dubbele oplegging onder plaat is deze rotatie stijf, wel torsie in plaat bij wiellast op naad. Mogelijk een dwarskoppeling nodig om dit te beheersen.	+	10	Massieve liggers zijn stijve elementen resulteerd in grotere stijfheid per m2,, dus gunstig voor beperken doorbuiging. Voor samenwerking tussen de liggers is een dwarskracht verbinding noodzakelijk. Dit is uitvoerbaar. Dubbele oplegging onder plaat is deze rotatie stijf. Is wel relatief zwaar, maar nog steeds licht in vergelijking tot de verkeersbelasting.	++	15	Een boogconstructie is stijf en goed om de globale doorbuiging te beperken. Afhankelijk van de keuze van dwarsdragers en dek is ook locale doorbuiging te beheersen (zolang brug maar niet te breed wordt). Boogconstructie is stijf en bij stijve dwarsdragers zijn trillingen beperkt.	++	15
De stevige connectie tusen liggers betekent een kleinere kans op scheuren van het waterdichte membraan. Dit is een voordeel. CLT-platen zijn waarschijnlijk een goede bescherming voor de hoofdligger (wordt in dezelfde wijze toegepast in de woningbouw). De verbinding is tijdens gebruik niet bereikbaar voor inspectie en onderhoud. Daardoor groot risico voor levensduur garantie.	+	20	Er is geen mogelijkeid tot vochtophoping tussen dek en ligger, wat voordelig is. Een nadeel is de integraliteit van hoofdligger en dek. Indien lokaal het dek vervangen dient te worden, zal het geheel van dek en ligger vervangen moeten worden. Bij goede verbinding prefab elementen is een dek met weining lokale vervormingen en hierdoor langere levensduur van bovenliggende membraam.	+	20	Zonder bovenliggend dek zal 100jr levensduur voor hoofdligger een uitdaging worden. Dit omdat bij lekkage van waterdicht membraam en aantasting onderliggend hout, dan zal de hele ligger vervangen dienen te worden. Als er rotatie is tussen platen dan heeft dit effect op levensduur van bovenliggende membraam.	-	10	Zonder bovenliggend dek zal 100jr levensduur voor hoofdligger een uitdaging worden. Dit omdat bij lekkage van waterdicht membraam en aantasting onderliggend hout, dan zal de hele ligger vervangen dienen te worden. Bij goede verbinding platen is een dek met weining lokale vervormingen en hierdoor langere levensduur van bovenliggende membraam.	-	10	Boog is aanzienlijk minder goed beschermd tegen vocht ten opzichte van optie 1-3 en 5. Dit zal gevolgen hebben voor de levensduur als geen beschermende 'huid' wordt gebruikt (of anders Accoya hout). Dek is SLT, dus zal relatief hoge onderhoudsintensiteit hebben.	--	0
Een bijna volledig geprefabiceerd ontwerp waar alleen de verbinding en een extra dekplaat geplaatst moet worden. Wel meerdere liggers dan bij optie 1.	+	20	Een bijna volledig geprefabiceerd ontwerp waar alleen de verbinding geplaatst moet worden.	++	30	Een bijna volledig geprefabiceerd ontwerp waar alleen de verbinding geplaatst moet worden. Wel zwaardere elementen dan bij optie 6.	++	30	Een bijna volledig geprefabiceerd ontwerp waar alleen de verbinding geplaatst moet worden. Wel zwaardere elementen dan bij optie 6.	++	30	De uitvoering van dit ontwerp is langzaam ten opzichte van de anderen. Het zal een uitdaging zijn om een boog van 50m overspanning te kunne prefabriceren. Daarnaast zal het transport ook aanzienlijk moeilijker zijn. Dit ontwerp zal met afstand de langste installatietijd en materiaalkosten hebben.	--	0
Bouwkosten: Hoeveelheid materiaal gebruik vergelijkbaar met optie 1tm5, lijkt relatief veel stalen verbindingen te hebben : score neutraal Onderhoud: dek is evt los van ligger te vervangen: score + Onderhoud: extra onderhoud aan stalen gaffel verbindingen. Deze is echter van onderaf niet bereikbaar voor inspectie en onderhoud score --	-	15	Bouwkosten: Hoeveelheid materiaal is gunstiger tov optie 1tm5, score + Onderhoud: extra onderhoud aan stalen gaffel verbindingen score -	+	30	Bouwkosten: Hoeveelheid materiaal gebruik vergelijkbaar met optie 1tm5 : score neutraal Onderhoud: dek is niet los van ligger te vervangen, levensduur ligger is mogelijk beperkt: score -	-	15	Bouwkosten: Hoeveelheid materiaal gebruik is groter dan optie 1tm5 : score - Onderhoud: dek is niet los van ligger te vervangen, levensduur ligger is mogelijk beperkt: score -	-	15	Bouwkosten; Gekromde boogliggers is duurder en wat complexere verbinding van hangers en bij booggeboorte. Onderhoud: Dwarsvoorspanning in dek zal mogelijk vaker gecontroleerd en nagespannen moeten worden: score - Onderhoud; de beschermende huid van de bogen heeft een kortere levensduur.	-	15
Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30	Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30	Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30	Ruim inzetbaar bij verschillende opdrachtgevers in binnen en buitenland.	+	30	Zal minder vaak inpasbaar zijn voor viaducten / bruggen vanwege grotere overspanning, inzetbaar bij verschillende opdrachtgevers in binnen en buitenland. Wel meer uitgesproken (aantrekkelijke) vormgeving	+	30
Door meer massa en vele staalverbindingen waarschijnlijk iets hogere mki impact. Wellicht voordeel dat doordat dek en liggers niet samenwerken er ook beter gedemonteerd kan worden(ntb). Nadeel is dat er meer materiaal benodigd is. Hoofdliggers hebben lange levensduur en zijn makkelijk her te gebruiken bij de juiste losmaakbare verbindingen. In breedte makkelijk aanpasbaar zonder al het verkeer van de brug te halen.	+	40	Door vele staalverbindingen waarschijnlijk iets hogere mki impact. Elementen goed te demonteren mocht het 'dekdeel sneller slijten dan de ligger. Verbinding dek-ligger is niet demonteerbaar. Bij verwijderen dek van ligger is de ligger weer inzetbaar voor andere functie als balk in de bouw. Voor dek is waarschijnlijk minder hoogwaardig hergebruik mogelijk. In breedte makkelijk aanpasbaar zonder al het verkeer van de brug te halen.	-	20	Veel massa (en lijm) t.o.v. andere ontwerpen, dus meer mki impact. (grondstofverbruik). De elementen zijn demontabel en herbuikbaar. Bij lekkage door waterdichtmembraam zal de bovenlaag beschadigd zijn en eerst weggefreesd dienen te worden voor hoogwaardig hergebruik. Dus wat verlies van sterkte en bij kleinere viaducten inzetbaar.	-	20	Veel massa (en lijm) t.o.v. andere ontwerpen, dus meer mki impact. (grondstofverbruik). De elementen zijn demontabel en herbuikbaar. Bij lekkage door waterdichtmembraam zal de bovenlaag beschadigd zijn en eerst weggefreesd dienen te worden voor hoogwaardig hergebruik. Dus wat verlies van sterkte en bij kleinere viaducten inzetbaar.	-	20	Boogbrug is als geheel te transporteren en ergens anders te plaatsen. Ook makkelijk in hoogte aan te passen. Aanpasbaarheid in breedte is niet goed mogelijk.	-	20
Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden, aandachtpunt is het koppelen van de stalen verbindingen mogelijk wel meer of langere stops nodig om : score 0 (neutraal) Aanpasbaarheid: makkelijk uit te breiden door toepassen losse liggers die onderling te koppelen zijn: score +	+	20	Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden, aandachtpunt is het koppelen van de stalen verbindingen mogelijk wel meer of langere stops nodig om : score 0 (neutraal) Aanpasbaarheid: makkelijk uit te breiden door toepassen losse liggers die onderling te koppelen zijn: score +	+	20	Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden: score + Aanpasbaarheid: makkelijk uit te breiden door toepassen losse liggers zonder dwars spreiding: score +	++	30	Montage: Liggers kunnen met verkeersstops in 1 of meerdere nachten over de onderliggende weg gelegd worden: score + Aanpasbaarheid: makkelijk uit te breiden door toepassen losse liggers zonder dwars spreiding: score +	++	30	Montage: Weekend afsluiting op onderliggende weg nodig tbv inrijden boogbrug: score - Aanpasbaarheid: Bovenliggende weg niet te verbreden door aanwezigheid boog: score -	--	0
		185			185			165			180			125

E

Karamba model build-up typology study

E.1. Build-up of geometry

INPUT VALUES which can be changed

GEOMETRY
- Length [m]
- Number of spans
- Skew [gon]
- Width [m]

DIMENSION OF MODULES
- L1 [m]
- L3 [m]
- B1 [m]
- B2 [m]

CROSS SECTIONS
Main girders
- Height [m]
- Width [m]

Cross girders
- Height [m]
- Width [m]

Deck panel
- Thickness [m]

Struts
- Height [m]
- Width [m]

CONNECTIONS
DP-DP
- Diameter of bolts [mm]
- Width steel plate [mm]

MG-MG
- Diameter of bolts [mm]
- Number of steel plates

CG-CG
- Diameter of bolts [mm]
- Number of steel plates

DP-MG
- Diameter of screw [mm]
- Length of screw [mm]
- Angle of inclination [degrees]

Strut
- Diameter of bols [mm]
- Number of steel plates
- Width steel plate [m]
- Diameter of pin [mm]

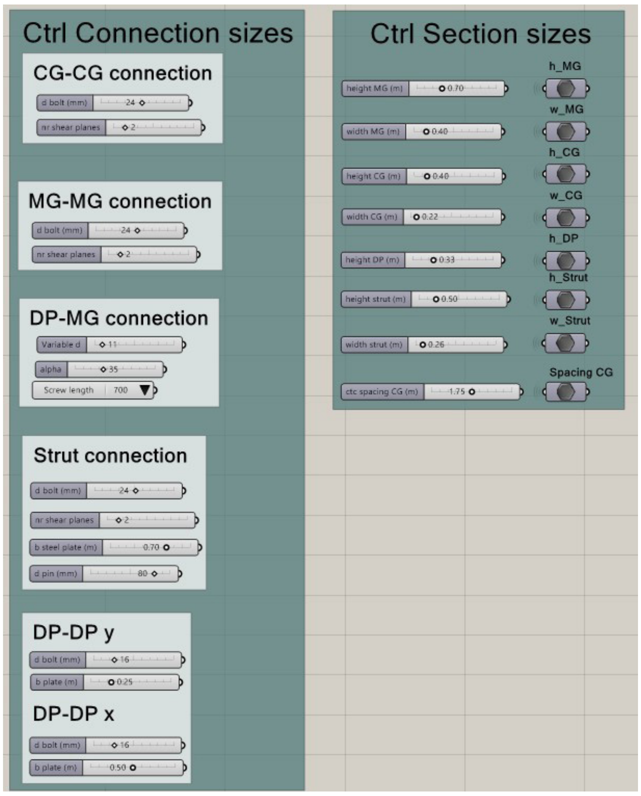
INPUT VALUES to keep constant

MESH DENSITIES
- Mesh density of deck
- Mesh density of loads

DIMENSIONS
Columns
- Height [m]
- Width [m]

Free height [m]

LOADS
- Load combinations
- Multiplication factor for self-weight
- Load values



OUTPUT

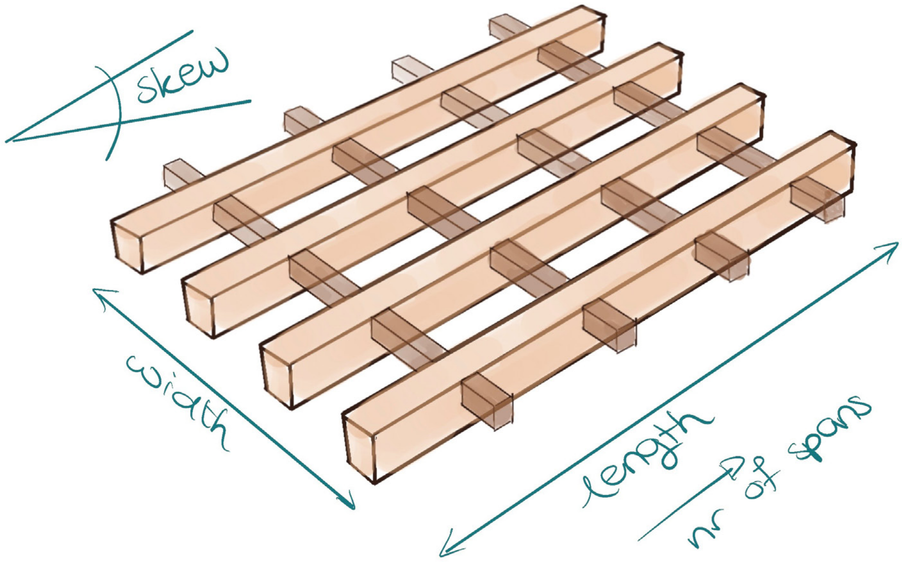
SUMMARIZED
- Mass [kg]
- Volume [m^3]
- UC SLS
- UC ULS
- UC of connections

STRUCTURAL
- Forces in members
- Stresses in members
- Deflections
- Support reactions

1

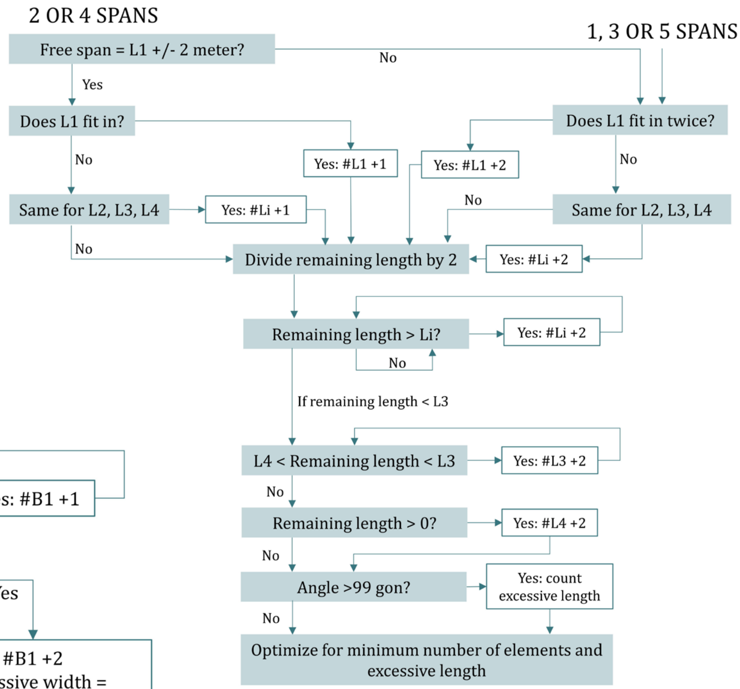
Input:

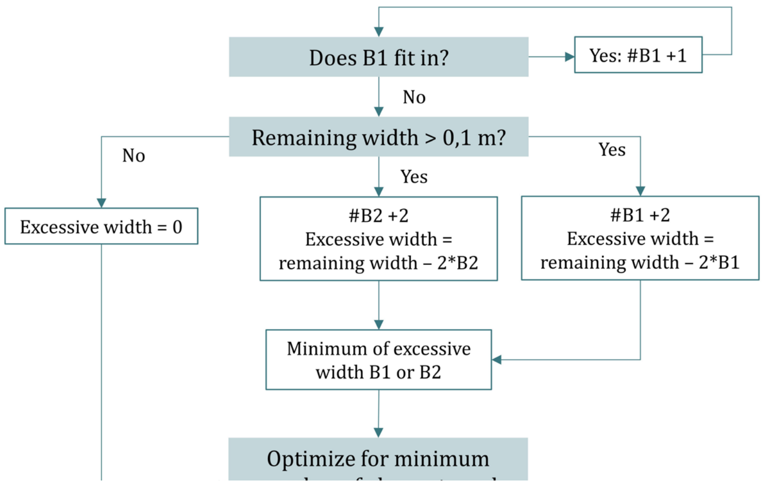
- Skew
- Width
- Length
- Number of spans



2

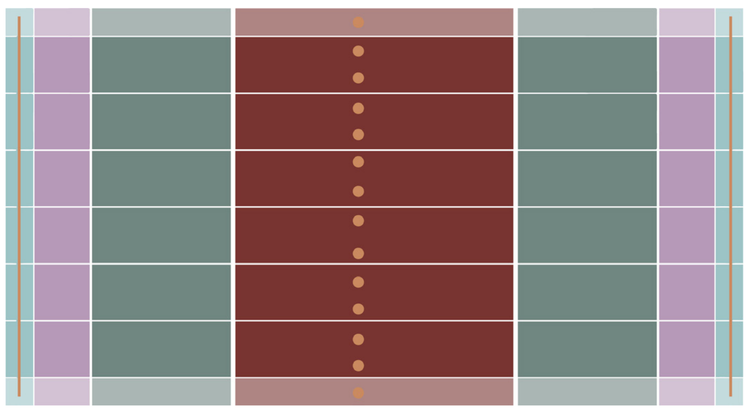
Script calculates the required amount of modules in length and width.





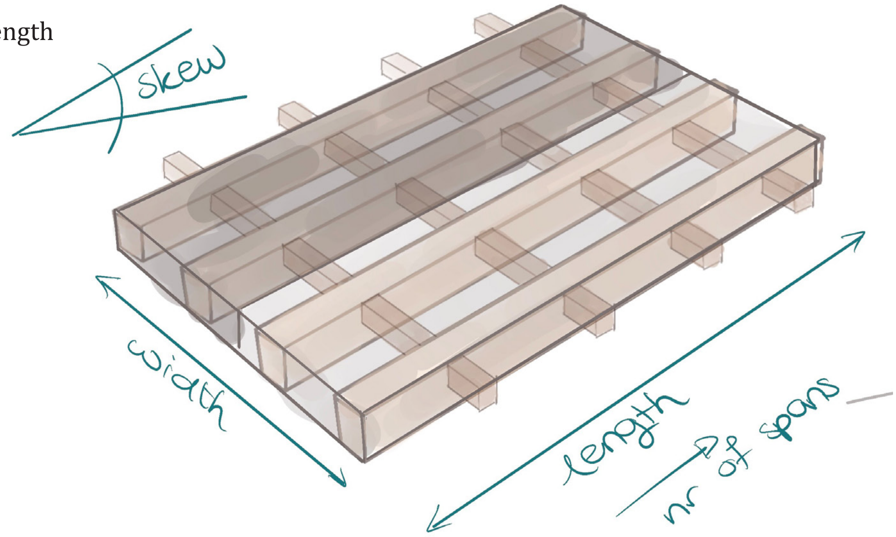
3

Modules are arranged in length and width: in both cases, largest modules from the middle, smallest at the outer edges.



4

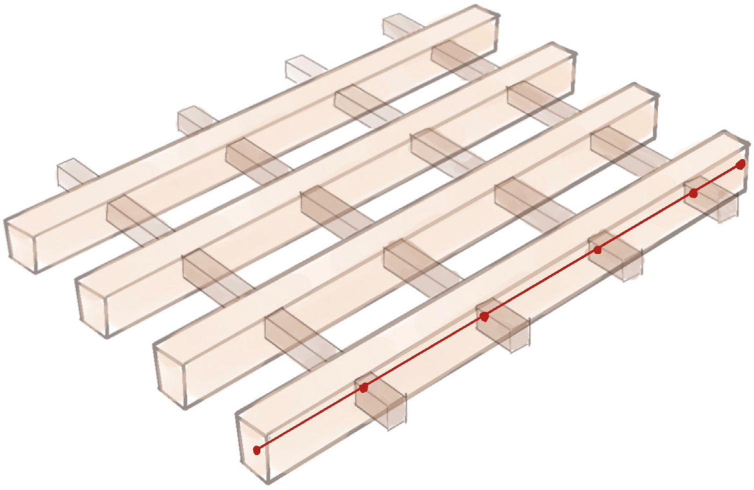
Arrangement of modules in length and width direction



5

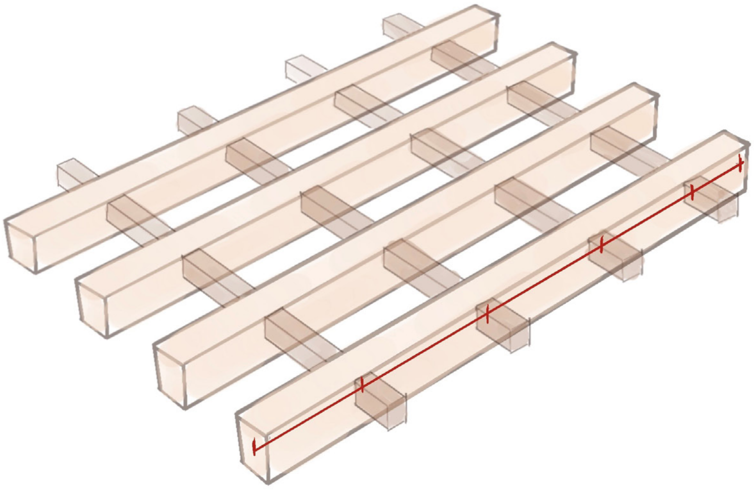
Creating main girder with nodes at:

- End points of modules
- Locations of cross girders



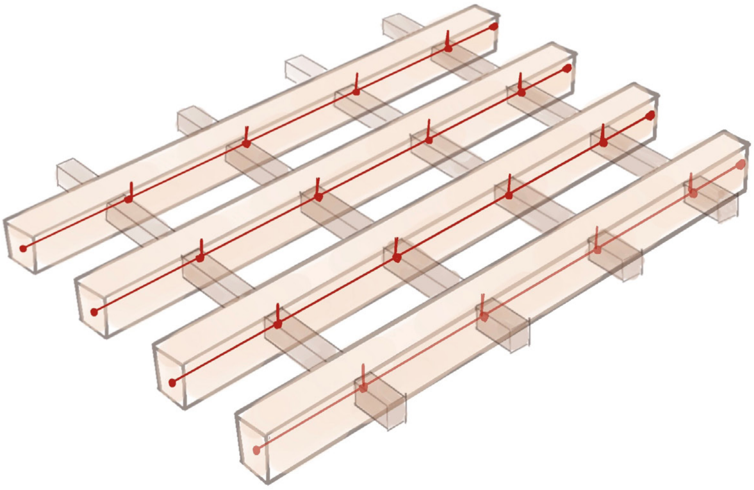
6

Creating lines to make links to cross girders and deck panel.
Lines of elements are at the center of the cross section, so to connect elements, links must be made.
One can assign a stiffness to this link to simulate the stiffness of the connection.



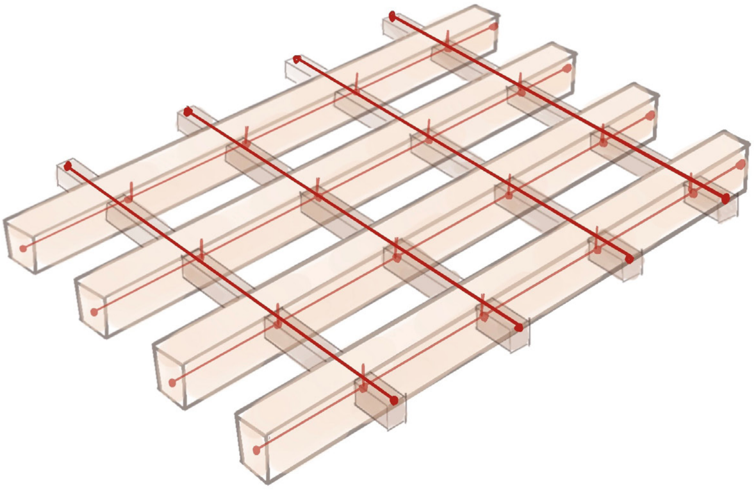
7

Copying main girders with links so th1at they are equally spaced within the module.



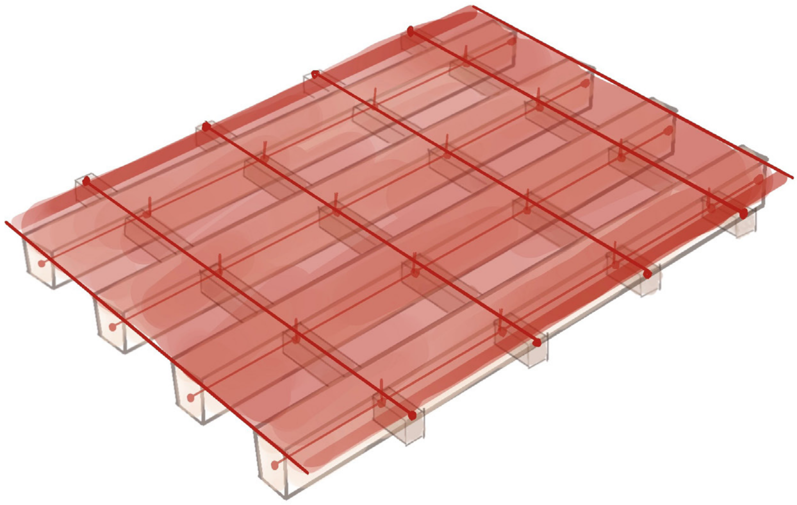
8

Creating cross girders between nodes in model, so that they are connected to the main girders.
Vertical distance to main girders is so that the upper surfaces of both are at the same height.



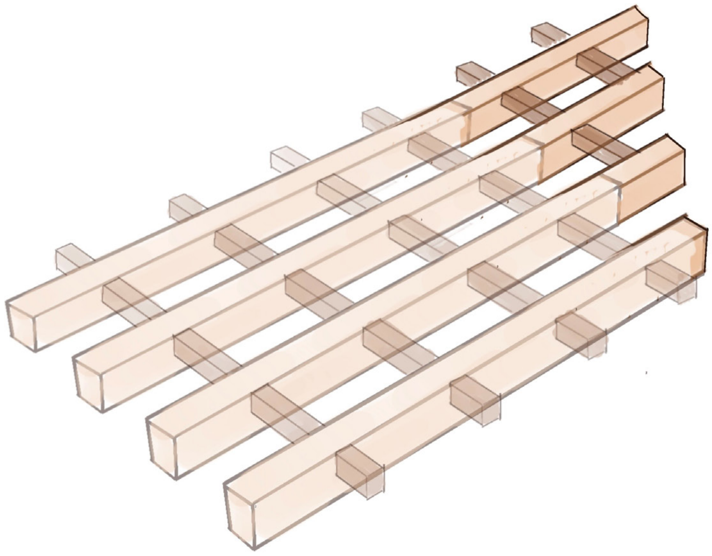
9

Making surface between cross girders and end of modules, and moving it up.
Bottom of the deck is at same height as top surfaces of girders.
Next, the surface is put into a mesh, and the meshes are jointed so that the actual panels are one mesh.



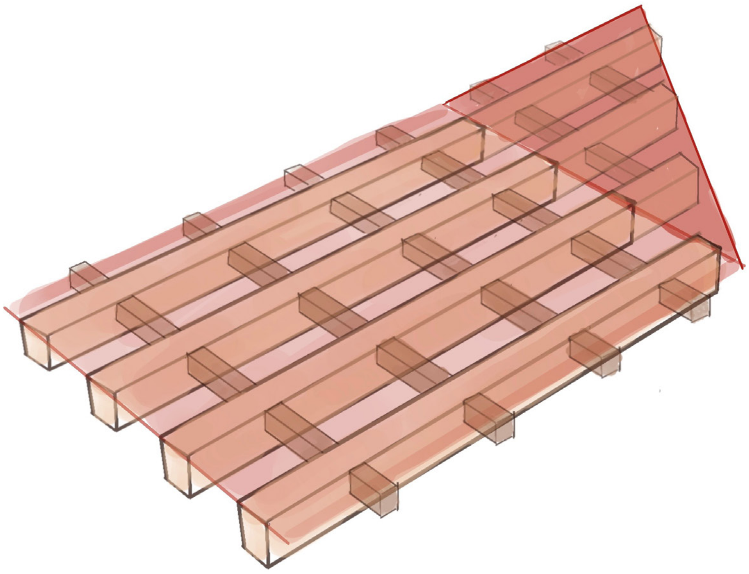
10

Creating the geometry of the triangular parts. This is done in the same manner as the rectangular geometry. The same is done at the other end.



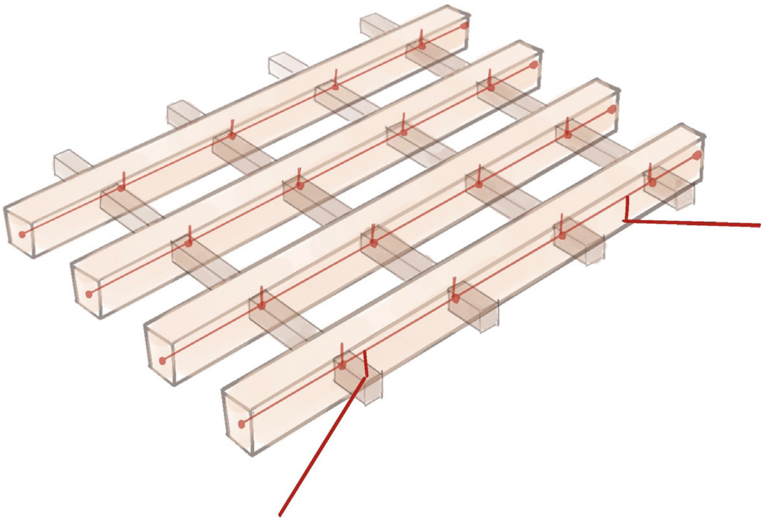
11

Creating the deck surface of the traingular part. The same is done at the other end.



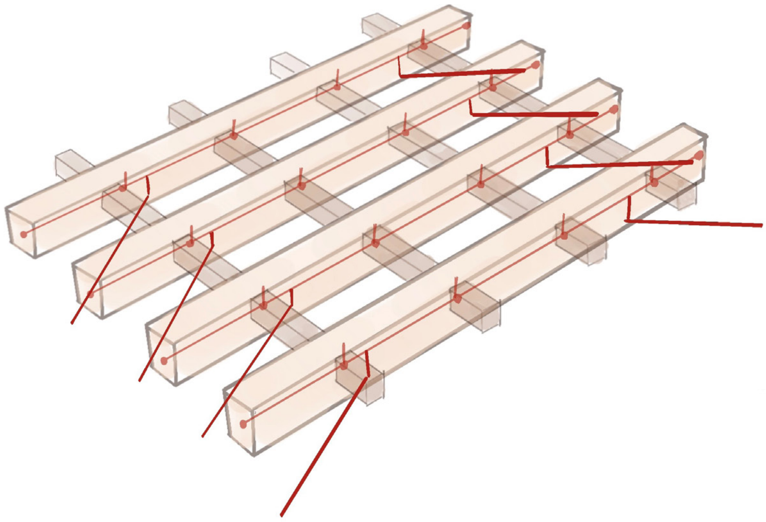
12

Creating struts and links to the main girders for the first main girder. Same principle applies to the supporting columns.



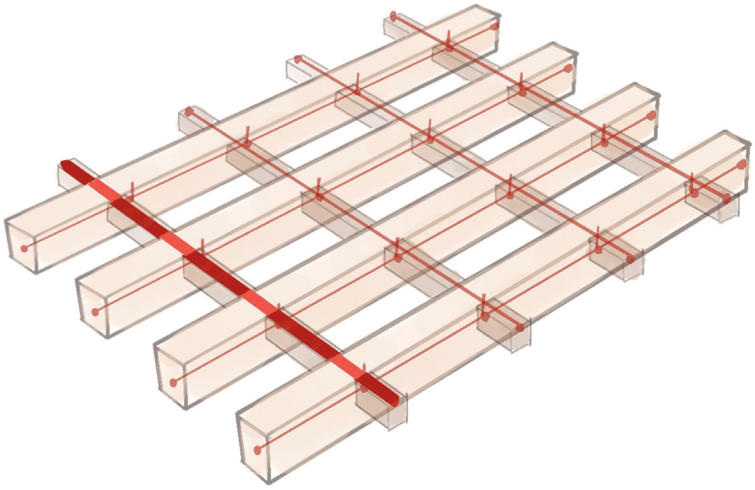
13

Moving the struts to all main girders. Same principle applies to the supporting columns.



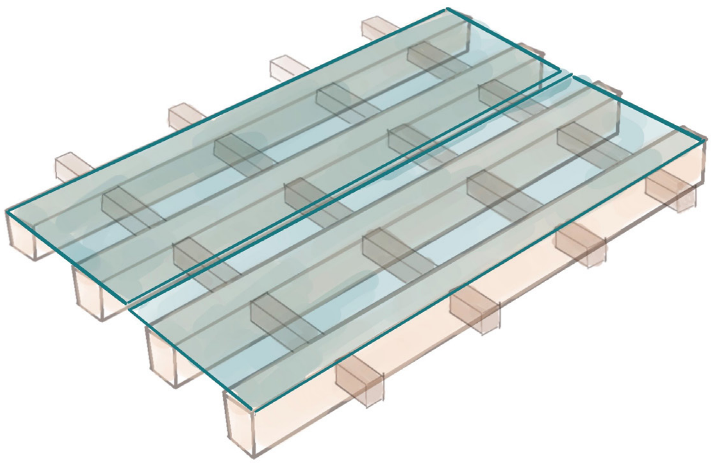
14

Subdividing the cross girder lines in actual cross girders and in ‘infinite stiff links’ that go through the cross section of the main girder.



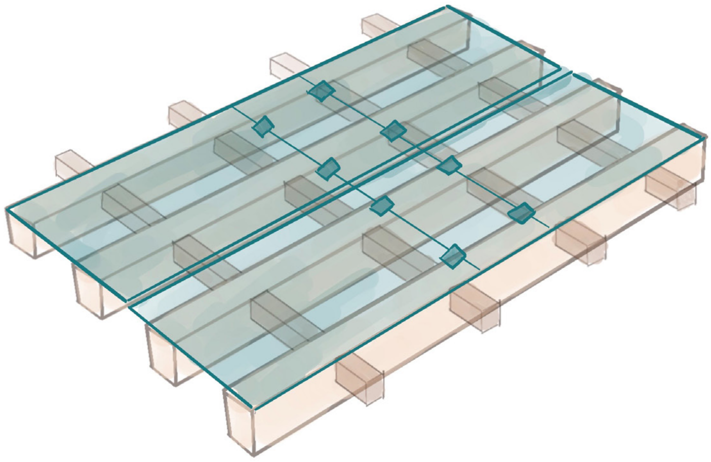
15

Creating surfaces at 0.5 meter from the outer edges. Creating virtual lanes of 3 meter width.



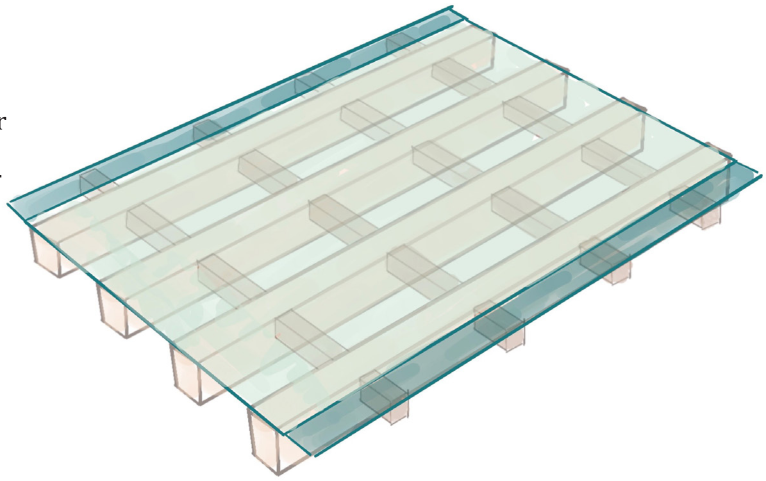
16

Creating surfaces of 0.4*0.4 meter for the axle loads. At middle of span and at the first three lanes in width direction.



17

Creating surfaces for static load. At outer edges, a strip is placed for the loads of the railings etc. In the middle, a surface for the weight of the asphalt and other permanent elements.



E.2. Supports

For each typology, the definition of the supports and their degrees of freedom are described.

Model v1 and v2

Model v1 and v2 are very similar in supports. The main girders are the lowest bearing elements in both models, and they connect to the abutments and the concrete columns.

Support points are applied at the bottom of the concrete columns. These are restricted in translation in global x and z direction. The middle column also has a restriction for translation in global y direction. To attach the structure to the abutments, supports are applied at the ends of the main girders as well, but only restricted in z direction. Furthermore, for the beam structure support points are applied 1 meter inwards from the ends of the main girders. The reason for this is that the main girders always overlap at least 500 mm with the abutment (Rijkswaterstaat, 2017). Because this makes the beam a continuous beam over a support point, the rotation is lowered and the deflection reduced.

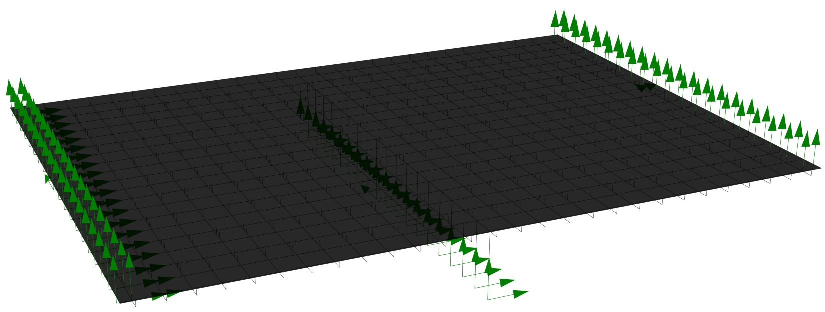


Figure E.1: Support points in beam structure

For the strut model, a stiff element is created that resembles the abutments. These are restricted at the bottom in global z direction. The struts are connected to these abutments. Again, the middle element is also restricted in y direction.

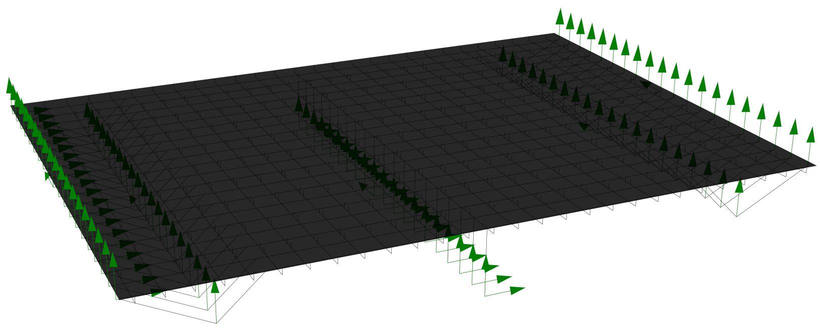


Figure E.2: Support points in strut structure

Truss model

For the truss model, support are applied at the bottom of the concrete columns, restricted in x and z direction. Similar to the other models, the middle one is restricted in y direction. At the end of the trusses, restrictions are only applied in z direction.

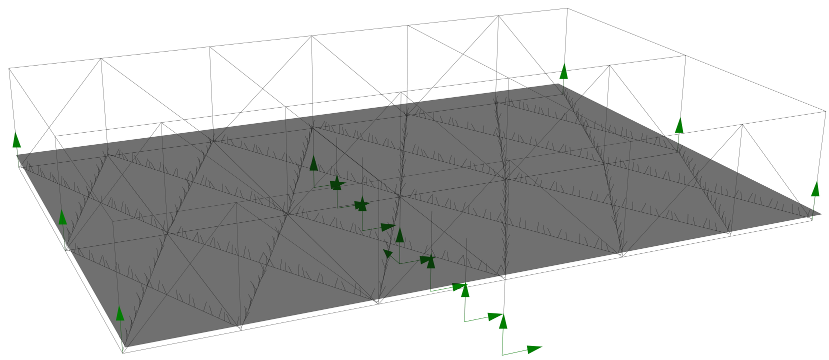


Figure E.3: Support points in truss structure

E.3. Connections in typology study

Assumptions are made for connections in the parametric model. In the detailed part of this research, the connections are worked out further. For the typology study, the assumptions are sufficient.

E.3.1. Main girders

Main girder connections occur at the linking of modules. Depending on the modules that are applied, the number of connections differ.

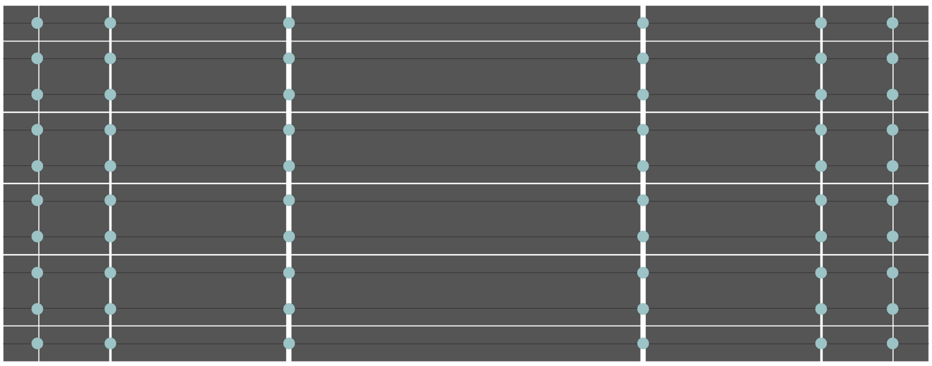


Figure E.4: Joints in main girders

To calculate the rotational stiffness of the connection, Equation E.1 and E.2 are used. This calculation is based on the book Timber Engineering, Design principles, Part E4 (Ehlbeck and Werner, 1995). The dimensions are used as shown in figure E.5.

$$K_r = K_{ser} \cdot \sum_{i=1}^{n} x_i^2 + y_i^2 \tag{E.1}$$

$$K_{ser} = \frac{\rho_m^{1.5} \cdot d}{23} \cdot 2 \tag{E.2}$$

where:

- ρ_m is the mean density of the timber [kg/m^3]
- d is the diameter of the bolt [mm].
- x_i and y_i are the distances to the center to the connection in respectively x and y direction [mm].

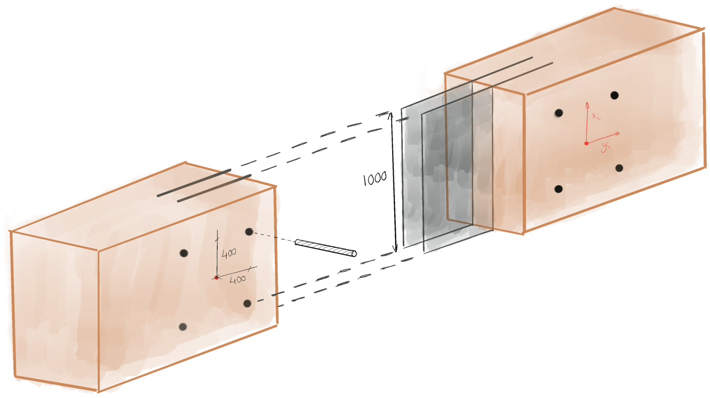


Figure E.5: Slotted-in steel plate connection, assumption for global design. Dimensions are in mm.

- K_r is the slip stiffness of the fastener, expressed in Nmm.
- K_{ser} is the rotational stiffness of the connection, expressed in kNm/rad.

In Equation E.2, the stiffness can be multiplied by two, since steel plates are used. When bolts M24 are used, with timber GL28h, the following result is obtained.

$$K_{ser} = \frac{460^{1.5} \cdot 24}{23} \cdot 2 = 20590 \tag{E.3}$$

Two slotted in steel plates are applied, resulting in four shear planes. With the distances from the center point of the connection, and the fact that four bolts are applied, the following result is obtained for the rotational stiffness [kNm/rad] for a girder of 1 meter height.

$$K_r = 20590 \cdot 4 \cdot (0.4^2 \cdot 4 + 0.4^2 \cdot 4) = 105419 \tag{E.4}$$

This calculation is implemented in the global model, on the ends of the main girders, for the connection between the modular elements. The stiffness of the connection is calculated automatically by using the applied cross sections.

E.3.2. Cross girders

The same principle is applied at the splice joints of the cross girders. Depending on the size of the cross girders, the stiffness of the connection can be determined. This connection is applied between the intersections of the standard elements, in width direction. Similar to the main girder connection, the calculation is implemented in the parametric model, using the dimensions of the cross girders as input.

E.3.3. Shear connection for composite behaviour

Composite behaviour between the main girders and the deck is assumed. The amount of composite behaviour has a high influence on the deflection of the structure. This should be defined carefully.

As stated in the literature study, to make the structure demountable, a connection with dowels is desired. The same properties for dowels are taken as for the connection of the main girder. For the slip modulus of one connecting dowel M24 K_{ser} = 10295 N/mm (Equation E.4). The value of the slip modulus is calculated by the parametric model, using the amount of dowels and the size as input. The connections are modelled as stiff links, with a slip modulus at the end of the member. By doing this, the eccentricity of the connected members is kept, and the composite behaviour is not overestimated.

The strength of the connection is determined by the number of connecting members, multiplied by the

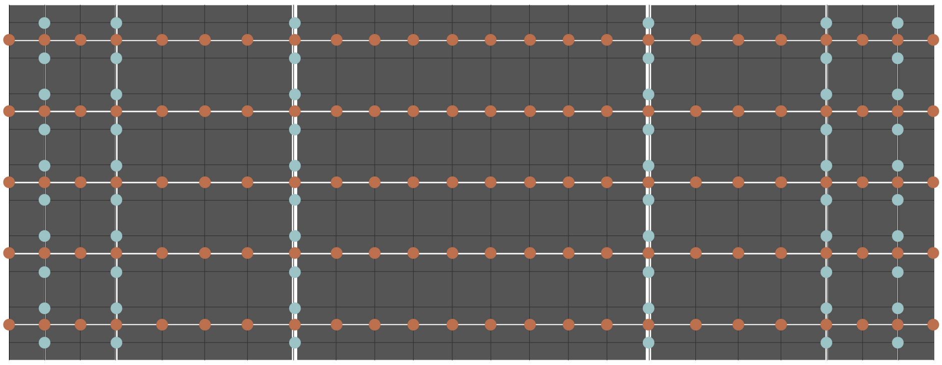


Figure E.6: Joints in main girders and cross girders

strength of one connecting member. For the first phase of this research, dowels are used. In the parametric model, a calculation is made to determine how many dowels can fit in the connection area. From Table 8.7 in NEN-EN 1995-1-1, the following values are obtained for the minimum distance.

Table E.1: Minimum distances of dowels

a_1	$(1.2 + 0.8 \cdot \lvert\cos(\alpha)\rvert) \cdot d_c$
a_2	$1.2 \cdot d_c$
a_3	$1.5 \cdot d_c$

The number of dowels parallel and perpendicular to the grain are determined by the minimum distances. In the detailed design, the strength is checked and verified. As the maximum value of dowels that can fit in is put in the model for all typologies, a fair comparison is obtained.

E.3.4. Links between deck panels

Each module has its own deck panel. These deck panels must also be connected to avoid differences in deformation. When differences in deformation occur, the asphalt cracks and more maintenance is required. Furthermore, to keep the watertight layer in tact, no significant differences in deformation can occur as well.

To simulate the connection between the decks, zero-length links are applied. These links are formed by taking the point closest to the edge from one deck panel, and one from the attaching deck panel. These points form a line, but are actually located in the same point. By doing this, a zero-length link is obtained. This link makes sure that the deformations on the edges of the deck panels align with each other, but no stiffness is applied to this connection yet. When more accurate calculations are performed on this connection, stiffness can be implemented into the model to obtain a more accurate result. The way it is modelled in the simulations of the typologies is conservative, so the behaviour can only get more efficient when the stiffness is applied.

F

Model verification typology study

F.1. Verification of mass

The first step of the verification is to check whether all elements that are created are actually included in the analysis. This is done for all three models.

V1 model

For the v1 model in bridge four, the mass is 773164 kg in the Karamba model. The concrete columns are not included in this mass.

Table F.1: Mass check for v1 model

Element	**Length (m)**	**Height (m)**	**Width (m)**	**Density (kg/m^3)**	**Mass (kg)**
Main girders	1309,18	1.2	0.6	433.37	408498.04
Cross girders	529.1	0.4	0.5	433.37	45859.21
Element	**Area (m^2)**	**Thickness (m)**		**Density (kg/m^3)**	**Mass (kg)**
Deck	1813	0.4		450	326349
Total					780697

1.0 % difference is obtained, which is negligible.

V2 model

For the v2 model in bridge four, the mass is 561393 kg in the Karamba model. The concrete columns are not included in this mass.

Table F.2: Mass check for v2 model

Element	**Length (m)**	**Height (m)**	**Width (m)**	**Density (kg/m^3)**	**Mass (kg)**
Main girders	1365	0.7	0.5	433.37	207042.5
Cross girders	613.13	0.4	0.4	433.37	42513.6
Element	**Area (m^2)**	**Thickness (m)**		**Density (kg/m^3)**	**Mass (kg)**
Deck	1813	0.4		450	326340
Total					575896

2.5 % difference is obtained, which is negligible.

Truss model

For the truss model in bridge four, the mass is 649306 kg in the Karamba model. The concrete columns are not included in this mass.

Table F.3: Mass check for truss model

Element	**Length (m)**	**Height (m)**	**Width (m)**	**Density (kg/m^3)**	**Mass (kg)**
Bottom chord	147	1	0.7	433.37	44593.8
Top chord	147	1	0.7	433.37	44593.8
Diagonals	243.56	0.6	0.6	433.37	37998.6
Vertical chord	216	0.6	0.6	433.37	33698.9
Top cross girders	140	0.4	0.2	433.37	4853.7
Cross girders	910.15	1	0.6	433.37	236659.0
Element	**Area (m^2)**	**Thickness (m)**		**Density (kg/m^3)**	**Mass (kg)**
Deck	1715	0.33		450	254677.5
Total					657075

1.2 % difference is obtained, which is negligible.

F.2. Verification of forces

F.2.1. Support reactions

To verify the loads in the model, the reaction forces are compared to the sum of support reactions that should occur on the basis of a hand calculation. This verification is performed for Load model 1 (LM1), with the Uniformly Distributed Load (UDL) and the Tandem system (TS).

V1 model

The reaction forces are obtained from the Karamba model. A total of 66 support points are applied. The sum of these reaction forces are presented in Table F.4.

Table F.4: Sum of reaction forces in Karamba model v1

Fx	0	kN
Fy	0	kN
Fz	7351.95	kN

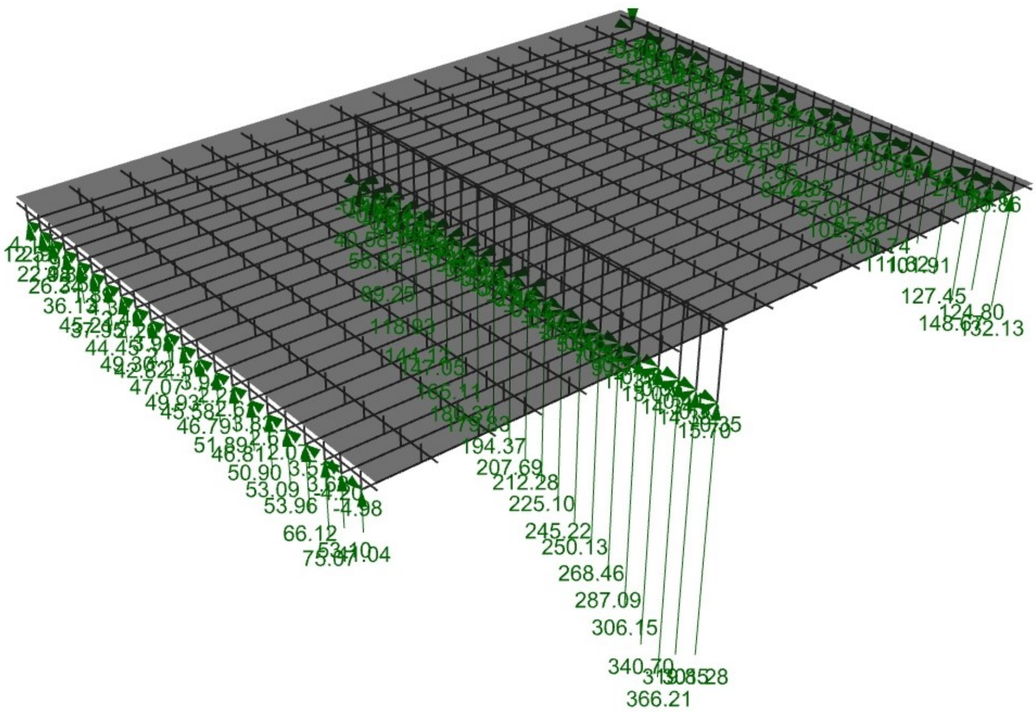


Figure F.1: Support reactions due to LM1 for model v1

The hand calculation is performed as shown in Table F.5.

Table F.5: Sum of reaction forces in Karamba model

Length	49	m
q_{UDL}	$3 \cdot 1.035 + 9 \cdot 3.5 = 125.55$	kN/m
TS	$2 \cdot (300 + 200 + 100) = 1200$	kN
Total	$49 \cdot 125.55 + 1200 = 7351.95$	kN

Both sums of reaction forces end in 7351.95 kN, so the forces are implemented into the model correctly.

V2 model

The reaction forces are obtained from the Karamba model. A total of 66 support points are applied. The sum of these reaction forces are presented in Table F.6.

Table F.6: Sum of reaction forces in Karamba model v2

Fx	0	kN
Fy	-2.47	kN
Fz	7351.95	kN

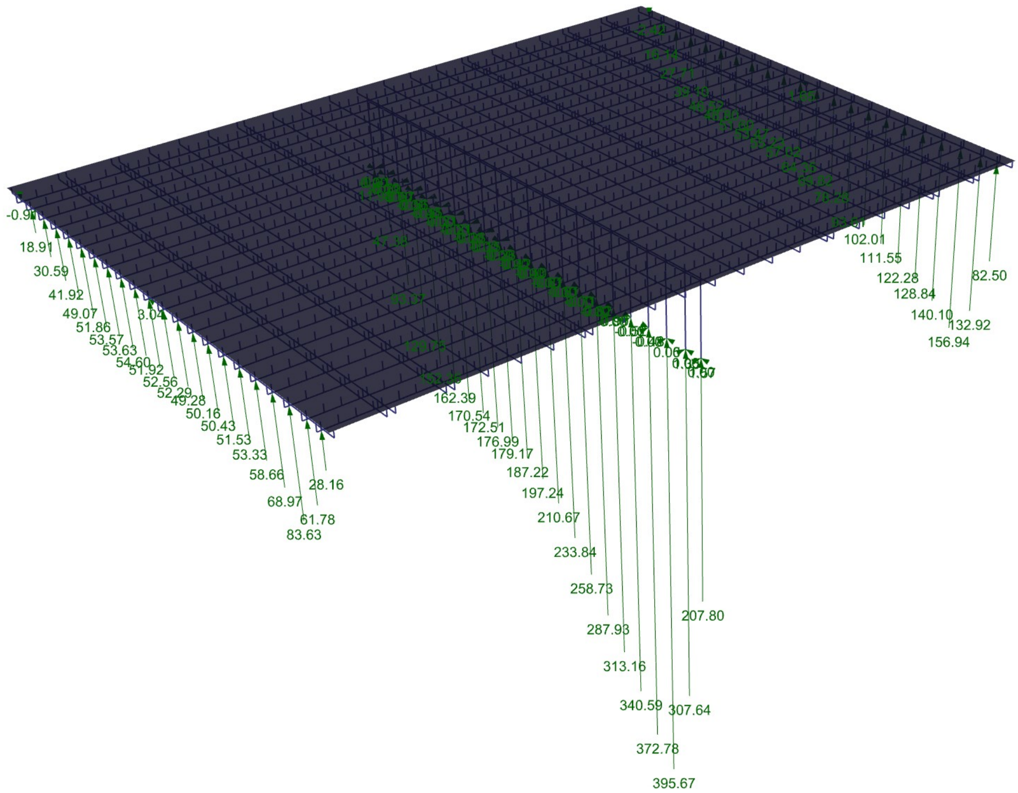


Figure F.2: Support reactions due to LM1 for model v2

The handcalculation is performed as in Table F.5, the same applies for model v2. Again, both sums of reaction forces end in 7351.95 kN, so the forces are implemented into the model correctly. The small difference of forces in y-direction is negligible.

Truss model

The reaction forces are obtained from the Karamba model. A total of 13 support points are applied. The sum of these reaction forces are presented in Table F.7.

Table F.7: Sum of reaction forces in Karamba truss model

Fx	0	kN
Fy	0	kN
Fz	7352.11	kN

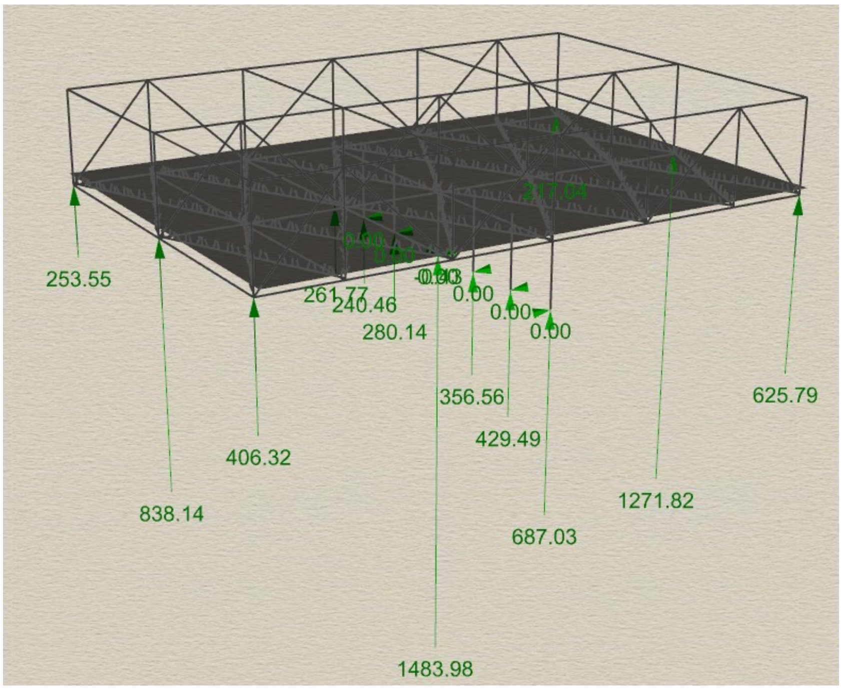


Figure F.3: Support reactions due to LM1 for truss model

The handcalculation is performed as in Table F.5, the same applies for the truss model. The sums of reaction forces is 7352.11 kN, so the forces are implemented into the model correctly.

F.2.2. Bending moments

V1 model

To verify the bending moments in the main girders, the model of v1 is modified. The cooperation between deck, cross girders and main girders is taken out. By cooperation, not all bending moments are transferred to the main girders, which makes it hard to check. Figure F.5 presents the bending moments in longitudinal direction at the location of the intermediate support. The location of this bending moment line over the width is presented in Figure F.4.

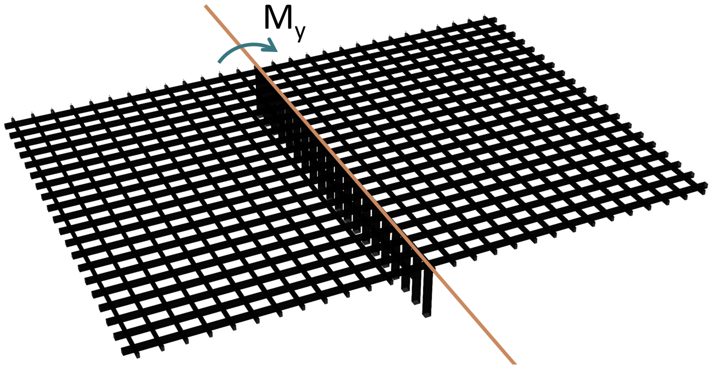


Figure F.4: Location of bending moment line

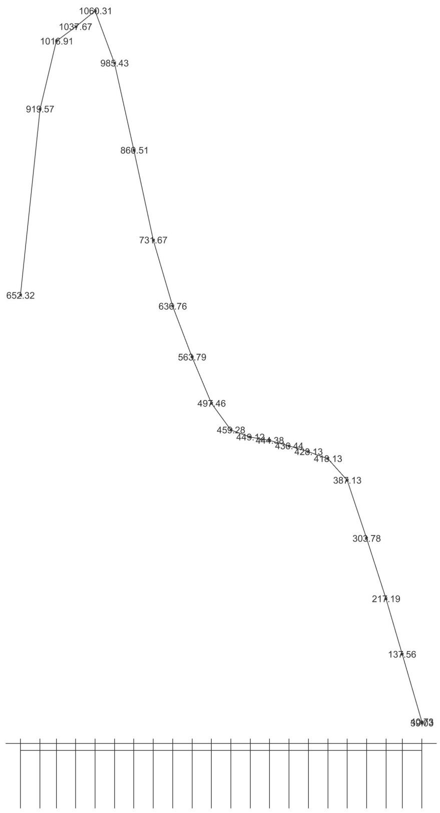


Figure F.5: Bending moments due to LM1 for model v1

The peak in the bending moment line is explained by the tandem loads applied on the bridge. These are not applied on the entire width of the bridge, causing a non-symmetrical pattern in the bending moment line.

To compare the bending moments with a hand calculation, the sum of these bending moments taken: 12723 kNm. In hand calculation, the bending moment [kNm] due to the uniformly distributed load is caused by the tandem loads and the distributed load. The load cases are explained in Section 3.2.2. The following data is included in the calculation:

- Length of bridge: $L = 49$ metre (2*24.5)
- Length of 1 span: $L_{1span} = 24.5$ metre
- Tandem load force:
  - 2*2 wheel loads of 150 kN
  - 2*2 wheel loads of 100 kN
  - 2*2 wheel loads of 50 kN
  - Total: $F_{TS} = 300 \cdot 2 + 200 \cdot 2 + 100 \cdot 2 = 1200$ kN
- Distributed load:
  - 1 lane with 10.35 kN/m^2
  - 9 lanes with 3.5 kN/m^2
  - Total: $q_{UDL} = 1.035 \cdot 3 + 9 \cdot 3.5 \cdot 3 = 125.55$ kN/m

With the data stated above, the bending moment [kNm] due to the distributed load is:

$$\frac{1}{8} \cdot q_{UDL} \cdot L_{1span}^2 = 9420 \tag{F.1}$$

The tandem system causes a bending moment [kNm] of

$$\frac{F_{TS} \cdot \frac{L}{4}^3}{\frac{L}{2}^2} = 3675 \tag{F.2}$$

A sum bending moment of 13095 kNm is obtained. This is 3% difference from the sum of bending moments from the Karamba model. The shape is according to the expectations as well, thus the bending moments can be considered as correct in the model.

V2 model

Model 2 is modified as well to check the bending moments. The cooperation between deck and main girders is taken out. When this operation is performed, the following results are obtained for the bending moments in longitudinal direction at the location of the intermediate support (Figure F.4).

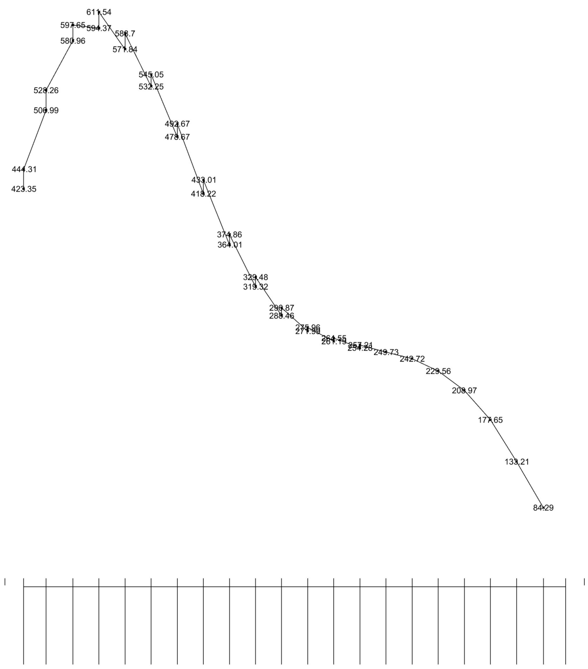


Figure F.6: Bending moments due to LM1 for model v2

The sum of these bending moments is 13232 kNm. The shape of the bending moments is similar to the bending moments of v1. Again, a hand calculation is performed to check the bending moments. The bending moment due to the load is equal to version 1, where a sum bending moment of 13095 kNm is obtained. This is 1% difference from the sum of bending moments from the Karamba model. Again, the shape is according to the expectations, which shows that the force distribution works well in the model.

F.3. Verification of deflection

To verify the deflection of the model, a dummy load is applied of 1 kN/m^2. The Karamba model is modified and a Matrixframe model is made to compare with. One combination of deck and girders is made in the Matrixframe model, and its deflection is measured.

V1 model

To make a fair comparison, the cross girders are modified as infinite stiff links in the Karamba model. In this model, the cross girders can not be taken out as the eccentricity of the deck to the main girders will not be maintained. The deck is still in place. The load of 1 kN/m^2 is put into the Karamba model and a deflection of 1.62 mm is obtained. The cross-sections applied are

- Height main girder: h_{MG} = 1000 mm
- Width main girder: w_{MG}= 600 mm
- Centre-to-centre distances main girders: w_{ctc} = 1750 mm
- Height cross girder: h_{CG}= 400 mm
- Height deck: h_{deck}= 400 mm

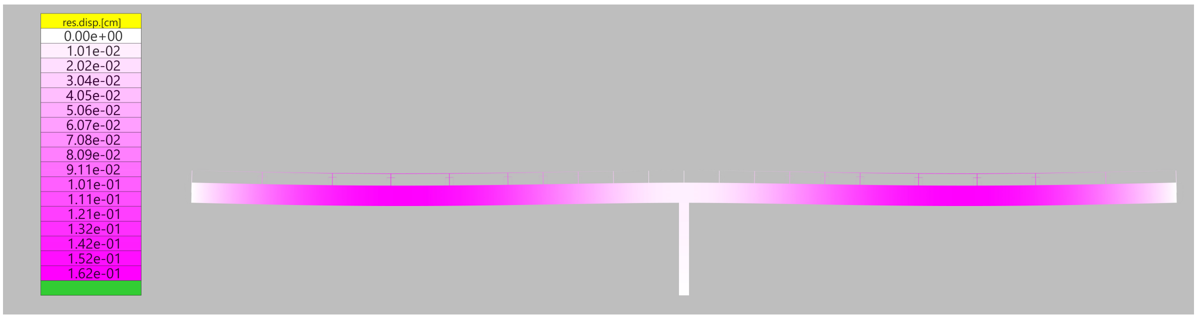


Figure F.7: Deflection Karamba model with dummy loads

A matrixframe model is constructed as well, as a 2D model. To simulate part of the deck and one main girder, a line load is applied of $1.75 \cdot 1 = 1.75$ kN/m on a main girder. The spans have the same sizes as the Karamba model: 2 times 24.5 metre. For the cross section, GL28h and CLT are taken as the materials, with the same E-modulus as in Karamba. The equivalent E-modulus is calculated. The moment of inertia [m^4] and area [m^2] are calculated by hand and used as input for a manually defined cross-section.

To determine the distances of the normal force centre of the elements to the normal force centre of the combined cross section, the location of the normal force centre should be defined. A reference axis is chosen, to which the distances of the normal force centres of the elements are defined (a). The reference axis is defined as the bottom layer of the main girder.

$$z_{nc} = \frac{\sum A \cdot a}{\sum A} \tag{F.3}$$

$$z_{nc} = \frac{1 \cdot 0.6 \cdot 0.5 \cdot 1 + 0.4 \cdot 1.75 \cdot (1 + 0.4 + 0.5 \cdot 0.4)}{1 \cdot 0.6 + 0.4 \cdot 1.75 + 0.4 \cdot 1.75} = 1.09 \tag{F.4}$$

Consequently, the following values are obtained for the distance to the normal force centres:

- $a_{MG} = 1.09 - 0.5 \cdot h_{MG} = 0.59$ m
- $a_{deck} = 1.09 - 0.5 \cdot h_{deck} - h_{CG} - h_{MG} = -0.51$ m
- $I_{MG} = \frac{1}{12} \cdot w_{MG} \cdot h_{MG}^3 + \cdot a_{MG}^2 \cdot A_{MG} = 0.260$ m^4
- $I_{deck} = \frac{1}{12} \cdot w_{ctc} \cdot h_{deck}^3 + \cdot a_{deck}^2 \cdot A_{deck} = 0.190$ m^4

- $I_{total} = I_{MG} + I_{deck} = 0.450\ \text{m}^4$
- $A = w_{MG} \cdot h_{MG} + w_{ctc} \cdot h_{deck} = 1.30\ \text{m}^2$

To calculate the equivalent E-modulus, the following equation is used. E_1 defines the E-modulus in the strong direction, E_2 is the E-modulus in the weak direction [kN/cm^2]. The main girders have their primary direction in span direction, the deck has its primary direction in the width of the bridge. Therefore, the stiffness of the weak direction works in the span direction (E_2).

- GL28h: $E_1 = 1150\ \text{kN/cm}^2$
- GL28h: $E_2 = 30\ \text{kN/cm}^2$
- CLT from C24: $E_1 = 934.5\ \text{kN/cm}^2$ (following calculation of Section B.2 for LL-400/11s (Derix, 2020))
- CLT from C24: $E_2 = 186.56\ \text{kN/cm}^2$ (following calculation of Section B.2 for LL-400/11s (Derix, 2020))

$$E_{eq} = \frac{E_{1,Gl28h} \cdot I_{MG} + E_{2,CLT} \cdot I_{deck}}{I_{total}} = 686 \tag{F.5}$$

A deflection of 1.1 mm is obtained, which is in the same order of magnitude. However, the difference is explained by the application of continuous beams in Matrixframe, where beam splices are introduced in the Karamba model. Furthermore, no three-dimensional effects are included in the Matrixframe analysis.

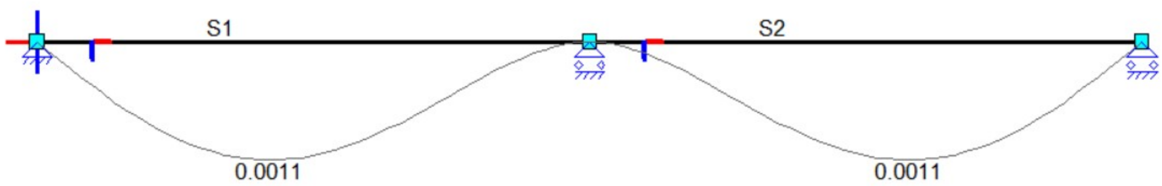


Figure F.8: Deflection Matrixframe model with dummy load

V2 model

To make a fair comparison, the cross girders are removed from the Karamba model. By doing this, load distribution between the main girders can occur. The deck is still in place. The load of 1 kN/m^2 is put into the Karamba model and a deflection of 18.1 mm is obtained. The cross-sections applied are

- Height beam = 400 mm
- Width beam = 400 mm
- Height deck = 400 mm

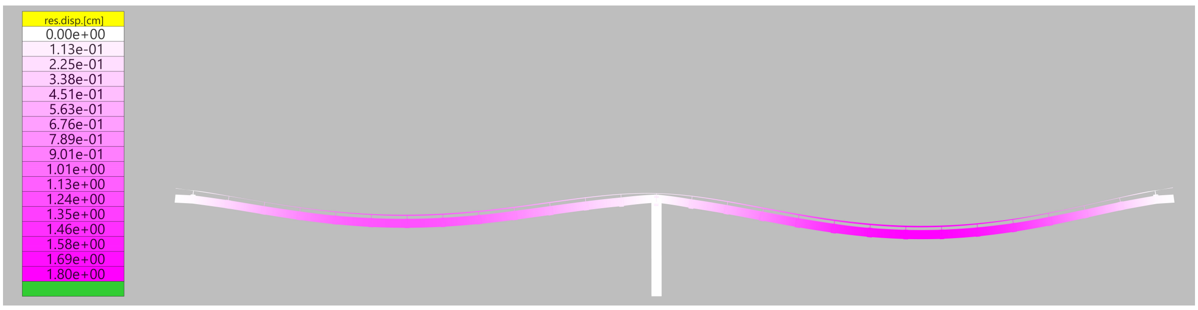


Figure F.9: Deflection Karamba model with dummy load

A matrixframe model is constructed, similar to v1. The only difference are the stiffness values of the cross-sections. These are defined below.

$$z_{nc} = \frac{0.4 \cdot 0.4 \cdot 0.5 \cdot 0.4 + 0.4 \cdot 1.75 \cdot (0.4 + 0.5 \cdot 0.4)}{0.4 \cdot 0.4 + 0.4 \cdot 1.75} = 0.53 \tag{F.6}$$

Consequently, the following values are obtained for the distance to the normal force centres:

- $a_{MG} = 0.53 - 0.5 \cdot h_{MG} = 0.0.33$ m
- $a_{deck} = 0.53 - 0.5 \cdot h_{deck} - h_{MG} = -0.07$ m
- $I_{MG} = \frac{1}{12} \cdot w_{MG} \cdot h_{MG}^3 + \cdot a_{MG}^2 \cdot A_{MG} = 0.019\ \mathrm{m}^4$
- $I_{deck} = \frac{1}{12} \cdot w_{ctc} \cdot h_{deck}^3 + \cdot a_{deck}^2 \cdot A_{deck} = 0.013\ \mathrm{m}^4$
- $I_{total} = I_{MG} + I_{deck} = 0.032\ \mathrm{m}^4$
- $A = w_{MG} \cdot h_{MG} + w_{ctc} \cdot h_{deck} = 0.86\ \mathrm{m}^2$

For v2, both the beams and deck have their primary direction in the span direction.

$$E_{eq} = \frac{E_{1,Gl28h} \cdot I_{MG} + E_{1,CLT} \cdot I_{deck}}{I_{total}} = 1002.8 \tag{F.7}$$

A deflection of 13.2 mm is obtained, which is in the same order of magnitude. The stiffer behaviour is explained by the two-dimensional analysis.

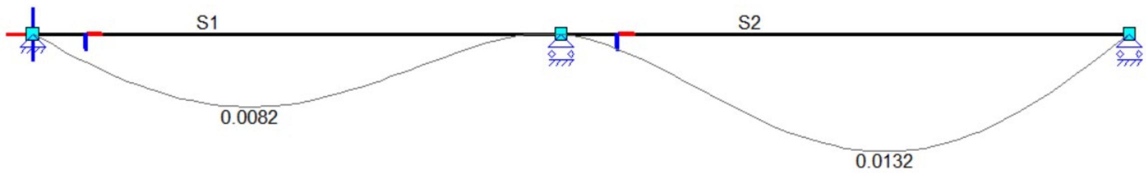


Figure F.10: Deflection Matrixframe model with dummy load

Truss model

To modify the truss model to become similar to a 2D model, bottom and top chords of the truss are modified into continuous beams. The connections of the diagonal and vertical chords are still hinged, which is also modelled in the 2D model. The direction of the diagonals changed later in the design process, but the verification of the model works for all configurations of trusses. A check for the deflection is made by applying a load of 100 kN on all truss nodes. A deflection of 1.6 mm is obtained. The cross sections applied are

- Height top and bottom chord = 1200 mm
- Width top and bottom chord = 1000 mm
- Height diagonals and vertical chords = 500 mm
- Width diagonals and vertical chords = 500 mm

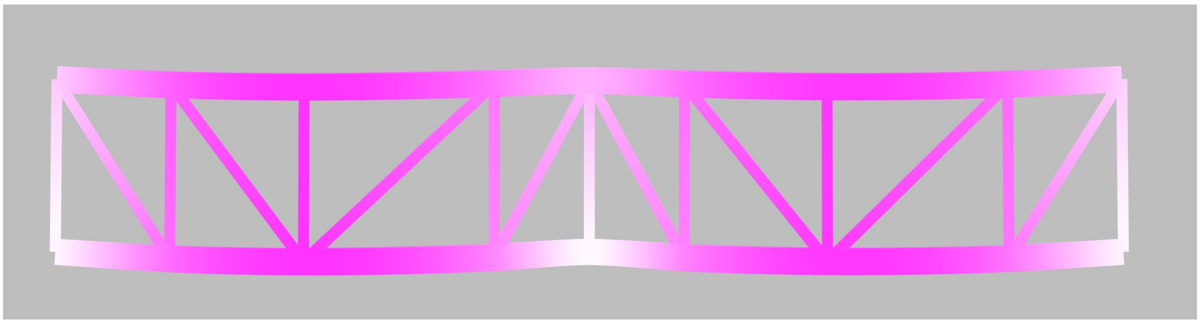


Figure F.11: Deflection Karamba model with dummy load

In the Matrixframe model, the spans have the same sizes as the Karamba model: 2 times 24.5 metre. GL28h is inserted as the material, the deck is left out of the 2D model.

A deflection of 1.4 mm is obtained, which is in the same order of magnitude.

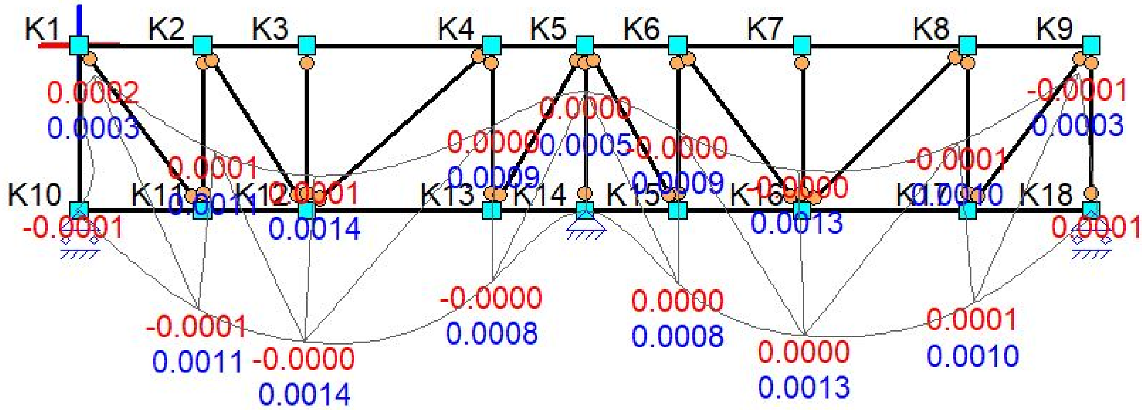


Figure F.12: Deflection Matrixframe model with dummy load

F.4. Verification of composite behaviour

As described in Chapter 7, typology v2s is chosen. Composite behaviour is essential in this load-bearing system. To check the composite behaviour of the model, a hand calculation for the cross-section forces is made.

A composite cross-section is assumed, with an infinite stiff connection (glue). Arbitrary cross-sections are applied, as shown in Figure F.13. First, the position of the neutral axis is calculated by hand. Afterwards, the location of the neutral axis is calculated from the stresses in the model.

To calculate the neutral axis, the material of the beam and deck are both set to glulam, so that the E-modulus is the same for all parts. The location of the neutral axis is calculated according to Equation F.3.

$$a_i = \frac{\sum z \cdot A}{\sum A} = \frac{160 \cdot 1750 \cdot 320 + 670 \cdot 700 \cdot 520}{1750 \cdot 320 + 700 \cdot 520} = 360 \tag{F.8}$$

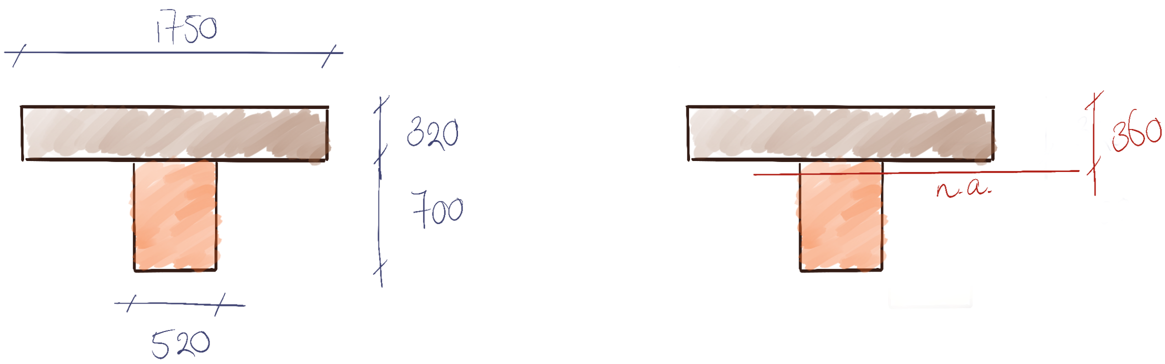


Figure F.13: Neutral axis according to hand calculation for geometry

When a dummy load of 1 kN/m^2 is applied, the following forces are obtained in the main girders: N=101.1 kN, My=-13.9 kNm. The stresses are calculated as follows:

$$\sigma_{max} = \frac{N}{A} + \frac{M_y \cdot z}{I} \tag{F.9}$$

$$\sigma_{min} = \frac{N}{A} - \frac{M_y \cdot z}{I} \tag{F.10}$$

where

- σ is the occurring stress [N/mm^2]
- N is the normal force [N]
- A is the area [mm^2]
- My is the bending moment [Nmm]
- z is the neutral axis of the beam [mm] ($0.5 \cdot h_{MG} = 350$ mm)
- I is the moment of inertia of the beam [mm^4], its calculation is explained in the previous section.

The following results are obtained:

$$\sigma_{max} = \frac{101.1 \cdot 10^3}{364 \cdot 10^3} + \frac{13.9 \cdot 10^6 \cdot 350}{1.486 \cdot 10^{10}} = 0.605 \tag{F.11}$$

$$\sigma_{min} = \frac{101.1 \cdot 10^3}{364 \cdot 10^3} - \frac{13.9 \cdot 10^6 \cdot 350}{1.486 \cdot 10^{10}} = -0.050 \tag{F.12}$$

The visual representation of the stresses is shown in Figure F.14, and the resulting location of the neutral axis is 53 mm from the top of the beam. This results in 373 mm from the top of the composite cross section, which is similar to the 360 mm as calculated by the geometry calculation. The small difference can be explained by the reduced effective width of the deck panel. The center-to-center distance of the main girders is now taken as the width of the deck panel, but the effective width is slightly smaller.

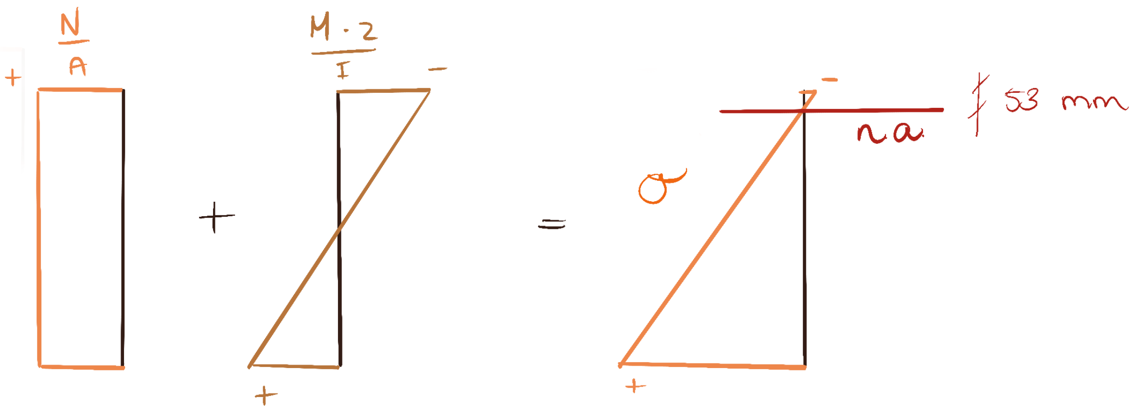


Figure F.14: Neutral axis according to hand calculation with forces

G Typology analysis

G.1. Structural analysis

Table G.1: Structural analysis results for bridges

Bridge	Typology	UC SLS	UC ULS	Mass (kg)	Construction height (m)
1	v1b	0.85	0.74	264531	2.2
1	v1s	0.9	0.86	214278	1.7
1	T	0.56	1	288328	1.2
1	v2b	0.86	0.76	260576	1.5
1	v2s	0.89	0.82	185445	1.1
2	v1b	0.64	0.86	540367	2.5
2	v1s	0.84	0.63	321065	1.6
2	T	0.58	0.83	356808	1.4
2	v2b	0.82	0.83	501295	1.6
2	v2s	0.92	0.54	344484	1.2
3	v1b	0.67	0.84	238918	1.7
3	v1s	0.76	0.51	139277	1.14
3	T	0.63	0.90	225030	1.0
3	v2b	0.77	0.83	249471	1.2
3	v2s	0.67	0.68	131682	0.64
4	v1b	0.78	0.92	583611	1.6
4	v1s	0.82	0.78	436000	1.2
4	T	0.73	0.81	674275	1.3
4	v2b	0.96	0.73	573673	1.1
4	v2s	0.92	0.71	371624	0.74
5	v1b	0.79	0.75	760176	1.6
5	v1s	0.9	0.83	694709	1.6
5	T	0.55	0.88	579063	0.94
5	v2b	0.64	0.87	731535	1.2
5	v2s	0.81	0.89	699215	1.2

Table G.2: Structural analysis results for bridges, values in graphs

Bridge	Typology	UCSLS * mass/area	UCULS * mass/area	Construction height/free width
1	v1b	435	379	0.0349
1	v1s	373	357	0.0270
1	T	313	558	0.0190
1	v2b	434	383	0.0238
1	v2s	319	294	0.0175
2	v1b	419	562	0.0514
2	v1s	326	245	0.0329
2	T	250	358	0.0288
2	v2b	498	504	0.0329
2	v2s	384	225	0.0247
3	v1b	279	350	0.0646
3	v1s	185	124	0.0433
3	T	247	353	0.0380
3	v2b	335	361	0.0456
3	v2s	154	156	0.0243
4	v1b	271	320	0.0327
4	v1s	213	203	0.0245
4	T	293	326	0.0266
4	v2b	328	250	0.0225
4	v2s	204	157	0.0151
5	v1b	309	293	0.0171
5	v1s	322	297	0.0171
5	T	164	262	0.0101
5	v2b	241	328	0.0128
5	v2s	292	320	0.0128

G.2. Trade-off matrix

G.2.1. Explanation of score weights

Effective: 35 points because in order to make a sustainable construction, the most important aspect is to make sure that no material is wasted.

- Mass-deflection: as shown in the SBIR and other studies on timber bridges, the design is often governed by serviceability limit state which results in deflection. Therefore, mass-deflection weighs the heaviest within the effective strategy.
- Mass-strength: all designs need to comply with strength regulations as well, so this is almost as important as deflection.
- Construction height: this does not make a difference in the amount of material used, but in the implementation into the existing infrastructure. In order to make the structure fit in the road system by not changing the embankments too much, this must also be effective. Nevertheless, it is scored lower than the mass requirements since that is directly linked to the amount of timber used.

Protected: 10 points as it is hard to predict what the influence of the geometry and typology is on the actual technical lifespan of the structure. However, protection is important as it can increase the durability of the bridge significantly. No in depth research on the lifespan of the structure is in the scope of this research.

Flexible: 25 points since this is an important strategy to extend the lifespan of the structures and to make sure that it fits in the replacement task of Rijkswaterstaat. However, it is not as important as effective and demountable so weighs lower.

- Widely applicable: Similar to construction height. The structure must be implemented in the existing infrastructure networks in the Netherlands. However, adaptability is a more important focus of this research so is therefore scored higher.
- One of the main aspects of this research is the adaptable system. Therefore this is scored relatively high.

Demountable: 30 points as the above strategy relies on the demountable principle. Furthermore, almost all R-principles that are applied rely on the demountability of the structure.

- Reaching connections: when the connections can't be reached, mounting and demounting is made difficult. However, it is considered less important than complexity due to the amount of labour.
- Complexity: this is correlated with the rigidity and the ability to reach the connections, but also has a cost aspect in it. When the complexity is high, a lot of labour is required to mount and demount the connections. Furthermore, more complex connections increase the costs of the manufacturing of the connection itself. Therefore, this has the largest weight within this strategy.
- Connection rigidity: the magnitude and location have influence on either the deflection as strength of the bridge. It is expected that most of the bridges are governed by SLS requirements, so bending moments and rotational capacity are important.

G.2.2. Explanation of scoring

Table G.3: Performance indicators for TOM effective

Strategy	Performance indicator	Criteria
Effective	Mass - deflection	How much mass is needed to satisfy the SLS requirements? Is the deflection evenly distributed?
	Mass - strength	How much mass is needed to satisfy the ULS requirements? Are the stresses evenly distributed? Are there very large peaks in the stresses?
	Construction height	What is the required construction height to satisfy both SLS and ULS requirements? Is this height used efficiently?

Table G.4: Score TOM on effective strategy: mass-deflection

Effective	**Mass-deflection**	**15**
v1b	A lot of mass is needed to comply with SLS requirements. This is caused by the relatively large span as no struts are applied. Furthermore, no composite behaviour in longitudinal direction can be obtained since the main girders are not directly connected to the deck. $UC_{SLS} * mass/area = 343$ kg/m^2	-
v1s	Struts act like an extra support, which reduces the free span and thus less mass is required. No composite behaviour is obtained, similar to v1b. $UC_{SLS} * mass/area = 284$ kg/m^2	+
T	Evenly distributed deflection due to high stability ensured by the truss. Truss requires quite a lot of material but makes an efficient structure. $UC_{SLS} * mass/area = 254$ kg/m^2	+ +
v2b	Composite behaviour of main girder and deck is essential to reduce deflections. This can be ensured by making a doweled connection, which is still demountable. The free span is not reduced. $UC_{SLS} * mass/area = 367$ kg/m^2	- -
v2s	Struts act like an extra support and lower the free span. Composite behaviour of main girder and deck is essential but reduce deflections. This can be obtained in the same manner as v2b. $UC_{SLS} * mass/area = 270$ kg/m^2	+

Table G.5: Score TOM on effective strategy: mass-strength

Effective	**Mass-strength**	**10**
v1b	SLS is governing in all situations. ULS requirements are not a problem in the elements. Again, more mass is required due to large free span compared to the strut options. Large jumps occur in the stresses in the main girders because the cross girders transfer the forces as point loads from the deck to the main girders. $UC_{ULS} * mass/area = 381$ kg/m^2	- -
v1s	SLS is governing in all situations. ULS requirements are not a problem in the elements. Scores better than v1b, again large jumps occur again in the stresses. $UC_{ULS} * mass/area = 245$ kg/m^2	+
T	Most situations are governed by ULS requirements. High unity checks occur in the truss members (as a consequence of axial stresses). $UC_{ULS} * mass/area = 372$ kg/m^2	- -
v2b	SLS is still governing in most situations, but UC's are closer to 1. Still, a lot of mass is needed to comply with ULS requirements. The free span is large compared to strut option, similar to v1b. $UC_{ULS} * mass/area = 365$ kg/m^2	- -
v2s	SLS is still governing in most situations, but UC's are closer to 1. Less mass is needed to comply with ULS requirements compared to v2b. This is caused by application of struts (and thus lowering the free span). $UC_{ULS} * mass/area = 231$ kg/m^2	+ +

Table G.6: Score TOM on effective strategy: construction height

Effective	**Construction height**	**10**
v1b	No composite behaviour occurs between main girders and deck. Therefore the girders need to transfer all forces to the embankments and provide stiffness. Large construction height required. Slenderness (L/H) = 30.	- -
v1s	Free span is reduced slightly, so smaller cross sections can be applied. Still no composite behaviour and thus high main girders needed. Score of 0,65 from analysis. Slenderness (L/H) = 38.	-
T	Truss provides a lot of stiffness for structure, but does not add to the effective height, which makes it efficient. Cross girders and deck are placed between bottom chords of the truss. No stacking occurs as in option 1, which reduces the effective height. However, bottom chords still need to be high because of ULS requirements. Slenderness (L/H) = 50.	+ +
v2b	No reduction due to global typology (no struts). Cross girders are integrated between main girders so less height needed than option v1b. Furthermore, composite behaviour can be obtained. Slenderness (L/H) = 43.	-
v2s	Free span is reduced, so smaller cross sections can be applied (and thus construction height. Cross girders are between main girders so less height needed than option v1s, and composite behaviour is obtained. Slenderness (L/H) = 57.	+ +

Table G.7: Performance indicators for TOM protected

Strategy	Performance indicator	Criteria
Protected	Moisture	Is the structure protected against weather influences? Can the structure ventilate and dry when it got wet?

Table G.8: Score TOM on protected strategy

Protected	**Moisture**	**10**
v1b	A lot of possibility for drying and ventilation due to limited contact surface. Beams can be easily protected against the weather by applying a cantilever. Deck must be protected by a waterproofing membrane.	+ +
v1s	A lot of possibility for drying and ventilation due to limited contact surface. Beams can be easily protected against the weather by applying a cantilever. Deck must be protected by a waterproofing membrane.	+ +
T	A lot of material needed to protect the trusses against weather influences. Girders below the deck can be kept dry by the deck. A lot of contact surface between the deck and girdes, so limited possibility for ventilation.	- -
v2b	A lot of contact surface between beams and deck, so no possibility for drying and ventilating there. Beams can be easily protected against the weather by applying a cantilever. Deck must be protected by a waterproofing membrane.	-
v2s	A lot of contact surface between beams and deck, so no possibility for drying and ventilating there. Beams can be easily protected against the weather by applying a cantilever. Deck must be protected by a waterproofing membrane.	-

Table G.9: Performance indicators for TOM flexibility

Strategy	Performance indicator	Criteria
Flexible	Widely applicable	Is the structure applicable for all skews within the scope? Is the structure applicable for all lengths within the scope? Is the structure applicable for all widths within the scope?
	Adaptability	Is the structure adaptable in width of the bridge? Is the structure adaptable in length of the bridge?

Table G.10: Score TOM on flexible strategy: widely applicable

Flexible	**Widely applicable**	**10**
v1b	Due to large construction height, the fitting into the existing infrastructure is difficult. Roads and embankments should be adjusted to arrive at the bridge. When large spans are applied, the height increases significantly compared to a concrete bridge.	- -
v1s	Some space under the bridge is taken by the struts, but this will be located above the embankments so will not reduce the free profile. Can be used for larger spans as the free span is reduced by struts. Larger construction height (compared to v2s) makes the implementation into the attaching roads more complicated.	-
T	Widely applicable as long spans can be made. In case of narrow bridges, a truss is not the most efficient structure, but can still be applied. For very wide bridges, the deflection of the cross girders becomes governing and not all loads can be transferred to the trusses effectively.	+
v2b	More applicable than option v1b, as the construction height is reduced. Still, not a slender structure and thus the fitting into the existing infrastructure is difficult. Roads and embankments should be adjusted to arrive at the bridge.	-
v2s	Some space under the bridge is taken by the struts, but this will be located above the embankments so will not reduce the free profile. Can be used for larger spans as the free span is reduced by struts. Lowest effective height so least adjustments to infrastructure, in case of replacing a concrete bridge.	+ +

Table G.11: Score TOM on flexible strategy: adaptability

Flexible	**Widely applicable**	**15**
v1b	Modules can be added in width and length. However, the construction height and mass are already high, and should be over-dimensioned to be adaptable.	+
v1s	Modules can be added in width and length. However, the construction height is already high, and should be over-dimensioned to be adaptable. Struts must be moved to adapt in length.	-
T	Can not be expanded in width due to the trusses. Can be expanded in length as modules contain one truss part.	- -
v2b	Modules can be added in width and length. Relatively effective structure due to composite behaviour, but still some over-dimensioning is required.	+ +
v2s	Modules can be added in width and length. Relatively effective structure due to composite behaviour, but still some over-dimensioning is required. Struts must be moved to adapt in length.	+

Table G.12: Performance indicators for TOM demountability

Strategy	Performance indicator	Criteria
Demountable	Reaching connections	Are the connections easy to reach? Is the space available to place the connections that are needed?
	Complexity	Are there many connections in the construction? How complex are the connections (how many members join in the connection)?
	Connection rigidity	Do large bending moments occur at the location of the connections? Is there much rotational stiffness needed in these connections?

Table G.13: Score TOM on demountable strategy: reaching connections

Demountable	**Reaching connections**	**5**
v1b	Connections can be easily reached due to stacking of elements, and not integrating the beams. Enough space is available to mount and demount connections.	+ +
v1s	Connections can be easily reached due to stacking of elements, and not integrating the beams. Enough space is available to mount and demount connections.	+ +
T	Connections can be easily reached since only one layer of beams is present, which has to be connected to the chords of the truss. Enough space is available to mount and demount connections.	+
v2b	Harder to reach because of compact construction, compared to v1.	-
v2s	Harder to reach because of compact construction, compared to v1.	-

Table G.14: Score TOM on demountable strategy: complexity

Demountable	**Complexity**	**15**
v1b	Low amount of connections. A maximum of three members join in one connection. Mostly transfer of vertical forces in stacked elements.	+ +
v1s	Slightly more connections due to struts and slightly more complex. A maximum of four members join in one connection. Mostly transfer of vertical forces in stacked elements.	+
T	Multiple members join in one connection, which make them complex and harder to mount. A maximum of six members join in one connection.	- -
v2b	More connections compared to v1b because of integrated beams and composite behaviour between deck and girders. A maximum of four members join in one connection.	+
v2s	Slightly more connections due to struts and slightly more complex compared to v2b. More connections compared to v1s because of integrated beams and composite behaviour between deck and girders. A maximum of four members join in one connection.	-

Table G.15: Score TOM on demountable strategy: connection rigidity

Demountable	**Connection rigidity**	**10**
v1b	Large bending moments in main girder at location of connection. Rotational stiffness is essential to reduce deflection.	+
v1s	Large bending moments in main girder at location of connection. Only normal forces in struts, but cause a jump in the bending moment line. Less rotational stiffness required compared to v1b because of struts that limit deflection and rotation.	-
T	Mostly normal forces, minimal rotational stiffness needed. Very effective transfer of forces.	- -
v2b	Large bending moments in main girder at location of connection. Rotational stiffness is essential to reduce deflection.	+ +
v2s	Large bending moments in main girder at location of connection. Only normal forces in struts, but cause a jump in the bending moment line. Less rotational stiffness required because of struts that limit deflection compared to v2b.	+

H

Variables in parametric model

The standard values for the variables in the parametric model are presented in Table H.1. When deviations occur, or changes are made, it is indicated in the text.

Table H.1: Variables in model, standard values

Element	Variable	Value	Unit
MG	Height	0.68	m
MG	Width	0.40	m
MG	Material	GL28h	-
CG	Height	0.40	m
CG	Width	0.22	m
CG	Material	GL28h	-
DP	Height	0.33	m
DP	Material	CLT LL-320/9s	-
Strut	Height	0.5	m
Strut	Width	0.26	m
Strut	Material	GL28h	-
MG-MG connection	Diameter bolt	30	mm
MG-MG connection	Bolt strength class	8.8	-
MG-MG connection	Number of steel plates	2	-
MG-MG connection	Number of bolts in x direction	2	-
MG-MG connection	Number of bolts in z direction	4	-
CG-CG connection	Diameter bolt	24	mm
CG-CG connection	Bolt strength class	8.8	-
CG-CG connection	Number of steel plates	1	-
CG-CG connection	Number of bolts in x direction	2	-
CG-CG connection	Number of bolts in z direction	2	-
DP-MG connection	Diameter dowel	24	mm
DP-MG connection	Number of dowels	6	-
DP-DP connection	Rotational stiffness	0	N/mm
DP-DP connection	Translational stiffness	Infinite	-
MG-CG connection	Translational stiffness	Infinite	-
Strut connection	Translational stiffness	Infinite	-

The parameters that are kept constant are shown in Table H.2.

Table H.2: Constant parameters in model

Parameter	Value	Unit
Mesh density of deck v-direction	5	-
Mesh density of deck u-direction	3	-
Mesh density of load v-direction	10	-
Mesh density of load u-direction	50	-
Amount of trucks per year per lane	2000000+	-
Factor to enlarge selfweight	1.05	-
Width of edges	1	m
Free height	4.6	m
Height concrete column	0.5	m
Width concrete column	0.5	m

Table H.3: Variables in model, values for screwed connection between main girder and deck panel

Element	Variable	Value	Unit
MG	Height	0.68	m
MG	Width	0.40	m
MG	Material	GL28h	-
CG	Height	0.40	m
CG	Width	0.22	m
CG	Material	GL28h	-
DP	Height	0.33	m
DP	Material	CLT LL-330/9s	-
Strut	Height	0.5	m
Strut	Width	0.26	m
Strut	Material	GL28h	-
MG-MG connection	Diameter bolt	30	mm
MG-MG connection	Bolt strength class	8.8	-
MG-MG connection	Number of steel plates	1	-
MG-MG connection	Number of bolts in x direction	2	-
MG-MG connection	Number of bolts in z direction	4	-
CG-CG connection	Diameter bolt	24	mm
CG-CG connection	Bolt strength class	8.8	-
CG-CG connection	Number of steel plates	1	-
CG-CG connection	Number of bolts in x direction	2	-
CG-CG connection	Number of bolts in z direction	2	-
DP-MG connection	Screw diameter	11	mm
DP-MG connection	Screw length	700	mm
DP-MG connection	sg	385	mm
DP-MG connection	Number of screw sets	3	-
DP-DP connection	Rotational stiffness	0	N/mm
MG-CG connection	Translational stiffness	Infinite	-
Strut connection	Translational stiffness	Infinite	-

Table H.4: Variables in model, values for final design

Element	Variable	Value	Unit
MG	Height	0.70	m
MG	Width	0.40	m
MG	Material	GL28h	-
CG	Height	0.40	m
CG	Width	0.22	m
CG	Material	GL28h	-
DP	Height	0.33	m
DP	Material	CLT LL-330/9s	-
Strut	Height	0.5	m
Strut	Width	0.26	m
Strut	Material	GL28h	-
Connection	Bolt/screw grade	10.9	-
MG-MG connection	Diameter bolt	24	mm
MG-MG connection	Number of steel plates	1	-
MG-MG connection	Number of bolts in x direction	2	-
MG-MG connection	Number of bolts in z direction	2	-
CG-CG connection	Diameter bolt	18	mm
CG-CG connection	Number of steel plates	1	-
CG-CG connection	Number of bolts in x direction	2	-
CG-CG connection	Number of bolts in z direction	2	-
DP-MG connection	Screw diameter	11	mm
DP-MG connection	Screw length	700	mm
DP-MG connection	sg	335	mm
DP-MG connection	α	35	degrees
DP-MG connection	Number of screw sets	4	-
DP-DP connection x	Number of bolts	4	-
DP-DP connection x	Diameter bolt	16	mm
DP-DP connection y	Number of bolts	1	-
DP-DP connection y	Diameter bolt	16	mm
DP-DP connection	Rotational stiffness	0	N/mm
MG-CG connection	Translational stiffness	Infinite	-
Strut connection	Translational stiffness	Infinite	-

The standard bridge for which the detailed design is performed is presented in Table H.5.

Table H.5: Variables in model

Variable	Value	Unit
Length	35	m
Width	15	m
Number of spans	1	-
Skew	100	gon

Modular system

I.1. Scope of modular system

Table I.1: Percentage of spans in database Rijkswaterstaat

Rounded free span	1	2	3	4	5
5	0.08	0	0.58	0.25	0
10	0.17	0.17	12.78	4.4	0.25
15	0.91	0.75	9.7	13.53	2.57
20	2.82	3.15	3.49	7.47	1.2
25	2.41	10.0	0.81	1.83	0.17
30	0.5	8.8	0.83	0.91	0.25
35	0.17	1.74	1.08	0.25	0.17
40	0.08	0.08	0.08	0.08	0.25
45	0.17	0	0	0	0
50	0.08	0	0	0	0

Table I.2: SLS and ULS results with part of structures with and without struts for demountable modules

Rounded free span	1 SLS	1 ULS	2 SLS	2 ULS	3 SLS	3 ULS	4 SLS	4 ULS	5 SLS	5 ULS
5	x	x	0.41	0.33	0.27	0.33	0.28	0.33	0.27	0.45
10	x	x	0.56	0.85	0.62	0.41	0.60	0.79	0.56	0.63
15	0.77	0.70	0.85	0.75	0.96	0.75	0.97	0.76	0.95	0.76
20	0.40	1.02	0.44	0.99	x	x	x	x	x	x
25	0.51	0.83	0.76	0.95	x	x	x	x	x	x
30	0.99	0.82	x	x	x	x	x	x	x	x
35	x	x	x	x	x	x	x	x	x	x
40	x	x	x	x	x	x	x	x	x	x
45	x	x	x	x	x	x	x	x	x	x
50	x	x	x	x	x	x	x	x	x	x

Table I.3: Percentage of free spans in design space, data according to database of Rijkswaterstaat

Rounded free span	1	2	3	4	5
5	0.08	0	0.58	0.25	0
10	0.17	0.17	12.78	4.4	0.25
15	0.91	0.75	9.7	13.53	2.57
20	2.82	3.15	3.49	7.47	1.2
25	2.41	10.0	0.81	1.83	0.17
30	0.5	8.8	0.83	0.91	0.25
35	0.17	1.74	1.08	0.25	0.17
40	0.08	0.08	0.08	0.08	0.25
45	0.17	0	0	0	0
50	0.08	0	0	0	0

I.2. Grade of demountability

I.2.1. Inclined screws

According to Tomasi et al. (2010), the stiffness of an inclined screwed connection can be defined with equation:

$$K_{ser} = K_{\perp} \cdot cos^2\alpha + K_{\parallel} \cdot sin^2\alpha \tag{I.1a}$$

$$K_{\perp} = \rho_m^{1.5} \cdot d/23 \tag{I.1b}$$

$$K_{\parallel} = K_{ser,ax,i} \tag{I.1c}$$

$$K_{ser,ax,i} = 30 \cdot s_g \cdot d \tag{I.1d}$$

where

- α is the inclination angle of the screw
- d is the outer diameter of the screw
- ρ_m is the density of the timber
- s_g is the embedment length of the threaded segment of the screw [mm]

For an inclined screw connection with the following properties, obtained from **?**:

Table I.4: Properties of inclined screws

Property	Value	Unit
Length	700	mm
s_g	385	mm
ρ_m	460	kg/m^3
d	11	mm
α	40	degrees

Which leads to the following slip modulus:

Table I.5: Slip modulus of inclined screws

Property	Value	Unit
$K_{\perp}$	4718.47	N/mm
$K_{\parallel}$	110550	N/mm
K_{ser}	31176	N/mm

For the dowelled connection, the slip modulus is 10295 N/mm per dowel (as calculated in Appendix E), thus three times less stiff compared to the inclined screws.

Table I.6 summarizes the slip modulus for the three types of deck-main girder connectors.

Table I.6: Slip modulus of three options

Connection	Value	Unit
Dowelled	6*10295	N/mm
Inclined screws	6*31176	N/mm
Glued	Infinite stiff	-

Results modules with inclined screws

Table I.7: SLS and ULS results with part of structures with and without struts for solid modules

Rounded free span	1 SLS	1 ULS	2 SLS	2 ULS	3 SLS	3 ULS	4 SLS	4 ULS	5 SLS	5 ULS
5	x	x	0.33	0.33	0.22	0.33	0.23	0.33	0.22	0.45
10	x	x	0.46	0.93	0.51	0.46	0.48	0.81	0.44	0.65
15	0.63	0.76	0.70	0.81	0.83	0.83	0.84	0.83	0.83	0.83
20	0.34	0.94	0.37	0.97	x	x	x	x	x	x
25	0.42	0.98	0.67	1.01	x	x	x	x	x	x
30	0.86	0.90	x	x	x	x	x	x	x	x
35	x	x	x	x	x	x	x	x	x	x
40	x	x	x	x	x	x	x	x	x	x
45	x	x	x	x	x	x	x	x	x	x
50	x	x	x	x	x	x	x	x	x	x

I.3. Application in practice

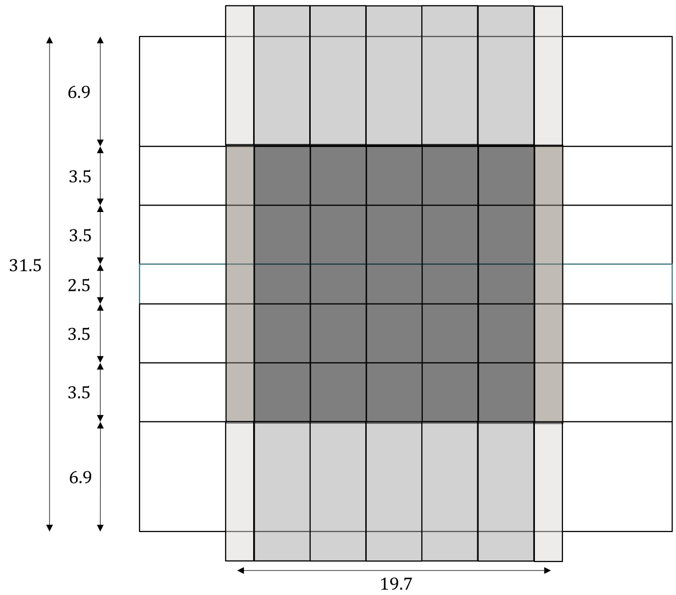


Figure I.1: Module configuration. Overpass: 2x2 lanes with a middle verge. Underpass: 2x2 lanes with a middle verge and taluds.

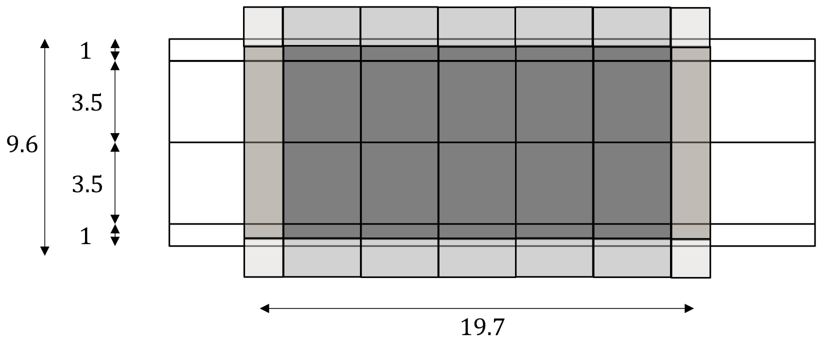


Figure I.2: Module configuration. Overpass: 2x1 lanes with a middle verge. Underpass: 2x1 lane.

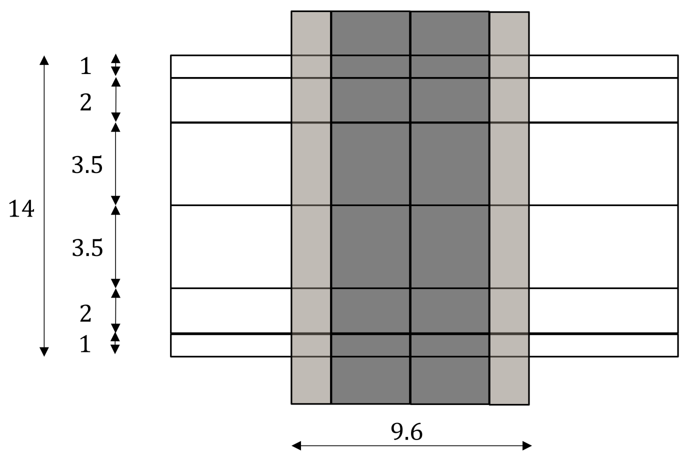


Figure I.3: Module configuration. Overpass: 2x1 lane. Underpass: 2x1 lane with a bike lane on both sides.

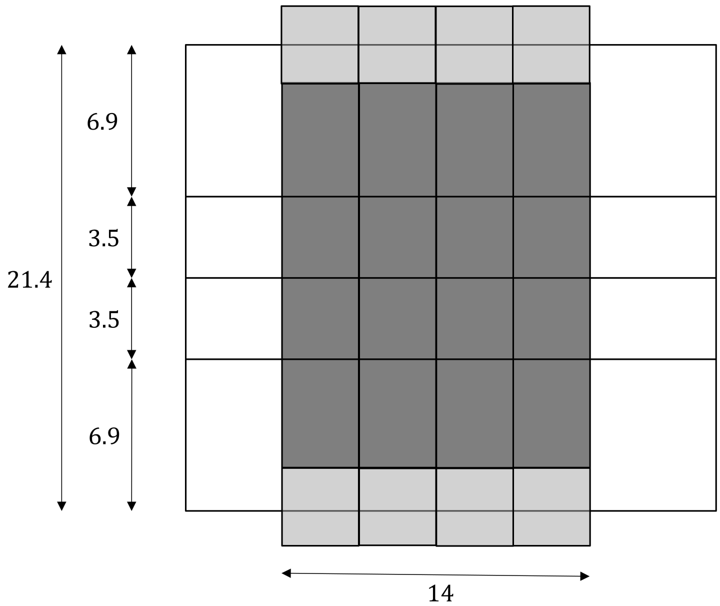


Figure I.4: Module configuration. Overpass: 2x1 lane with bikelanes. Underpass: 2x1 lane with taluds on both sides.

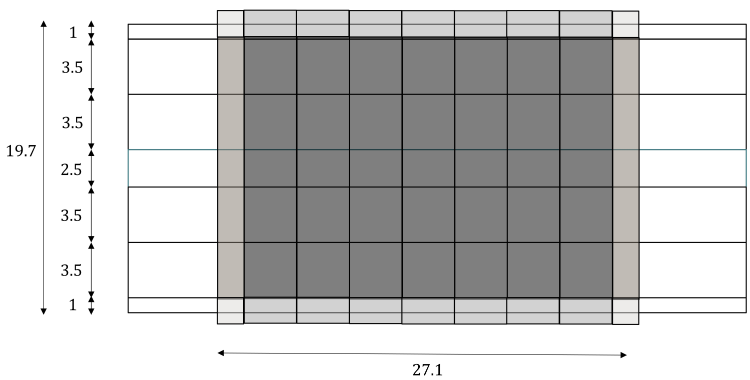


Figure I.5: Module configuration. Overpass: 2x3 lanes with a middle verge. Underpass: 2x2 lane with a middle verge.

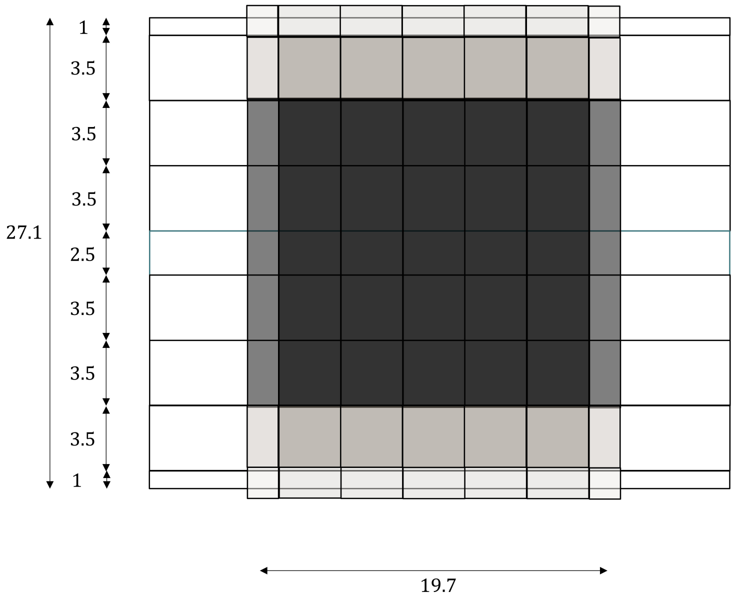


Figure I.6: Module configuration. Overpass: 2x2 lane with a middle verge. Underpass: 2x3 lane with a middle verge.

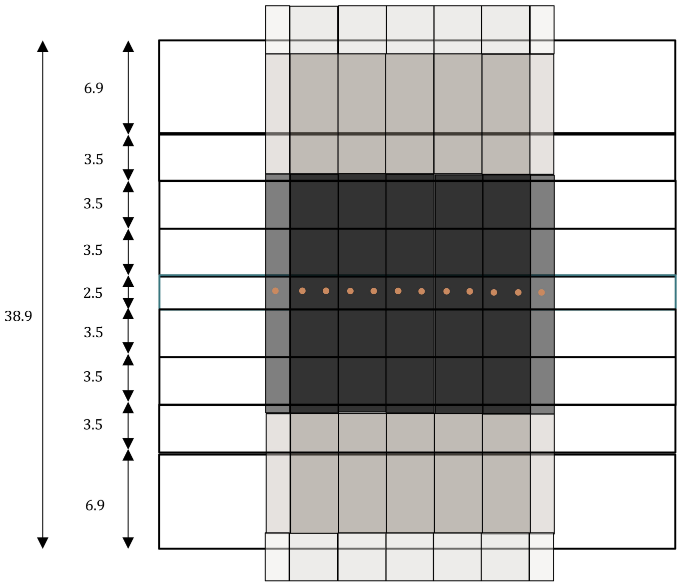


Figure I.7: Module configuration. Overpass: 2x2 lane with a middle verge. Underpass: 2x3 lane with a middle verge and taluds. Support points are required as the maximum span is exceeded.

I.4. Camber

According to Thelandersson (1995), the minimum required camber can be defined by the following equation:

$$h_{camber,min} = u_{G,k} + 0.5 \cdot u_{Q1,k} \cdot \psi_{1,Q1} \tag{I.2}$$

$\psi_{1,Q1}$ is 0.75 for the tandem system, and 0.4 for the uniformly distributed load. For the deflection due to self-weight, the long term deflection is taken into account, including creep. This is described in the chapter on loads and resulted in $1.8 \cdot u_{inst,G}$ Both values result in the following equation for the load case for camber.

$$h_{camber,min} = u_{inst,G} \cdot 1.8 + u_{TS,k} \cdot 0.375 + u_{UDL,k} \cdot 0.2 \tag{I.3}$$

The deflection according to the load case from Equation I.3. For all bridges that are in the selection of Table 7.5, the deflection due to the load case is calculated. To convert this into a required radius, Equation I.5 is applied.

$$R^2 = (R - C)^2 + (0.5l)^2 \tag{I.4}$$

$$R = \frac{C^2 + 0.25l^2}{2C} \tag{I.5}$$

This is calculated for all eighteen bridges in the selection, the results are presented in Figure I.8. The cross sections are used as presented in Table H.3. The size of the dots are according to the percentage of bridges with that span and the number of spans. The minimum radius to apply for the selection is 1981 meter.

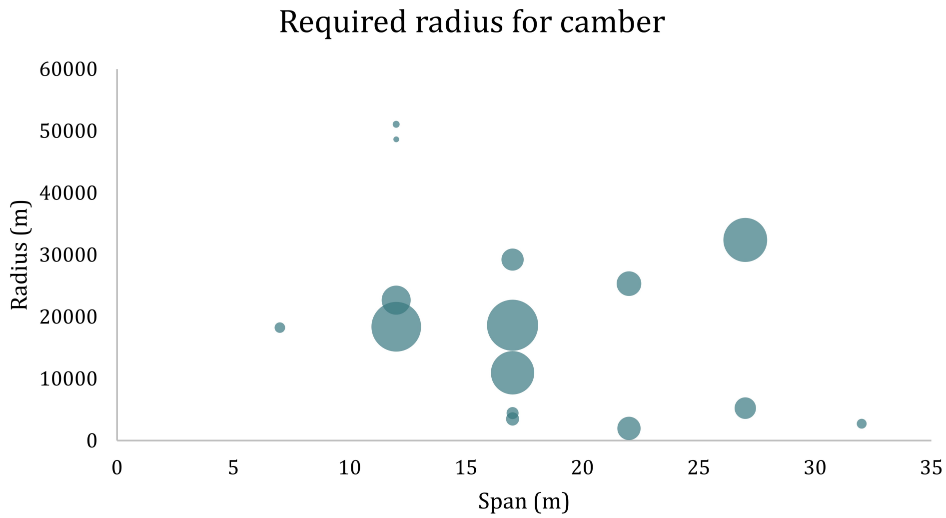


Figure I.8: Minimum radius for bridges in selection

For a module of 17.5 meter length, the following height difference occurs, with α in rad and C in mm:

$$\alpha = \sin^{-1}\left(\frac{0.5 \cdot l}{R}\right) = \sin^{-1}\left(\frac{8.75}{1981}\right) = 4.41 \cdot 10^{-3} \tag{I.6}$$

$$C = R - cos(\alpha) \cdot R = 19.3m \tag{I.7}$$

To be able to realise this camber, the different elements must be connected with the same radius. An issue occurs here: glulam can easily be produced with a curvature, but CLT can not. However, the glulam has the largest structural height and therefore ensures the most stiffness. The CLT panel is relatively thin, and therefore more flexible. To make sure that the CLT panel can fit to the glulam curvature, the deflection due to its self-weight is calculated. The equivalent EI is calculated for the CLT panel with the dimensions as given in Table H.4. An equivalent EI of $2.921 \cdot 10^{13}$ Nmm is obtained. When the panel is placed on one support point in the middle (which is the highest point of the glulam beam), the following deflection is obtained [mm].

$$w = \frac{ql^4}{8EI} = \frac{1.49 \cdot 8750^4}{8 \cdot 2.921 \cdot 10^{13}} = 37.38 \tag{I.8}$$

The self-weight of 1.49 kN/m^2=1.49 N/mm is obtained from the brochure of X-LAM from Derix (2020). As shown in the equation, a deflection of 37.38 mm occurs due to self-weight. A camber of 19.3 mm is applied for the large module, so the deflection due to self-weight is sufficient. The CLT panel will not deflect more than the camber, as it touches the glulam beams and is screwed to that. The execution of the connection process is presented in Figure 7.36.

J

Structural verification of members

J.1. Material analysis

The results of the structural analyses as described below are according to the calculations as explained in Section J.3.

J.1.1. Beam material analysis

Table J.1: Unity checks beam material analysis, cross section is kept constant

Check	GL24h	GL28h	GL32h	LVL-S
Tension parallel to grain	0.690	0.705	0.525	0.367
Tension perpendicular to grain	0.232	0.231	0.220	0.335
Compression parallel to grain	0.043	0.040	0.025	0.023
Compression perpendicular to grain	0.041	0.042	0.040	0.041
Bending moment	0.097	0.099	0.081	0.066
Combination of tension and bending	0.329	0.331	0.301	0.401
Shear	0.029	0.029	0.027	0.062
Deflection	0.93	0.87	0.84	0.81

J.1.2. Deck material analysis

Table J.2: Unity checks deck material analysis, cross section is kept constant

Check	CLT	LVL-X	LVL-S	SLT C24	SLT GL28h
Tension primary direction ($N_{xx,T}$)	0.377	0.261	0.179	0.280	0.218
Tension secondary direction ($N_{yy,T}$)	0.058	0.025	0.040	0.295	0.281
Compression primary direction ($N_{xx,C}$)	0.551	0.517	0.342	0.432	0.393
Compression secondary direction ($N_{yy,C}$)	0.085	0.073	0.017	0.081	0.084
Rolling shear	0.329	0.329	0.263	0.263	0.263
M_{xx}	0.391	0.237	0.332	0.416	0.431
M_{yy}	0.125	0.069	0.002	0.054	0.048
SLS	0.87	0.88	1.06	1.08	0.95

J.2. Connections

All connections are described and calculated in this section. Checks in ULS are provided and the stiffness of the connection as well. After the calculation of the last connection, the outcome for SLS requirements is given, as these influence each other. If required, the timber cross sections are adjusted.

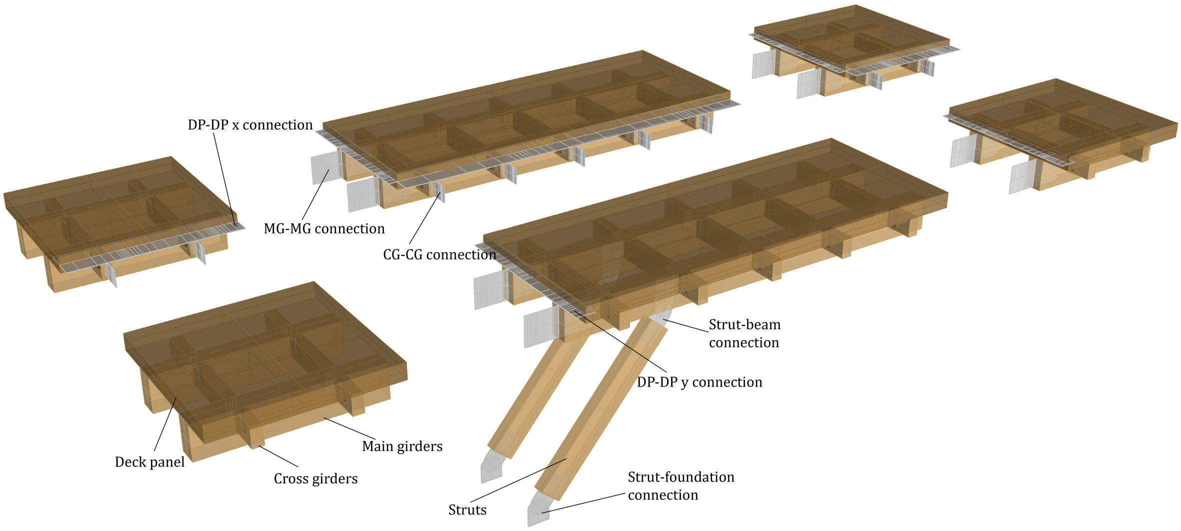


Figure J.1: Location of connections

J.2.1. Deck panel to main girder

As defined in Section 7.3.2, the connection between the deck panel and main girders is realised by inclined screws. This is an important connection for the global behaviour, as composite behaviour is enabled. The relative slip between the girders and deck create this composite behaviour. The slip can occur because of the inclined screws, with significant slip stiffness. The stiffness of the connection is already defined, and the strength must be verified.

According to EN1995-1-1, the following equation must be used for the strength of the connection:

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leq 1 \tag{J.1}$$

where

- $F_{ax,Ed}$ is the design axial stress [N]
- $F_{ax,Rd}$ is the design axial strength [N]
- $F_{v,Ed}$ is the design lateral stress [N]
- $F_{v,Rd}$ is the design shear strength [N]

When the following two conditions are satisfied:

- 6 mm $\leq$ d $\leq$ 12 mm
- 0.6 $\leq d_1/d \leq$ 0.75

the characteristic embedment strength is defined as:

$$F_{ax,k,Ed} = \frac{n_{ef} \cdot f_{ax,k} \cdot d \cdot l_{ef} \cdot k_d}{1.2 \cdot cos^2\alpha \cdot sin^2\alpha} \tag{J.2a}$$

$$f_{ax,k} = 0.52d^{-0.5} \cdot l_{ef}^{-0.1} \cdot \rho_k^{0.8} \tag{J.2b}$$

$$k_d = min\left(\frac{d}{8}; 1\right) \tag{J.2c}$$

where

- n_{ef} = effective number of fasteners [-]
- d = diameter of the screw, including thread [mm]
- l_{ef} = effective length of the screw = length of the threaded part [mm]
- ρ_k = characteristic density of the timber [kg/m^3]

For the shear strength, the Johanssen model is applied, of which the equations are shown in Equation J.3. To include the rope effect, the contribution of the axial resistance can not be more than the contribution of the first part of the equation. Otherwise the resistance is two times the Johansen part of the equation for screws. This should be checked for equation c until f.

$$f_{h,1,k} \cdot t_1 \cdot d \tag{J.3a}$$

$$f_{h,2,k} \cdot t_2 \cdot d \tag{J.3b}$$

$$\frac{f_{h,1,k} \cdot t_1 \cdot d}{1+\beta} \cdot \left(\sqrt{\beta + 2 \cdot \beta^2 \cdot \left(1 + \frac{t_2}{t_1} + \left(\frac{t_2}{t_1}\right)^2\right) + \beta^3 \cdot \left(\frac{t_2}{t_1}\right)^2} - \beta \cdot \left(1 + \frac{t_2}{t_1}\right) + \frac{F_{ax,Rk}}{4}\right. \tag{J.3c}$$

$$1.05 \cdot \frac{f_{h,1,k} \cdot t_1 \cdot d}{2+\beta} \cdot \left(\sqrt{2\beta(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_1^2}} - \beta\right) + \frac{F_{ax,Rk}}{4} \tag{J.3d}$$

$$1.05 \cdot \frac{f_{h,1,k} \cdot t_2 \cdot d}{2+\beta} \cdot \left(\sqrt{2\beta^2(1+\beta) + \frac{4\beta(2+\beta)M_{y,Rk}}{f_{h,1,k} \cdot d \cdot t_2^2}} - \beta\right) + \frac{F_{ax,Rk}}{4} \tag{J.3e}$$

$$1.15\sqrt{\frac{2\beta}{1+\beta}} \cdot \sqrt{2M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4} \tag{J.3f}$$

In which the following equations are used for $f_{h,k}$, $M_{y,Rk}$, n_{ef} and β:

$$f_{h,k} = 0.082 \cdot (1 - 0.01 \cdot d) \cdot \rho_k \tag{J.4a}$$

$$M_{y,Rk} = 0.3 \cdot f_u \cdot d^{2.6} \tag{J.4b}$$

$$n_{ef} = min\left(n; n^{0.9} \cdot \left(\frac{a_1}{13d}\right)^{0.25}\right) \tag{J.4c}$$

$$\beta = \frac{f_{h,k,1}}{f_{h,k,2}} \tag{J.4d}$$

where:

- t_1 = thickness of timber element 1 in which the fastener is present [mm]
- t_2 = thickness of timber element 2 in which the fastener is present [mm]
- f_u = tensile strength of the fastener [N/mm^2]
- a_1 = distance between fasteners, perpendicular to the fastener [mm]

As the screws are inclined, the definition of shear and axial strength change. After multiple iterations, changing the angle of the screws, the following input values are used which lead to the result as stated below.

Table J.3: Input values for calculation strength inclined screws

Variable	Value	Unit
α	35	degrees
d	11	mm
length	700	mm
s_g	335	mm
ρ_{mean}	460	kg/m^3
ρ_k deck	350	kg/m^3
ρ_k beam	425	kg/m^3
f_u	1000	N/mm^2
t_1	330	mm
t_1	243	mm
a_1	401	mm
n	6.0	-

The centre-to-centre distance of the screws does not match the mesh of the deck in the parametric model, the ctc distance of this mesh is taken and the number of fasteners is adjusted. The number of fasteners over the width of the main girders is determined by three parameters:

- a_2=5d, distance between pairs of inclined screws;
- across=1.5d, distance between two screws within a pair;
- a_2,cg=4d, distance to the outer edge of the element.

For a main girder with a width of 400 mm, four pairs of screws can be applied.

The following shear strengths according to the Johansen model are obtained:

Table J.4: Resistance of failure modes according to Johansen model for inclined screws

Failure mode	Resistance [kN]
a	100.73
b	90.22
c	51.49
d	79.51
e	64.98
f	24.81

In conclusion, the shear strength of one screw $F_{v,Rd}$=24.81 kN. With 5.06 effective screws, the shear force of the group is 125.63 kN. The axial strength of the group is 1498.91 kN, following equation J.2. The safety factor for connections in timber (1.3) gives $F_{ax,loc}$=1185.65 kN and $F_{v,loc}$=84.40 kN With an inclination of 35 degrees, the axial and shear capacity in the direction of the forces on a global scale is as follows:

$$F_{ax,gl} = \sin(\alpha) \cdot F_{ax,loc} + \cos(\alpha) \cdot F_{v,loc} \tag{J.5}$$

$$F_{v,gl} = \sin(\alpha) \cdot F_{v,loc} + \cos(\alpha) \cdot F_{ax,loc} \tag{J.6}$$

Resulting in $F_{ax,gl}$=999.92 kN and $F_{v,gl}$=740.52 kN. The occurring normal force is 858.93 kN and the shear force is 237.27 kN. The unity check of the connection is as follows:

$$\left(\frac{F_{ax,Ed}}{F_{ax,Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 = \left(\frac{858.93}{999.92}\right)^2 + \left(\frac{237.27}{740.52}\right)^2 = 0.84 \tag{J.7}$$

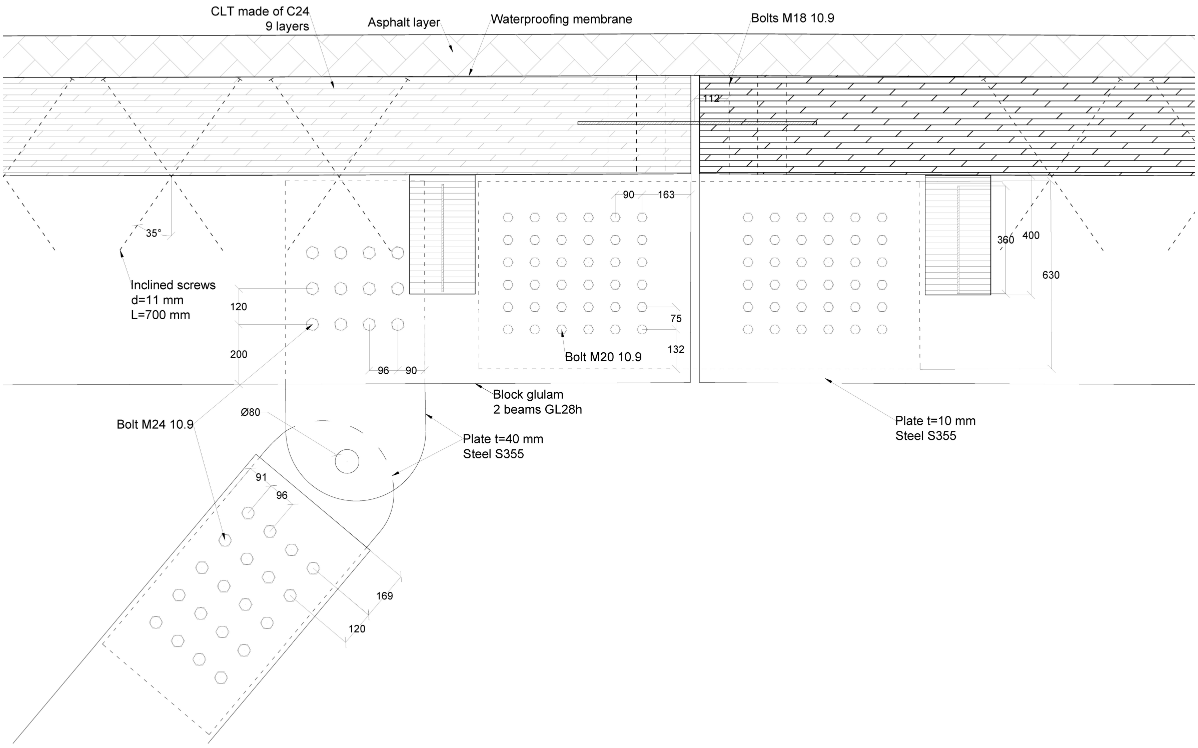


Figure J.2: Connection between main girders and deck

J.2.2. Beam splices

For the serviceability limit state requirements, the beam splice connections are crucial. Rotational capacity should be obtained here, in order to reduce the deflection. As described in Section 5.3, a connection with slotted-in steel plates is suited to obtain this rotational capacity.

$$K_r = K_{ser} \cdot \sum_{i=1}^{n} x_i^2 + y_i^2 \tag{J.8}$$

$$K_{ser} = \frac{\rho_m^{1.5} \cdot d}{23} \cdot 2 \tag{J.9}$$

where ρ_m is the mean density of the timber and d is the diameter of the bolt. x_i and y_i are the distances to the center to the connection in respectively x and y direction. In Equation J.9, the stiffness can be multiplied by two, as steel plates are used.

Main girders

The main girders are 700 mm in height, made out of GL28h. For the analyses as performed in the previous chapters, bolt M30 was used in strength class 8.8. This section defines whether the connection satisfies both SLS and ULS requirements. The final design of the connection is shown in Figure J.4

As the connection is at the location of a beam splice in the main load-bearing element, prevention of failure is crucial. The module arrangement is optimized to occur at locations with limited bending moment and shear force, but in theory, it can occur at any location. Therefore, the connection is dimensioned on 50% of the capacity of the timber beam. This requirement is used for steel structures, as defined in NEN-EN1992-1.

The capacity of the timber girders is the defined according to the equations in Section J.3.1. The following capacities are obtained:

Table J.5: Resistance timber main girders

Failure mode	Resistance [kN]
N (tension)	4368 kN
V	784 kN
M	731.73 kNm

Both shear and bending moment should be checked for this connection. A normal force in the main girder is a shear force for the connection. The combination of forces is checked by calculating the effective force on the bolts, induced by a combination of the bending moment and the normal and shear forces. For a rectangular arrangement, the outer fastener must be checked.

$$F_{VM} = \sqrt{\left(F_N + \frac{x_{max}}{\sqrt{x_{max}^2 + y_{max}^2}} \cdot F_M\right)^2 + \left(F_V + \frac{y_{max}}{\sqrt{y_{max}^2 + x_{max}^2}} \cdot F_M\right)^2} \tag{J.10}$$

The resultant force from the capacity is 5036 kN. The design value =5036*0.6=3022 kN. For 50% of this shear strength, a strength of 1511 kN must be obtained.

To determine the strength of one dowel, the Johansen model is applied again. The rope effect can account of 25% of the Johansen part, as dowels are applied. The strength of the connecting member is defined as the minimum of the values of Equation J.11, consisting of the failure modes in the Johansen model (f,g,h) for double shear timber-steel connections.

$$f_{h,1,k} \cdot t_1 \cdot d \tag{J.11a}$$

$$f_{h,1,k} \cdot t_1 \cdot d \cdot \left(\sqrt{2 + \frac{4M_{y,Rk}}{f_{h,k} \cdot d \cdot t_1^2}} - 1\right) + \frac{F_{ax,Rk}}{4} \tag{J.11b}$$

$$2.3\sqrt{M_{y,Rk} f_{h,1,k} d} + \frac{F_{ax,Rk}}{4} \tag{J.11c}$$

The three equations for the values of the strength are implemented in the model and are calculated according to the applied cross sections. Multiple iterations were performed and a sufficient design was obtained with six M20 bolts in x direction and six in z direction. For M20 in strength class 10.9, the following results are obtained:

Table J.6: Resistance of failure modes according to Johansen model for dowelled connection

Failure mode	Resistance [kN]
f	111.52
g	148.18
h	57.77

The lowest value is 57.77 kN, this is multiplied by the number of dowels, to result in the total shear capacity. 36 bolts are used in the connection on each side, and according to NEN 1995-1-1 (Equation 8.35) the effective number of bolts is equal to the number of bolts when loading is perpendicular. This results in a resistance of 1599.77 kN.

Next to the bolts, the steel plate has to be checked. This is done according to NEN-EN 1993-1-1:2016. The following variables are used for the steel plate design:

Table J.7: Variables for steel plate design main girder - main girder connection

Variable	Value	Unit
Grade	S335	-
f_y	355	N/mm^2
f_u	490	N/mm^2
γ_{M0}	1.0	-
γ_{M2}	1.25	-
t	30	mm
h	660	mm

The following conditions should be satisfied for the steel plate:

$$\frac{N_{Ed}}{N_{t,Rd}} \leq 1.0 \tag{J.12}$$

where for steel plates with holes in it (due to connections) the resistance is the minimum of:

$$N_{pl,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \tag{J.13a}$$

$$N_{u,Rd} = \frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} \tag{J.13b}$$

where the units are applied as stated in Table J.7, forces are in N and Anet is the net area, excluding the holes. This is calculating by reducing the height of the cross section by the diameters of the holes.

$$\frac{N_{Ed}}{N_{c,Rd}} \leq 1.0 \tag{J.14a}$$

$$N_{c,Rd} = \frac{A \cdot f_y}{\gamma_{M0}} \tag{J.14b}$$

$$\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0 \tag{J.15a}$$

$$M_{c,Rd} = M_{el,Rd} = \frac{W \cdot f_y}{\gamma_{M0}} \tag{J.15b}$$

where M is in Nmm and W in mm^3. Holes in the steel plate can be neglected for calculation of the bending moment capacity if the following is satisfied:

$$\frac{0.9 \cdot A_{net} \cdot f_u}{\gamma_{M2}} \geq \frac{A_f \cdot f_y}{\gamma_{M0}} \tag{J.16}$$

With the values of the forces as stated in Table J.5, the following unity checks are obtained:

Table J.8: Unity checks steel plate in MG-MG connection

Check	Occurring force	Resistance	UC
Tension	4368 kN	6477.41 kN	0.67
Bending	731.73 kNm	773.19 kNm	0.95

The steel plate satisfies the requirements.

Table J.9: Resistance of failure modes according to Johansen model for bolted connection in CG-CG

Failure mode	Resistance [kN]
f	69.92
g	110.73
h	78.18

Cross girders

The process for the cross girders works in the same manner as the main girders.

The initial design for this connection satisfies the ULS requirements and is described below. 2 bolts in x-direction and 2 bolts in z-direction of size M24 are applied, in strength 10.9. Table J.9 presents the resistances of the failure modes f until h.

Therefore the resistance of one bolt is 69.92 kN. The maximum forces occurring in the main girders at the location of this connection are:

Table J.10: Occurring forces in governing CG-CG connection

Force	Maximum	Minimum	Unit
N	50.88	-1.75	kN
V	59.42	-2.81	kN
M	0.78	-0.16	kNm

The occurring force on the bolt is 78.24 kN. 2 * 2 bolts can be accounted as four effective bolts, and have a resistance of 215.15 kN (with a safety factor of 1.3 applied), this configuration is therefore assumed to be sufficient (UC=0.36).

Similar to the main girder, the steel plate has to be checked as well. The checks as described in the previous section are summarized below. First, the properties of the steel plate are given in Table J.11, then the results of the calculation are stated in Table J.12.

Table J.11: Variables for steel plate design cross girder - cross girder connection

Variable	Value	Unit
Grade	S335	-
f_y	355	N/mm^2
f_u	490	N/mm^2
γ_{M0}	1.0	-
γ_{M2}	1.25	-
t	10	mm
h	360	mm

Table J.12: Unity checks steel plate in CG-CG connection

Check	Occurring force	Resistance	UC
Tension	50.88 kN	1100.74 kN	0.046
Compression	1.75 kN	1278 kN	0.001
Bending	0.78 kNm	76.7 kNm	0.010

This is again sufficient. The final design is shown in Figure J.3.

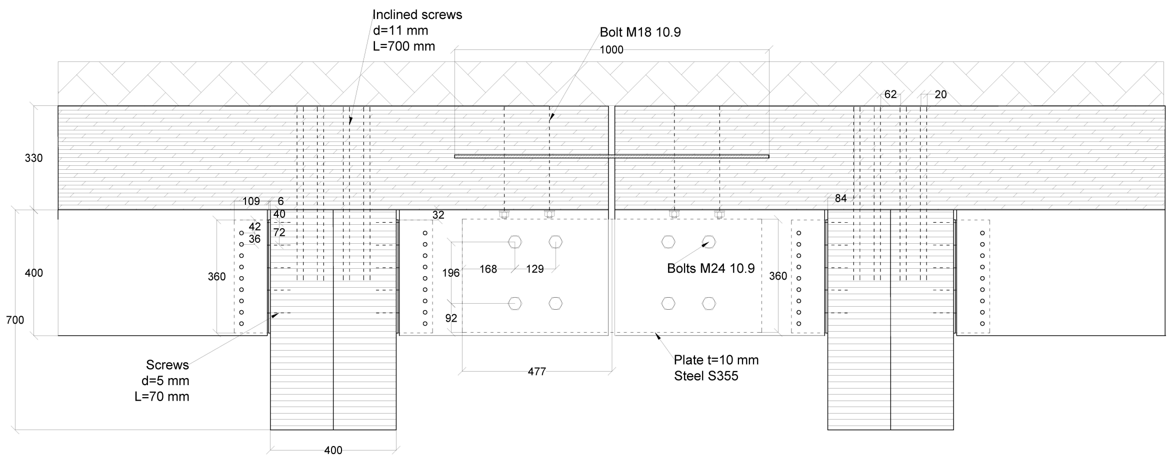


Figure J.3: Connection of cross girders to cross girders and cross girders to main girders. Joint between deck panels in global x-direction is shown as well.

J.2.3. Deck panels

In the first phase of this research, the stiffness of the links between the deck panels was set to zero. By doing so, the deck panels would not move individually from each other, but they would not transfer any forces or provide stiffness. In the actual design, they can transfer forces and contribute to the stiffness of the structure.

As shown in Section 5.3, the slotted-in steel plate connection scores best for the deck panel to deck panel connection. These are placed along the edge of a module, to make sure that the panels can transfer forces between themselves. As panels are connected, the steel plates can not be inserted as a beam (higher than wide) but they are inserted as plates. Again, bolts are used to attach the timber to the steel plate.

The bolts are checked according to the Johansen model with steel plates, as described before. A separate design is made for the connection in global x and global y direction.

Global x direction

The connection along the global x-direction (span direction) is considered here. It is placed along the long edges of the modules. Bolts of size M16 are applied, in strength class 10.9. Three bolts are applied along the edge. Below, the resistances of the failure modes are given, followed by the unity checks. The geometry is shown in Figure J.3.

Table J.13: Resistance of failure modes according to Johansen model for bolted connection in deck panels, global x direction

Failure mode	Resistance [kN]
f	63.65
g	85.48
h	35.95

Therefore the resistance of one bolt is 35.95 kN. The maximum normal force in the deck is 1606 kN/m. With 4*14.2 bolts per running meter, the design resistance is 1659 kN, and therefore sufficient.

The steel plate is checked as well, similar to the other slotted-in steel plate connections. First, the properties of the steel plate are given in Table J.14, then the results of the calculation are stated in Table J.15.

This is sufficient.

Table J.14: Variables for steel plate design DP-DP connection, x direction

Variable	Value	Unit
Grade	S335	-
f_y	355	N/mm^2
f_u	490	N/mm^2
γ_{M0}	1.0	-
γ_{M2}	1.25	-
t	10	mm
b	500	mm

Table J.15: Unity checks steel plate in DP-DP connection, x direction

Check	Occurring force	Resistance	UC
Tension	346.2 kN	1538.2 kN	0.23
Compression	1695.8 kN	1775.0 kN	0.96

Global y direction

Bolts of size M16 are applied, in strength class 10.9. Two bolts are applied along the edge. Below, the resistances of the failure modes are given, followed by the unity checks.

Table J.16: Resistance of failure modes according to Johansen model for bolted connection in deck panels, global y direction

Failure mode	Resistance [kN]
f	63.64
g	85.48
h	35.95

Therefore the resistance of one bolt is 35.95 kN. The maximum normal force in the deck is 279.91 kN/m. With 15.2 bolts per running meter, the design resistance is 497.87 kN, and therefore sufficient (UC=0.56).

The steel plate is checked as well, similar to the other slotted-in steel plate connections. First, the properties of the steel plate are given in Table J.17, then the results of the calculation are stated in Table J.18.

Table J.17: Variables for steel plate design DP-DP connection, y direction

Variable	Value	Unit
Grade	S335	-
f_y	355	N/mm^2
f_u	490	N/mm^2
γ_{M0}	1.0	-
γ_{M2}	1.25	-
t	10	mm
b	250	mm

Table J.18: Unity checks steel plate in DP-DP connection, y direction

Check	Occurring force	Resistance	UC
Tension	248.8 kN	825.5 kN	0.26
Compression	233.1 kN	887.5 kN	0.30

This is again sufficient.

Stiffness

No rotational stiffness is provided by this connection in the parametric model. Some rotational stiffness can be created, but a conservative approach is taken, as a lot of uncertainty lies in this stiffness. Therefore, when the model satisfies the SLS requirements without rotational stiffness, the behaviour can only improve. Slip occurs in the connection and is implemented accordingly.

J.2.4. Cross girder to main girder

The connection between the cross girders and main girders is simulated as a hinged connection. Consequently, the cross girders work as two-force members, and no moments need to be transferred in the connection. As stated before, connections that need to transfer bending moments are hard to establish with timber. The connection of a concealed bracket is used for this connection. This connection is stronger than a beam hanger.

Among others, Rothoblaas produces these concealed brackets. A design software is provided by Rothoblaas, called MyProject. In this software, the dimensions, materials and acting force can be inserted, and the unity check is calculated by the program. The following properties are used to calculate the strength of the bracket.

Table J.19: Variables for steel plate design DP-DP connection, y direction

Variable	Value	Unit
$F_{v,Ed}$	64.5	kN
γ_M	1.3	-
$h_{bracket}$	360	mm
d_{screw}	5	mm
l_{screw}	70	mm
Number of screws	5	-
d_{dowel}	12	mm
l_{dowel}	220	mm
Number of dowels	9	-

A design resistance of 102.8 kN is obtained, which is sufficient for the acting load of 64.5 kN.

Stiffness

No rotational stiffness is provided by this connection. However, slip occurs in the connection, which is defined by the slip in the dowels: $K_{ser} = \frac{460^{1.5} \cdot 12}{23} \cdot 2 = 5147$ kN/m. Nine dowels are applied over the height, thus a slip of 9*5147=47327 kN/m is obtained.

J.2.5. Strut connection

For the strut connection, a connection is applied as shown in Figure 5.20. To calculate this connection, the pin connection must be designed, and the connection between the steel plate and the timber. It is decided that the connection to the strut has one steel plate, and the connection to the timber beam has two. This is shown in Figure J.5. By doing so, the strength of the connection can be considered twice the strength of the failure mode with the lowest strength. Consequently, less bolts are required in the connection to the main girder, which is beneficial as the space available is limited.

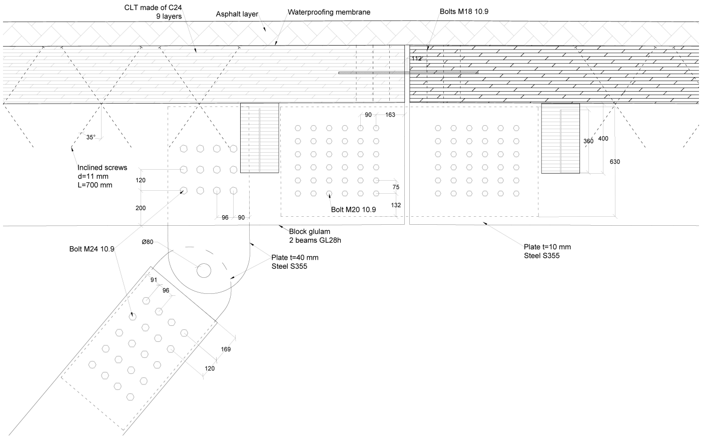


Figure J.4: Section of connection strut to main girders. Also shows connection between main girders and deck

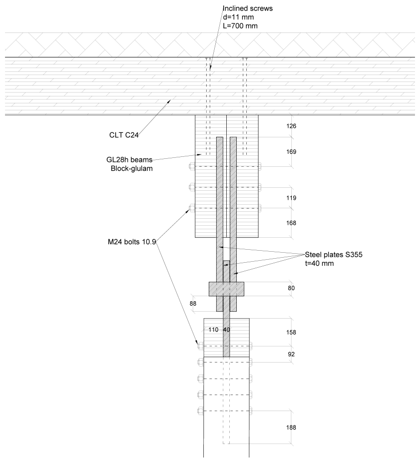


Figure J.5: Side view of connection strut to main girders

First, the connection between the steel plate and timber should be verified. This is done first, using the Johansen model, as shown in equation J.11. A rope effect of 25% is included, and bolt size M24 is used in strength class 10.9.

Table J.20: Resistance of failure modes according to Johansen model for strut timber-steel connection

Failure mode	Resistance [kN]
f	82.64
g	123.66
h	78.18

Thus the resistance of one bolt is 78.18 kN. Five bolts are applied in the direction parallel to the grain, and four perpendicular to the grain. In the main girder, this is three bolts in the height of the beam. As the connection to the strut is less strong, this one is elaborated below. A design resistance of 1202.7 kN is obtained. With a design resultant force of 1007.2 kN, this is sufficient.

Secondly, the steel plate is designed for compression, following expression J.14. A compressive force of 1007.2 kN occurs in the connection. A minimum requirement is set for the plate thickness:

$$t \geq 0.7 \cdot \sqrt{\frac{F_{Ed}}{f_y}} \tag{J.17}$$

Resulting in a thickness of 37 mm for this pinned connection. However, 37 is not a common used thickness. Therefore, a thickness of 40 mm is applied. With this thickness and 480 mm width, a design resistance of 6567.5 kN is obtained. This is sufficient.

Table J.21: Variables for steel plate design DP-DP connection, y direction

Variable	Value	Unit
f_{up}	490	N/mm^2
f_{yp}	355	N/mm^2
γ_{M0}	1.0	-
γ_{M2}	1.25	-
$\gamma_{M6,ser}$	1.0	-
d_{pin}	80	mm
t_{plate}	40	mm

Thirdly, the pin itself is designed. According to EC1993-1-8 (Chapter 3.13), the following failure modes can occur and must be checked:

$$F_{v,Rd} = \frac{0.6 \cdot A \cdot f_{up}}{\gamma_{M2}} \geq F_{v,Ed} \tag{J.18a}$$

$$F_{b,Rd} = \frac{1.5 \cdot t \cdot d \cdot f_y}{\gamma_{M0}} \geq F_{b,Ed} \tag{J.18b}$$

$$F_{b,Rd,ser} = \frac{0.6 \cdot t \cdot d \cdot f_y}{\gamma_{M6,ser}} \geq F_{v,ser} \tag{J.18c}$$

$$M_{Rd} = \frac{1.5 \cdot W_{el} \cdot f_{yp}}{\gamma_{M0}} \geq M_{Ed} \tag{J.18d}$$

$$M_{Rd,ser} = \frac{0.8 \cdot W_{el} \cdot f_{yp}}{\gamma_{M6,ser}} \geq M_{ser} \tag{J.18e}$$

$$W_{el} = \frac{\pi \cdot d^3}{32} \tag{J.18f}$$

$$\left(\frac{M_{Ed}}{M_{Rd}}\right)^2 + \left(\frac{F_{v,Ed}}{F_{v,Rd}}\right)^2 \leq 1 \tag{J.18g}$$

where

- d = diameter of the pin [mm]
- A = area of the pin [mm^2]
- t = thickness of the steel plate [mm]
- f_{up} = tensile strength of the pin [N/mm^2]
- f_{yp} = yield strength of the pin [N/mm^2]

With the variables as stated in Table J.21, the following result is obtained.

Table J.22: Resistances of pin connection

	Occurring force	Resistance	UC
$F_{v,Rd}$	998.3 kN	1182.2 kN	0.84
$F_{b,Rd}$	998.3 kN	1581.7 kN	0.63
$F_{b,Rd,ser}$	296.7 kN	632.7 kN	0.47
M_{Rd}	10.3 kNm	26.77 kNm	0.38
$M_{Rd,ser}$	3.05 kNm	14.28 kNm	0.21
Combination			0.86

J.2.6. Stiffness

The stiffness of the structural system is highly determined by the stiffness of the connections. In the end, the stiffness requirements are checked according to the deflection requirements. Below, the stiffness as inserted in the final structural model are listed. In the case of bolted connections, the stiffness is according to Equation J.8. The degrees of freedom are listed as done in Karamba, where t is translational and r is rotational. The axes are given according to the global coordination system. The vectors are given respectively to the degrees of freedom. Translational stiffness is expressed in kN/m and rotational stiffness in kNm/rad.

Table J.23: Stiffness of connections

Connection	Degrees of freedom	Stiffness
DP-DP x	tx, ty, rx	205897, 54906, 0
DP-DP y	tx, ty, ry	13726, 247076, 0
MG-DP	ty, tz	239753, 239753
MG-MG	tx, tz, ry, rz	41179, 41179, 22377, 0
CG-CG	tx, tz, ry, rz	41179, 41179, 3890, 0
MG-CG	rx, rz, tx, ty, tz	92654, 92654, 2224, 2224, 2224
Strut	tx, ry	411794, 0

J.3. Timber members

J.3.1. Checking of Glulam

According to NEN-EN1995-1-1, the following checks should be performed to verify the elements in a structural way:

$$\sigma_{t,0,d} \leq f_{t,0,d} \tag{J.19}$$

where $\sigma_{t,0,d}$ is the design value of the tension stress [N/mm^2], parallel to the grain direction. $f_{t,0,d}$ is the design value of the tension strength parallel to the grain direction [N/mm^2].

$$\sigma_{c,0,d} \leq f_{c,0,d} \tag{J.20}$$

where $\sigma_{c,0,d}$ is the design value of the compression stress, parallel to the grain direction [N/mm^2]. $f_{c,0,d}$ is the design value of the compression strength parallel to the grain direction [N/mm^2].

$$\sigma_{c,90,d} \leq k_{c,90,d} \cdot f_{c,90,d} \tag{J.21}$$

where

$$\sigma_{c,90,d} = \frac{F_{c,90,d}}{A_{ef}} \tag{J.22}$$

$\sigma_{c,90,d}$ is the design value of the compression stress, perpendicular to the grain direction [N/mm^2]. $F_{c,90,d}$ is the design value of the compression force perpendicular to the grain direction [N]. A_{ef} is the effective contact area for compression perpendicular to the grain [mm^2]. $f_{c,90,d}$ is the design value of the compression strength perpendicular to the grain direction [N/mm^2]. $k_{c,90,d}$ is a factor that takes into account the development of compression, the load configuration and possibility of separating. A value of 1.75 can be taken for $k_{c,90,d}$, since discrete supports are applied, and the elements are laminated.
For bending, the following equations should be satisfied:

$$\frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{J.23}$$

$$k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{J.24}$$

where $\sigma_{m,y,d}$ and $\sigma_{m,z,d}$ are design values for the bending stresses around the main axes [N/mm^2]. $f_{m,y,d}$ and $f_{m,z,d}$ are the design values for the bending strength [N/mm^2]. k_m is a factor that takes into account the redistribution of stresses, and is 0.7 for rectangular laminated sections.
For shear, the following check should be satisfied:

$$\tau_d \leq f_{v,d} \tag{J.25}$$

where τ_d is the design value of the shear stress. $f_{v,d}$ is the design value of the shear strength. To check for torsion, the following equation is used:

$$\tau_{tor,d} \leq k_{shape} \cdot f_{v,d} \tag{J.26}$$

where k_{shape} is

$$k_{shape} = min(1 + 0.15\frac{h}{b}, 2.0) \tag{J.27}$$

Karamba 3D calculates the value of the shear stress due to shear forces in Y and Z direction, combined with the shear stress due to torsion.

For a combination of bending and tension, the following should be checked:

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{J.28}$$

$$\frac{\sigma_{t,0,d}}{f_{t,0,d}} + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{J.29}$$

And for a combination of bending and compression:

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + \frac{\sigma_{m,y,d}}{f_{m,y,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{J.30}$$

$$\left(\frac{\sigma_{c,0,d}}{f_{c,0,d}}\right)^2 + k_m \frac{\sigma_{m,y,d}}{f_{m,y,d}} + \frac{\sigma_{m,z,d}}{f_{m,z,d}} \leq 1 \tag{J.31}$$

These combinations are retrieved from the Karamba analysis, as only stresses in the same parts of beams must be checked in a combination.

J.3.2. Checking of CLT

In general, the same checks should be performed for CLT as for glulam beam. However, as the grains are not oriented in one direction, the calculation of the strength is more complicated. The deviating values are given, with references to the equations from the previous section.

In equation J.19, $\sigma_{t,0,d}$ is defined as follows:

$$\sigma_{t,0,d} = \frac{N_{0,d}}{A_{0,net}} \tag{J.32}$$

The same applies for tension in the other direction, and compression in both directions.

Rolling shear is an often occurring failure mode for CLT panels due to the cross layers (Li et al., 2014). The compression transverse to the grain direction must be checked, as local failure can occur due to rolling shear.

$$\frac{N_{90,d}}{k_{c,90} \cdot A_{ef}} \leq f_{c,90,d} \tag{J.33}$$

where $k_{c,90}$ is 1.90 in case of a punch that is not close to a support, and $k_{c,90}$ is 1.40 when it is close to a support. For A_{ef}, the width can be taken 30 mm larger on both sides.

To verify the bending moments, the following equation must be satisfied:

$$\sigma_{m,d} \leq f_{m,d} \tag{J.34}$$

$$\sigma_{m,d} = \frac{M_{0,d}}{W_{0,net}} \tag{J.35}$$

This must also be performed for the bending moment in the other direction.

Finally, to verify the shear in the panel:

$$\tau_{V,R,d} = \frac{V_{0,d} \cdot S_{0,R,net}}{I_{0,net} \cdot b} \tag{J.36}$$

Again, the same must be performed for the other direction.

Most stresses can be calculated by the output of the Karamba model. However, as one link between the main girders and deck panel simulate multiple connectors, concentrated shear stresses occur. The result of this mesh density is described in Chapter 9. Consequently, the development of the shear stresses can not be predicted accurately enough by the Karamba software.

J.3.3. Results

As a consequence of the connection design in the previous section, the unity check in SLS ended up at 0.97. This is close to 1.0 and not desired, thus the height of the main girder is adjusted to 0.70 meter. A unity check in SLS of 0.92 is obtained.

The members are checked according to the equations as described above. This is done for a bridge of 35 meter length, with one span. A width of 15 meter is applied. The following cross sections are applied to the members:

Table J.24: Cross sections

Dimension	Size (m)
Height MG	0.70
Width MG	0.40
Height CG	0.40
Width CG	0.22
Height DP	0.33
Height strut	0.50
Width strut	0.26

GL28h is applied for the beam elements, where C24 is applied for the CLT deck. A script is written in Karamba to obtain the unity checks for each element type. The results are presented in Table J.25 until J.28.

Main girders

Table J.25: Unity checks for main girders

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	12.10	17.8	0.68
Tension in transverse direction	0.06	0.38	0.17
Compression in longitudinal direction	1.34	22.4	0.06
Compression in transverse direction	0.08	2.0	0.04
Bending	2.24	22.4	0.10
Shear	0.06	2.8	0.02
Combined bending and tension	-	-	0.78
Combined bending and compression	-	-	0.10

Cross girders

Table J.26: Unity checks for cross girders

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	4.27	17.8	0.24
Tension in transverse direction	0.14	0.38	0.38
Compression in longitudinal direction	0.67	22.4	0.03
Compression in transverse direction	0.16	2.0	0.08
Bending	1.34	22.4	0.06
Shear	0.45	2.8	0.16
Combined bending and tension	-	-	0.30
Combined bending and compression	-	-	0.06

Struts

Table J.27: Unity checks for struts

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	-	-	-
Tension in transverse direction	0.02	0.38	0.04
Compression in longitudinal direction	9.86	22.4	0.44
Compression in transverse direction	0.02	2.0	0.01
Shear	0.11	2.8	0.04

Deck

Table J.28: Unity checks for deck

	Occurring stress [N/mm^2]	Allowed stress [N/mm^2]	Unity check
Tension in longitudinal direction	4.26	11.2	0.38
Tension in transverse direction	2.83	11.2	0.25
Compression in longitudinal direction	9.07	16.8	0.54
Compression in transverse direction	3.02	16.8	0.18
Bending in longitudinal direction	8.45	19.2	0.44
Bending in transverse direction	4.22	19.2	0.22
Rolling shear	0.66	2	0.33

Overview

Table J.29: Summary of unity checks

UC	Main girders	Cross girders	Struts	CLT deck
Tension in longitudinal direction	0.68	0.24	-	0.38
Tension in transverse direction	0.17	0.38	0.04	0.25
Compression in longitudinal direction	0.06	0.03	0.44	0.54
Compression in transverse direction	0.04	0.08	0.01	0.18
Bending (longitudinal direction for deck)	0.10	0.06	0.04	0.44
Bending (transverse direction for deck)	-	-	-	0.22
Shear in longitudinal direction	0.02	0.16	0.04	-
Rolling shear	-	-	-	0.33
Combined bending and tension	0.78	0.30	-	-
Combined bending and compression	0.10	0.06	-	-

K

Technical drawings

Steel	Concrete
Asphalt	CLT
Soil	Glulam

Project
Thesis modular timber bridges

Drawing title
Overview drawing, section A

Drawn by
Vera Sluijter

Date
22/10/2021

Unit	Scale	Sheet no.
mm	1:100	1

Steel		Concrete	
Asphalt		CLT	
Soil		Glulam	

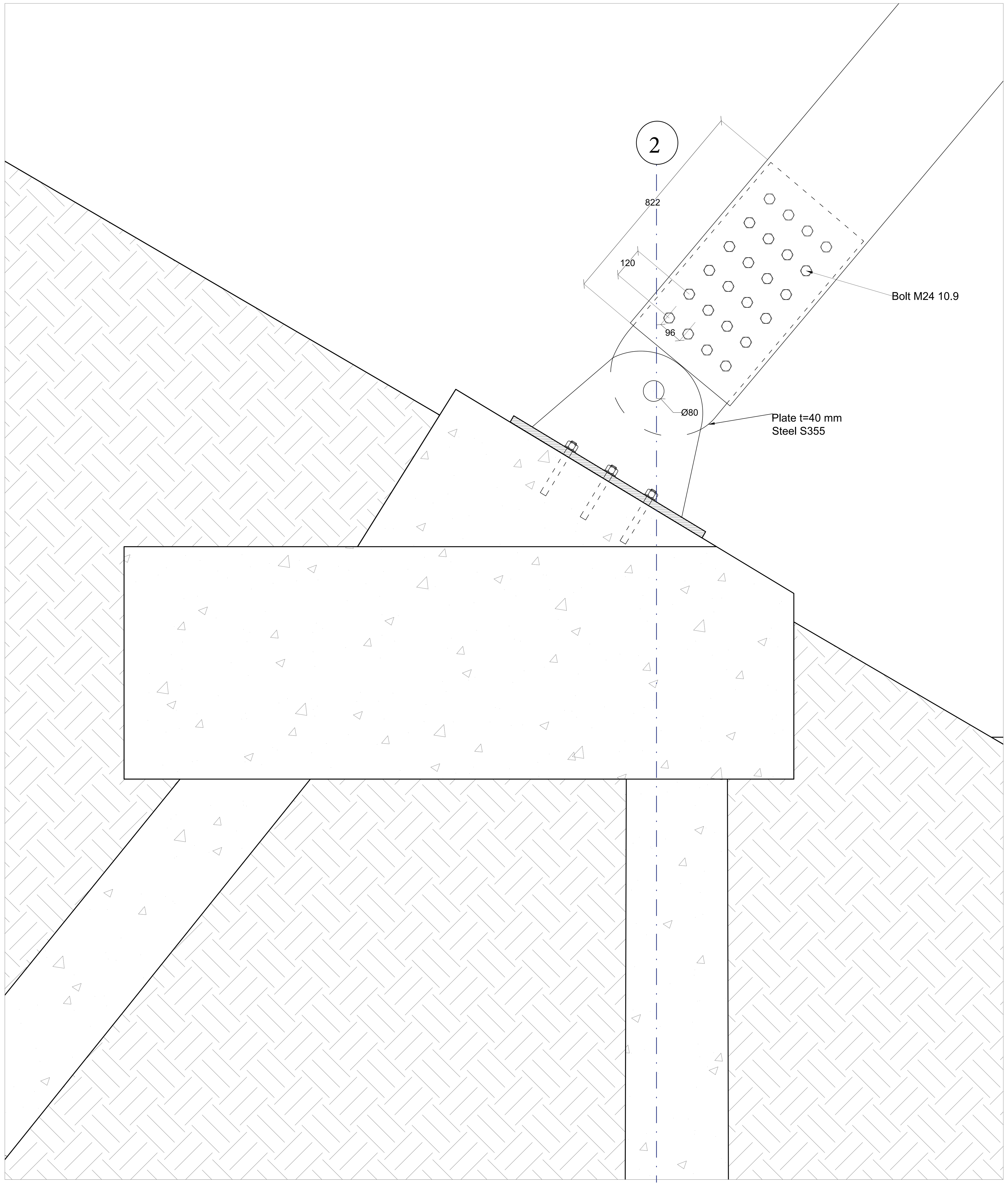
Project
Thesis modular timber bridges

Drawing title
Top view

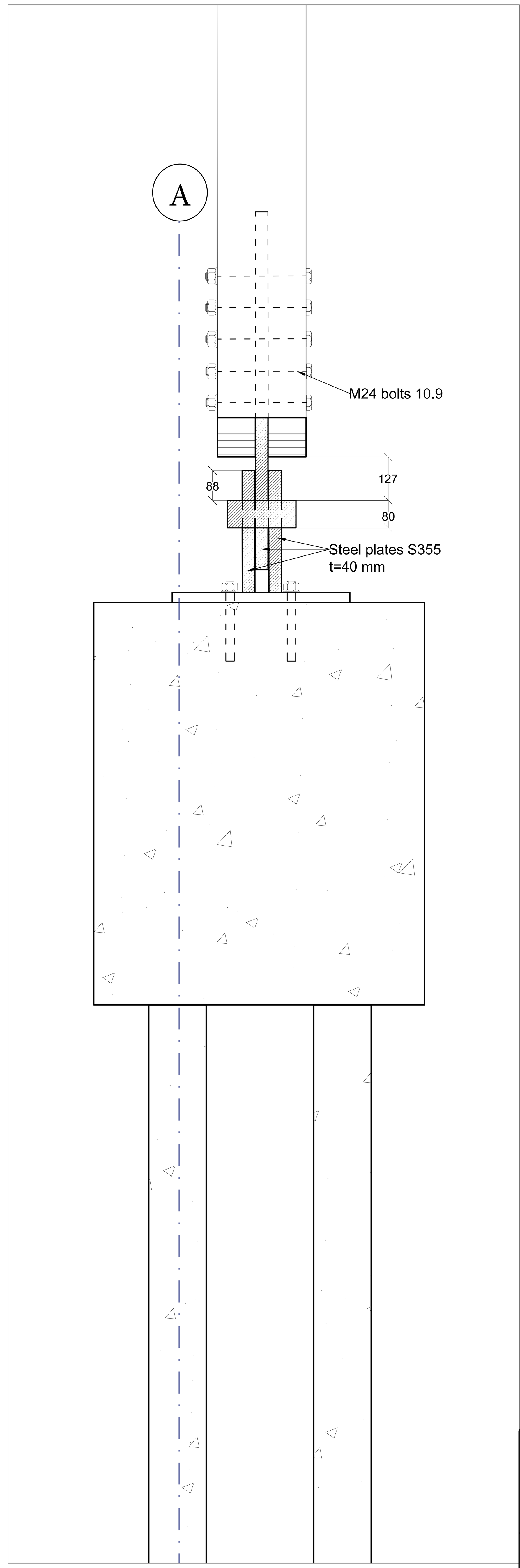
Drawn by
Vera Sluijter

Date
22/10/2021

Unit	Scale	Sheet no.
mm	1:100	2



Section A



Section 2

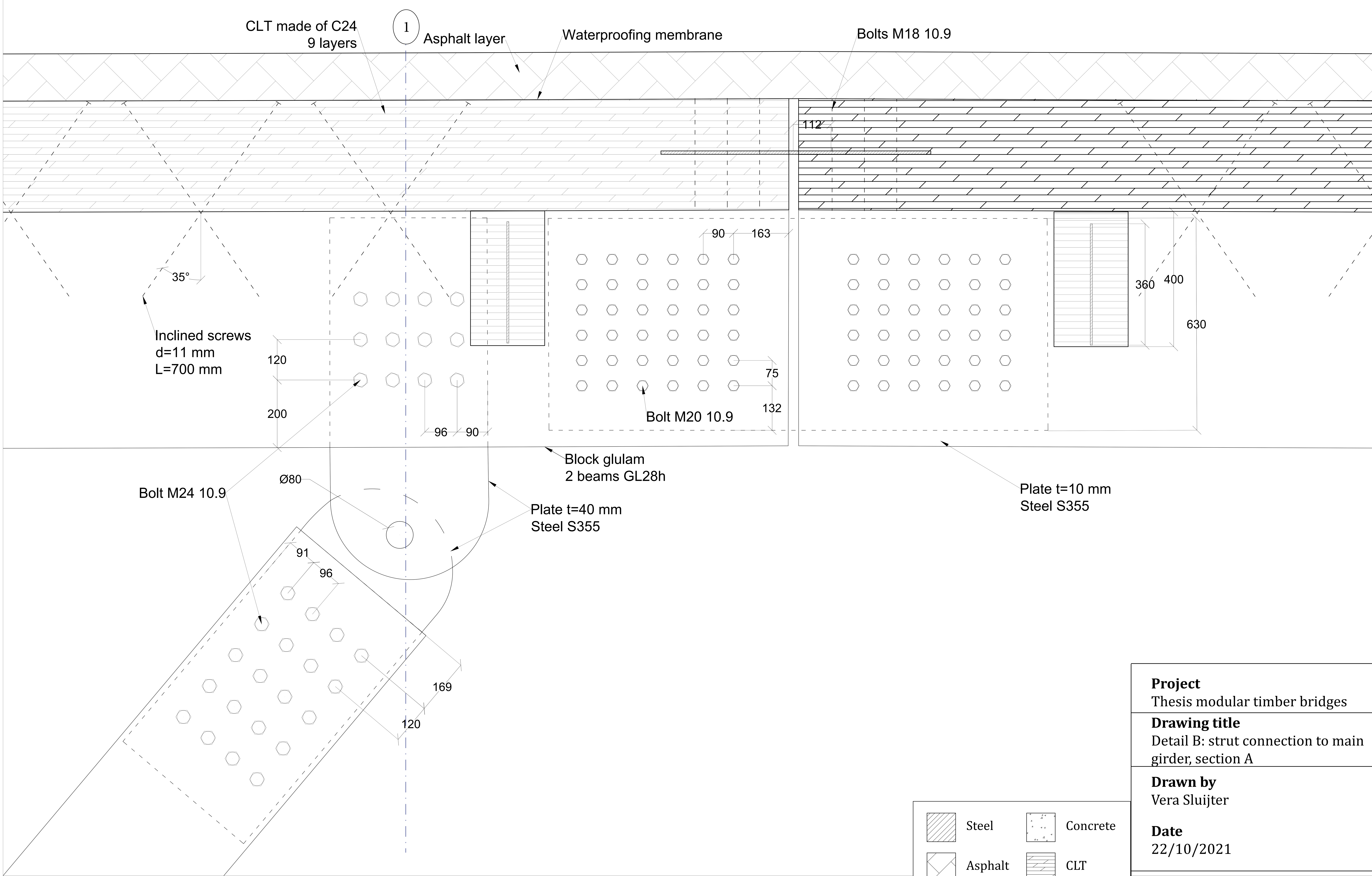
Steel	Concrete
Asphalt	CLT
Soil	Glulam

Project
Thesis modular timber bridges

Drawing title
Detail A: strut support connection section 2 and A

Drawn by
Vera Sluijter
Date
22/10/2021

Unit	Scale	Sheet no.
mm	1:10	3



Steel	Concrete
Asphalt	CLT
Soil	Glulam

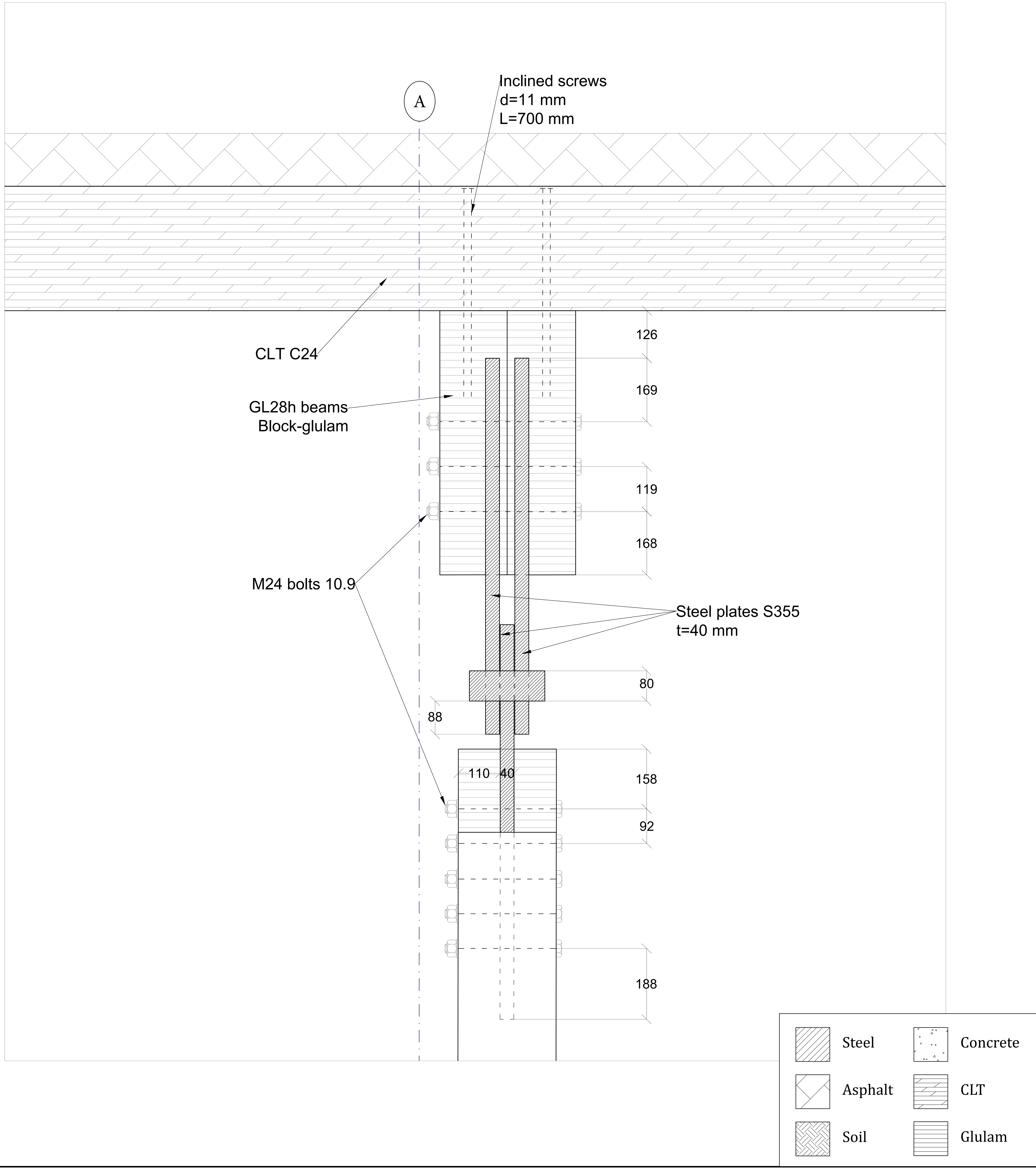
Project
Thesis modular timber bridges

Drawing title
Detail B: strut connection to main girder, section A

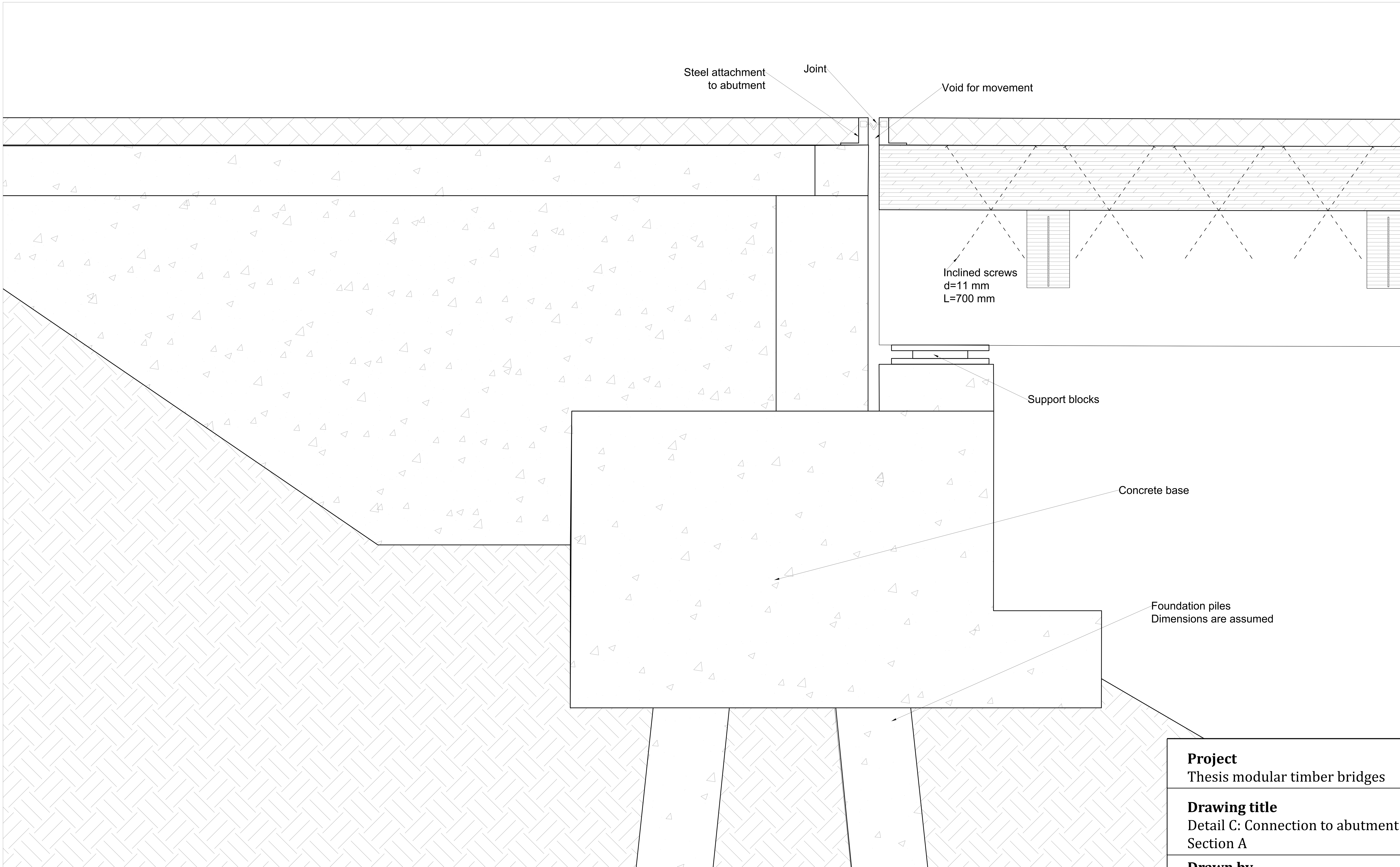
Drawn by
Vera Sluijter

Date
22/10/2021

Unit	Scale	Sheet no.
mm	1:10	4



Project Thesis modular timber bridges		
Drawing title Detail B: strut connection to main girder, section 1		
Drawn by Vera Sluijter		
Date 22/10/2021		
Unit mm	**Scale** 1:10	**Sheet no.** 5



Steel
Concrete
Asphalt
CLT
Soil
Glulam

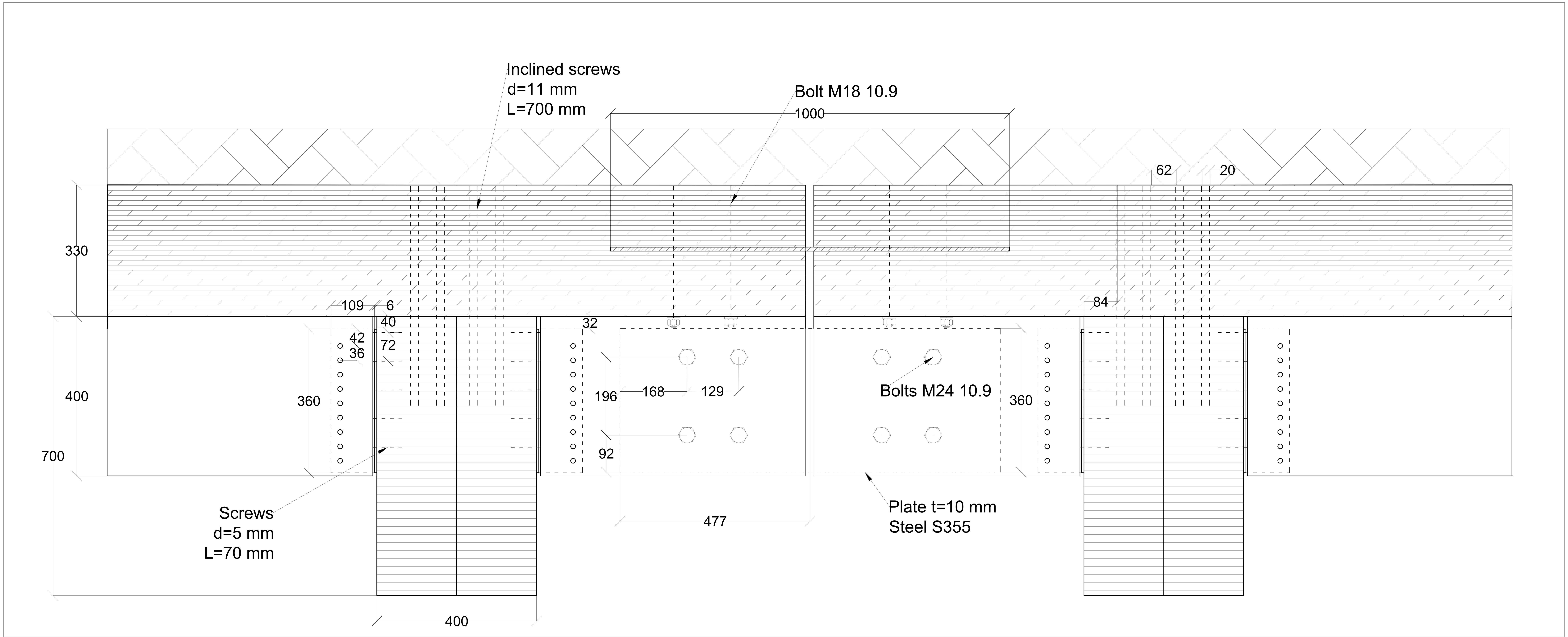
Project
Thesis modular timber bridges

Drawing title
Detail C: Connection to abutment
Section A

Drawn by
Vera Sluijter

Date
22/10/2021

Unit	Scale	Sheet no.
mm	1:20	6



Steel

Concrete

Asphalt

CLT

Soil

Glulam

Project
Thesis modular timber bridges

Drawing title
Detail D: Connection cross girders
Section 2

Drawn by
Vera Sluijter

Date
22/10/2021

Unit	Scale	Sheet no.
mm	1:10	7