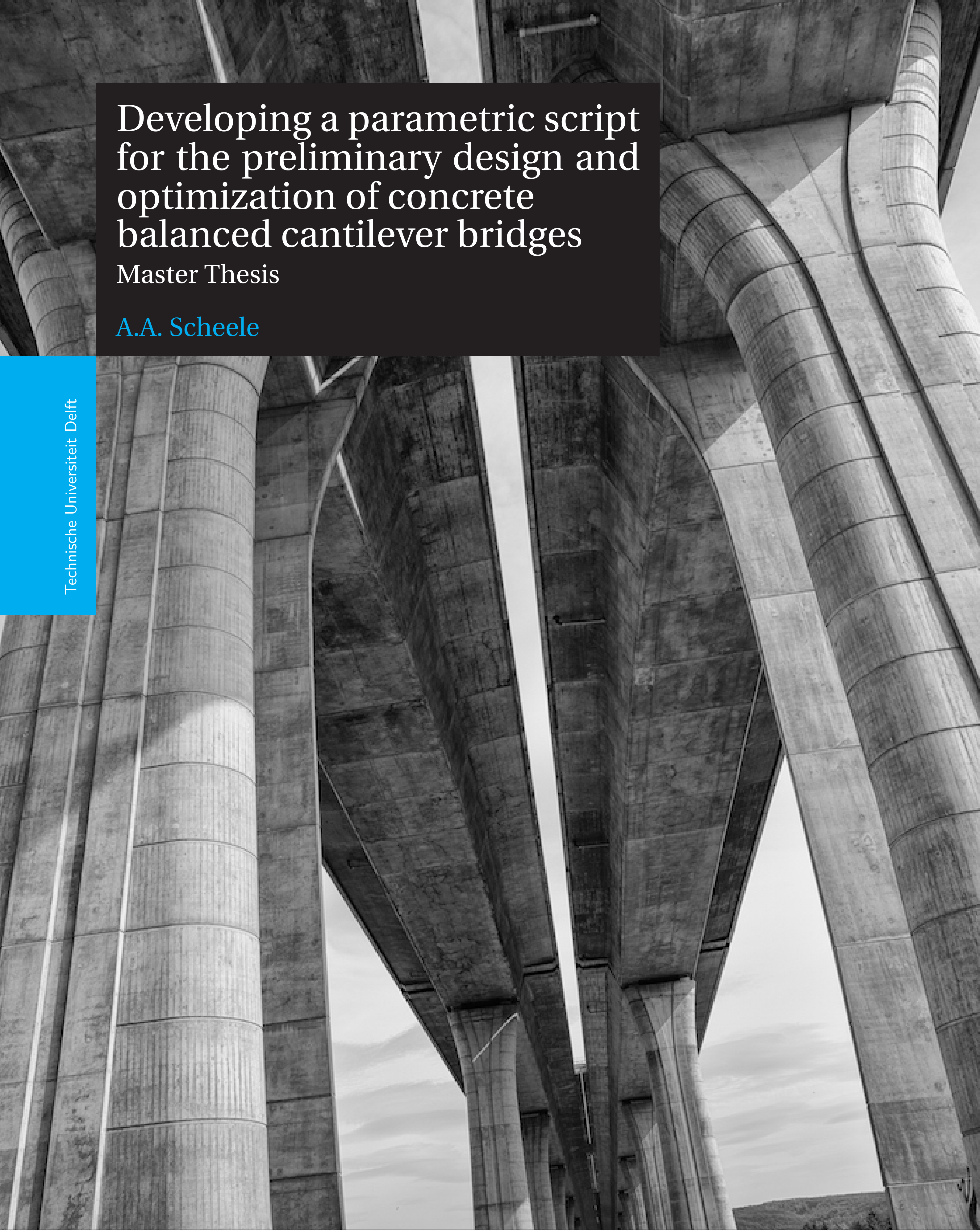

Developing a parametric script for the preliminary design and optimization of concrete balanced cantilever bridges

Master Thesis

A.A. Scheele

Technische Universiteit Delft

TU Delft Delft University of Technology

Challenge the future

Developing a parametric script for the preliminary design and optimization of concrete balanced cantilever bridges

Master Thesis

by

A.A. Scheele

in partial fulfillment of the requirements for the degree of

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Supervisor:	Ir. J. van der Sanden,	Movares
Thesis committee:	Prof. ir. R. Nijsse,	TU Delft
	Dr. H. M. Jonkers,	TU Delft
	Ir. L. P. L. van der Linden,	TU Delft

This thesis is confidential and cannot be made public until February 19, 2020.

TU Delft Delft University of Technology

Preface

In this report, the results of my master thesis project on parametric concrete bridge design will be described. Included in this thesis are some background information about parametric design, bridge types, materials, environmental impacts and software packages. Furthermore the research questions and the overall approach will be discussed combined with the expected result, which is a parametric concrete bridge design model.

In partial fulfilment of the requirements for a masters degree in Building Engineering at the Delft University of Technology, I am doing an internship at Movares in Utrecht. Movares is a engineering company specialized in infrastructure, with around 1000 employees. At the moment there is an ongoing project called ReDesign, the aim of which is to automate and optimize the design process of all engineering fields including infrastructure. Some of the design processes, e.g. for underpasses, are already finished and in use. For concrete bridges this project has just started, so there still is a demand for additional research and investigation regarding this topic. This is where my research will come into play.

As soon as the design process is automated and optimized, it will take less time to design and compare multiple bridges. So it will be possible to generate a higher quality of design in a smaller amount of time. This will be extremely valuable for the clients as well as for Movares itself. When this whole evolution is finished, the engineers will have more time to think about really important or innovative solutions.

In the first place i want to thank my supervisors at Movares, Wilco Sponselee and Janine van der Sanden, who provided the topic for this master thesis, but also many thanks to all other colleagues who guided and helped me this year. It was a very pleasant time and I liked the friendly atmosphere at Movares. In fact, after this project I will start my career at Movares and I am thrilled to participate in the real world of engineering.

I also want to thank the rest of my graduation committee, Rob Nijsse, Henk Jonkers and Lennert van der Linden, for asking critical questions, providing feedback and guiding me through the sometimes difficult graduation process.

Finally, I want to thank my family, friends and especially my girlfriend for supporting me and giving me the motivation and energy to continue with this project!

For now, I hope you enjoy reading this thesis.

A.A. Scheele
Utrecht, February 2019

Cover image: Radotín Bridge in Prague, photo by Jiri Vatka Photography.

SUMMARY

The main goal of this Master Thesis is to develop a script which makes it possible to speed-up and improve the process for the preliminary design of concrete cantilever bridges and to be able to present multiple design alternatives during a meeting with a client. It is obvious that such a program must meet the structural requirements stated in the Eurocode and the additional codes. Besides, this program will provide the possibility to compare and optimize the alternatives on use of materials, costs and environmental impact. In this way it will be an interactive tool to fulfil the client's needs by giving a rough impression of the final design early in the design process. However, a bridge design process using a parametric design program might also have detrimental side effects. Therefore this thesis is based on the following research question:

What can be achieved by using a parametric model in the preliminary design phase, with the possibility to optimize and compare concrete bridges on material usage, costs and environmental impact?

To achieve an efficient parametric model, in the Chapter 2 "Background Information" first all information about parametric design, road design, bridge design for main and approach bridges, substructure design, foundation design, material properties, load combinations and optimization of the preliminary design is presented. Furthermore the load models and design verifications are investigated by studying the Eurocode and additional design codes like the ROK (Richtlijnen Ontwerp Kunstwerken). This is the result of the performed literature study. The information from this chapter is used as basis for the parametric bridge designs in Chapter 3 "Bridge Designs", Chapter 4 "Parametric Model" and the optimization process in Chapter 5 "Optimization".

In Chapter 3 "Bridge Designs", the theoretical bridge design with all elements and components from the previous chapter is further elaborated and translated into parametric designs. Also the optimization parameters and constraints of the bridge are determined. Furthermore the analysis, checking and verification phase are explained. The bridge design as described in this chapter, is implemented in the parametric model.

In Chapter 4 "Parametric Model", the set-up and the working of the parametric model itself is explained by using a flowchart. Subsequently the script is described in more detail with help of illustrations and visualizations. Also the bridge parameters are visualized in this chapter.

Subsequently the optimization process is explained in Chapter 5 "Optimization", supported by illustrations of the workflow. Furthermore in this chapter two existing bridges, the Stichtsebrug II and the Dintelhaven (east), are redesigned by the model. And by optimizing these bridges, the effectiveness of the model is judged. The results of this redesigning process is described and visualized in Chapter 6 "Results".

The results of the redesigning process are presented, combined with graphs, tables and visualizations in Chapter 6 "Results". The optimized bridges from the parametric model are compared with the existing bridges and the results are discussed in this chapter. Also design graphs are presented , with these graphs it is possible to estimate the approximate construction height for different span lengths.

Finally in Chapter 7 "Conclusion", the main research question is answered. Furthermore, the reliability of the model and it's limitations are discussed. Subsequently there are given possible improvements and extensions for the model. Based on the previous chapters in this master thesis and the results of the different optimizations, the following can be concluded:

By using a parametric model for the preliminary design of concrete bridges, the design time is decreased significantly, the design is more flexible and the chance for (human) errors is minimized. Furthermore, the model produces an instant visualisation and overview of the costs and environmental impact. Due to a smart evolutionary solver, the model is also able to optimize this results in a fast way.

However, the parametric model has also it's limitations due to the high investment of developing the model against the expected low frequent use. Also the model is only reliable for long and small cantilever bridges in the preliminary design phase, due to the select analysis which is performed by the model. Furthermore, since the optimization of the designs is only based on material usage, the user should be careful with the interpretation of the results. Other costs aspects are not taken into account.

Since there now is a solid and working parametric model, expanding and improving this model is easy. So, the limitations as described above can be solved and the reliability of the parametric model can be increased. This all together makes the parametric script for concrete balanced cantilever bridges a useful tool, with high potential in the preliminary design phase.

Contents

1

Introduction

For decades, the Dutch ports and roads are used for importing and exporting cargo in and out of Europe. This transportation benefits from a well designed sustainable road network. Therefore, the Netherlands is a country with a high density road network which is still growing every year, see Figure 1.1. Also the use of the roads is increasing each year, which makes the availability of the roads very important, so the construction method is adapted in order to minimize the hindrance. In 2017 there was a total highway length of 5357 kilometres; these roads are supervised and maintained by Rijkswaterstaat. In total there is about 139.000 km of roads, 3.000 km railways, 35.000 km bicycle lanes and 6.000 km of waterways[1]. Especially this last category creates a challenge for the roads to cross them. This crossing of rivers, roads, railways or other obstacles is the basis for this project in which the author will investigate the design of concrete bridges by making use of a parametric design model. For an overview of the dense motorway and waterway network in the Netherlands, see Figure 1.2.

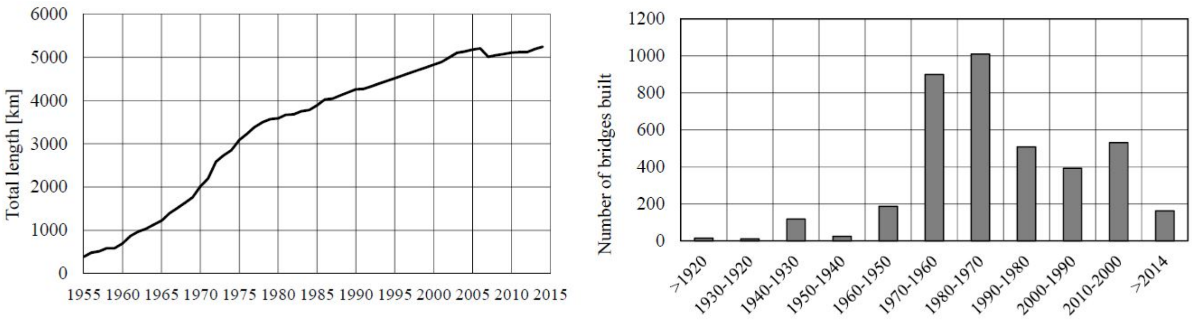


Figure 1.1: Left: Development of the total length of the Dutch main road network. Right: Overview of the number of bridges built per decade since 1920[2].

In Figure 1.1 on the right, there is an overview of the number of bridges that are constructed in the last decades. As can be seen, many of those bridges were build in the last 50 years. Since in most of the cases this is also the design service lifetime and since the amount of heavy traffic is increased over the last decades, many bridges have to be replaced or strengthened. This is also the case for the Merwede-bridge (1961), the Hagesteinse-bridge (1981) and the Keizersveer-bridge (1968). These steel bridges will preferably be replaced by concrete bridges, to reduce maintenance costs but also to decrease roadworks. Furthermore, replacing the old steel bridges by concrete bridges gives a noise reduction[3]. Since all bridges have different boundary conditions and requirements, this requires a smart, flexible and fast way of designing.

Further in this Chapter the topic of this Master Thesis will be explained by stating the problem definition and the main question which should be answered to solve this problem. In order to answer this main question, first a set of sub questions need to be answered. Thereafter the approach strategy for this project will be described and the scope of the project will be put into writing. In the end of this chapter there will be an overview of the expected result.

1.1. PROBLEM DEFINITION

Nowadays, designing bridges (and other structures) is a quite time-consuming process. Many bridges have to be designed from scratch and adjustments of the designs costs money and time. Furthermore, the flexibility for the client is severely limited as soon as the design process is started. This problem came forward during a meeting at Movares.

Figure 1.2: Overview of the dense infrastructure, with in red the motorways and in blue the waterways [1].

A possible solution for this problem would be an innovative approach to bridge design, using an interactive parametric design program. Using this design program, the design process could be more efficient. Besides, modifications in the design could be made in seconds. Presumably, the design process will be more accurate as well; once the program is correctly developed, the chance for errors will be greatly reduced. And misunderstandings between designer and client are less likely. Other advantages include a more cost-effective design process and increased flexibility towards the client.

However, a bridge design process using a parametric design program might also have detrimental side effects. When the boundary conditions become too complex, so a special design is required, the parametric design program will not be able to perform the task. Since it is not possible within this limited time span for this master thesis project to develop a script including all possibilities, the goal is to develop a couple of bridges for standard situations. This limitation in complexity and diversity of bridge types should be taken into account when using the program.

1.2. RESEARCH QUESTION

What can be achieved by using a parametric model in the preliminary design phase, with the possibility to optimize and compare concrete bridges on material usage, costs and environmental impact?

1.3. OBJECTIVE

The main goal of this Master Thesis is to develop a script which makes it possible to speed-up and improve the process for the preliminary design of concrete bridges and to be able to present multiple design alternatives during a meeting to a client. It is obvious that such a program must meet the structural requirements stated in the Eurocode and the additional codes. Besides, this program will provide the possibility to compare and optimize the alternatives on use of materials, costs and environmental impact. In this way it will be an interactive tool to fulfil the client's needs by giving a rough impression of the final design early in the design process.

1.4. METHODOLOGY

In this section the approach strategy for answering the main question will be explained, by giving a schematic overview of the thesis set-up, see Figure 1.3. Furthermore an overview of the subquestions per chapter with a brief explanation will be given.

Chapter 2 – Background Information

In this chapter all information about parametric design, road design, bridge design for main and approach bridges, substructure design, foundation design, material properties, load combinations and optimization of the preliminary design is presented. Furthermore the load models and design verifications will be investigated by studying the Eurocode and additional documents like the ROK (Richtlijnen Ontwerp Kunstwerken). This is the result of the performed literature study. The information from this chapter will be used as basis for the parametric bridge designs in Chapter 3 and 4 and the optimization process in Chapter 5. The following subquestions will be answered in this chapter:

2.1 What is the current state of affairs in parametric design?
2.2 What are the important parameters for road design in the Netherlands?
2.3 Which bridge types are best suited to design using a parametric design program?
2.4 How is the substructure and foundation of concrete bridges designed?
2.5 What are the mechanical properties of the materials? And what are the costs and environmental impact?
2.6 What load models and combinations must be considered?
2.7 What software programs are suited best for this topic?
2.8 What is evolutionary optimizing?

Chapter 3 – Bridge Design

In this chapter the bridge design with all elements and components will be further elaborated and translated into parametric designs. Also the optimization parameters and constraints of the bridge will be determined, furthermore the analysis, checking and verification phase will be explained. The bridge design described in this chapter, is implemented in Chapter 4. Subquestions for this chapter are:

3.1 How to translate the structural designs into parametric designs?
3.2 What are the important parameters for the design of concrete bridges?
3.3 What analyses should be performed by the model?
3.4 What are the important requirements for the preliminary bridge design?

Chapter 4 – Parametric Model

In this chapter the working of the parametric model itself is explained by using a flowchart. Subsequently the script is described in more detail with help of illustrations and visualizations. This chapter is based on the following subquestions:

4.1 How to compose the parametric model?
4.2 What are the important steps in the process of designing concrete bridges parametrically?
4.3 How to create clear visualizations for the user?

Chapter 5 – Optimization

The optimization process for this specific project will be explained, supported by illustrations of the workflow. Furthermore two existing bridges will be redesigned by the model. And the effectiveness of the model will be judged, by optimizing and comparing these bridges. The results of this redesigning process will be made visual in Chapter 6. The subquestions for this chapter are:

5.1 How can the bridges be optimized to reduce costs?
5.2 How to redesign and optimize existing bridges by using the parametric model?

Chapter 6 – Results
Here the results from the redesigning process are presented, combined with graphs, tables and visualizations. The optimized bridges from the model will be compared with the existing bridges and the results will be discussed in this chapter. The following subquestions will be answered here:

6.1 What is the performance of the existing bridges?
6.2 How to present the redesigned bridges?

Chapter 7 – Discussion
In this chapter, the main research question will be answered. Furthermore, the reliability of the model and it's limitations will be discussed. Subsequently, possible improvements and extensions for the model will be given. The subquestions for this chapter are not ordered per section, but subquestions could be: How reliable is the model? What are the limitations? How can the model be improved? Can the model be easily extended?

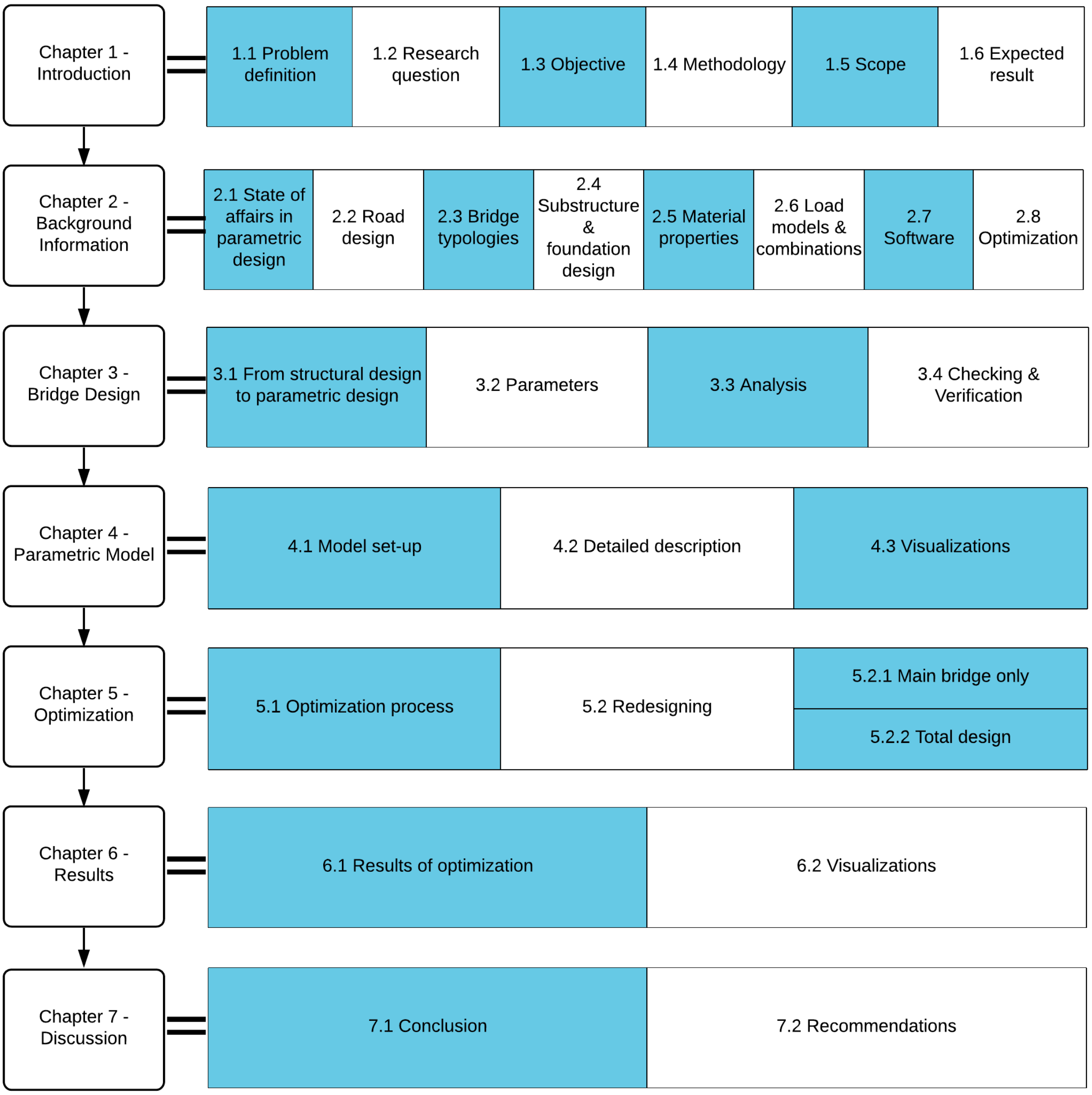


Figure 1.3: Schematic overview of the thesis set-up and chapter layout.

1.5. SCOPE

In the scope, the boundaries of this master thesis will be defined. The scope consists of four parts: Design, Analysis, Optimization and Visualization. These four parts will be described in the following sections supported with a schematic overview, see Figure 1.4.

Design

In this project, the focus will be on the preliminary design of concrete bridges in the overall design process of the road alignment. This holds that there are no detailed calculations involved and the main goal is to give a good estimation of how the design would look. The calculations will be globally in 2D for the main bridge, but also 3D calculations will be made for the load distribution in the approach bridges. There will be no calculations about detailing of the reinforcement or long-term effects as fatigue and creep.

So the bridges will be designed parametrically, globally analysed with FEM software (Finite Element Method) and optimized on material usage, costs and environmental impact.

The focus will be on concrete bridges for main roads (highways) with a variable amount of lanes. Exiting lanes or minor roads will not be part of this project. Since the length of the alignment is quite small, the decision is made to not include the horizontal alignment, this will be taken as a straight line. The vertical alignment is included in the model and is limited by minimal radii, the free height of the passage and start and end points (A and B). Between the start and end points there is the total design, consisting of: embankments, approach bridges, the main bridge and the substructure. With the substructure, the piers, columns, foundation slabs and piles are meant, these parts are designed with help of rules of thumb. For this part of the model, some estimations will be made for the load bearing capacity of the soil, the actual soil stiffness will not be part of the project. Also in the analysis stage, the supports will be assumed as fully rigid instead of elastic supported. Furthermore, the detailed parts like expansion joints and bearings will not be designed in the model. They may be taken into account in a quantitative way for the estimation of the total project costs, more joints and bearings give higher costs.

The model will be build around a fictional case study, but it should be applicable to any location. The chosen case consists of a river crossing with on both side wetlands, because that is the most challenging situation. Obviously the lengths of the spans and the total bridge length can differ in this situation. All these input values should be free and user-defined.

Because the model will be mainly developed for projects in the Netherlands, the boundaries for the model are also based on Dutch limitations and regulations. For each location, the length of the alignment, the span and the height of the bridge is different. The maximum span length for the model is set to 200 meters, since the longest concrete bridge in the Netherlands has a span of 192 meters[1]. And the minimum length is set to 15 meters, this is mainly for the approach bridges.

Analysis

The analysis of the bridge consists of multiple checks and verifications. The focus will be on the main span, but the approach bridges, substructure and foundation have to satisfy on some level as well. The structural analysis will be done with Karamba, a plug-in for the algorithm editor add-on, Grasshopper. Later in this report, the used plug-ins and programs are explained.

Since Karamba only gives the force and stress distribution in the geometry, the forces have to be translated into prestressing loads. This will only be done for the main span and the approach bridges, by adding calculations/formulas to the Grasshopper script. The cross-sections should be checked on bending moment capacity, buckling capacity, but also deflection and fitness checks will be performed. The shear capacity of the bridge will not be analysed, since this has little effect on the main bridge design.

To verify the results from the Karamba analysis, hand calculations will be performed (in Grasshopper). The structure has to satisfy for the Ultimate Limit State (ULS) and the Serviceability Limit State (SLS), this will be proven by determining the Unity Checks (UC). For the cast-in-situ balanced cantilever bridge, as well the construction phase as the end phase will be analysed, since there is a changing moment distribution during the construction and the end phase of the bridge. Also the approach bridges will be analysed in multiple stages. The rest of the elements will only be analysed for the end phase.

Detailed reinforcement modelling or calculations will not be performed.

The substructure will be designed and verified on load bearing capacity and buckling capacity, so the capacity has to be higher than the applied load from the superstructure and the traffic loads, $N_{Rd} > N_{Ed}$. The

[1] www.wegenwiki.nl/Dintelhavenbrug

amount of reinforcement will be estimated for all elements, this is also implemented for the resulting material amounts and costs. Stability, dynamic and fatigue calculations will not be taken into account.

Optimization
The optimization of the bridges will be performed on material use, costs and environmental impact expressed in shadow costs. The costs will be determined per cubic meters or tons material. So the construction of the bridge and transport of the material will not be taken into account. These two costs and shadow costs are quite specific per location and would make the costs calculation too complex. The bridges can be compared on the results of the optimization.

Visualization
With the visualization of the model, the most important goal is to create a clear 3-dimensional digital model of the total design combined with cross-sections of the bridge.
Since the main objective is to optimize the bridges on costs, the aesthetics are inferior compared to optimized (economic) designs in this project.
Also important, is the clarity of the User Interface (UI). The user must have a clear overview of the steps he has to take and the input that he has to provide.
Finally, it is possible to export the 3-D model to other software programs like Revit and SCIA, for the following design stages.

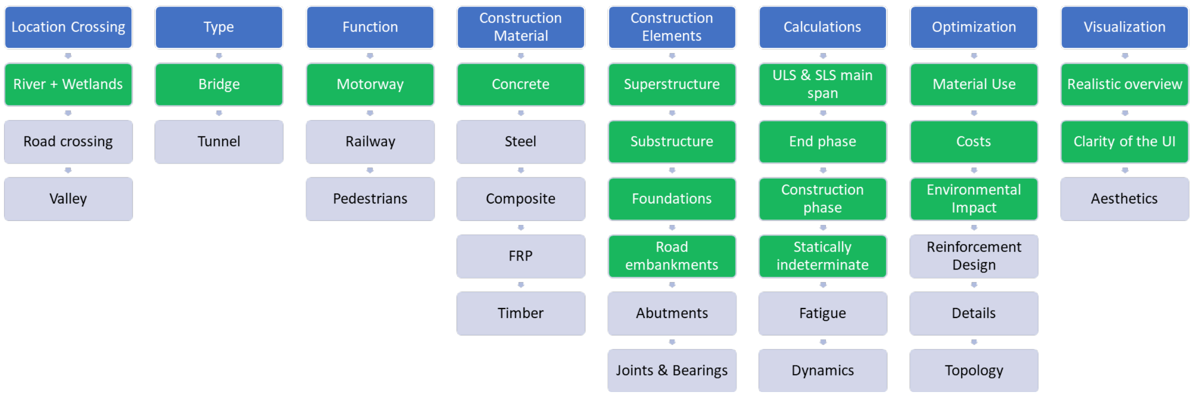


Figure 1.4: Scope of Master Thesis Project

Not in the Scope
Not included in the scope are: Other road types, tunnels, railway or pedestrian bridges, bridges made out of other materials, detailing and soil properties, shear calculations, dynamic calculations and aesthetics. Also fatigue, shrinkage and detailed creep calculations are not taken into account. These choices are made to prevent the model from becoming too complicated.

Program set-up

To accomplish the main goal of this master thesis project, a set-up for the parametric model is made. In this model the boundaries and other input provided by the user generates a geometry which will be analysed. This geometry is split into the main span, approach bridges and substructure. At the same time, the model will calculate the material amounts, costs and shadow costs, which are used for the optimization stage. After the optimization stage, the best design is chosen as the preliminary design for the specific location. This design forms the basic for the later design phases. Figure 1.5 shows a coarse schematic overview of the above mentioned steps in the program set-up. Note that this process is an iterative process, so some steps will be done several times. An enlarged and more detailed version of this scheme an be found in Appendix A.

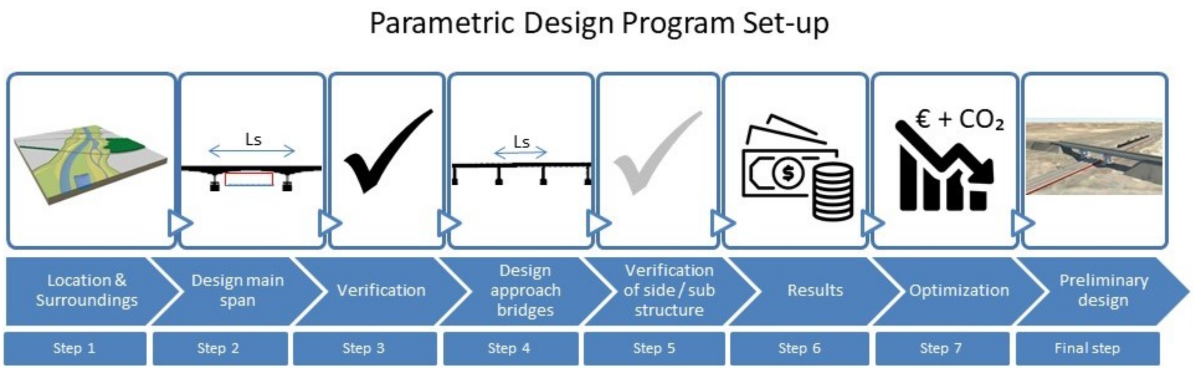


Figure 1.5: Parametric Design Program Setup.

2

Background Information

This chapter will give an overview of the information which is a result of the literature study. An overview will be given of the state of affairs in parametric design to give an impression of the possibilities and also to show the difference with the way of designing decades ago. Subsequently, more project specific information will be given, about road design, bridge design and materials. Finally, the software packages and the optimization theory are described.

2.1 What is the current state of affairs in parametric design?
2.2 What are the important parameters for road design in the Netherlands?
2.3 Which bridge types are best suited to design using a parametric design program?
2.4 How is the substructure and foundation of concrete bridges designed?
2.5 What are the mechanical properties of the materials? And what are the costs and environmental impact?
2.6 What load models and combinations must be considered?
2.7 What software programs are suited best for this topic?
2.8 What is evolutionary optimizing?

2.1. STATE OF AFFAIRS IN PARAMETRIC DESIGN

The last years there is a huge increase of new programs and tools for designing and analysing structures. In this section a brief description of the possibilities will be presented. But before the state of the art in parametric design nowadays will be described, first a quick look into the history of parametric design as described by Daniel Davis in 2013. [4]

2.1.1. HISTORY OF PARAMETRIC DESIGN

One of the first designers who used an automated parametric design tool is Gaudí, although he did not use a computer. Gaudí's "hanging model" technique (Figure 2.1) which he also used to design the Sagrada Família, is in fact parametric optimizing of the design without having to calculate the exact mathematical shape of the curves by hand. Of course this model had only a few parameters namely, the length of the chains and the location of the supports. One of the limitations of the hanging model is the dependency on physical laws, in this case gravity, the same counts for Frei Otto with his soap films which he used for so called form finding. When the computer came into use those limitations disappeared and it gave the opportunity to use relationships like parallel and orthogonal. A program that used this relationships was Sketchpad developed by Ivan Sutherland in 1963. This program had seventeen "atomic constraints", which work as small operators. Each atomic constraint has input variables and a specific function which automatically provides the output. Sketchpad gave designers the freedom of not only giving the input, but also changing the underlying relationship between the input and the output. Since the computer was quite expensive in that time, there were only a few companies who could afford them, mainly companies in the automotive or the aerospace engineering. One of the first electronic drafting machines cost US$500.000 equivalent to almost 2,8 million Euro nowadays [5] Because of this, the use of computers was minimal in the structural design practice. It took around twenty years until the computer was affordable for most of the companies and there were even people who could afford a personal computer, this technological development resulted in the release of AutoCAD in 1982. Eighteen versions later, in 2009, the first parametric functions were introduced in AutoCAD2010.

Figure 2.1: Picture of one of Gaudí's hanging chain models, used for the design of compression arches. Source: Little Big Welt, www.littlebigwelt.co.uk/barcelona

Simultaneous to this development, there was the release of Pro/ENGINEER by Samuel Geisberg in 1985. With this software program it was possible to design three-dimensional geometry using various parametric equations.The vision behind the program is described by Geisberg in an interview in 1993;

> *The goal is to create a system that would be flexible enough to encourage the engineer to easily consider a variety of designs. And the cost of making design changes ought to be as close to zero as possible. In addition, the traditional CAD/CAM software of the time unrealistically restricted low-cost changes to only the very front end of the design-engineering process.*[1]

So two important things can be noted from this quote, one is that parametric design software gives the opportunity to easily provide multiple designs to a client. And the second important thing is that with parametric design software, changes can be made more easily and less costly even in a later stage of the design process. These two advantages are also of importance for this master thesis project.

[1] Geisberg quoted by J.Teresko in IndustryWeek 1993, 28

In 2000 the Parametric Technology Corporation released a new software program especially for the building industry, called Revit. On their website the developers gave an interesting description of the words *Parametric Building Modeler* or its synonym: *Revit*, see Figure 2.2.

Figure 2.2: Definition of Parametric Building Modeler or Revit.[4]

Although Revit can be seen as a fully parametric program, all the parametric equations are hidden for the users. For instance: when a building was designed and the floor height had to be changed, all the elements like walls and stairs but also building plans and sections would change accordingly. So the designers were using parametric modelling in their designs, but they did not have influence in the parametric model itself.

When the software developers noticed that there was a high demand on specific functions or applications within their software, they realized that they could avoid those questions by providing a scripting interface. In this scripting interface, designers would be able to create their own automated functions with help of some basic components, or even fully write their own script. Those scripting interfaces are now further developed into visual or "no-code" programming tools like Explicit History (later called Grasshopper), developed by David Rutten at Robert McNeel & Associates in 2007.[6]

2.1.2. PARAMETRIC DESIGN NOWADAYS

Nowadays it is possible to create your own parametric model using tools like Grasshopper or Dynamo. These two visual scripting tools are quite easy to understand, even if you do not have any programming experience. A more detailed description of Grasshopper will be given later in the software Section 2.7. Although parametric design is used most of the time for architectural purposes, the use of parametric design software in structural design is also rapidly increasing. An example of the application of parametric design is the Heydar Aliyev Center in Baku, Azerbaijan by Zaha Hadid, see Figure 2.3. In this project parametric design is used to create fluid shapes and to integrate the structural elements in the outer shell of the building, to create a column free space. This type of structures would be quite difficult to design without the current software packages for design and analysis of structures. Although there are some exceptions, Gaudí for instance with his Sagrada Família. Note that this project still is not finished due to the complexity of the design. With nowadays programs, it is also possible to investigate the constructibility and to optimize the amount of unique elements or use robotics for the manufacturing process.

Figure 2.3: Heydar Aliyev Center in Baku, Azerbaijan by Zaha Hadid (2013) Source: www.archdaily.com

Furthermore it is possible to use external data from Geographic Information System Maps (GIS-Maps) and data as soil parameters as input for the parametric models. For the analysis of structures/buildings there are lots of tools available, for instant the use of Computational Fluid Dynamics (CFD) can be used to analyse the airflow in a building to design the ventilation system. Or for urban planning and public transport infrastructure, it is possible to use People Flow Technology to design the roads or stations. Also the visualizations become more realistic and the use of virtual reality applications is increasing.

2.2. ROAD DESIGN

To determine the correct alignment for the road from A to B including the bridge, the ROA (Richtlijn Ontwerp Autosnelwegen)[7] by Rijkswaterstaat will be used. In this document all the design rules for the motorways in the Netherlands are stated. Important parameters for this master thesis are the maximum slopes, the minimal radii for the horizontal and the vertical alignment and the transversal cross-section of the road. This transversal cross-section includes the road embankment, the lane division and additional elements like safety barriers.
These design regulations are based on safety guidelines and comfort for driving. One of the guidelines is the minimal sight distance which is required for anticipating situations.

2.2.1. ALIGNMENT

For the alignment of the road in horizontal and vertical direction the guiding factor is the sight distance, a safety parameter. To set some boundaries, in this project only continuous situations will be taken into account. This means a constant road configuration, so the number of lanes stays the same and there are no road split-ups or merging lanes. The focus will be on the main roads, with a design speed of 120 km/h, and not on exiting or parallel lanes, these have other design regulations and limitations.

Sight distance

For the sight distance there are three different types. The first type is the anticipating sight distance, this distance is determined by the speed of the car driver and his ability to distinguish the situation further ahead. If necessary, the driver must be able to undertake action. For a speed of 120 km/h the required sight must be at least 335 meters, this gives limitations for the sharpness of the road curvature. The second type is the sight distance regarding the overview of the road. This is the minimal distance for the driver to be able to follow the road and to control the transverse position of his vehicle in a comfortable way. This length has a minimum of 165 meters. The third type is the stopping sight distance, this is the required length for noticing an obstacle and to be able to perform an emergency stop. Here the length is dependent on the slope of the road, for a horizontal road the required length is 260 meters but for a downwards slope of 5 % this length increases to almost 300 meters. The governing sight distance for the design of roads is the largest value, so 335 meters.

Horizontal alignment

The horizontal alignment exist of different types and has multiple functions. There are straight road parts, curved parts and the parts which form the transition from one curve to another.
The straight parts are limited to a maximum length of 2400 meters. Main reason for this limitation is the fact that on long straight roads the driver will focus to much on the horizon and is less sensitive for the situations close by. To minimize the straight parts, instead slightly curved parts with a radius of 40000 meters are applied. Because in this project the road parts are quite short, the decision is made just to use straight parts.
For the curved parts, the sight distance is governing but it is also dependent on the transversal slope. When a standard transversal slope of 2,5 % is used, the minimal radius for the road is 1500 meters. For the roads without a transversal slope, an minimum radius of 4000 meters is prescribed. This radius is taken from the middle of the road. In this project a standard slope of 2,5 % will be used, so a minimum radius of 1500 meters. The parts in between the curves should have a length of at least 240 meters for curves in opposite direction and 480 meters for curves in the same direction.

Vertical alignment

For the vertical alignment concave and convex curves are used to ensure a comfortable passage, preferably without a straight part in between. This straight part is only used when the height difference is more than 12 meters. For an overview of the vertical alignment see Figure 2.4.
The convex curve has a radius of 12400 meters, this is determined by the overview sight distance with Formula 2.1 [7].

$$R_{convex,min} = \frac{L_{sight}^2}{2(\sqrt{h_0} + \sqrt{h_h})^2} \tag{2.1}$$

In this equation all parameters are in meters. L_{sight} is the overview distance of 165 meters, h_0 is the height of the object (in this case this is the road itself, so zero) and h_h is the eye level of the driver, assumed at 1,1 meter.

The radius of the concave curve depends on the height difference to avoid kinks in the road alignment, see Table 2.1. When the height difference increases, the transition from the concave curve to the convex curve becomes to steep, therefore this concave radius is enlarged to guarantee a smooth transition.

Height Difference (Δh)	Relation $R_{\smile,min} = k * R_{\frown,min}$	Resulting $R_{\smile,min}$
≤ 3,0 m	k = 2	24800 m
4,5 m	k = 3	37200 m
6,0 m	k = 4,5	55800 m
7,5 m	k = 6	74400 m
≥ 9,0 m	k = 7	86800 m

Table 2.1: The relation between the height difference, the convex curve and the concave curve for the vertical alignment of motorways. [7]

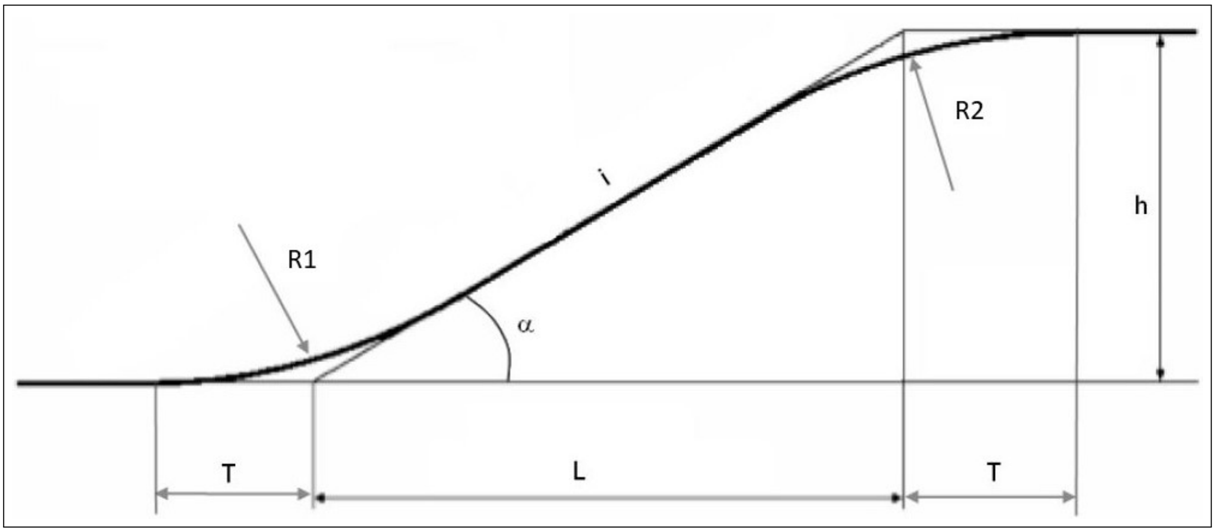


Figure 2.4: Vertical alignment with radii and slope. In this figure from the ROA[7], H1 is the concave radius and H2 is the convex radius

The maximum steepness and length of the motorway is also defined in the ROA. For a slope of 3% the length should not be larger than 1300 meters and for a slope of 5% the length is limited to 500 meters. Since the height differences in the Netherlands are quite small, this limitations will not be governing.

2.2.2. CROSS-SECTION OF THE ROAD AND EMBANKMENT

The cross-section of the road embankment consists of the highway set-up and the soil body underneath. This information will be used in the design phase, but also in the result phase for the determination of the total costs and material use.
In the road set-up the number of lanes is part of the input provided by the user, so this will be one of the free parameters. The rest of the road elements like stripes, safety barriers and emergency lanes are predefined according to the ROA. For an overview of the set-up of the road on concrete bridges, see Figure 2.5.

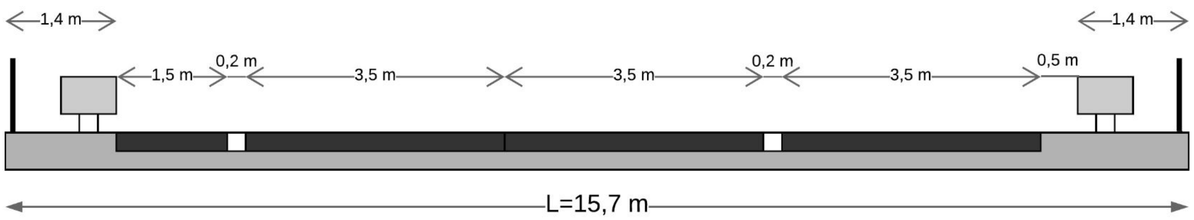


Figure 2.5: Cross-section of the deck set-up with a width of 15,7 meters based on two driving lanes and a safety lane.

As can be seen in Figure 2.5, the traffic lanes have a width of 3,50 meters and they are separated by 0,15 m wide stripes (not visible in the figure above). On the sides of the road there is a 0,20 meters wide solid line with on the right side a hard shoulder including the safety lane which is also 3,50 meters wide. The safety lane might be used as traffic lane during rush hours or when other lanes are blocked due to an accident or maintenance. On the left side of the carriage way there is a hard strip of 1,50 meters wide. In the 1,4 meters on both sides of the asphalt pavement, a safety barrier is situated with a width of approximately 0,60 meters and on the outside a footpath for maintenance and a parapet. For a two lane highway the bridge deck becomes 15,7 meters in total, when lanes are added or removed the total width will be 15,7 ± 3,50 meters. The maximum number of lanes in this project is three, this results in a bridge width of 19,2 meters.

For an overview of the road design including a safety lane, see Figure 2.6. In this Figure, half of the embankment is drawn, with a slope of 1:2. Under the bridge, at the location of the abutment, the embankment will have a slope of 2:3. For the amount of materials in the result phase, the cross-section of the embankment has to be determined, this will be done with the formula for the surface of a trapezium, see Equation 2.2.

$$Area = [(W_{top} + W_{bottom}) * h]/2 \tag{2.2}$$

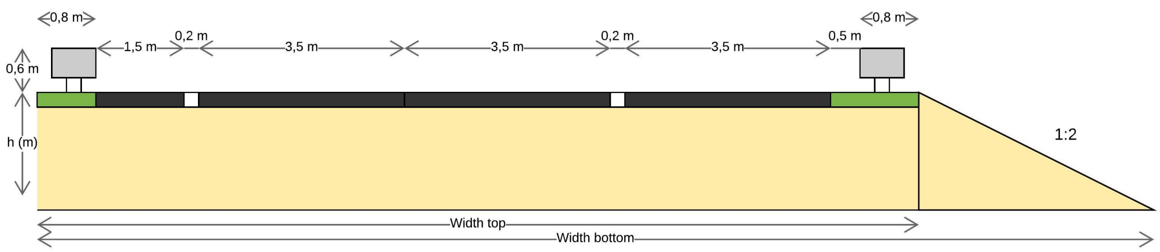


Figure 2.6: Cross-section of the embankment including emergency lane.

When there is no emergency lane present, the embankment will have a smaller top width, see Figure 2.7. Instead of an emergency lane, there has to be some safety zone (A=1,50 m) and the safety barrier zone (W=0,80 m).

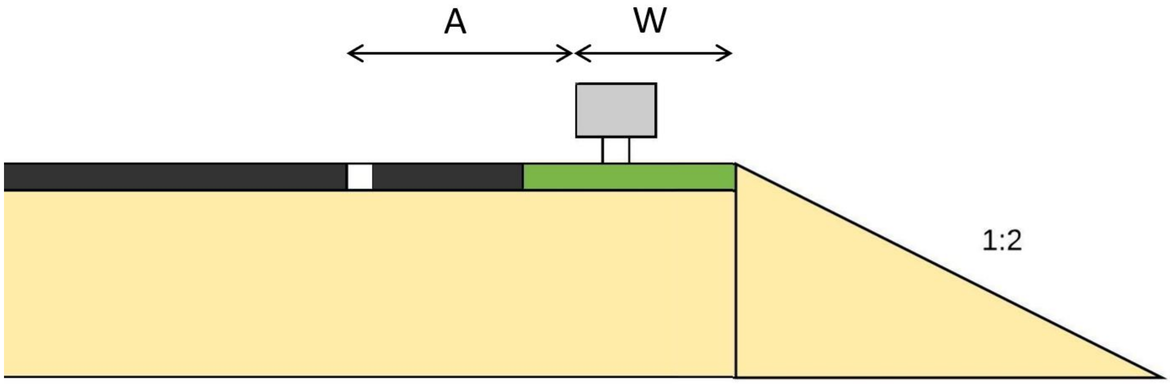


Figure 2.7: Cross-section of the embankment without emergency lane.[8]

2.2.3. INTEGRATION INTO THE PARAMETRIC MODEL

The alignment and the cross-section will be integrated into the parametric model with help of formula's to construct the points and scalable lines to connect them. The user input will be the starting point of the road (in the model point A: $\{x, y, z\} = \{0, 0, 0\}$), the clearance envelop and the length of the main bridge. The parametric model should construct the alignment from A to B, which will form the base for the design.
Note that the clearance by a river crossing is dependent on the water level, since this is not a constant value, the highest value should be governing. According to a research report by Brolsma Advies the clearance should be 11,35 meters above the highest water-level [9]. For the clearance above roads, 4,6 meters is prescribed by the ROA [7]. This is determined by the truck height, additional height due to movement of the truck plus an extra safety zone. To increase the safety in this project there is chosen for 12 meters above river and 5 meters above roads.
In addition, there is also the possibility to choose a self-defined bridge width, so not depending on the number of lanes. This gives the user the freedom to design every possible bridge width and to use the model also for other road types.

2.3. BRIDGE TYPOLOGIES

In this section the different types of structural solutions for the main span and the approach bridges will be described, these will be the input for the parametric model. For each type the basic geometry will be described combined with some visualizations and mechanical schemes for the flow of forces. This will help to better understand the bridge structure before the actual modelling of the bridge in the next phase will take place. The parameters and variables for the parametric design and a more detailed description of the simplified bridge model will be described in Chapter 3.

In the overview in Figure 2.8 some different bridge types per span length are shown. Not all bridges in this graph will be included in the parametric model; note also that the steel bridges are mainly used to have some additional comparable bridge types for the longer span lengths. The main focus in this thesis will be on concrete bridges.

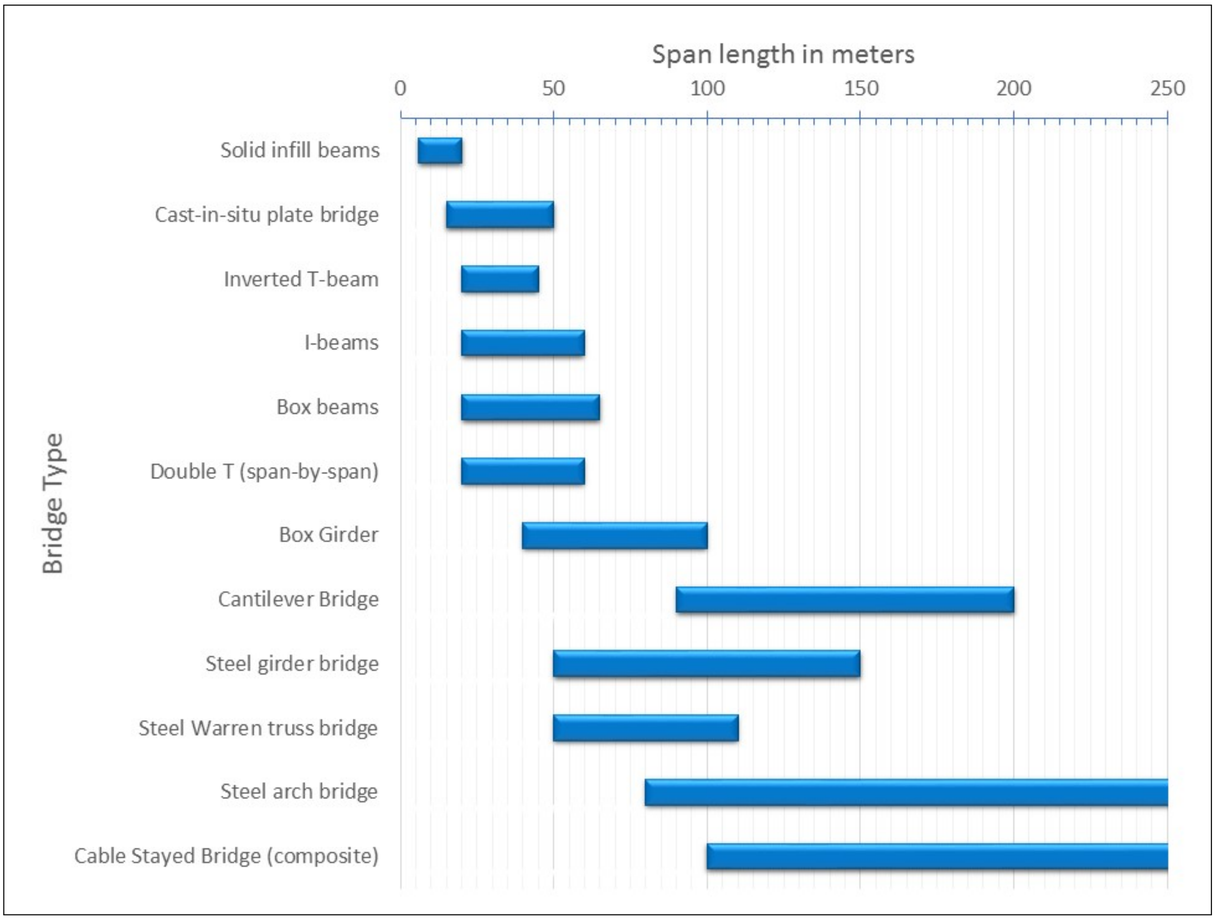


Figure 2.8: Bridge types per span length. Source: Lecture material CIE5127 Concrete bridges[10], Spanbeton and Master Thesis Report by Edward Fransen.[11]

As can be seen in the overview above, there are not many possibilities for large span bridges unless the choice will be made to take also composite or steel bridges into account. But this falls outside the scope, since in this thesis the focus is on bridges with concrete as their main structural material. This makes the balanced cantilever bridge the largest possible bridge to design as the main bridge.

For the approach bridges the choice is made to focus only on the pre-stressed prefabricated beams to reduce the hindrance for underlying roads during the construction phase. Two types of beams are chosen, namely the inverted-T- or I-beam and the box beam.

2.3.1. MAIN BRIDGE - BALANCED CANTILEVER BRIDGE

Geometry

In this section the balanced cantilever bridge, see Figure 2.9, will be described. This bridge is known for it's parabolic shape with changing structural height, consisting of multiple 3-5 meter long segments build-up from the supports towards the middle. From previous research dissertations follows, that a second or third degree function for the bottom shape of the bridge is not the most optimal choice. Instead a 2.5 degrees function proves to be more suitable for the structure.[12] This shape has the advantage of staying horizontal over a larger distance at midspan compared to the second degree function, which means that the total bridge height can be reduced to meet the same clearance. At the hammerhead the height of the bridge is still sufficient to take up the large bending moments.

Figure 2.9: Cantilever bridge at the A67 across the river Maas, Grubbenvorst 1967. Source: Cement article: "Maasbrug Grubbenvorst beoordeeld" in number 7 2017 [13]

The balanced cantilever bridge consists of a box shape with one or multiple cells and can be made with prefabricated elements or cast in place with travelling form-work, both in combination with prestressing cables and or reinforcement. The top flange of the box girder is cantilevering out approximately 3 meters of the box itself, in this way there is a wider deck possible without using extra material for the bottom part of the structure.

Advantages

One of the special features of the cantilever bridge is the fact that during construction there is no supporting structure needed on the ground, so there is almost no hinder for the surroundings. Only the hammerhead is supported on the ground during construction, the segments which follow are all hung up by lifting equipment which is anchored in the previously cast segment. That is why this type of bridges is mostly applied over rivers or waterways with a large width. Another advantage is the relatively high construction speed, because of the high repetition rate. The form-work can be used multiple times with only changing the height of the structure and the thickness of the bottomflange. This also saves costs because there is no need to design and build new form-work for each segment.

Box Girder Bridge
Keeping the construction height of the bridge constant, and by doing so constructing a box girder bridge, would be even more cost efficient for the re-use of the form-work. But this would also mean that the bridge would be inefficiently high in the middle of the span. This type of bridges is therefore mostly applied to smaller spans (40 to 90 meters) with preferably multiple spans of the same length. In that case, the incremental launching technique could be applied, see Figure 2.10.
With incremental launching, the bridge will slide horizontally over the piers and each time there will be a new segment connected behind the previously cast element. This method has the advantage of building all bridge elements at the same location and does not need any supporting structure on the ground. A large disadvantage of this construction method is the constantly changing moment distribution during the sliding of the elements over the piers, causing stress changes in the cross-section. This requires a larger construction height and more reinforcement than the amount that is needed in the end phase of the bridge. The slenderness ratio for this type of bridges is 25 when supported by false form-work and 18 when using the incremental launching technique. The slenderness ratio is determined by dividing the span length by the height of the bridge. The difference in construction method is also visible in the maximum span length. When the supported construction method is used, this results in 90 meters maximum and for the sliding method the maximum span length is only 60 meters.

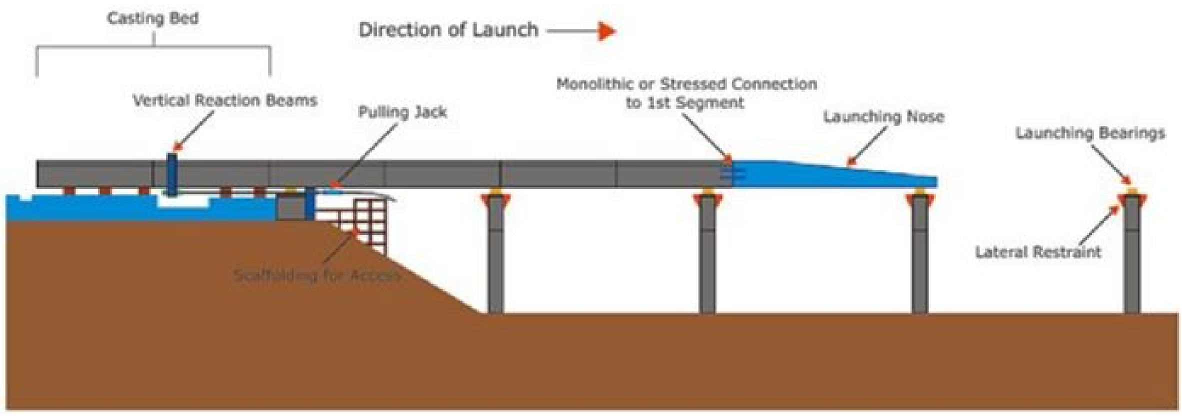


Figure 2.10: Building sequence of the box girder bridge with incremental launching. Source: en.vsl.cz/incremental-launching-method/

Disadvantages
A large disadvantage of the balanced cantilever bridge is the heavy self-weight compared to steel bridges (between 80% and 90% of the total weight). This requires a substantially larger foundation and this results in higher costs. A solution for this could be the use of high(er) strength concrete to decrease the cross-sectional area or the use of lightweight concrete. Note that this last option is forbidden by the Dutch Richtlijn Ontwerpen Kunstwerken (ROK) since there where some damage problems discovered during the prestressing of earlier designed bridges containing lightweight concrete. Most of the time C55/67 up to C70/85 is used to decrease the amount of concrete and so the self-weight.[14] The use of higher strength concrete is not investigated in this thesis.
Other disadvantages are potential colour differences between the elements (aesthetic) and the lower construction speed in case of the cast in place method. Reason for this is that every reinforcement cage must be made in a small amount of time and space, so it is not possible to double the amount of workers doing the braiding. Also there must be some time for the concrete to harden, before the prestressing cables can be stressed.

Construction Sequence for the Balanced Cantilever Bridge

The construction sequence of the bridge is displayed in Figure 2.11. At first, the piers are constructed with the hammerhead structure on top, this is the starting point of the cantilevering parts. The second phase is the connecting of the elements on both sides of the hammerhead, if planned well, there can be one new element on each side per week. The advantage of building on both sides at the same time is the fact that the structure is almost completely in balance. Only small bending moments have to be taken up by the hammerhead and the extra supporting structure, which is just a tension element a few meters next to the hammerhead piers.
When either the embankment or the other half of the span is reached a connection will be made, in the middle also called the stitch element. With this step the superstructure is completed and the asphalt package and other road elements can be build on top of the bridge.
When the side span is smaller than half the main span, there has to be extra weight on the sides to balance the total structure, this will be done by filling the box elements and so creating an extra counterweight. In practice this (partly) filling of the box elements is also applied when the span lengths are in balance, just to avoid negative reaction forces at the embankments due to unequal loading of the bridge.

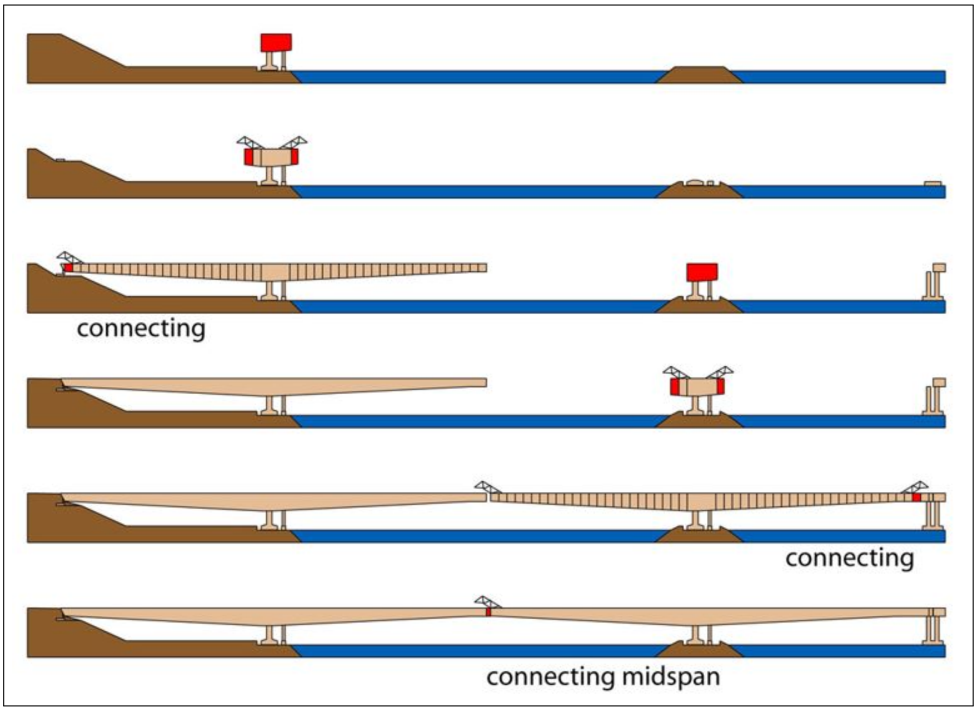


Figure 2.11: Building sequence of the balanced cantilever bridge. Source: Lecture slides CIE5127 Concrete Bridges [10]

Force schemes

Once the bridge is finished, it can be simplified to a line on four supports with a parabolic moment distribution. But just before the last segment is cast, the bridge has another moment distribution, see moment line A in Figure 2.12. Moment line B gives the distribution due to the self weight and the additional (traffic) load.
In situation A, there is tension in the topflange of the bridge and in situation B, there is tension in the bottomflange at midspan. The rest of the bridge will be mainly in compression.

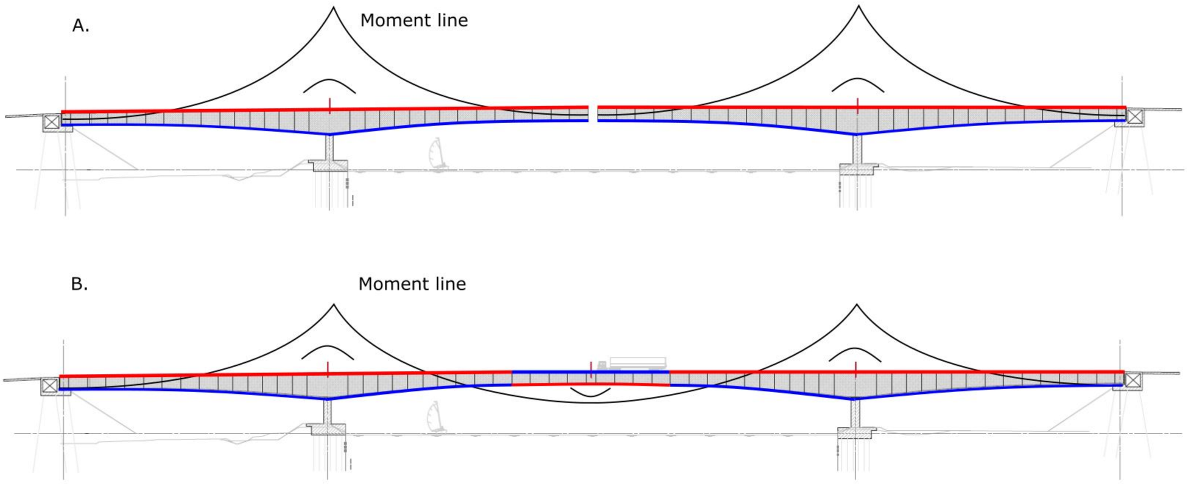


Figure 2.12: Moment lines for the bridge in the last construction phase (A) and during the use phase (B). Red is tension, blue is compression. Figure edited from an article in Betoniek. [14]

Prestressing Methods

Since the bridge is cantilevering out during the construction phase and defecting in the middle during the use phase, the concrete cross-section has tensile forces to withstand. Since these tensile forces are to large to be taken up by the concrete, there must be reinforcement or prestressing steel to take up the tensile force and strengthen the concrete.

With reinforced concrete there is a deformation needed to activate the steel rebars, this means the concrete must crack before the steel takes over the tension force. This cracking of the concrete effects the durability of the structure. Also the maximum deflection is higher compared to pre-stressed structures. Another disadvantage is the connecting or stitching of the reinforcement between each segment, this is needed to create a continuous tension element through the concrete cross-sections. So each reinforcement cage must be coupled to the previously cast segment, this problem does not exist when prestressing cables are used. These cables go through multiple segments, so the coupling of the elements is not an issue any more.

The basic idea of prestressing is bringing the concrete under compression by tensioning the steel tendons. By doing so, the cracking of the concrete is prevented unless the structure will deform substantially due to external loading, so the compression stresses will change into tension stresses. This two types of concrete strengthening will be explained including mechanical properties in Section 2.5.

The prestressing of the bridge is sometimes desirable in three directions; longitudinal from the web to the deck of the box-beams, transverse in the deck slab and vertical in the webs. In practice it is nowadays more common to use only straight cables in the deck instead of cables which smoothly flow from the web into the deck slab. In this case there must be quite some shear reinforcement to "hang" the structure onto the previously cast element.

The tendons are post-tensioned, this means that the concrete is hardened before the cables will be stressed. To make this possible, there are hollow tubes cast into the concrete segment. Once the concrete is hardened enough, the tendons will put through the tube and stressed at the ends. After this the tubes will be grouted to protect the steel cables.

Another possibility is the use of external prestressing, here the cables are completely outside the concrete cross-section and are connected to the concrete with help of deviators. These deviators are highly reinforced concrete consoles which stick out of the bridge internal surface. See Figure 2.13.

Figure 2.13: External prestressing in a balanced cantilever bridge. Source: Schäfer Bauten, www.researchgate.net/figure/Inside-of-a-box-girder-of-a-post-tensioned-concrete-bridge-\with-deviated-external$_fig3_242280102$

Applicability

In the Netherlands, the common length for the main span of a concrete cantilever bridge is between 90 and 200 meters, with a slenderness ratio varying from 15 to 26 but most of the time a slenderness ratio of 22 is used. This ratio counts for the height of the bridge at the supports. At midspan and at the outer supports the height will be approximately 2-3 meters. The world record for this type of bridges is with a span of 301 meters the Stolmabrua in Norway, see Figure 2.14, other long cantilever bridges can be found in Turkey or countries in south-east Asia. In these countries the soil mainly consists of rock instead of weak soils like in the Netherlands, so the foundation of the bridge is less difficult.

Figure 2.14: Largest balanced cantilever bridge of the world, Stolmabrua in southwest Norway.
Source: www.vareveger.no/artikler/rekordbrua-er-ferdig/224727

Design Verification

The bridge has to be roughly checked at multiple places to make sure it is structurally sound. The checks which will be performed in the model are the normal force capacity and the moment capacity of each cross-section. Also there is a fitness check for the maximum amount of cables. This check makes sure that the width of the cross-section is sufficient enough to contain all the cables. This fitness check will be performed only at the location of the hammerhead, where the number of cables will be maximal.

In order to verify the model, the bridge geometry is also put into an Excel-file which determines the force distribution numerically. For the working of this Excel-file, see Appendix I.
The results of the model and the Excel-file are checked by structural engineers at Movares. Also a couple of checks with comparable bridge designs were performed. Due to confidentiality reasons, the results of these comparisons with existing bridge designs is not included in this master thesis report.

2.3.2. APPROACH BRIDGES

In this section several structural solutions for the approach bridges will be explained briefly, combined with information about the applicability. The approach bridges are sorted on span range, see Figure 2.15. Prefabricated elements are preferred due to the high construction speed and low hindrance. In this category three types are distinguished namely solid deck bridges, girder bridges and (curved) box beam bridges. These bridge designs are based on bridge types which are produced by Consolis Spanbeton, a Dutch company which is specialized in prefabricated concrete products. In Figure 2.16, an example of a prefabricated concrete viaduct can be seen.

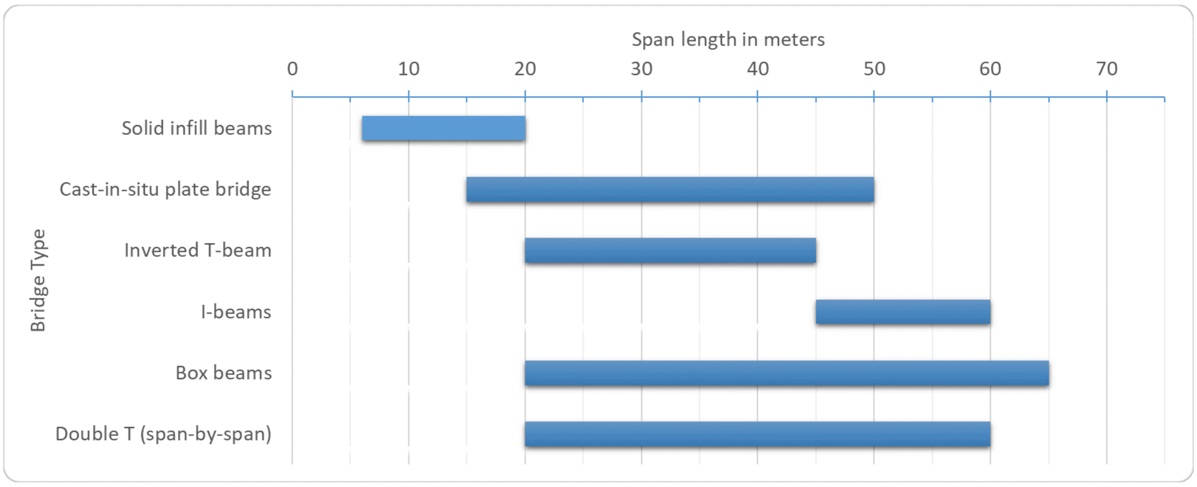


Figure 2.15: Bridge types per span length for the approach bridge. Source: Lecture material CIE5127 Concrete bridges[10] and Spanbeton

Figure 2.16: Construction of box girders with a span of 61 meters across the Dutch highway A12 near Zoetermeer. Source: Spanbeton, photo by Ineke Key Fotografie

SOLID DECK BRIDGES

Solid deck bridges are mainly used for small spans or when it is not possible to place the supporting columns in a regular pattern [15]. This structure is an alternative for the cast-in-place reinforced solid deck bridges, but has as a large benefit a high construction speed and low use of formwork.
As soon as the span increases the self weight of the bridge increases also rapidly, so the choice for a solid bridge deck is no longer economically justified.

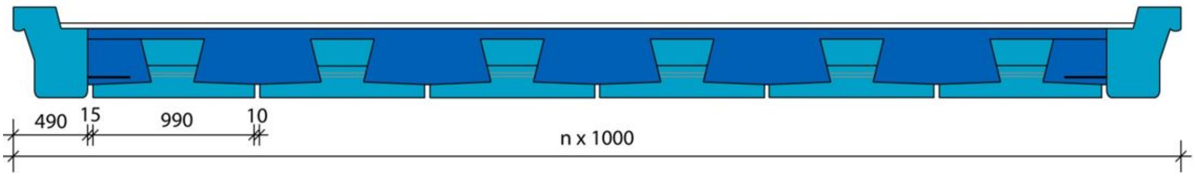


Figure 2.17: Solid deck with in-filled beams. Source: Lecture slides CIE5127 Concrete Bridges [10]

When slabs are used the span is 4-8 m maximum and when beams are used the maximum span is 6-20 m. These structures have a slenderness ratio of 20-25. The solid deck bridges consists of pre-stressed slab elements for the short spans or for the larger spans pre-stressed beams with a cast-in-place topping, see Figure 2.17. For the strengthening reinforced as well as pre-stressed concrete is used. For the design of these type of bridges, Figure 2.18 can be used. Note that the axis-titles are in Dutch, so to clarify; on the vertical axis the height of the profile is shown in millimetres and on the horizontal axis the length of the span in meters.

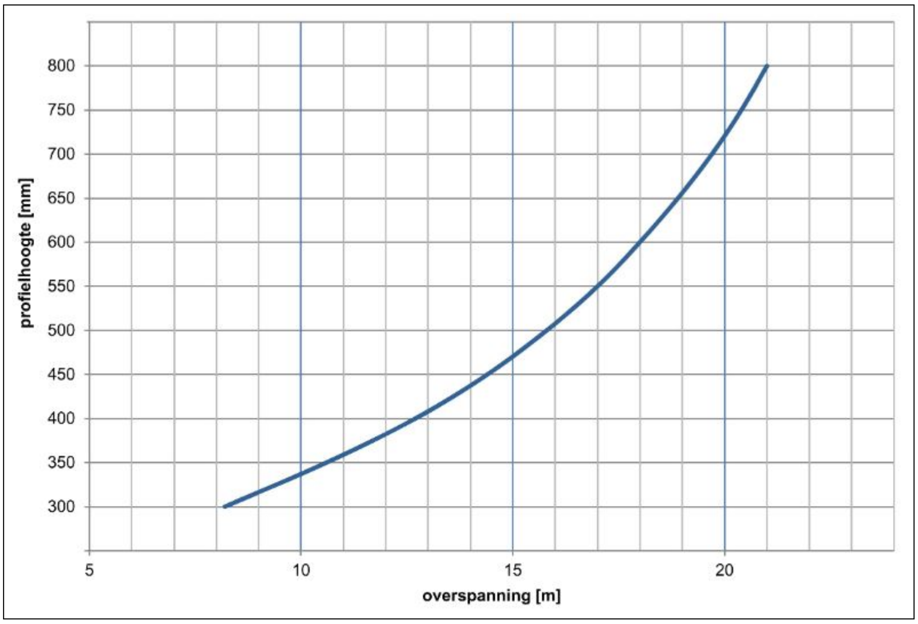


Figure 2.18: Design graph for solid deck bridges. Source: Spanbeton

Due to the complexity of modelling in-filled elements in Grasshopper/Karamba (in multiple stages), this type of prefabricated beams is not used in the model. As a consequence only the girders and box beams are left to apply in the model.

GIRDER BRIDGES

One can distinguish pre-stressed girder bridges between: inverted T-shaped and I-shaped beams, see Figure 2.19 Both beams are used in combination with cast-in-place cross beams at the supports and a cast-in-place deck. Compared with the solid deck bridges, the girder bridge designs use a significantly lower amount of material since there is a hollow space between the beams. In some cases it is possible to create a larger centre-to-centre distance to reduce the use of material even more. The girder bridges have a slenderness ratio of 20-28. The inverted T-shaped beams are suitable for spans of 15-45 m, while the I-shaped beams span up to 60 m due to a higher stiffness. In Figure 2.20 the design graph for inverted-T and I-beams is shown.

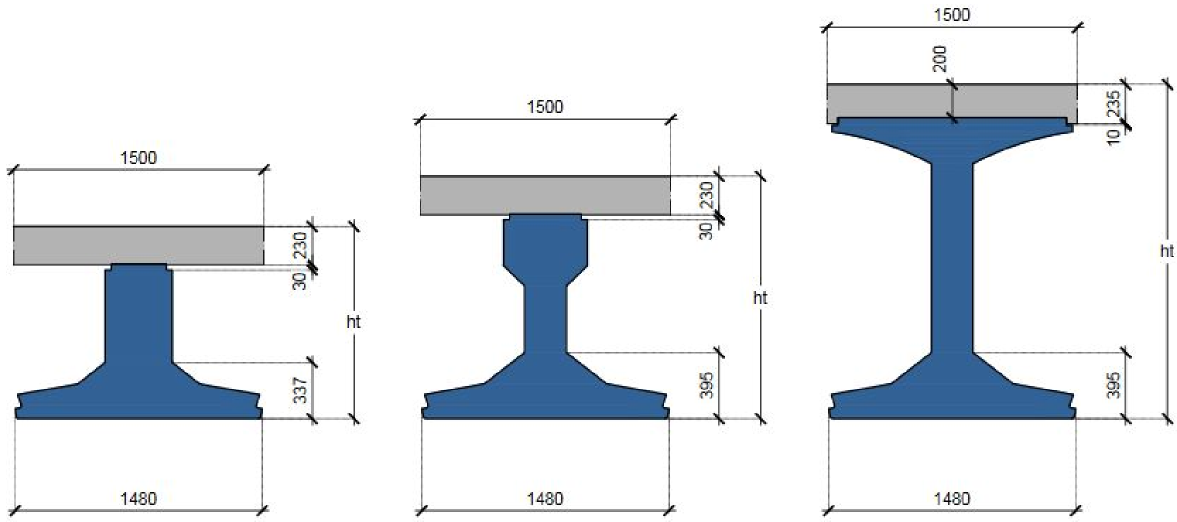


Figure 2.19: Girder bridges from inverted T-beam (left), inverted T-beam with compression zone (middle) and the I-beam (right) Source: Spanbeton

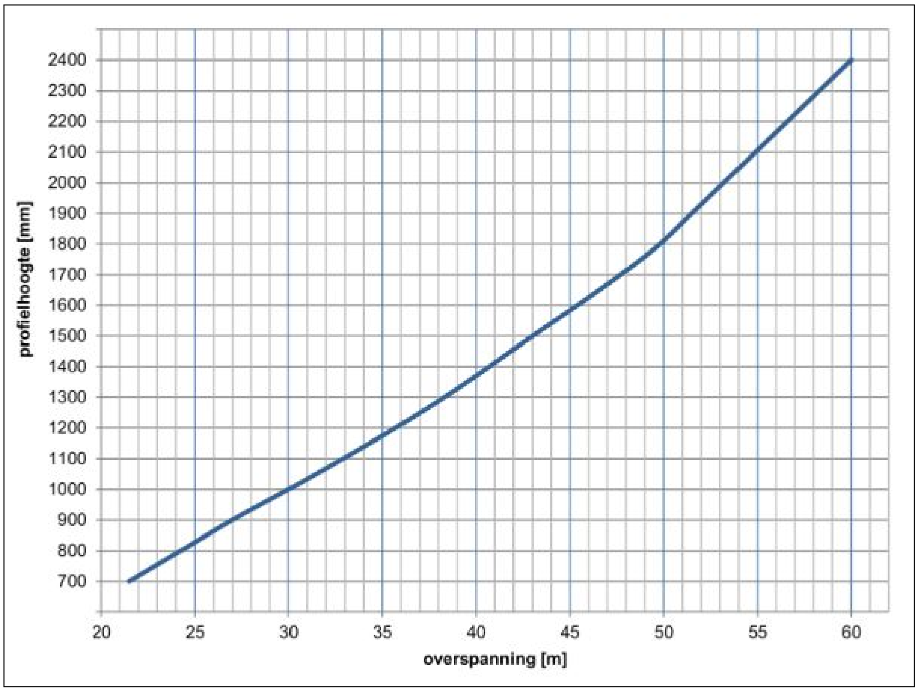


Figure 2.20: Design graph for girder bridges. Source: Spanbeton

Construction sequence

The concrete beams are cast in a factory in a steel mould. In this mould there is the required amount of reinforcement and prestressing steel or tendons. These tendons are stressed by use of large hydraulic jacks and after prestressing the cables are locked onto an anchor plate. Now the concrete will be cast between the stressed tendons.
After the concrete is hardened up to a part of it's final strength the tendons will be cut from the anchoring plate. In this project a concrete class of C35/45 is assumed during the cutting of the tendons, the final strength which is reached after 28 days is C60/75.
Once the tendons are cut from the anchoring plate, the steel will transfer the tension force to the concrete over a so called transmission length. After this transmission length of approximately one meter, the initial force of the prestressing steel is reached in the cross-section. Now the beams are ready for transport and further application.
At the construction site, the beams are hoisted into place by use of mobile cranes, Figure 2.21. The beams are supported by capping beams, horizontal beams which transfer the load from each specific beam towards the columns beneath. Each two beams have a small gap between the bottom flanges which has to be filled to secure a continuous bottom flange. The reason for this is to make sure that all beams take up the load of for instance a collision by a truck on the road underneath. To reduce the self weight, there must be hollow spaces between the beams, that is why there will be formwork between the beams to support the cast-in-place compression layer. This compression layer is cast on top of the girders and connected with shear reinforcement which makes the elements into a whole bridge deck. This bridge deck also makes sure that point loads will be spread over multiple beams in transverse direction.
Subsequently the asphalt layers and other road elements are build and the bridge is ready for use.

Figure 2.21: Construction of a bridge span with I-beams. Source: Spanbeton

Crossbeams

The crossbeams are designed to take up the torsional forces in the bridge and to distribute the forces between the beams by creating a fixed end. For the box beams there are no crossbeams needed, since these beams already have enough torsion capacity and the ends of the box beams are made of solid concrete. The crossbeams are not further explained in this project and also not implemented in the parametric model. For an impression of these crossbeams, see Figure 2.22.

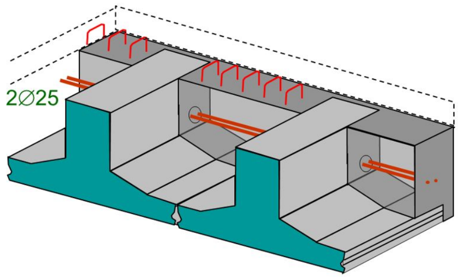


Figure 2.22: Crossbeam between the girders with reinforcement.[10]

BOX BEAM BRIDGES

The Box beam is known for its slender appearance with a low self-weight, but also for its torsion stiffness, that is why the box beams are suitable for curved bridges as well. The minimum radius for the horizontal curvature of the box beams is 50 m. The beams are prestressed in the longitudinal direction with in between cast-in-place joints, the beams are tied together with transverse prestressing. Since the box bridge does not need any structural floor on top of the beams, there is quite some saving of construction height, see Figure 2.23.

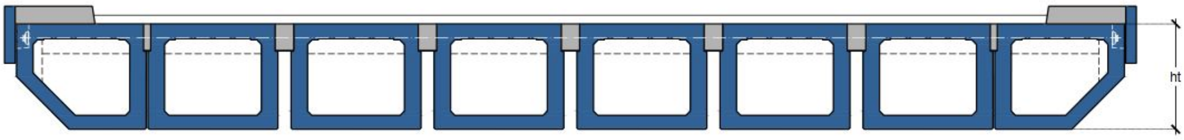


Figure 2.23: A box beam construction with small construction height (ht). Source: Spanbeton

Another advantage of the box beam bridges is the high construction speed, after a week the bridge deck is ready to use due to the fact that it is almost completely prefabricated. The box beams are suitable for statically determinate constructions and can be used perpendicular and under an angle. The slenderness ratio for the box beams is 28-32 and they are applicable in the Netherlands until a span length of 68 m [2]. In Figure 2.24, the design graph for box beam bridges is shown, with on the vertical axis the height of the beam in millimetres and on the horizontal axis the span length in meters.

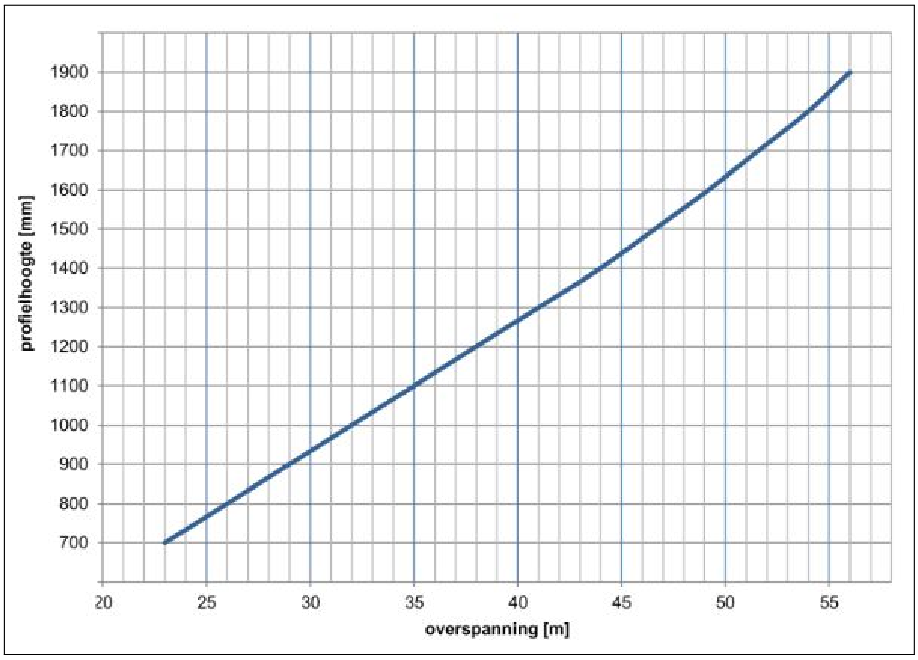


Figure 2.24: Design graph for box beam bridges. Source: Spanbeton

[2]This is the upper boundary, limited by the transport possibilities of the beams [16]

2.4. SUBSTRUCTURE & FOUNDATION DESIGN

For the substructure and foundation, which consists out of the piers, abutments, capping beams, foundation slabs and piles, the ROK (Richtlijnen Ontwerp Kunstwerken) version 1.4 Appendix B[17] is used. Also the document by Rijkswaterstaat "Vuistregels voor het ontwerpen van betonnen bruggen en viaducten"[18] will be used. Although this document is quite old (2004), it is still applicable for the preliminary design phase. The design of the substructure will be simplified as much as possible, so small details or joints will not be taken into account. The design of the substructure should satisfy the compressive capacity that is needed as well as the buckling capacity for the columns, this will be checked by simple calculations in Grasshopper.

2.4.1. PIERS

The piers consist of round columns for the approach bridges with a diameter of minimal 0,8 m or walls with a cross-section of minimal 0,6 * 0,8 m^2 combined with a foundation slab of minimal 1,0 m height. When the capacity of the pier has to be enlarged, the foundation slab will become tapered to provide a better load distribution. The width of the slab will increase as well as the number of foundation piles. Note that the width of the slab should never exceed the width of the bridge deck. This enables the possibility for future road extensions, for instance the increase of the number of lanes and thereby the widening of the bridge. For an overview of the possible pier solutions with the foundation beneath, see Figure 2.25.
The main bridge will be supported at the two middle supports by hammerhead piers, these are large rectangular columns or walls which carry most of the load (approximately 90 %). At the outer ends of the bridge, the bridge is supported by the columns and capping beams which also carry the approach bridge. Since there is a large concentrated load at the location of the hammerheads, the foundation slab underneath should be quit big to ensure sufficient spreading of the loads towards the foundation piles. Thickness of the foundation slab of the hammerhead could be a couple of meters, depending on the weight of the bridge.

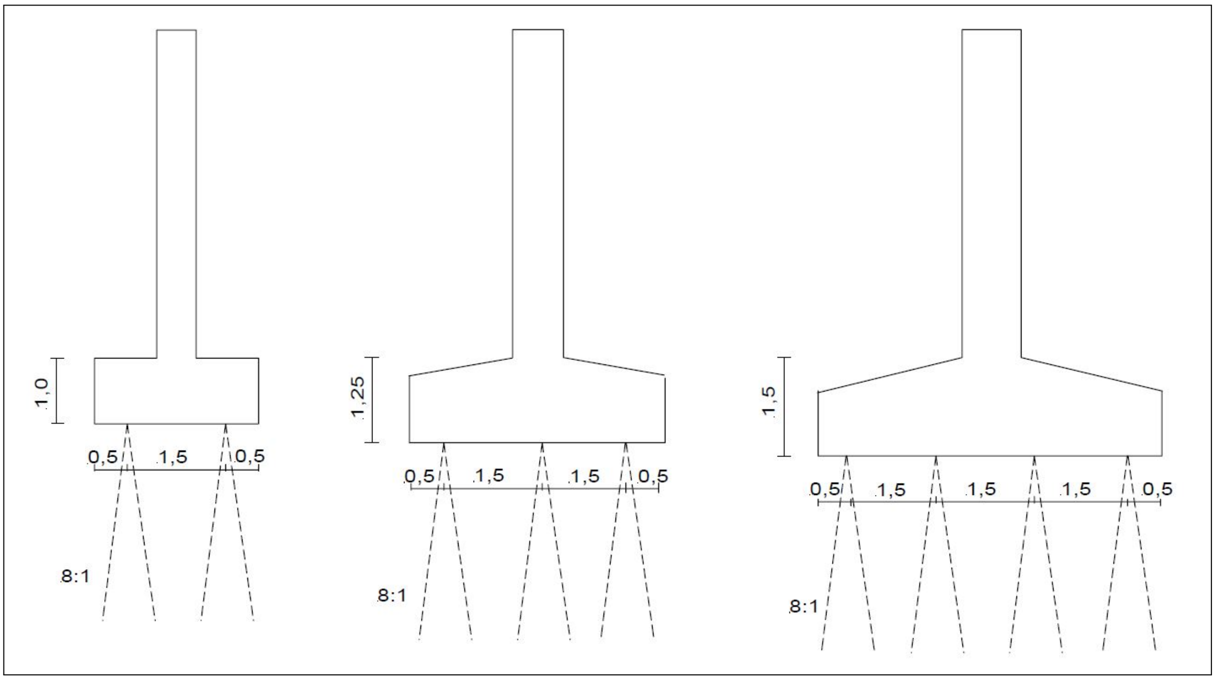


Figure 2.25: Design guidelines for the intermediate supporting piers[18]

2.4.2. CAPPING BEAMS

On top of the piers is for the approach bridges a capping beam to support the individual beams, see Figure 2.26. This beam is around 1,0 m high (minimal) and is supported by the piers each 4-5 meters. The capping beams has a rectangular cross-section with a width minimal equal to the diameter of the columns. Also, since the capping beam has to support two spans, the width of the capping beam is determined by the angle between the column row and the longitudinal direction of the crossing. The maximum width, which follows from an angle of 45 degrees, is $2 * 870 + 100 = 1840mm$. In this project only straight crossings (90 degrees) are taken into account, this results in a width for the capping beams of $2 * 620 + 100 = 1340mm$ this will be rounded to 1,5 m to keep it roughly as a preliminary design.

Figure 2.26: Fly-over with box girders supported by capping beams. Source: Spanbeton

2.4.3. ABUTMENTS

The abutments (Figure 2.27) are only used on the begin or end of the bridge and they are embedded, in the embankments. Since the abutments only have to carry one half of the span length, the number of foundation piles under these abutments should be less then under the normal supporting piers. The dimensions of the abutment are estimated and they will not be calculated in this project.

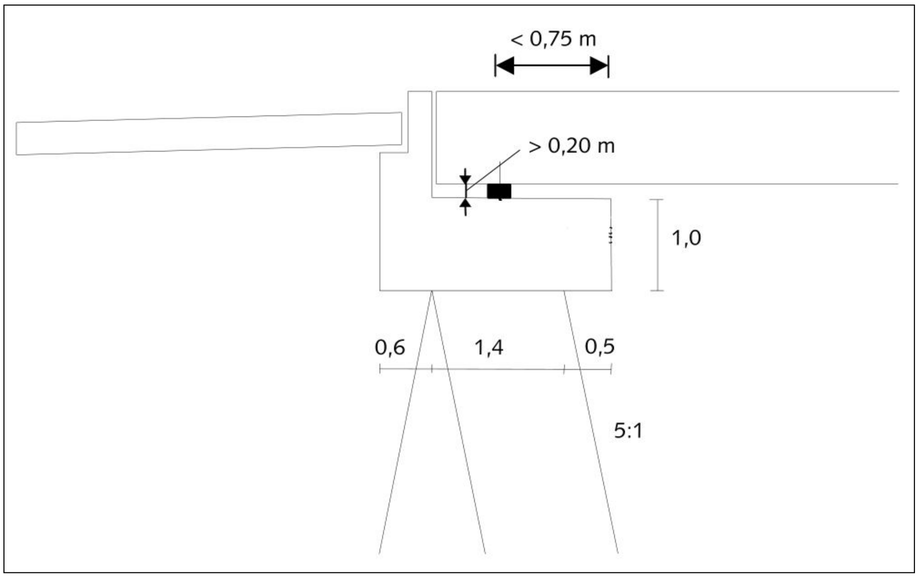


Figure 2.27: Design guidelines for the supporting structure at the outer-ends of the bridge [18]

2.4.4. FOUNDATION PILES

The number of foundation piles depends on the capacity that is needed to support the bridge weight, traffic loads and the self weight of the piers. Since the soil properties differ for each location, an estimation for the pile capacity is made in this project to simplify the model. The following assumptions are made.[3]

- The concrete class that will be used is C55/67
- The cross-section is 450*450 mm^2 with a length of 20 meters
- The compressive strength of the soil is taken as 8 MPa (or 8.000 kN/m^2)
- The piles are spaced 1,5 meters c-t-c (possible under an angle of 8:1), see Figure 2.28
- Capacity of the pile with the above mentioned values, is assumed as 1500 kN
- Total volume per pile: 4.05 m^3 concrete
- Prestressing steel estimated as a single tendon with 7 strands Ø 9,3 mm – Y1860

In the model, these piles are only used as straight piles and the length, cross-section, capacity and centre-to-centre distance are user defined. In this way it is possible to give a realistic estimation of the material amounts for each location.

Figure 2.28: Prefab concrete foundation piles in a rectangular grid. Source: De Groot Funderingstechnieken NV, www.dgft.eu/funderingen/fotos/palen/geheide-prefabpalen-2/

[3]Based on information from the manufacturers Haitsma beton http://www.haitsma.nl/heipalen/funderingen-techniek/ and Vroom funderingstechnieken https://www.vroom.nl/nl/products/15-prefab-heipalen-beton

2.5. MATERIAL PROPERTIES

In this section the material properties of concrete and steel will be briefly discussed to get a better understanding of the behaviour and the (dis)advantages of the materials. Also this will explain certain choices later on in the design process of the bridges.
Since the goal is to find an optimal design in terms of material usage, cost and environmental impact, these terms will be described also in this section. The materials which are used in the model and the different concrete classes per element will be described. After that, the costs of the materials will be explained with a detailed specification of the costs for each element. Subsequently the environmental impact of the material will be described by first explaining the reason for implementing this sustainability aspect in this project.

2.5.1. MECHANICAL PROPERTIES

Concrete

The material concrete consists out of different base materials, namely cement, sand, aggregates and water. These materials are mixed together in a certain distribution and there will be a chemical reaction which forms the concrete. The composition of the material and the additives determine the strength of the end product, for those different strength classes see Figure 2.29 from the Eurocode, NEN-EN 1992-1-1, section 3.1.2.

	Sterkteklassen voor beton														Vergelijking/Verklaring
f_{ck} (MPa)	12	16	20	25	30	35	40	45	50	55	60	70	80	90	
$f_{ck,cube}$ (MPa)	15	20	25	30	37	45	50	55	60	67	75	85	95	105	
f_{cm} (MPa)	20	24	28	33	38	43	48	53	58	63	68	78	88	98	$f_{cm} = f_{ck}+8$(MPa)
f_{ctm} (MPa)	1,6	1,9	2,2	2,6	2,9	3,2	3,5	3,8	4,1	4,2	4,4	4,6	4,8	5,0	$f_{ctm}=0,30 \times f_{ck}^{(2/3)} \leq$C50/60 $f_{ctm}=2,12 \cdot \ln(1+(f_{cm}/10))$ > C50/60
$f_{ctk,\,0,05}$ (MPa)	1,1	1,3	1,5	1,8	2,0	2,2	2,5	2,7	2,9	3,0	3,1	3,2	3,4	3,5	$f_{ctk;0,05} = 0,7 \times f_{ctm}$ 5 % fractiel
$f_{ctk,0,95}$ (MPa)	2,0	2,5	2,9	3,3	3,8	4,2	4,6	4,9	5,3	5,5	5,7	6,0	6,3	6,6	$f_{ctk;0,95} = 1,3 \times f_{ctm}$ 95 % fractiel
E_{cm} (GPa)	27	29	30	31	33	34	35	36	37	38	39	41	42	44	$E_{cm} = 22[(f_{cm})/10]^{0,3}$ (f_{cm} in MPa)
ε_{c1} (‰)	1,8	1,9	2,0	2,1	2,2	2,25	2,3	2,4	2,45	2,5	2,6	2,7	2,8	2,8	zie figuur 3.2 ε_{c1} ($^0/_{00}$) = 0,7 $f_{cm}^{0,31} \leq 2,8$
ε_{cu1} (‰)	3,5									3,2	3,0	2,8	2,8	2,8	zie figuur 3.2 voor $f_{ck} \geq 50$ MPa ε_{cu1}($^0/_{00}$)=2,8+27[(98-f_{cm})/100]^4
ε_{c2} (‰)	2,0									2,2	2,3	2,4	2, 5	2,6	zie figuur 3.3 voor $f_{ck} \geq 50$ MPa ε_{c2}($^0/_{00}$)=2,0+0,085(f_{ck}-50)^{0,53}
ε_{cu2} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	zie figuur 3.3 voor $f_{ck} \geq 50$ MPa ε_{cu2}($^0/_{00}$)=2,6+35[(90-f_{ck})/100]^4
n	2,0									1,75	1,6	1,45	1,4	1,4	voor $f_{ck} \geq$ 50 MPa n=1,4+23,4[(90- f_{ck})/100]^4
ε_{c3} (‰)	1,75									1,8	1,9	2,0	2,2	2,3	zie figuur 3.4 voor $f_{ck} \geq$ 50 MPa ε_{c3}($^0/_{00}$)=1,75+0,55[(f_{ck}-50)/40]
ε_{cu3} (‰)	3,5									3,1	2,9	2,7	2,6	2,6	zie figuur 3.4 voor $f_{ck} \geq 50$ MPa ε_{cu3}($^0/_{00}$)=2,6+35[(90-f_{ck})/100]^4

Figure 2.29: Overview of the different concrete strengths and material properties. Source: NEN-EN 1992-1-1, section 3.1.2 [19]

As can be seen in the table above, the compressive strength of the concrete (f_{ck}) is around 10 to 15 times as strong as the tensile strength (f_{ctm}). This is because, when the material is in compression the forces can be transferred partly through the strong grain structure. But in the transverse direction, the interface between the grains and the mortar will be loaded in tension. Since this interface is the weakest link, the concrete will fail at these locations first. This is the reason that concrete will fail most of the time on tension.

When the tensile capacity is reached, the concrete will show micro-cracks at first and these cracks will grow into macro-cracks if the loading of the concrete continues. This will in the end lead to the breaking of the concrete section.

This tensile collapse problem could be solved by changing the shape of the structure so mainly compressive stresses do occur, an example structure is the arch. Since it is quite hard to prevent every tensile force in the structure due to unequal load distribution, this is not always a solid solution. A better solution for the low tensile capacity of concrete, is the use of other materials at the location where tensile forces occur. This is the reason why reinforced concrete is developed.

Reinforced Concrete
In reinforced concrete, the concrete will be used in it's strong points, namely compression and the steel will be used in tension. For a simple example, see Figure 2.30.

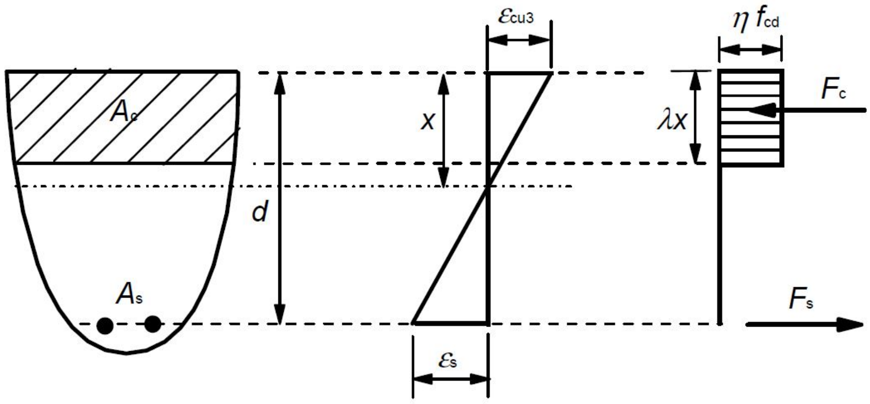


Figure 2.30: Example of a reinforced concrete section loaded in bending. Source: NEN-EN 1992-1-1, section 3.1.7 [19]

In this example, a beam with a certain cross-section is loaded in bending. This causes the top of the section to become loaded in compression and the bottom part will be loaded in tension. Since the concrete tensile strength is quite limited, the concrete will crack and the steel bars will take over the force. This is the principle of reinforced concrete, it is allowed to crack but the cracking is limited.

Prestressed Concrete
To prevent the concrete from cracking, another solution could be used, namely prestressing of concrete. With this technique the concrete will be pre-loaded in compression which will prevent the bottom part from becoming loaded in tension in case that the beam is loaded by a bending moment. Since in the case of prestressing, the concrete cross-section is not cracked, so the stiffness of the element will remain intact, this technique allows for longer and more slender structures. So the amount of concrete can be decreased by choosing for prestressed bridge designs.
Prestressing can be done in several ways. One option is to use external prestressing cables, here the steel cables are on the outside of the concrete cross-section but connected to the structure with deviator blocks. The second option is to embed prestressing cables in the concrete section, this can be done in two ways. One way is to pull the steel strands in the factory during the casting of the concrete, here the strands are directly in contact with the concrete and they can transfer shear forces along the length of the strands, this is called pre-tensioning. The second way is by using ducts embedded into the concrete, once the concrete is hardened, a bundle of strands or tendon will be shifted through the duct, brought in tension and anchored at the end. After the stressing of the tendons, the ducts are filled with mortar to conserve the tendons and thereby increase the service lifetime. This second method is called post-tensioning.

In this project the last two types of prestressing will be used. For the prefabricated beams, internal prestressing is used (pre-tensioning, with bond) and post-tensioning will be used for the cantilever bridge.

Pre-tensioning

When using prestressing strands in the factory, the strands will preferably have a straight alignment in the bottom of the cross-section. Although for mechanical reasons, a parabolic shape is more efficient since the moment distribution due to self-weight has the same shape $(1/8 * q * l^2)$.

In order to prevent high bending moments due to the eccentricity at the ends of the beam, the strands are smoothed for a certain length from the begin and end of the beam. In this way, the strands do not transfer any load to the concrete in these parts of the beam, so the bending moment is reduced. See Figure 2.31 for a schematic view of the above mentioned principle.

Also there is a so called transmission length, a distance over which the tendon force is transmitted to the concrete. At the start of this transmission length there is no axil force in the concrete and at the end there is the initial prestressing force active in the concrete section. The reason for this increasing force distribution from zero to initial, is that after the concrete is hardened, the strands are cut, so at the cutting location there is no stress present due to relaxation of the strands.

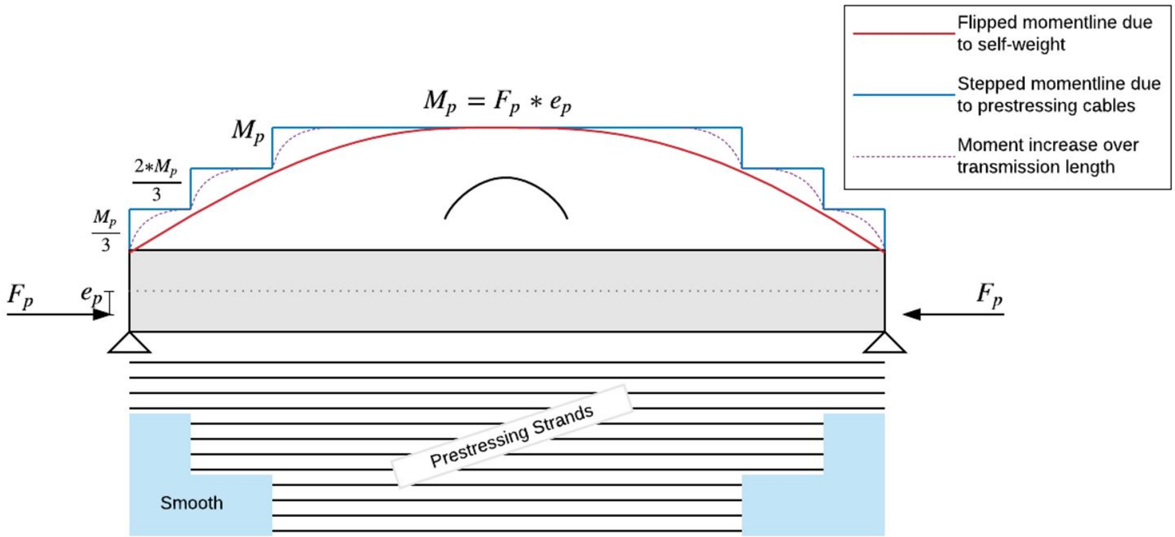


Figure 2.31: Principle of prestressing with straight bonded cables in prefabricated beams.

Post-tensioning with bonded tendons

With post-tensioning the steel strands are bundled into tendons and are stressed after the concrete is hardened. This technique will be used for the balanced cantilever bridge.

After casting the hammerhead, the first segments can be cast at both sides of the hammerhead. These segments are cast with help of tailor-made formwork and the prestressing ducts are already in the top flange. Also practical reinforcement is present to support the ducts and ensure the durability of the segment. After a couple of days, the concrete is hardened enough and the prestressing tendons are pulled through the ducts and stressed with help of a hydraulic jack. After the stressing of the tendons, the ducts will be grouted to conserve the tendons and to enable force transfer from the cable into the concrete. After a week, the first segments are ready and the process repeats itself with the following segments.

At the location of the anchors, the prestressing force is introduced in the segment and since the tendons have a certain eccentricity to the neutral axis of the cross-section, there is a moment generated. This moment is given by $M_p = \Delta F_p * e_p$, with all parameters as a function of the position along the length of the bridge. When the grout is hardened, the concrete and prestressing steel work together like a composite structure, both deform in the same way but stresses differ. This behaviour is comparable with reinforced concrete behaviour.

Steel

As described in the previous paragraphs, there will be used reinforcement and prestressing steel to strengthen the concrete cross-section. In the next paragraphs the material properties of these steel types will be explained briefly, supported by tables and graphs.

Reinforcement Steel

For the reinforcement steel, standard B500B will be used with a characteristic yield strength of 500 N/mm^2. Since in this project there will be no calculations on the reinforcement design, the detailed material properties are not discussed in this thesis. The material is mainly taken into account for the material use, costs and shadow costs. Therefore an overview is made with the assumed amount of reinforcement per structural element, see Table 2.2. The price of the reinforcement is taken as 1200 €/ton and an environmental impact value or costs indicator (ECI) of 162,84 €/ton. This ECI factor will be explained in Section 2.5.4.

Table 2.2: Overview of the amount of reinforcement per element or bridge type. Source: Movares

Element type	**Concrete Strength**	**Reinforcement** $[kg/m^3]$
Cantilever Bridge	≥C55/67	180
Prefabricated beams	C60/75	100
Concrete Deck	C30/37	100
Columns/Piers	C30/37	135
Capping beams	C30/37	135
Foundation Piles	C55/67	100
Foundation slab	C30/37	125

Prestressing Steel

For the prestressing of the cantilever bridge and the prefabricated beams, there is used a standard prestressing steel type of Y1860S. This are strands with a design strength of 1454 N/mm^2 and after prestressing losses are taken into account, a strength of $1454 * 0,8 = 1163 N/mm^2$ is left.
There are tow types of strands used, one with a cross-sectional area of $100mm^2$ for the cantilever bridge and one with a cross-sectional area of $150mm^2$ for the prefab beams. In the model, all these material properties can be changed by the user.
For an overview of the mechanical properties of the prestressing steel, see Figure 2.32. The price of the prestressing steel is taken as 1750 €/ton and ECI of 431,53 €/ton.
Note: this ECI value is based on the old environmental database since it is not present in the new version. The ratio between the reinforcement and prestressing steel is taken equal to the ratio in the old version.

staal-soort	voorspan-element	f_{pk} (N/mm²)	ε_{uk} (‰)	$f_{p0.1k}$ (N/mm²)	f_{pd} (N/mm²)	max. trekspanning: tijdens spannen $\sigma_{p.max}$ (N/mm²)	max. trekspanning: na aflaten max $\sigma_{p,0}$ (N/mm²)	knik in σ-ε-diagram UGT: f_{pd}/E_p (‰)	knik in σ-ε-diagram UGT: f_{pk}/γ_s (N/mm²)	knik in σ-ε-diagram BGT: $f_{p0.1k}/E_p$ (‰)
Y1030H	staaf	1030	35	834	758	751	709	3,70	936	4,07
Y1100H	staaf	1100	35	891	810	802	757	3,95	1000	4,35
Y1230H	staaf	1230	35	1082	984	974	920	4,80	1118	5,28
Y1570C	draad Ø > 8	1570	35	1303	1185	1173	1108	5,78	1427	6,36
Y1670C	draad Ø ≤ 8	1670	35	1436	1306	1293	1221	6,37	1518	7,01
Y1770C	draad Ø ≤ 8	1770	35	1522	1384	1370	1294	6,75	1609	7,43
Y1860C	draad Ø ≤ 8	1860	35	1600	1454	1440	1360	7,09	1691	7,80
Y1860S	streng	1860	35	1600	1454	1440	1360	7,46	1691	8,20
Y2060S	streng	2060	35	1772	1610	1594	1506	8,26	1873	9,09

Figure 2.32: Properties of prestressing steel. Source: GTB 2013, paragraph 17.1.a

A visual representation of the strength properties of the prestressing steel in the form of the stress-strain diagram, is shown in Figure 2.33. In this diagram, we take the horizontal line at $f_{pd} = f_{p0,1k}/\gamma_S$ as the assumed stress-strain development.

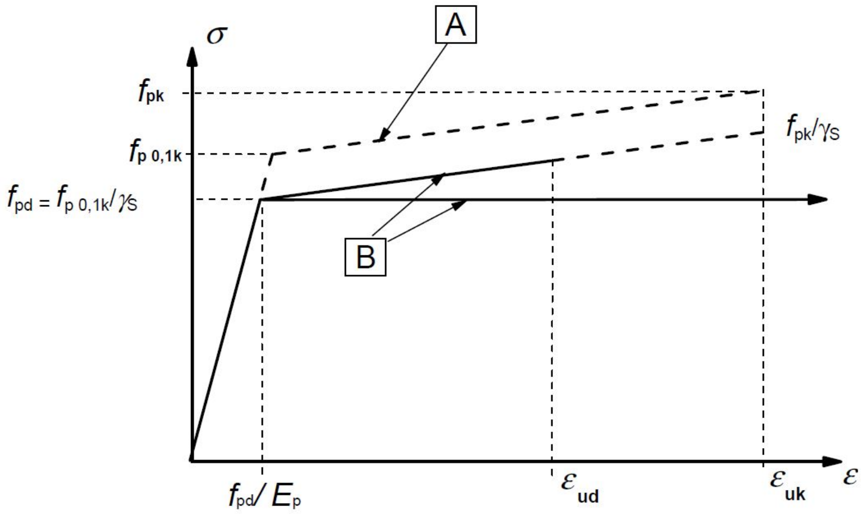


Figure 2.33: Stress-strain diagram for prestressing steel. Source: NEN-EN 1992-1-1, section 3.3.6 [19]

2.5.2. MATERIAL AMOUNTS

In the Grasshopper script, the amount of material will be determined automatically, by multiplying the length of the elements with the area (in case of beams) or by multiplying the surface area with the thickness (in case of slabs or plates). These material amounts are split-up per element, but also for the different concrete classes, since the environmental impact differs per concrete class. Also there is a separate material calculation for the main bridge. The division of concrete classes per element will be elaborated in the next paragraph.

Note that the materials described in this chapter are only the ones used in the program. So the different concrete classes, the reinforcement and prestressing steel, the asphalt and the soil for the road embankments.

Concrete classes

Since each element (prefab beam, cast in place bridge deck, foundation slab or pile) is made with another concrete class, there is performed an investigation into the most used strength class per element. This standard concrete class will be used in the parametric model.

In Table 2.3, an overview of the standard concrete classes or steel strength is given per structural element. This will be used in the next section on costs and in the following section on environmental impact.

Table 2.3: Standard material per element or bridge type. Source: Movares

Element type	**Concrete/Steel Strength**
Cast-in-situ	C30/37
Prefabricated beams	C60/75
Cantilever Bridge	≥C55/67
Substructure	C30/37
Foundation Piles	C55/67
Concrete Deck	C30/37
Reinforcement Steel	B500B
Prestressing Cables	Y1860S

The exact composition of the different concrete classes can be found in section 2.5.4.

2.5.3. COSTS

The costs of the bridge design will be determined per element and/or per volume (tons or m^3). The following prices per unit were taken into account. These prices are based on estimations for the preliminary design phase which are provided by Movares.

- Concrete: 100 Euro/m^3
- Reinforcement steel: 1200 Euro/ton
- Prestressing steel: 1750 Euro/ton
- Casting: 25-50 Euro/m^3
- Formwork: 25-250 Euro/m^2
- Asphalt: 110-150 Euro/m^3
- Soil: 15 Euro/m^3

As can be seen in the list above, the different concrete classes do not effect the material price, all concrete classes are around the 100 Euro/m^3.
The price for the prestressing anchors and spiral reinforcement is not included in the unit price of the prestressing steel, but a price of 700 Euro per "live end" and 350 Euro per "dead end" is taken into account. The "live end" is the end of the cable where the stressing with the hydraulic jack takes place, the other passive end where the cable is only connected to the concrete, is called the "dead end".
The casting price and the price for formwork differ quite a lot. Reason for this is the difficulty of the bridge element. For instance the casting of a balanced cantilever bridge above the water, requests a more complex method, compared to the casting of a foundation on land. Therefore a more specified overview of the costs per structural element can be found in Appendix E.
The price for asphalt depends on whether the asphalt package is on the soil embankment, here an extra foundation layer is needed, or on the superstructure where it can be directly applied.

For the calculation of the total price in the parametric model, there will be used unit prices (Euro/ton or Euro/m^3) for each element. To achieve a more detailed price for the balanced cantilever bridge, the prices for this bridge are split up as can be seen in Appendix E. For an overview of the unit prices, see Table 2.4.

With all prices and material amounts known, the price for the materials of the main bridge and the total design can automatically be determined and used as goal for the optimization.

Table 2.4: Unit prices per element, as implemented in the model

Element:	**Chosen Factor:**	**Units:**
Cantilever Bridge	splitted price	-
Hammerhead	650	€/m^3
Prefab beam	500	€/m^3
Deck layer	275	€/m^3
Capping beam	650	€/m^3
Column	650	€/m^3
Foundation slab	400	€/m^3
Foundation pile	150	€/m

NOTE: The costs as determined by the model are only material based, so transport of the material and construction of the bridge is not taken into account.

2.5.4. ENVIRONMENTAL IMPACT

Before the environmental impact of the materials will be determined, there will be an explanation about the necessity of including the environmental impact in projects. This will be done by stating a few historic events which influence the (Dutch) society and by giving the consequence of this events for this project.
Since a couple of years there is an increasing demand for sustainable and durable solutions, also in the construction industry. Also Climate Change, inducing sea-level rise and extreme weather conditions, caused by industrial pollution asks for serious measures. As an example of one of these measures there is the Paris Agreement.

The Paris Agreement

This Paris Agreement is signed by 194 countries in late 2015, with as main goal; to limit global warming to well below 2 °C and if possible only 1.5 °C. This increase in temperature is with respect to the global temperature before the industrialization. The limit of 2 °C is based on disastrous problems which will arise according to scientist when this 2 °C temperature rise (or worse) is reached. These consequences will not be further explained in this thesis.

So how does the Paris Agreement work? All parties have declared that they will cut on their greenhouse gas emissions, preferably in the 2020s. And each five years the ambitions to cut in their emissions will be revised progressively, in other words: the goal is to set the bar higher every time. Also there is every five years from 2023 a "global stock-take", this is a moment to reflect in all transparency on the accomplishments and expectations of each party. To make sure that every country will invest to their potential, in this agreement is stated that each country's pledge must “represent a progression” on their previous one “and reflect its highest possible ambition”. [4] Since developing countries do not have the (financial) ability to reach these goals on their own, they will be funded by the developed countries.

This Paris Agreement is also translated into national measures. An example of this measures is the signing of the Concrete Agreement (Betonakkoord) in 2018 by various organisations including the Directorate-General for Public Works and Water Management (Dutch: Rijkswaterstaat), but also Contractors and Engineering firms like Movares.
In this Concrete Agreement the various parties have agreed on two main goals, one is growing a more Circular Economy, the other one is limiting the carbon emissions. These two goals influence each other and for this project the choice is made to include both goals.
This is made concrete in the following way. Since prefabricated beams are easy to re-use this contributes to the Circular Economy in the highest possible way, that is why this construction type is chosen over the cast-in-place method for the approach bridges. With this construction method, only the material itself could be recycled or re-used. For the other goal, not only the carbon emission is investigated and determined but all environmental pollution factors are taken into account. This will be explained in the next section.

Environmental Costs Indicator

For the environmental impact of the materials the GWW-materials database will be used. (GWW stands for Grond-, Weg- en Waterwerken). In this database the materials and their environmental impacts are determined with help of the NMD (Nationale Milieu Database). Part of this database can be found in Appendix G.

In this project the impact will be determined by multiplying the material amounts with the environmental impact of the material. All eleven pollution factors are based on the complete life cycle of the materials, so from cradle to grave. Because it is quite difficult to compare the different materials on each of the eleven types of pollution, there is determined an "equivalent impact value" in Euro for each of the pollution factors. This value is based on, how easy it is to prevent or resolve the pollution and how harmful it is for the environment. All these values together are combined into the Environmental Costs Indicator value (ECI)[5], this is the so called "shadow price" per volume material.

So, the environmental impact in this project is calculated by determining the emissions of the material itself, so transport, construction method and the emissions by the equipment is not taken into account.

[4] www.telegraph.co.uk/business/0/paris-agreement-climate-change-everything-need-know/
[5] in Dutch: MKI = Milieu Kosten Indicator

Concrete Compositions
Since the environmental impact of concrete is mainly relying on the amount of cement, this cement usage is further investigated. Overall counts that the higher the concrete class, the higher the use of cement. But there is another aspect which is important here, namely the type of cement.

There are two main types of cement, CEMI and CEMIII. CEMIII has a lower impact compared to CEMI so this looks like the cement to use, but the exact composition of the concrete classes still has to be determined. This composition of the various concrete strength classes is based on real mixtures and can be found in Appendix F. For all mixtures the environment in which the concrete will be applied, is taken into account. Since the concrete will be used in an environment with high chance on wet conditions, frost and possible chlorides from de-icing salts, the used exposure class is XF4.
As an example, below there is a concrete composition for a concrete mixture C30/37, see Table 2.5.

Table 2.5: Composition of Concrete Class C30/37 Source: Heijmans

Compound:	kg/m3:	€/m3:
C30/37		ECI:
CEM I 52.5N	108	6.54728
Hoogovenslak	252	0.387912
Grind 4-22	343	0.028958
Grind 4-32	801	0.067625
Zand 0-4	778	0.354145
SP Sky 648 con 20%	1.71	0.157748
Bronwater	107	0
	sum:	7.54

Since there are many possibilities for picking a concrete mixture, the decision is made to use mean values for each concrete class. This mean values are based on the regression line trough the selection of the eleven example mixtures, see Figure 2.34.

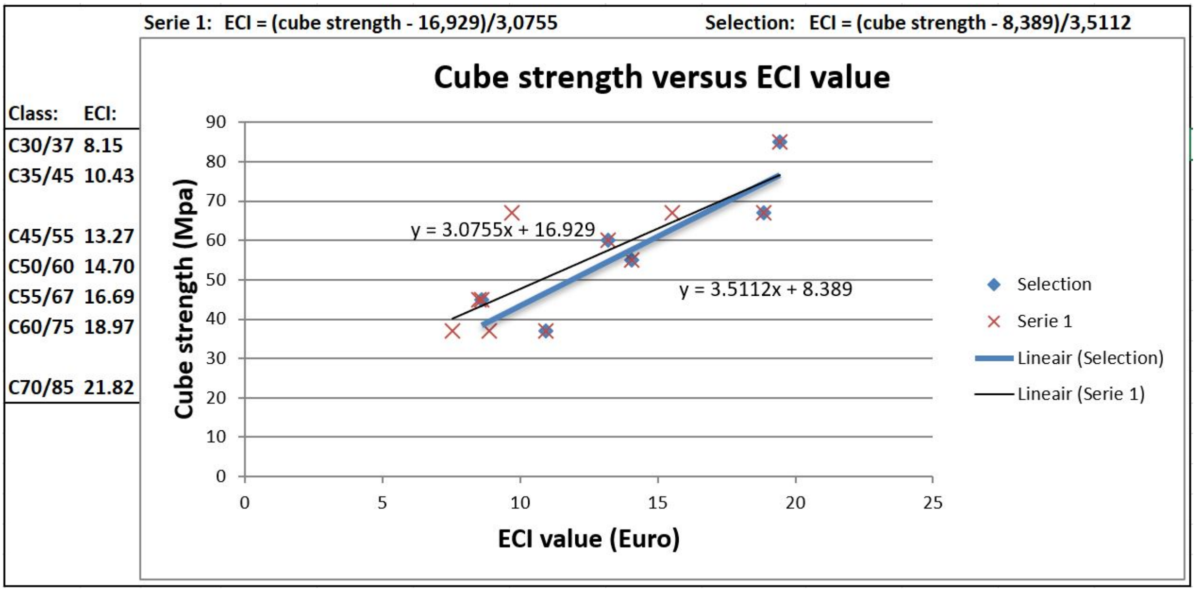


Figure 2.34: Regression line which determines the concrete environmental costs indicator

In the selection of mixes, only the higher ECI value per concrete class is used to draw the regression line. In the table on the left side of the Figure, the ECI values per concrete class can be found.
For the ECI of the prestressing anchors, the material amounts are estimated with help of the dimensions specified by the manufacturer.[20] This results in an ECI of 31 Euro per anchor.

2.6. LOAD MODELS & COMBINATIONS

Since the model should present structurally sound bridges, there must be an analysis and verification stage in which the bridges are loaded and checked. To determine the loads which have to be applied on the bridges, the Eurocode 1: Actions on structures - Part 2: Traffic loads on bridges[21] and the National Annex[22] are investigated. From this investigation follows that for a non-detailed bridge calculation it is sufficient to use only LM1 and LM4 (Load model 1 & 4). Since the project focus will be on the preliminary design, these two load models will be applied. Next to the traffic loads which are live loads, there is also the self-weight of the bridge superstructure and super imposed loads consisting of the asphalt layers, the safety barriers and other road elements. Other loads, for instance wind load, dynamic loads or horizontal (collision) loads will not be taken into account.

2.6.1. TRAFFIC LOADS

For the traffic loads there are four load models possible, from which in this project LM1 and LM4 will be applied. In the next paragraphs, these two load models will be described in detail. The other load models are; load model 2, consisting of a single axle load meant for semi-local and local verifications and load model 3, consisting of a set of special vehicles meant for general and local verifications. These will not be further explained in this thesis.

Load Model 1

Load model 1 is based on a Uniform Distributed Load (UDL) combined with Tandem System loads (TS), this load model represents the general loading due to lorries and cars. Since the position of the vehicles is not bounded by the width of the lanes, a system of notional lanes is defined. With this method, the whole carriageway will be divided into notional or theoretical lanes with a width of 3 meters. On these notional lanes there are two different loads present. The heavy lane is loaded by a UDL of 9,0 kN/m^2, the other notional lanes and remaining area of the carriageway are loaded with a UDL of 2,5 kN/m^2. So, on the most unfavourable notional lane, which is most of the time directly next to the safety barrier, there will be the highest UDL and tandem axle, with the largest eccentricity. For a schematic overview of the load model, see Figure 2.35.

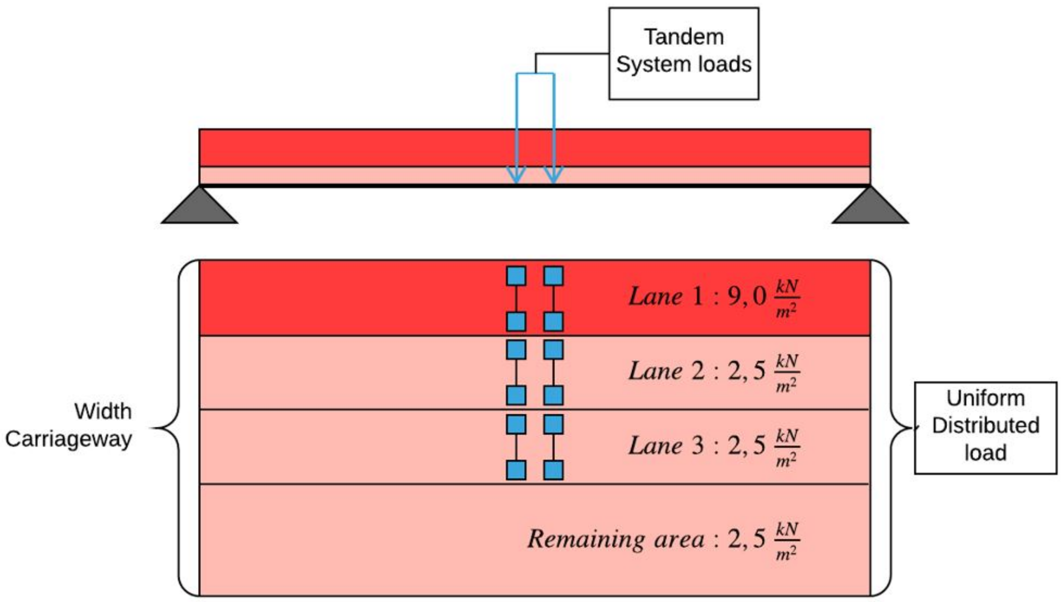


Figure 2.35: Notional lanes on bridge deck with characteristic loads and in green the axle loads. [10]

Since these loads are characteristic values, a factor α should be applied for the different lanes. For an overview of the application of these α factors see Table 2.6.

Location	Total Axle Load [kN]	UDL [kN/m^2]
Notional lane 1	2*α_{Q1}*300	α_{q1}*9,0
Notional lane 2	2*α_{Q2}*200	α_{q2}*2,5
Notional lane 3	2*α_{Q3}*100	α_{q3}*2,5
Other lanes	0	α_{qi}*2,5
Remaining area	0	α_{qr}*2,5

Table 2.6: Load set-up for the notional lanes,

In chapter 4.3.2 of the national appendix for this code, the values for the α_q factors are defined for roads with more than 3 notional lanes and 2 million trucks per year. Since the expectation is that the traffic amount will increase in the coming years, these factors will be used for all roads. So in this project α_{Qi}=1,0 , α_{q1}=1,15 and for i>1, α_{qi}=1,4.[22] This gives a UDL of 10,35 kN/m^2 for the heavy lane and 3,5 kN/m^2 for the remaining carriageway. The highest axle load will be placed in the heavy lane, to create an unfavourable effect. During the analysis, the tandem axle loads and the UDL are tested on multiple locations to find the most unfavourable situations.

Load Model 4
Load model 4 consists of a crowd load of 5 kN/m^2 over the full width of the bridge, so it might be that for wide bridges Load model 4 is governing compared to Load model 1. Therefore per individual bridge design, the governing load model is investigated.

Since the position of the traffic load does influence the moment distribution in the bridge, there are multiple load cases used in this project. Note that LM1 always is applied with the tandem axle loads in the middle of the distributed load domain.

- Load case 0: Only self-weight
- Load case 1: Superimposed Dead loads
- Load case 2: Whole bridge LM1 or LM4
- Load case 3: One side span LM1 or LM4
- Load case 4: Midspan LM1 or LM4
- Load case 5: Two side spans LM1 or LM4

After applying the different load cases, there can be concluded that the full traffic loading of the bridge has the most unfavourable effect. Therefore load case 2 is used as variable load for the analysis of the bridge in the parametric model.

2.6.2. SELF-WEIGHT AND SUPERIMPOSED DEAD LOADS

In this project, the loads for the self-weight of the concrete structures are automatically implemented, by adding the gravity. The superimposed dead loads consists of all additional loads which are added after the superstructure is finished. This are the asphalt deck, safety barriers and other road elements.
The asphalt deck is build-up out of two layers. The bottom layer is minimal 50 millimetres thick, water retaining asphalt (2500 kg/m^3), this protects the concrete slab for salts and other aggressive substances. On top of this layer is an open asphalt layer (ZOAB, 2150 kg/m^3) of minimal 70 millimetres thick, to decrease the risk for aquaplaning during heavy rainfall and to reduce noise.[17] The total asphalt package should be at least 140 millimetres and maximal 170 millimetres thick, this is depending on the length of the bridge span and its deflection.
Since the weight per meter safety barrier is around 45 kg/m, this will not be taken into account. Also other small loads from for instance street lights will not be taken into account for the structural analysis of the bridge. Instead, the asphalt package is assumed over the full width of the bridge.

2.6.3. LOAD COMBINATIONS

Ultimate Limit State

For the load combinations Eurocode 0: Basis of Structural Design section 6.4.3.2 is used. In this Eurocode, the following load combinations for Ultimate Limit State (ULS) and Serviceability Limit State (SLS) are found in chapter 6.4 and 6.5.[23]

$$SLS: \sum_{j\geq1} G_{k,j} + P + Q_{k,1} + \sum_{i>1} \psi_{0,i} * Q_{k,i} \tag{2.3}$$

$$ULS_1: \sum_{j\geq1} \gamma_{G,j} * G_{k,j} + \gamma_P * P + \gamma_{Q,1} * \psi_{0,1} * Q_{k,1} + \sum_{i>1} \gamma_{Q,i} * \psi_{0,i} * Q_{k,i} \tag{2.4}$$

$$ULS_2: \sum_{j\geq1} \xi_j * \gamma_{G,j} * G_{k,j} + \gamma_P * P + \gamma_{Q,1} * Q_{k,1} + \sum_{i>1} \gamma_{Q,i} * \psi_{0,i} * Q_{k,i} \tag{2.5}$$

Since the infrastructural works for main roads have a high risk for loss of human lives, Consequence Class 3 (CC3) will be applied. This determines the value for the γ factors in the load combinations. $\psi_0 = 0{,}8$, $\gamma_{G,j} = 1{,}4$, $\xi_j * \gamma_{G,j} = 1{,}25$, $\gamma_{Q,1} = 1{,}5$ and $\gamma_{Q,i} = 1{,}65$, from National Annex to Eurocode 0: Basis of structural design Tabel NB.9 – A2.1 and Tabel NB.13 – A2.4(B).[24] These factors result in the following equations for concrete bridges in ULS. In these equations G is the permanent load consisting of the self-weight and the superimposed dead loads and P is the load due to prestressing. The variable load Q is already filled in with the Tandem System (TS) and the Uniformly Distributed Load (UDL).

$$SLS: G + P + (TS + UDL) \tag{2.6}$$

$$ULS_1: 1{,}4 * G + P + 1{,}2 * (TS + UDL) \tag{2.7}$$

$$ULS_2: 1{,}25 * G + P + 1{,}5 * (TS + UDL) \tag{2.8}$$

Serviceability Limit State

In the Serviceability Limit State (SLS), the structure will be checked on the maximum deformation due to the variable load only. The assumption is made here, that the deflections due to permanent loads are already compromised for during the construction of the bridge. In other words, the bridge will be build with a small upwards camber to be able to neglect the deflections due to the self-weight and superimposed dead loads.

The SLS verification check for the deformation has to satisfy the requirement that the maximum deflection of the bridge should not exceed the $\frac{L_{span}}{500}$. This check will be performed during the verification phase.

2.7. PARAMETRIC DESIGN SOFTWARE

After researching multiple software programs and plug-ins during the initial literature study, the decision is made to use Rhino with Grasshopper as the central player and Karamba for the structural analysis in this project. The choice for this software package is made, based on; personal experience, a reference master thesis by Edward Fransen, also performed at Movares[11] and the enormous availability of (free) plug-ins. Also the thriving community behind Grasshopper, was part of the reason behind this choice. Due to the large amount of students and others using Grasshopper, issues are quickly resolved and solutions are easily shared on the forum.
In this chapter, the software package and plug-ins will be explained shortly and there will be given a scheme to create a more visual understanding of how the different programs and plug-ins interact with each other.

Figure 2.36: Rhinoceros logo, Source: www.rhino3d.com

Rhino
Rhino (Figure 2.36) is the three-dimensional CAD environment in which the visualization is modelled or baked. For this project, the modelling will be done by the parametric add-on Grasshopper.

Grasshopper
Grasshopper (Figure 2.37) is an algorithm editor add-on for Rhino, with this plug-in it is possible to create geometry, read different types of files, process data, perform operations and much more. The plug-in works with "Parameters" and "components" or operating blocks, and "wires" or "connectors", which together form a definition. These definitions are followed by the program from left to right, or upstream to downstream. Changes in the upstream processes are directly updated in the downstream components. And simultaneously, the possible visualizations are updated in the Rhino viewport.
Since Grasshopper uses a visual programming language, it is easy to understand for beginners and the building of definitions is an intuitive job. There is no need to learn "old" program language, although it is possible to script your own components with use of Python, Visual Basic or C#.

Figure 2.37: Grasshopper logo, Source: www.grasshopper3d.com

2.7.1. PLUG-INS DESCRIPTION

There are numerous plug-ins and applications available for Grasshopper or Rhino, just a slight selection of them will be used in this project. Below there will be a brief description of the plug-ins used in this project.

Karamba
Karamba (Figure 2.38) is a parametric Finite Element plug-in for Grasshopper, suitable for the early design phase. Since the plug-in is fully embedded into Grasshopper, it works together easily with other plug-ins and optimization processes. The way in which Karamba operates is simply summarized in the following 6 steps[25]:

- **Create** a wire-frame or points with Grasshopper
- **Convert** the lines and meshes into beams and shells
- **Define** the support conditions and loads
- **Assemble** the model with elements, cross-sections, materials and connections
- **Analyze** the model for the chosen load cases and analysis types
- **View** the results from the structural analysis, but also the total weight and the utilization rate

The geometry from Grasshopper which is mainly lines and surfaces has to be converted into Karamba geometry, this will be done with the "line-to-beam" component or the "index-to-beam" component. These components create a cross-section for the specific line or index, so creating a three-dimensional element. The cross-sections can be user defined, based on standard shapes, but it is also possible to read-in cross-sections from a CSV-file or chose one from the standard library.

Figure 2.38: Karamba logo, Source: www.karamba3d.com

For the materials, steel is the default but also concrete is included in the plug-in. The default concrete class is C25/30, other classes are also available or can be defined by supplying the mechanical properties.

One of the problems for this project will be to translate the beam or element forces into cross-section reinforcement, this is not included in Karamba. The use of prestressing can be done by applying an initial strain to the beams resulting in a normal force or just by adding point loads at the outer ends of the beams. Other load types are self-weight, point loads, line loads and surface loads.
The standard connections between elements in Karamba are set to rigid, but each supporting or connection point can be adapted for six degrees of freedom.

As soon as the model is assembled and all te elements are correctly connected, the analysis can be performed. For this project a static analysis for small deflections will be performed, which takes into account only first order theory. Note that this type of analysis, does not take the deformed structure into account, so it might be an underestimation of the real force distribution. Because of the limited time which is available for this project, other analysis will not be performed.

Once the analysis has finished, the results can be made visual in different ways. There is the model-view, in which the deformations of the model are visualized. Another option is the beam-view, this presents the stresses distribution and the performance of each element resulting in an utilization rate. When a beam has a utilization rate of more than 100 % it is likely to fail. As an extra safety, it can be decided to accept only utilization below the 80 %. The results can be visualized with different colour gradients or numerical values. The self-weight of the model is expressed in kilograms.

To conclude, an overview of the pro's and con's for using Karamba in this project.

Pro:

- Karamba is fully integrated within Grasshopper
- Visualization of results is clearly visible in combination with a legend
- Three-dimensional calculations are easily performed in Karamba
- Every adjustment in the design is immediately adapted in Karamba

Con:

- The force/moment results for shells are quite difficult to obtain
- The accuracy of Karamba is lower compared to traditional FEM packages
- For prestressed or reinforced concrete the model is quite complex to make
- When building a large model, Karamba will take a lot of time for performing calculations
- Cross-sections are not visible as solids in Karamba
- It is not possible to create each cross-sectional shape

Since the model will be applied during the preliminary design phase and there are a few disadvantages as described above, for example the three-dimensional modelling of the main bridge which is quite complex. Therefore the choice is made to design the cantilever bridge in two-dimensions instead of three-dimensions. This calculation and analysis will be explained in detail in Chapter 3.

Galapagos, Colibri & Design Explorer
Galapagos is the optimization plug-in which will be used in this project. It uses an evolutionary algorithm to solve complex problems with many variables and constraints. A more detailed explanation of this can be found in Section 2.8, where the principal of evolutionary solving and the working of Galapagos is explained. The results from Galapagos can be visualized by uploading the data via Colibri (another Grasshopper plug-in) to an online tool called Design Explorer. This tool creates genome graphs, in which all the solutions are drawn.

2.7.2. Interaction Schemes

Below there are the visualizations of the different stages with their software packages and the mutual interaction between the programs, see Figure 2.39. Not all the programs or plug-ins from the scheme will be used in this project, but this will give an insight in the possibilities for further development of the model and the position of Grasshopper as central player. The yellow outlined shapes are the ones used in this project.
There are three basic steps in the model, one the Design phase which is at the start user defined and later optimized in an iterative process, two the Analysis and Verification of the design which is linked back to the Design phase and step three the Visualization of the design which is the final phase. Since the Optimization phase gives the new input for the design, it is included in the Design phase. All processes are monitored in Grasshopper which is the central player in this scheme.

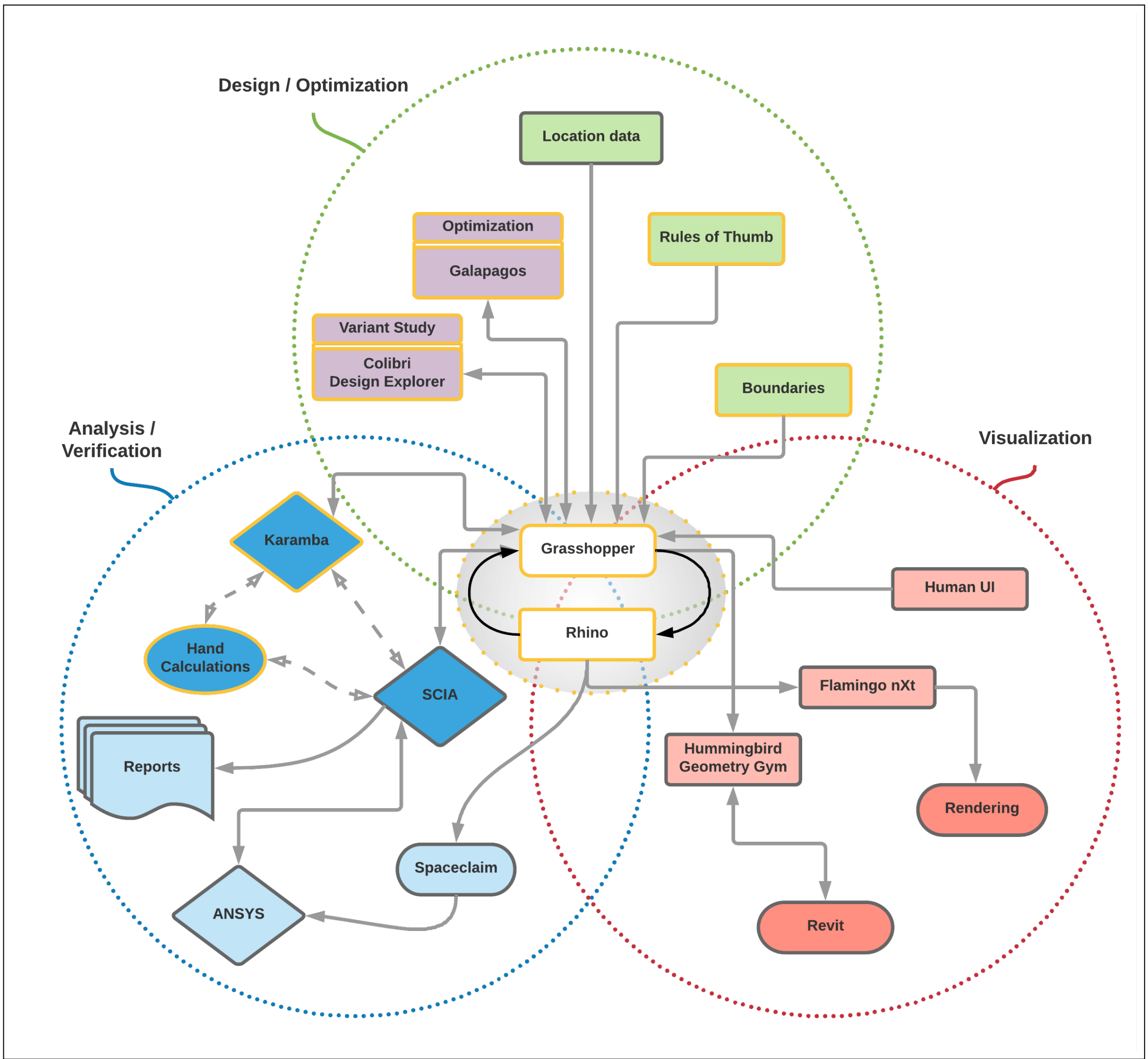


Figure 2.39: Software interaction scheme

2.8. OPTIMIZATION

In this section, the Grasshopper plug-in Galapagos and the principle of evolutionary solving will be described in detail. Subsequently, the bridge optimization process will be explained in Chapter 5. There the goal and the optimization parameters for this project will be described. After that, the results of this optimization process will be described in Chapter 6.

2.8.1. EVOLUTIONARY OPTIMIZING WITH GALAPAGOS

Since optimization of a complex problem with multiple variables takes a lot of time when all possible solutions must be calculated (brute force method). There is need for a smart solver that uses the input from previously calculated options, as extra information for the following options.
In Grasshopper there is such a solver, called Galapagos (Figure 2.40), which makes use of the evolutionary algorithms to solve complex problems. In this section, first the pros and cons will be explained followed by a description of the steps taken by Galapagos and the underlying process.[26]

Figure 2.40: Galapagos Evolutionary Solver, Grasshopper component by David Rutten

Pros and Cons

An advantages of using an evolutionary solver like Galapagos is the high applicability for various over- or under-constrained problems where other methods of solving, for example Generalized Reduced Gradient (GRG) Non-linear, do not longer apply. GRG Non-linear solvers use the method of finding the optimum value by searching for the derivative equal to zero. This might be a local optimum and possibly not even close to the global optimum.[27] Another advantage is the possibility to extract intermediate results from the optimization process. This could be done by simply interrupting the process (this might not give very good results), by setting a certain threshold for the results, or by setting a maximum number of iterations. In general it holds, that the later in the process, the higher the quality of the result. So the combinations of variables or genomes will improve themselves in search for the peak solution. Furthermore, Galapagos allows the user to interact with the optimization process by using predefined start values and boundaries for the variables. In this way, the user can force the optimization in a certain direction, this may be useful to investigate the influence of a certain parameter.
One of the disadvantages of such a solver is the fact that they take a lot of time to solve a problem. Also it does not guarantee a real solution or result, unless intermediate results above a certain threshold are accepted as well.

Process

The process of Galapagos will be described with the use of Darwinian principles (survival of the fittest) in the following example. There are two variables or "genes". When all possible solutions with these genes are combined, they give a solution which can be presented as a landscape in three-dimensional shape. In this case, the fitness of the solution is presented by its height. If this theory is further elaborated, a problem with 10 variables will create a 11-dimensional landscape which is quite hard to understand. Therefore, the example will have only two variables to keep it simple, the parametric model will probably have more than two variables.

Since the solver does not know the shape of the landscape, the first step is to create a random distribution of solutions by using random combinations for the two variables. These solutions all have an own performance, the better the performance, the higher their position in the landscape. Now to further optimize the results as a second step, the worst solutions are killed and there will be searched around the better solutions, or in terms of Darwinism; the children of the stronger parents. By repeating these steps for a couple of times, eventually the peaks in the landscape are found. See Figure 2.41 for a visualization of the optimization process.

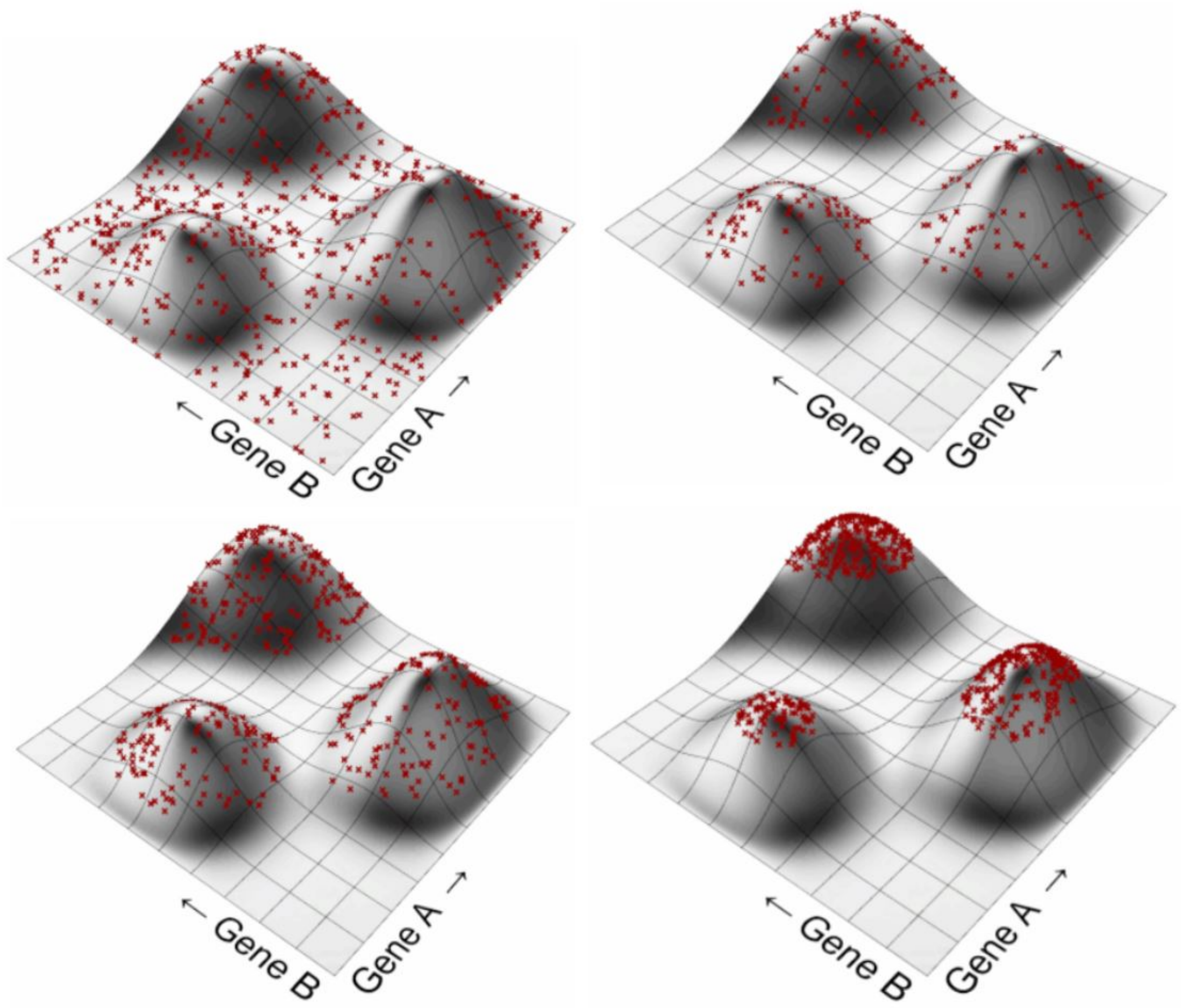


Figure 2.41: Fitness Landscapes with optimization process visualized[26]

The question is, how does the solver know which values to discard and which to further investigate? This will be explained by describing the following terms.

The first one is the **fitness function**. This is a visualization of all solutions, based on their performance on the required result (for instance, project costs). For the example above this was a rather easy understandable landscape, but there might be more difficult functions. There are fitness functions which have many local optima, so the real optimum is less easy to find since the solver will evolve around those local peaks. Or the fitness function might have a noisy or distorted surface with many small peaks, here the solver has too many options so the process will take a long time. See Figure 2.42.

The second one is the **selection mechanism**, how to determine which results are fit enough to keep and which results have a high potential for even better results.
There are three selection possibilities. The isotropic selection; each result will be further investigated, no matter how fit. The exclusive selection; only the best X% will be further looked into. Or the biased selection; the higher the result, the more investigation around these results is performed. Galapagos is able to perform all these kind of selection mechanisms.

The third one is the **coupling algorithm**, this determines which two genomes are combined to form a new genome in the optimization process. There are multiple options, there can be made a combination with a genome which is close-by, so almost similar. This presumably gives also an almost similar result. Or there can be made combinations with genomes further away, which are more different. This option has the risk

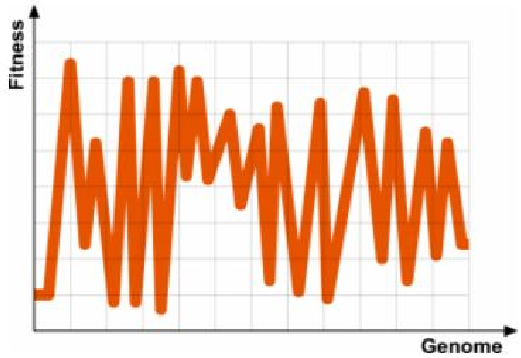


Figure 2.42: Fitness Landscapes in 2-D with many highs and lows close together (distortion) [26]

of landing somewhere in between two peaks. Therefore the best option would be somewhere in between, this can be specified in Galapagos by giving an in-breeding factor which provides the coupling area. For a visualization of the coupling area, see Figure 2.43. Note that in this coupling algorithm of Galapagos, the results or fitness of the specific genomes is not taken into account. This might be developed further in the future.

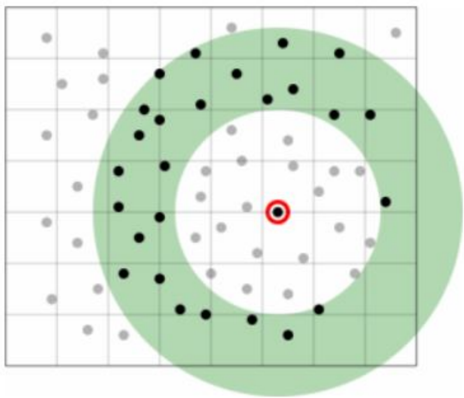


Figure 2.43: Coupling area for the genomes in green.[26]

The fourth one is the **coalescence algorithm**, which determines the values for the new formed genomes. One of the possibilities is cross-over coalescence, here the new genome has part of the values from the earlier used genome A and part of the values from genome B. So all parameters keep the same value, but the combinations are different. Another possibility is blend coalescence, this creates new genomes with values in between the values from the earlier used genomes. It is also possible to use the fitness of the specific genomes to create a more guided blend coalescence, so the new genome will be closer to the fit genome. As a visual support, see Figure 2.44.

The fifth term is **mutation factories**, with this mutation it is possible to manually increase the diversity in solutions. The only way of performing mutations in Galapagos is by point mutation, this is basically changing one of the parameters to create another solution. A simple way of projecting the used genomes and the different solutions, is by drawing "genome graphs" or "parallel coordinates plots" (PCP). In the graphs there is for each variable a vertical axis and each genome forms a line through the specific variable values towards the result on the right of the graph, see Figure 2.45. In this way an clear overview of all the results and the effect of each variable is given.

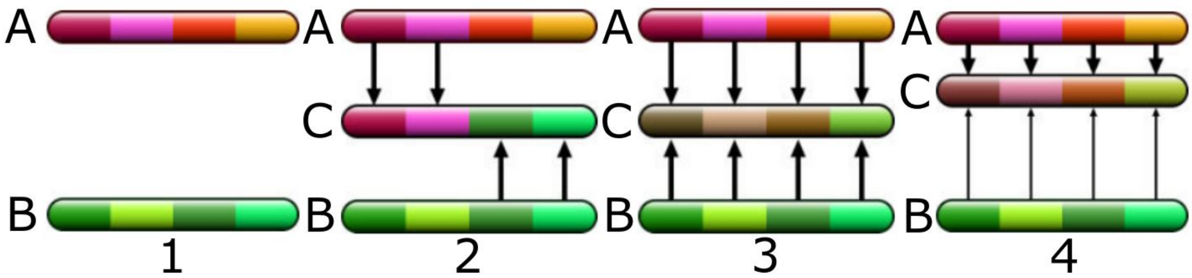


Figure 2.44: Coalescence Algorithm visualized. From left to right; 1. first generation genomes A and B, 2. forming of genome C with use of cross-over coalescence, 3. forming of genome C with blend coalescence and 4. use of preference based blend coalescence to form genome C.[26]

These genome graphs or parallel coordinates plot can be made with Design Explorer, an online open source tool developed by Thornton Tomasetti, CORE studio. [6] It is also possible in Design Explorer to select part of the solutions by adapting the range of the specific vertical axis.
To upload the data from Grasshopper to Design Explorer, another plug-in called Colibri is used.

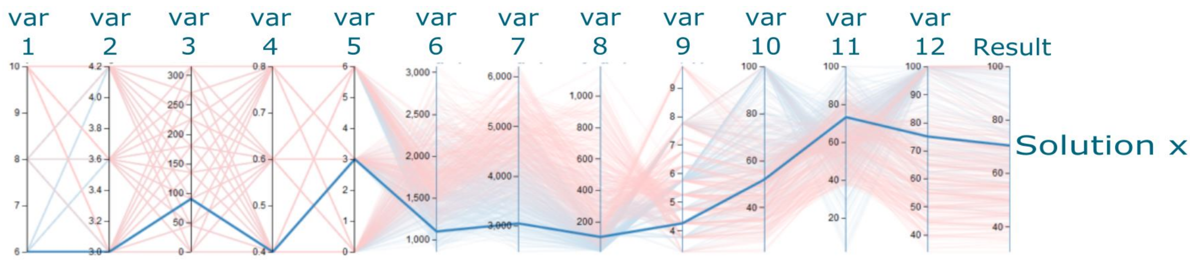


Figure 2.45: Genome graphs with parameters on the vertical axis and the result on the right. Source: Design Explorer | CORE studio http://core.thorntontomasetti.com/design-explorer/

In Galapagos all these principles are used in the optimization process.

Colibri
The plug-in Colibri, Figure 2.46 is able to transfer the data from the free sliders and the output into a CSV-file which can be opened with Design Explorer. This plug-in works with two main components, an "Iterator" and an "Aggregator". The Iterator gathers all input parameters, loops them over for each iteration and transfers the data to the Aggregator. This component collects all data from the Grasshopper script which is needed for Design Explorer and processes this in the CSV-file. The result of this process can be seen in Figure 2.45 and in Chapter 5, the project specific information is described.

Figure 2.46: Colibri, Grasshopper plug-in by CORE studio | Thornton Tomasetti

[6] http://mebd.azurewebsites.net/DesignExplorer/

3

Bridge Design

In this chapter, the approach of the project will be further explained, in this case the structural designs of the bridge types will be translated into parametric designs. For this parametric design, it is important to know which variables are standard or predefined and which are free, so changeable during the optimization process. It is important to limit the amount of free variables, so the optimization process will not become too complex.
The description in this Chapter will consist of the bridge geometry and other elements, the analysis and verification phase. The description will be supported by figures and graphs. After describing the different bridge types and substructure elements, the parametric model itself will be explained in Chapter 4. The optimization processes and the model results are described in Chapter 5.
The subquestions for this chapter are:

3.1 How to translate the structural designs into parametric designs?
3.2 What are the important parameters for the design of concrete bridges?
3.3 What analyses should be performed by the model?
3.4 What are the important requirements for the preliminary bridge design?

3.1. FROM STRUCTURAL DESIGN TO PARAMETRIC DESIGN

To be able to design parametric bridges, an investigation is performed on the concept of the bridge types and the important parameters in these concepts. As a result, the design of each element in the model will be described in detail.
The parametric model will be split into different parts, namely; the main bridge, approach bridge, substructure and the foundation. Therefore the same division is used in this chapter. After this section there will be an overview of all the parameters which are variable (so can be optimized), the parameters which are set by the user (not optimizable) and the parameters which are predefined in the model.

3.1.1. MAIN BRIDGE - BALANCED CANTILEVER BRIDGE

The main bridge in this project is represented by the balanced cantilever bridge. For this bridge, first the parametric geometry will be explained followed by the cross-sectional shape and the parameters which are important for this bridge type. The hammerhead piers and other support structures are not taken into account for the model of the main bridge.

Bridge Geometry

The geometry of the bridge is build up with a 2.5 degrees function for the height under the bridge and the thickness of the bottom flange, see Equations 3.1 & 3.4.[28] In these formulas, the shape of the bottom flange of the bridge will be described for one cantilevering span of the total main bridge. Since the bridge is symmetric, this is enough to design the shape of the whole bridge by mirroring the quarter a couple of times. This can also be done by changing "x" in the formulas into "(x-L)" for the midspan and into "(x-2L)" for the other side span.

$$H_{bottom,z}(x) = a_h * 10^{-6} x^{2.5} + c \tag{3.1}$$

In this formula a_h is a parameter which determines the steepness of the curve, c is the vertical clearance under the bridge and x has a horizontal domain from 0 to 400 (in this example the length of the main bridge). The steepness parameter, a_h, is depending on the length of the main span, the height at the support and the height at midspan, in the following way, see Equation 3.2.

$$a_h = \frac{(h_{midspan} - h_{hammerhead}) * 10^6}{\left(\frac{L_{span}}{2}\right)^{2.5}} \tag{3.2}$$

The same formula can be used for the thickness of the bottom flange, here the parameter a_t is depending on the length of the main span, the thickness of the slab at the ends and at the support, see Equation 3.3.

$$a_t = \frac{(t_{hammerhead} - t_{midspan}) * 10^6}{\left(\frac{L_{span}}{2}\right)^{2.5}} \tag{3.3}$$

$$t_{bot.flange}(x) = a_t * 10^{-6} x^{2.5} + t_{midspan} \tag{3.4}$$

These equations are first written in Excel and later translated to Grasshopper and are fully parametric, so they will change with the boundaries of the bridge.

To make it easier to understand, the mentioned formulas are made around a realistic but fictional case in which a bridge with a span of 200 meters is designed. See Figure 3.1 for the shape of the bridge with a clearance of 12 metres, a height at midspan of 2,5 metres and a height at the hammerhead of 10 metres. For the thickness of the bottom flange in a quarter of the bridge, with a gradient from 1,0 metre at the hammerhead, to 0,3 metre at the end, see Figure 3.2.

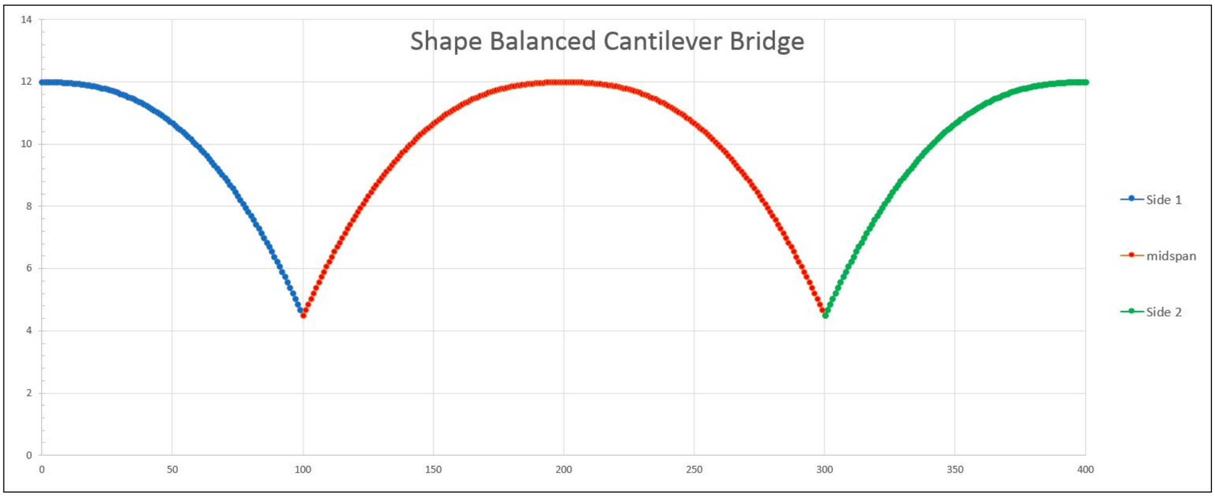


Figure 3.1: Excel graph for the shape of the balanced cantilever bridge (200 m span).

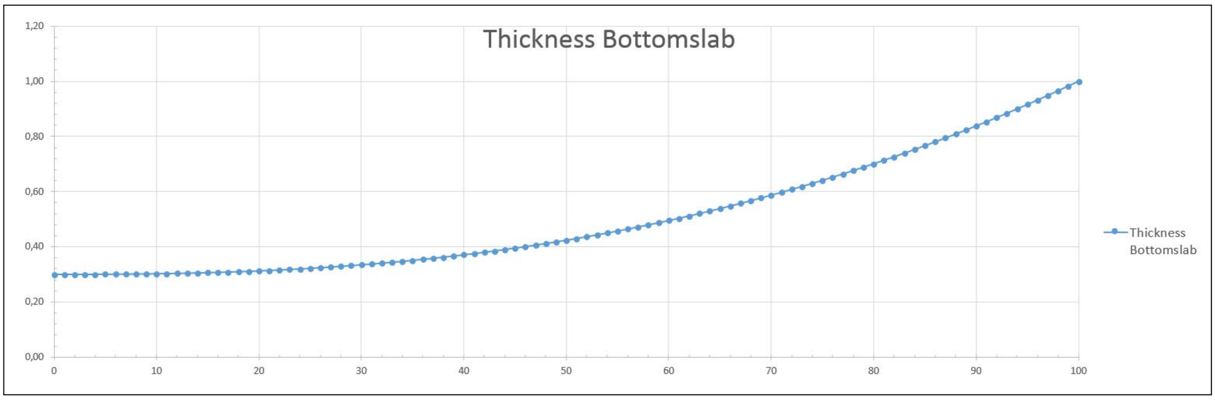


Figure 3.2: Excel graph for the thickness of the bridge's bottom flange. (only for one side span of the bridge)

In reality, the bridge will be cast-in-place in segments from 3-5 m length, depending on the height of the cross-section. Reason for this is, that the segments are limited by the carrying capacity of the expansion wagon which is used by building the bridge segments.
In this project, the length of the segments will be constant, so the carrying capacity will be neglected. Also the calculations are not based on segments but on a standard distance dx. So it will be a more theoretical solution.

The bridge will be modelled in 2D, since it is too complicated to model the bridge in 3D combined with prestressing parametrically in Karamba. This will also be too detailed for the application of the model, namely during the preliminary design phase. About the analysis of the bridge, more can be read in Section 3.3.

The bridge will be designed with straight prestressing tendons in the deck flange over the full length, this is the "cantilever prestressing". At midspan and at the outer ends, there can still appear tension in the bottom flange, therefore another type of prestressing is applied there called "continuity prestressing". These tendons will also be applied in a straight line at a fixed distance from the top of the bridge. For an overview of the location of the prestressing, see Figure 3.3. It must be said that in reality the applied prestressing is not fully optimized along the length of the bridge since it is not expedient to change the amount of strands or tendons every meter. In reality a standard of two tendons per web is taken as minimum amount, but it is possible to change the amount of strands per tendon.

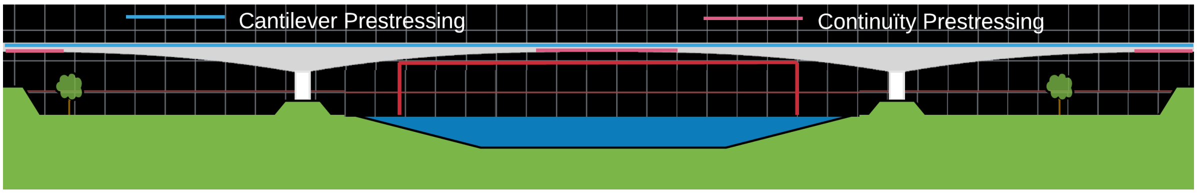


Figure 3.3: Balanced cantilever bridge with the blue line for the cantilever prestressing and the pink lines for the continuity prestressing.

Cross-section

The cross-section of the cantilever bridge will be simplified to ease the modelling of the bridge geometry in Grasshopper and Karamba. This holds that the haunches, the sloping parts in the cross-section, are not taken into account. So only straight box bridges are used, see Figure 3.4 & 3.5. These choice should be verified by comparing the Simplified (straight) box girder with a Real box girder on the cross-section area (A), the section modulus (W) and the second moment of area (I). When these values do not differ significantly, the choice for straight boxes is allowed.

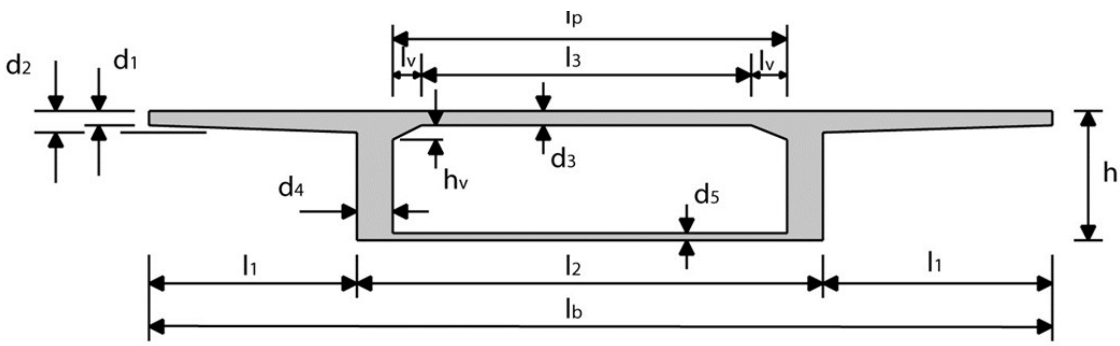


Figure 3.4: Cross-section for the balanced cantilever bridge [10], see table below for the values of the cross-section parameters.

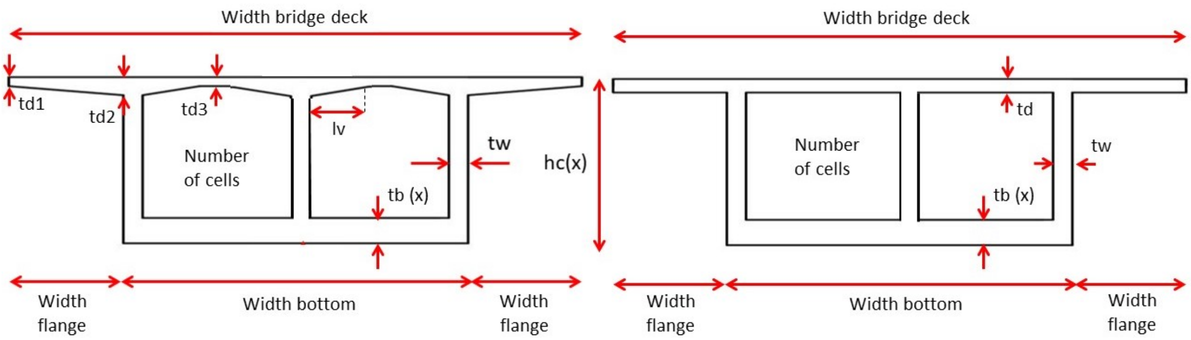


Figure 3.5: From real cross-section design to simplified straight design for modelling in Grasshopper/Karamba.

As can be seen in Table 3.1, the difference between the real box and the simplified box at the location of the hammerhead are quite small (around 4%), therefore the expected difference in outcome for the analysis of the bridge can be neglected. Since it might be that for other, for instance smaller cross sections, this difference will be slightly bigger, the decisions is made to check all cross-sections in a quarter of the bridge. These cross-sections are compared in Excel on physical properties as well as stress differences. An overview of the difference in stress between the simplified model and the real model is visualized in Figure 3.6.

Table 3.1: cross-section comparison at location of the hammerhead

Rules of thumb	**Real Box**	**Simplified Box**	**Difference**
w_b=by number of lanes	w_b=15,7 m	w_b=15,7	-
l_{span}/h = 22	$h_{support} = 200/22 \approx 10m$	$h_{support} \approx 10m$	-
$l_1 \approx$ 2 - 3.5 m	l_1=3,5 m	l_1=3,5 m	-
$l_2 \approx$ 5 - 7 m	l_2=8,7 m	l_2=8,7	-
$l_v/l_p \leq 0.2$	l_v=1,5 m l_p=l_b-2*(l_1+d_4)=7,7 m	l_v=0 l_p=7,7 m	(no haunches)
h_v=l_v/2	h_v=0,75 m	h_v=0	(no haunches)
$d_1 \geq$ 0.2 m	d_1=0,20 m	d_1=0,35 m	(straight deck slab)
$d_1/d_2 \approx 0.5$	d_2=0,40 m	d_2=0,35 m	(straight deck slab)
$d_3 > l_p/30 > 0.25m$	d_3=0,25 m	d_3=0.35 m	-
$d_4 \geq 0.35m$	d_4=0,50 m	d_4=0,50 m	-
$d_5 \geq 0.15m$	$d_{5,support}$=1,0 m	$d_{5,support}$=1,0 m	-
Cross-Sectional Area	22,850 m^2	22,110 m^2	3%
Second Moment of Area	348,9 m^4	335,7 m^4	4%
Section Modulus (top)	62,3 m^3	60,0 m^3	4%
Section Modulus (bottom)	79,4 m^3	76,3 m^3	4%

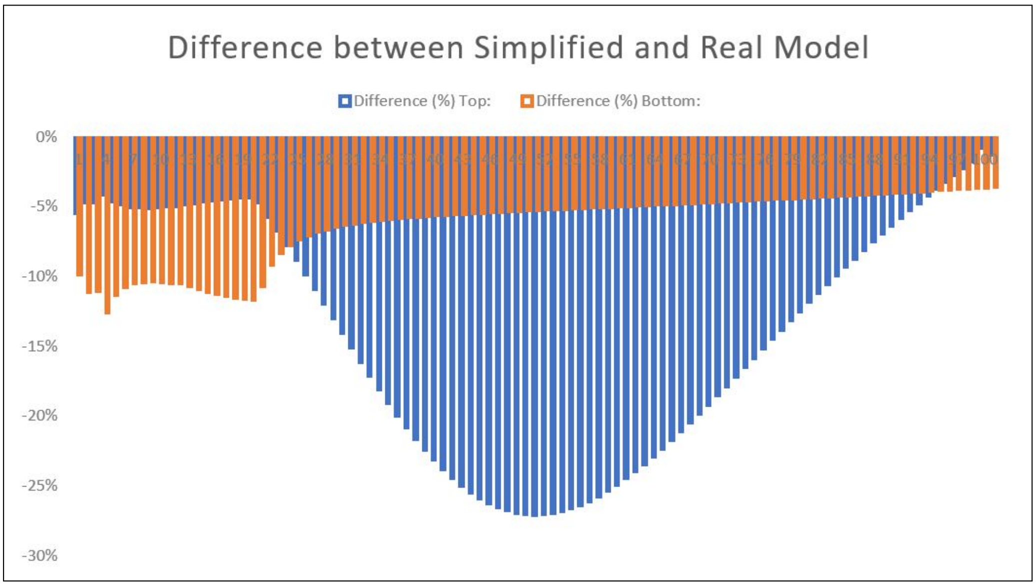


Figure 3.6: Stress difference between the simplified and the real cross-section

As can be seen in Figure 3.6, the stress difference reaches a maximum of -26 % in the middle of the bridge quarter. Since this difference is negative, which means that the simplified model has higher compressive stresses compared to the real bridge, the choice for the simplified model is a conservative one. So most of the time, the stresses in the real bridge will be lower compared to stresses in the model. This should be checked in the phases following after the preliminary design phase.

Parameters

For the cantilever bridge, there are two free variables and the rest of the input parameters are predefined. The free variables are the construction height and the thickness of the bottom flange. These variables both have two input values, namely the values at the hammerhead and at midspan. The intermediate values are determined with help of the earlier described formulas. During the optimization process, these four input values are varied until the most optimal design is found.

3.1.2. APPROACH BRIDGES

The structural model of the approach bridges consist of one span including the prefabricated beams with a deck slab and two capping beams. The capping beams are only used in the model to achieve the right physical boundaries for the analysis of the prefabricated beams.
For the structural design of the prefabricated beams, the description by the manufacturer will be used, in this project Spanbeton. These designs are translated into parameters with help of an Excel table in which all the dimensions per beam type are specified, see Appendix C. This Excel table is also translated into Grasshopper.

INVERTED T- & I-BEAMS

Geometry & Physical properties

The ZIPXL girders are build up as straight lines with standardized cross-sections. The width of the girders is standard 1480 millimetres wide and the bottom flange has two different thickness's, one for the profiles up-to ZIPXL 900 and another for the profiles from ZIPXL 1000 up-to ZIPXL 2400. For the higher profiles there is an additional top flange which forms the compression zone, this reduces the thickness of the deck (200 mm instead of 230 mm). The thickness of the web also differs, from 400 for the small profiles to 250 for the higher profiles. For an overview of the used profiles, see the document as provided by the manufacturer in Appendix B.

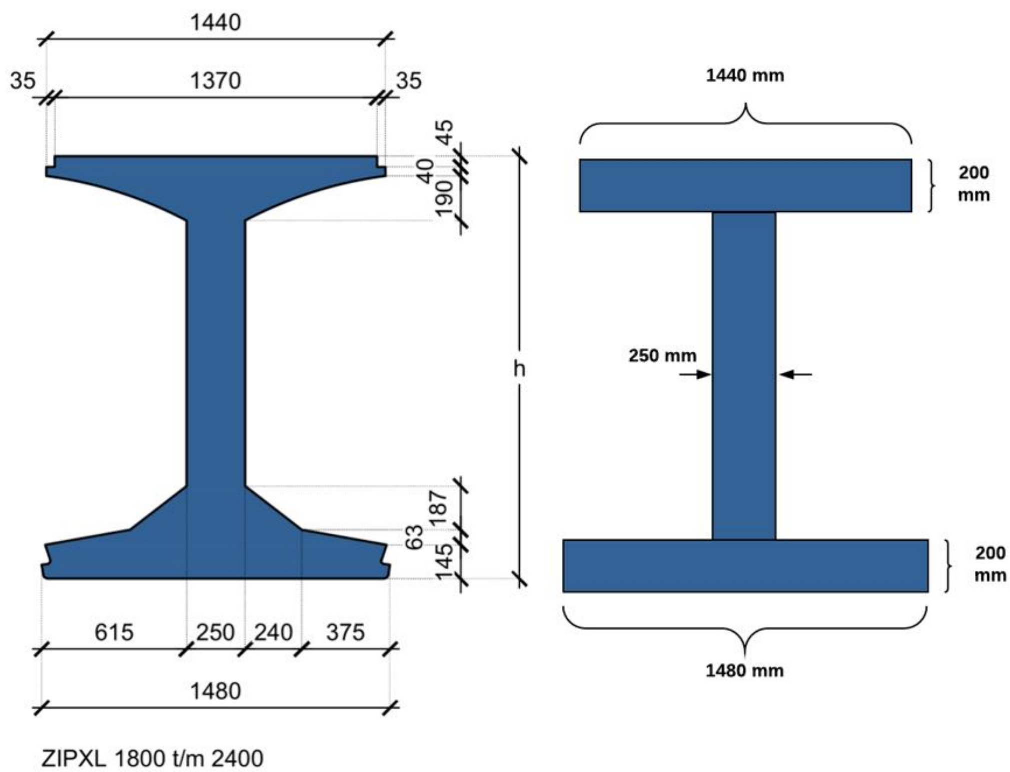


Figure 3.7: Cross-sections ZIPXL beams for the approach bridge, real and simplified. Source: Spanbeton

Since the profile geometry of the prefabricated beams is quite complicated these cross-sections will be simplified to normal T- and I-shapes, see Figure 3.7. In this way the physical properties of the beams will differ from the ones specified by the manufacturer, but this will be compensated by changing the cross-sectional area, moment of inertia and the resistance to the factory specifications. This can be done with help of the "Modify Cross-section" component from Karamba, see Figure 3.8. With this component, it is possible to replace certain cross-section values with self defined values. As a result of this modification, the beam will act as the one specified by the manufacturer, but will have the simple visual dimensions as specified by the author. The profiles are selected automatically per span length with help of rules of thumb. These rules of thumb are set after trying out different settings, but can be changed by the user.

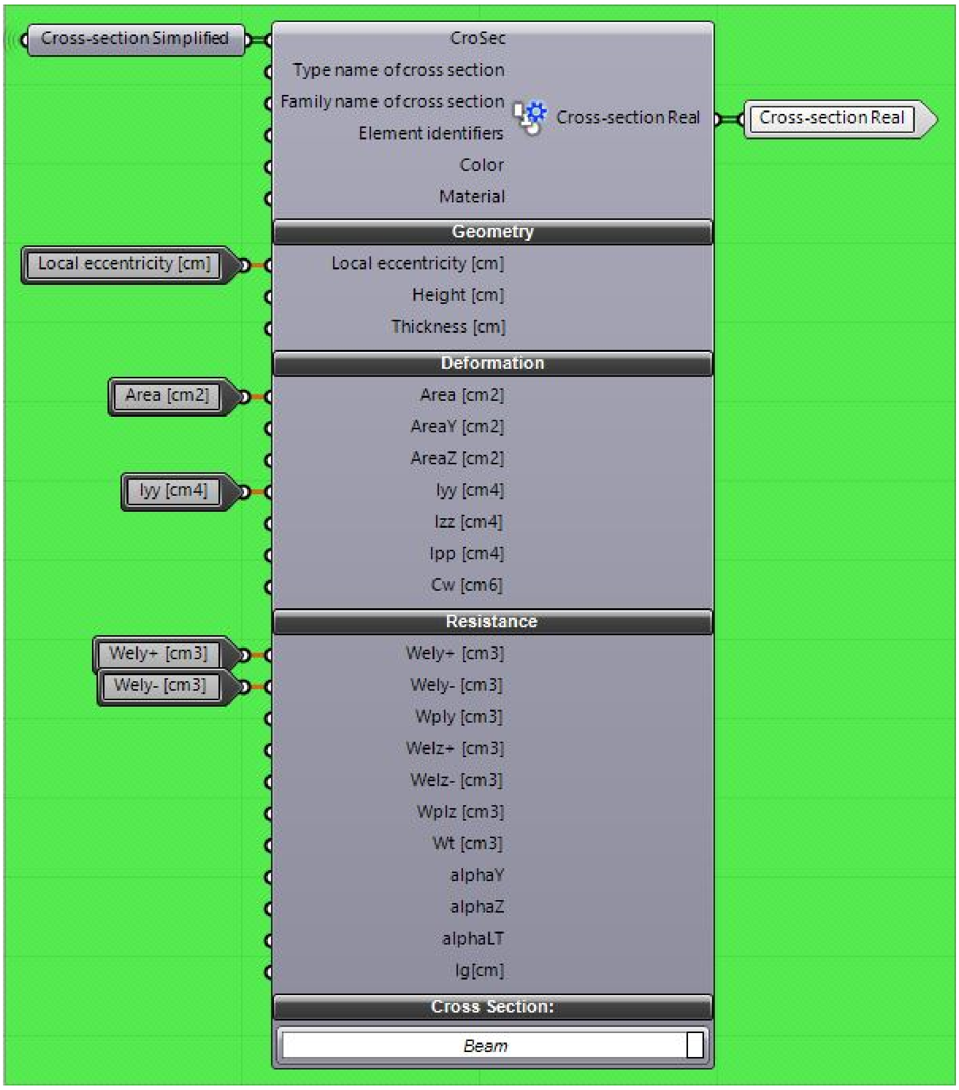


Figure 3.8: Modifying of the simplified cross-section into the cross-section as specified by the manufacturer. On the left is the simplified cross-section and the correct cross-section properties, on the right of the component is the new formed cross-section.

Parameters

The approach bridges are not optimized on cross-sectional level, since it is more efficient to use standardized beams. The only parameter which is a variable for the approach bridges is the ratio between the span length and the height of the profile ($\frac{L_span}{h}$). From this ratio follows the right profile for each span. Another way of optimizing the approach bridges can be varying the span length and so the amount of spans. This last option has also influence on the substructure and the foundation. This optimization is not automatically implemented in this project.

BOX BEAM GIRDERS

Geometry & Physical properties

The box beams (SKK) are just like the ZIPXL beams, build up as straight lines with standardized cross-sections. The cross-sections of the box beams are split up into two categories, one for the profiles with a height up to 1600 millimetres (SKK 700 - SKK 1600) and another for the higher profiles (SKK 1700 - SKK 1900). For category one; the width is 1480 millimetres, the thickness of the bottom flange is 157 millimetres and the thickness of the top flange is 170 millimetres. For category two; the width is 1180 millimetres, the thickness of the bottom flange is 205 millimetres and the thickness of the top flange is 220 millimetres. The thickness of the webs is for all profiles the same, namely 188 millimetres, and for all profiles, the top flange has notches on both sides with a width of 30 millimetres.
For an overview of the used profiles, see the document as provided by the manufacturer in Appendix B.

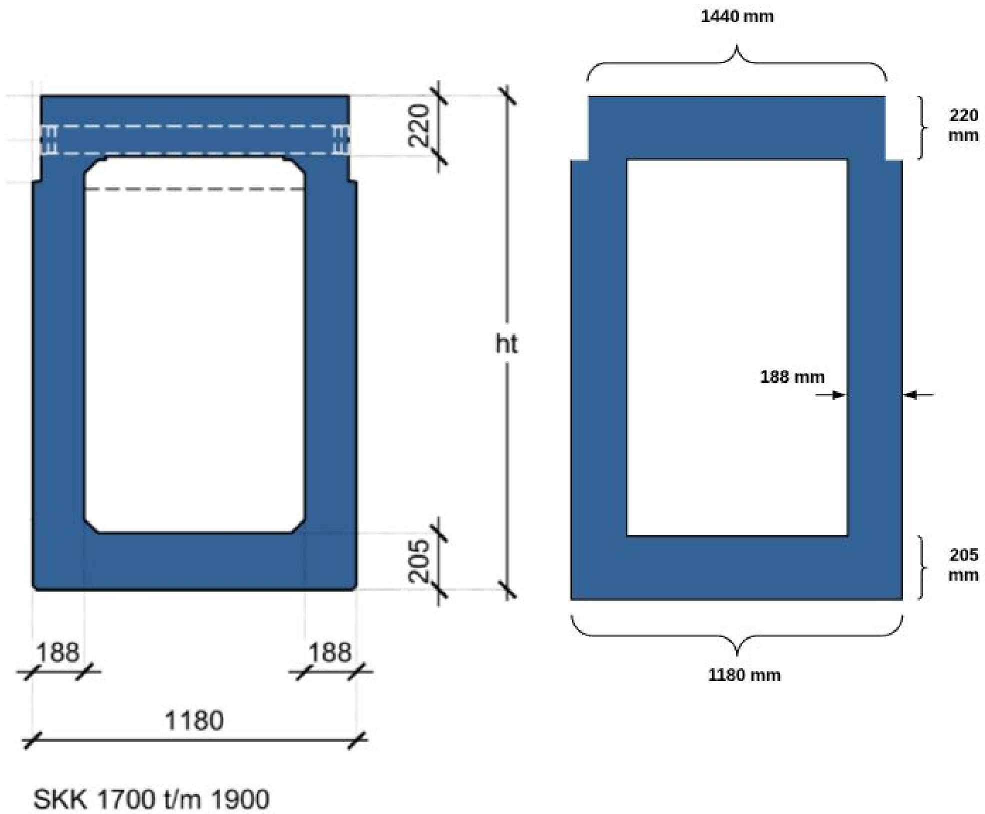


Figure 3.9: Cross-sections SKK beams for the approach bridge, real and simplified. Source: Spanbeton

Since the profile geometry of the prefabricated beams is more detailed than desirable for the model, these cross-sections will be simplified to simple rectangular shapes, see Figure 3.9. In this way the physical properties of the beams will slightly differ from the ones specified by the manufacturer, but this will be compensated by changing the cross-sectional area, moment of inertia and the resistance to the factory specifications. This is done in the same way as described for the ZIPXL beams, with help of the "Modify Cross-section" component from Karamba, see Section 3.1.2.
The profiles are selected automatically per span length with help of rules of thumb. These rules of thumb are set after trying out different settings, but can be changed by the user.

3.1.3. SUBSTRUCTURE

The substructure model is split up into a model for the main bridge, consisting of the hammerhead piers. And a model for the approach bridges, consisting of the columns and capping beams.

For the first model, the only input parameter specified by the user is the length of the hammerhead (this is the dimension in longitudinal bridge direction). The other dimension of the hammerhead is predefined, this is the width of the bottom flange of the main bridge. The hammerhead is standard designed as a rectangular wall, other shapes are not used in the model.
The second model, for the approach bridges, is designed as two columns with a capping beam on top. The dimensions of the capping beam have to be estimated, as well as the diameter of the round columns. The position of the columns is predefined.

For the substructure, the resulting support reactions of the superstructure (main bridge and approach bridge) are used as point loads in the analysis. This will be further explained in Section 3.3.

3.1.4. FOUNDATION

In the same way as the substructure, the foundation model is split up in a model for the main bridge and a model for the approach bridge. In these models, the thickness of the foundation slabs is estimated by the user, as well as the pile dimensions, capacity and centre-to-centre distance. The width of the slabs is predefined and equal to the width of the bridge. From the total reaction force coming from the substructure above, follows the amount of piles which is needed and thereby the length of the slab. In this way, these slab dimensions differ for the location where there are two spans connecting and for the location where there is the connection between the main bridge and the approach bridge.

3.2. PARAMETERS

There are three types of parameters which are used in the model; user-defined, pre-defined and optimization parameters. User defined parameters are variables which can be changed according to the clients needs, they are most of the time different for each project. Pre-defined parameters are parameters which are already set by the designer of the script, they can be changed when desirable but are standard for all bridges. The last type, optimization parameters are variables which have to be set at the beginning by the user, but later they are automatically used to optimize the design. In this section, each of these parameters will be further explained combined with example parameters.

User Defined Parameters

First of all, there are many general parameters to design the bridge model. These parameters follow from the location where the bridge has to be build, the location of existing roads, the width of obstacles which must be crossed, etc. See Figure 3.10.

The main bridge has many variables which are free to choose by the user. These are for example the thickness of the bridge webs, thickness of the deck slab, length of the cantilevering flanges, number of cells and the height of the clearance under the bridge. Together these user-defined parameters generate the bridge design, consisting of a visualization of the bridge combined with the first results.

For the analysis of the bridges and substructure, the mechanical properties of the different materials have to be specified. For instance, the design stress of the concrete and the cross-sectional area per prestressing strand, but also safety factors and centre-to-centre distance. Once specified, these values can be used every time a bridge is designed, there is no need to specify them again.

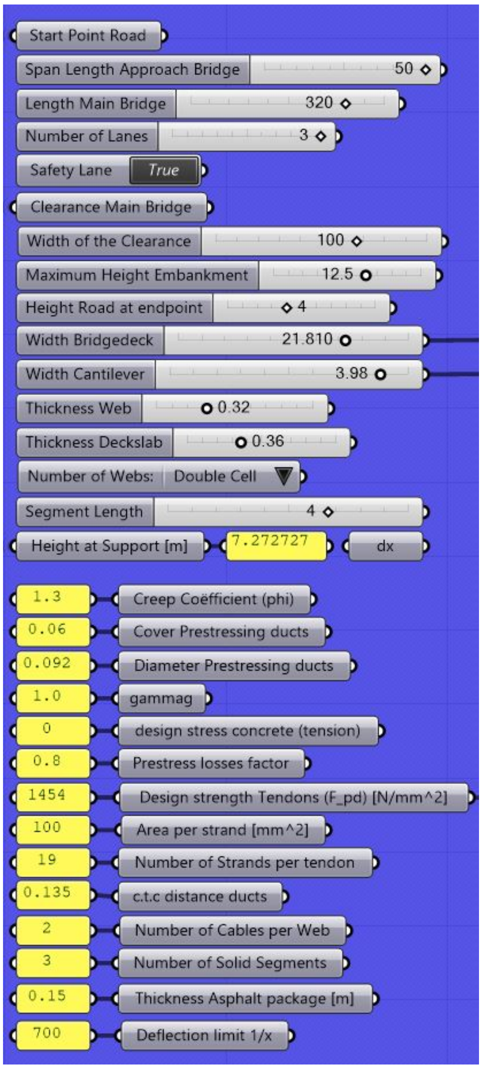


Figure 3.10: User defined and predefined parameters in the parametric model

Predefined Parameters

There are also parameters which are predefined based on previously designed bridges in the Netherlands, or based on regulations. For instance the slope of the bridge design, which is automatically designed for each span length, the length of the approach bridges, which is depending on the clearance and thereby the height of the bridge, but also traffic loads and superimposed dead loads which are fixed values in the model. These parameters are fully integrated in the model, but can be adapted if for instance regulations change.
For the approach bridges, the predefined parameters are the profile dimensions and cross-sectional properties. Also the location of the traffic loads is set in this project, but this can be changed if needed.

For all bridge elements, a standard or most-used concrete class is used. The choice for these concrete classes is based on experience and reference designs. For an overview, see Section 2.5.2.

Optimization Parameters

Optimization parameters are the parameters which are used for the automatic optimization of the main bridge and the total bridge design. These parameters are:

- Height of the Bridge (hb)
- Construction Height at the Hammerhead (h1)
- Construction Height at Midspan & Ends (h2)
- Thickness of the Bottomslab at the Hammerhead (t1)
- Thickness of the Bottomslab at Midspan & Ends (t2)

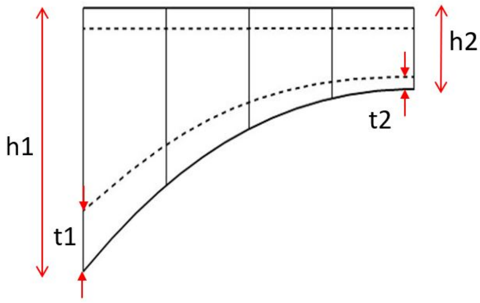


Figure 3.11: Optimization parameters for the main bridge

For a visual overview of the above stated parameters, see Figure 3.11 & 3.12. The values of the construction height and the bottom thickness in between the two boundary values are determined by the 2.5 degree functions as described in Section 3.1.1. The details for the different optimization processes and set-ups will be described in Chapter 5.

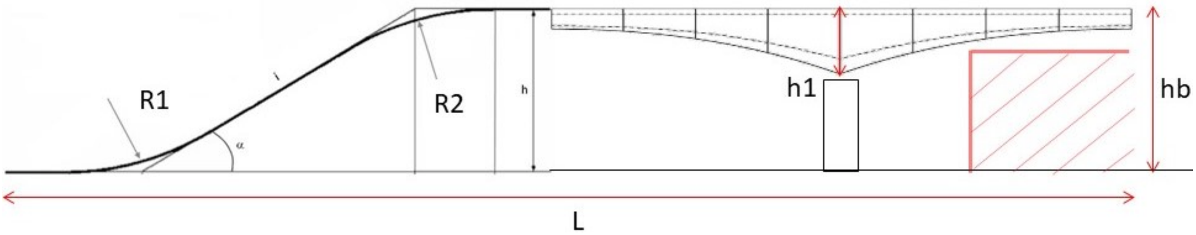


Figure 3.12: Optimization parameters for the main bridge and total bridge design.

The only optimization parameters for the approach bridges are the different type of beams and beam heights. The beam types (ZIPXL or SKK) can be selected manually and the profile height is selected automatically by making use of rules of thumb. To optimize the approach bridge, the ratio for the rules of thumb can be changed.

3.3. ANALYSIS

There are multiple analyses which have to be performed by the model, these are explained in this section. The order of analysis as described in this section is also used in the model, so first the main bridge analysis will be described following by the approach bridge, the substructure and at last the foundation.

3.3.1. MAIN BRIDGE ANALYSIS

For the analysis of the main bridge, the Grasshopper elements will be translated into Karamba elements. All elements together will be combined in a 2D Karamba model which is used to add loads and load-combinations, to specify the support conditions and to determine the force distribution. With the force distribution known, the model should be able to determine the amount of prestressing cables for which the stresses in the bridge will be limited.

For the determination of the amount of prestressing cables, there are two methods possible. One method for statically determinate structures, called the "cross-section method" and another one for statically determinate and statically indeterminate structures, called the "equivalent prestressing load method". See Figure 3.13.

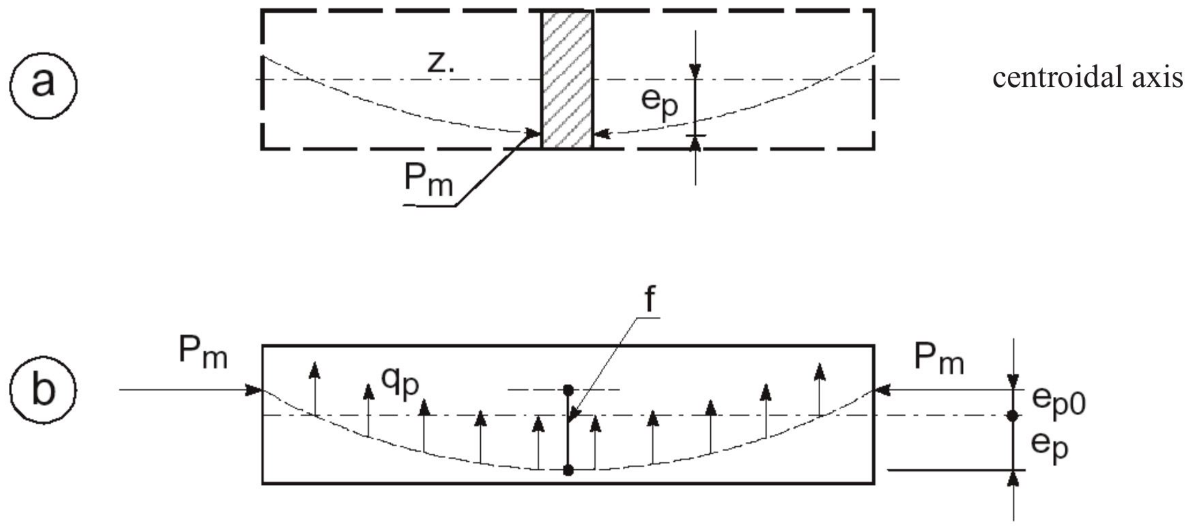


Figure 3.13: Statically determinate prestressed concrete beam with: A. Cross-section method, B. Equivalent prestressing load method [29]

Cross-section method

The cross-section method, is used to determine the stresses directly in the cross-section with help of Formulas 3.5 and 3.6. In these formulas, P_m is the prestressing force in the cable, A_c is the concrete cross-sectional area, e_p the eccentricity of the cable and W_{ct} the section modulus. The stress in the cross-section is determined by the prestressing force divided by the area (a constant stress distribution over the height), minus the prestressing moment due to eccentricity divided by the section modulus (changing stress over the height). Since the structure is able to deform freely, the moment follows directly from the force and the eccentricity. When this free deforming is not possible because the structure is statically indeterminate, this $M_p = P_m * e_p$ does not apply any more and there will be additional reaction forces and moments. In that case the "equivalent prestressing load method" should be applied.

$$\text{Stress at bottomside: } \sigma_{cb} = -\frac{P_m}{A_c} - \frac{P_m * e_p}{W_{cb}} \tag{3.5}$$

$$\text{Stress at topside: } \sigma_{ct} = -\frac{P_m}{A_c} + \frac{P_m * e_p}{W_{ct}} \tag{3.6}$$

Equivalent prestressing load method
In the equivalent prestressing load method, the curvature is used to determine the loads introduced by the cable. In this method, the force at begin and end of the beam is assumed to be only in horizontal direction, while the force due to the curvature pressure along the length of the cable is assumed to be only vertical. Reason for this is that, the angle of the cable with the neutral axis of the beam is quit small[29].
The vertical component of the cable pressure is determined by Equation 3.7. So the upward cable force is the prestressing force divided by the radius, or the prestressing force multiplied with the curvature. In most cases the length of the beam is much larger than the drape, therefore it is permitted to use Equation 3.8.

$$q_p = \frac{P_m}{R} = P_m * K \tag{3.7}$$

$$R = \frac{l^2}{8 * f} \tag{3.8}$$

In this way, Equations 3.7 and 3.8 can be substituted into Equation 3.9.

$$q_p = \frac{8 * P_m * f}{l^2} \tag{3.9}$$

Note that the moment at midspan is $\frac{q*l^2}{8} = P_m * f$, this checks out with the theory.

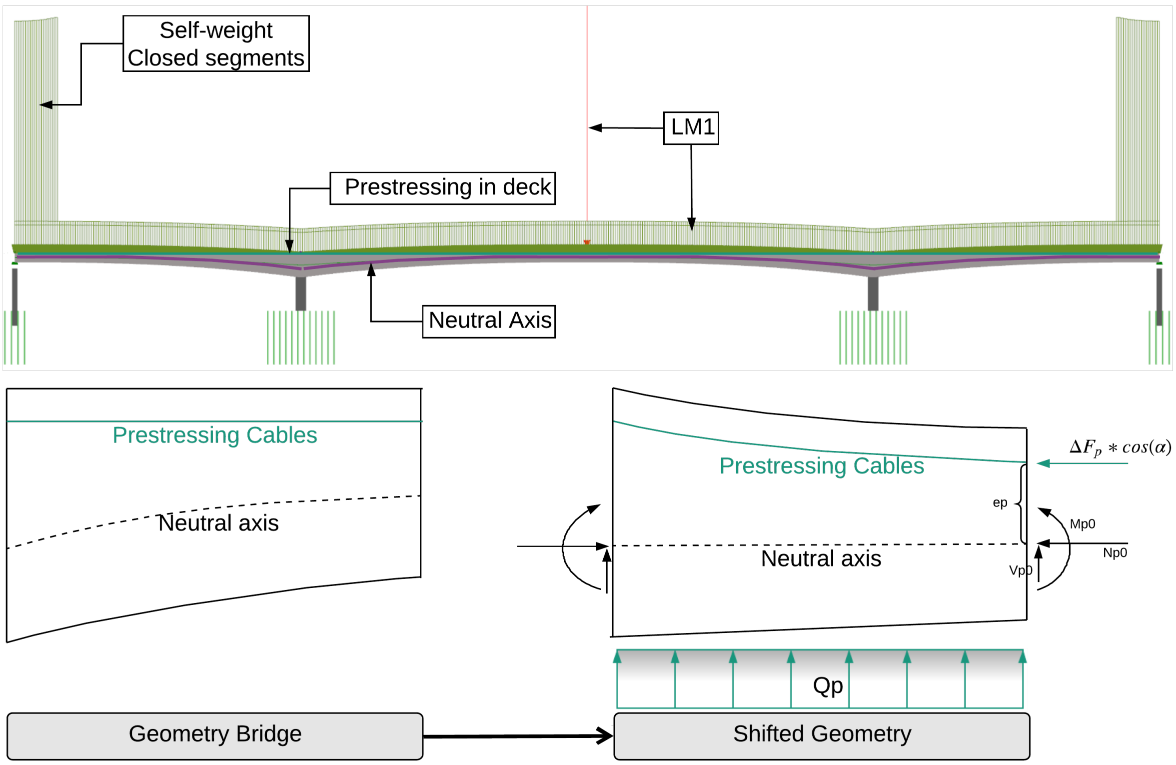


Figure 3.14: Structural model of the main bridge and shifting of the neutral axis and tendons to achieve a straight system line.

In the case of the balanced cantilever bridge, the cables are straight lines but the neutral axis of the bridge itself, has a changing curvature over the length of the bridge. In that case it is allowed to create a straight neutral axis, by shifting the cross-section and the prestressing cables upwards, see Figure 3.14. Consequently, the cable now has a curvature and therefore gives an upward pressure on the segment.

While for a simply supported beam, the drape of the cable and thereby the cable pressure is easy to determine and there are only a few components, with the balanced cantilever bridge this is more complex. To achieve the right cable pressure per dx, the second derivative of the shape of the bridge is taken to get the curvature over the length x ($K(x)$). This $K(x)$ is multiplied with the horizontal component of the prestressing force creating the Q_p, see green line-load at the bottom of Figure 3.14. Another component which has to be added at every segment dx, is the moment due to eccentricity of the additional force ΔF_p. The difference in shear force between the right side and the left side of the segment, also creates an extra moment. Furthermore there is a difference in work line between the normal forces N_1 and N_2 due to the changing curvature.

Since this method has many small components and it is not easy to implement them all in the parametric model, it is hard to acquire a correct prestressing distribution. Also there are some simplifications in the theory of this method, like the neglecting of the horizontal cable pressure. This is only possible for small segment lengths where the curvature of the cable is small, or for symmetrical structures where the horizontal forces cancel each other out.

Conclusion: After applying both methods and checking the results with existing calculations at Movares, the decision is made to use the cross-section method in this project. This will be further explained in the next paragraph about the change of statical system.

Change of Statical System

For the construction phase, in which the cantilever prestressing is determined, it is still allowed to use the cross-section method. So the amount of cantilever prestressing which follows from this method is applied at the construction phase, as well as the user phase. But since the main bridge changes during the construction phase from a cantilevering structure (statically determinate) to a continuous bridge on four supports once the bridge is closed (statically indeterminate). The method which should be applied for determining the continuity prestressing force is; the "equivalent prestressing load method". Nevertheless this method will not be applied in the parametric model. The reason for this decision will be explained with help of the following example.

When a simply supported beam loaded by gravity is considered, the moment at midspan is known as $\frac{1}{8} * q * l^2$. Now assume that the same beam is clamped at both ends, making it a indeterminate structure. This gives a change in moment distribution as can be seen in Figure 3.15.

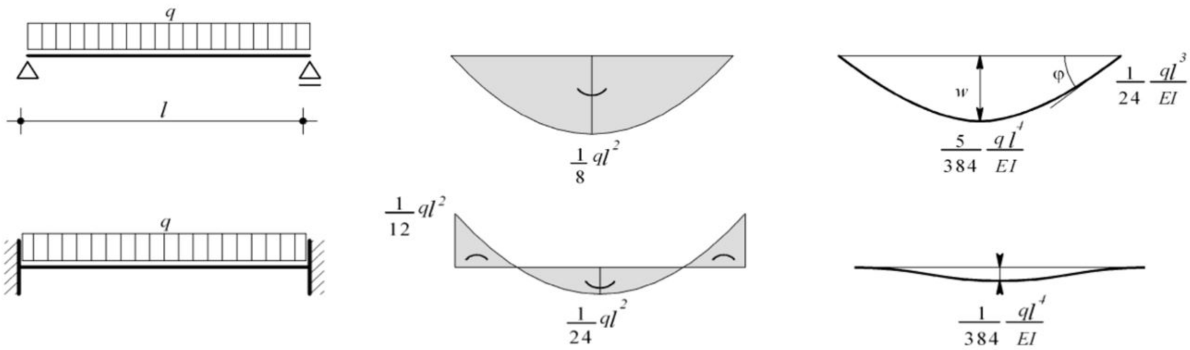


Figure 3.15: Change of moment distribution due to statical indeterminacy

Note that the total moment due to the self weight is still the same, $\frac{1}{12} + \frac{1}{24} = \frac{1}{8}$, but it is shifted upwards. This change of statical system will have the same effect on the moment distribution due to prestressing, therefore the cantilever prestressing still has sufficient capacity to withstand the weight of the bridge after the bridge is closed. Since the moment at midspan is reduced, also the moment due to the continuity prestressing will reduce. So instead of $M_p = F_p * e_p$, the moment distribution will be $M_p = F_p * e_p * RF$ with Reduction Factor (RF) equal to 0,5. This is an assumption based on previous bridge designs by Movares.

When the factor must be determined exactly, it can be done by calculating the rate of fixation of the two supports, which is depending on the stiffness of the side spans. With stiffness equal to zero leading to the statical determinate solution and an infinite stiffness leading to the statical indeterminate model.

To conclude, due to the change of statical system, there is more continuity prestressing needed in the middle of the bridge span. And in this way it is allowed to use the "cross-section method", to simplify the analysis and thereby reduce the amount of force components in the model.

Ultimate Limit State Analysis
In addition to the ULS load combinations as described in Chapter 2.6.3, for the analysis of the main bridge, there is another load combinations which should be taken into account, namely the moment due to the change of statical system. This moment distribution (M_{ed}) is depending on the moments in the old system combined with the moments in the new system based on a certain creep coefficient. The old system in this case is the bridge in the construction phase, just before closing of the bridge. The new system is the bridge in the end phase, after the bridge is finished and the moment distribution is transformed into its new shape due to creeping of the concrete.

For the creeping of the concrete, a couple of remarks should be made. For instance, the phased construction of the bridge is not accounted for, otherwise the segments would all have a different age and therefore a different creeping coefficient (ϕ). Also the size of the cross-section changes over the length of the bridge, which also leads to a difference in creep coefficient. To be able to assume a realistic creep factor for this project, a few creep coefficients are determined by hand with the graphs as can be seen in Figure 3.16.

In this graph, $h_0 = \frac{2*A_c}{u}$ is the notional size of the cross-section calculated by two times the concrete area divided by the perimeter which is in contact with the outside air, t_0 is the number of days between the casting and the loading of the concrete and there is assumed a quick cement type R.

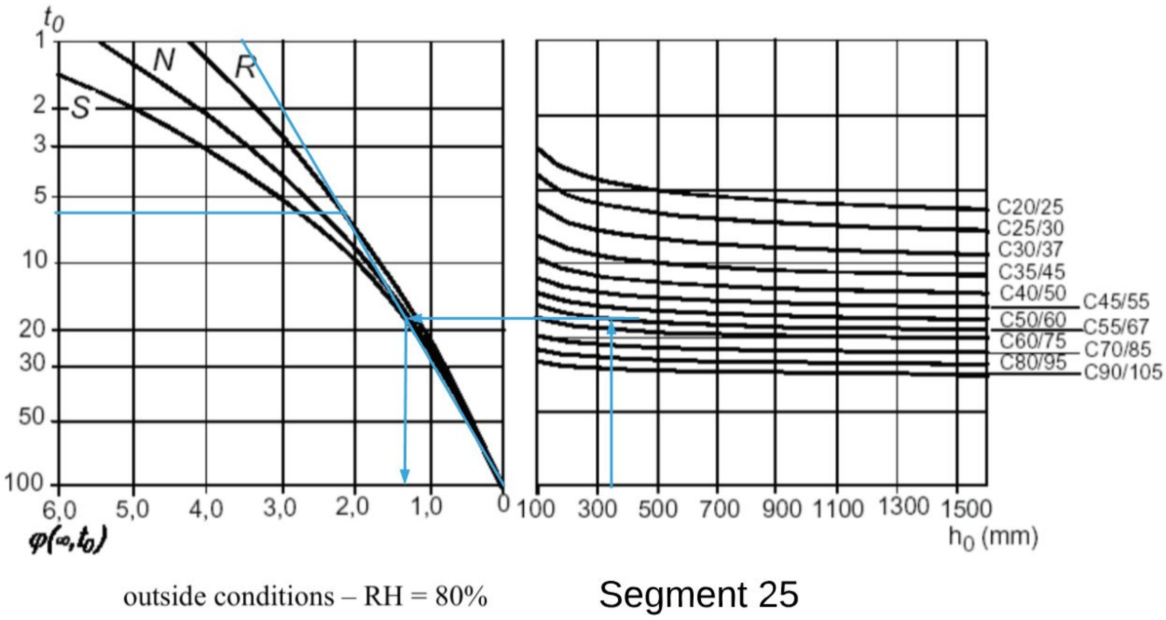


Figure 3.16: Determination of the creep coefficient, leads to $\phi = 1,3$. Source: NEN-EN 1992-1-1 paragraph 3.1.4[**?**]

From these simple determinations of the creep coefficient for multiple cross-sections, followed that the coefficient does not come above 1,3. But to achieve an extra conservative design, there is chosen to use a creep coefficient of 1,5. This is in line with other research into the balanced cantilever bridge. [28] This assumption leads to Equation 4.3 and 4.4 with $\phi = 1,5$, $\gamma_{G1} = 1,4$, $\gamma_Q = 1,5$ and $\gamma_{G2} = 1,25$ these are also implemented in the parametric model.

$$\begin{gathered} M_{ed}(6.10a) = (\gamma_{G1} * M_{sw,construction} + M_{p,cantilever,construction}) * e^{-\phi} \\ +(\gamma_{G1} * M_{sw,end} + M_{p,cantilever,end}) * (1-e^{-\phi}) \\ +\gamma_{G1} * M_{superimposeddeadloads} + \psi * \gamma_Q * M_{variable} + M_{p,continuity} \end{gathered} \tag{3.10}$$

$$\begin{gathered} M_{ed}(6.10b) = (\gamma_{G2} * M_{sw,construction} + M_{p,cantilever,construction}) * e^{-\phi} \\ +(\gamma_{G2} * M_{sw,end} + M_{p,cantilever,end}) * (1-e^{-\phi}) \\ +\gamma_{G2} * M_{superimposeddeadloads} + \gamma_Q * M_{variable} + M_{p,continuity} \end{gathered} \tag{3.11}$$

3.3.2. APPROACH BRIDGE ANALYSIS

The analysis of the approach bridges in 3D[1] is less complex and extended compared to the analysis of the main bridge. Here also the prestressing design is made, based on the previously described load cases and combinations. The difference is that in the approach bridges there is only one type of prestressing, but the amount of prestressing is determined in three stages. For this calculation the cross-section method is used, which is a logic choice since the prefab beams are statically determinate. For an overview of the structural model, see Figure 3.17 with In the red outline surface a UDL of 9 kN/m^2 and in the orange outline surface 2,5 kN/m^2. The red point loads, representing the Tandem axle loads, are eccentrically applied in the middle of the span.

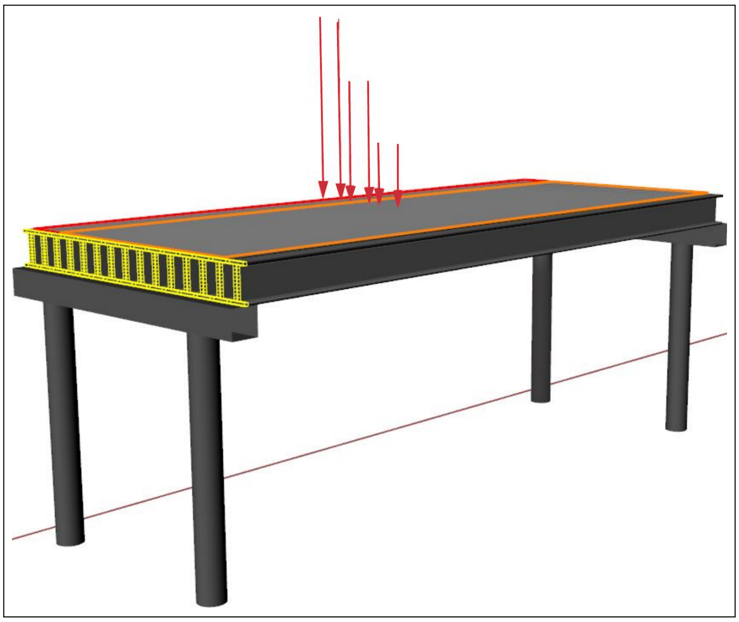


Figure 3.17: Structural model for calculation of an approach bridge span.

Initial Stage

The first stage in which a calculation is performed, is the "Initial Stage". This is during the stressing of the beams in the factory. The only load which is present during the stressing is the self-weight of the beams. The analysis at this stage consists of two checks, the stress at the top and the bottom of the beams due to prestressing. From the requirements, that the tensile stress at the top of the beam does not exceed the tensile capacity and the compressive strengths does not exceed the compressive capacity of the concrete, follows an initial allowed prestressing force ($F_{p,i}$). This force forms the upper boundary for the initial prestressing force.

In this project there is chosen for an extra conservative design, by setting the tensile capacity to -1 N/mm^2 (slightly compressive).

Intermediate Stage

The second stage is the "Intermediate Stage", in this stage the beams are hoisted on location and the casting of the compression layer takes place. At this stage, the lower boundary for the prestressing force is determined by again checking the stresses at top and bottom. The governing (highest) required prestressing force from these checks, gives the minimal amount of prestressing strands. Note that in this case, the self-weight of the deck layer is present, but this layer does not help for the strength of the cross-section.

Final Stage

The third stage is the "final stage", here all the loads are present including the traffic loads. Also the deck and the beams work together in this stage, so the section moduli are combined and the stresses are checked again.

[1]Note: Although the model for the approach bridge is in 3D, no calculations in transverse direction are performed. The third dimension is only used for the application of the loads, once this is carried out, the beam with the highest load is selected and analysed in 2D.

The lowest force determined by these checks gives the upper boundary for the working prestressing force.

When all checks are performed, the working prestressing force coming from the last stage is multiplied with the stress loss factor of 0,85 to achieve the initial prestressing force. This force is checked with the allowable initial prestressing force, and leads to the required number of prestressing strands per beam.

The prestressing is added according to the principle described in the paragraph about pre-tensioning in Section 2.5.1. So the prestressing will be added in three steps to simulate a parabolic prestressing distribution and a transmission length of 1 meter is assumed. After application of the prestressing, the stresses are additionally checked and the final bending moment is determined to perform a unity check. This will be described in Section 3.4.

3.3.3. SUBSTRUCTURE ANALYSIS

The substructure analysis only focusses on the columns and hammerhead piers, these are loaded by the support reactions of the main bridge and the approach bridge. Also the traffic loads are taken into account. The capping beam is not calculated in this project and there is chosen for a standard solution with two columns. For an overview of the structural model, see Figure 3.18.

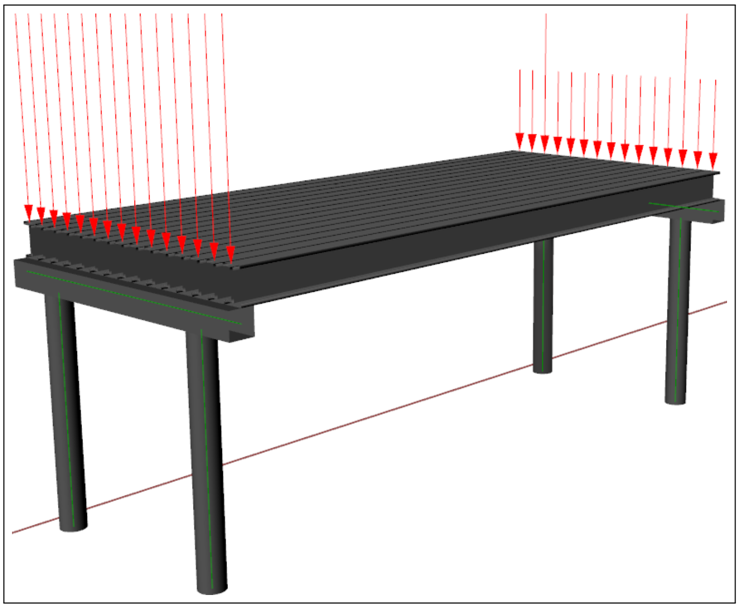


Figure 3.18: Structural model for calculation of the substructure.

As can be seen in the Figure above, the point loads on the left side are twice as large compared to the right side, while there are also two large point loads. The reason for this is that the left side is between two spans and the right side is at the location of the connection between the approach bridge and the main bridge.

In the analysis, the force in the columns is determined and the columns is checked on compression and buckling capacity. When the diameter of the column is smaller than the maximal value required by the two checks, a larger diameter should be chosen by the user.

3.3.4. FOUNDATION ANALYSIS

The analysis of the foundation is simplified into a calculation of the number of foundation piles, other calculations will not be performed. The number of foundation piles follow from the vertical support reaction coming from the superstructure and substructure above. These vertical loads are multiplied with a factor 1,5 to compensate for potential horizontal loads. In addition of these loads, there is the self-weight of the foundation slab which is estimated by the user.
Once the number of piles is calculated, this will generated the length of the foundation slab depending on a user defined grid. The width of the foundation slab is equal to the bridge width.
The foundation slab and the piles itself are not checked on capacity or deformations.

3.4. CHECKING & VERIFICATION

Since the bridge design should be realistic and constructable, it should meet a couple of requirements. Therefore multiple checks are performed, including capacity checks and fitness checks. Also the model itself should be verified with traditional design methods and reference projects.

3.4.1. MOMENT CAPACITY CHECK - ULS

The first check which will be performed in the model, is for the Ultimate Limit State (ULS). In this check, the moment capacity (M_{rd}) of the cross-sections are determined and the check holds that the acting moment (M_{ed}) should be smaller for each cross-section, so $M_{ed} < M_{rd}$.

Determine M_{rd}

For the determination of the moment capacity for each cross-section, it is first necessary to calculate the height of the compression zone (X_u). For the main bridge, the difficulty here is the non-uniform cross-section over the height. For the compression zone, a bi-linear stress-strain diagram is assumed.

With help of horizontal equilibrium, first the normal force in the concrete (N_c) is determined. Once this N_c is known, the height of the compression zone can be calculated in three different ways.

1. Xu in bottomslab
2. Xu > bottomslab and a < bottomslab
3. Xu > bottomslab and a > bottomslab

To clarify the different options, see Figure 3.19. For every cross-section, all three options are calculated and the correct one is selected. This same set-up is also used to determine the moment capacity for each position along the main bridge.

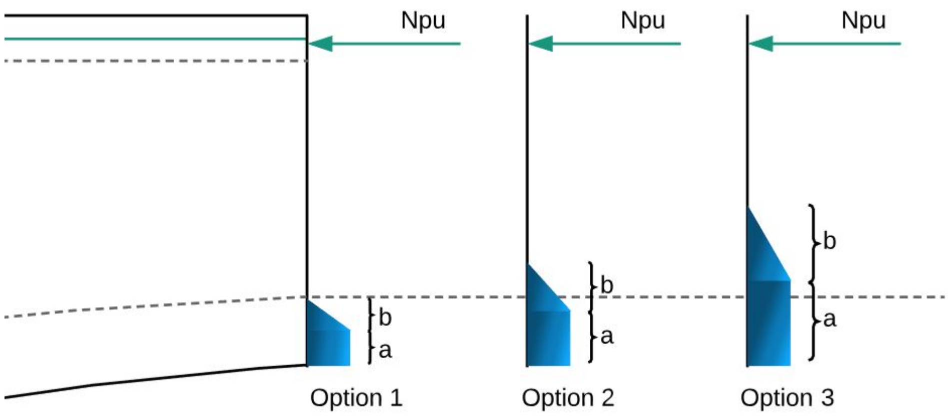


Figure 3.19: Three different options for the height of the compression zone X_u

For this capacity test a unity check of 1,0 is taken as limit, $\frac{M_{ed}}{M_{rd}} \leq 1,0$, although there is a trend going on in the sector where there is chosen to accept only structural solutions with a unity check of 0,8 or lower. The reason why in this thesis, this lower limit is not taken into account is that there are already additional safety measures present in the model. So to prevent the model from becoming too conservative, a unity check of 1,0 is kept. To substantiate this, an overview of the included safety measures is given in the next paragraph.

Safety measures

In the calculation of the prestressing, a stress loss factor of 0,80 instead of the prescribed 0,85 is taken (5% extra safety). Furthermore, the relaxation of the steel tendons in the compression zone of the main span is not taken into account and there is only calculated with either the cantilever prestressing or the continuity prestressing, this is an underestimation of the moment capacity of the cross-section. (approximately 5%) In reality these two prestressing types will help each other. Also when the use of reinforcement steel is taken into account, the stresses in the cross-section will be less problematic and the strength of the bridge increases (approximately 10% extra capacity). Since these additional safety measures are present in the parametric model, it is justified to use UC=1,0.

3.4.2. DEFORMATION CHECK - SLS

For the serviceability limit state (SLS), the design will be checked on the maximum displacement, which occurs in the middle of the main span. The maximum allowable deflection follows from the ROK1.4 paragraph 4.1[30], here $\frac{L}{300}$ is taken as limit. To ensure an additional conservative way of designing, there is chosen for a limit of $\frac{L}{500}$. This value can also be adapted by the user.
When the deflection of the bridge exceeds the allowable deflection limit, the design does not suffice and a penalty is given to the overall costs.

3.4.3. CONSTRUCTIBILITY CHECKS

An important check which is performed for each design, is if the bridge stays out of the clearance gauge. This is the construction free space where the ships pass. When the structure coincides with the clearance gauge, once again a penalty will be given.

The last automatic check which has to be satisfied is the constructibility check, with other words, do the tendons fit in the deck and bottom flange. For this check, an standard distance between the tendons of 135 mm is assumed, this number is based on previous designs. Another assumption is, that there is only one layer of tendons.

3.4.4. STRESS CHECK

The stress check is not automated in the model, but the stresses will be plot for each of the bridges. In this way it is possible for the user to check whether the stresses do exceed certain limits, or if they stay within the boundaries of the material. As an addition, there could be feedback of the model when the stresses are to high, but this is not further implemented in the model.

3.4.5. BUCKLING CAPACITY OF THE PIERS

The piers and columns of the model are all checked on compression capacity by comparing the acting load with the capacity of the cross-section. Since the columns are quite high, they are also checked on buckling capacity with help of the formula sheets which can be found in Appendix D.
In these formulas, which are also implemented in the parametric model, the minimal dimension of the column is determined based on the axial load and the length of the column. The model does not automatically change the dimension of the columns, this should be done by hand, reason for this is the minimal influence of this optimization for the total material usage.

3.4.6. MODEL VERIFICATION

The model itself is verified with help of previously designed bridges by Movares. These bridges were recreated in the parametric model and the cross-section parameters, force distribution, amount of prestressing cables and other material amounts are compared. From these comparisons followed that there are some minor differences between the models, but this can be attributed to another way of modelling. The models by Movares are made out of large segments (approximately 4 m), while the parametric model uses a standard segment length equal to $dx = 1m$. Due to confidentiality reasons, the results of this comparison are not embedded in this thesis.

4

Parametric Model

In this chapter the working of the parametric model itself is explained by using a flowchart. Subsequently the script is described in more detail with help of illustrations and visualizations. For an overview of the parametric model, see Figure 4.1.
This chapter is based on the following subquestions:

4.1 How to compose the parametric model?
4.2 What are the important steps in the process of designing concrete bridges parametrically?
4.3 How to create clear visualizations for the user?

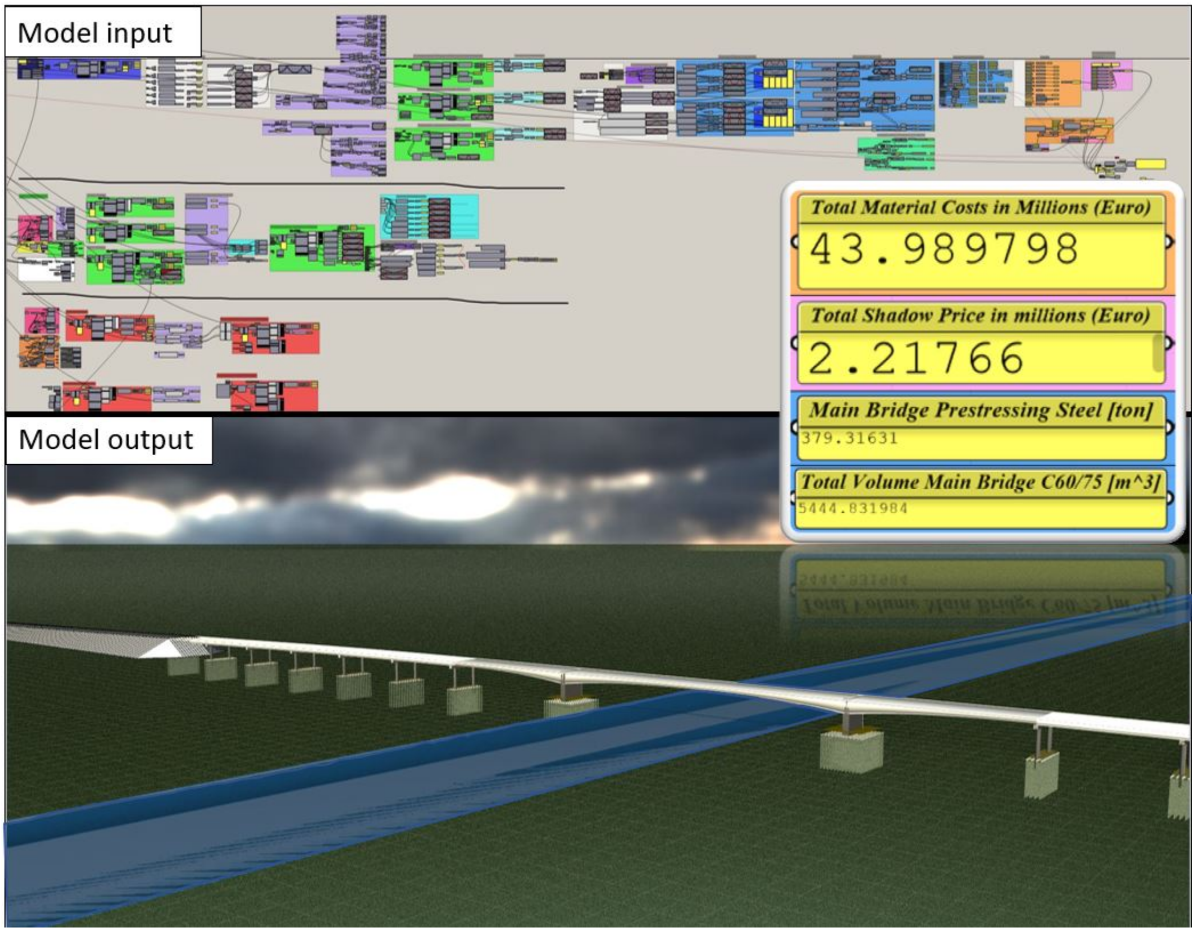


Figure 4.1: Picture of the parametric model with input and output

4.1. MODEL SET-UP

In this section, the model set-up will be described by first following a flowchart where the steps of the model are visualized in general, see Figure 4.2.

The first step in the model is the input step. In this step all project and location specific input will be given by the user, the other input parameters are already provided by the script designer. With this input, the model has all information to construct the geometry and subsequently perform all analyses and calculations. The geometry is split up into, the main bridge, approach bridges and substructure to speed up the process, but there is also a visual model with all separate parts combined.
Simultaneously, the material amounts, costs and environmental impact of the design is determined and these results are immediately visible. After this step, the user may want to optimize the design. This optimization process will eventually lead to the design with the lowest costs and/or environmental impact, which will be the preliminary design.

In the next section, this simple flowchart will be described in more detail combined with illustrations from the model and an extended flowchart.

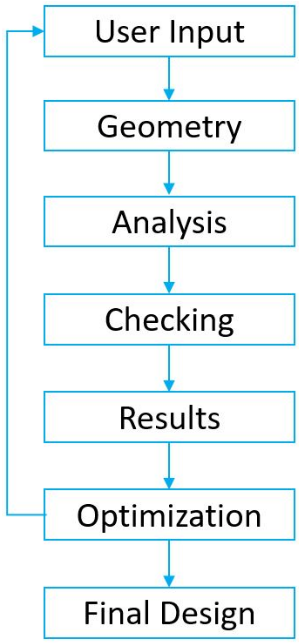


Figure 4.2: Flowchart of the Parametric Model

4.2. DETAILED DESCRIPTION

In this section, the model will be described in more detail. Also the flowchart is extended into a more detailed flowchart, see Figure 4.3.

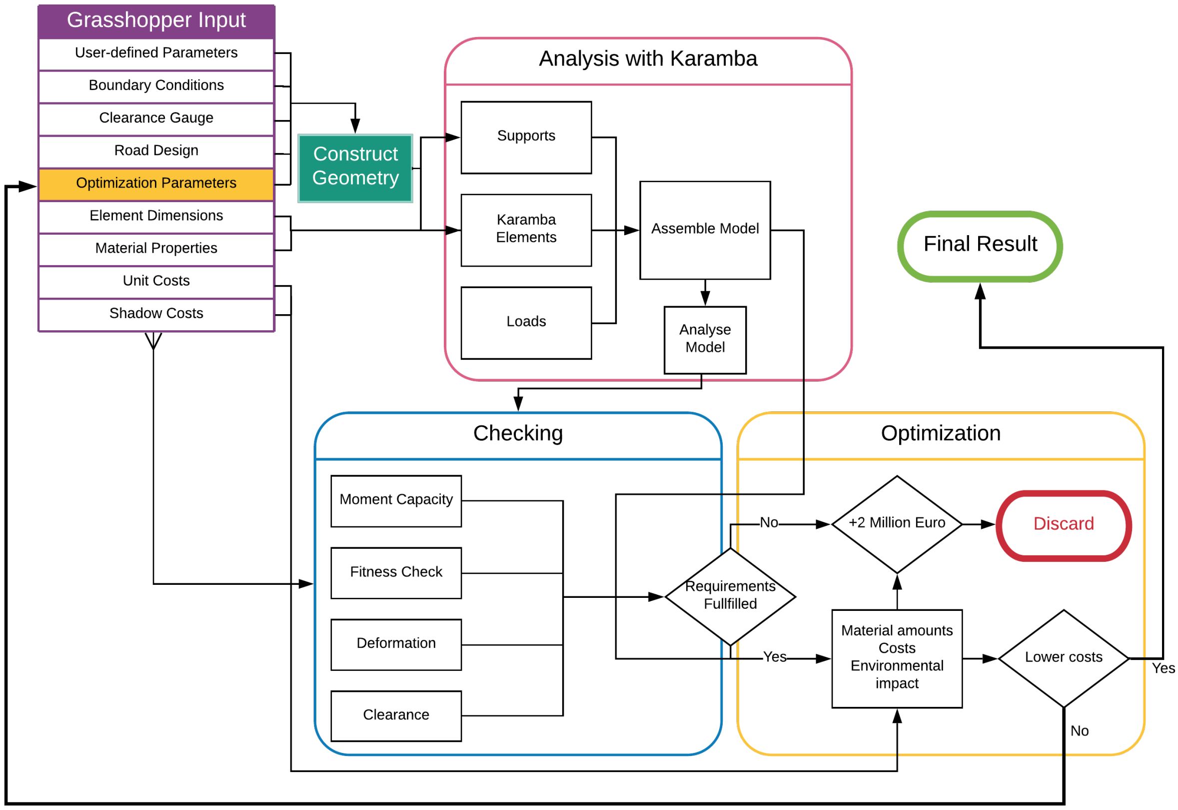


Figure 4.3: Detailed flowchart of the Parametric Model

The sequence of the processes is similar to the simple scheme, but there is more information about the steps. In the next subsections each of these steps will be described and the link with other steps is also explained.

In section 4.3, there will be focussed more on the visualizations of the parametric model although there are some previews in the geometry subsection.

4.2.1. INPUT

As earlier described, the input for the model consists of many parameters. This input is divided in location and boundary parameters, main bridge parameters, approach bridge parameters, substructure parameters, material parameters and (shadow) costs parameters. Most of the parameters are predefined by the script designer or they are similar for most of the projects. These are for example the unit costs, the material properties of the various construction elements and the parameters which follow from the Eurocode or additional design regulations.
In this subsection, the parameters will be listed and in the following subsection the geometry parameters will be supported with visualizations.

General parameters
The parameters that are important for each specific location are:

- The height of the start point of the road, which is at {0,0,h} in Rhino.
- The length of the main bridge, this is two times the expected length of the main span.
- The height of the main bridge above ground level.
- The span length for the approach bridges.
- The dimensions of the clearance gauge, width and height of the construction free space.
- The height of the road embankment.
- The width of the carriageway or the number of lanes and presence of safety lane.
- The height of the road at end point.

Once these parameters are filled in, the main dimensions of the road design are determined.

Bridge parameters
The next step is to provide the estimated dimensions for the various construction elements. Starting with the main bridge, this are the important dimensions:

- The assumed height of the bridge at the hammerhead. $\left(h1 = \frac{L_{span}}{22}\right)$
- The height at midspan and ends. (h2)
- The thickness of the bottomslab at the hammerhead. (t1)
- The thickness of the bottomslab at midspan and ends. (t2)
- The width of the bridge deck. [1]
- The width of the cantilevering deck flanges
- The number of webs. This is linked to the choice for a single, double or triple cell bridge
- The segment length which is used for the practical amount of prestressing.
- The thickness of the deckslab, which is constant over the width of the deck.
- The thickness of the webs

[1] This parameter is only needed when the number of lanes is not already specified.

With this input, the geometry of the main bridge can be generated.

The only parameter for the approach bridges is, the type of prefabricated beams; inverted-T / I beam or box beam. The dimensions of the beams are specified in a cross-section table and are selected by using rules of thumb.

Substructure parameters
The dimension parameters for the substructure are:

- The height and width of the capping beams.
- The number of columns and the diameter.
- The width of the hammerhead piers. The length is equal to the width of the bottomslab of the bridge.

Foundation parameters
For the foundation of the bridge also dimensions must be provided, these are:

- The thickness of the foundation slabs for the main and approach bridge.
- The estimated length of the slab. The width of the slab is equal to the bridge width.
- The distance between the foundation piles.
- The dimensions of the piles and the assumed capacity.
- The length of the foundation piles to determine the right material amounts and costs.

The capacity of the foundation piles and the loads of the bridge and substructure, determine the required amount of piles and the actual dimensions of the foundation slab. This dimension overrules the estimated length of the slab.

The parameters for the analysis of the bridge, like the properties of the prestressing tendons and the mechanical properties of the concrete, are predefined by the script designer. These are based on the Eurocode and/or most used values by structural engineers at Movares. These parameters will not be changed by the user, unless changes occur in the regulations or for instance the material price changes.

In the next subsection, the above listed parameters are visualized.

4.2.2. GEOMETRY

In this subsection, the focus is on the visual Rhino output or schematic overview of the previous described parameters. This output is created in Grasshopper with help of the plug-in Karamba.
At first the points, lines and surfaces are created with standard Grasshopper components. Subsequently, the simple geometry is translated into Karamba elements with the "Line to Beam" and the "Mesh to Shell" components, see Figure 4.4. In these components the lines or shells are linked to a certain identifier, which can be a unique name. This can be used to link the beam to a specific material and cross-section. The results of these components is a three-dimensional visualization of the geometry.

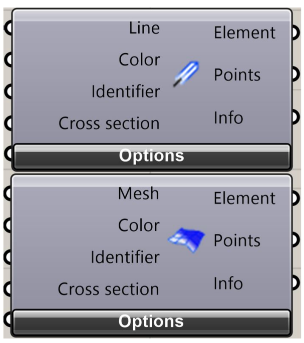


Figure 4.4: Karamba components "Line to Beam" and "Mesh to Shell".

The first geometry which is visualized, is based on the general road design and boundary parameters, see Figure 4.5 & 4.6. The formula for the vertical alignment of the beam is here implemented, the length of the alignment depends on the maximum height of the bridge.

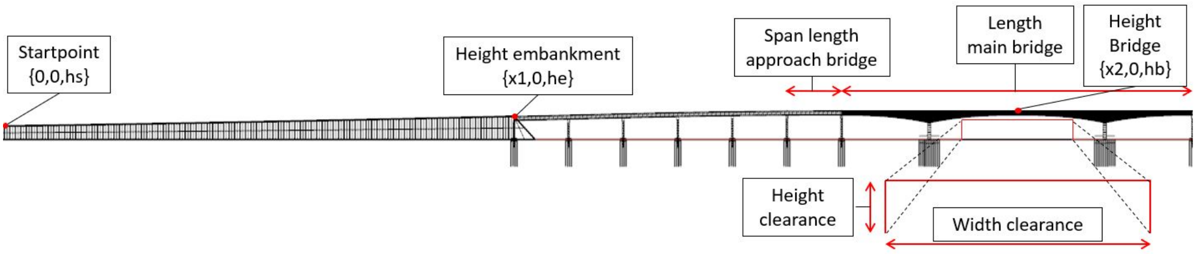


Figure 4.5: Schematic overview of the road design.

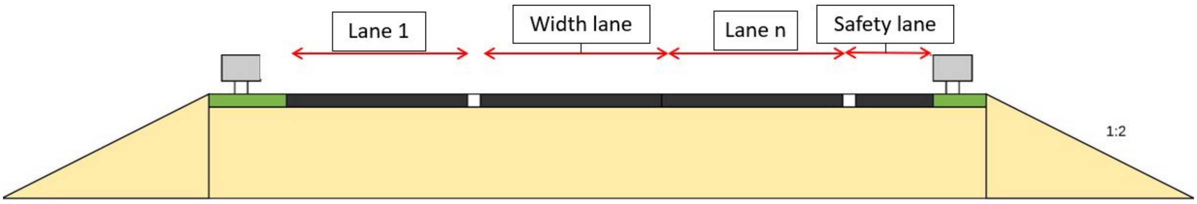


Figure 4.6: Overview of the carriageway.

Note that also the more detailed bridge geometry is visualized here, although this is not generated by the general parameters, this will be explained in the following figures.

Subsequently, the geometry of the main bridge is generated, based on the outer dimensions and the cross-section parameters. See Figure 4.7 & 4.8. For the shape of the main bridge, the 2.5 degrees function as described in Chapter 3.1.1 is used.

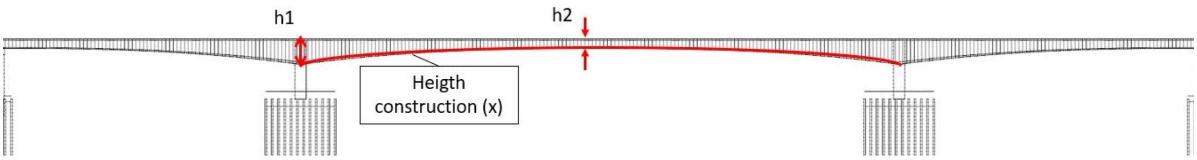


Figure 4.7: Geometry of the main bridge.

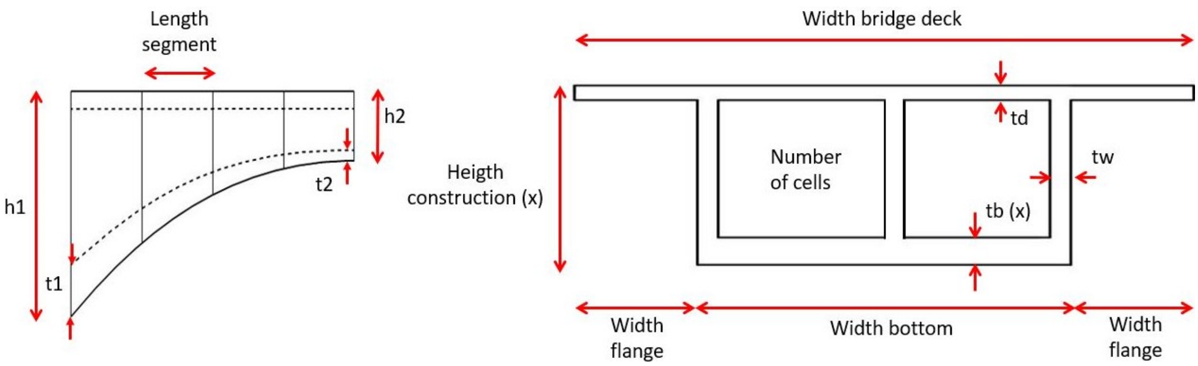


Figure 4.8: Cross-section of the main bridge.

For the geometry of the approach bridges, the only variable parameter is the type of prefabricated beam. The two types can be seen in Figure 4.9. The height of the beams depends on the span length.

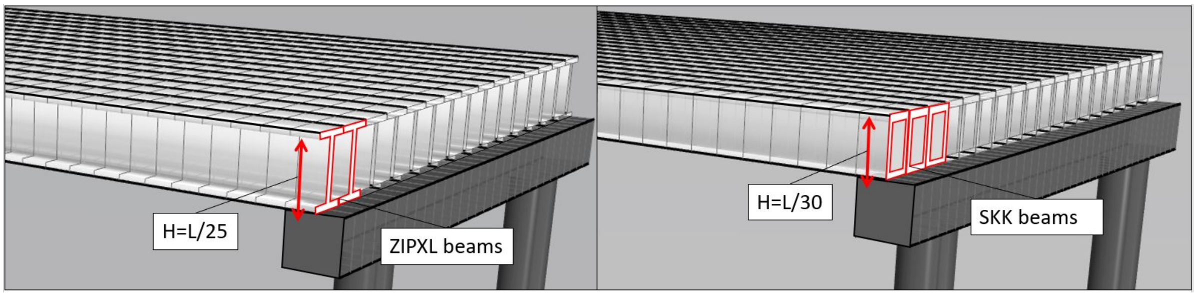


Figure 4.9: Approach bridge span with inverted-T or I-beams and box beams.

The geometry of the substructure can be seen in Figure 4.10.

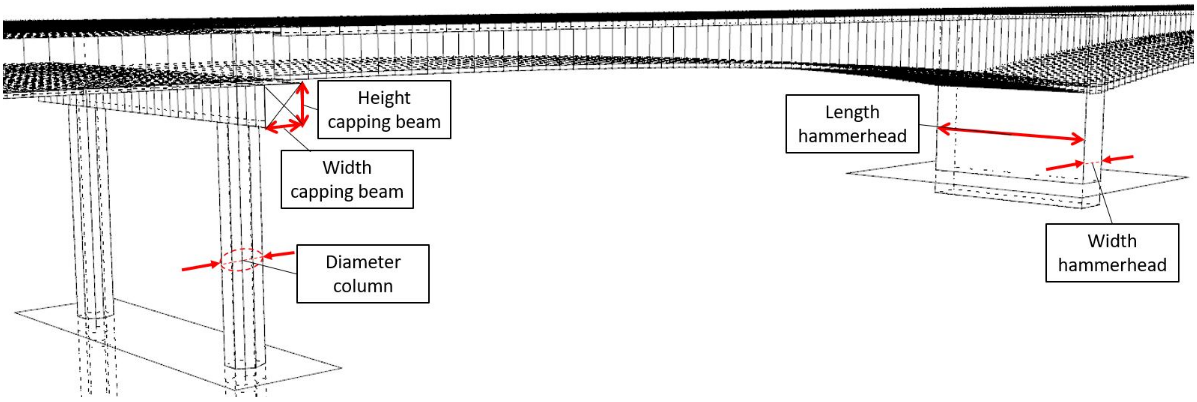


Figure 4.10: Overview of the substructure geometry.

At last the geometry of the foundation is also generated, see Figure 4.11.

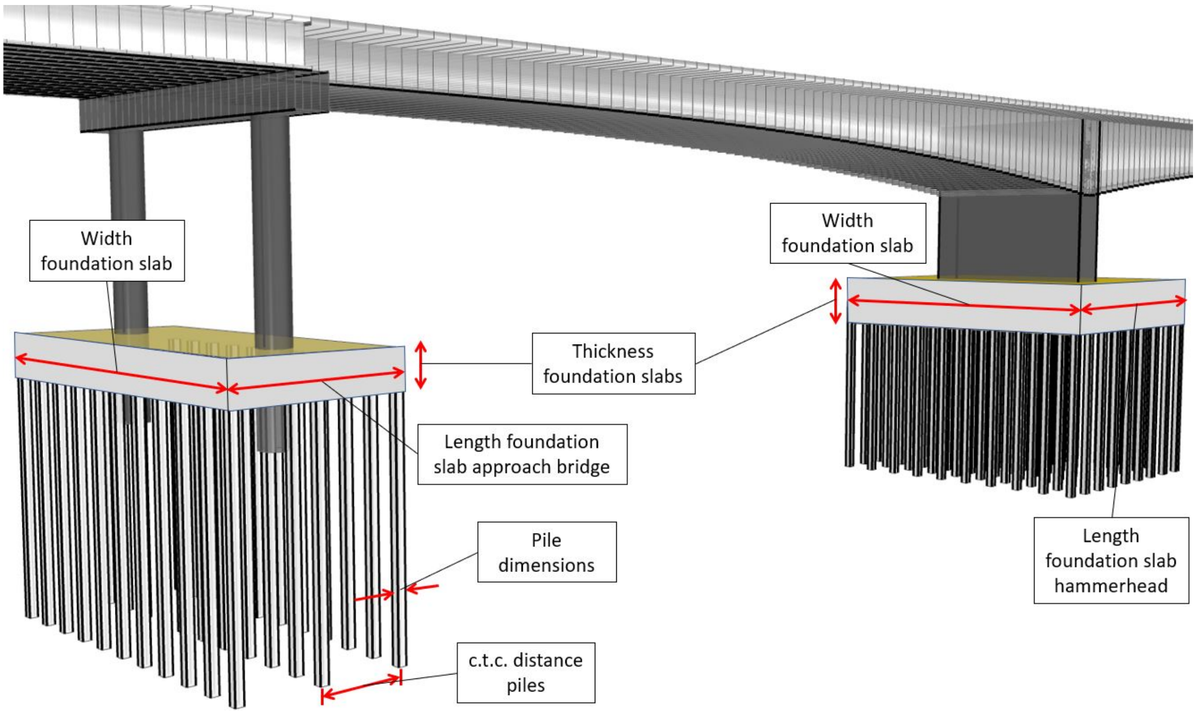


Figure 4.11: Geometry of the foundation.

4.2.3. ANALYSIS

The structural analysis of the various construction parts is performed by Karamba, but before the analysis can start, the Grasshopper geometry are translated into Karamba elements. The visual output of this action can be seen in the previous subsection.
Once the elements are created and the materials are assigned, the physical relations between the different elements and the supports of the structure should be defined. Also the gravity load, live loads and superimposed dead-loads must be added. These loads are divided over multiple load cases, so the influence of each specific load can be extracted from the model and used in the calculations.
In this section, the description will focus on the analysis of the main bridge, but the analysis of the approach bridge and other analyses are performed in a similar way.

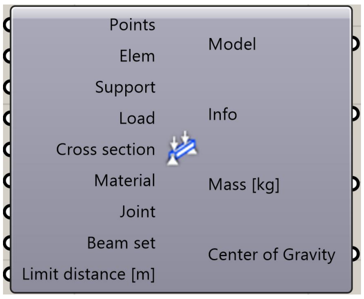


Figure 4.12: The "Assemble Model" component in Karamba.

When all the elements and relations are created, they can be assembled into one model with the "Assemble Model" component, see Figure 4.12.

The next step is the calculation of the model with the "Analyse ThI" component. This component calculates the force distribution and the deflections of the model using first order theory for small deflections.[25] With this force distribution, the amount of prestressing is determined in phases (first the cantilever prestressing, subsequently the continuity prestressing) and the prestressing is added to the model as point-loads and bending moments. The following moment lines from the model are the results of the different phases and prestressing components, see Figures 4.13 & 4.14. The moment values in the figures are based on a bridge with a span of 160 meters and a width of 22 meters.

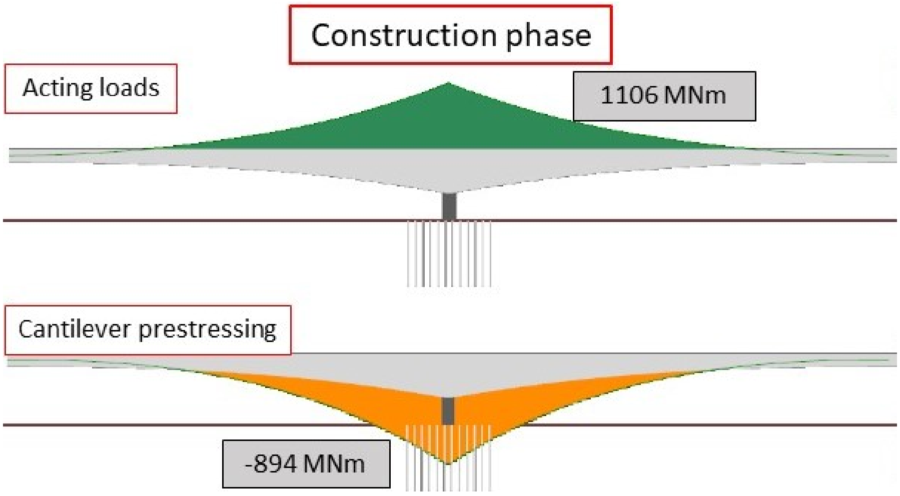


Figure 4.13: Momentlines during construction phase.

Construction phase, cantilever prestressing

During the construction phase, only the self-weight of the bridge is present. Although, for the determination of the maximum bending moment, also the variable load and the superimposed dead loads are used which are applied in the end phase. This maximum bending moment, $M_{max} = M_{selfweight,constructionphase} + M_{superimposeddeadloads} + M_{variable}$, is used to calculate the amount of cantilever prestressing cables with use of the "cross-section method", see Equation 4.1.

$$N_{wp.cantilever}[kN] = \left(\sigma_c - \frac{\gamma_g * M_{ed}}{W_{top}}\right) / \left(\frac{-e_{pt}}{W_{top}} - \frac{1}{A_c}\right) \tag{4.1}$$

The equation above, is written in the "Expression" component in Grasshopper, this component allows the adding of user defined formula's and calculations. In the equation, M_{ed} is the maximum acting moment, W_{top} is the section modulus for the top of the cross-section, σ_c is the allowed concrete stress, γ_g is the load factor equal to 1.0, e_{pt} is the eccentricity of the cantilever prestressing cables and A_c is the area of the concrete cross-section. All values except σ_c and γ_g are taken over the length of the bridge, so they are a function of x. As a result, the required cantilever prestressing force per meter length $N_{wp.cantilever}$ is calculated. This force is applied in the next stage and here also the continuity prestressing is determined.

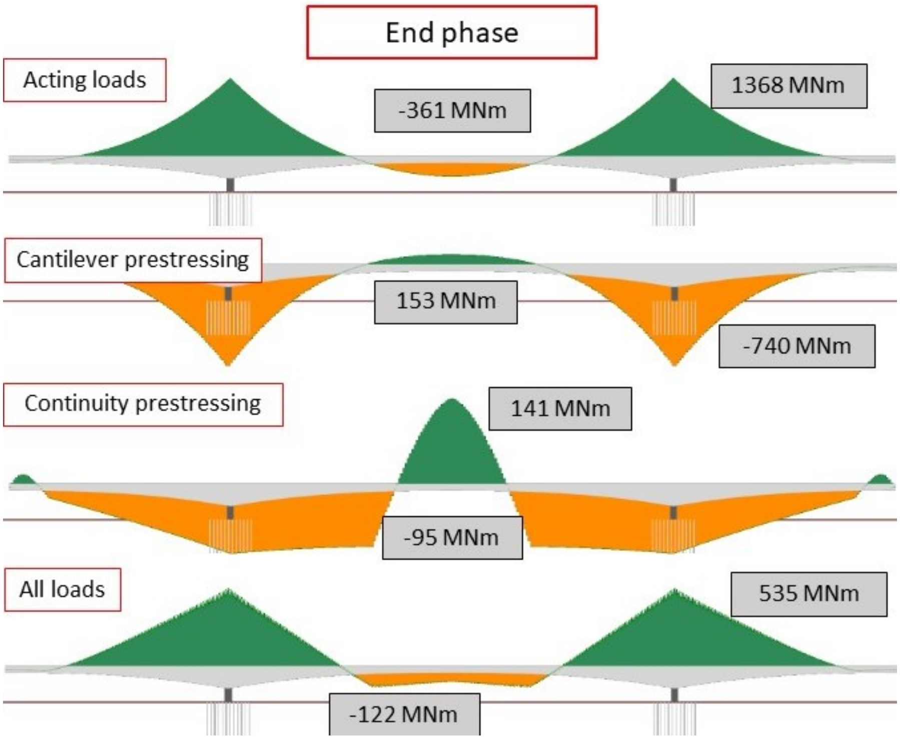


Figure 4.14: Momentlines in the end phase.

End phase, continuity prestressing
When the cantilever prestressing is determined and added to the model, at midspan of the bridge still tension does occur in the bottomslab due to variable loads and the change of statical system. See Figure 4.15. This tension should be taken up by the continuity prestressing.

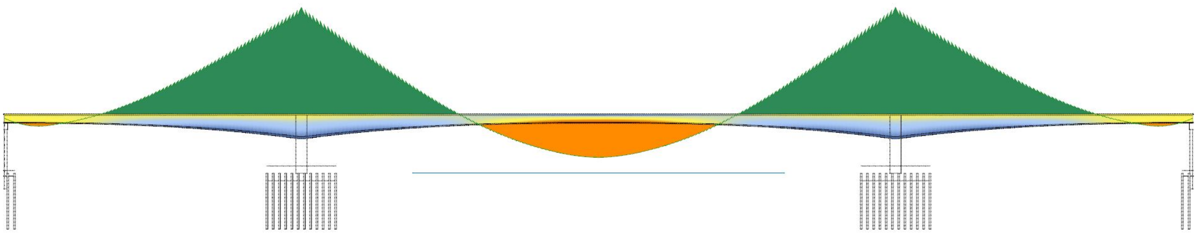


Figure 4.15: The moment distribution after application of the cantilever prestressing.

From the moment distribution above, only the negative bending moments (orange) are used for determining the required continuity prestressing force. This force will be applied in the bottomslab of the bridge. Equation 4.2 is used to calculate this required continuity prestressing force $N_{wp.continuity}$ per meter length.

$$N_{wp.continuity}[kN] = \left(\sigma_c + \frac{\gamma_g * M_{ed}}{W_{bot}}\right) / \left(\frac{-e_{pb} * RF}{W_{bot}} - \frac{1}{A_c}\right) \tag{4.2}$$

In this equation, M_{ed} is the resulting negative bending moment from the previous phase, W_{bot} is the section modulus for the bottom of the cross-section, σ_c is the allowed concrete stress, γ_g is the load factor equal to 1.0, e_{pb} is the eccentricity of the continuity prestressing cables, A_c is the area of the concrete cross-section and RF is the earlier described reduction factor (Chapter 3.3.1).

By using the "Beam Forces" component, the resulting internal forces in the structure can be determined for different load cases. These values are used for the determination of the final acting moment (M_{ed}), by implementing Equations 4.3 & 4.4 into the expression component in Grasshopper. These equations are earlier described in Chapter 3.3.1 . This M_{ed} is used in the next subsection for the moment capacity check.

$$M_{ed}(6.10a) =$$
$$(\gamma_{G1} * M_{sw,c} + M_{pcan,c}) * e^{-\phi} + (\gamma_{G1} * M_{sw,end} + M_{pcan,end}) * (1 - e^{-\phi}) + \gamma_{G1} * M_{sdl} + \psi * \gamma_Q * M_{var} + M_{pcon} \tag{4.3}$$

$$M_{ed}(6.10b) =$$
$$(\gamma_{G2} * M_{sw,c} + M_{pcan,c}) * e^{-\phi} + (\gamma_{G2} * M_{sw,end} + M_{pcan,end}) * (1 - e^{-\phi}) + \gamma_{G2} * M_{sdl} + \gamma_Q * M_{var} + M_{p,con} \tag{4.4}$$

In these equations, $M_{sw,c}$ and $M_{pcan,c}$ are the bending moments due to self-weight and cantilever prestressing during construction. $M_{sw,end}$ and $M_{pcan,end}$ are the bending moments due to self-weight and cantilever prestressing at the end phase. M_{sdl} is the moment due to superimposed dead-loads, M_{var} is the moment due to load model 1 and $M_{p,con}$ is the moment due to the continuity prestressing these three components are only present at the end phase.

Another type of analysis which is also performed by the model, is the determination of the material amounts for the various construction elements. These material amounts are coupled to unit prices during the result phase where the total costs and shadow costs are calculated, see Section 4.2.5.

4.2.4. CHECKING

After the analysis of the bridges, in this subsection the bridges are tested on capacity (ULS), serviceability (SLS) and fitness.
For the capacity checks, the material properties and dimensions from the input phase are used for the resisting moment of the bridge, while the final force distribution from the model is used for the acting moment. Furthermore there is also performed a buckling check for the columns, but since this is not taken into account for the automatic optimization of the bridge, this will not be further explained in this section.
The SLS check that is performed by the model, is the deformation check of the main bridge. And the fitness checks are split-up into; the placement of the required amount of prestressing tendons in the deck- and bottomslab and the boundary condition that the bridge stays out of the clearance gauge.

Moment capacity check
The first check that is performed by the model, is the Moment capacity check. For this check, the resulting normal force from the model is used to determine the height of the compression zone X_u by using horizontal force equilibrium, $\sum F_h = 0$. With this compression zone the moment capacity (M_{rd}) is calculated for each meter of the bridge.

Since the acting moment is also known from the analysed model, it is possible to perform a unity check by dividing the acting moment by the resisting moment, $\frac{M_{ed}}{M_{rd}} \leq 1.0$. See Figure 4.16 for the above described process in the model.

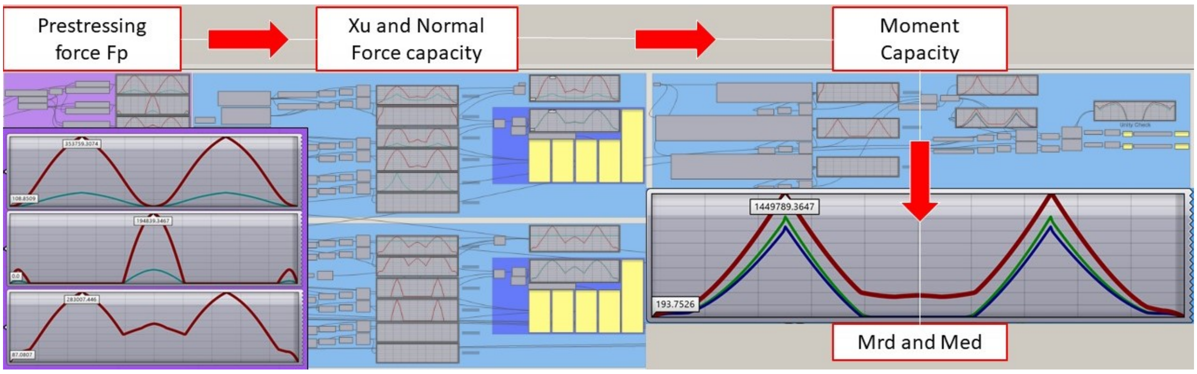


Figure 4.16: Moment capacity check, as performed by the parametric model.

In the graph at the right side of the figure, the red line is the M_{rd} and the green and blue line are the M_{ed} as determined with the two different load combinations for the ultimate limit state. (see also Figure **??**)

Deformation check
For the deformation check, the deflection of the main bridge is determined with help of Karamba. For this check there is assumed the following; the deflections during the construction of the bridge, so due to the self weight and superimposed loads are already compromised for. This means that the bridge is build with a slight camber and is perfectly straight at the time that the bridge is in use. So the only load that is causing the bridge to deflect is the variable load.

When the variable load is added to the bridge, the "AnalyseThI" component gives a maximum displacement in meters. This value is compared with the deflection limit of $\frac{L_{span}}{500}$ and when the displacement exceeds this limit, a penalty is given. See Figure 4.17.

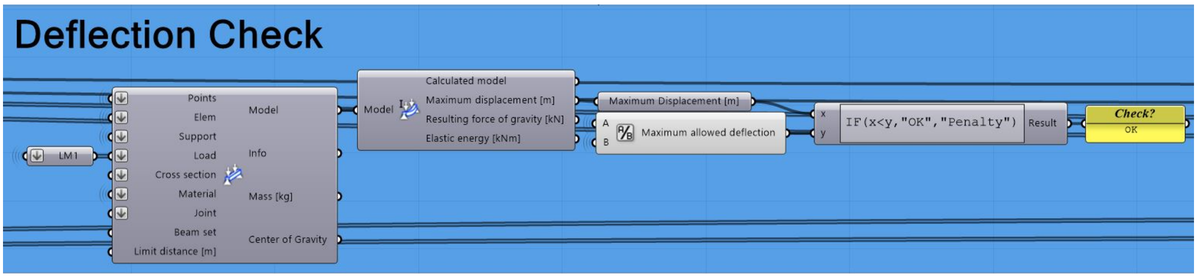


Figure 4.17: Deformation Check, as performed by the parametric model.

Fitness checks

The first fitness check is performed right after the determination of the amount of prestressing cables during the analysis stage. Since from the manufacturers, the dimensions and the centre-to-centre distance of the ducts is known, the required width for the maximum amount of cables can be determined. This check is performed for the deck, so the cantilever cables, and for the bottomslab, the continuity cables.

When the required width exceeds the available width, once again a penalty is given. See Figure 4.18 for a visualization of the "Cable Fitness Check" in the parametric model.

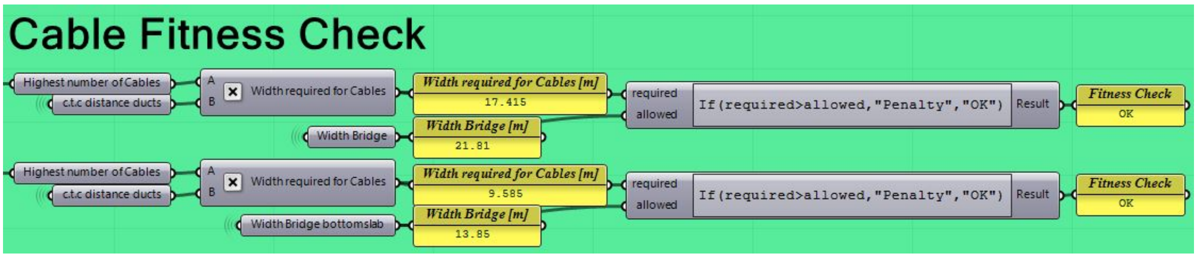


Figure 4.18: Cable Fitness Check, as performed by the parametric model.

The other fitness check is the "Clearance Check". With this check there is verified whether the bridge stays out of the clearance gauge. Therefore the z-coordinate of the critical point is determined. This point is at the bottom of the bridge at the outer ends of the clearance width.
This value is compared with the height of the clearance and when the z-coordinate is smaller, a penalty is given, see Figure 4.19.

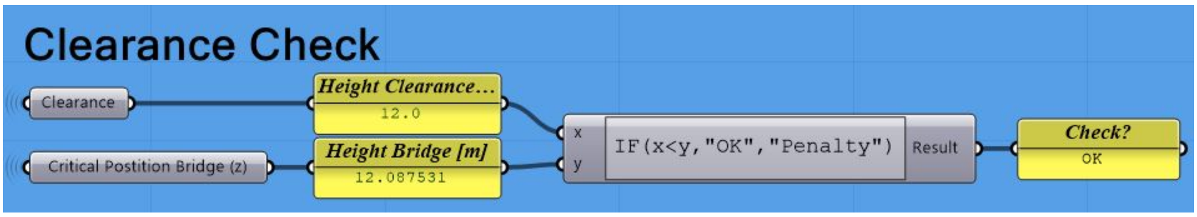


Figure 4.19: Clearance Check, as performed by the parametric model.

Penalty system

For each requirement that is not fulfilled by the design, a penalty is given to the total costs. The height of the penalty depends on the expected project costs. This penalty system is used during the optimization process to filter out the non-satisfying designs. This will be explained in detail in Chapter 5.

4.2.5. RESULTS

In the result phase of the parametric model, the total material amounts are determined and this is used to determine the costs and shadow costs.

The first step is analysing the different elements and calculating the material amounts per element, see Figure 4.20.

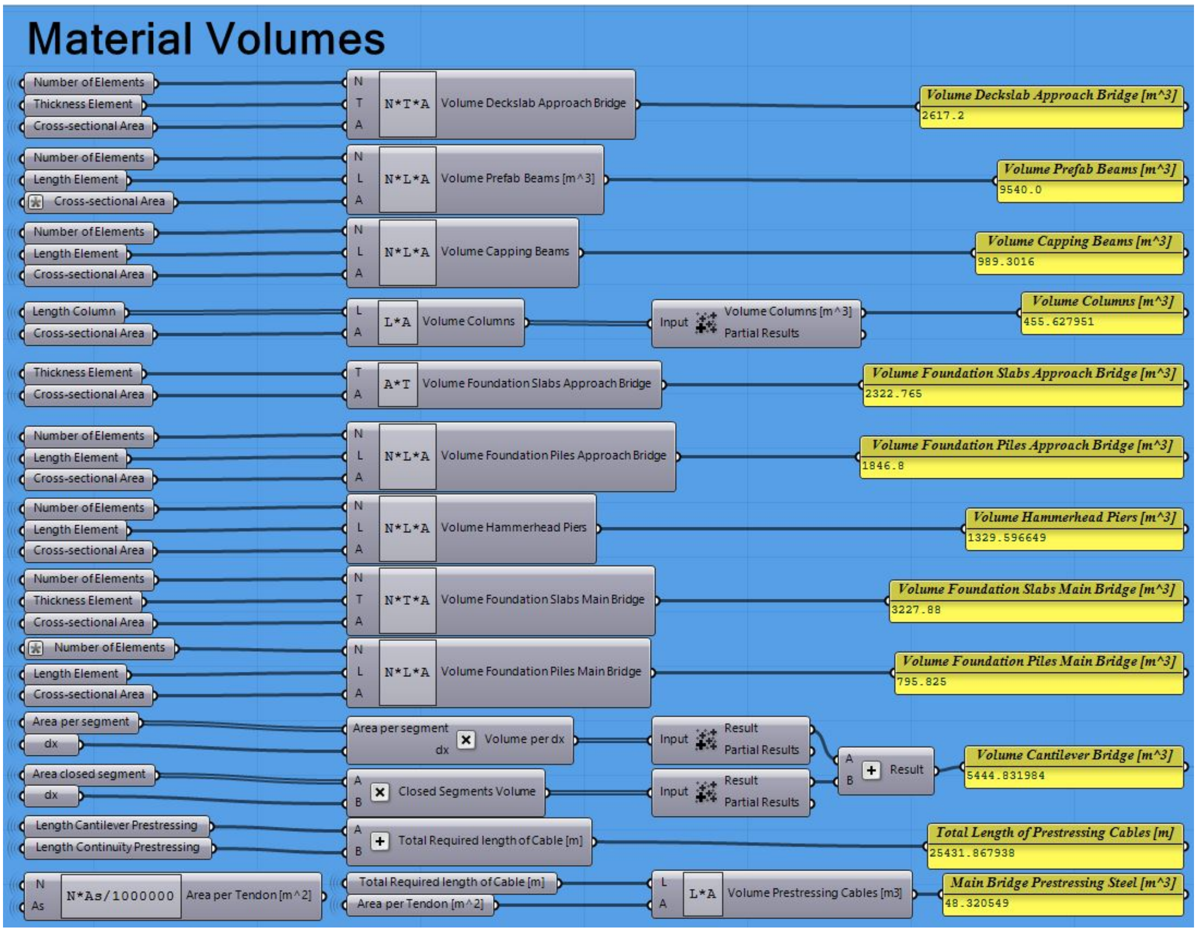


Figure 4.20: Calculation of the material amounts per element

Subsequently, these material amounts are bundled per concrete class or type of steel (reinforced or pre-stressed), see Figure 4.21.

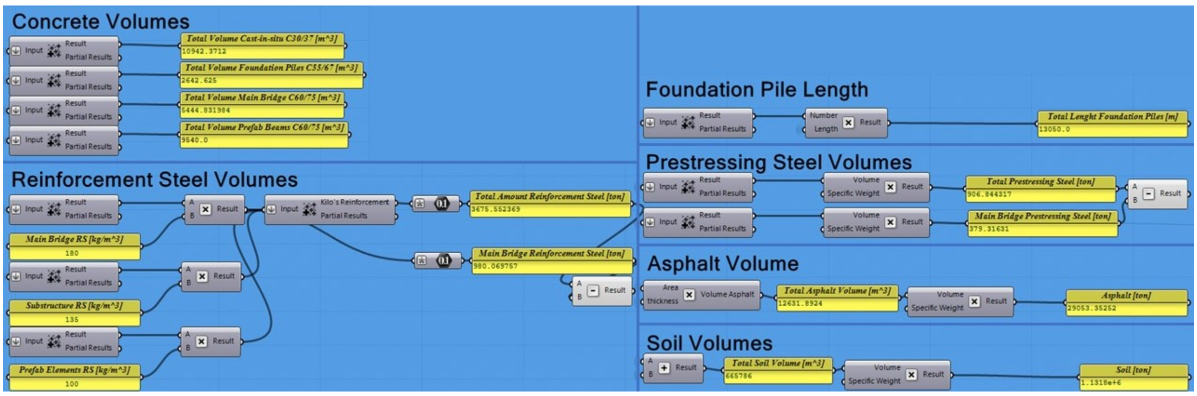


Figure 4.21: Calculation of the material amounts per type.

With the unit prices, the model now is able to calculate the total project costs and the environmental impact, see Figure 4.22 & 4.23.

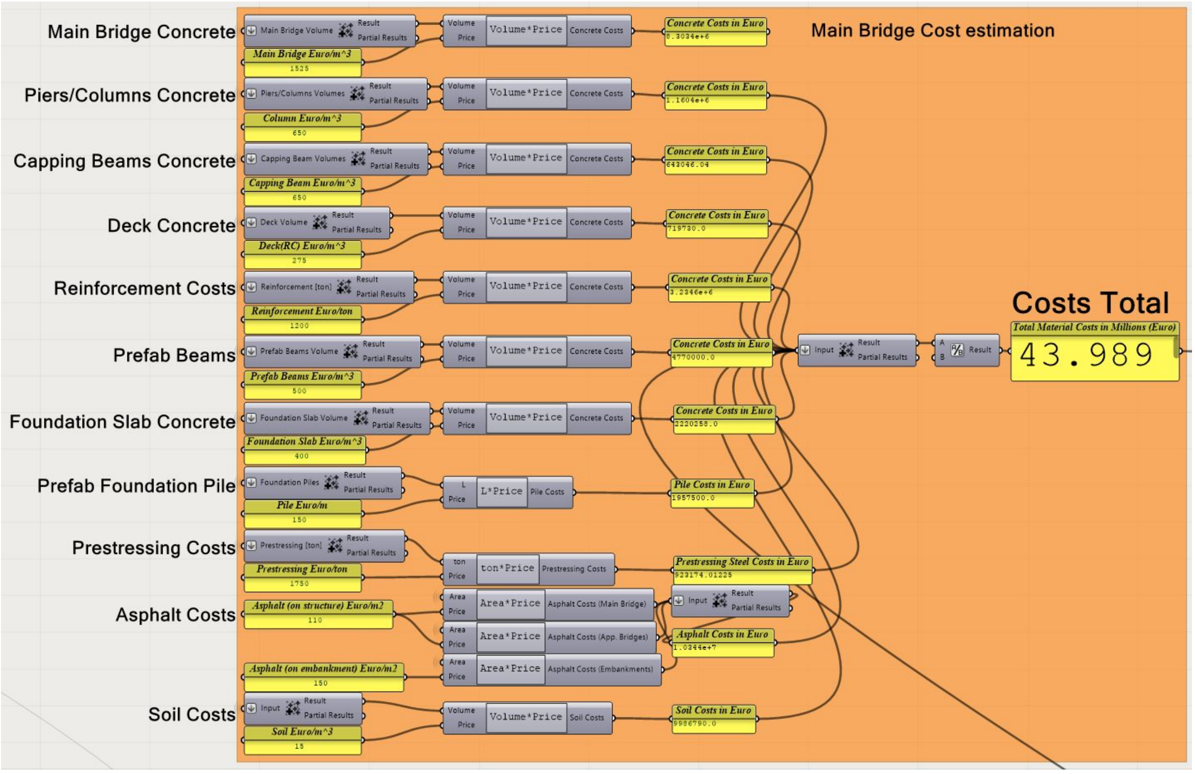


Figure 4.22: Calculation of project costs based on the material amounts.

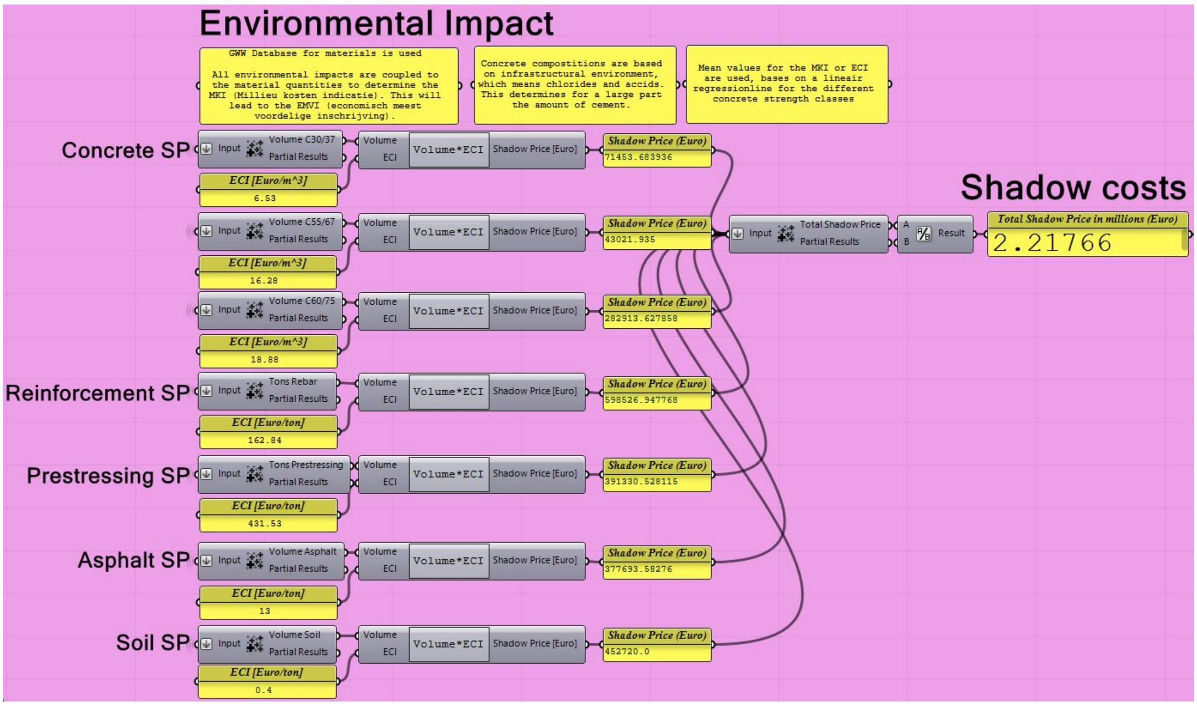


Figure 4.23: Calculation of environmental impact or shadow costs based on the material amounts.

4.2.6. OPTIMIZATION

The optimization process will be described in detail in Chapter 5. In this section only the layout of the optimization process in the parametric model is explained, see Figure 4.24.

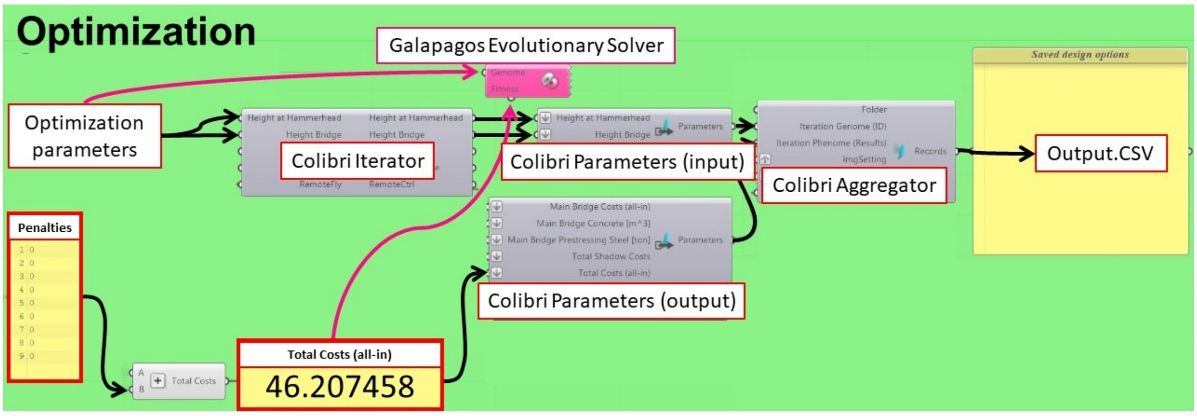


Figure 4.24: The optimization process in the parametric model.

In the figure above, a pink component can be seen, this is the "Galapagos Evolutionary Solver". This is the component that does the optimization of the main bridge.
The component has two input parameters. One the "Genome", here the optimization parameters are linked to, and two the "Fitness", this is the parameter that must be optimized, in this case the Total costs. Although the Galapagos optimization can only be towards one parameter, in the Colibri components, also other result parameters can be added.

The "Colibri Iterator" gathers the optimization parameters and performs the same iterations as the Galapagos component, loops them over and transfers the data through the Parameters component to the Aggregator. This component collects also the output parameters and processes this in a CSV-file. This CSV-file can be used by Design Explorer to visualize the iteration results and to filter the bad performing designs out of the collection. In this way, the optimal solution can be found, this is called the preliminary design.

4.3. VISUALIZATIONS

In this section, the visual output of the model will be highlighted. The same order as previously used will be handled, so starting with the main bridge, subsequently the approach bridge, substructure and foundation.

Main Bridge

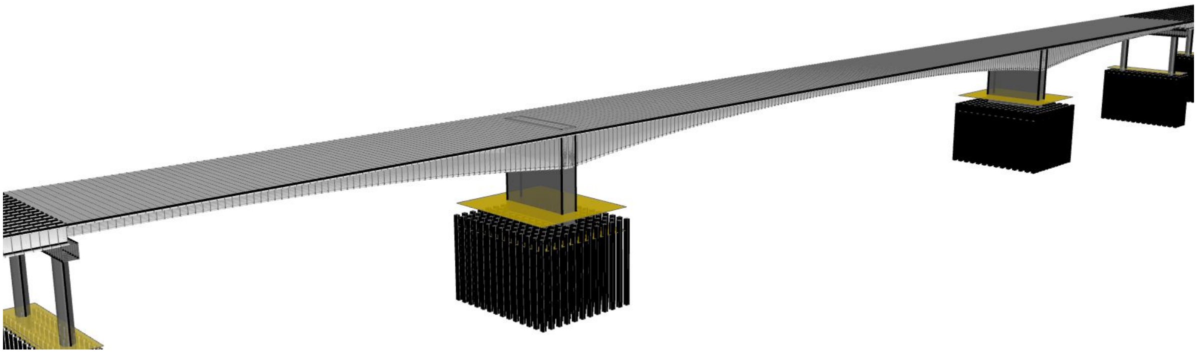


Figure 4.25: Perspective view of the main bridge.

Figure 4.26: Front view of the main bridge.

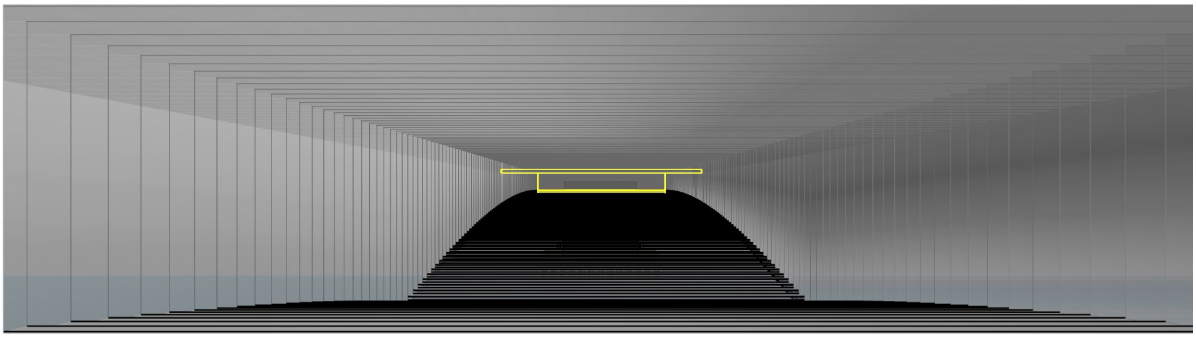


Figure 4.27: Inside view of the main bridge.

Approach Bridge

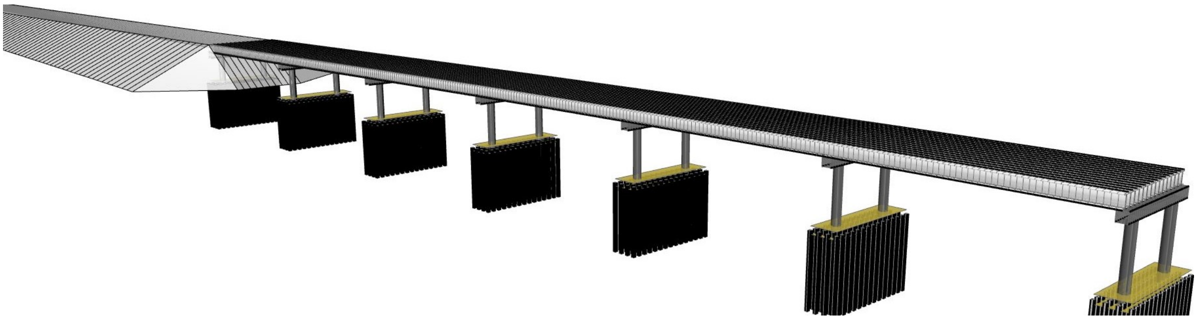


Figure 4.28: Perspective view of the approach bridge with substructure.

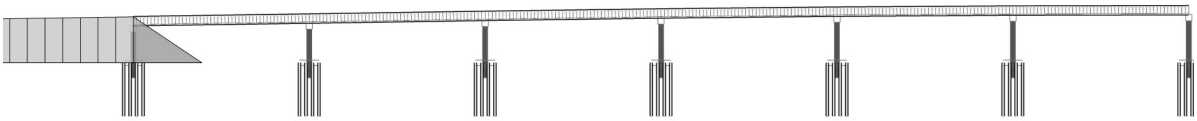


Figure 4.29: Front view of the approach bridge with substructure.

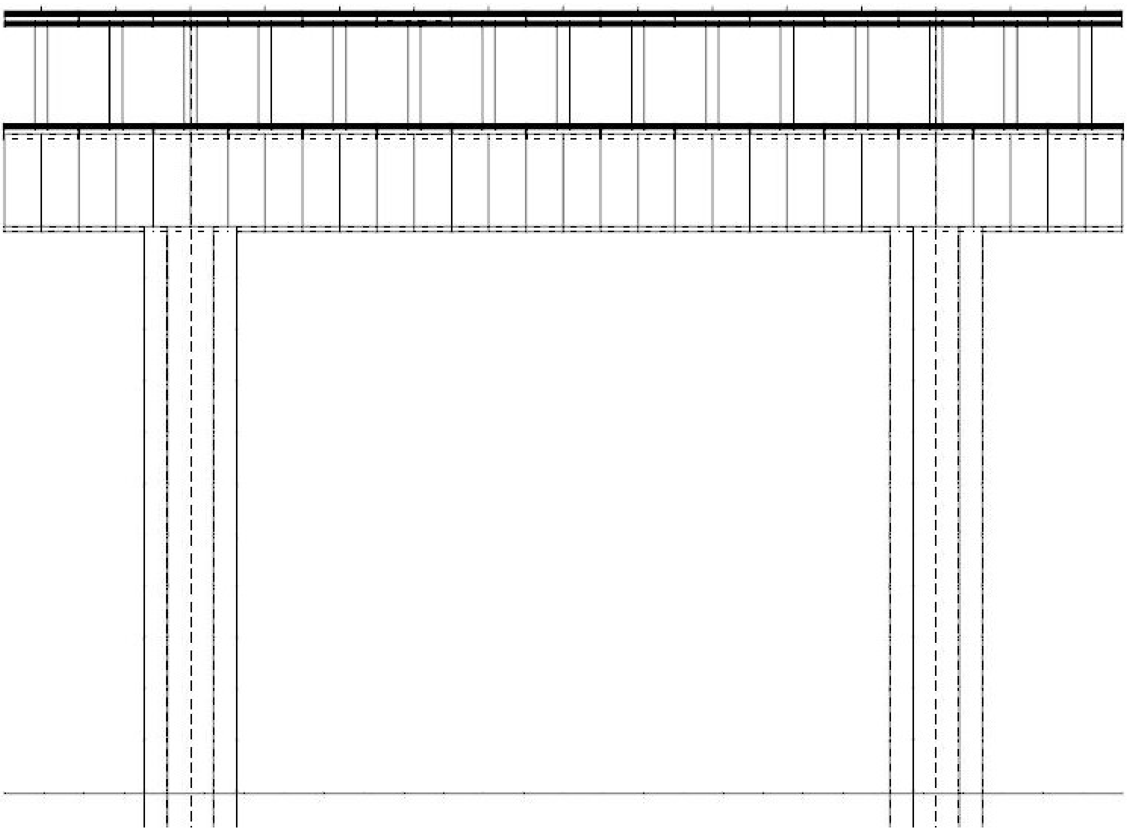


Figure 4.30: Cross-sectional view of the approach bridge with substructure.

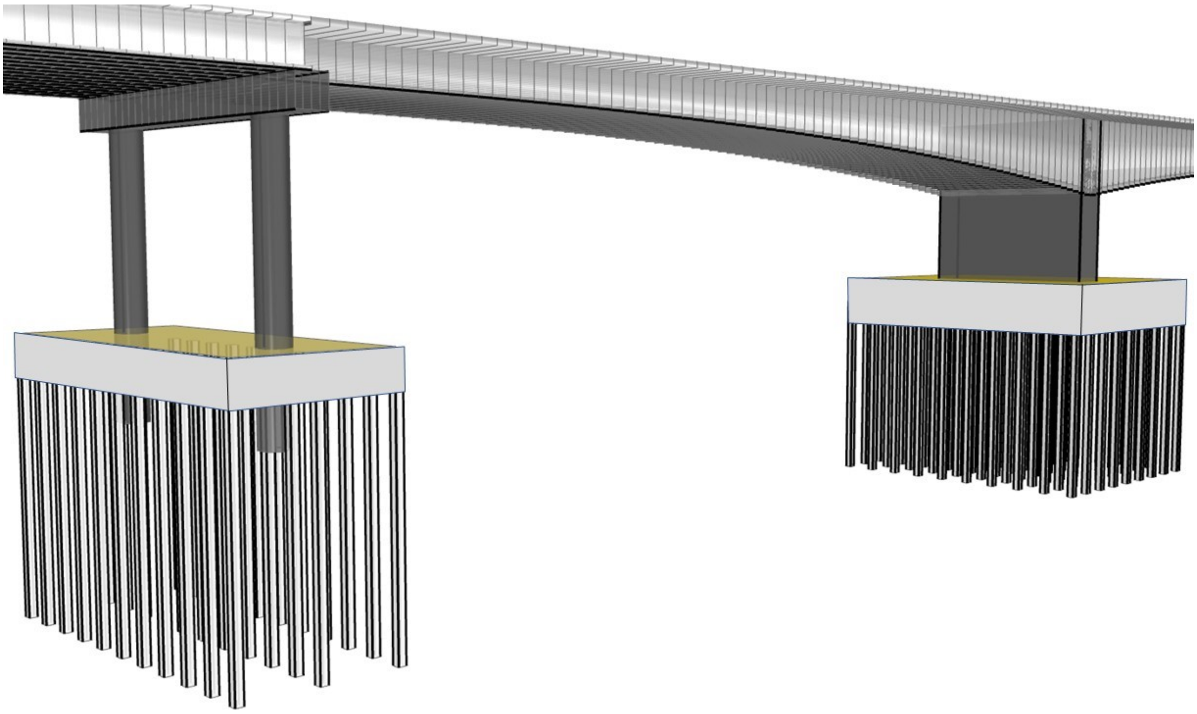


Figure 4.31: Ground-level perspective view of the combined structure.

Substructure

Figure 4.32: Perspective view of the substructure.

Foundation

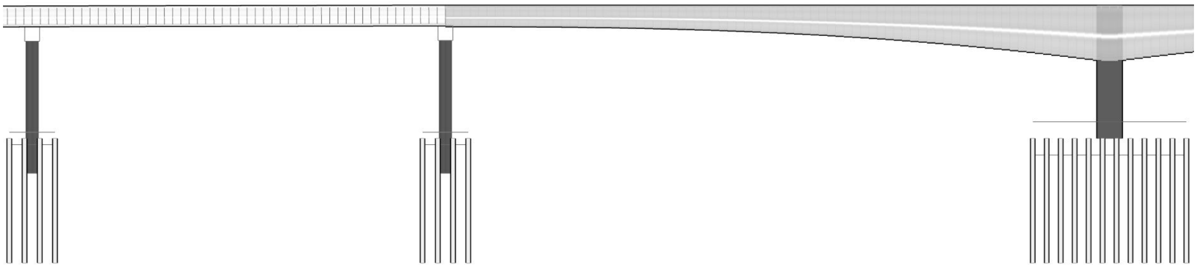


Figure 4.33: Front view of the foundation with pile centre-to-centre distance.

Figure 4.34: Top view of the foundation with pile grid.

5

OPTIMIZATION

In this chapter, the optimization process for this specific project will be explained, supported by illustrations of the workflow. Furthermore two existing bridges will be recreated by the model and by optimizing these bridges, the effectiveness of the model will be judged. The results of this redesigning process will be made visual in Chapter 6. The subquestions for this chapter are:

5.1 How can the bridges be optimized to reduce costs?
5.2 How to redesign and optimize existing bridges by using the parametric model?

5.1. OPTIMIZATION PROCESS

In this section there will be a brief definition of the general idea behind optimizing and subsequently the optimization process for the bridge designs will be explained.

5.1.1. GOALS

The main goal of the optimization phase is to minimize the costs and the environmental impact of the various designs. This will be done by optimizing the designs on the total costs, these are the costs of the material including the shadow costs. Another option is to optimize on material amounts or environmental impact, but this will not be performed in this project.

A definition of the word optimization is:

> "The process of making something as good or effective as possible"[1]

When this definition will be applied to the parametric bridge design model, first there should be investigated where the biggest costs advantage can be gained. Therefore, the total road design must be thoroughly examined, see Figure 5.1.

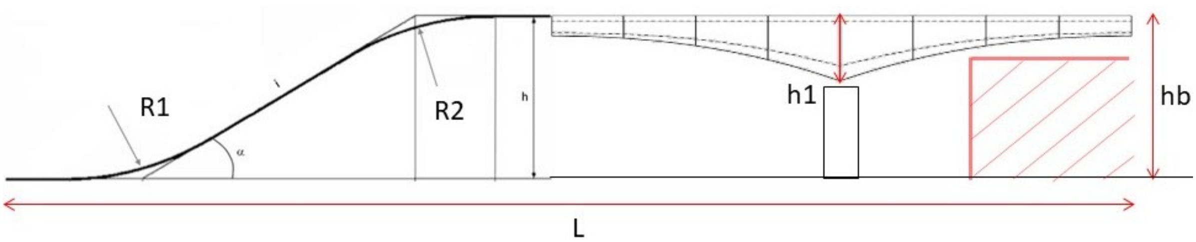


Figure 5.1: General road design, based on the ROA (Richtlijn Ontwerp Autosnelwegen)[7] including the optimization parameters for the total design.

As earlier described in Chapter 2.2, the height of the vertical alignment determines the length of the alignment since the radii of the alignment are limited. So when the height of the alignment can be minimized the

[1] Definition achieved from Cambridge Dictionary, www.dictionary.cambridge.org/dictionary/english/optimization

length of the total road design is also minimized, ensuring the biggest costs advantage.

Since the height of the road alignment depends on the structural height of the main bridge, this is the parameter which should be optimized.
Simply put, when the structure of the main bridge is optimized by minimizing the structural height, the length of the approach bridges can be smaller, this causes a lower material usage, so the total costs will also drop.

Design graphs
Another goal during the optimization stage is, to investigate the impact of the span length on the structural height for different bridge widths. By optimizing the structural height in the model for different set-ups, it is possible to create "Design graphs". With these graphs, the user of the model is able to know the approximate structural height of the bridge before starting the model. This could speed-up the design process, since the user is able to limit the range for the optimization process. The design graphs can be found in Chapter 6.1.3.

5.1.2. OPTIMIZATION PARAMETERS
The main goal is to minimize the structural height of the main bridge, but to stay out of the clearance envelop and suffice all other checks as described in Chapter 3.4. This will have the largest influence on the ***total design***. But first, there will be an optimization for the ***main bridge only***, so without the approach bridges and substructure.

Main bridge only
For this optimization there will be four parameters, namely the construction height at hammerhead (**h1**) and midspan (**h2**) and the bottom slab thickness at these locations (**t1** and **t2**). See Figure 5.2. The other cross-section parameters are standard values. The goal of this optimization is to reach the structural limit of the bridge. Also this optimization process is used to determine the optimal dimensions for the thickness of the bottom slab and the structural height of the bridge at midspan. These three parameters are standardized for the optimization of the total design, to limit the amount of variables and to speed-up the process.

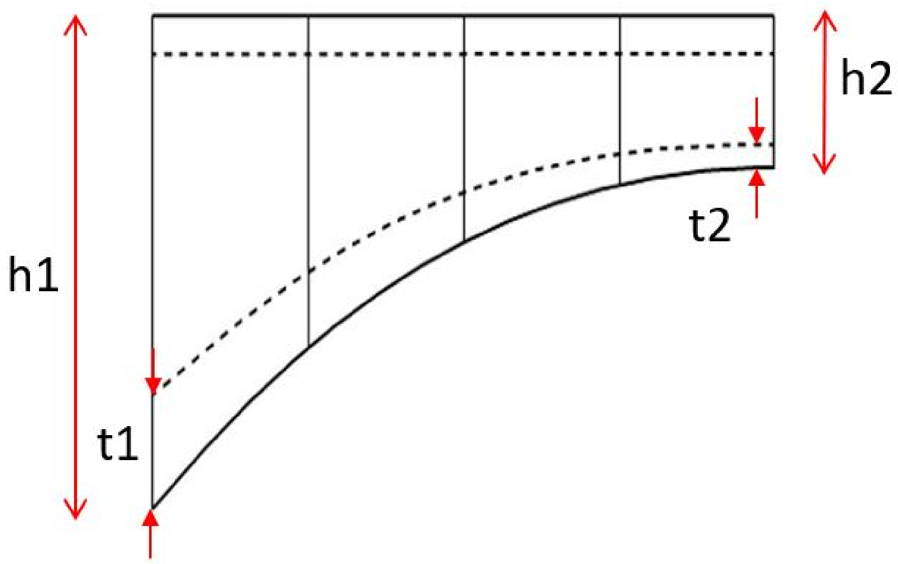


Figure 5.2: Optimization parameters for the main bridge only.

Total design
The two parameters which are used for the total design optimization, are the height of the box girder at the location of the hammerhead (**h1**) and the overall height of the bridge (**hb**), which guarantees a free clearance. (See Figure 5.1)
With this optimization, the total project costs are determined based on the material amounts and the influence of the structural height on the costs is investigated.

5.1.3. FITNESS FUNCTION

Since the goal is to minimize the total costs, it is necessary to investigate the fitness of the designs and how this fitness function is build-up.

The total costs consists of the material costs and the environmental costs, both based on the same amount of material. This gives the following function, see Equation 5.1.

$$Total\ costs = \sum(Volume_{material\ i} * (Unit\ Costs_{material\ i} + Shadow\ Costs_{material\ i}) \qquad (5.1)$$

In this equation, all material amounts are taken into account but some of the materials effect the amounts of others. For instance; the amount of concrete is inversely proportional to the amount of prestressing steel in the main bridge. When the cross-section of the bridge enlarges for a specific span length, so the amount of concrete increases, the required amount of prestressing decreases. So, a smaller structural height means more prestressing and a large structural height means less prestressing. Therefore it is not completely true that: the smaller the construction height, the lower the costs for the main bridge.

For the total project costs, it generally applies that the lower the structural height the lower the costs, because the height positively effects the amount of material in the approach bridges and embankments.

5.2. REDESIGNING

Part of the optimization process consists of the redesigning of two real bridges, namely the Stichtsebrug II (L=320 m) and the Dintelhaven bridge (L=384 m). Both bridges with and without approach bridges. In this section, the approach of the redesigning process for both bridges is described, along with visualisations from the optimization process. The results of the optimized bridges are compared with the existing bridges when entered in the parametric model. This ensures a fair comparison.

Before describing the redesigning process, there must be noted something about the Stichtsebrug II. This bridge is known as a structural masterpiece and is the first balanced cantilever bridge, built with high strength concrete. Since this bridge is designed "on the edge" and the dimensions are minimized, this bridge is used to set a boundary for the deflection limit in the parametric model.

Deflection limit

The deflection limit is determined by entering the dimensions of the Stichtsebrug II into the parametric model and calculating the deformation due to the variable loads (LM1). This deformation is used to determine the deflection limit, $w_{max} = \frac{L_{span}}{x}$.

From the model followed a deflection of 0,22 meter and the bridge has a span length of 160 meters, this gives $x = \frac{L_{span}}{w_{max}} = \frac{160}{0,22} = 727$. To stay on the conservative side, a deflection limit of $w_{max} = \frac{L_{span}}{700}$ will be used in the model.

General set-up

With the redesigning of the two bridges, the goal is to verify the parametric model and to check the performance of the already built bridges. After the optimization, the optimized bridge will be compared with the existing bridge and eventual differences will be discussed.

The redesigning of the bridges will be done in two different ways, as described in the previous section; one by searching for the structural limit of the main bridge only and two by optimizing the bridge in the total design. For the first set-up, all bridge cross-section parameters will be copied from the existing bridge, but the construction height and thickness of the bottom slab, at hammerhead and midspan will be variable.
For the second set-up, only the height of the bridge and the construction height at the hammerhead are variable. The height at midspan will be taken as 2,5 meters and the thickness of the bottom slab has a gradient from 0,6 meters at the hammerhead to 0,25 meters at midspan. These values are based on standard applied dimensions from existing bridges in the Netherlands and based on the result of the "main bridge only" optimization. For an overview of the balanced cantilever bridges in the Netherlands, see Appendix H.

After the two optimizations, the difference will be discussed at the end of the subsections.

From the optimization, the total costs and environmental impact, along with the dimensions of the optimized design will be described in Chapter 6.1.1.

5.2.1. STICHTSEBRUG II

The optimization of the Stichtsebrug II, is performed in two different takes, the main only and the total design. In this section, the set-up of the redesigning and optimization process of the Stichtsebrug II is described along with visualisations of the parameters. The bridge cross-section parameters are copied and entered into the model and for the clearance gauge, a width of 100 meters is chosen with a height of 12 meters.

Optimization "main bridge only"

The input parameters for the Stichtsebrug II can be seen in Table 5.1 and are visualized in Figure 5.3. The results of the existing bridge are produced by the parametric model and are only to compare the performance with the optimized structure, this will be done in Chapter 6.1.1.

Table 5.1: Input and output of the Stichtsebrug II in the parametric model

Input	**Stichtsebrug II**
Optimization Parameters	
Height at Hammerhead	6,75 m
Height at Midspan	2,5 m
Thickness Bottomslab Support	0,55 m
Thickness Bottomslab Midspan	0,22 m
Fixed Parameters	
Length Bridge	320 m
Width Bridge Deck	21,81 m
Width Cantilever	3,98 m
Thickness Webs	0,32 m
Number of Cells	2
Thickness Deck	0,36 m
Deflection Limit	L/700
Results from parametric model	
Main Bridge Concrete	5445 m^3
Main Bridge Prestressing Steel	379 ton
Shadow Costs (in million Euro)	0,452
Total Costs (in million Euro)	8,706

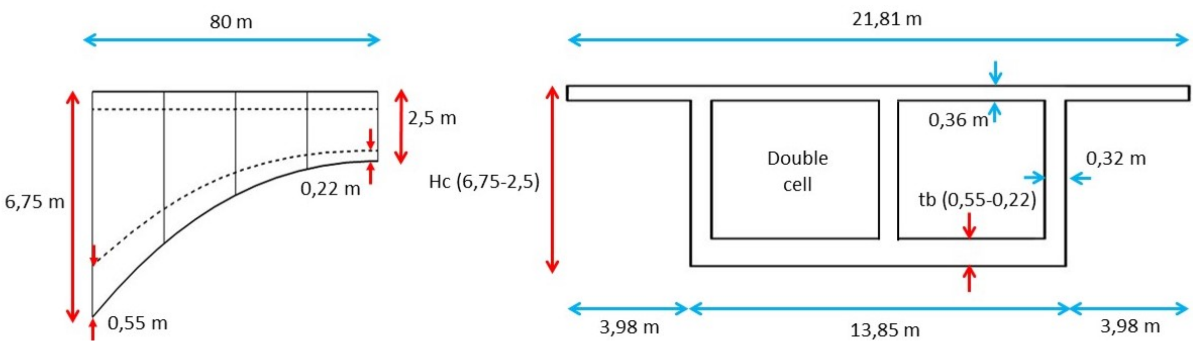


Figure 5.3: Input parameters Stichtsebrug II, with in red the variable parameters.

As a visual representation of the optimization process, the results of the different iterations are shown in the Parallel Coordinates Plot (PCP) made with Design Explorer, see Figure 5.4. The highlighted red line, is the Stichtsebrug II with the existing dimensions. In the next chapter, the optimized design will be discussed.

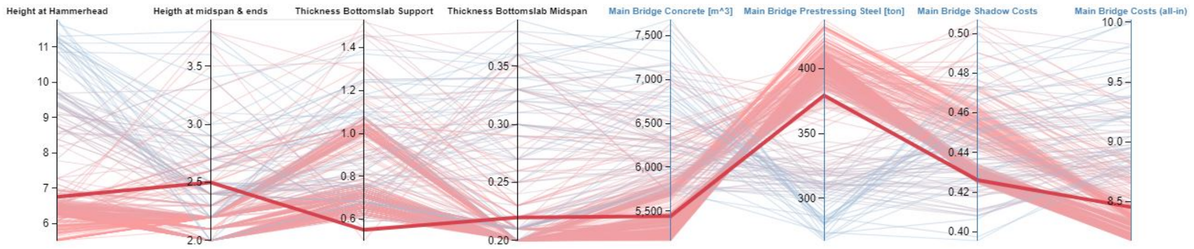


Figure 5.4: Parallel Coordinates Plot of the first variant study for the main bridge. Source: Design Explorer | CORE studio http://tt-acm.github.io/DesignExplorer/?ID=jizyeq

Optimization "total design"

The input parameters for the total optimization of the Stichtsebrug II can be seen in Table 5.2 and are visualized in Figure 5.5. Furthermore, the same settings will be used compared to the previous optimization process. Note: the thickness of the bottomslab for the optimization process differs compared to the existing bridge. This is based on the standard minimal values in practice.

Table 5.2: Input and output of the Stichtsebrug II in the parametric model

Input	**Stichtsebrug II**
Optimization Parameters	
Bridge Height	13,5 m
Height at Hammerhead	6,75 m
Fixed Parameters	
Height at Midspan	2,5 m
Thickness Bottomslab Support	0,55 m
Thickness Bottomslab Midspan	0,22 m
Length Bridge	320 m
Width Bridge Deck	21,81 m
Width Cantilever	3,98 m
Thickness Webs	0,32 m
Number of Cells	2
Thickness Deck	0,36 m
Deflection Limit	L/700
Results	
Main Bridge Concrete	5445 m^3
Main Bridge Prestressing Steel	379 ton
Main Bridge Costs (million Euro)	8,706
Total Shadow Costs (million Euro)	2,218
Total Design Costs (million Euro)	46,207

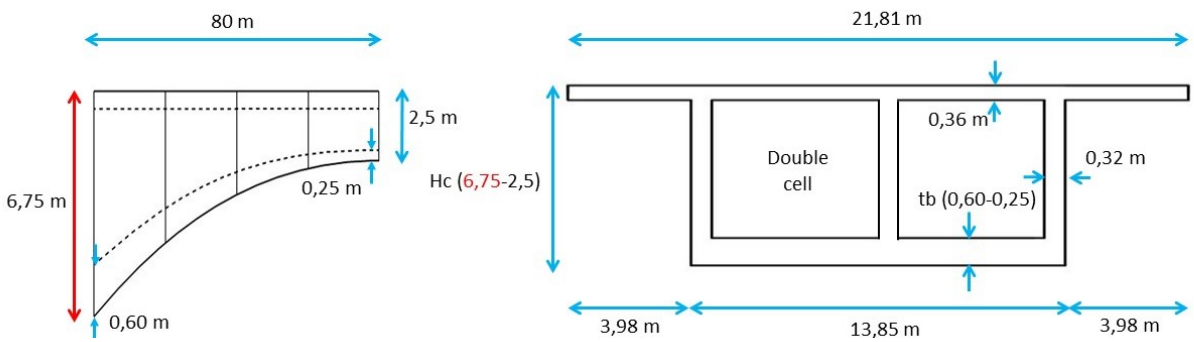


Figure 5.5: Input parameters Stichtsebrug II, with in red the variable parameters.

In Figure 5.6, the iterations of the optimization process are visible. In this optimization process, the approximate dimensions of the bridge were already known from the previous optimization, therefore there is only searched around these values and not randomly. The highlighted line is the existing bridge with a total costs of 46,2 million Euro.

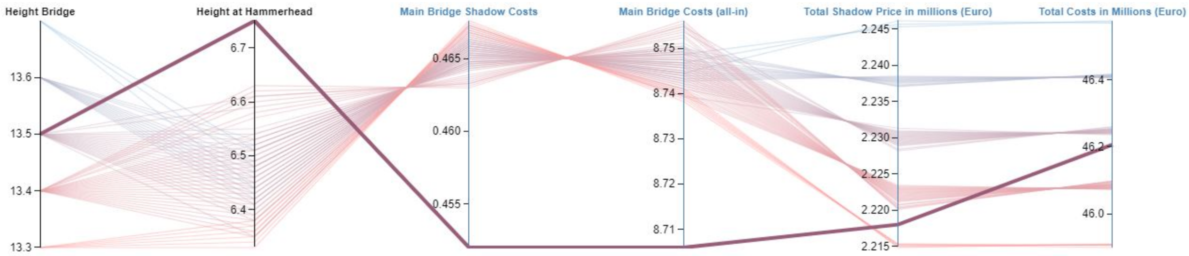


Figure 5.6: Parallel Coordinates Plot of the first variant study for the total bridge design. Source: Design Explorer | CORE studio

5.2.2. DINTELHAVEN (EAST)

The optimization of the Dintelhaven Bridge (east), is performed in two takes. One optimization for only the main bridge and the other optimization for the total design, with a minimal height of 3,0 meters for the midspan cross-section. For the clearance envelope, a width of 150 meters is assumed with a height of 12 meters.

Optimization (Main Bridge Only)

For the main bridge optimization almost the same set-up is used as for the Stichtsebrug II. Only the clearance envelope is different. For the input parameters and results, see Table 5.3 and Figure 5.7.

Table 5.3: Input and output of the Dintelhaven (East) bridge in the parametric model

Input	**Dintelhaven (East)**
Optimization Parameters	
Height at Hammerhead	8,50 m
Height at Midspan	3,0 m
Thickness Bottomslab Support	0,60 m
Thickness Bottomslab Midspan	0,25 m
Fixed Parameters	
Length Bridge	384 m
Width Bridge Deck	22,15 m
Width Cantilever	3,6 m
Thickness Webs	0,38 m
Number of Cells	2
Thickness Deck	0,43 m
Deflection Limit	L/700
Results from parametric model	
Main Bridge Concrete	8108 m^3
Main Bridge Prestressing Steel	623 ton
Shadow Costs (in million Euro)	0,694
Total Costs (in million Euro)	32,161*

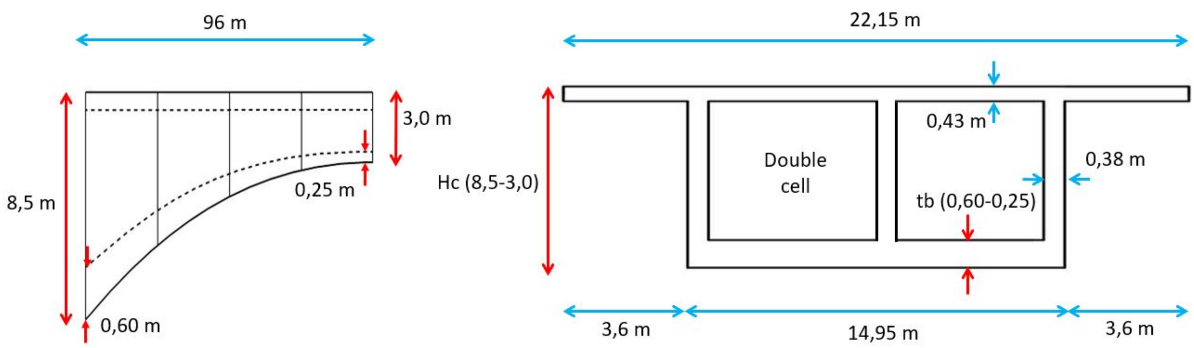


Figure 5.7: Input parameters Dintelhaven (East), with in red the variable parameters.

Remarks

As can be seen in the last row of the table, there is an extremely high total costs for the existing Dintelhaven (east) bridge. This is a result of a penalty of 20 million Euro which is added to the design costs. The penalty is given for the number of cables which do not fit in the deckslab; a width of 23,625 meters is required while there is only 22,15 meters available. This problem can be solved by putting the cables partly in two rows in the deckslab or placing some cables in the webs of the bridge. The resulting costs for the main bridge are 12,161 million Euro.

For the optimization of this bridge there is decided to ease the fitness check for the required width, because the small amount of cables can be placed somewhere else. Therefore an extra width for the cables of 2 meters is allowed in the optimization process. The results of this optimization process are discussed in Chapter 6.1.1. Below in Figure 5.8 there is an overview of the variant study. The highlighted line in the plot is the existing Dintelhaven (east) bridge.

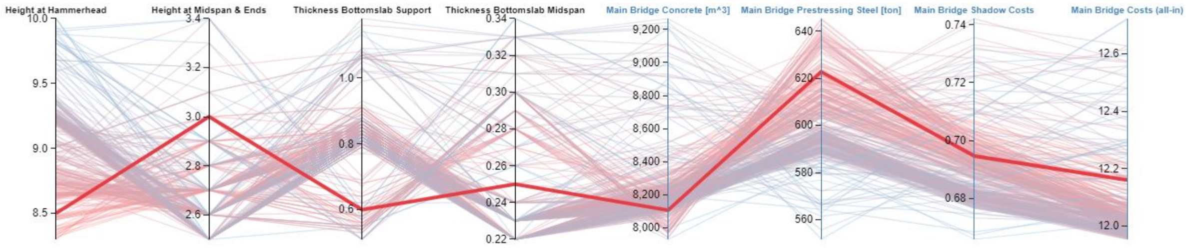


Figure 5.8: Parallel Coordinates Plot of the first variant study for the main bridge. Source: Design Explorer | CORE studio

Optimization (Total Design)

For the total design of the Dintelhaven Bridge (east), the two optimization parameters which are used here, are the height of the box girder at the location of the hammerhead and the overall height of the bridge. The other optimization parameters from the previous optimization processes are now fixed. Here is chosen for a structural height at midspan of 3,0 meters, since this bridge is longer compared to the Stichtsebrug II. The thickness of the bottom slab is 0,8 at the hammerhead and 0,25 at midspan. For all input parameters and the result of the optimization, see Table 5.4 and Figure 5.9.

Table 5.4: Input and output of the Dintelhaven Bridge (east) in the parametric model

Input	**Dintelhaven Bridge (east)**
Optimization Parameters	
Bridge Height	15,1 m
Height at Hammerhead	8,5 m
Fixed Parameters	
Height at Midspan	3,0 m
Thickness Bottomslab Support	0,60 m
Thickness Bottomslab Midspan	0,25 m
Length Bridge	384 m
Width Bridge Deck	22,15 m
Width Cantilever	3,6 m
Thickness Webs	0,38 m
Number of Cells	2
Thickness Deck	0,43 m
Deflection Limit	L/700
Results	
Main Bridge Concrete	8108 m^3
Main Bridge Prestressing Steel	623 ton
Main Bridge Costs (million Euro)	12,161
Total Shadow Costs (million Euro)	2,915
Total Design Costs (million Euro)	57,164

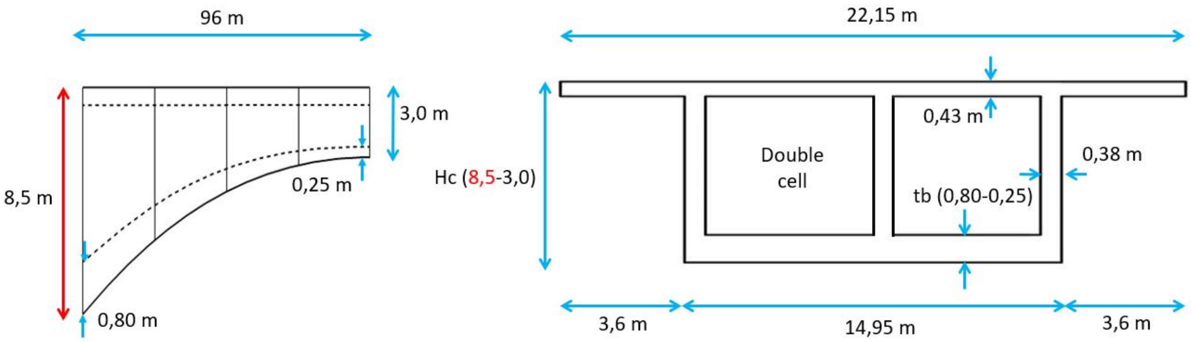


Figure 5.9: Input parameters Dintelhaven (East), with in red the variable parameters.

Note that in the table above, the main bridge costs is written without the penalty. This guarantees a fair comparison with the optimized design.

In Figure 5.10, the results of the variant study is visualized with Design Explorer. The highlighted line in the plot is the existing Dintelhaven (east) bridge.

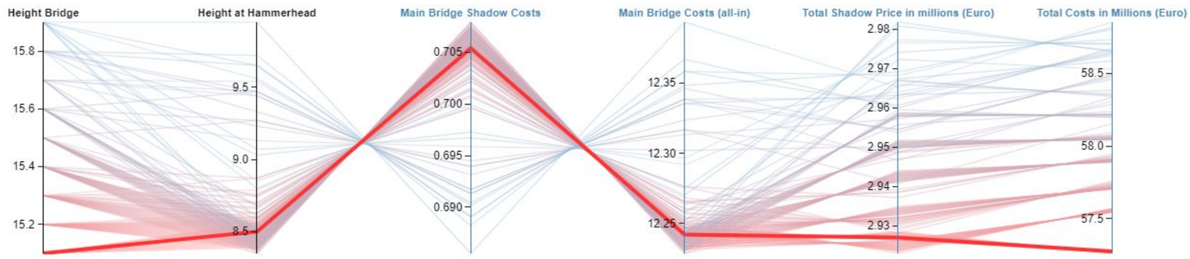


Figure 5.10: Parallel Coordinates Plot of the optimization process for the Dintelhaven total design. Source: Design Explorer | CORE studio

5.2.3. CONCLUSION

In the first optimization set-up, the main bridge is optimized and the expectation beforehand was that there would be a different outcome for the height at hammerhead when optimizing the total design. Therefore the optimization of the bridge was split-up in two takes. But after the optimization it turned out that the height of the main bridge at hammerhead as determined at the first set-up, is almost equal to the most cost-effective height for the total design.

One might wonder, why splitting up the optimization process? The reason for this split-up is simple. The process for the main bridge only takes 2,5 seconds per iteration, while the optimization of the total design takes approximately 20 seconds per iteration. Therefore it is more effective to first search for the optimal dimensions of the main bridge and subsequently search for the height of the bridge above ground level and the optimal structural height at hammerhead.

In the next Chapter, the results of the optimization process will be discussed and also the optimized designs will be compared with the existing bridges.

6 Results

To demonstrate the value of the parametric model, in this chapter the results from the redesigning process are presented, combined with graphs, tables and visualizations. The optimized bridges from the model will be compared with the existing bridges and the results will be discussed in this chapter. Also the influence of the different boundary values or parameters will be made visual with help of design graphs.
At the end of this chapter there is also an overview of the visualisations produced by the script in the Rhino viewport. Subquestions for this chapter are:

6.1 What is the performance of the existing bridges?
6.2 How to present the redesigned bridges?

6.1. RESULTS OF OPTIMIZATION

The results of the optimization stage are divided in two parts. The first part describes and discusses the redesigning process. The second part describes the general use of the parametric model, by showing design graphs and influence lines for specific bridge parameters.

6.1.1. REDESIGNING

In this section, the results of the redesigning process of the Stichtsebrug II and the Dintelhaven (east) cantilever bridges from the previous chapter are described. The results will be presented in tables, in which the input parameters and the output parameters of the existing bridge and the optimized bridge will be visualized. The difference of the optimized bridge, compared to the existing bridge are coloured in red and green. Red, when the parameter is higher compared to the existing bridge and green, when the parameter is an improvement compared to the existing bridge.

Stichtsebrug II - main bridge only

With the evolutionary solver from Galapagos, the values as represented in the last column of Table 6.1 are determined.

Table 6.1: Input and output of the optimization process for the Stichtsebrug II (main bridge only)

	Stichtsebrug II	**Optimized Bridge by Galapagos**
Input		
Optimization Parameters		
Height at Hammerhead	6,75 m	6,73 m
Height at Midspan	2,5 m	2,5 m
Thickness Bottomslab Support	0,55 m	0,52 m
Thickness Bottomslab Midspan	0,22 m	0,22 m
Fixed Parameters		
Length Bridge	320 m	320 m
Width Bridge Deck	21,81 m	21,81 m
Width Cantilever	3,98 m	3,98 m
Thickness Webs	0,32 m	0,32 m
Number of Cells	2	2
Thickness Deck	0,36 m	0,36 m
Deflection Limit	L/700	L/700
Results from parametric model		
Main Bridge Concrete	5445 m^3	5407 m^3
Main Bridge Prestressing Steel	379 ton	381 ton
Shadow Costs (in million Euro)	0,452	0.452
Total Costs (in million Euro)	8,706	8.696

As can be seen in the table, the height and bottomslab at hammerhead are slightly decreased compared to the real bridge dimensions and the other optimization parameters are equal.[1] This results in a smaller concrete cross-section, so less volume, but a slightly higher amount of prestressing cables. Overall this has a positive effect on the total cost of the bridge and the shadow costs are more or less the same.

It can be concluded that the Stichtsebrug II is already quite optimized, there is not much room for improvements. This was already the expectation, so this optimization process also shows that the parametric model is able to generate realistic designs. Which is line with the outcome of the earlier model verification, see end of Chapter 3.4.6.

[1]The construction height at midspan and the thickness of the bottomslab at midspan are minimum values, based on practical dimensions. See also Appendix H.

Stichtsebrug II - total design

With the knowledge of the previous optimization results, the optimization for the total bridge design for Stichtsebrug II is performed. The results from this optimization can be seen in Table 6.2.

Table 6.2: Input and output of the optimization process for the Stichtsebrug II (total design)

Input	Stichtsebrug II	Optimized Bridge by Galapagos
Optimization Parameters		
Bridge Height	13,5 m	13,3 m
Height at Hammerhead	6,75 m	6,33 m
Fixed Parameters		
Height at Midspan	2,5 m	2,5 m
Thickness Bottomslab Support	0,55 m	0,60 m
Thickness Bottomslab Midspan	0,22 m	0,25 m
Length Bridge	320 m	320 m
Width Bridge Deck	21,81 m	21,81 m
Width Cantilever	3,98 m	3,98 m
Thickness Webs	0,32 m	0,32 m
Number of Cells	2	2
Thickness Deck	0,36 m	0,36 m
Deflection Limit	L/700	L/700
Results		
Main Bridge Concrete	5445 m^3	5547 m^3
Main Bridge Prestressing Steel	379 ton	399 ton
Main Bridge Costs (million Euro)	8,706	8,738
Total Shadow Costs (million Euro)	2,218	2,215
Total Design Costs (million Euro)	46,207	45,899

As can be seen in the table above, in the last column once again there is the result of the optimization. There is one new parameter added to the table: the "Bridge Height", which takes care of the vertical placement of the bridge, so that the clearance gauge remains free. This parameter is not influencing the design of the main bridge itself, but has the largest impact on the total design. Therefore this parameter is governing in the optimization process.

Also there can be seen that the construction height at the hammerhead is quite a bit smaller compared to the real dimensions, which is a result of the slight increase in the bottomslab and the adding of the bridge height parameter. This decrease in construction height enables a lower bridge height.

Furthermore, there is the fixed bottomslab which has slightly larger dimensions.

These new dimensions result in a main bridge which is less efficient compared to the existing bridge. There is an increases in concrete volume as well as in prestressing steel. This causes logically a higher costs for the main bridge. The total design costs however are less compared to the existing bridge.

Dintelhaven (east) - main bridge only

The results of the optimization process and the real Dintelhaven (east) bridge parameters and results can be seen in Table 6.3. The same approach is applied here as for the Stichtsebrug II in the previous paragraphs, so below the table will be the explanation of the result.

Table 6.3: Input and output of the optimization process for the Dintelhaven (east) (main bridge only)

	Dintelhaven (east)	**Optimized Bridge by Galapagos**
Input		
Optimization Parameters		
Height at Hammerhead	8,5 m	9,24 m
Height at Midspan	3,0 m	2,5 m
Thickness Bottomslab Support	0,60 m	0,79 m
Thickness Bottomslab Midspan	0,25 m	0,22 m
Fixed Parameters		
Length Bridge	384 m	384 m
Width Bridge Deck	22,15 m	22,15 m
Width Cantilever	3,6 m	3,6 m
Thickness Webs	0,38 m	0,38 m
Number of Cells	2	2
Thickness Deck	0,43 m	0,43 m
Deflection Limit	L/700	L/700
Results from parametric model		
Main Bridge Concrete	8108 m^3	8069 m^3
Main Bridge Prestressing Steel	623 ton	594 ton
Shadow Costs (in million Euro)	0,694	0.677
Total Costs (in million Euro)	12,161	11,954

As can be seen in the last column of the table, the height at the hammerhead is considerably enlarged. While the height at midspan is decreased. The same trend applies to the thickness of the bottomslab, at hammerhead there is an increase while at midspan the bottomslab is minimized. Surprisingly, this change in dimensions leads to a lower concrete volume and a significantly lower prestressing amount. At the bottom of the table, this results in a costs saving of 0,2 million Euro and also a lower environmental impact.

Dintelhaven (east) - total design

In Table 6.4, the results of the optimization of the total design for the Dintelhaven (east) bridge is shown.

Table 6.4: Input and output of the optimization process for the Dintelhaven (east) (total design)

Input	**Dintelhaven (east)**	**Optimized Bridge by Galapagos**
Optimization Parameters		
Bridge Height	15,1 m	15,0 m
Height at Hammerhead	8,5 m	8,35 m
Fixed Parameters		
Height at Midspan	3,0 m	3,0 m
Thickness Bottomslab Support	0,60 m	0,80 m
Thickness Bottomslab Midspan	0,25 m	0,25 m
Length Bridge	384 m	384 m
Width Bridge Deck	22,15 m	22,15 m
Width Cantilever	3,6 m	3,6 m
Thickness Webs	0,38 m	0,38 m
Number of Cells	2	2
Thickness Deck	0,43 m	0,43 m
Deflection Limit	L/700	L/700
Results		
Main Bridge Concrete	8108 m^3	8397 m^3
Main Bridge Prestressing Steel	623 ton	622 ton
Main Bridge Costs (million Euro)	12,161	12,229
Total Shadow Costs (million Euro)	2,915	2,831
Total Design Costs (million Euro)	57,164	56,111

Equal to the optimization process for the Stichtsebrug II, in this optimization take, the "Bridge Height" is added as optimization parameter. Compared to the existing Dintelhaven bridge, the only fixed parameter which differs from reality is the thickness of the bottomslab at the location of the hammerhead. This thickness of 0,80 meters is in line with the thickness which came out of the optimization process for the main bridge only, which was 0,79 meters. The height of the bridge at midspan is kept equal tot the existing design, although the optimization gave a smaller height. This choice is based on the length of the bridge and the standard applied height for bridges with a span length of 180 meters or more, see also Appendix H.

As can be seen in the table, the "Height at Hammerhead" is less compared to the existing bridge, which enables also a slightly lower "Bridge Height". Although the construction height at the hammerhead is decreased, there still is an increase in concrete volume due to the enlargement of the bottomslab dimension. However, this quite large increase of concrete volume leads to only a small decrease in prestressing steel. This logically results in higher costs for the main bridge.

The higher costs for the main bridge are not fatal for the total design costs. Due to the decreasing of the bridge height with only 0,1 meter, the total design costs drop with more than one million Euro. Also the environmental impact of the material is less compared to the existing Dintelhaven bridge.

6.1.2. DISCUSSION

As can be seen in the previous paragraphs, there is a difference between the results of the optimization of the main bridge only and the total design. It can be concluded, that the influence of the construction height is the largest for the total design, while this is not always the case for the performance of the main bridge only. See for instance the high construction height at hammerhead for the Dintelhaven bridge in the main bridge only optimization, Table 6.3

Possible explanation for this difference is the fact that a high construction height for the main bridge has a positive effect on the force distribution in the main bridge. The shape of the bridge is more like an arch, so the loads are partly taken up by the compression arch in the bridge.

Another reason for the high construction height has to do with the price difference between concrete with a unit price of 100 €/m^3 and prestressing steel with a unit price of 1750 €/ton. Of course, the price difference is not exactly the difference between these two values, since there is also the formwork, reinforcement steel and casting of the concrete which should be added somehow to the unit price. This detailed price difference is not further elaborated in this project.

Where in the optimization of the main bridge only, the costs of this part must be minimized, the model increases the amount of the relatively cheap concrete, to lower the amount of the more expensive prestressing steel (including anchors). This gives higher cross-sections but in the end a lower price.

In the total design optimization, these higher cross-sections have a negative impact on the total design costs, because the total road alignment must be higher, causing longer approach bridges and embankments. Therefore in this optimization, the height of the bridge is minimized as much as possible, with a high use of prestressing steel as result. See for instance Table 6.2, where there is a small increase in concrete volume of 1,9 % but a larger increase of 5,3 % in prestressing steel to create a bridge with a lower construction height and thereby lower total costs.

There must be noted once again, that the costs as determined by the model are only the material costs, so other costs aspects are not taken into account. When also the other costs are taken into account, it might possibly lead to other results, but the lowering of the bridge height is an effect for the total design which is in most cases positive for the project costs.
After the various optimizations, there can also be concluded that the governing checks for the main bridge are most of the time; the deflection of the bridge at midspan and the fitting of the cables in the bridge deck or bottomslab. So these are the checks which determine if the design is valid or if there must be a penalty.
Capacity checks are rarely governing for the design.

6.1.3. DESIGN GRAPHS

The general performance of the model is investigated by trying different optimization set-ups. The results of these optimization processes are described in this section, combined with graphs and tables.

To investigate the working of the model, there are performed optimizations for two different bridge widths and both are optimized for four different span lengths. The range for the span length is determined by the span length of the existing bridges in the Netherlands. The set-up for this process can be seen in Table 6.5.

Table 6.5: Input and output for the different optimization processes

general dimensions		**Optimization parameters**				**Main bridge only**		**Total design**	
Width	Span length	H1	H2	T1	T2	Costs	shadow costs	shadow costs	Costs
16	140	5.3	2.5	0.5	0.22	5.745	0.296	1.618252	33.850
16	160	6.6	2.5	0.64	0.22	6.874	0.354	1.657	35.179
16	180	8.7	2.5	0.77	0.22	8.167	0.416	1.820	37.987
16	200	10.5	2.6	0.9	0.23	9.612	0.492	1.857	38.820
21	140	5	2.5	0.5	0.22	7.904	0.402	2.103	43.872
21	160	6.2	2.5	0.53	0.22	9.370	0.482	2.194	45.763
21	180	8.3	2.5	0.57	0.22	11.077	0.559	2.295	47.998
21	200	10.1	2.6	0.6	0.24	13.003	0.657	2.407	50.272

In the table above on the left the general dimensions are shown. The width of 16 meters corresponds with a double cell bridge and a width of 21 meters corresponds with a triple cell bridge. For both bridge widths, there is a length division between 140 meters and 200 in steps of 20 meters. For each of the general dimensions there is performed an optimization on construction height. The gray highlighted column marks the thickness of the bottomslab at the location of the hammerhead, this is linear interpolated between the minimum and the maximum span length to simplify the design process. For the construction height and thickness of the bottomslab at midspan, only the longest span is divergent. This is a result of the optimization process.

The results of the optimization processes can be seen in Figure 6.1, where the "Design Graphs" are plotted and in Figure 6.2, where the "Total Costs" are plotted.

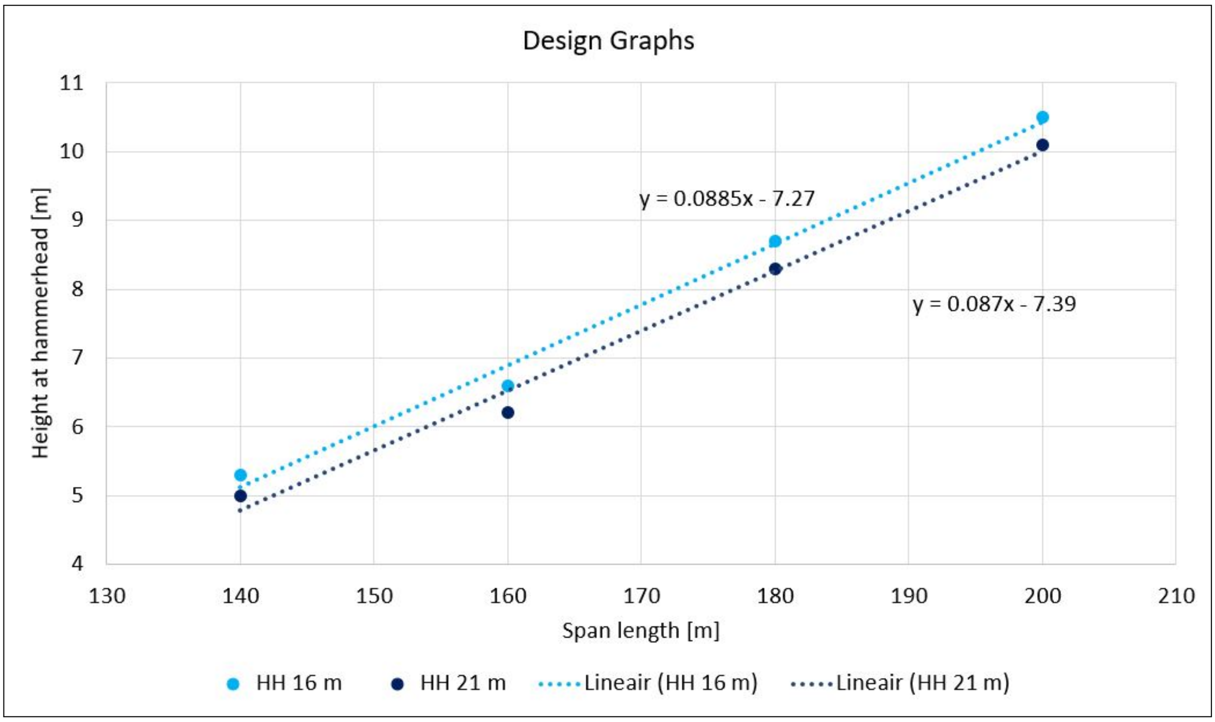


Figure 6.1: Design graphs for double and triple cell cantilever bridges with a span length between 140 and 200 meters.

In the "Design Graph" on the vertical axis, the construction height at hammerhead can be found while the span length is on the horizontal axis. As described earlier, this construction height has the largest influence on the total design costs and this parameter is minimized during the optimization process.

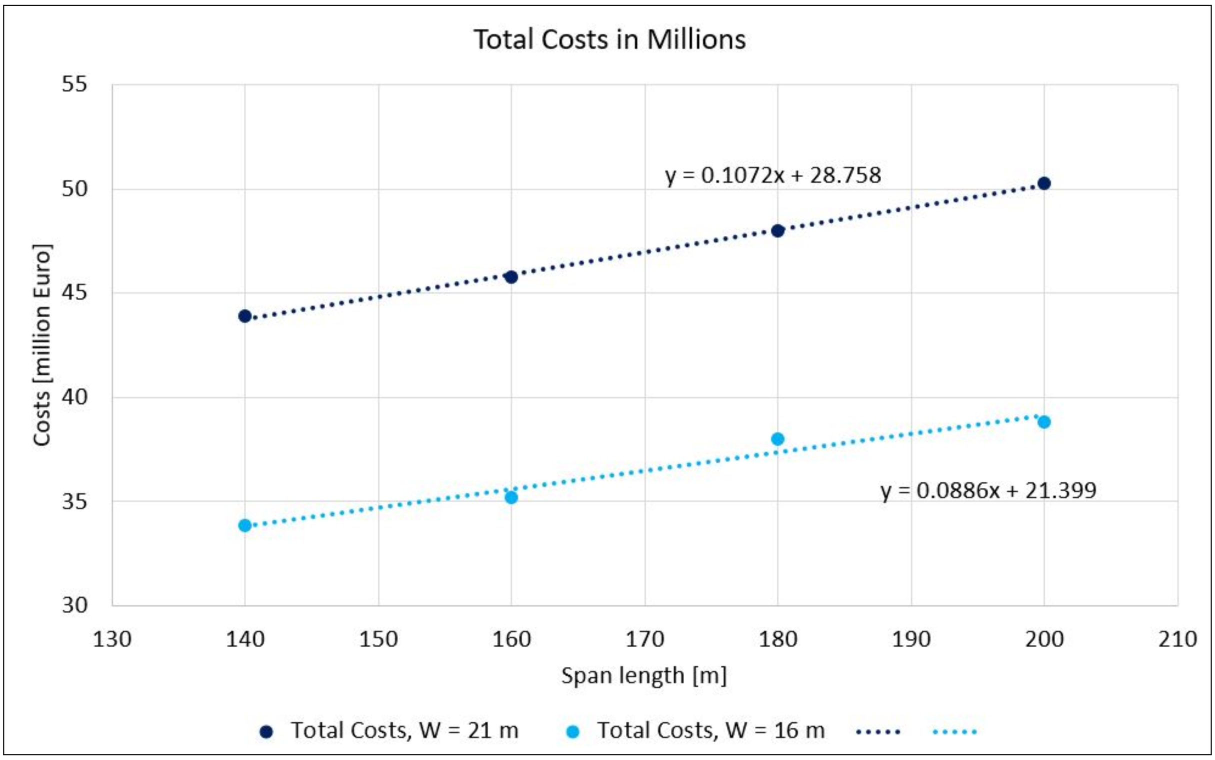


Figure 6.2: Total costs graph for double and triple cell cantilever bridges with a width of 16 or 21 meters and a span length between 140 and 200 meters.

Discussion part 1

As can be seen in the graphs above, the width of the bridge has a positive influence on the construction height. The values for the construction height in the design graph for the 21 meter bridge is lower compared to the values for the 16 meter bridge. Reason for this is the ratio between the variable loads and the self-weight which is decreasing for wider bridges. Also the 21 meter bridge has a slightly higher web-to-width ratio, $\frac{4}{21}$ vs $\frac{3}{16}$. This means that the moment capacity for the same construction height, should for the wider bridge in comparison always be higher.

Furthermore, it is remarkable that the lines in the design graphs are almost linear. This is also the case for the material costs, which means that the increase in span length gives almost the same proportional increase in costs.

One would expect that the design graph for H/L would have a more increasingly ascending shape, but this is not the case. An explanation for this is that although the self-weight of the bridge is increasing, when enlarging the span, the construction height changes as well. For a prismatic beam, the curve would be a second degree function based on the moment distribution which is a second degree function. This explains why the line is almost linear for the cantilever bridge; the shape of the bridge follows the inverse shape of the moment distribution due to it's self-weight. Why the line is not completely linear is because there also is a variable load which is effecting the design.

Another explanation why the line for the costs is also almost linear, is because the applied length during the optimization is probably within the economic range of this bridge type. When trying out a span length outside the span length domain, there could be investigated whether the result deviates from the line. This will be performed in the next subsection.

Design graphs out of range

In the discussion above, the shape of the design graph and the costs graph was questioned. That is why in this subsection the range of the bridge is extended to investigate the course of the graph outside the standard applied range. Also the number of prestressing cables is visualized, to show the impact of the increasing span length.

The range for the cantilever bridge is extended and the new span range is 120-250 meters. In this optimization process, only the double cell bridge with a width of 16 meters is calculated.

Below, the new Design Graph for W = 16 m is plotted, see Figure 6.3

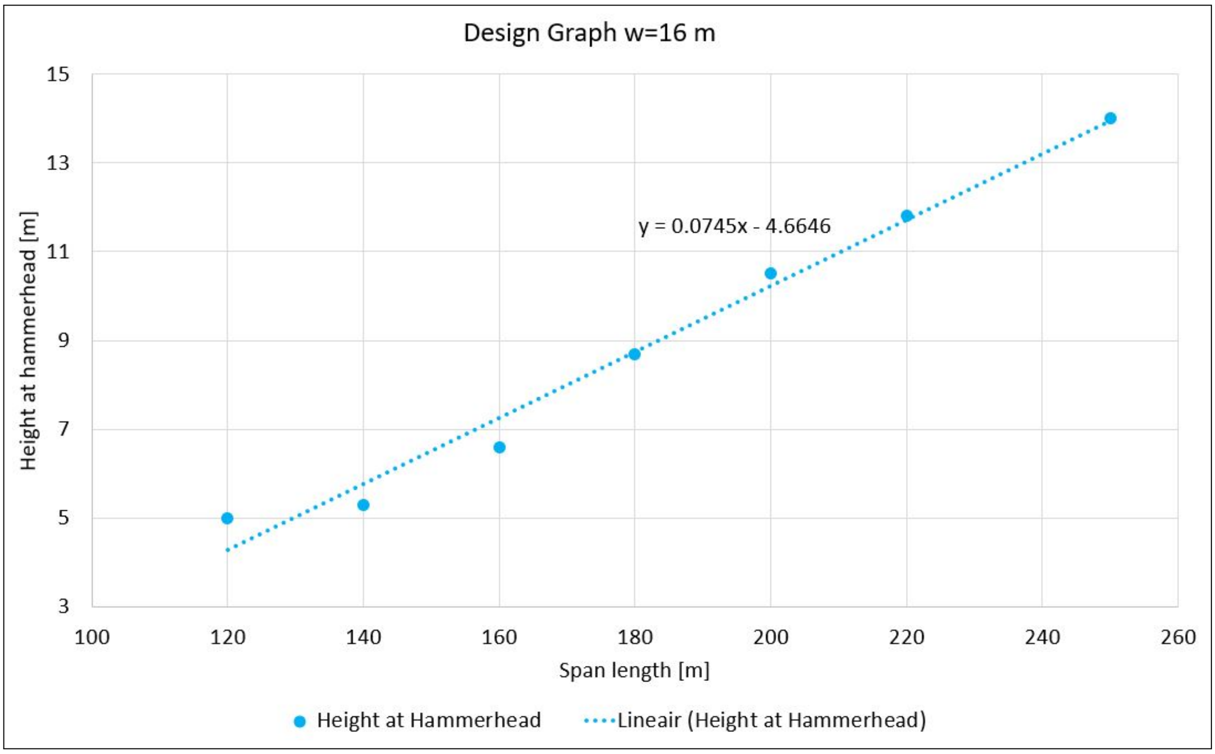


Figure 6.3: Design graphs for double cell cantilever bridges with a span length between 120 and 250 meters.

As can be seen in the graph, the course of the height versus the span length is still linear, so the theory of the non-prismatic cross-section from the previous discussion still applies. However, when the total costs graph is plotted for the same span range, the course of the graph changes compared to the previous graph.
See Figure 6.4

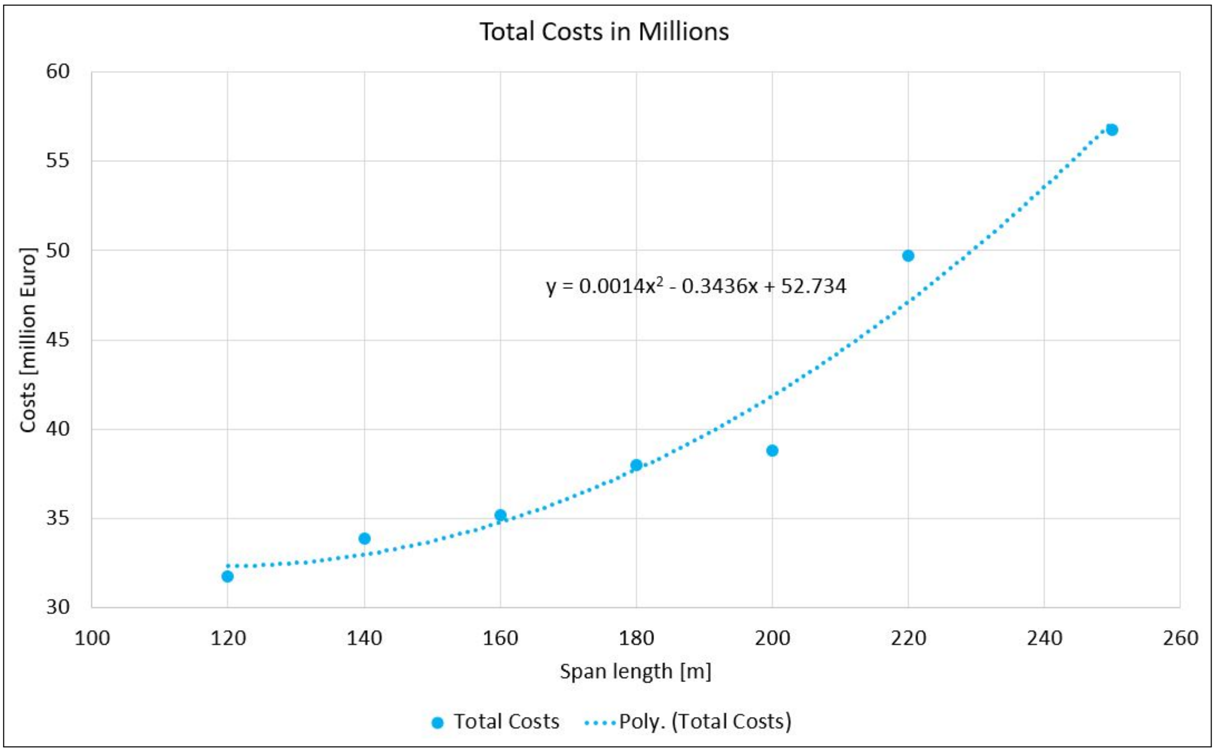


Figure 6.4: Total costs graph for double cell cantilever bridges with a span length between 120 and 250 meters.

Discussion part 2

The new costs graph has a clear increasingly ascending shape, which is caused by the increase in construction heights and thickness's of the bottomslab. Furthermore, the number of prestressing cables is also increasing. And since the length of the road alignment is linked to the height of the bridge, this linear increase is also notable in the approach bridges and the foundation.

In the design of the approach structure (embankment plus approach bridge), the maximum height of the embankment is fixed, so the costs for the embankment stay more or less equal. The bridge part however is increasing and each additional span which is required when the alignment lengthens, means also a costs increase in the substructure and foundation. Also the increased weight of the main bridge results in a larger foundation.

From the difference between the first plotted costs graph and the new costs graph can be concluded that the range extension is outside the economic range for this bridge type. Because the costs increases faster compared to the increase in span length. Also the constructibility of the longer bridges is becoming more difficult since the required amount of prestressing is exceeding the allowable maximum number of cables in the cross-section. This can be seen in Figure 6.5.

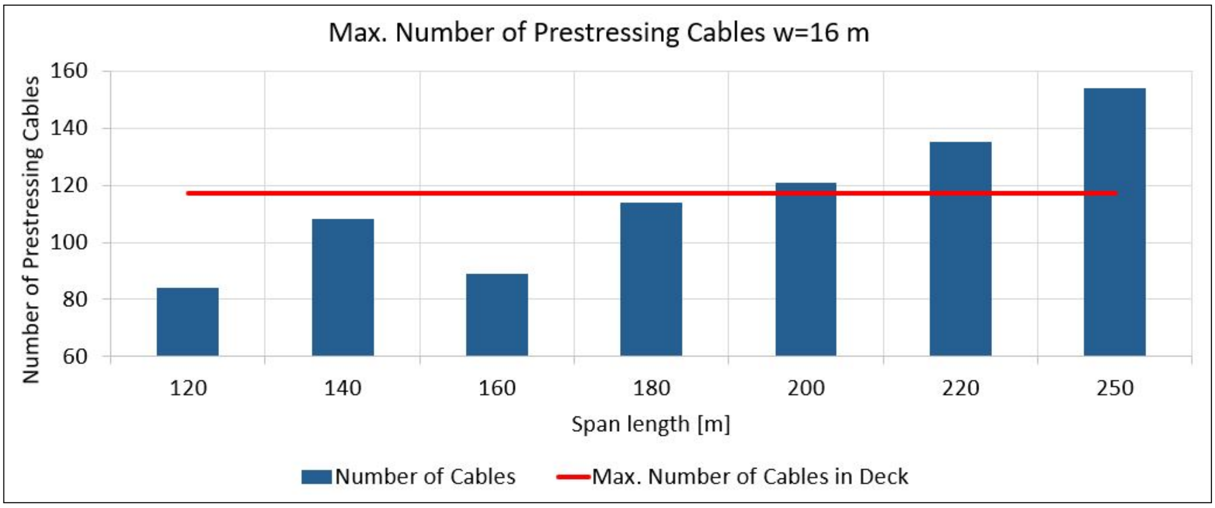


Figure 6.5: Number of cantilever prestressing cables in the deck of the bridge and the maximum number of cables which can be applied.

The graph shows that from a span length of 200 meters, the number of cables which is required for the design exceeds the maximum number of cables which can be placed in the deck. For the 200 meters span, these are only a few cables which possibly can be placed in a second layer or in the web of the bridge, but for the 220 and 250 meter spans the limit is exceeded significantly.

One of the options for these longer spans could be to increase the height of the concrete cross-section, because by increasing the internal lever arm, the amount of prestressing cables can be decreased. However, this would be quite costs inefficient since the length of the alignment is rapidly increasing as well. Furthermore, the height of the cross-section would also effect the slenderness of the webs. To make sure that the webs will not buckle, the thickness must be increased which results in additional self-weight.
Another option is placing the cables double-layered in the deck. For this, the concrete deckslab must be thickened (locally), which again gives an increase in self-weight. So most of the time these measures require additional prestressing in the end.

Therefore it is safe to conclude that; the span range from 140 to 200 meters, as applied in the Netherlands, is economically beneficial. However, in other countries, for instance Norway, the applied span range is larger. In these countries bridges up to 300 meters are built.
The main reason for this difference is the fact that in these countries, height differences are easily taken due to the presence of hills and mountains. Also the stronger rock bottom makes the foundation of the heavy-weight bridges more easily.

6.2. VISUALIZATIONS

Below there are some output visualizations of the two redesigned and optimized bridges.

Stichtsebrug II

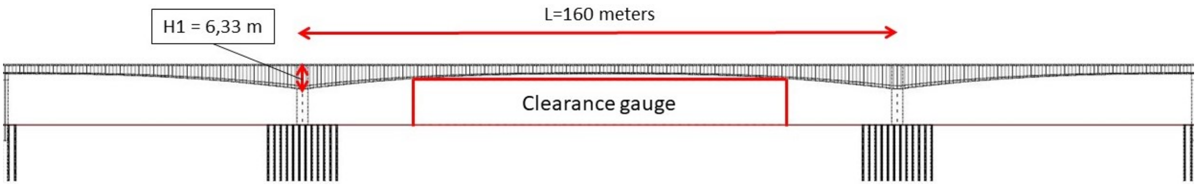


Figure 6.6: Visualisation of the Stichtsebrug II, designed by the parametric model

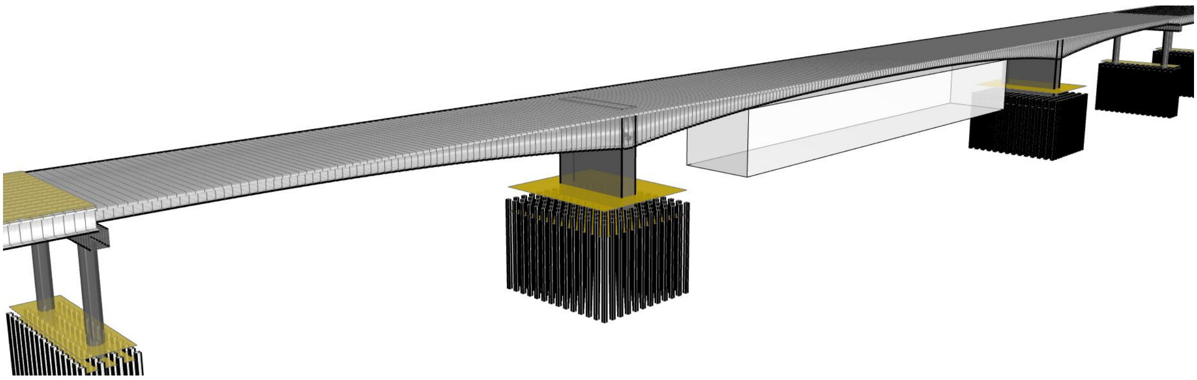


Figure 6.7: Visualisation of the Stichtsebrug II, designed by the parametric model

Dintelhaven (east) bridge

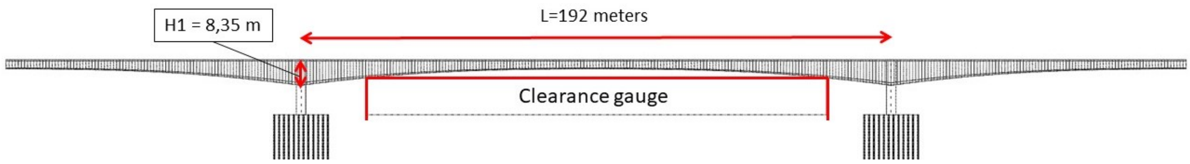


Figure 6.8: Visualisation of the Dintelhaven (east), designed by the parametric model

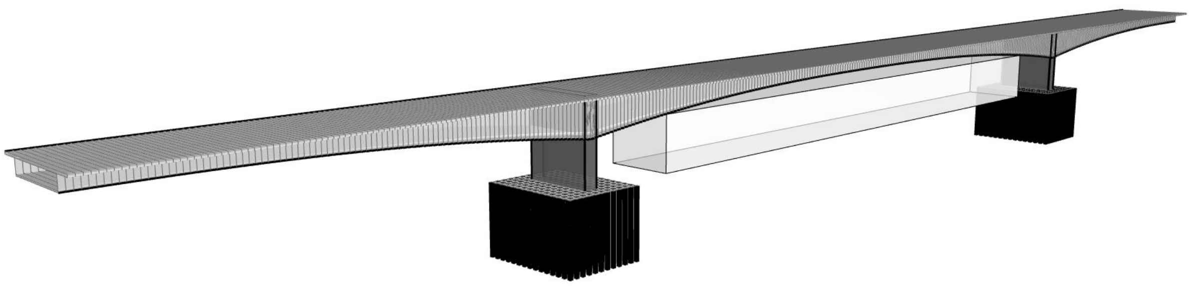


Figure 6.9: Visualisation of the Dintelhaven (east), designed by the parametric model

7

Discussion

In this chapter, the answer on the main question will be given and the conclusions of this master thesis project will be discussed along with recommendations for future research into parametric bridge design. Also possible improvements for the Grasshopper script will be given.
The main question for this master thesis is:

What can be achieved by using a parametric model in the preliminary design phase, with the possibility to optimize and compare concrete bridges on material usage, costs and environmental impact?

Answer: Although the parametric model for the preliminary design of concrete cantilever bridges took a long time to develop, this time investment can be regained by using the model several times. Because by using the parametric model for the preliminary design of concrete bridges, the design time per procurement is decreased significantly. Furthermore, the design process of the balanced cantilever bridge has become more flexible since changes are automatically implemented for the whole model. Also the chance for (human) errors is decreased, for instance the risk of data loss is minimized due to the model's single environment. Since only a limited amount of calculations is performed by the model, the results of the analysis must be used with caution. Another advantage is the instant results as produced by the model. These results consist of a visualisation of the design and an overview of the material amounts, with their costs and environmental impact. Furthermore, the model is also able to optimize these results in a fast way due to a smart evolutionary solver. This all together makes the parametric model a useful tool with high potential, but it should be further developed and improved.

7.1. Conclusions

In this section, the answer on the main question is supported by stating all conclusions in detail. First the advantages and subsequently the limitations of the parametric model will be enumerated. The recommendations will be discussed in the next section.

7.1.1. Advantages

Speed
Once the input for the design is provided by the user, it is possible with the parametric model to design concrete bridges in a small amount of time, as demonstrated in Chapter 5.2.3. Generating a design takes around 20 seconds for the model and the optimization of the main bridge only takes 2,5 seconds per iteration. This means that it is possible to design a bridge within a couple of hours instead of days or weeks.

Flexibility
The parametric model is also an improvement in flexibility for the design of cantilever bridges. When in the traditional design process the length of a cantilever bridge would change, the structural model must be adapted and all cross-sections of the bridge must be redesigned by hand. The same counts for changes in the width of the bridge or other changes in the boundary conditions. In the parametric model, these changes are

automatically taken care of and the new cross-sections are redesigned in seconds. Therefore, the model can be used at multiple locations with only changing the location input. This makes the model easy to use and quite suitable during the preliminary design phase, where high flexibility is demanded.

Completeness
Where in traditional design, multiple software programs are used for the design, analysis, costs calculations and optimization, which is performed most of the time on a single parameter. In the parametric model, these processes are combined in a single environment and optimization can be performed for multiple parameters at the same time. The advantage of this single environment is, that all input and output are in the same program and changes in the design are instantly shown in the visualizations and results. Another advantage is that the chance for loss of data or unmeant changes in data is minimized since there are no transfers between different software programs.

Smart optimization
As also described in Chapter 5.2.3, a split is made in the model between the main bridge, which has the largest impact on the total costs, and the total bridge design. This gives a faster way of optimizing and by using the evolutionary solver of Galapagos it is effective enough to calculate only a selection of the possible iterations.

Further investigations
The model can be used for further investigations since it has a working design process and it is fast and flexible. For instance, it is possible to investigate the influence of certain parameters on the total costs, as also described in Chapter 6.1.3. Or to use the model for further research into difficult mechanical problems like the shrinkage, creep and relaxation of concrete bridges. This could also strengthen the model itself. Furthermore, the model can be easily extended since there is a solid base.

7.1.2. LIMITATIONS

Applicability
For now, the model is only applicable for concrete cantilever bridges with large spans in the preliminary design phase. A disadvantage is that these bridges are only build once in a couple of years, so the model can not be used everyday. Furthermore, the development of a parametric model of this scale, takes a long time so it is a large investment. The question is whether this investment can be earned back.

Reliability
The model is only reliable for cantilever bridges with a small width compared to their length since the calculations are in 2D. When the transverse direction of the bridge is also added to the analysis, the eccentricity of the variable loads has a detrimental effect on the designs. Also calculations for shear force capacity are not included in the model, since this has a smaller impact compared to the bending moment capacity. However, for local effects, the shear force calculations are important. Besides that, the used software for the analysis is not as accurate as the traditional finite element programs.

Complexity
A detrimental effect of the complicated and extended model is the fact that inexperienced users will get lost quite easily and the use of the model for people without knowledge of the script is difficult. Therefore, the use of the model should be explained by adding notes and remarks to the model. Another disadvantage is the fact that the smoothness of the design process is decreasing for extended models.

Results
Regarding the results of the costs calculation, in the model the costs are only determined on material use. For a more reliable result, also the location of the bridge should be taken into account and the transport, use of machinery and construction method should be added to the cost calculation. This could lead to different optimal designs. This will be further explained in the recommendations.

7.2. RECOMMENDATIONS

In this section, the recommendations for future research or improvements for the model are stated. The recommendations are split-up into; additional literature study, reliability improvements for the model, more detailed costs calculation and adding a user interface.

User interface
Since the parametric model in Grasshopper is quite overwhelming and confusing, especially for inexperienced users, an improvement is to add a user interface (UI). This UI must be supported by notes and explanations from the script designer. This would improve the user-friendliness off the model. The Grasshopper plug-in "Human UI" could be used for this purpose.

Additional literature study
To extend and improve the model, there should be performed additional literature study into other bridge designs. This gives extra comparisons per location. Also the balanced cantilever bridge could be analysed in more detail.

For the design of the balanced cantilever bridge, in this project there is chosen for a 2,5 degree function based on earlier research. Although it is not clear why this shape is more efficient compared to a second or third degree function. This shape of the bridge could be further investigated with help of the parametric model. Also there should be investigated which span lengths are within the economic range and what happens if the span lengths are outside of this domain.

Also the effect of creep, shrinkage and relaxation and the phased calculations during the construction of the bridge should be further investigated. For now there is only a difference between construction phase just before closing and the end phase, with a constant creep parameter.

In the model there is used only a deflection limit for the deformation due to the variable loads, but the exact requirements for this type of bridges is not clear. Also the effect of the dynamic behaviour and vibrations of the bridge must be investigated.

Reliability improvement
The reliability of the model can be improved by adding additional analyses to the model. Part of these analyses are already described above, but there are also analyses which do not have the necessity of a complex literature study. For instance, 3D calculations which require a different modelling approach, shear calculations mainly for local effects and the use of soil parameters like the results of Cone Penetration Tests (CPT). These additional calculations and validations help to improve the reliability of the parametric model, but a detrimental effect is the increased complexity of the model and presumable time-lag.

Detailed costs calculation
In the current parametric model, the costs are only based on the material usage. To achieve a more realistic result, the other costs aspects must be added as well. Additional costs aspects are; location data, transport costs, construction method, maintenance costs, equipment and labour costs.

For the location data, it is important to investigated the difference of building in an urban environment or at the countryside. Also the location has impact on the accessibility and the distance between the construction site and for instance the concrete mixing plant. This accessibility effects the transport costs.

Furthermore, the model does take into account the difference between certain construction elements, but the construction method itself and the use of expensive equipment is not implemented in the costs calculation. Also an estimation of the amount of labour should be added to the model.

To be able to compare the costs of the bridge also with bridges made out of other materials, it is also interesting to add the maintenance costs. This could make the difference between the choice for a concrete or a steel bridge.

A

Parametric Design Program Setup

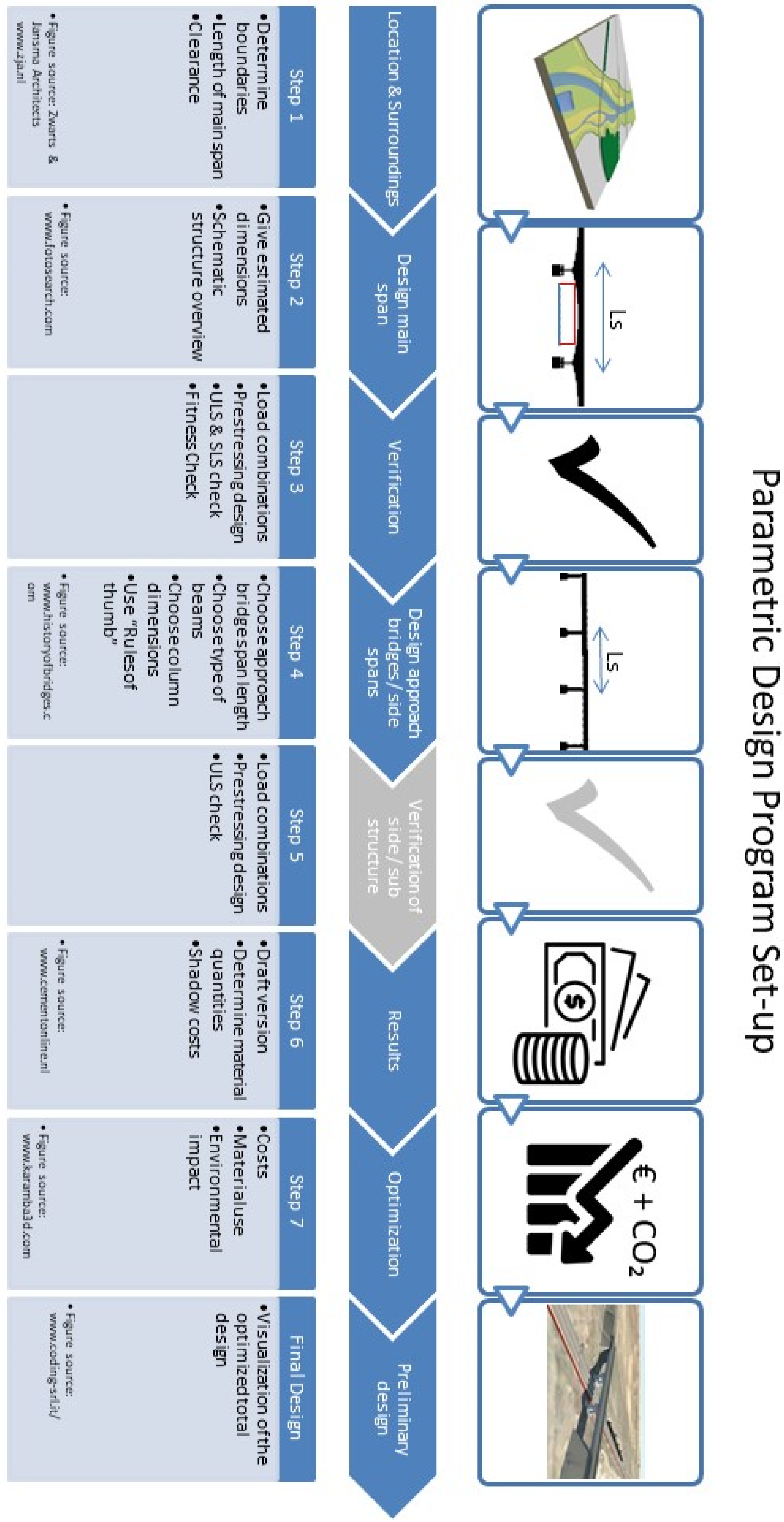
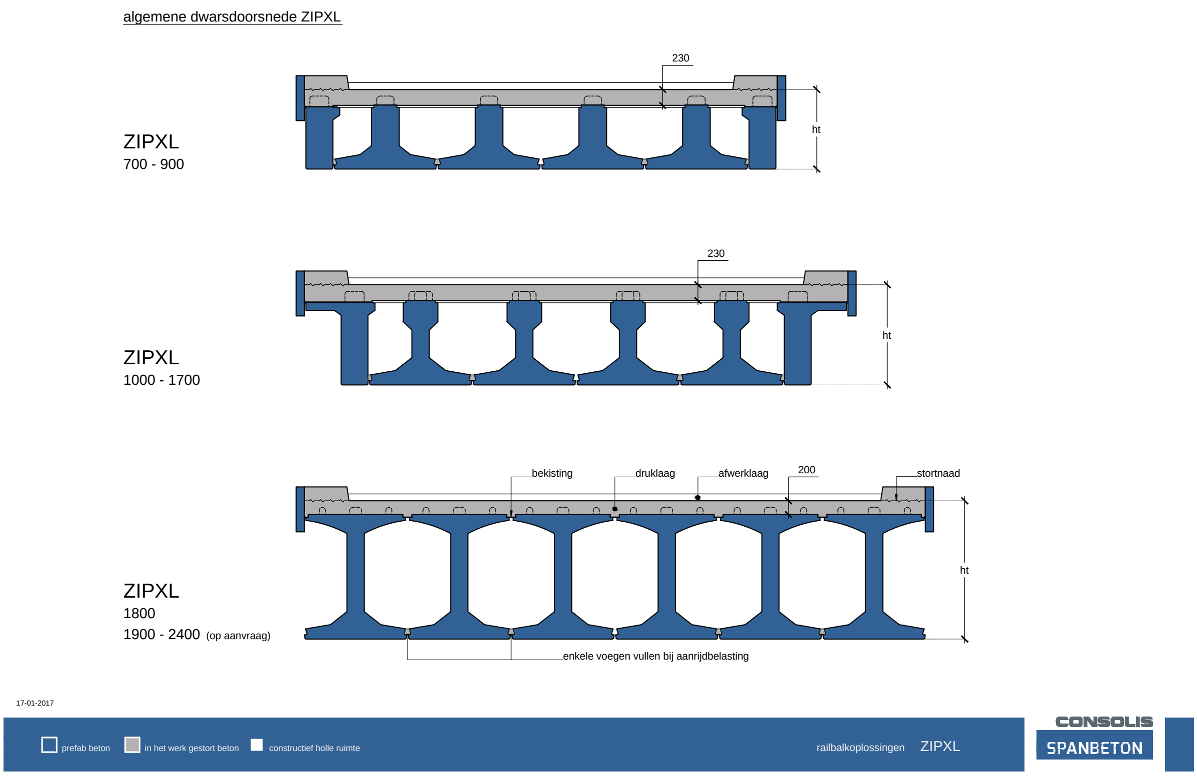


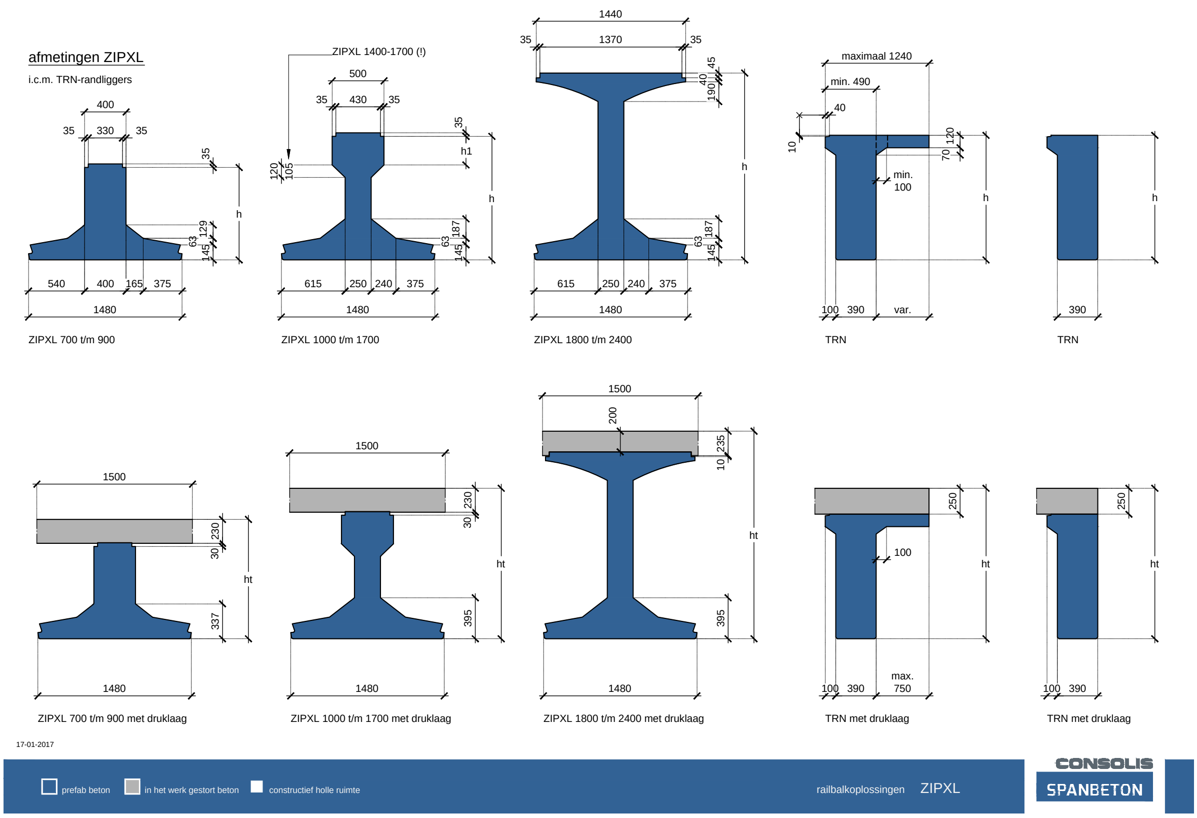
Figure A.1: Parametric Design Program Set-up.

B

Prefabricated Beams as specified by Spanbeton



algemene dwarsdoorsnede ZIPXL
ZIPXL
700 - 900
230
ht
ZIPXL
1000 - 1700
230
ht
ZIPXL
1800
1900 - 2400 (op aanvraag)
bekisting
druklaag
afwerklaag
200
stortnaad
ht
enkele voegen vullen bij aanrijdbelasting
17-01-2017
prefab beton
in het werk gestort beton
constructief holle ruimte
railbalkoplossingen ZIPXL
CONSOLIS
SPANBETON



afmetingen ZIPXL
i.c.m. TRN-randliggers
400
35 330 35
35
129
63
145
h
540 400 165 375
1480
ZIPXL 700 t/m 900
ZIPXL 1400-1700 (!)
500
35 430 35
35
h1
120
105
187
63
145
h
615 250 240 375
1480
ZIPXL 1000 t/m 1700
1440
35 1370 35
45
40
190
187
63
145
h
615 250 240 375
1480
ZIPXL 1800 t/m 2400
maximaal 1240
min. 490
40
10
120
70
min.
100
h
100 390 var.
TRN
h
390
TRN
1500
230
30
ht
337
1480
ZIPXL 700 t/m 900 met druklaag
1500
230
30
ht
395
1480
ZIPXL 1000 t/m 1700 met druklaag
1500
200
235
10
ht
395
1480
ZIPXL 1800 t/m 2400 met druklaag
250
100
ht
100 390
max.
750
TRN met druklaag
250
ht
100 390
TRN met druklaag
17-01-2017
prefab beton
in het werk gestort beton
constructief holle ruimte
railbalkoplossingen ZIPXL
CONSOLIS
SPANBETON

Afmetingen en gewichten

Afmetingen				Gewichten		
type	h [mm]	h_1 [mm]	h_t [mm]	prefab [kN/m]	druklaag [kN/m]	totaal [kN/m]
ZIPXL 2400	2400	-	2600	26,5	7,5	34,0
ZIPXL 2300	2300	-	2500	25,8	7,5	33,3
ZIPXL 2200	2200	-	2400	25,2	7,5	32,7
ZIPXL 2100	2100	-	2300	25,6	7,5	33,1
ZIPXL 2000	2000	-	2200	24,0	7,5	31,5
ZIPXL 1900	1900	-	2100	23,3	7,5	30,8
ZIPXL 1800	1800	-	2000	22,7	7,5	30,2
ZIPXL 1700	1690	500	1950	21,2	8,6	29,8
ZIPXL 1600	1590	400	1850	19,9	8,6	28,5
ZIPXL 1500	1490	300	1750	18,7	8,6	27,3
ZIPXL 1400	1390	200	1650	17,4	8,6	26,0
ZIPXL 1300	1290	375	1550	17,9	8,6	26,5
ZIPXL 1200	1190	275	1450	16,7	8,6	25,3
ZIPXL 1100	1090	175	1350	15,4	8,6	24,0
ZIPXL 1000	990	75	1250	14,2	8,6	22,8
ZIPXL 900	890	-	1150	14,6	8,6	23,2
ZIPXL 800	790	-	1050	13,6	8,6	22,2
ZIPXL 700	690	-	950	12,6	8,6	21,2

Doorsnedegrootheden (exclusief wapening)

	Enkel profiel (prefab)			Samengesteld profiel	
	A_{EP} $\cdot 10^3$ [mm^2]	zp_{EP} [mm]	I_{EP} $\cdot 10^9$ [mm^4]	zp_{SP} [mm]	I_{SP} $\cdot 10^9$ [mm^4]
ZIPXL 2400	1060	1121	831	1386	1219
ZIPXL 2300	1035	1074	748	1333	1105
ZIPXL 2200	1010	1026	670	1288	1008
ZIPXL 2100	985	978	597	1235	907
ZIPXL 2000	960	931	529	1181	811
ZIPXL 1900	935	883	466	1127	722
ZIPXL 1800	910	836	408	1074	639
ZIPXL 1700	845	719	288	1012	565
ZIPXL 1600	795	660	240	955	494
ZIPXL 1500	745	599	196	897	428
ZIPXL 1400	695	537	156	838	367
ZIPXL 1300	716	524	137	793	314
ZIPXL 1200	666	468	107	738	264
ZIPXL 1100	616	412	81	682	219
ZIPXL 1000	566	354	58	625	179
ZIPXL 900	583	328	43	569	144
ZIPXL 800	543	288	31	519	113
ZIPXL 700	503	251	21	469	86

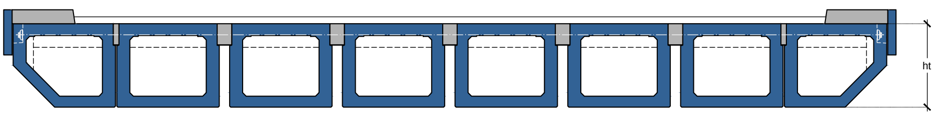
uitgangspunten:
betonsterkteklasse prefab: C60/75
betonsterkteklasse druklaag: C35/45

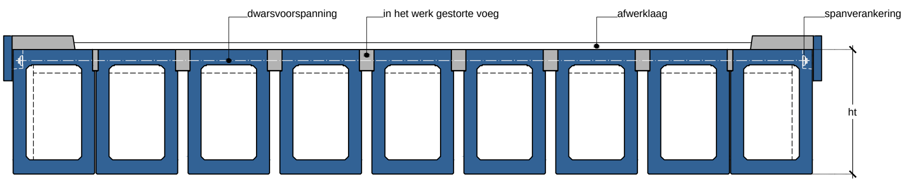
weerstandsmomenten EP :
$W_{EP,boven}$ $= I_{EP} / (h - zp_{EP})$
$W_{EP,onder}$ $= I_{EP} / zp_{EP}$

traagheidsmoment I_{SP} is bepaald met gewogen E op basis van stijfheid prefab

algemene dwarsdoorsnede SKK

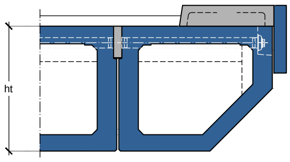
SKK
700 - 1600



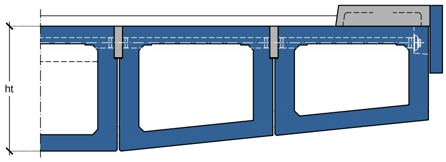


SKK
1700 - 1900 (op aanvraag)

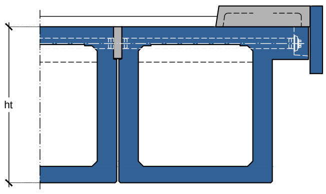
randoplossingen SKK



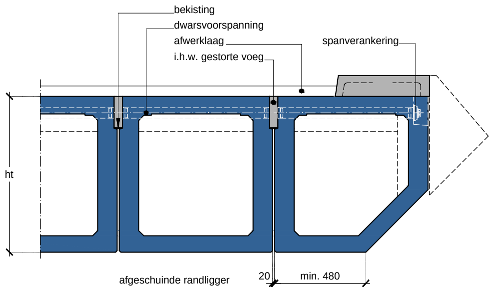
afgeschuinde randligger



afschuining over meerdere liggers



randligger met uikraging (max. 350 mm)

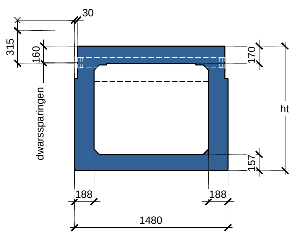


afgeschuinde randligger

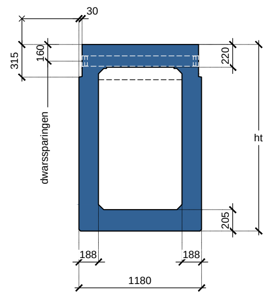
17-01-2017

Afmetingen en gewichten

Afmetingen		Gewichten	
type	$h=h_t$ [mm]	massief [kN/m]	hol [kN/m]
SKK 1900	1900	55,6	28,6
SKK 1800	1800	52,6	27,6
SKK 1700	1700	49,7	26,5
SKK 1600	1600	58,7	29,3
SKK 1500	1500	55,0	28,1
SKK 1400	1400	51,3	25,3
SKK 1300	1300	47,6	24,2
SKK 1200	1200	43,0	23,0
SKK 1100	1100	40,2	21,9
SKK 1000	1000	36,5	20,1
SKK 900	900	32,8	19,0
SKK 800	800	29,1	17,9
SKK 700	700	25,4	16,7



SKK 700 t/m 1600



SKK 1700 t/m 1900 (op aanvraag)

Doorsnedegrootheden (exclusief wapening)

	Enkel profiel (prefab)			Samengesteld profiel	
	A_{EP} $\cdot 10^3$ [mm^2]	zp_{EP} [mm]	I_{EP} $\cdot 10^9$ [mm^4]	zp_{SP} [mm]	I_{SP} $\cdot 10^9$ [mm^4]
SKK 1900	1038	945	448	961	461
SKK 1800	1001	895	389	910	401
SKK 1700	963	845	335	860	346
SKK 1600	1053	798	348	811	357
SKK 1500	1015	748	295	760	303
SKK 1400	903	716	228	729	234
SKK 1300	865	665	188	677	193
SKK 1200	827	614	154	625	158
SKK 1100	790	563	123	573	126
SKK 1000	719	498	93	508	96
SKK 900	681	448	71	457	73
SKK 800	644	398	52	406	54
SKK 700	606	348	37	355	38

uitgangspunten:
betonsterkteklasse prefab: C60/75
betonsterkteklasse voegen: C35/45

weerstandsmomenten EP :
$W_{EP.boven} = I_{EP} / (h - zp_{EP})$
$W_{EP.onder} = I_{EP} / zp_{EP}$

traagheidsmoment I_{SP} is bepaald met gewogen E op basis van stijfheid prefab

17-01-2017

C

Profile library Excel

Type	h (cm)	w_top(cm)	w_bottom(cm)	t_top(cm)	t_bottom(cm)	t_web(cm)	h_deck(cm)	A_profile (cm²)	ZP_from bottom(cm)	I_profile (cm4)	W_top(cm³)	W_bottom(cm³)	h_total(mm)	I_combined (mm4)	ZP_combined	W_comtop(mm³)	W_combot(mm³)
ZIPXL 2400	240	144	148	20	20	25	20	10600	112.1	83100000	649726	741302	2600	1219000000000	1386	1004118616	879509380
ZIPXL 2300	230	144	148	20	20	25	20	10350	107.4	74800000	610114	696462	2500	1105000000000	1333	946872322	828957239
ZIPXL 2200	220	144	148	20	20	25	20	10100	102.6	67000000	570698	653021	2400	1008000000000	1288	906474820	782608696
ZIPXL 2100	210	144	148	20	20	25	20	9850	97.8	59700000	532086	610429	2300	907000000000	1235	851643192	734412955
ZIPXL 2000	200	144	148	20	20	25	20	9600	93.1	52900000	494855	568206	2200	811000000000	1181	795878312	686706181
ZIPXL 1900	190	144	148	20	20	25	20	9350	88.3	46600000	458210	527746	2100	722000000000	1127	742034943	640638864
ZIPXL 1800	180	144	148	20	20	25	20	9100	83.6	40800000	423237	488038	2000	639000000000	1074	690064795	594972067
ZIPXL 1700	172.5	50	148	50	20	25	22.5	8450	71.9	28800000	286282	400556	1950	565000000000	1012	602345416	558300395
ZIPXL 1600	162.5	50	148	40	20	25	22.5	7950	66	24000000	248705	363636	1850	494000000000	955	551955307	517277487
ZIPXL 1500	152.5	50	148	30	20	25	22.5	7450	59.9	19600000	211663	327212	1750	428000000000	897	501758499	477146042
ZIPXL 1400	142.5	50	148	20	20	25	22.5	6950	53.7	15600000	175676	290503	1650	367000000000	838	451970443	437947494
ZIPXL 1300	132.5	50	148	37.5	20	25	22.5	7160	52.4	13700000	171036	261450	1550	314000000000	793	414795244	395964691
ZIPXL 1200	122.5	50	148	27.5	20	25	22.5	6660	46.8	10700000	141347	228632	1450	264000000000	738	370786517	357723577
ZIPXL 1100	112.5	50	148	17.5	20	25	22.5	6160	41.2	8100000	113604	196602	1350	219000000000	682	327844311	321114370
ZIPXL 1000	102.5	50	148	7.5	20	25	22.5	5660	35.4	5800000	86438	163842	1250	179000000000	625	286400000	286400000
ZIPXL 900	92.5	40	148	0	18	40	22.5	5830	32.8	4300000	72027	131098	1150	144000000000	569	247848537	253075571
ZIPXL 800	82.5	40	148	0	18	40	22.5	5430	28.8	3100000	57728	107639	1050	113000000000	519	212806026	217726397
ZIPXL 700	72.5	40	148	0	18	40	22.5	5030	25.1	2100000	44304	83665	950	86000000000	469	178794179	183368870
SKK 1900	190	112	118	22	20.5	18.8	10	10380	94.5	44800000	469110	474074	1900	461000000000	961	490947817	479708637
SKK 1800	180	112	118	22	20.5	18.8	10	10010	89.5	38900000	429834	434637	1800	401000000000	910	450561798	440659341
SKK 1700	170	112	118	22	20.5	18.8	10	9630	84.5	22500000	263158	266272	1700	346000000000	960	467567568	360416667
SKK 1600	160	142	148	17	15.7	18.8	10	10530	79.8	24800000	309227	310777	1600	357000000000	811	452471483	440197287
SKK 1500	150	142	148	17	15.7	18.8	10	10150	74.8	29500000	392287	394385	1500	303000000000	760	409459459	398684211
SKK 1400	140	142	148	17	15.7	18.8	10	9030	71.6	22800000	333333	318436	1400	234000000000	729	348733234	320987654
SKK 1300	130	142	148	17	15.7	18.8	10	8650	66.5	18800000	296063	282707	1300	193000000000	677	309791332	285081241
SKK 1200	120	142	148	17	15.7	18.8	10	8270	61.4	15400000	262799	250814	1200	158000000000	625	274782609	252800000
SKK 1100	110	142	148	17	15.7	18.8	10	7900	56.3	12300000	229050	218472	1100	126000000000	573	239089184	219895288
SKK 1000	100	142	148	17	15.7	18.8	10	7190	49.8	9300000	185259	186747	1000	96000000000	508	195121951	188976378
SKK 900	90	142	148	17	15.7	18.8	10	6810	44.8	7100000	157080	158482	900	73000000000	457	164785553	159737418
SKK 800	80	142	148	17	15.7	18.8	10	6440	39.8	5200000	129353	130653	800	54000000000	406	137055838	133004926
SKK 700	70	142	148	17	15.7	18.8	10	6060	34.8	3700000	105114	106322	700	38000000000	355	110144928	107042254

Figure C.1: Overview of the prefabricated beams and cross-section parameters

D

Buckling Calculation Sheets

Required Diameter regarding Buckling Capacity

$l_0 := 0.7 \cdot 15\mathrm{m}$ $\quad f_{cd} := 20\mathrm{MPa}$ $\quad N_{Ed} := -2000\mathrm{kN}$ $\quad A := 0.7$ $\quad B := 1.1$ $\quad C := 0.7$

$$A_c(d) := d^2 \cdot \frac{\pi}{4}$$

$$I_c(d) := d^4 \cdot \frac{\pi}{64}$$

$$i(d) := \sqrt{\frac{I_c(d)}{A_c(d)}}$$

$$\lambda(d) := \frac{l_0}{i(d)}$$

$$\lambda_{lim}(d) := 20 \cdot \frac{A \cdot B \cdot C}{\sqrt{\frac{-N_{Ed}}{A_c(d) \cdot f_{cd}}}}$$

$d := 1200\mathrm{mm}$ $\quad test := 0$

Given

$test = \lambda(d) - \lambda_{lim}(d)$

$(d_{column}) := \mathrm{Find}(d)$

Round Column

$d_{column} = 1.179\,\mathrm{m}$

$\lambda(d_{column}) = 35.621$

$\lambda_{lim}(d_{column}) = 35.621$

$$d_{columntest} := \sqrt[4]{\frac{l_0^{\,2} \cdot -N_{Ed}}{\frac{\pi}{64} \cdot (20 \cdot A \cdot B \cdot C)^2 \cdot f_{cd}}}$$

$d_{columntest} = 1.179\,\mathrm{m}$

If squared cross-section factor pi/64 becomes 1/12
For rectangular factor pi/64 becomes for example 1,2/12 (when b/h=1,2)

Required Thickness regarding Buckling Capacity

$l_0 := 0.7 \cdot 6m$ $f_{cd} := 20MPa$ $N_{Ed} := -97754kN$ $A := 0.7$ $B := 1.1$ $C := 0.7$ $wb := 15.875m$

wb is long side of the rectangular cross-section, d_{column} is the short side

$$A_c(d) := d \cdot wb$$

$$I_c(d) := d^3 \cdot \frac{wb}{12}$$

$$i(d) := \sqrt{\frac{I_c(d)}{A_c(d)}}$$

$$\lambda(d) := \frac{l_0}{i(d)}$$

$$\lambda_{lim}(d) := 20 \cdot \frac{A \cdot B \cdot C}{\sqrt{\frac{-N_{Ed}}{A_c(d) \cdot f_{cd}}}}$$

$d := 1200mm$ $test := 0$

Given

$test = \lambda(d) - \lambda_{lim}(d)$

$(d_{column}) := Find(d)$

Rectagular Column

$d_{column} = 0.825\,m$

$\lambda(d_{column}) = 17.643$

$\lambda_{lim}(d_{column}) = 17.643$

$$d_{column.test} := \sqrt[3]{\frac{l_0^2 \cdot -N_{Ed} \cdot 12}{wb \cdot (20 \cdot A \cdot B \cdot C)^2 \cdot f_{cd}}}$$

$d_{column.test} = 0.825\,m$

E

Costs Specification

Figure E.1 shows an overview of the costs for the different elements which are used in the model. Although the size of the elements in the model differ, the factor in the last column is chosen to give an estimation of the total project costs.

Element:	Concrete		Reinforcement		Prestressing		Formwork		Casting	Total price	Calculated Factor	Chosen Factor:
Units:	m³	[€/m³]	[kg/m³]	[€/ton]	[ton]	[€/ton]	[m²]	[€/m²]	[€/m³]	[€]	[€/m³]	[€/m³]
Cantilever Bridge	5890	100	180	1200	385	1750	24063	250	50	€ 8,845,240	€ 1502	←splitted price
Hammerhead (9*1,5*5m)	67.5	100	135	1200	0	1750	105	150	30	€ 35,460	€ 525	€ 650
prefab beam (50m)	51.9	100	100	1200	3.37	1750	265	25	30	€ 25,498	€ 491	€ 500
Deck layer (50*15*0.2)	150	100	100	1200	0	1750	26	75	25	€ 38,700	€ 258	€ 275
Capping beam (1,8*1,8*15m)	48.6	100	135	1200	0	1750	114.48	150	30	€ 31,363	€ 645	€ 650
Column Ø1,8 (5m)	12.7	100	135	1200	0	1750	28.3	150	30	€ 7,953	€ 626	€ 650
Foundation slab (5*15*1.5)	112.5	100	125	1200	0	1750	135	95	30	€ 44,325	€ 394	€ 400
Foundation pile (0,45*0,45*20)	4.05	100	100	1200	0.08	1750	36	25	30	€ 2,053	€ 103	€ 150

Figure E.1: Costs specification with on the right of the table, the chosen price per unit.

The price of the balanced cantilever bridge is split-up in the model, so each piece (concrete volume, prestressing steel and formwork) is separately priced. For the other elements, the fixed unit price is taken.
All prices are based on experience and reference project by Movares.

F

Concrete Compositions

For the concrete compostitions there are a couple of base materials needed, for these materials the ECI value is determined with help of the NMD (Nationale Milieu Database). The ECI values per kilogram base material can be found in Table F.1.

Table F.1: ECI values per kilogram base material.

$CURConcreteData$		$MKI[Euro/kg]$
$CEMINL$	$Cement$	$6.062E-2$
$CEMIIIANL$	$Cement$	$3.363E-2$
$CEMIIIBNL$	$Cement$	$2.345E-2$
$BlastFurnaceSlag(GGBFS)$	$Pozzolan/filler$	$1.539E-3$
$Flyashfromcoal$	$Pozzolan/filler$	$3.450E-4$
$Silicafume$	$Pozzolan/filler$	$5.259E-4$
$LimestonepowderNL$	$Filler$	$3.130E-3$
$Sand, river0-4mmNL$	$primaryAggregateFine$	$4.552E-4$
$Sand, sea0-4mmNL$	$primaryAggregateFine$	$1.859E-3$
$Sand, crushed0-4mmNL$	$primaryAggregateFine$	$1.084E-3$
$Sand, crushedrecycled0-4mm$	$secondaryAggregate$	$1.607E-4$
$Gravel, river>4mmNL$	$primaryAggregateCoarse$	$8.443E-5$
$Gravel, sea>4mmNL$	$primaryAggregateCoarse$	$1.849E-3$
$Gravel, crushed>4mmNL$	$primaryAggregateCoarse$	$2.254E-3$
$Granulate, crushedrecycled>4mm$	$secondaryAggregate$	$1.703E-4$
$Plasticizer-waterreducer$	$chemicalAdmixture$	$8.785E-2$
$Superplasticizer-highrange$	$chemicalAdmixture$	$9.225E-2$
$Tapwater$	$Water$	$3.191E-5$
$Surface/wellwater$	$Water$	0
$Steelrebar$	$Reinforcement$	0.163
$Steelfibres$	$Reinforcement$	0.195
$Plasticfibres$	$Reinforcement$	0.200

Below there is an overview of a couple of mixtures which are composed by a concrete specialist at Heijmans, a Dutch contractor. From this mixtures, there is made a selection of which the regression line is drawn to receive the ECI value for each concrete class. These values are implemented in the parametric model.

$RecepturenbetonmengselsNL$		
$ECI = EnvironmentalCostIndicatorinEuro$		
$Compound:$	$kg/m3:$	
	$C30/37$	$ECI:$
$CEMI52.5N$	108	6.547
$Hoogovenslak$	252	0.388
$Grind4-22$	343	$2.896E-2$
$Grind4-32$	801	$6.762E-2$
$Zand0-4$	778	0.354
$SPSky648con20\%$	1.71	0.158
$Bronwater$	107	0
	$sum:$	€7.544
$Compound:$	$kg/m3:$	
	$C30/37$	$ECI:$
$CEMIII/b42.5N$	348	8.161
$Zeezand0-2$	110	0.205
$Rivierzand0-4$	665	0.303
$Riviergrind4-32$	1018	$8.594E-2$
$SPLR-9400con30\%SPL$	0.52	$4.797E-2$
$Cretoplastcon35\%$	0.7	$6.458E-2$
$Oppervlaktewater$	170	0
	$sum:$	€8.866
$Compound:$	$kg/m3:$	
	$C30/37$	$ECI:$
$CEMIII/b42.5N$	237	5.558
$CEMI52.5R$	79	4.789
$Vliegas$	20	$6.899E-3$
$Zand0-4$	851	0.387
$Grind2-8$	563	$4.753E-2$
$Grind4-16$	561	$4.736E-2$
$SPPantarhit175$	1.22	0.113
$Water$	91	$2.904E-3$
	$sum:$	€10.951
$Compound:$	$kg/m3:$	
	$C35/45$	$ECI:$
$CEMIII/b42.5N$	336	7.879
$PKvliegas$	20	$6.899E-3$
$Zand0-4$	865	0.394
$Grind4-16$	1057	$8.924E-2$
$SPPantarhit175$	1.29	0.119
$Water$	107	$3.415E-3$
	$sum:$	€8.492

RecepturenbetonmengselsNL		
ECI = EnvironmentalCostIndicatorinEuro		
Compound:	kg/m3:	
	C35/45	ECI:
CEMI52.5N	126	7.638
Hoogovenslak	224	0.345
Grind4 − 22	316	2.668E − 2
Grind4 − 32	783	6.610E − 2
Zand0 − 4	826	0.376
SPSky648con20%	1.49	0.137
Bronwater	100	0
	sum:	€8.590
Compound:	kg/m3:	
	C45/55	ECI:
CEMI52.5R	102	6.184
CEMIII/b42.5N	307	7.199
Zand0 − 2	127	5.781E − 2
Zand0 − 4	658	0.230
Grind4 − 16	1025	8.654E − 2
SPVC1550con30%	1.8	0.166
SPBV1 − M36%	0.61	5.627E − 2
Water	144	4.595E − 3
	sum:	€14.053
Compound:	kg/m3:	
	C50/60	ECI:
CEMI52.5R	200	12.125
Hoogovenslak	200	0.308
Vliegas	40	1.380E − 2
Zand0 − 4	761	0.346
Grind4 − 16	989	8.350E − 2
SPS350con20%	3.6	0.332
Water	155	4.946E − 3
	sum:	€13.213
Compound:	kg/m3:	
	C55/67	ECI:
CEMIII/b42.5N	412	9.661
CEMI52.5R	137	8.305
Vliegas	50	1.725E − 2
Zand0 − 4	647	0.295
Grind4 − 16	464	3.917E − 2
Grind4 − 32	466	3.934E − 2
SPSky69620%	3.36	0.310
SP380 − Rcon40%	1.8	0.166
Water	168	5.361E − 3
	sum:	€18.838

$Recepturenbetonmengsels NL$		
$ECI = EnvironmentalCostIndicatorinEuro$		
$Compound:$	$kg/m3:$	
	$C55/67$	$ECI:$
$CEMI52.5N$	240	14.550
$Hoogovenslak$	180	0.277
$Zand0-4$	775	0.353
$Grind4-16$	737	$6.222E-2$
$Grind4-22$	316	$2.668E-2$
$SPSky64820\%$	2.94	0.271
$Water$	123	$3.925E-3$
	$sum:$	€15.543
$Compound:$	$kg/m3:$	
	$C55/67$	$ECI:$
$CEMIII/b42.5N$	380	8.911
$Vliegas$	100	$3.450E-2$
$Zand0-4$	832	0.379
$Grind4-16$	922	$7.784E-2$
$SPChrysoOptima254$	3.12	0.288
$Water$	104	$3.319E-3$
	$sum:$	€9.693
$Compound:$	$kg/m3:$	
$Prefab$	$C70/85$	$ECI:$
$CEMIII/a$	550	18.499
$Zand0-4$	1053	0.479
$Grind4-16$	696	$5.876E-2$
$SPHR27con35\%$	4.13	0.381
$Water$	109	$3.478E-3$
	$sum:$	€19.421

G

National Environmental Database

	Costs Euro/unit		0.16	0.16	0.05	30	2	4	9	0.09	0.03	0.0001	0.06
Impact category	Material/ Process Type		Abiotic Depletion,	Abiotic Depletion	Global Warming Potential	Ozone Layer Depletion	Photochemical Oxidation Potential	Acidification Potential	Eutrophication Potential	Human Toxicity Potential	Ecotoxicity Potential, Fresh water	Ecotoxcity Potential, Marine water	Ecotoxicity Potential, Terrestrial environment
Unit			non fuel (ADnf)	fuel (ADf)	(GWP)	(ODP)	(POCP)	(AP)	(EP)	(HT)	(FAETP)	(MAETP)	(TETP)
		Unit	kg Sb eq	kg Sb eq	kg CO2 eq	kg CFC-11 eq	kg C2H4 eq	kg SO2 eq	kg PO42- eq	kg 1,4-DB eq	kg 1,4-DB eq	kg 1,4-DB eq	kg 1,4-DB eq
CEM I NL	Cement	kg	6.70E-07	5.70E-04	8.20E-01	5.20E-09	2.10E-04	2.70E-03	3.60E-04	5.00E-02	6.90E-04	5.10E+00	6.80E-04
CEM IIIA NL	Cement	kg	6.70E-07	7.70E-04	4.40E-01	5.40E-09	1.20E-04	1.50E-03	1.70E-04	3.30E-02	4.40E-04	7.30E+00	4.50E-04
CEM IIIB NL	Cement	kg	6.50E-07	8.00E-04	3.00E-01	5.20E-09	9.00E-05	1.00E-03	1.00E-04	2.70E-02	3.40E-04	7.80E+00	3.60E-04
Blast Furnace Slag (GGBFS)	Pozzolan/ filler	kg	7.60E-10	1.70E-04	1.90E-02	1.10E-09	1.00E-06	5.80E-06	1.40E-06	3.60E-03	4.60E-06	2.00E+00	2.70E-06
Fly ash from coal	Pozzolan/ filler	kg	8.50E-10	2.30E-05	3.30E-03	2.60E-10	1.20E-06	1.50E-05	3.50E-06	6.70E-04	2.10E-05	2.10E-01	7.40E-06
Silica fume	Pozzolan/ filler	kg	4.80E-09	3.90E-05	5.20E-03	3.90E-10	1.60E-06	1.40E-05	3.30E-06	1.50E-03	3.00E-05	3.20E-01	4.80E-05
Limestone powder NL	Filler	kg	1.90E-08	2.40E-04	3.20E-02	2.40E-09	9.70E-06	8.60E-05	1.80E-05	8.40E-03	1.80E-04	2.00E+00	7.40E-05
Sand, river 0-4mm NL	primary Aggregate Fine	kg	1.30E-09	2.00E-05	2.90E-03	3.10E-10	2.30E-06	1.80E-05	4.20E-06	1.90E-03	3.10E-05	2.00E-01	1.10E-05
Sand, sea 0-4mm NL	primary Aggregate Fine	kg	3.40E-09	7.40E-05	1.10E-02	1.30E-09	1.00E-05	7.90E-05	1.80E-05	8.00E-03	1.30E-04	7.40E-01	2.30E-05
Sand, crushed 0-4mm NL	primary Aggregate Fine	kg	1.90E-08	6.90E-05	9.00E-03	5.70E-10	3.10E-06	2.90E-05	6.10E-06	4.30E-03	5.40E-05	5.50E-01	3.20E-05
Sand, crushed recycled 0-4 mm	secondary Aggregate	kg	5.10E-10	1.40E-05	2.00E-03	3.00E-10	6.90E-07	3.00E-06	5.20E-07	3.30E-04	1.60E-05	1.00E-01	2.80E-06
Gravel, river >4 mm NL	primary Aggregate Coarse	kg	1.20E-10	7.10E-06	1.10E-03	1.70E-10	3.60E-07	1.40E-06	2.30E-07	1.60E-04	8.40E-06	5.20E-02	7.00E-07
Gravel, sea >4 mm NL	primary Aggregate Coarse	kg	3.10E-09	7.20E-05	1.10E-02	1.30E-09	1.00E-05	7.90E-05	1.80E-05	7.90E-03	1.30E-04	7.30E-01	2.00E-05
Gravel, crushed >4 mm NL	primary Aggregate Coarse	kg	3.10E-09	4.30E-05	6.20E-03	6.80E-10	7.10E-06	5.70E-05	1.30E-05	1.70E-02	8.90E-05	4.40E-01	1.70E-05
Granulate, crushed recycled >4 mm	secondary Aggregate	kg	7.70E-10	7.00E-06	1.10E-03	1.10E-10	8.30E-07	6.70E-06	1.50E-06	7.20E-04	1.10E-05	6.80E-02	5.60E-06
Plasticizer - water reducer	chemical Admixture	kg	0.00E+00	2.20E-03	3.90E-01	2.20E-07	8.90E-04	1.00E-02	1.60E-03	1.10E-01	6.60E-03	1.70E+01	2.60E-04
Super plasticizer - high range	chemical Admixture	kg	0.00E+00	8.10E-03	7.20E-01	9.60E-08	1.40E-03	9.70E-03	4.60E-04	8.20E-02	3.00E-02	9.10E+00	3.60E-04
Tap water	Water	kg	2.60E-10	2.70E-06	3.40E-04	1.60E-11	1.10E-07	8.00E-07	1.40E-07	8.30E-05	1.30E-06	2.20E-02	1.50E-06
Surface/ well water	Water	kg	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
Steel rebar	Reinforcement	kg	1.10E-06	1.30E-02	1.50E+00	6.00E-08	1.20E-03	5.10E-03	7.00E-04	5.50E-01	1.80E-02	5.00E+01	2.70E-02
Steel fibres	Reinforcement	kg	3.50E-06	1.40E-02	1.90E+00	9.00E-08	1.30E-03	6.10E-03	8.30E-04	6.10E-01	2.80E-02	6.20E+01	2.90E-02
Plastic fibres	Reinforcement	kg	4.40E-07	3.60E-02	2.40E+00	3.00E-08	2.30E-03	8.30E-03	8.30E-04	2.80E-01	1.90E-02	3.30E+01	2.60E-03

Figure G.1: National Environmental Database Source: www.milieudatabase.nl/index.php

H

Overview of balanced cantilever bridges in the Netherlands

In the following overview, Figure H.1, all cantilever bridges in the Netherlands with a span of at least 120 meters are visible. The yellow marked cells are the minimal values as used in practice. Note that for the thickness of the bottomslab at midspan, the thickness is depending on the applied prestressing cables. For this value the thickness of the Stichtsebrug is taken a minimum. These values are used as input for the parametric model.

Cantilever bridges with Lspan > 120 m	Bridge		General dimensions		Hammerhead							Midspan		Mean values					Ratio		
						Floor			Webs			Floor		Surface Area							
Name bridge	Year	Concrete class	L_span	Width	Height	Thickness	Width	Area	Number	Thickness	Height	Thickness	Area	Floor	Webs	Deck	Total	Area/ Width	L/Hh	L/Hm	Webs c-t-c
Brug over de Maas bij empel	1969	B37,5	120.50	6.95	5.50	0.90	4.00	3.60	2	0.40	3.00	0.20	0.80	2.20	2.96	1.16	6.32	0.909	21.9	40.2	4.0
Brug over de Rijn bij Heteren	1968	B37,5	120.75	16.85	5.50	0.85	10.85	9.22	3	0.50	2.75	0.20	2.17	5.70	5.40	5.05	16.15	0.958	22.0	43.9	5.1
Brug over de Maas bij Grubbenvorst	1968	B37,5	120.75	14.35	5.50	0.85	8.85	7.52	3	0.45	3.00	0.20	1.77	4.65	5.03	4.06	13.74	0.957	22.0	40.3	4.5
Brug over de IJssel bij Zutphen	1974	lichtbeton	125.00	14.40	5.50	0.65	7.40	4.81	2	0.60	2.50	0.20	1.48	3.15	3.95	4.96	12.05	0.837	22.7	50.0	6.8
Brug opver de Maas bij Ravenstein	1972	B37,5	139.40	17.10	6.25	1.20	10.85	13.02	3	0.50	3.00	0.20	2.17	7.60	5.89	5.37	18.86	1.103	22.3	46.5	5.3
IJsselbrug Zwolle	1967	B37,5	150.50	16.85	7.00	1.50	10.85	16.28	3	0.50	3.00	0.20	2.17	9.22	6.23	4.44	19.89	1.180	21.5	50.2	5.2
IJsselbrug Deventer	1969	B37,5	150.50	17.35	6.84	1.45	10.85	15.70	3	0.50	3.00	0.15	1.63	8.66	6.18	4.51	19.35	1.115	22.0	50.2	5.3
Brug over de Maas bij Heumen	1979	B37,5	151.50	18.35	7.00	1.20	8.50	10.20	2	0.45	2.75	0.20	1.70	5.95	3.76	6.41	16.12	0.878	21.6	55.1	9.9
Zeeburgerbrug	1986	B37,5	155.00	21.01	7.00	1.00	14.90	14.90	3	0.50	2.75	0.20	2.98	8.94	6.41	7.27	22.62	1.077	22.1	56.4	7.2
Brug over de Maas bij Boxmeer	1982	lichtbeton	157.50	14.06	7.00	0.80	7.56	6.05	2	0.50	2.85	0.25	1.89	3.97	4.40	5.03	13.40	0.953	22.5	55.3	7.1
Stichtsebrug 2	1995	B85	160.00	21.81	6.75	0.55	13.86	7.62	3	0.32	2.50	0.22	3.05	5.34	4.07	7.90	17.30	0.793	23.7	64.0	6.8
Stichtsebrug 1	1980	lichtbeton	160.00	21.65	6.75	1.00	14.80	14.80	3	0.50	2.50	0.20	2.96	8.88	6.04	6.82	21.74	1.004	23.7	64.0	7.2
Jan Blankenbrug	2000	B65	165.00	28.90	7.00	1.35	11.20	15.12	2	0.95	2.75	0.30	3.36	9.24	7.70	10.08	27.02	0.935	23.6	60.0	10.7
Dintelhaven west	2001	B85	184.95	15.74	8.50	0.60	8.54	5.12	2	0.40	3.00	0.25	2.14	3.63	4.26	6.63	14.52	0.923	21.8	61.7	8.1
Dintelhaven oost	2001	B85	192.00	22.15	8.50	0.60	14.95	8.97	3	0.38	3.00	0.25	3.74	6.35	6.07	9.44	21.86	0.987	22.6	64.0	7.3

Figure H.1: Overview of balanced cantilever bridges in the Netherlands with a minimal span of 120 meters.

I

MAIN BRIDGE ANALYSIS - WITH EXCEL

I.1. STRUCTURAL ANALYSIS

The structural analysis of the main bridge will be done in two different ways. One: via Karamba, in which the flow of forces is automatically determined which will be used to design the prestressing cables. Two: via an Excel sheet, in this Excel sheet are the formulas to determine the flow of forces and the moment distribution. This will be used to determine the amount of prestressing cables.

While Karamba is directly connected in the script, Excel is an external program which will be used only for verification of the model. The Excel file can possibly be guided by the Grasshopper components "Write Excel" and "Read Excel". So specific data will go from Grasshopper to Excel, then in Excel the formulas and calculations will perform their task, consequently the Excel output will be loaded into Grasshopper again. The whole design can be managed in this way in only one script, which is a real advantage for the smoothness of the design process.

I.1.1. EXCEL CALCULATIONS

The Excel calculations take place in two different Excel sheets, one for the main bridge and another one for the prefabricated beams of the approach bridge.

Balanced Cantilever Bridge

In the Excel sheet for the balanced cantilever bridge, already the shape of the bridge was modelled parametrically. So to determine the force and moment distribution, the cross-section properties of the bridge is determined along the length of the span. From Paragraph 3.1.1, already the shape of the bridge is known. For the weight of the bridge, the area is calculated so the area times a length dx times the weight of concrete gives the load distribution over the length of the bridge, see Equation I.1. Note that for now, only the self weight of the structure is taken into account.

$$q_{selfweight}(x) = A(x) * dx * \gamma_{concrete} \tag{I.1}$$

From this load distribution the shear distribution can be computed by taking the cumulative value of the force distribution, or in different words, by integrating the force distribution over the length, see Equation I.2.

$$V_{selfweight}(x) = \int_0^L q(x)dx \tag{I.2}$$

To go from the shear force distribution to the moment distribution, the formula should be integrated once more, see Equation I.3.

$$M_{selfweight}(x) = \int_0^L V(x)dx \tag{I.3}$$

In Excel this formulas are used as numeric functions with dx is equal to one, instead of a closed analytical function. The reason for this is the complexity of the function for the bottom flange, which is also indirectly inside the formulas for the area and the load distribution. So integration of these functions would give mathematical problems.

When from the moment distribution the rotation of the cross-section has to be determined, the values of the bending moment distribution should be superimposed from the hammerhead to the end of the bridge. So the maximum rotation will be at the end of the structure and it will be zero at the hammerhead. For the analytical function of the angular rotation, see Equation I.4.

$$\theta(x) = \frac{\int M(x)dx}{E * I_{yy}} \tag{I.4}$$

From the angular rotation (θ), the deflection (w) can be obtained by integrating for one last time, see Equation I.5. Here also the deflection of the individual parts with length dx is superimposed from the hammerhead, where the deflection is assumed to be zero.

$$w(x) = \int \theta(x)dx \tag{I.5}$$

Tendon calculation

For the calculation phase, it is necessary to determine the location of the neutral axis of the cross-section along the length of the bridge. This is done by multiplying the individual area parts of the cross-section with their distance to the bottom of the cross-section. Thereafter, the sum of those values has to be divide by the total cross-sectional area to receive the location of the neutral axis from the bottom. For the formula to determine the neutral axis, see Equation I.6.

$$z_{neutralaxis}(x) = \frac{\sum(A_i(x) * z_i)}{A(x)} \tag{I.6}$$

Once the neutral axis of the cross-section is known, it is possible to calculate the second moment of area (I_{yy}). This section property is determined by calculating the second moment of area of the individual area parts and adding the Steiner part to it. The Steiner rule is the addition of the inertia created by a cross-section part at a certain distance to the neutral line. Equation I.7 is used in Excel to determine the second moment of area along the length of the bridge.

$$I_{yy}(x) = \sum(\frac{1}{12} * b_i * h_i^3 + A_i * d_i^2) \tag{I.7}$$

Where d_i is the distance from the local centre of gravity to the neutral axis of the total cross-section. With the second moment of area known, it is possible to calculate the section modulus for the top and bottom of the cross-section by dividing the second moment of area by the distance from the neutral axis to the top and bottom fibre. See Equations I.8 & I.9.

$$W_{top}(x) = \frac{I_{yy}(x)}{h_c(x) - z(x)} \tag{I.8}$$

$$W_{bottom}(x) = \frac{I_{yy}(x)}{z(x)} \tag{I.9}$$

The W_{top} is used in the calculations for the amount of tendons and both of the section moduli can be used to determine the stress levels in the outer fibres of the concrete cross-section. These stresses can be calculated by dividing the bending moment by the section modulus, see Equation I.10 & I.11.

$$\sigma_{top}(x) = \frac{M_y(x)}{W_{top}(x)} \tag{I.10}$$

$$\sigma_{bottom}(x) = \frac{M_y(x)}{W_{bottom}(x)} \tag{I.11}$$

For the calculation of the tendon distribution, it is logical to consider not every meter of the bridge but only the segment length. Since it is not possible to change the tendon distribution within a segment. Therefore the above mentioned bending moment distribution is redistributed over the segments. So only at the location of the split between two segments the values of the bending moment line and other parameters are requested.

Once the moment and all cross-section parameters are known, the excel sheet is able to calculate the number of tendons along the length of the bridge. See Figure I.1.

segment number	M_ed per Segment	e_p Segment	A_cross section Segment	W_top Segment	N_working prestress	N_initial prestress	Ap [mm²]	number of strands	number of tendons	min. Number of tendons	Length Tendons [m]	Total length Tendon [m]	Volume Prestressing Steel [m³]
0	0	0	0	0	0	0	0	0	0	0		**68448**	**130,1**
1	2543	0,874	10,175	9,051	1890	2362	1625	16	0,86	6	100	600	
2	9158	0,880	10,199	9,109	6731	8413	5786	58	3,05	12	96	1152	
3	19858	0,892	10,251	9,234	14416	18020	12393	124	6,52	18	92	1656	
4	34667	0,912	10,337	9,442	24704	30880	21238	212	11,18	24	88	2112	
5	53622	0,942	10,461	9,745	37232	46540	32008	320	16,85	30	84	2520	
6	76779	0,982	10,630	10,159	51536	64420	44305	443	23,32	36	80	2880	
7	104209	1,033	10,846	10,697	67088	83860	57676	577	30,36	42	76	3192	
8	136003	1,098	11,114	11,373	83346	104182	71652	717	37,71	48	72	3456	
9	172275	1,178	11,437	12,201	99796	124745	85794	858	45,15	54	68	3672	
10	213160	1,272	11,818	13,199	115998	144997	99723	997	52,49	60	64	3840	
11	258816	1,384	12,262	14,385	131600	164500	113136	1131	59,55	66	60	3960	
12	309427	1,513	12,770	15,778	146354	182942	125820	1258	66,22	72	56	4032	
13	365203	1,662	13,346	17,401	160104	200130	137641	1376	72,44	78	52	4056	
14	426382	1,831	13,993	19,280	172779	215974	148538	1485	78,18	84	48	4032	
15	493229	2,021	14,713	21,441	184372	230465	158504	1585	83,42	90	44	3960	
16	566041	2,233	15,508	23,916	194923	243654	167575	1676	88,20	96	40	3840	
17	645143	2,469	16,381	26,740	204506	255633	175813	1758	92,53	102	36	3672	
18	730892	2,729	17,336	29,949	213212	266515	183298	1833	96,47	108	32	3456	
19	823679	3,014	18,373	33,587	221142	276427	190115	1901	100,06	114	28	3192	
20	923926	3,326	19,495	37,699	228398	285498	196353	1964	103,34	120	24	2880	
21	1032092	3,663	20,705	42,336	235081	293852	202099	2021	106,37	126	20	2520	
22	1148668	4,028	22,004	47,553	241285	301606	207432	2074	109,17	132	16	2112	
23	1274185	4,421	23,395	53,411	247095	308869	212427	2124	111,80	138	12	1656	

Figure I.1: tendon calculation

BIBLIOGRAPHY

[1] Statistics Netherlands, *Transport and mobility 2016*, Tech. Rep. (Centraal Bureau Statistiek, 2016).

[2] A. Reitsema, D. Hordijk, and N. V. Heijmans, *Towards Slender, Innovative Concrete Structures for Replacement of Existing Viaducts,* (2016).

[3] Rijksoverheid, *Extra geld voor toekomstvaste aanpak bruggen project A27 | Nieuwsbericht | Rijksoverheid.nl,* (2017).

[4] D. Davis, *Modelled on Software Engineering: Flexible Parametric Models in the Practice of Architecture*, Ph.D. thesis, School of Architecture and Design, RMIT University (2013).

[5] D. E. Weisberg, *The engineering design revolution: The people, companies and computer systems that changed forever the practice of engineering.* `www.cadhistory.net.` (2008).

[6] A. O. Payne, *The Grasshopper Primer (EN),* third edit ed. (2014) p. 242.

[7] Rijkswaterstaat GPO m.m.v. Witteveen + Bos, *Richtlijn Ontwerp Autosnelwegen 2017*, Tech. Rep. november (2017).

[8] T. Olsthoorn, *Richtlijn Ontwerp Autosnelwegen - Veilige Inrichting van Bermen*, Tech. Rep. (Rijkswaterstaat, 2017).

[9] J. Brolsma, *Rapportage Containerhoogtemetingen*, Tech. Rep. april (2015).

[10] C. van der Veen, *Lecture slides from cie5127 concrete bridges,* (2017).

[11] E. Fransen, *Towards the Automation of Preliminary Bridge Designs with Parametric Design Software*, Master thesis, TU Delft (2018).

[12] I. van der Ven, *Optimalisatie van Hoge Sterkte Beton uitbouwbruggen*, Master thesis, TU Delft (2005).

[13] H. Nosewicz, T. de Goede, R. Vergoossen, and H. Sliedrecht, *Maasbrug grubbenvorst beoordeeld,* Cement **7**, 28 (2017).

[14] T. Reijnen, *Vrijevoorbouwmethode belicht,* Betoniek **4**, 8 (2015).

[15] R. Benaim, *The Design of Prestressed Concrete Bridges: Concepts and principles* (Taylor & Francis, 2007).

[16] E. Beerda, *Nederlands grootste kokerligger over máxima naar bouwplaats,* (2010).

[17] Rijkswaterstaat, *Richtlijnen Ontwerp Kunstwerken 1.4 Bijlage B,* (2017), RTD 1001:2017.

[18] D. Weertman and W. de Rijke, *Vuistregels voor het ontwerpen van betonnen bruggen en viaducten*, Tech. Rep. (Rijkswaterstaat, 2004).

[19] European Committee for Standardization, *NEN-EN 1992-1-1 + C2*, Tech. Rep. (2011).

[20] VSL, *VSL STRAND POST-TENSIONING SYSTEMS*, Tech. Rep. (2013).

[21] European Committee for Standardization, *Eurocode 1: Actions on structures -Part 2: Traffic loads on bridges,* (2015).

[22] European Committee for Standardization and the Dutch Subcommittee TGB Basiseisen en belastingen, *Eurocode 1: Actions on structures -Part 2: Traffic loads on bridges - Nationale bijlage,* (2015).

[23] European Committeee for Standardization, *Eurocode 0: Basis of structural design,* , 121 (2011).

[24] European Committee for Standardization, *National Annex to Eurocode 0: Basis of Structural Design,* (2011).

[25] C. Preisinger, *PARAMETRIC STRUCTURAL MODELING User Manual for Karamba Version 1.2.2* (2016) p. 142.

[26] D. Rutten, *Evolutionary Principles applied to Problem Solving - Grasshopper,* (2010).

[27] C. Young, *Excel Solver: Which Solving Method Should I Choose?* (2017).

[28] I. van der Ven, *Optimalisatie van Hoge Sterkte Beton uitbouwbruggen - Bijlagen*, Master thesis, TU Delft (2005).

[29] P. h.c.j. Walraven and d. C. Braam, *Prestressed Concrete* (Faculty of Civil Engineering and Geosciences, 2015) p. 373, arXiv:267 .

[30] Rijkswaterstaat, *Richtlijnen Ontwerp Kunstwerken 1.4,* , 1 (2017).